LECTURE NOTES – 5

« HYDROELECTRIC POWER PLANTS »

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CHAPTER 5

Main Types of High – Head Power Plant Developments

Power plants operating under a head higher than 50 m may be termed as *high-head power plants*. Three main types of high-head power developments may be discerned.

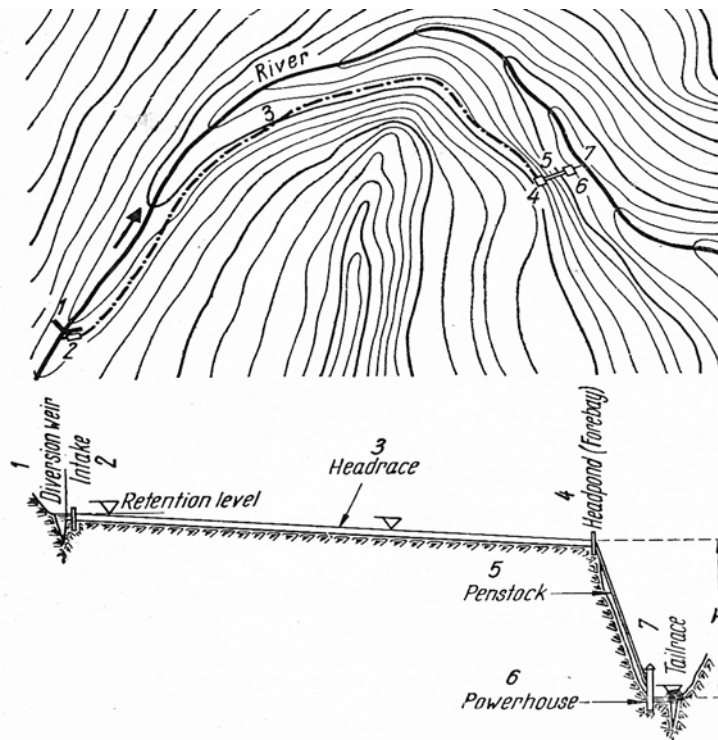### A) Diversion Canal Type Plant

Figure. General layout and profile of a high-head diversion canal development

The main parts of a high-head diversion canal type plant are:

1) the weir,
2) the canal intake,
3) the head race,
4) the headpond with spillway and gate or valve chamber,
5) the penstock,
6) the powerhouse,
7) the tailrace.

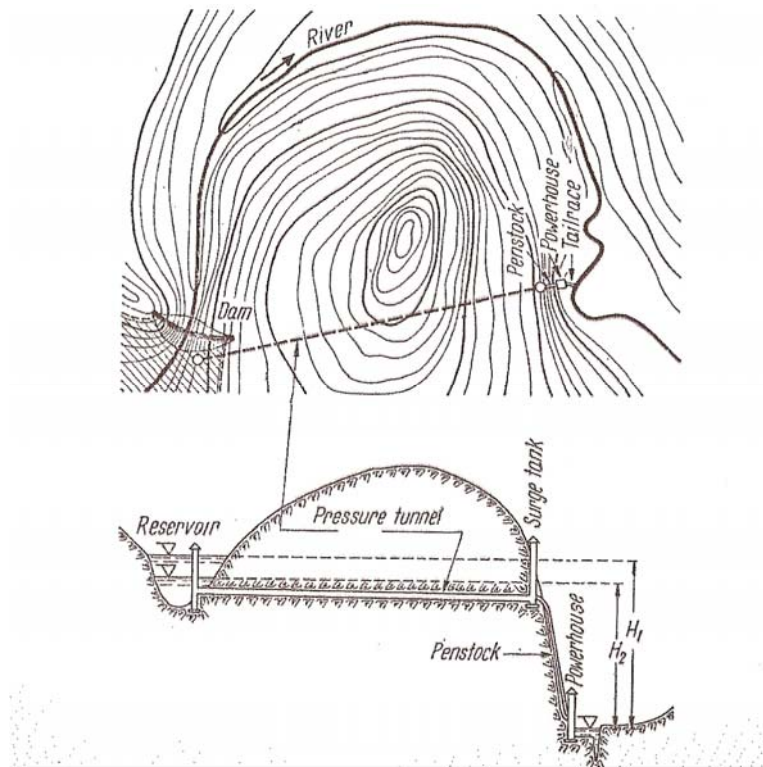*Prof. Dr. Atıl BULU*

B) Plants Fed by a Pressure Tunnel

Figure. General layout and profile of a pressure tunnel development

1) the dam (sometimes only weir),
2) the intake or headworks,
3) the pressure tunnel,
4) the surge tank,
5) the penstock,
6) the power house,
7) the tailrace.

The tunnel intake should be located close to the bottom of the reservoir to ensure the greatest effective storage volume. Under certain topograhic and geologic conditions, the conveyance of water through a tunnel under the dividing range may, even in case of low dams, be preferable to building a long, meandering power canal. Since here the tunnel is not necessarily a pressure conduit, free-surface flow conditions may prevail therein. Such arrangements should be regarded as *diversion canal type developments*.

C) Plants with Concentrated Fall

Developments, where the powerhouse is located close to, or within, a high dam or high-head river barrage, constitute the third main type of high-head installations. This arrangement, which could be termed *plant with concentrated fall*, or valley dam station, is essentially similar to that of low-head run-of-river plants. The head for the power station approaches the height of the dam.
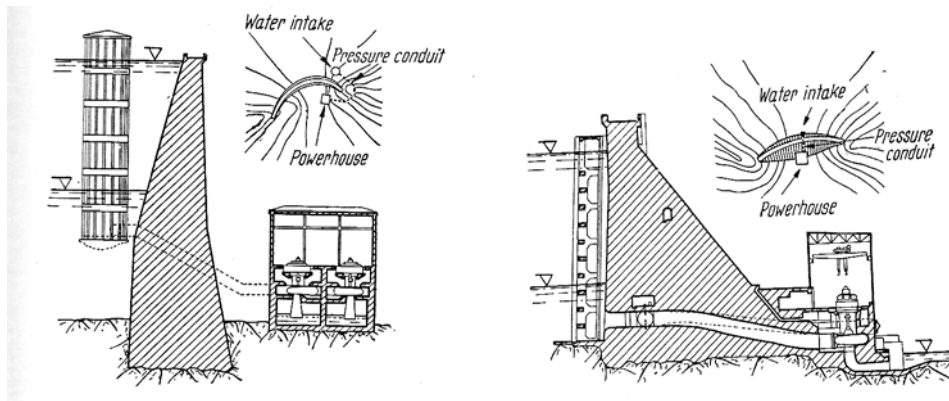
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Figure. Development with the powerhouse located at the toe of the dam
(plant with concentrated fall)

The three main parts of this type of power plants:

1) the water intake (generally built on the upstream face of the dam),
2) the pressure conduit (generally transversing the dam body, sometimes bypassing the dam adjacent rock),
3) the powerhouse.

The output of diversion canal developments is closely governed by the discharge available in the river, while the small storage capacity created by the low weir is sufficient to meet daily fluctuations in load only. This type may be called as *high-head run-of-river plant*. The other two types may be referred to as *reservoir plants*. Pressure tunnel developments are valuable for those fed from a large reservoir under high head.

Free Surface Intakes

Settling basins and sand traps are very important for high-head water power plants and they should designed on the basis of hydraulic computations. Suspended load, especially sharp edged fine sand transported by mountain streams causes rapid wear of the penstock and steel parts of the turbines. Water flowing at high velocity and carrying heavy sediment load attacks the lining of power canal and power channels.

Scratch effects become generally more pronounced with increasing head, therefore, in case of heads higher than 100 m, sand should be carefully settled out and with heads higher than 200 m even the greatest part of silt fraction should be retained.

The main parts of an intake are:

1) the inlet section including the sill and coarse rack,
2) the inlet gate and transition section,
3) the settling basin and sand flushing canal.
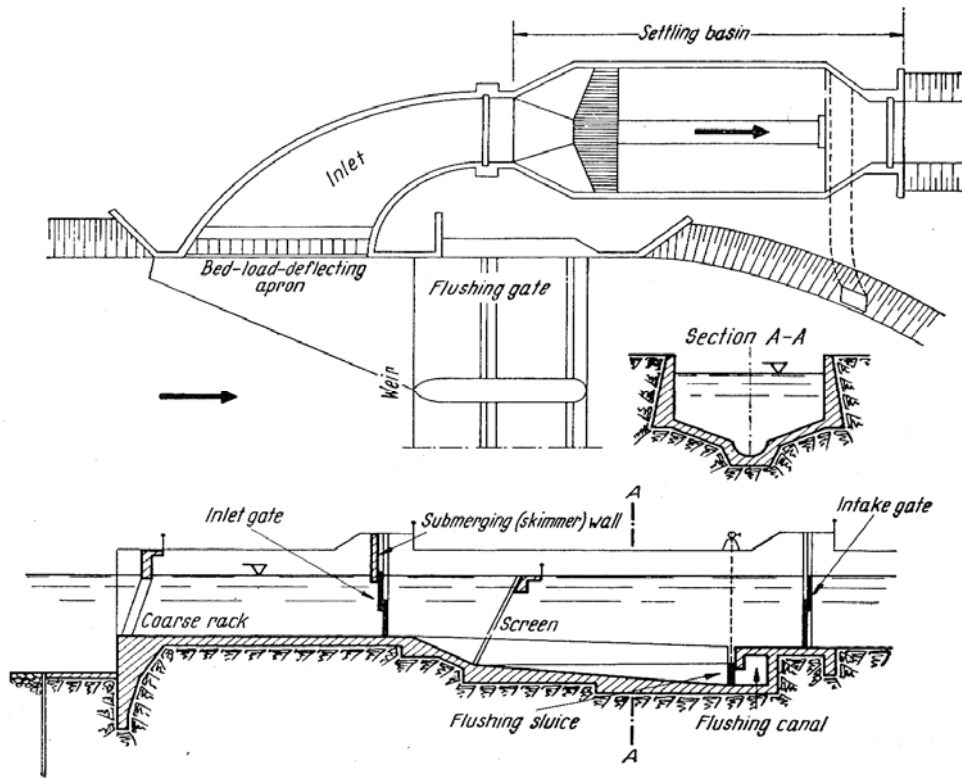
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Figure. General arrangement of intake

Before the inlet section of the intake a bed load deflecting apron should be applied permitting a periodical flushing of bed load hold by the sill. The apron extends to the flushing gate of the weir.

Protection against Silting

In preventing entrance of bed load, or rather in promoting a desilting effect at the inlet section, in protecting both inlet and canal against sedimentation, the proper choice of intake site is of vital importance.
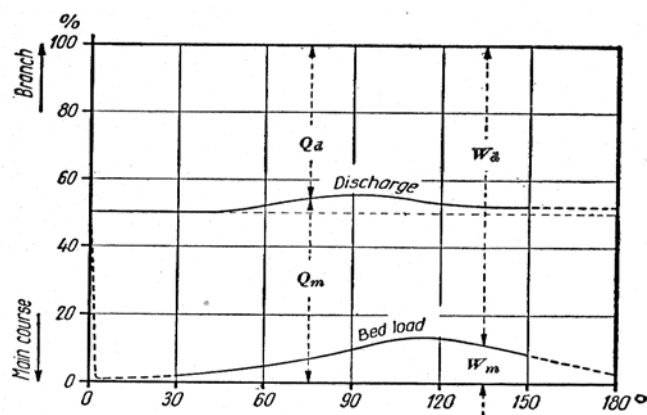

Fig. 1/21. The amount of bed load entering the canal plotted *vs.* the angle of diversion, with approximately half of the flow diverted from a straight river stretch. (Experiments of *H. Bulle.*)

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Changes in the angle of diversion (the angle between the outside wall of the intake structure and the direction of main flow) hardly affect the quantity of sediment entering the canal. More than 90% of transported matter enters the diversion canal branching off at an angle of $30^0 - 90^0$ from the main course, yet carrying only half of the main discharge. At the same time it can be seen that the angle of diversion has no significant effect upon silting conditions.
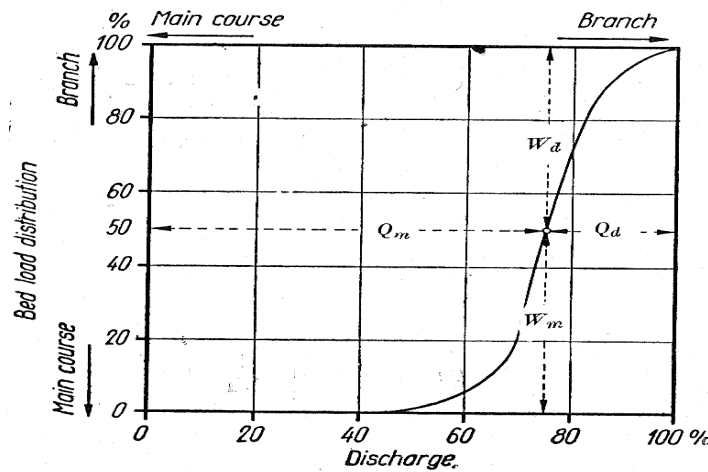

Fig. 2/21. Variation of the amount of entering silt with discharge in case of diversion under acute angle from straight river stretch (Experiments of *H. Bulle*)

The curve shown above illustrates the distribution of discharges and silt quantities for a 30^0 angle of diversion. A balanced (50 – 50 per cent) distribution of silt is attained only if some 25% of the original discharge is allowed to enter the canal. If the discharge carried by the diversion canal exceeds 60%, the entire sediment load enters to the branch canal.

Hydrodynamical considerations will yield a very simple explanation for the above phenomenon, that in case of diversion from a straight stretch, the flow entering the power canal carries extremely great quantities of bed load. The velocity component opposite the inlet section and vertical to it, denoted by v, is due to the transverse head loss Δh, the latter given by the equation,

$$v = \mu \sqrt{2g\Delta h}$$

In one vertical along cross-section (x-x) velocities v due to Δh will be almost uniform. The velocity of direction x-x varies only in direction x within one section and shows an increase from the opposite bank towards the side intake. Since the velocity distribution within the main course above the point of diversion involves a velocity considerably smaller at point B than at the surface, it follows that the deeper the examined subsurface point is situated, the greater the angle formed by the resultant of velocities V and v and by the axis of main flow. Consequently, more water is drawn from deeper layers into the canal than from those nearer to the surface. Thus a lamination according to depth arises in the main flow causing the water entering the power canal to be drawn, for the greater part, from the lower layers that are heavily silted, while water in the upper layers containing considerably less silt overfalls the diversion dam and streams forth in the main course.
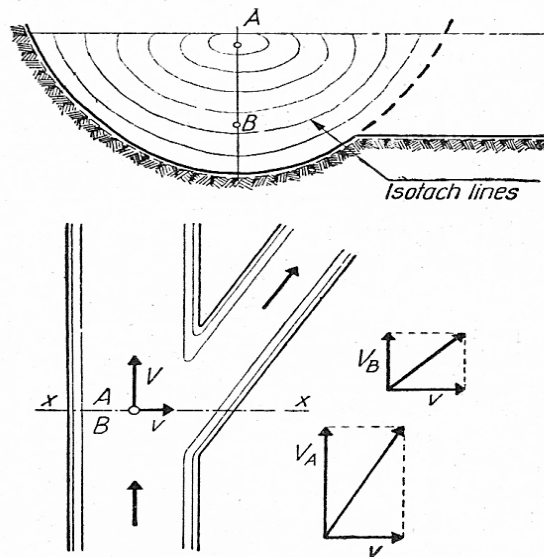
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Figure. Velocity distribution in the
river at the intake

The whole bed load is practically is carried into the power canal for big discharges, and the joint application of high sill, silt-sliding apron and sluiceway gives no substantial relief to the problem.

Let us examine the bed load conditions of intakes located in a bend.
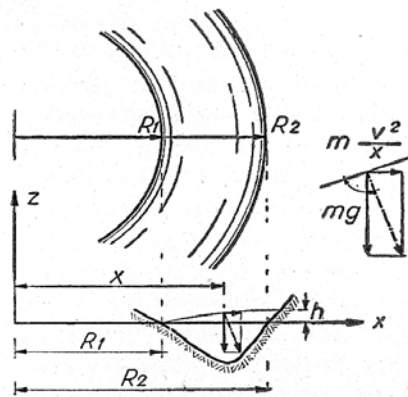

Figure. Profile of the water
surface in bends

Considering that the water surface as a potential surface normal to the resultant of acting forces, the equation for the sloping water level in the curve may be developed as follows:

$$\frac{dz}{dx} = \frac{m\dfrac{v^2}{x}}{mg}$$

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$$gdz = v^2 \frac{dx}{x}$$

Solving the above differential equation,

$$gz = v^2 Lnx + C$$

Consequently, at point x = R_1, z = 0. Thus,

$$C = -v^2 LnR_1$$

$$z = \frac{v^2}{g} Ln \frac{x}{R_1}$$

The maximum rise of the surface occurring at the concave bend,

$$h = \frac{v^2}{g} Ln \frac{R_2}{R_1} = \frac{v^2}{2g} \times 0.43 \times \log \frac{R_2}{R_1}$$

Velocity of flows shows a tendency to decrease along the same vertical towards the bottom. Particles of water moving in beds at and near the surface are thus subject to greater centrifugal force than those traveling near the bottom. Consequently, particles at and near the surface are forced towards the concave bank. An equal quantity of water is bound to follow at the bottom in the opposite direction towards the convex band due to the principle of continuity.

Particles of water submerging with great velocity cause erosion of the bottom along the concave bank. Part and occasionally the whole of eroded matter is then deposited by the flow slowing down towards the convex bank. So the original rectangular cross-section takes and asymmetrical shown as shown in the Figure.
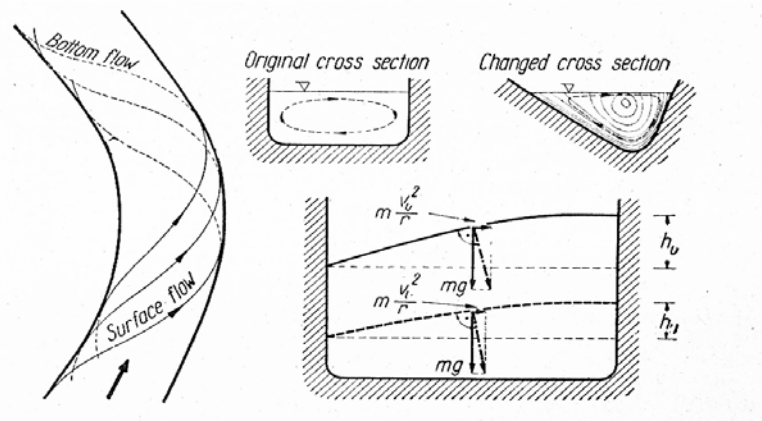

Figure. Development of spiral flow in bends

Silt of different particle size reaching the bend separates according to size at the peak of the curve. Fine silt settles fairly high on the sand bank formed under water along the convex bank, while coarser grains are carried forth and deposit mostly in or around the holes along the concave bank. It is advisable to have the power canal branch from the concave side in as much as relatively desilted water is required.
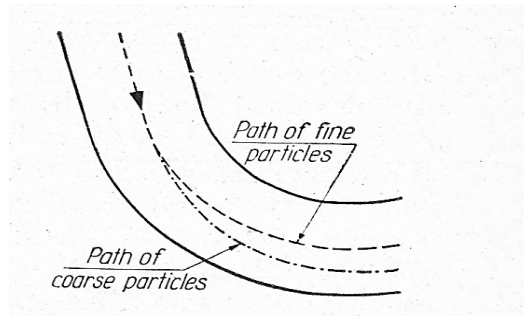
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Figure. Separation of bed load according
to particle size in bends

The intake structure is to be built at a point where the spiral flows is strong and the weir is to be located so that the sluiceway or the lateral opening of the movable gate system also falls within the sphere of spiral flow.
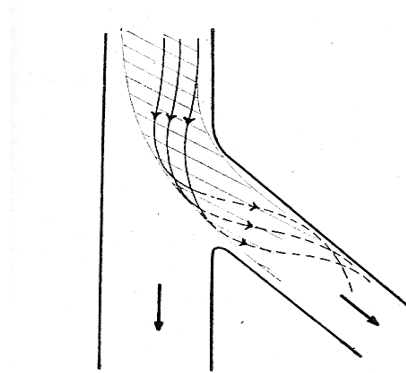

Figure. Spiral flow at
the intake

Water discharging into branch does not diverge in a sharp angle but follows a curved route; spiral flow will develop at the upstream end of the canal too, should the diversion be placed either in a straight or in a curved stretch of the water course. Surface flows tend towards the concave side of the curved streamway caused by the diversion, while bottom flow transporting debris is directed towards the canal.

With a 50-50 percent ratio of discharges, distribution of bed load is as follows and given in the Figure.

1. Bed load: Canal 100%, main water course 0%,
2. Bed load: Canal 50%, main water course 50%,
3. Bed load: Canal 5%, main water course 95%,
4. Bed load: Canal 100%, main water course 0%,
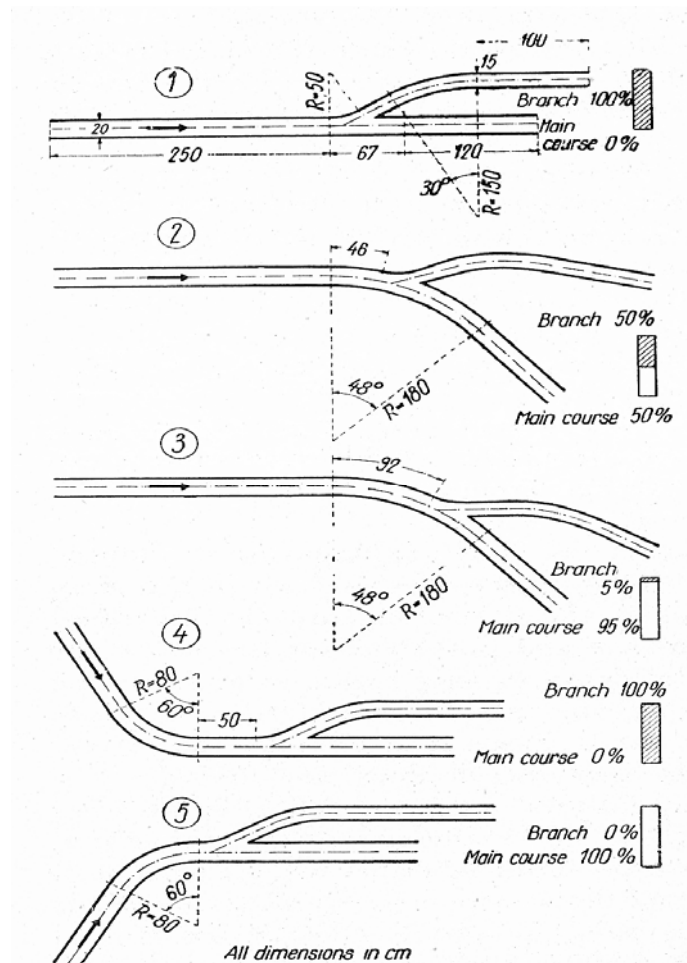5. Bed load: Canal 0%, main water course 100%.

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Figure. Typical shapes of diversion

No. 5 proved most unfavorable as here silt transportation into canal is intensified by synchronizing spiral flows in both original water course and branch canal. No.1 is also highly unfavorable. No. 2, 3, and 5 may be regarded as favorable. No. 2, diversion is at the upstream end of the curve where spiral flow is not yet fully developed and so the effect of flow in the original water course is largely decreased by spiral flow in the branch canal. No 3. and 5 are the most favorable, as here fully developed spiral flow at the downstream end of the bend cannot be considerably lessened by spiral water movements of opposite direction of the branch.

The following basic principles governing selection of the intake site can be suggested:

1. Intakes should be located, whenever possible, on the concave side of a curved stretch,
2. Efficiency of the intake in preventing sedimentation increases with the sharpness of the bend,
3. The amount of bed load transported into the canal decreases, as the ratio of the total discharge to the amount increases.
4. Intakes are most favorably located along the downstream reach of the curve, near the end.
5. The lower the head, the more effective the intake.

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6. Conditions in a straight stretch are opposite to those described under No. 3; with diverted flow being constant, any increase in the river discharge will involve more extensive sedimentation in the canal.

7. The silt releasing sluice of the diversion weir, the canal sill and the desilting sluice can only be operated at good efficiency if more or less favorable bend conditions are created through proper design and arrangement in keeping with the above principles.

8. With intakes from straight stretches, but more so along the convex side, the afore-mentioned measures offer no significant contribution to the protection against sedimentation in the canal. In such cases both canal sill and desilting canal give satisfactory results if heads are considerable even during high-head periods.

9. Intakes from straight stretches can be made more favorable by forcing water to follow a curved route with the convex side of stream curve facing the intake structure.
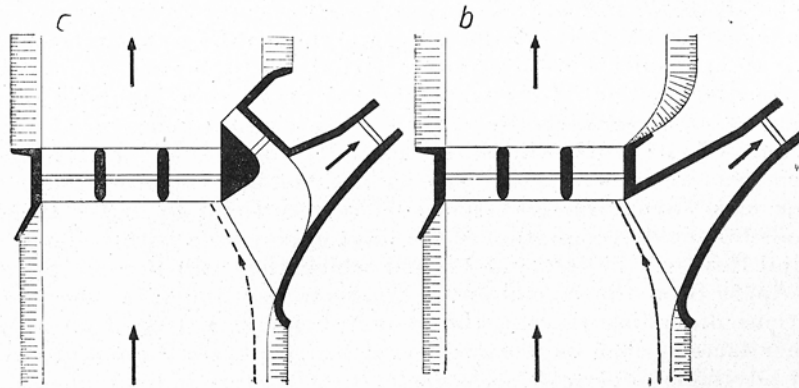

Figure. Intakes from straight stretches with bed contraction, a) with flushing (desilting) canal, b) without flushing canal

This can be achieved by arrangements illustrated in the above Figure, where an inlet section extending crosswise into rive bed, and a weir shorter than the width of flow above the intake make the flow to follow a curved route.

10. The quantity of bed load can be reduced by a longitudinal baffle wall as shown in the below Figure, if $Q_d < Q_B$ where Q_d is the diverted discharge and Q_B the discharge conveyed in the river in width B of the intake. As a result of inequality $Q_d < Q_B$ part of the water is compelled to deviate on a curved path from the bank, thereby bringing about a silt-diverting spiral flow.
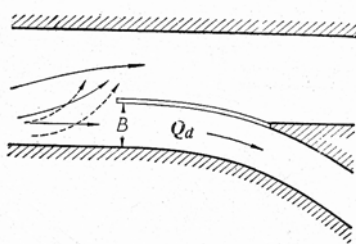

Fig. 11/21. Reduction of entering bed load by means of a baffle wall extending into the main course. (After *M. M. Grishin*)
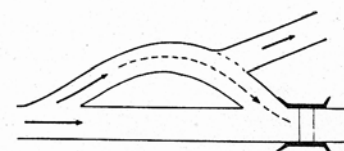

Fig.12/21. Diversion from straight river stretch with curved lateral branch.(Suggestion of *Habermaas*)

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Protective measures against bed load may be completed by a few remarks.

   a) The minimum discharge Q_0 capable of inducing bed load movement will be decisive of the choice of the site and the arrangement of the intake. The ratio of the plant discharge capacity Q_p and of the above limit discharge Q_0 is the one of the factors governing the design of intake.
   b) The individual features of an intake may require the discharge diverted into canal, in certain periods, less than available in the river for power generation.
   c) Quantitative relations may be established as to the reduction of the discharge diverted. The degree of reduction depends upon the character of the river section.
   d) Special care should be taken if a power plant having a relatively great discharge capacity is projected on a mountain river. The necessary reduction of the discharge may in this case permit the diversion of a volume corresponding to the maximum plant discharge capacity at times of flood only.
   e) On mountainous rivers carrying a heavy sediment load, the intake works should always be located on the concave side of the bend even if this side is otherwise less favorable.
   f) Flow velocity in the inlet section of the intake should preferably be 0.75 m/sec on the average as indicated by experiments conducted between velocity limits of o.50 and 1.10 m/sec.

Care should be taken during the hydraulic designing of the settling basin to ensure the calculated velocity in the structure would range 0.40 to 0.60 m/sec.

The hydraulic design in settling basins is broadly outlined in the following paragraph.

   1) Exploration of sediment conditions, involving the quantitative and qualitative analysis of sediment carried by the river. In mountain rivers or in steep, upper river sections at the average sediment concentration varies from 2 to 10 kg/m^3.
   2) Following the investigations of sediment conditions, the necessary degree of load removal, should be determined. Attempts have been made to approximate operating requirements by specifying the diameter of the smallest particles to be settled out (limit particle size).

   At medium-head plants, the removal of particles larger than 0.2 – 0.5 mm is usually specified. Instead of using the limit particle size, the degree of removal is frequently defined by the removal ratio of concentrations after and before settling expressed in percentages. If the concentration of raw water is C, and that of the clarified water is specified as the permissible value C_p, the required removal ratios is obtained as,

$$100\frac{C_p}{C}$$

   By specifying or assuming the limit particle size, the removal ratio may easily be calculated.

   3) Having determined the basic data as suggested, design can be done. First the settling velocity of the smallest fraction; i.e., of the limit particle size to be removed should be calculated theoretically or be established by tests.

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The so-called horizontal-flow settling system is usually applied at power developments. For this system, the dimensions of the settling basin may in principal be determined by two computation methods.

The effect of turbulent flow upon settling velocity is neglected in the simple settling theory. Three basic relations may be written for the determination of the required basin length. Denoting the depth of the basin by h and its width by b, the discharge passing through basin is,

$$Q = bhV \quad (\text{m}^3/\text{sec})$$

Where V is the flow velocity. The second equation expressing the relation between the settling velocity w, the depth of the basin and the settling time t,

$$t = \frac{h}{w} \quad (\text{sec})$$

Finally, the length of the basin will be governed by the consideration saying that water particles entering the basin and sediment particles conveyed by them with equal horizontal velocity should only reach the end of the basin after a period longer than the settling time. Thus even the smallest settling particle may strike the bottom of the basin within the settling zone. The retention period should not be shorter than the settling time. The required length of the basin is,

$$l = Vt \quad (\text{m})$$

Eliminating t from the last two relations will be established between the six values governed the hydraulic design:

$$Q = bhV$$
$$lw = hV$$

A solution of the problem is not possible unless four quantities are known. The discharge Q can always be considered given, the settling velocity w is defined by the initially specified degree of removal and can be established by calculation. The highest permissible flow velocity should also be specified in order to prevent particles once settled from picked up again. The actual flow velocity should not exit this limit, whereas excessive dimensions computed by substantially lower velocities would again result in uneconomical design. Velocities higher than the permissible velocity tend to scour the material settling to the bottom, which may even become suspended again. This limit velocity may in fact be considered equal to the theoretical suspending velocity, or to the critical velocity encountered in the theory of sediment transport.

The critical velocity,

$$V = a\sqrt{d} \quad (\text{cm/sec})$$

Where d is the diameter of particles in mm and the constant a:

<center>a = 36, for d > 1 mm,
a = 44, for 1 mm > d > 0.1 mm
a = 51 for 0.1 mm> d</center>

The fourth dimension that can be assumed in advance is one of the main dimensions of the basin. The depth of the horizontal flow settling basins employed in water power projects is generally between 1.5 and 4.0 m with velocities not higher than from 0.4 to 0.6 m/sec.

The water mass conveyed during settling time should equal the capacity of the settling basin. Owing to the retarding effect of turbulent flow on sinking particles, settling is slower in flowing water. By using a lower settling velocity (w − w'), the reduction in settling velocity w' to be closely related to the flow velocity,
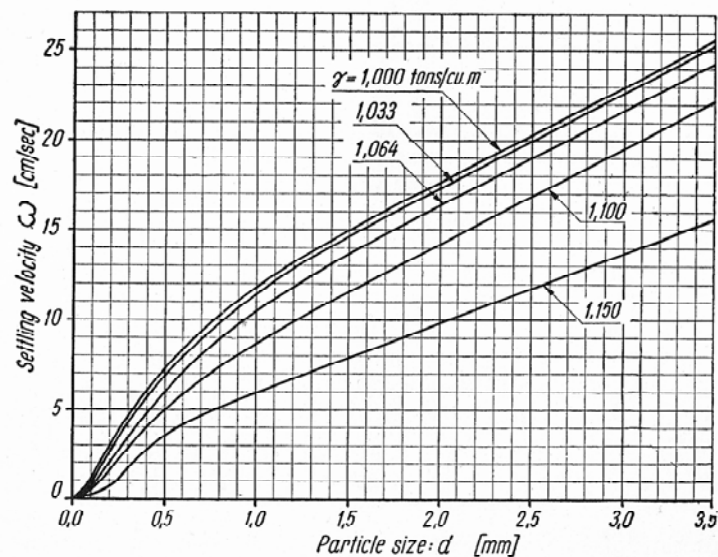
$$w' = aV \quad \text{(m/sec)}$$

The coefficient a may be computed from the relation,

$$a = \frac{0.132}{\sqrt{h}}$$

Where h is the water depth in m. The settling length is therewith,

$$l = \frac{hV}{w - aV} = \frac{h^{3/2}V}{h^{1/2} - 0.132V} \quad \text{(m)}$$

A negative denominator is an indication of the fact that no settling can be attained under the assume conditions. The computation should be repeated using the modified dimensions.

10/78. Settling velocity in stagnant water plotted against the density of silty water and the particle diameter. (After *L. Sudry*)

<center>13</center>

The most important factors affecting the design of the settling basin are the quality of sediment (specific weight and shape of particles) density of water carrying sediment and water temperature. All estimates involving the direct application of the afore-mentioned data should be regarded as approximate only.

The necessary settling length for turbulent flow is computed from the settling velocity in stagnant water w and from the flow velocity. The settling length,

$$l = \frac{\lambda^2 V^2 \left(\sqrt{h} - 0.2\right)^2}{7.51 w^2} \quad \text{(m)}$$

Where λ depends on the removal ratio defined previously. Values of λ defined by the function,

$$\lambda = f(w)$$

W denotes the ratio of settled sediment to the total load entering with the flow and can be computed from the afore-mentioned removal ratio as follow:
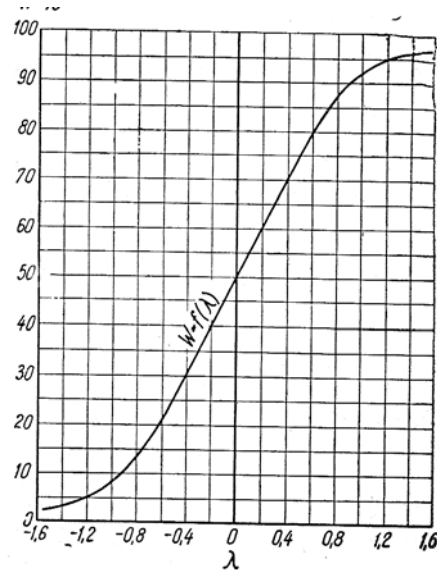
$$w = 100 - 100 \frac{C_P}{C}$$

Fig. 11/78. *Velikanov's* relationship
w = f (λ) for designing settling basins

The settling velocity pertaining to the limit particle size of the fraction to be settled out without assuming 100% removal. Satisfactory values can be obtained by using coefficients pertaining to a 95 to 98% removal of the limit particle size.

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Continuous operation can be ensured by one of the following arrangements:

a) Series of basins, some of which can be flushed while others operating,
b) Permanent operation of basins can also be realized by continuously flushing settled sediment. An inflow exceeding the water demand by 10% should be admitted into the basin and sediment accumulating at the bottom can be flushed continuously by discharging the excess water to waste.

Example: Design a settling basin for a high-head power plant by using the settling theory. The basin should serve to remove particles greater than 0.5 mm diameter from the water conveying mainly sand. The design discharge is 5 m³/sec and assume an initial value of 3.20 m for the depth of the basin.

Solution: Determine first the permissible velocity flow velocity. Owing to economical considerations this should equal the critical velocity for which,

$$V = 44\sqrt{d} = 44\sqrt{0.5} = 31.2\,cm/\sec$$

In designing the basin, V = 30 cm/sec flow velocity will be used. The following step is to determine the settling velocity according to the limit particle size of 0.5mm to be removed. From the settling velocity – particle size Figure, w = 6 cm/sec (for γ = 1.064). The required length of the basin is,

$$l = h\frac{V}{w} = 3.20 \times \frac{30}{6} = 16m$$

And the width,

$$b = \frac{Q}{hV} = \frac{5}{3.2 \times 0.3} = 5.21m$$

Checking: The settling time is,

$$t = \frac{h}{w} = \frac{3.20}{0.06} = 53.4\,\sec$$

The discharge conveyed during this period is,

$$V = Qt = 5 \times 53.4 = 267m^3$$

Should be equal to the capacity of the basin;

$$V = hbl = 3.20 \times 5.21 \times 16.0 = 267m^3$$

Determine the length of the basin using identical basic values by the method of Velikanov's Figure for a removal ratio of 97% (W=0.97).

The Figure yields λ = 1.50 for W = 0.97. The length of the basin is,

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$$l = \frac{\lambda^2 V^2 \left(\sqrt{h} - 0.2\right)^2}{7.51 w^2}$$

$$l = \frac{1.5^2 \times 0.3^2 \times \left(\sqrt{3.2} - 0.2\right)^2}{7.51 \times 0.06^2} \cong 19m$$

Example: Compute for the conditions of the preceding example the settling length by considering the retarding effect of turbulent.

Solution: The coefficient governing the reduction of settling velocity is,

$$a = \frac{0.132}{\sqrt{h}} = \frac{0.132}{\sqrt{3.20}} = 0.0737$$

And thus the velocity decrement,

$$w' = aV = 0.0737 \times 0.30 = 0.0221 m/\sec$$

The settling length,

$$l = \frac{hV}{w - w'} = \frac{3.20 \times 0.30}{0.060 - 0.0221} = 25.30m$$

The unchanged width of the basin is,

$$b = \frac{Q}{hV} = \frac{5.0}{3.20 \times 0.30} = 5.21m$$

And its capacity,

$$V = hbl = 3.20 \times 5.21 \times 25.30 = 422m^3$$

Example: Compute the modified dimensions for a reduced depth of 2.40 m.

Solution:

$$a = \frac{0.132}{\sqrt{h}} = \frac{0.132}{\sqrt{2.4}} = 0.0851$$

$$l = \frac{hV}{w - aV} = \frac{2.40 \times 0.30}{0.06 - 0.0851 \times 0.30} = 20.90m$$

Width of the basin is,

$$b = \frac{Q}{hV} = \frac{5}{2.40 \times 0.30} = 6.95m$$

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And the reduced capacity,

$$V = 2.40 \times 6.95 \times 20.9 = 348 m^3$$

Example: A power plant is fed by a river carrying very coarse suspended sediment load. As indicated by the gradation curve obtained for the sediment, 70% are held on the 1 mm screen. In order to protect the turbines the entire over 1 mm diameter should be settled.

Solution:

$$100 \frac{C_p}{C} = 30\%$$

The basin will be designed for a discharge of 12 m³/sec with the retarding effect of turbulent and a depth of 2.80 m will be taken. The critical velocity is,

$$V = 44\sqrt{d} = 44 \times \sqrt{1} = 44 \, cm/\sec$$

The settling velocity in stagnant water is obtained from the Figure (for γ = 1.064) W = 10 cm/sec. The settling velocity decrement due to the turbulent,

$$W' = \frac{0.132}{\sqrt{h}} V = \frac{0.132}{\sqrt{2.8}} \times 0.44 = 0.0346 \, m/\sec$$

The actual settling velocity is,

$$W - W' = 0.100 - 0.0346 = 0.0654 \, m/\sec$$

And the settling length,

$$l = \frac{hV}{W - W'} = \frac{2.80 \times 0.44}{0.0654} = 18.85 m$$

The required width of the basin is,

$$b = \frac{Q}{hV} = \frac{12}{2.80 \times 0.44} = 9.74 m$$

A settling basin 20 m long and 10 m wide will have a capacity of,

$$V = 10 \times 20 \times 2.8 = 560 m^3$$

Compute the length of the basin also by the equation of Velikanov (W = 0.97), λ = 1.50,

$$l = \frac{1.50^2 \times 0.44^2 \times \left(\sqrt{2.8} - 0.2\right)^2}{7.51 \times 0.10^2} = 12.50 m$$

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