5. Sewer conduit hydraulics

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5. Sewer conduit hydraulics

Goals of this chapter

To know the design concept for sewer conduits

- Which situations are critical?
- Which discharges have to be considered?
- What is the minimum longitudinal conduit slope given by hydraulics?
- Which cross sections and conduit lengths are optimal?
- Which hydraulic phenomena have to be considered?
- How rough is a sewer conduit?

Note

Conduit = object that conveys a fluid with undefined section

Pipe = circular conduit

5. Sewer conduit hydraulics

Free surface flow, i.e. partial pipe filling!
Here: design of conduits. Manholes are discussed later

Content:

Literature:
W.H. Hager (1999). *Wastewater Hydraulics*, Springer, Berlin
ATV (1995). *Bau und Betrieb der Kanalisation*. Ernst & Sohn, Berlin
SIA 190:2017, SIA *Documentation* 40, DWA 110
Séminaire VSA/EPFL (2013). *Hydraulique des canalisations*

5.1 Relevant discharges

5.1 Relevant discharges

Extreme discharges are considered for design

Maximum discharge

- Defines conveyance capacity
- Uniform flow conditions are assumed
- No undulating conditions! ($F<0.8$ or $F>1.2$)
- Flowing full condition (transition between free-surface and pressurized flow)
- Design using Colebrook & White or Strickler

⇨ Sewer dimensions, conduit diameter D

Minimum discharge

- Solids transport, avoid deposition
- Minimum wall shear stress, minimum velocity

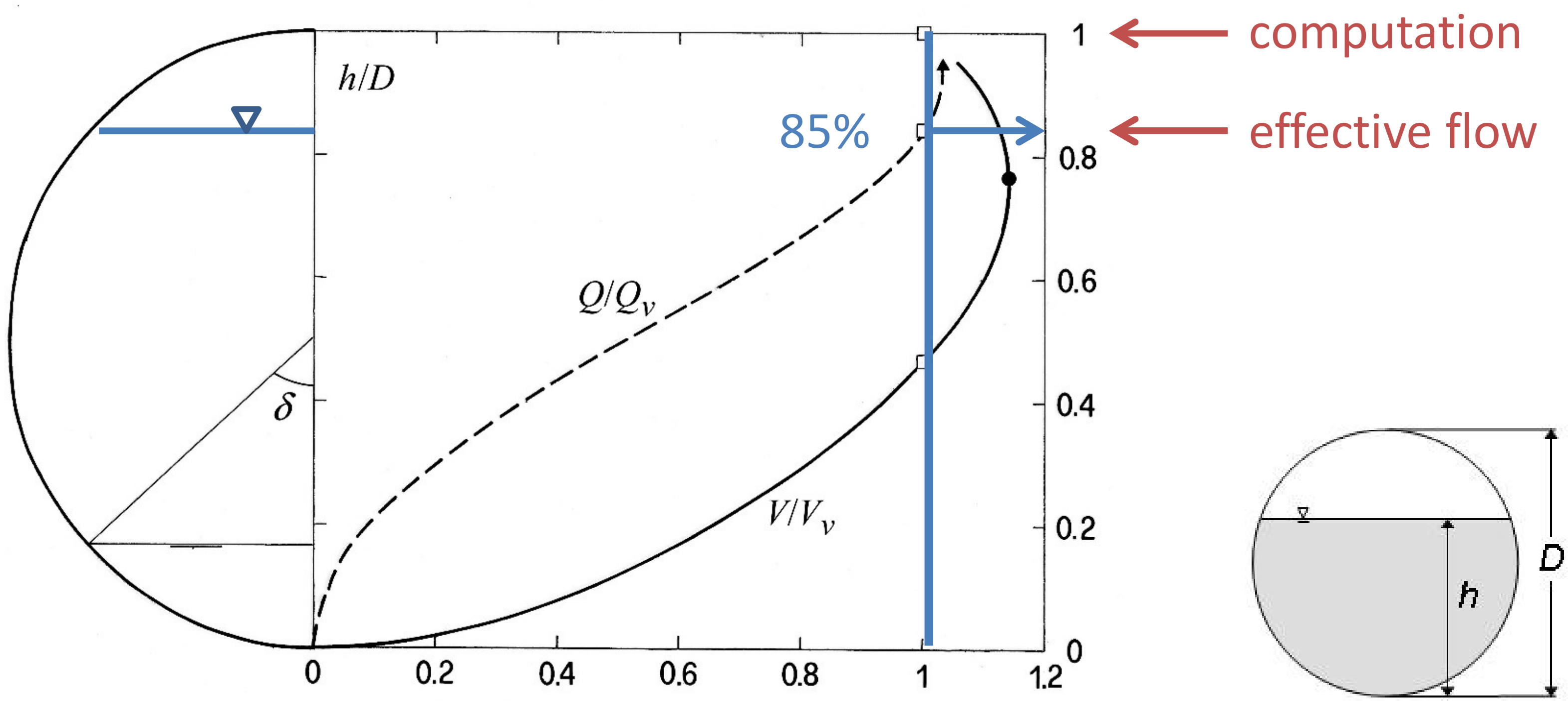
⇨ Defines conduit slope S_O

5.1 Relevant discharges

Maximum discharge

- maximum capacity $Q_M \approx$ flowing full condition Q_v (hydraulically equal to Q with 85% partial filling)
- Uniform flow, i.e. S_E=S_O

⇨ simple geometry and hydraulic computation (flowing full condition)



Old or irregular conduits: use 0.95*D* for capacity check

5.1 Relevant discharges

Minimum discharge

- Minimum flow depth of 3 cm (DWA 110)
- Minimum wall shear stress of $\tau \geq 1.0$ N/m^2 (DWA 110).
- Concept: D ⇨ S_{Om}

DWA 110, for $h/D \geq 0.50$

***D* [m]**	**0.25**	**0.50**	**1.00**	**1.50**	**2.00**
V_m [m/s]	0.49	0.64	0.95	1.19	1.39
S_{om} [‰]	1.63	1.12	1.01	0.96	0.91

If h/D is smaller than in the table, S_{Om} increases (h from Q_m)
Note that small slopes are difficult to realize and change due to settlements

In general $S_{Om} \geq 1$‰

5.1 Relevant discharges

Minimum discharge

- Macke (1980, 1983) gives the minimum velocity V_m is

  $V_m = 0.5 + 0.55D$

- Schütz (1985) gives the minimum slope S_{Om} with D in [mm]

  $S_{Om} = 1/D$, but $S_{Om} \geq 1‰$ for $D \geq 1'000$ mm

- Sander (1994) gives for small sewer ($D<1.0$ m)

  $S_{Om} = 1.2‰/D$, for $D<1.0$ m follows $S_{Om} = 1.2‰$

- SIA 190

D [m]	V_m [m/s]
<0.4	0.7
0.4 to 1.0	0.8
>1.0	1.0

ASCE V_m=0.60 m/s

D [m]	S_{Om} [‰]
0.25	2.9
0.45	1.32
0.60	0.90

5.2 Concept

5.2 Concept

- Uniform flow and flowing full condition in conduits
- Minimum conduit diameter $D \geq 0.30$ m
- Maximum conduit length, i.e. maximum distance between manholes is 120 m
- All changes related to the conduit in manhole (e.g. shape, slope, diameter, direction, discharge, junction, elevation, roughness)
- Between manholes exclusively straight, unchanged conduits (maintenance)
- Minimum depth of earth cover 1.0 m
- Wastewater conduit below fresh water pipe
- Energy line at ground elevation or below, if possible

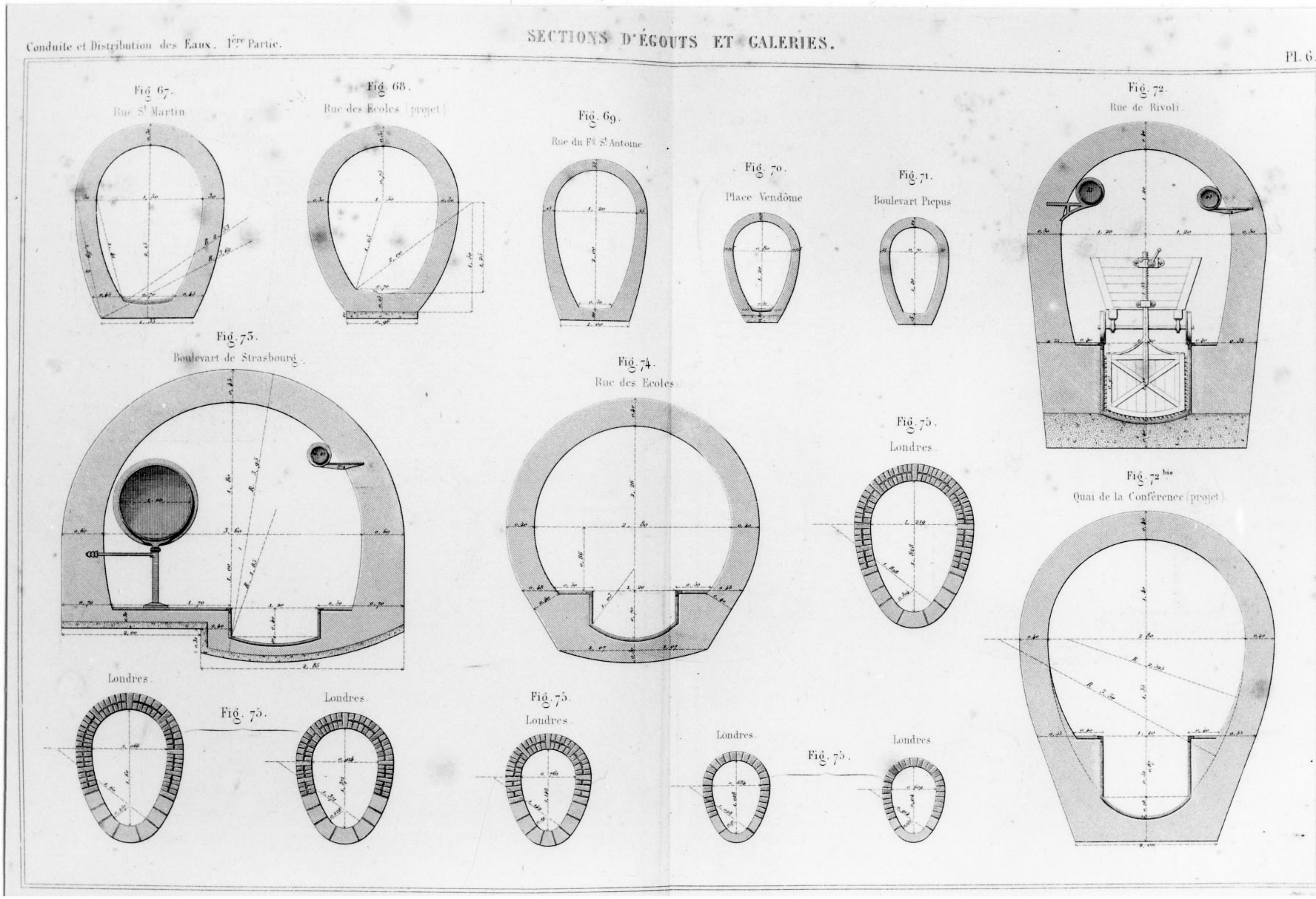
SIA 190:2017

5.2 Concept

Main Sewer Lausanne (Assainissement Lausanne)

5.2 Concept

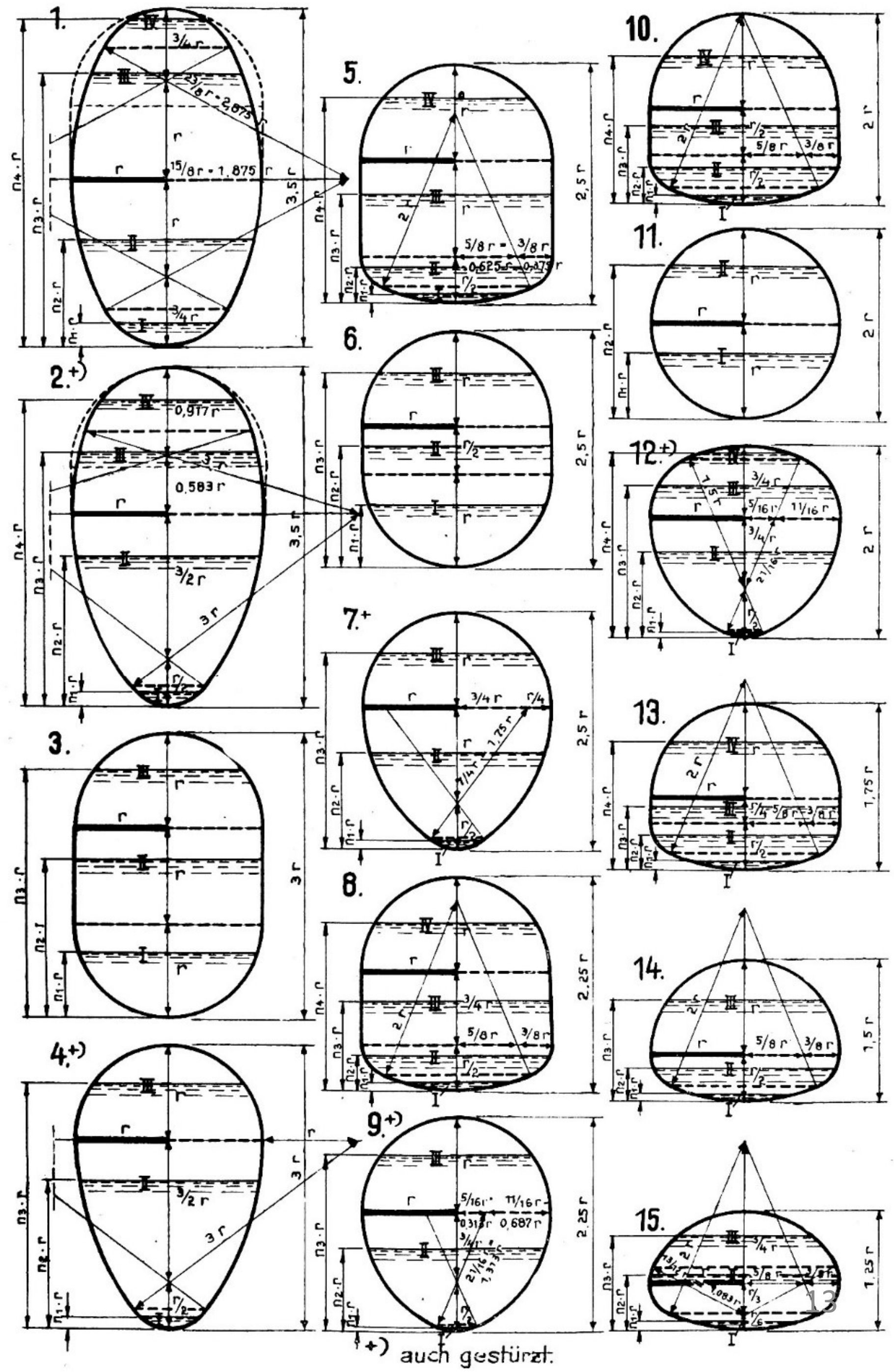
Cross-sections of sewers used in Paris (Dupuit 1845)



5.2 Concept

Conduit types

- Thormann (1944) proposed 15 standard profiles
- Shape is often given by structural aspects and difficult to construct
- Typically constriction at bottom, to increase flow velocity and wall shear stress (solids transport)

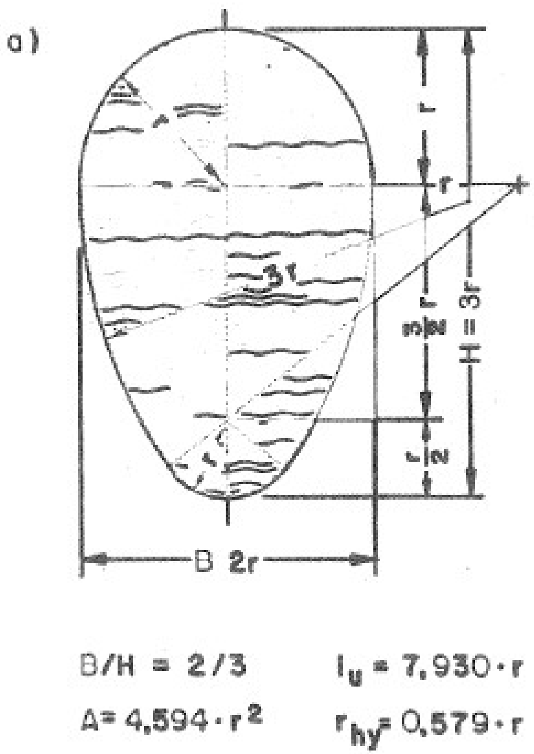


5.2 Concept

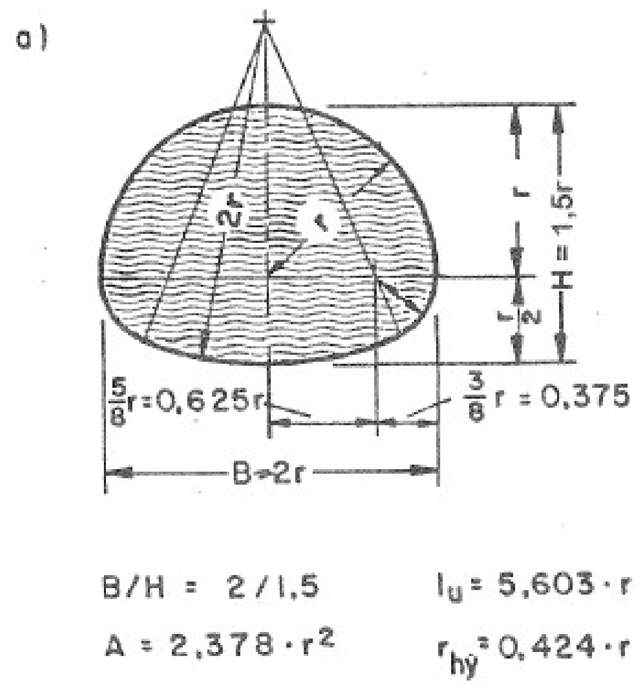
Use exclusively standard profile, if possible circular conduits (= pipes)!

DWA 110 defines as «standard» profiles:

- circular cross-section (conduit)
- egg-shaped cross-section 2:3 (structural of hydraulic aspect)
- horseshoe cross-section 2:1.5 (for small height)



egg-shaped



horseshoe

5.3 Hydraulic design

5.3 Hydraulic design

Code SIA 190

4.1 Dimensionnement hydraulique et calculs

4.1.1 Principes

4.1.1.1 Le dimensionnement hydraulique des conduites est effectué par tronçons pour un débit de dimensionnement Q_{Dim}, généralement défini par le plan général d'évacuation des eaux (PGEE).

4.1.1.2 Lorsque de l'air est entraîné par l'écoulement, le dimensionnement hydraulique doit être effectué pour le débit du mélange eau-air, en respectant le degré de remplissage admissible.

4.1.1.3 Pour éviter la formation de dépôts dans les canalisations, les vitesses d'écoulement par débit de temps sec doivent satisfaire aux conditions minimales du tableau 5. Des approches alternatives basées sur la tension de frottement sont autorisées.

4.1.1.4 L'existence de dépôts n'est pas considérée pour le dimensionnement hydraulique, car un nettoyage périodique des canalisations est admis.

4.1.1.5 Les exigences de la norme SN EN 16933-2 doivent être respectées.

4.1.1.6 En règle générale, la ligne d'énergie du débit de dimensionnement Q_{Dim} devrait être inférieure au niveau du terrain.

4.1.2 But

4.1.2.1 Le dimensionnement hydraulique doit apporter la preuve, par calcul, par des essais sur modèle ou par des mesures in situ, que la canalisation est capable d'évacuer le débit affluent selon les conditions du PGEE et que, pour Q_{Dim}, elle ne se mette pas en charge et ne produise pas de refoulements.

4.1.2.2 L'écoulement en charge de Q_{Dim} peut se produire dans les siphons et les étranglements. Dans un tel cas, l'exploitant du réseau définit la hauteur de charge admissible.

5.3 Hydraulic design

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4.1.3 **Formules d'écoulement**

4.1.3.1 Il est recommandé de calculer les pertes de charges linéaires avec la formule de Prandtl-Colebrook-White. La capacité hydraulique Q en m^3/s d'une conduite simple est ainsi obtenue par la relation:

$$Q = v\,A \qquad (4)$$

$$Q = -2\sqrt{8g}\,\sqrt{RJ_e}\cdot\log\left[\frac{k_b}{14{,}8R} + \frac{2{,}51\nu}{4R\sqrt{8g}\,\sqrt{RJ_e}}\right]A \qquad (5)$$

Pour des avant-projets, en régime turbulent rugueux et en écoulement uniforme, la formule de Manning-Strickler peut être appliquée comme alternative, en particulier lorsque la condition $v \geq 1050\ \nu/k_b$ est satisfaite:

$$Q = K_S\,J_e^{1/2}\,R^{2/3}\,A \qquad (6)$$

v Vitesse moyenne d'écoulement en m/s
A Surface mouillée en m^2
g Accélération gravitaire en m/s^2
R Rayon hydraulique en m
J_e Pente de frottement (perte de charge par unité de longueur)
k_b Rugosité opérationnelle en m
ν Viscosité cinématique en m^2/s
K_S Coefficient de rugosité de Strickler en $m^{1/3}/s$

4.1.3.2 Si aucune variation significative de section, de pente ou de rugosité n'intervient et si l'influence des regards peut être négligée, la condition d'écoulement uniforme peut être admise. Dans ce cas, la pente de frottement J_e est assimilée à la pente de la canalisation J_s (annexe B, figure 17: k_b = 1,0 mm et figure 18: k_b = 1,5 mm).

5.3 Hydraulic design

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4.1.4 **Remplissage partiel**

4.1.4.1 Le taux maximal admissible de remplissage partiel z_{max} doit permettre d'éviter l'occupation totale de la section, c'est-à-dire le passage d'un écoulement à surface libre à un écoulement en charge. L'influence de remous et de pulsations peut ainsi être réduite. Lorsqu'un mélange eau-air se forme dans l'écoulement, il doit être considéré dans la définition du taux maximal admissible de remplissage.

4.1.4.2 Le taux maximal admissible de remplissage partiel z_{max} vaut:

- pour les profils circulaires $z_{1,max} = h/d_i = 0{,}85$ (8)
- pour les autres profils $z_{2,max} = A/A_{plein} = 0{,}85$ (9)

h Hauteur d'eau moyenne perpendiculaire à l'axe du tuyau en m
d_i Diamètre intérieur du tuyau circulaire en m
A Surface mouillée perpendiculaire à l'axe du tuyau en m^2
A_{plein} Surface intérieure totale perpendiculaire à l'axe du tuyau en m^2

4.1.4.3 Le taux maximal admissible de remplissage partiel z_{max} doit être réduit dans les situations suivantes:

- lorsque le nombre de Froude *Fr* est compris entre 0,8 et 2,0, (10)
- aux confluences et aux raccordements de canalisations,
- en cas de réduction de section,
- lorsque l'écoulement est fortement torrentiel.

4.1.5 **Conduites à forte pente**

4.1.5.1 Une distinction est faite entre canalisations à faible et à forte pente. Dans les canalisations à forte pente, l'écoulement est caractérisé par un mélange eau-air qui accroît la section mouillée. Ce phénomène peut se produire pour les pentes supérieures à 6 % dans les grandes canalisations ($d_i \geq 1500$ mm) et supérieures à 11 % dans les petites canalisations ($d_i \leq 350$ mm).

4.1.5.2 Lorsqu'un mélange eau-air se produit, la section de conduite doit être augmentée. Le dimensionnement doit alors être effectué avec le débit du mélange, et le taux maximal de remplissage partiel ne doit pas dépasser 85 % (annexe B, figure 20).

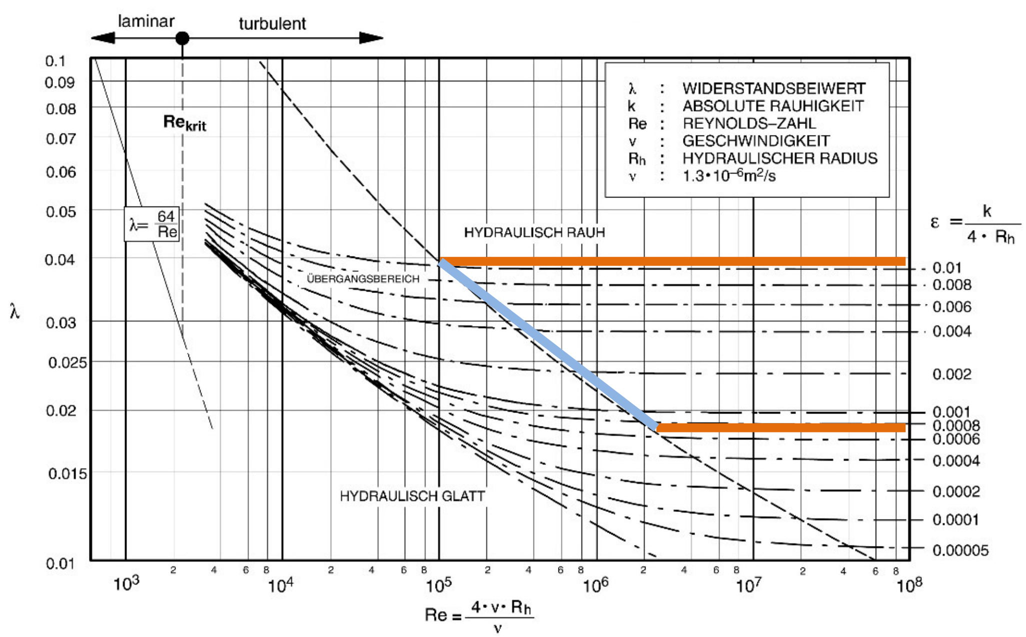
5.3 Hydraulic design

If uniform flow condition emerges for hydraulic rough regime, then the GMS equation may be *explicitly* applied as

$$V = K\, S_O^{1/2}\, R_h^{2/3}$$

Precondition (Colebrook & White): relative roughness $7\cdot10^{-4} < k_s/(4R_h) < 7\cdot10^{-2}$ and viscosity $k_s > 30\nu[g^2 S_O^2 Q]^{-1/5}$

Precondition (GMS): $18 < K < 87$ and $K < 170(S_O^2 Q)^{1/30}$, K in $m^{1/3}s^{-1}$



5.3 Hydraulic design

Strickler (GMS) is based on 4 hydraulic hypotheses, all for Q_M

1. Conduit «flowing full»
2. Uniform flow
3. turbulent rough regime, substitution
4. Circular flow section

If these hypotheses are respected, then the prediction is precise, otherwise it represents a good approximation

5.3 Hydraulic design

Design equation based on GMS (uniform flow, turbulent rough regime) and continuity eq. (Hager 1999)

$$Q_V = AV = \frac{\pi D^2}{4} K S_O^{1/2} R_h^{2/3}$$

and

$$R_h = \frac{A}{U} = \frac{\pi D^2}{4\pi D} = \frac{D}{4}$$

results in (uniform flow and flowing full condition)

$$Q_V = \frac{\pi}{4^{5/3}} K S_O^{1/2} D^{8/3} = 0.31 K S_O^{1/2} D^{8/3}$$

with

$$\frac{K k_s^{1/6}}{\sqrt{g}} = 8.2$$

5.3 Hydraulic design

EXAMPLE
Q_M=10 m^3/s

Which *D* is adequate?

Circular profile, concrete conduit
From topography S_O=0.005

5.3 Hydraulic design

SIA 190:2017: If assuming flowing full condition (geometry), Darcy & Weisbach and Colebrook & White results in

$$Q = -2\sqrt{8g}\sqrt{RJ_e} \cdot \log\left[\frac{k_b}{14.8R} + \frac{2.51\nu}{4R\sqrt{8g}\sqrt{RJ_e}}\right]A$$

Variables:

Q_v	discharge for multiple partial filling ratios
R	hydraulic radius
v	kinematic viscosity (1.31 10^{-6} m^{-2}/s for 10°)
J_e	energy line slope
k_b	operational sand roughness
g	acceleration of gravity
A	flow (wetted) surface

Valid for all hydraulic regimes, for $S_E \neq S_O$ and $S_E = S_O$, and all partial filling ratios!

5.3 Hydraulic design

DWA A110: If assuming flowing full condition (geometry), Darcy & Weisbach and Colebrook & White results in

$$\frac{Q_v}{\sqrt{g S_E D^5}} = -\frac{\pi}{\sqrt{2}} \log\left[\frac{2.51\nu}{\sqrt{2 g S_E D^3}} + \frac{k_s}{3.71 D}\right]$$

Variables:

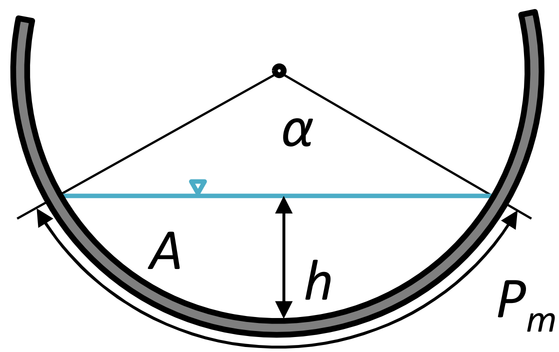
Q_v	discharge for full flow condition
D	conduit diameter implicit computation
v	kinematic viscosity (1.31 10^{-6} m^{-2}/s for 10°)
S_E	energy line slope
k_s	equivalent sand roughness (for standard plastic conduits 0.1 mm)
g	acceleration of gravity

Valid for all hydraulic regimes, and for $S_E \neq S_O$ and $S_E = S_O$

Hager (1999), DWA 110

5.3 Hydraulic design

Partial pipe filling



Géométrie du segment

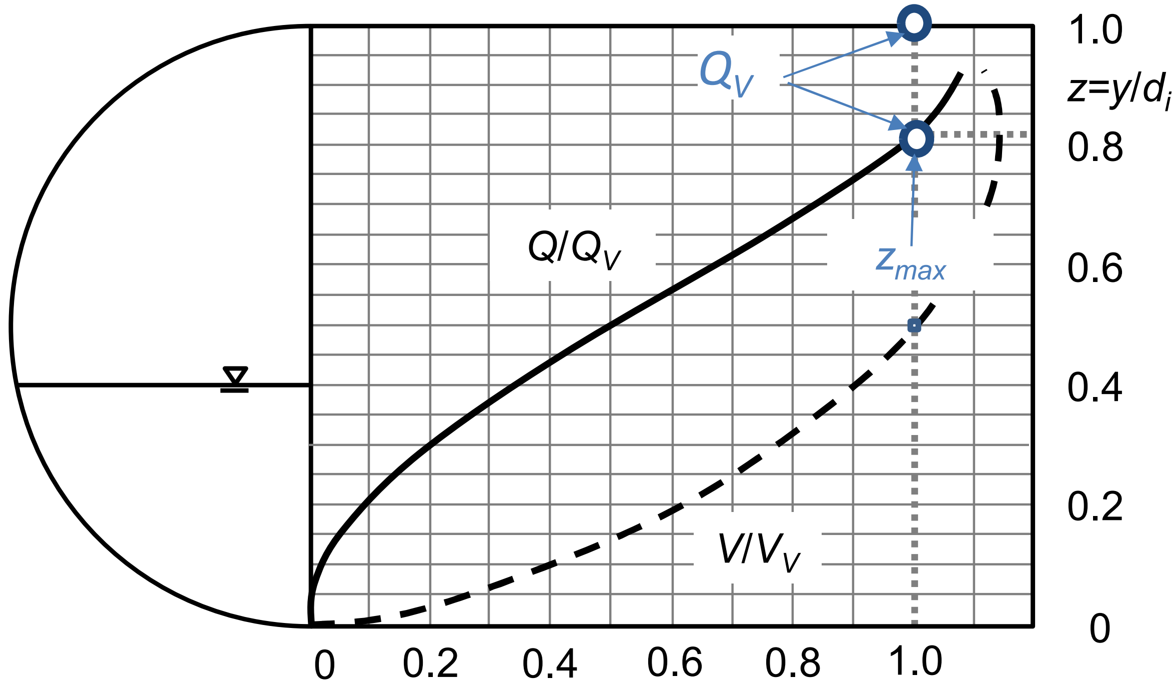
- 0≤*α*≤360
- Surface mouillé *A*, hauteur *h*, périmètre mouillé P_m
- $R_h(\alpha)$ et $h/D(\alpha)$ suivent

Ecoulement (Strickler)

$$\frac{Q}{Q_V} = \frac{K\sqrt{J}R_h^{2/3}A}{K\sqrt{J}R_{hV}^{2/3}A_V} = \frac{R_h^{2/3}A}{R_{hV}^{2/3}A_V}$$

Ecoulement (Darcy-Weisbach, Franke 1955)

$$\frac{Q}{Q_V} = \frac{A}{A_V}\left(\frac{R_h}{R_{hV}}\right)^{1/8}\sqrt{\frac{R_h}{R_{hV}}}$$



5.3 Hydraulic design

Partial pipe filling of circular profile (Hager 1999)

- $Q_{(M, m)} \leq Q_V$
- Precise solution is complicated because of geometry (precedent slide)
- Approximation for uniform flow (index *N*), with GMS and Sauerbrey (1969)
- Error <5%, on safe side

Relative uniform discharge $$q_N = \frac{Q}{K S_O^{1/2} D^{8/3}}$$

Partial filling ratio $$y_N = \frac{h_N}{D} = 0.926\left[1 - \sqrt{1 - 3.11 q_N}\right]^{1/2}$$

Wetted cross section area $$\frac{A}{D^2} = \frac{4}{3} y^{3/2}\left(1 - \frac{y}{4} - \frac{4y^2}{25}\right)$$

5.3 Hydraulic design

EXAMPLE

Q_m=0.5 m^3/s

K=80 m$^{1/3}$s^{-1}

S_O=0.005

D=2.00 m

Solids transport at Q_m. What is V_m?

5.3 Hydraulic design

For Q_M (uniform flow & flowing full condition): no undulating flow conditions (0.8<F<1.2), otherwise flow choking is probable.

For circular, partial filled profiles

$$\mathsf{F} \neq \frac{V}{\sqrt{gh}}$$

but

$$\mathsf{F}^2 = \frac{Q^2}{gA^3}\frac{\mathrm{d}A}{\mathrm{d}h} \qquad \mathrm{Fr} = \sqrt{\frac{Q^2 b}{gA^3}} \qquad \mathsf{F} = \frac{Q}{\sqrt{gDh^4}}$$

and

$$h_c = \sqrt{\frac{Q}{(gD)^{1/2}}}$$

(Hager 1999)

5.3 Hydraulic design

Why is F different in a circular than in rectangular profile?

Bernoulli (with continuity eq.)

$$H = h + \frac{Q^2}{2gA^2}$$

Minimum energy if dH/dh=0 (derivative)

$$\frac{\mathrm{d}H}{\mathrm{d}h} = 1 - \frac{2Q^2}{2gA^3}\frac{\mathrm{d}A}{\mathrm{d}h} = 0$$

Definition of F following William Froude [1810 - 1879]

$$1 = \frac{Q^2}{gA^3}\frac{\mathrm{d}A}{\mathrm{d}h} = \mathrm{F}^2$$

For rectangular channel $Q^2=h^2b^2V^2$ and $A=bh$, so that dA/dh=b

$$\mathrm{F}^2 = \frac{h^2b^2V^2}{gh^3b^3}b = \frac{V^2}{gh}$$

5.3 Hydraulic design

EXAMPLE

Q_m=0.5 m^3/s

K=80 $m^{1/3}s^{-1}$

S_O=0.005

D=2.00 m

Is the flow in the critical regime?

5.4 Choking

5.4 Choking

Choking is the abrupt transition from free surface flow (partial filling up to 85%) to pressurized flow

Main reasons

- the flow touches the conduit ceiling
- air is mixed into flow
- air is entrapped at conduit ceiling

Consequences

- Pressurized flow in conduit
- air entrainment and partial transport, or accumulation
- reductions of discharge capacity
- pulsations with pressure peaks (fatigue)
- geysiring

⇨ Air is relevant for conduit design

5.4 Choking

Undulating flow conditions generate choking

Choking number $C_o = y_o F_o$

No choking if

$C_o<0.9$ for $1<F_o<2$
(undular jump)

Local choking. Free surface flow conditions in downstream possible, if aerated sufficiently

(Gargano and Hager, JHE 128(11), 2002)

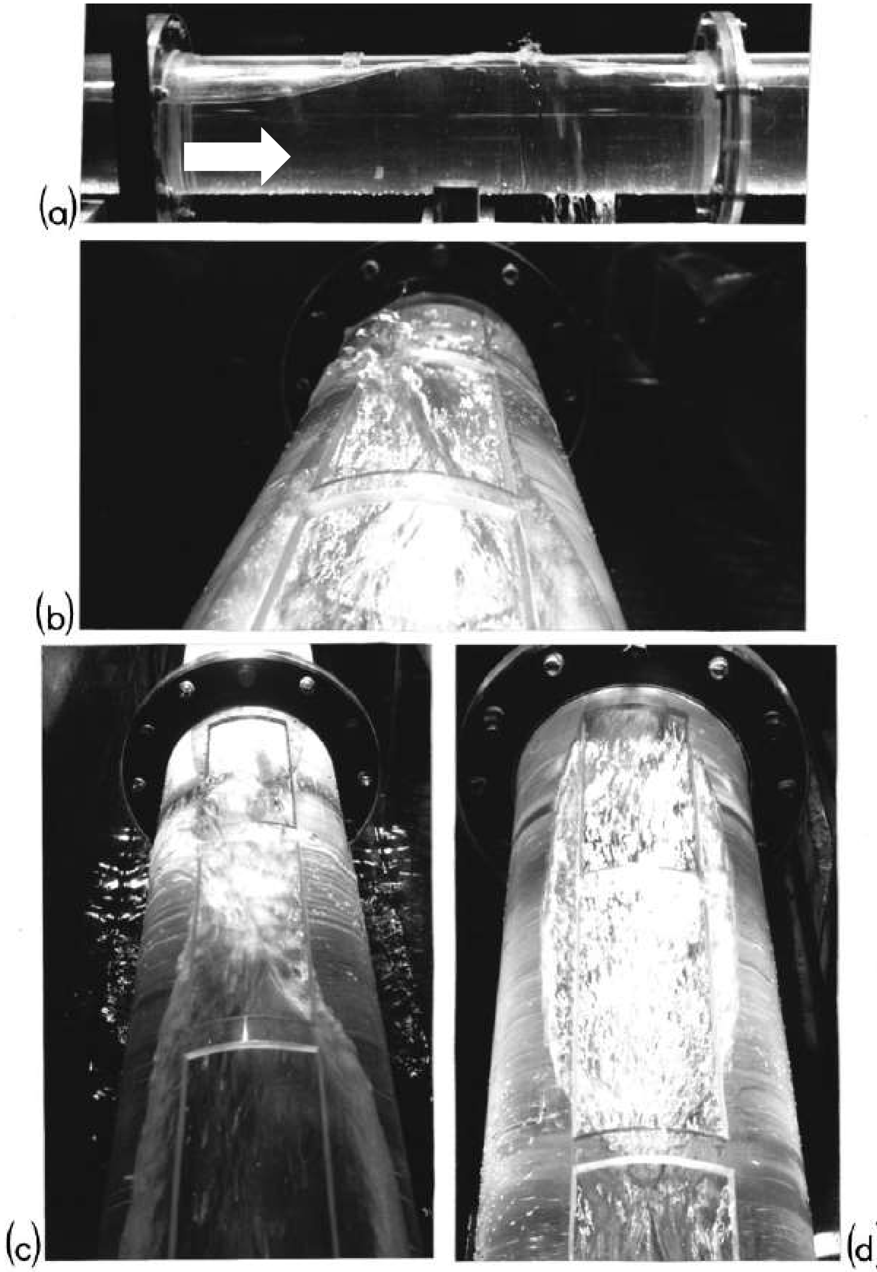


Fig. 2. Undular hydraulic jump for $y_o=0.62$ and $F_o=1.50$ involving choking flow

5.4 Choking

EXAMPLE

Q_M=10 m^3/s

Q_m=0.5 m^3/s

K=80 m$^{1/3}$s^{-1}

S_O=0.005

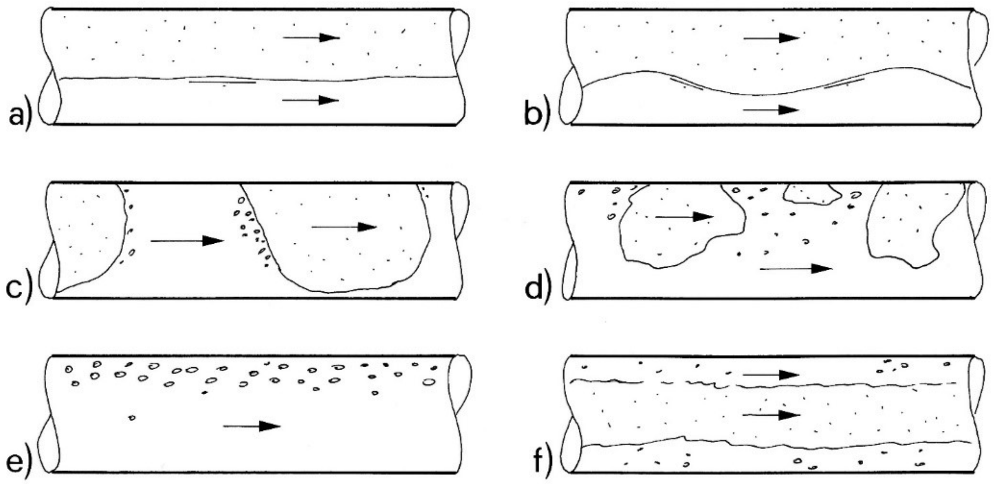
D=2.00 m

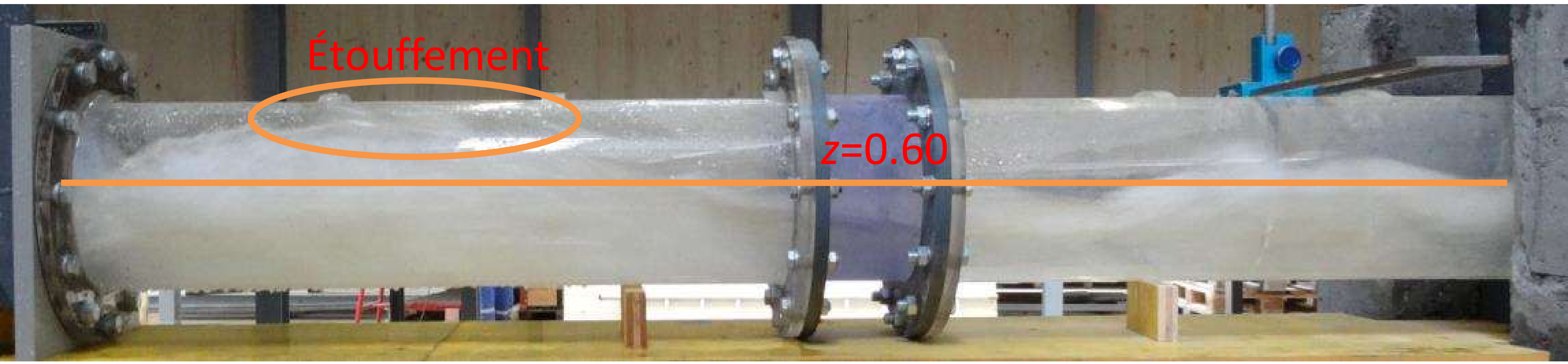
Is there a problem with undulating (choking due to critical) flow?

5.4 Choking

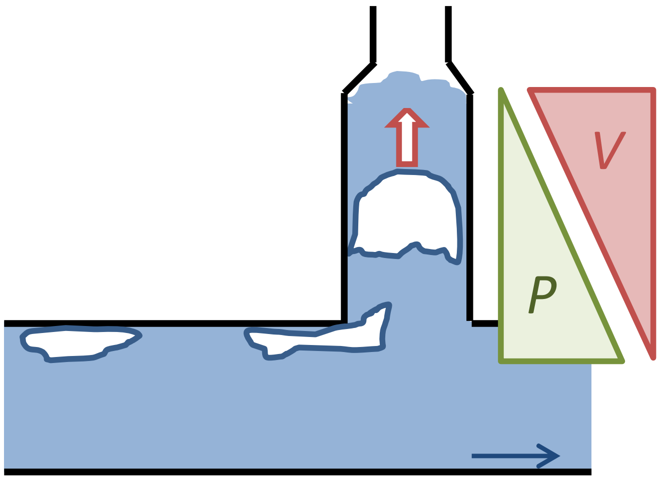
Air transport in conduit flows (Hager 1999)

a) Stratified flow
b) Wave flow
c) Slug flow
d) Plug flow
e) Bubbly flow
f) Annular flow





5.4 Choking

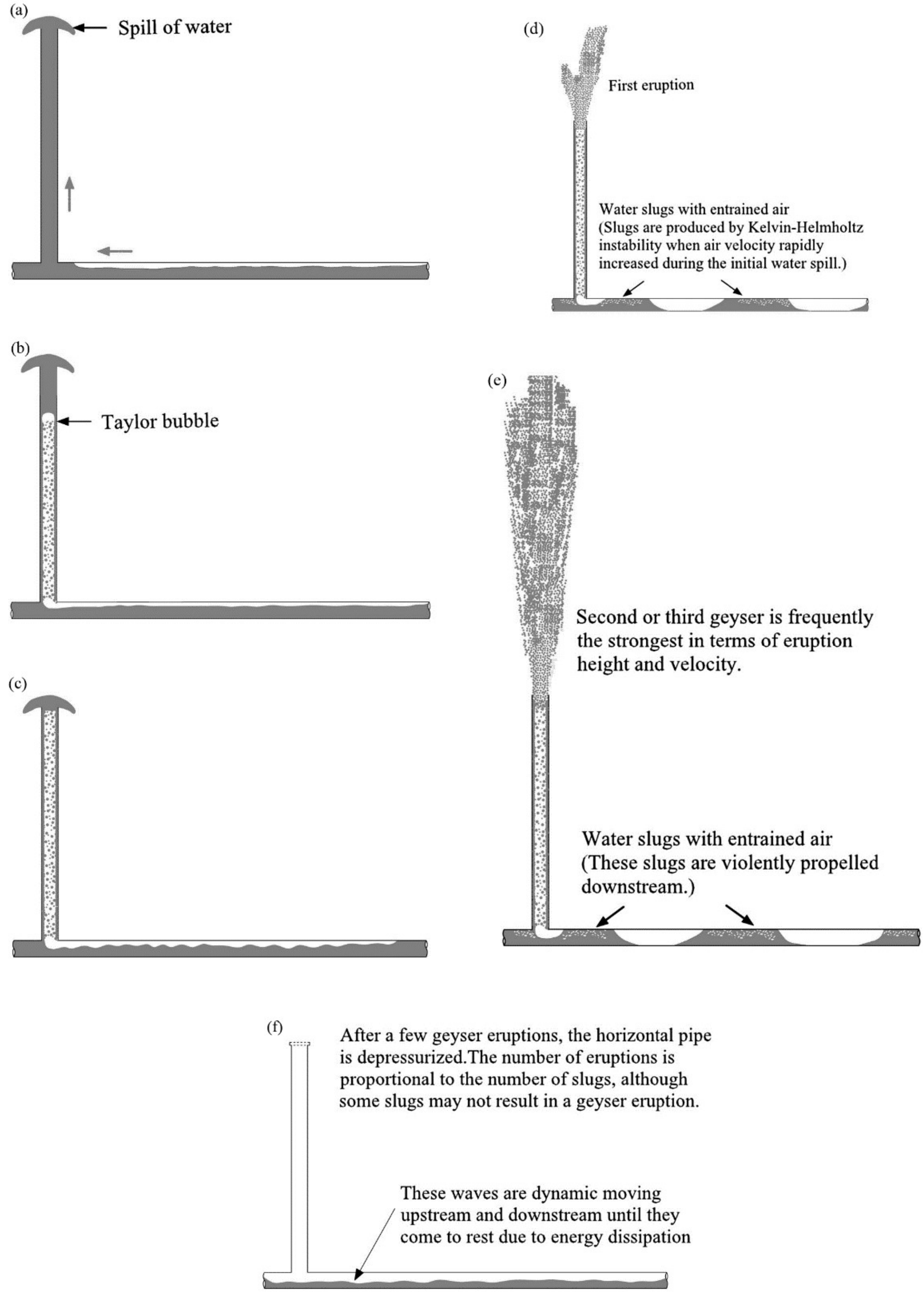


Minnesota storm sewer ⇨

5.4 Choking

Geysering in vertical Pipe

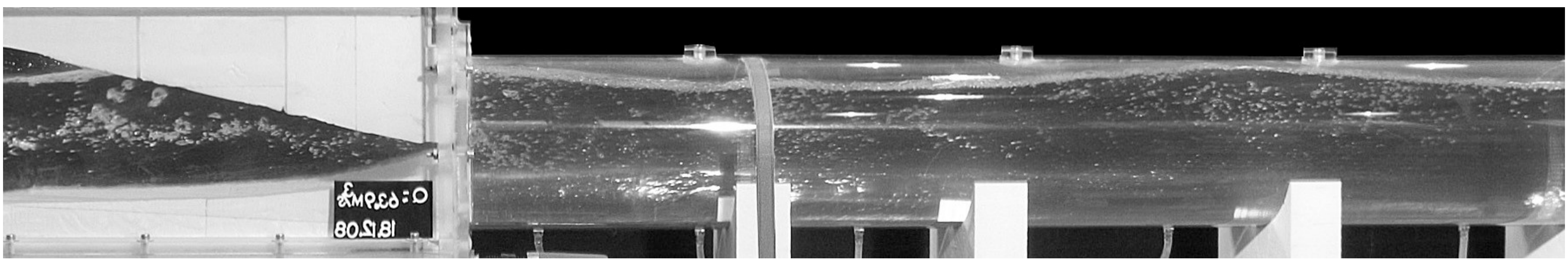
(Leon, Elayeb & Tang, JHR, 2018)

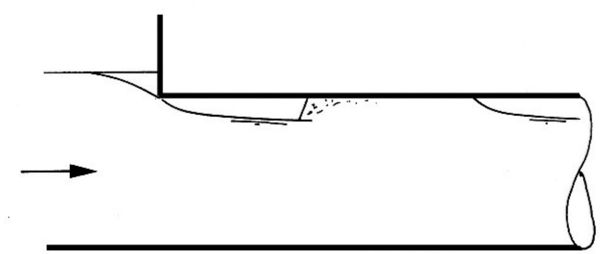


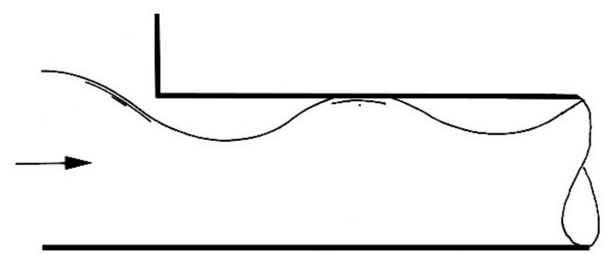
5.4 Choking

Formation

- At conduit inlets (manholes, small downstream *D* = reduction cross-section)
- Shock waves (supercritical flow)
- Undulating flow surface (F=1)
- Air entrainment or detrainment
- Discharge pulsations
- Submergence from the downstream (hydraulic jump)





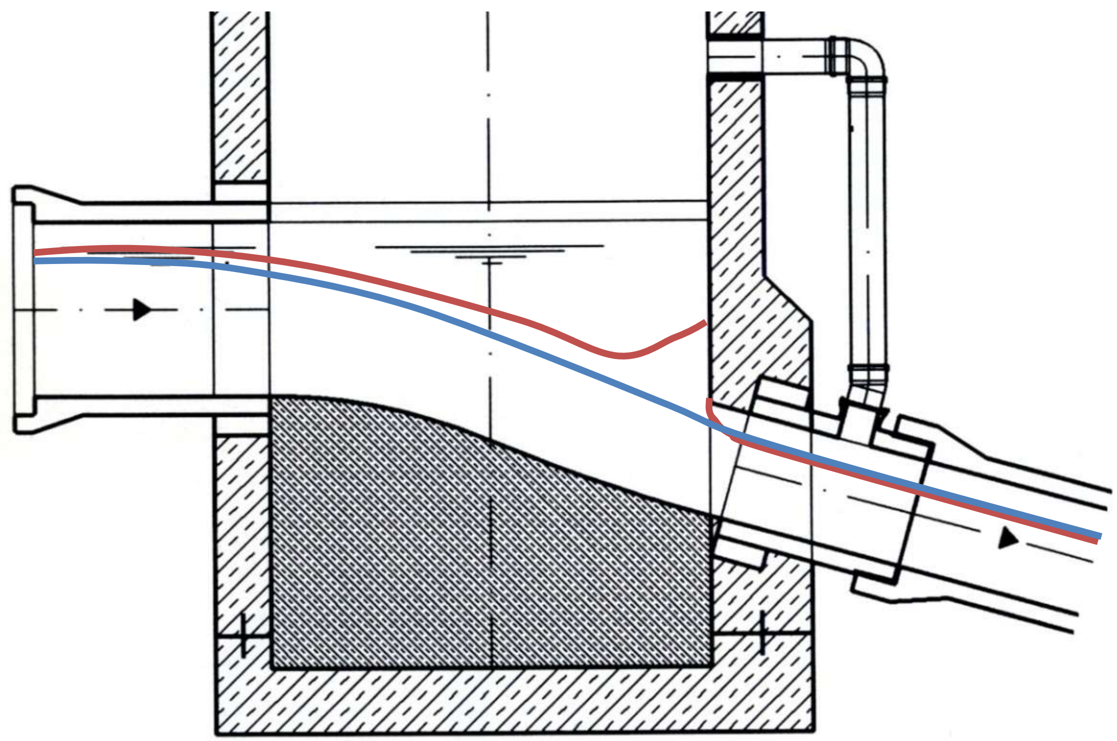


5.4 Choking

Consequences of air entrainment: Pulsations, air accumulation, discharge reduction, pressure peaks, “milk”

For manhole (conduit inlet):
- Increase downstream conduit diameter
- Provide aeration

Discharge capacity decreases, as
$Q=f(H^{3/2})$ for overflow
$Q=f(H^{1/2})$ for gate flow



Bottle experiment

5.4 Choking

In conduit because of undular or supercritical flow (shock waves)

Reduce partial filling ratio $y=h/D$ below the maximum given by Sauerbrey (1969)

- For $0 \leq S_o \leq 12‰$ ⇨ $y_{MC}=0.92-30S_o$
- For $S_o>12‰$ ⇨ $y_{MC}=0.55$

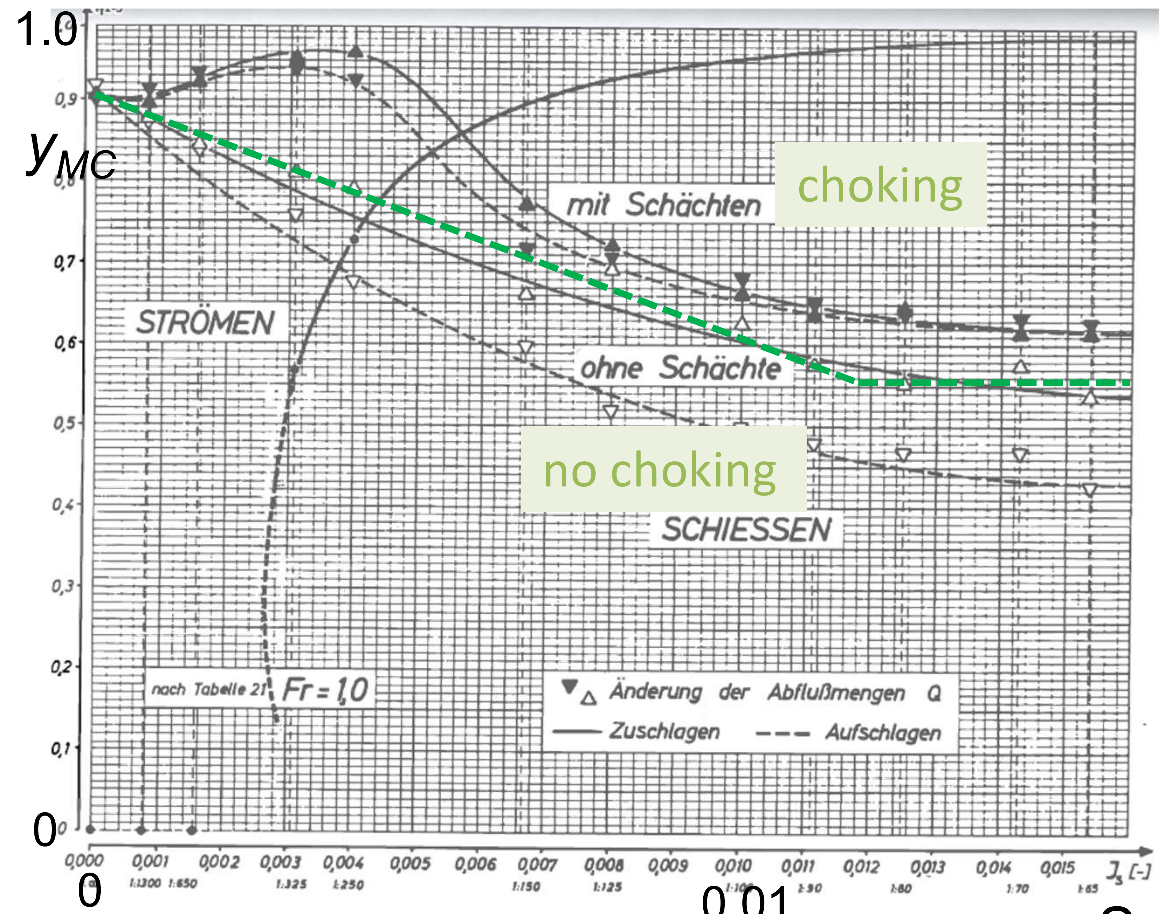


Abb. 56 Umschlagpunkte und Fließarten in der Versuchsanlage

5.4 Choking

EXAMPLE

Q_M=10 m^3/s

Q_m=0.5 m^3/s

K=80 $m^{1/3}s^{-1}$

S_O=0.005

D=2.20 m

Choking due to supercritical flow (shockwaves)?

5.5 Steep sewer

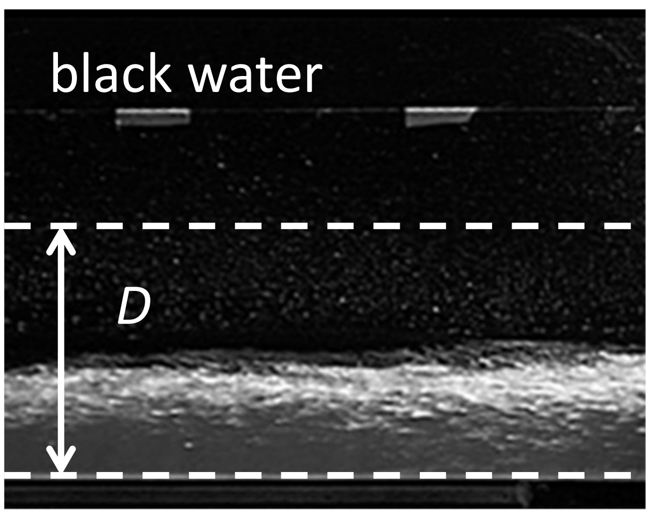
5.5 Steep sewer

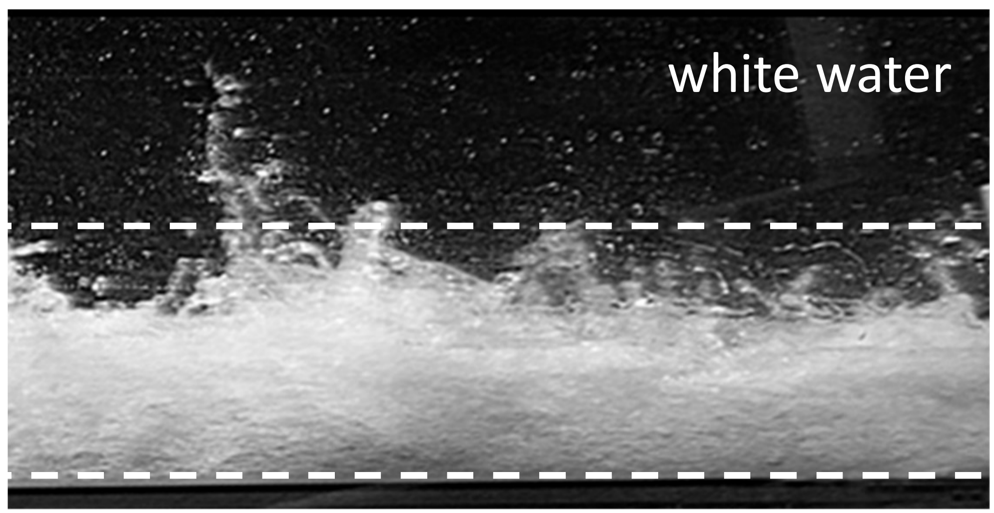
Steep topography ⇨ steep sewer conduits or fall manhole (chapter 7)

High Reynolds number = high turbulence = air self-entrainment into flow

"White water"

Consequences: flow depth increases as two-phase flow, rough flow surface, choking



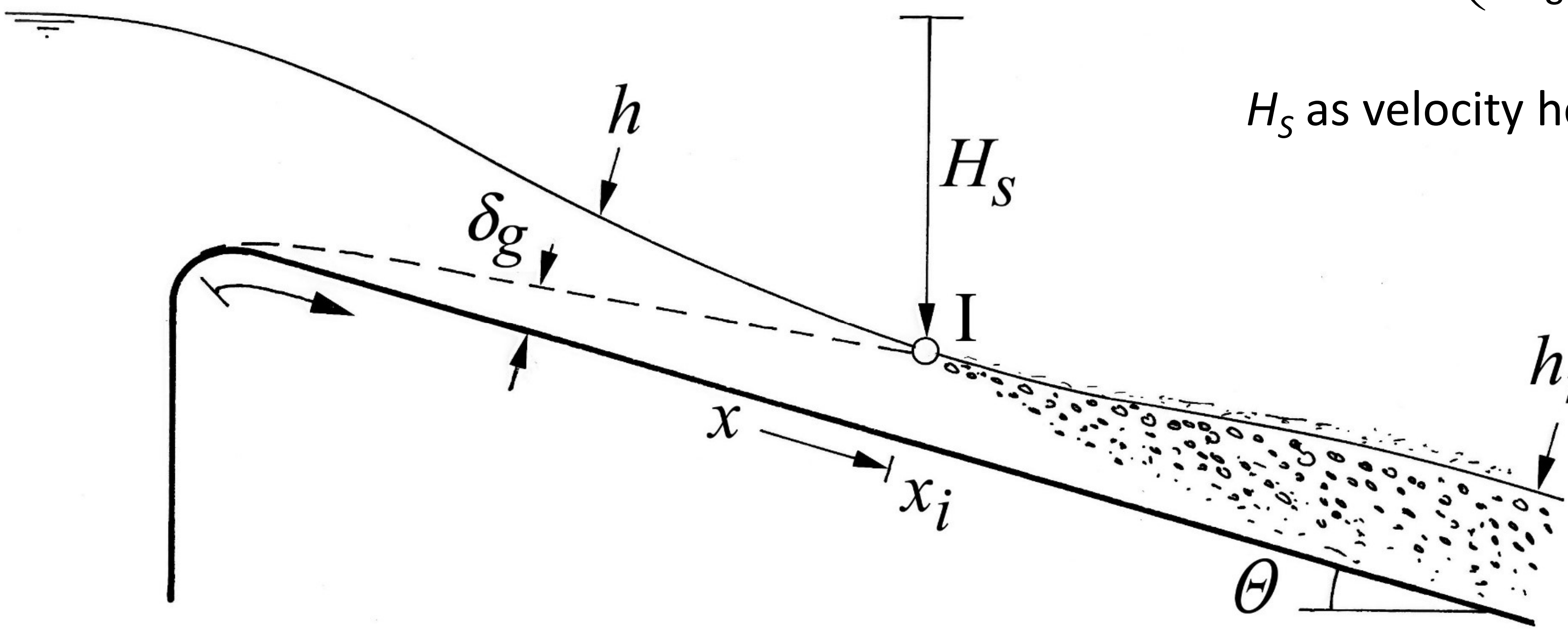


5.5 Steep sewer

General air entrainment mechanism

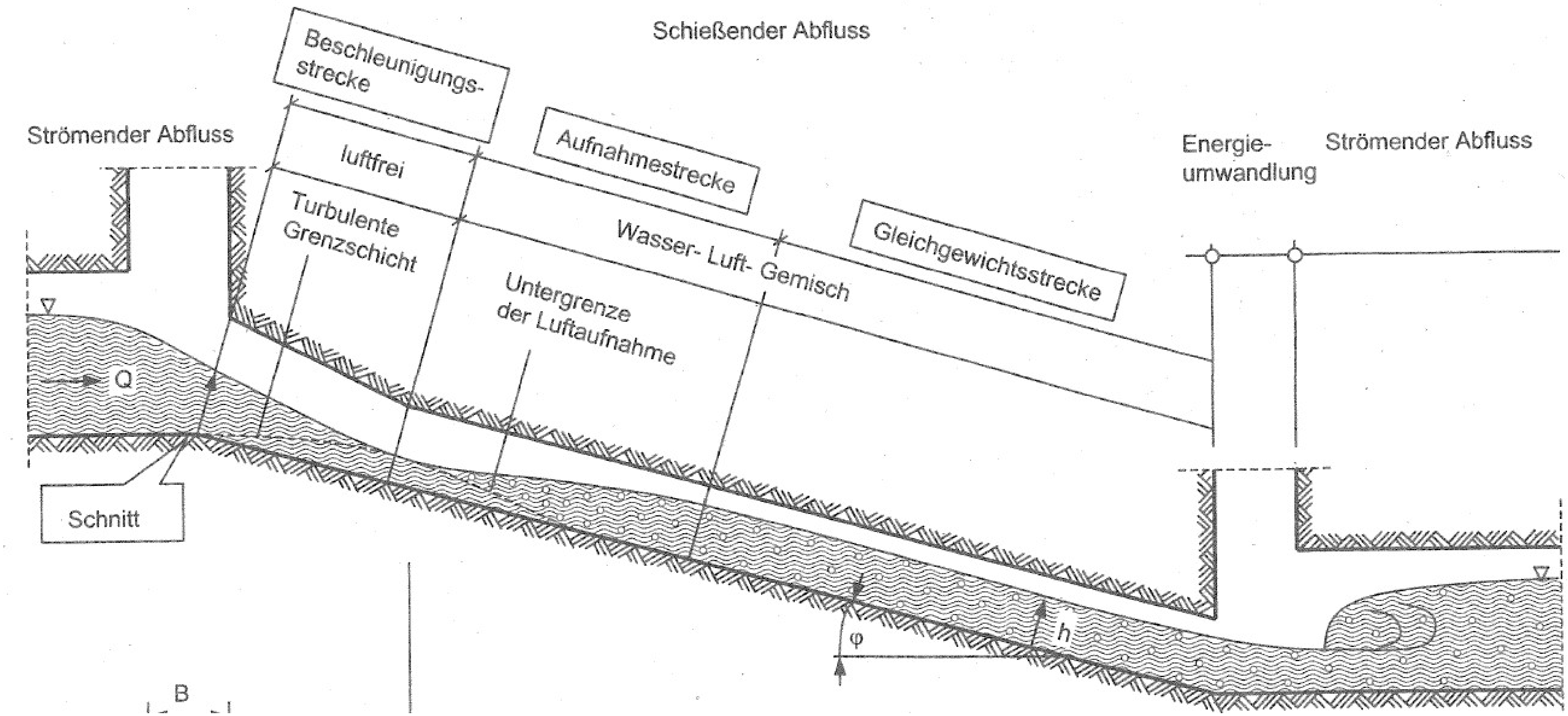
$$\frac{\delta_g}{x} = 0.021\left(\frac{k_s}{H_s}\right)^{0.1}$$

H_S as velocity head



5.5 Steep sewer

Sewer air entrainment mechanism (ATV A110)



5.5 Steep sewer

General approach

- Derive draw-down curve for black-water as $h(x)$
- Compute thickness of TBL $\delta_g(x)$
- Self aeration point at location x where $h=\delta_g$

For sewer (Hager 1999)

Roughness characteristics for circular profile

$$\chi = \frac{K {S_O}^{1/2} D^{1/6}}{g^{1/2}}$$

If $\chi<8$ no self-aeration in uniform flow

If $\chi>8$ self-aeration in uniform flow

Aeration increases flow depth!

Two-phase air-water flow depth is larger than black-water flow depth

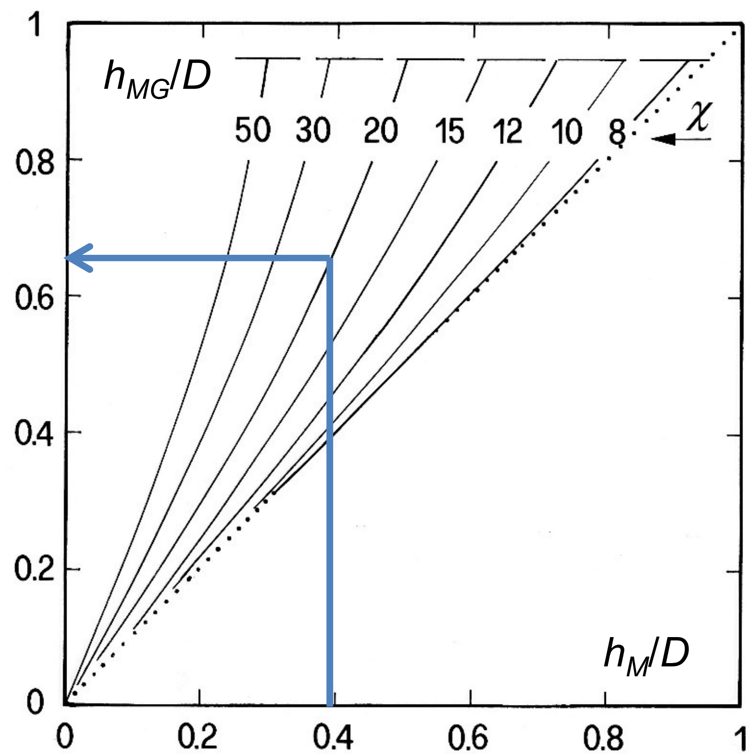
5.5 Steep sewer

Approximation of two-phase air-water uniform flow depth h_{MG}

$$\frac{y_{MG}}{d_i} = \frac{1}{4}\chi^{2/3}\left(\frac{y_M}{d_i}\right)^{10/9}$$

The partial filling ration y_M is higher for h_{MG} than for h_M

General hydraulic characteristics of flow relates to black-water!



Example: If y_M=h_M/D=0.4 and χ=20
⇨ y_{MG}=0.65

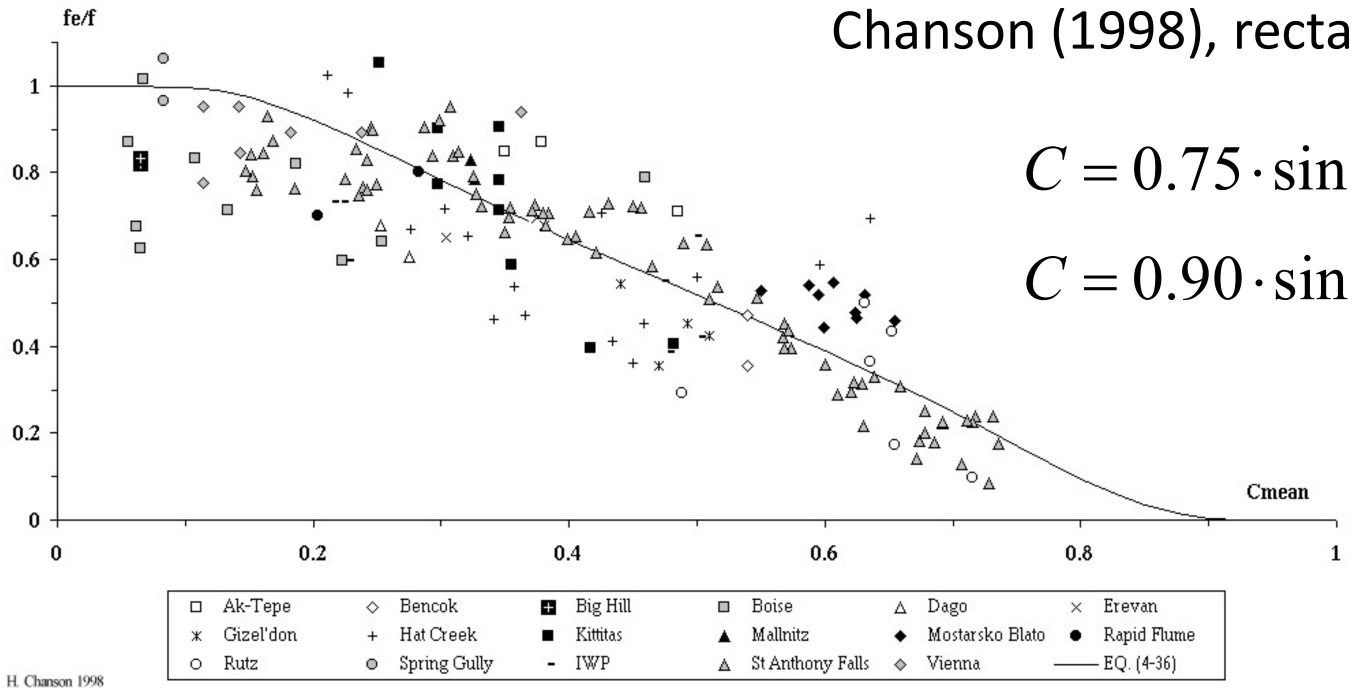
5.5 Steep sewer

EXAMPLE
Q_M=10 m^3/s
Q_m=0.5 m^3/s
K=80 m$^{1/3}$s^{-1}
S_O=0.005
D=2.20 m

Choking due to two-phase air-water mixture flow (steep sewer)?

5.5 Steep sewer

Flow velocity of air-water mixture flow is increased



Chanson (1998), rectangular flow section

$$C = 0.75 \cdot \sin\varphi^{0.75}$$ Hager (1991)

$$C = 0.90 \cdot \sin\varphi$$ Chanson (1996)

- SIA 190 says $\frac{V_{MG}}{V_M} = 1 - C^2$ following Volkart (1978), contradicting literature
- Volkart (1978) says y_{MG}<0.9 (Sauerbrey applies for black-water only)!

5.6 Effect of manhole

5.6 Effect of manhole

The conduit is designed for *assumed* uniform flow

- Different flow characteristics at upstream and downstream end
- Check inflow conditions and draw down curves near manholes

Critical cases

- Super-critical inflow and sub-critical d/s pipe uniform flow ⇨ hydraulic jump
- Sub-critical inflow and super-critical conduit flow with reduction of *D* ⇨ choking

Check

- Connection of conduit to manhole (change of bottom slope or diameter)
- Draw drawn curves
- Changes in flow regime (e.g. from super- to subcritical)

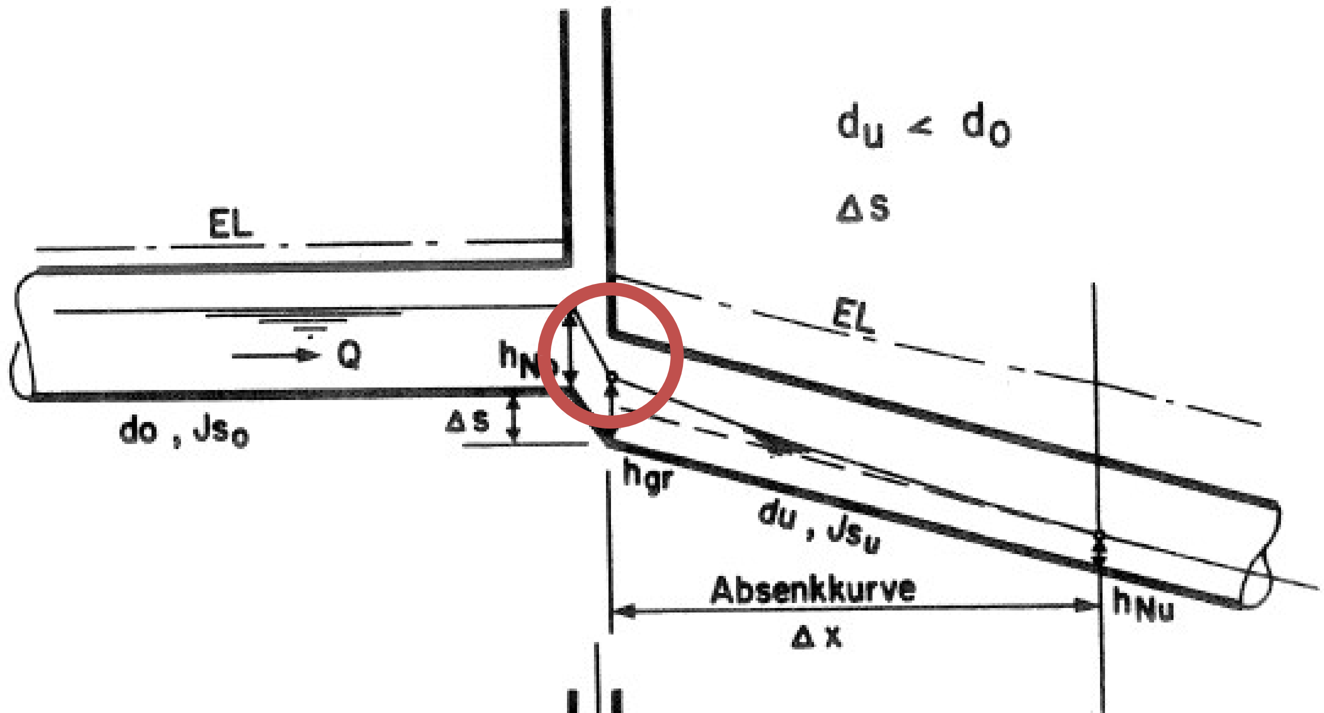
5.6 Effect of manhole

From flat to steep slope

(SIA 53)

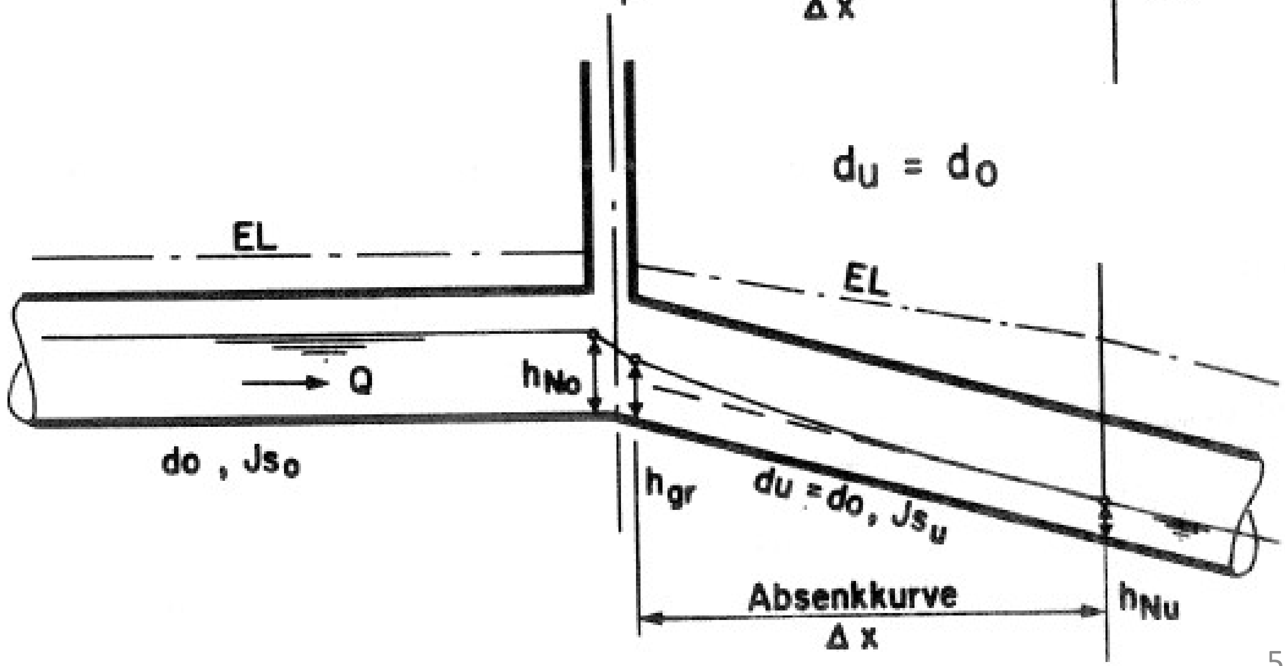
Offset

If D_u<D_o, then Δs



No offset

D_u=D_o



5.6 Effect of manhole

From steep to flat slope

(SIA 53, ATV A110)

Check if hydraulic jump occurs

Depth ratio in conduit

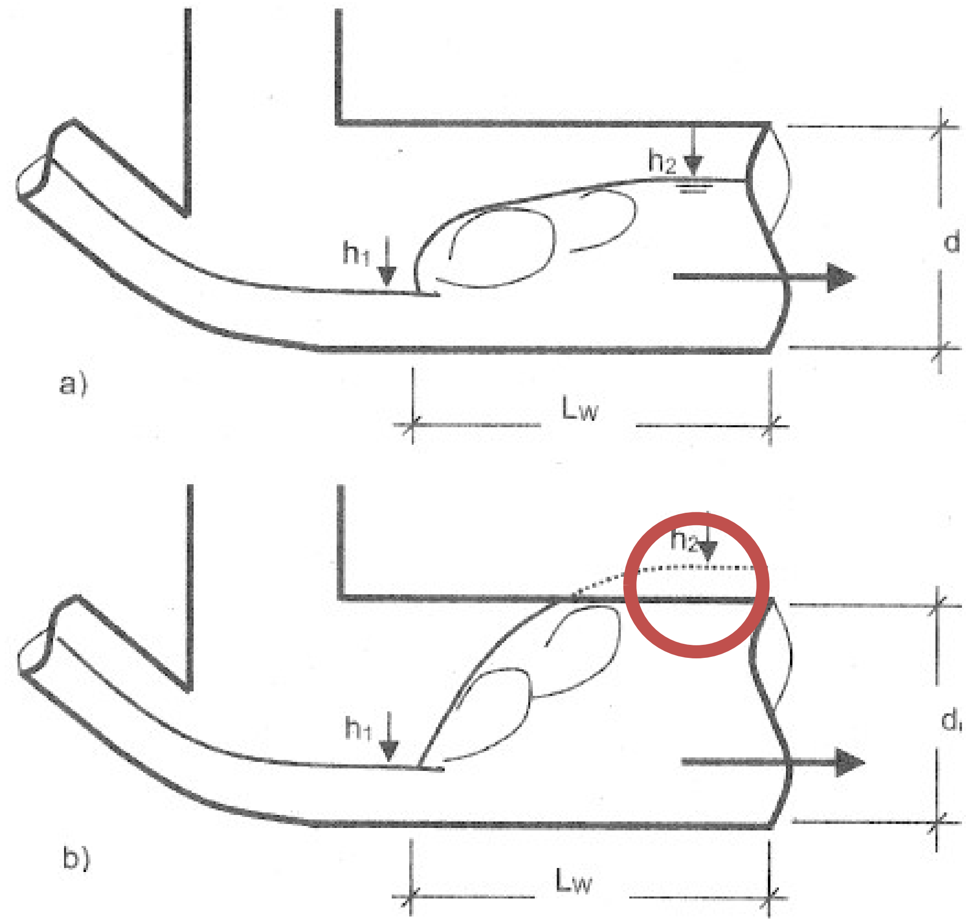
$$Y = \frac{h_2}{h_1} = \mathsf{F}_1^{\,0.9}$$

Roller length L_w

$$\frac{L_w}{h_2} = 4\mathsf{F}_1^{\,0.5}$$

Air entrainment

$$\beta = \frac{Q_a}{Q} = 0.0066(\mathsf{F}_1 - 1)^{1.4}$$



5.7 Losses in flows

5.7 Losses in flows

Two approaches for linear and local losses:

- Operational roughness (simplification)
- Bernoulli (including all the hydraulic aspects)

Operational roughness (SIA 190): local losses are included in “exaggerated” linear roughness

Die Rauigkeitsbeiwerte k_b nach Prandtl-Colebrook-White bzw. K_S nach Strickler berücksichtigen die betriebliche Rauigkeit der Kanalisation.

Tabelle 4 Betriebliche Rauigkeitsbeiwerte

Leitungen	k_b mm	K_S $m^{1/3}/s$
Kreisförmige und kreisähnliche Kanäle mit Schächten und/oder mit Anschlüssen in Schächten	1,0	80
Leitungen mit direkten Anschlüssen zwischen den Schächten	1,5	75
Leitungen aus nicht genormten Rohren	1,5	75
Rechteckkanäle in Beton	$\geq$ 1,7	$\leq$ 75
Gegliederte oder asymmetrische Querschnitte	$\geq$ 2,6	$\leq$ 70

5.7 Losses in flows

ATV Operational roughness (ATV 110)

“Operational” (over-all) roughness k_b for entire system instead of individual values, including friction and local losses

Contains: wall roughness, inexact and altered construction, joints, inlets at manholes, standard manholes, some junctions

Advantage: all-inclusive design without detailed verification
Disadvantage: sometimes too “rough” and thus not economic

Depending on conduit and manhole type k_b=0.5 to 1.5 mm

5.7 Losses in flows

Energy losses according to Bernoulli (individual concept)

Every manhole and conduit is considered individually. Values of k_s and ξ are provided by SIA 190 and ATV 110. For instance, k_s =0.1 mm for PVC conduits

They consist of the sum of

- local losses (manholes) due to streamline curvature (flow separation) because of a modified wall geometry, and
- frictional losses (conduit) due to fluid viscosity and wall roughness. According to Darcy & Weisbach and Colebrook & White. For uniform flown the frictional losses are included since S_E=S_O

If ρ, Q and g are constant between two sections, then the energy head [m] is

$$z_1 + \frac{p_1}{\rho g} + \frac{{V_1}^2}{2g} = z_2 + \frac{p_2}{\rho g} + \frac{{V_2}^2}{2g} + \left(\sum\left(f_i \frac{L_i}{D_i}\right) + \sum \xi_i\right)\frac{{V_i}^2}{2g}$$

Exercise, Homework

Q_M=64 m^3/s
S_O=5.6‰
D=3.80 m
TBM, concrete segment surface
Free surface flow required

Does it work?

Exercise, Homework

Projet A

- Q_M=10 m^3/s
- J_s=0.005 (0.5%)
- K=80 $m^{1/3}/s$
- (Aucun influence des regards, transport des sédiments assuré, application M-S adéquate)
- Diamètre *D* ?

Projet B

- Q_M=1 m^3/s
- J_s=0.2 (20%)
- K=80 $m^{1/3}/s$
- (Aucun influence des regards, transport des sédiments assuré, application M-S adéquate)
- Diamètre *D* ?

5.8 Pro memoria

5.8 Pro memoria

How to design a sewer conduit?

1. define conduit shape
2. derive Q_M ⇨ D with GMS
3. ensure that $F \neq 1$ ⇨ increase D
4. Assure that choking number $C<0.9$ if $1<F<2$ ⇨ increase D
5. Check choking for supercritical flow (maximum y from Sauerbrey) ⇨ increase D
6. air entrainment? If yes, increase D
7. Has the manhole an effect on the conduit diameter?
8. derive Q_m , check if V_m and S_{Om} are sufficient, otherwise increase D or S_{Om}

Iterative approach! Every item typically demands for a re-computation of preliminary items. Measures:

- increase D
- increase S_{Om}
- sewage pumping
- drop manholes

5.8 Pro memoria

- Discharge capacity ⇨ uniform flow & flowing full condition with *y*=85%
- Minimum discharge ⇨ solids transport, depositions
- $Q_m < Q_M \leq Q_V$

- (Preliminary) Design for uniform flow conditions, GMS (respect limits)
- Uniform flow = equilibrium between driving and retaining forces
- EL, PL and S_O are parallel ⇨ every streamwise flow cross-section is similar
- For Darcy & Weisbach and Colebrook & White use $D=4R_h$

- Use exclusively straight conduits without any changes
- Use standard sewer conduits, best circular pipes
- Locate every change in a manhole
- Respect maximum manhole distance and minimum conduit diameter
- ATV concept of operative roughness

5.8 Pro memoria

- Transition from “flowing full condition” to pressurized flow occurs abruptly
- Choking is equal to a failure of the system
- Choking generates pressure peaks, a discharge capacity reduction, pulsations and geysiring

- Steep sewers are delicate: provide aeration or drop manhole
- Criterion χ>8 for self-aeration in uniform flow
- Sufficient conduit diameter at manhole exit branch

Questions?