Hydraulic design of small hydro plants

Profession

Version 2 STANDARDS/MANUALS/ GUIDELINES FOR SMALL HYDRO DEVELOPMENT Civil Works – Hydraulic Design Of Small Hydro Plants Lead Organization: Sponsor: Alternate Hydro Energy Center Indian Institute ofTechnologyRoorkee Ministry of New and Renewable Energy Govt. of India May 2011 AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 1 1. GUIDELINES FOR HYDRAULIC DESIGN OF SMALL HYDRO PLANTS This section provides standards and guidelines on the design of the water conductor system.

This system includes; head works and intake, feeder canal, desilter (if required), power canal or alternative conveyance structures (culverts, pipelines, tunnels, etc), forebay tank, penstock and surge tank (if required) up to the entry of the turbine, tailrace canal below the turbine and related ancillary works. 1. 1 HYDRAULIC DESIGN OF HEAD WORKS In general head works are composed of three structural components, diversion dam, intake and bed load sluice. The functions of the head works are: Diversion of the required project flow from the river into the water conductor system.

Control of sediment. Flood handling. Typically a head pond reservoir is formed upstream of the head works. This reservoir may be used to provide daily pondage in support of peaking operation or to provide the control volume necessary for turbine operation in the water level control mode. This latter case would apply where the penstock draws its water directly from the head pond. Sufficient volume must be provided to support these functions. There are three types of head works that are widely used on mini and small hydro projects, as below: Lateral intake head works Trench intake head works

Reservoir / canal intakes Each type will be discussed in turn. 1. 1. 1 Head Works with Lateral Intakes (Small Hydro) Head works with lateral intakes are typically applied on rivers transporting significant amounts of sediment as bed load and in suspension. The functional objectives are: To divert bed-load away from the intake and flush downstream of the dam (the bed load flushing system should be operable in both continuous and intermittent modes). To decant relatively clean surface water into the intake. To arrest floating debris at intake trashracks for removal by manual raking.

To safely discharge the design flood without causing unacceptable upstream flooding. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 2 The following site features promote favourable hydraulic conditions and should be considered during site selection: The intake should be located on the outside of a river bend (towards the end of the bend) to benefit from the spiral current in the river that moves clean surface water towards the intake and bed load away from the intake towards the centre of the river.

The intake should be located at the head of a steeper section of the river. This will promote removal of material flushed through the dam which may otherwise accumulate downstream of the flushing channel and impair its function. Satisfactory foundation conditions. Ideal site conditions are rare, thus design will require compromises between hydraulic requirements and constraints of site geology, accessibility etc. The following guidelines assume head works are located on a straight reach of a river. For important projects or unusual sites hydraulic model studies are recommended.

A step by step design approach is recommended and design parameters are suggested for guidance in design and layout studies. Typical layouts are shown in Figures 2. 2. 1 to 2. 2. 3. 1. 1. 2 Data Required for design. The following data are required for design: Site hydrology report as stipulated in Section 1. 3 of this Standard giving: - Qp (plant flow) - Q100 (design flood flow, small hydro) - Q10 (design flood flow, mini hydro) (data on suspended sediment loads) - Cw - H-Q Curves (W. L. rating curves at diversion dam) Topographic mapping of the site including river bathymetry covering all head works structure sites.

Site geology report. 1. 1. 3 Site Selection: Selection of the head works site is a practical decision which involves weighing of several factors including hydraulic desiderata (Section 2. 2. 1/1. 0), head optimization, foundation conditions, accessibility and constructability factors. Given the importance of intake design to the overall performance of the plant it is recommended that an experienced hydraulic engineer be consulted during studies on head works layout. 1. 1. 4 Determination of Key Elevations: AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 3

For the illustrative example: Qp = 10. 0 m3/s Determine V0 = 0. 5 Q0. 2 (= 0. 792, say 0. 80 m/s) (= 12. 5 m2) A0 = Q ? V0 A0 H= (= 1. 77 m, say 1. 80 m) 4 Assume L = 4H (= 7. 08 m, say 7. 0 m) ye = greater of 0. 5 yo or 1. 5 m (= 1. 80m) yd = L. S (= 0. 28 m) NOL = Z0 + ye + yd + H NOL = 97. 5 + 1. 80 + 0. 28 + 1. 80 (= 101. 38m, say 101. 50 m) Sill = NOL – H (= 99. 7m) Crest of weir or head pond NOL = 101. 5 m Height of weir = 4. 0 m These initial key elevations are preliminary and may have to be adjusted later as the design evolves. 1. 1. 5 Head Works Layout

The entry to the intake should be aligned with the river bank to provide smooth approach conditions and minimize the occurrence of undesirable swirl. A guide wall acting as a transition between the river bank and the structure will usually be required. Intake hydraulics are enhanced if the intake face is slightly tilted into the flow. The orientation of the intake face depends on river bank topography, for straight river reaches the recommended values for tilt vary from 10o to 30o depending on the author. When this angle becomes too large the intake will attract excessive amounts of sediment and floating debris.

It is recommended that the sill level of the intake is kept sufficiently higher than the sill level of the under sluice. The under sluice should be located adjacent to the intake structure. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 4 For development of the head work plan, it is recommended that the following parameters be used for layout: Axis of intake should between 100° to 105° to axis of diversion structure The actual inclination may be finalized on the basis of model studies. Divide wall, if provided, should cover 80% to 100% of the intake.

Assume flushing flow equal to twice project flow then estimate the width and height of the flushing gate from orifice formula,: Example should be in appendix. Qf = 0. 6 ? 0. 5W2 Where: Qf = flushing flow W = gate width H = gate height (= 0. 5W) Yo = normal flow depth as shown in 2. 2. 1. 1/2. 0 Sill should be straight and perpendicular to the flow direction. In the sample design (Fig. 2. 2. 1. 1) the axis of the intake = 105° & Qf = 2. 0? 10. 0 = 20m3/s ? 20. 0 = 0. 6 ? 0. 5 W2 ? W = 2. 8 m (say 3. 0m) and H = 1. 5 m. 1. 1. 6 Flood Handling, MFL and Number of Gates.

For small hydro a simple overflow diversion weir would be the preferred option if flood surcharge would not cause unacceptable upstream flooding. For purpose of illustration, the following design data are assumed (see Figure 2. 2. 2): Design flood, Q100 = 175 m3/s A review of reservoir topography indicated that over bank flooding would occur if the flood water level exceeded 103. 0 m. Select this water level as the MFL. This provides a flood surcharge (S) of 1. 20 m. Assume weir coefficients as below: Gate, Cw = 1. 70 - - - sill on slab at river bottom. Weir, Cw = 1. 0 - - - - - - -ogee profile. Assume gate W/H ratio = 1: 2 H = 4. 0 m ? W = 4. 8 (say 5. 0 m) MFL. = NOL + 1. 50 (= 103. 0m) Qgate = Cw. W. (MFL - ZS)1.. 5 Qweir = Cw. Lw. S1. 5 Capacity check for MFL = 103. 0 m No. of Length of Overflow QG Gates Section (m) (m3/s) 0 35. 0 0. 0 1 29. 0 109. 6 QW (m3/s) 82. 8 68. 6 QT (m3/s) 82. 8 178. 2 > 175 AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 5 Therefore one gate is sufficient. Where: MFL = Maximum flood level (m) NOL = Normal operating level (m) S = flood surcharge above NOL (m)

W = width of gate (m) H = height of gate (m) ZS = elevation of gate sill (m) = weir coefficient (m0. 5s-1) Cw QG, QW, QT = gate, weir and total flows The flow capacity of the sediment flushing gate may also be included in calculating flood handling capacity. 1. 1. 7 Diversion structure and Spillway Plains Rivers: Stability of structures founded on alluvial foundations typical of plains rivers, is governed by the magnitude of the exit gradient. The critical gradient is approximately 1. 0 and shall be reduced by the following safety factors: Types of foundation

Shingles / cobbles Coarse sand Fine sand Safety factor 5 6 7 Allowable Exit Gradient 0. 20 0. 167 0. 143 Also diversion structures on plains rivers will normally require stilling basins to dissipate the energy from the fall across the diversion structure before the water can be returned safely to the river. Design of diversion weirs and barrages on permeable foundation should follow IS 6966 (Part 1). Sample calculations in Chapter 12 of “ Fundamentals of Irrigation Engineering” (Bharat Singh, 1983) explain determination of uplift pressure distributions and exit gradients.

Further details on structural aspects of design are given in Section 2. 3. 3 of this Standard. Mountain Rivers: Bedrock is usually found at relatively shallow depths in mountain rivers permitting head works structures to be founded on rock. Also the beds of mountain rivers are often boulder paved and are much more resistant to erosion than plains rivers. Therefore there may be no need for a stilling basin. The engineer may consider impact blocks on the downstream apron or simply provide an angled lip at the downstream end of the apron to “ flip” the flow away from the downstream end of the apron.

A cut-off wall to bed rock of suitable depth should AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 6 also be provided for added protection against undermining by scour. The head works structures would be designed as gravity structures with enough mass to resist flotation. For low structures height less than 2. 0 m anchors into sound bedrock may be used as the prime stabilization element in dam design. Stability andstressdesign shall be in accordance with requirements of Section 2. 3. 3 of this Standard. 1. 1. 8 Sediment Flushing Channel To be reviewed

The following approach is recommended for design of the flushing channel: Select flushing channel flow capacity (Qf) = 2? Qp Estimate maximum size of sediment entering the pocket from site data or from transport capacity of approaching flow and velocity. In case of diversion weir without gates assume sediment accumulation to be level with the weir crest. (Assume continuous flushing with 3? Qp entering the pocket, for this calculation). Establish entrance sill elevation and channel slope assuming an intermittent flushing mode (intake closed) with Qs = 2Qp, critical flow at the sill, supercritical flow downstream (FN ? 1. 0) and a reservoir operating level 0. 5m below NOL. Determine slope of channel to provide the required scouring velocity, using the following formula which incorporates a safety factor of 1. 5: i = 1. 50 io d 9/7 i0 = 0. 44 6 / 7 q Where: io = critical scouring velocity d = sediment size q = flow per unit width (m3/s per m) Verify that flow through pocket in continuous flushing mode (Qs = 3Qs) will be sub critical, if not lower entrance sill elevation further. Determine height of gate and gate opening based on depth of flow at gate location and corresponding gate width. Increase the above theoretical gate height by 0. 5 m to ensure unrestricted open channel flow through the gate for intermittent flushing mode and a flushing flow of 2 Qp. For initial design a width to height ratio of 2: 1 for the flushing gate is suggested. 1. 1. 9 Intake/Head Regulator: In intake provides a transition between the river and the feeder canal. The main design objectives are to exclude bed-load and floating debris and to minimize head losses. The following parameters are recommended:

Approach velocity at intake entrance (on gross area) 0. 20 Ve = 0. 5 Q p m / s For trashracks that are manually cleaned, V should not exceed 1. 0 m/s.

AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 7 Convergence of side walls 2. 5: 1 with rate of increase in velocity not exceeding 0. 5 m/s per linear m. • Height of sill above floor of flushing channel (ye) = greater of 1. 5m or 50% flow depth. • The floor of the transition should be sloped down as required to join the invert of the feeder canal. Check that the flow velocity in the transition is adequate to prevent deposition in the transition area. If sediment loads are very high consider installing a vortex silt ejector at the downstream end of the transition. Provide coarse trashracks to guard entry to the head gate. The trashrack would be designed to step floating debris such as trees, branches, wood on other floating objects. A clear spacing of 150 mm between bars is recommended. Trashrack detailed design should be in accordance with IS 11388. • The invert of the feeder canal shall be determined taking into consideration head losses through the trashrack and form losses through the structure. Friction losses can be omitted as they are negligible: V2 Calculate form losses as: H L = 0. 3 2 2g Where: V2 = velocity at downstream end of contraction.

Calculate trashrack losses as: 4/3 V2 ? t? H L = K f ? ? . Sin? . 2g ? b? Where: Kf = head loss factor (= 2. 42 assuming rectangular bars) T = thickness of bars (mm) B = clear bar spacing (mm) ? = angle of inclination to horizontal (degrees) V = approach velocity (m/s) 1. 1. 10 References on Lateral Intakes and Diversion Weirs. IS Standards Cited: IS 6966 (Part 1) IS 11388 USBR (1987) Singh, Bharat Nigam, P. S. Hydraulic Design of Barrages and Weirs -

Guidelines Recommendations for Design of Trashracks for Intakes Design of Small Dams Fundamentals of Irrigation Engineering Nem Chand & Bros. Roorkee (1983) Handbook of Hydroelectric Engineering (Second edition) ….. pages 357 to 365 Nem Chand & Bros. - Roorkee (1985) 1. 1. 11 Other References: Bucher and Krumdieck Guidelines for the Design of Intake Structures for Small Hydro Schemes; Hydro ’88/3rd International Conference on Small Hydro, Cancun – Mexico. Bouvard, M. Mobile Barrages and Intakes on Sediment Transporting AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 8 Razvan, E. 1. 2. Rivers; IAHR Monograph, A. A. Balkema – Rotterdam (1992) River Intakes and Diversion Dams

Elsevier, Amsterdam (1988) SEMI PERMANENT HEADWORKS (MINI HYDRO) For mini hydro projects the need to minimize capital cost of the head works is of prime importance. This issue poses the greatest challenge where the head works have to be constructed on alluvial foundations. This challenge is addressed by adoption of less rigorous standards and the application of simplified designs adapted to the skills available in remote areas. A typical layout is shown in Figure 2. 2. 3. 1. 2. 1 Design Parameters Hydraulic design should be based on the following design criteria: Plant flow Qp) = QT + QD Where: QT = total turbine flow (m3/s) QD = desilter flushing flow (= 0. 20 QT) m3/s QFC = feeder canal flow (= 1. 20 QT) m3/s QF = gravel flushing flow (= 2. 0 QP) Spillway design flow (SDF) = Q10 Where: Q10 = flood peak flow with ten year return period. 1. 2. 2 Layout ? To be reviewed Intake approach velocity = 1. 0 m/s Regulator gate W/H = 2 Flushing channel depth (HD) = 2H + W/3 Flushing channel minimum width = 1. 0 m Assumed flushing gate W/H = 2, determine H from orifice equation, as below: Q f = 0. 53? 2 H 2 . 2 gY1 Y1 = HD for design condition Where: W width of gate (m) H = height of gate (m) Yi = upstream depth (m) = depth of flushing channel (m) HD Select the next largest manufactures standard gate size above the calculated dimensions. 1. 2. 3 Weir AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 9 Determine weir height to suit intake gate and flushing gate dimensions, as shown in Figure 2. 2. 3. For weirs founded on permeable foundations the necessary structure length to controlfailureby piping should be determined in accordance with Section 2. 2. 1/4. 1 of this Standard.

A stepped arrangement is recommended for the downstream face of the weir to dissipate hydraulic energy. The height of the steps should not exceed 0. 5 m and the rise over run ratio should not less than 1/3, the stability of the weir cross-section design should be checked for flotation, over turning and sliding in accordance with Section 2. 3. 1. 1. 3 TRENCH INTAKES Trench intakes are intake structures located in the river bed that draw off flow through racks into a trench which conveys the flow into the project water conductor system. A characteristic of trench intakes is that they have minimum impact on river levels.

Trench intakes are applied in situations where traditional headwork designs would be excessively expensive or result in objectionable rises in river levels. There are two quite different applications: on wide rivers and on mountainous streams, but the basic equations are the same for both types. The trench intake should be located in the main river channel and be of sufficient width to collect the design project flow including all flushing flows.

If the length of the trench is less than the width of the river, cut off walls will be required into each bank to prevent the river from bypassing the structure.

Trench weirs function best on weirs with slopes greater than 4%-5%, for flatter slopes diversion weirs should be considered. The spacing between racks is selected to prevent entry of bed load into the trench. The following terms are sometimes used in referring to trench intake designs. Trench weir, when the trench is installed in a raised embankment. • Tyrolean or Caucasian intakes, when referring to trench intakes on • mountainous streams. Features: AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 10 1. 3. 2 Design Parameters

The following design parameters are suggested for the dimensioning of trench weirs. • Design Flows: The following design flows are recommended: Bedload flushing flow (from collector box) = 0. 2 QT • Desilter flushing flow = 0. 2 QT • Turbine flow = 1. 0 QT • Total design flow • = 1. 4 QT Dimensional Layout AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 11 The following factors should be considered in determining the principal dimensions: length, breadth and depth of a trench weir: Minimum width (B)= 1. 25 m (to facilitate manual cleaning) Length should be compatible with river cross section. It is • recommended that the trench be located across main river channel. Maximum width (B) ? 2. 50m. Trashrack bars longer than about 2. 50 m • may require support as slenderness ratios become excessive. Invert of collector box should be kept a high as possible. • • Racks • • • • The clear spacing between bars should be selected to prevent entry of bed-load particles that are too large to be conveniently handled by the flushing system. Generally designs are based on excluding particles greater than medium gravel size from (2 cm to 4 cm).

A clear opening of 3. 0 cm is recommended for design. A slope across the rack should be provided to avoid accumulation of bed load on the racks. Slopes normally used vary from 0° to 20°. Rectangular bars are recommended. Bar structural dimension shall be designed in accordance with Section 2. 2. 1/5. 0 of this Standard. An appropriate contraction coefficient should be selected as explained in the following sub-section. Assume 30% blockage. Spacing between racks is designed to prevent the entry of bedload but must also be strong enough to support superimposed loads from bedload accumulation, men and equipment.

This issue is discussed further in Subsection 2. 2. 3 / 2. 0. 1. 3. 3 Hydraulic Design of Trench Intake The first step in hydraulic design is to decide the width of the trench intake bearing in mind the flow capacity required and the bathymetry of the river bed. The next step in hydraulic design is to determine the minimum trench breadth (B) that will capture the required design flow. The design approach assumes complete capture of river flow, which implies, that river flow is equal to plant flow for the design condition. Hydraulic design is based on the following assumptions: Constant specific energy across racks. • Effective head on screen is equal to base pressure (depth) • Approach velocity is subcritical with a critical section at the entry to the structure as shown in figure 2. 2. 3/1. The set of equations proposed is based on the method given by Lauterjung et al (1989). • First calculate y1: AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of

Small Hydro Plants /May 2011 12 2 y 1 = k. H0 3 - - - - - - - - - - - - (1) Where: y1 = depth at upstream edge of rack Ho = the energy head of the approaching flow k = an adjustment factor (m) m) (-) k is a function of inclination of the rack and can be determined from the following table: Values of k as a Function of Rack Slope (?) Table: 2. 2. 1/1 ? = 0° 2° 4° 6° 8° 10° 12° k = 1. 000 0. 980 0. 961 0. 944 0. 927 0. 910 0. 894 ? = 14° 16° 18° 20° 22° 24° 26° k = 0. 879 0. 865 0. 851 0. 837 0. 852 0. 812 0. 800 Then calculate the breadth of the collector trench from the following equations (2) to (4) 1. 50 q - - - - - - - - - - - - - - (2) L= E1. E 2 C. cos? 3/2 . 2gy 1 Where: L = sloped length across collector trench (m) E1 = blockage factor E2 = Effective screen area = e/m

C = contraction coefficient ? = slope of rack in degrees y1 = flow depth upstream from Equation 1. (m) q = unit flow entering intake (m3/s per m) e = clear distance between bars (cm or m) m = c/c spacing of bars (cm or m) Assume E1 = 0. 3 (30%) blockage. “ C” can be calculated from the following formula (as reported by Raudkivi) Rectangular bars: ? e? C = 0. 66 ? ? ? m? ? 0. 16 ? m? .? ? ? h? 0. 13 Assume h = 0. 5 y1. This formula is valid for 3. 5> - - - - - - - - - - - - - - (3) h e > 0. 2 and 0. 15 < < 0. 30 m m Finally, the required breadth (B) can be determined as below: B = L cos ? - - - - - - - - - - - - -(4) AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 13 1. 3. 4 Hydraulic Design of Collector Trench Normally a sufficient slope on the invert of the trench is provided to ensure efficient flushing of bed-load particles that would otherwise accumulate on the invert of the trench. A suitable scouring slope can be estimated from the following equation: Ss = 0. 66 d 9 / 7 6/7 qo Where: d =

sediment size (m) qo = flow per unit width (Q/B) at outlet of trench (m3/s per m) Ss = design slope of trench invert.

The minimum depth of the trench at the upstream and is normally between 1. 0m to 1. 5 m, based on water depth plus a freeboard of 0. 3 m. For final design the flow profile should be computed for the design slope and the trench bottom profile confirmed or adjusted, as required. A step-by-step procedure for calculating the flow profile that is applicable to this problem can be found in Example 124, page 342-345 of " Open-Channel Hydraulics" by Ven. T. Chow (1959). In most cases the profile will be sub critical with control from the downstream (exit) end.

A suitable starting point would be to assume critical flow depth at the exit of the trench. 1. 3. 5 Collector Chamber The trench terminates in a collector box. The collection box has two outlets, an intake to the water conductor system and a flushing pipe. The flushing pipe must be design with the capacity to flush the bed-load sediment entering from the trench, while the project flow is withdrawn via the intake. The bottom of the collection box must be designed to provide adequate submergence for the flushing pipe and intake to suppress undesirable vortices.

The flushing pipe should be lower than the intake and the flushing pipe sized to handle the discharge of bed load. If the flushing pipe invert is below the outlet of the trench, the Engineer should consider steepening the trench invert. If the trench outlet invert is below the flushing pipe invert, the latter should be lowered to the elevation of the trench outlet or below. The deck of the collector box should be located above the design flood level to provide safe access to operate gates. AHEC/MNRE/SHP Standards/ Civil Works -

Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 14 1. 3. Flushing Pipe The flushing pipe should be designed to provide a high enough velocity to entrain bed-load captured by the weir. A velocity of at least 3. 0 m/s should be provided. If possible, the outlet end of the pipe should be located a minimum of 1. 0m above the river bed level to provide energy to keep the outlet area free from accumulation of bed load that could block the pipeline. 1. 3. 7 References on Trench weirs CBIP, (2001): Manual on Planning and Design of Small Hydroelectric Scheme Lauterjung et al (1989): Planning of Intake Structures Freidrich Vieweg and Sohn, Braunswchweig - Germany

IAHR (1993): Hydraulic Structures Design Manual: Sedimentation: Exclusion and Removal of Sediment from Diverted Water. By: Arved J. Raudkivi Publisher: Taylor & Francis, New York. Chow (1959): Open- Channel Hydraulics Publisher: McGraw-Hill Book Company, New York. 1. 4 RESERVOIR, CANAL AND PENSTOCK INTAKES The designs of reservoir, canal and penstock intakes are all based on the same principles. However, there are significant variations depending on whether an intake is at the forebay reservoir of a run-of-river plant or at storage reservoir with large draw down or is for a power tunnel, etc.

Examples of a variety of layouts can be fond in IS 9761 Hydropower Intakes – Criteria for Hydraulic Design or Guidelines for Design of Intakes for Hydropower Plants (ASCE, 1995). The features common to all designs are shown in the following sketch: AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 15 The objectives of good design are: To prevent entry of floating debris. • To avoid formation of air entraining vortices. • To minimize hydraulic losses. • 1. 4. 1 Control of floating debris

To prevent the entry of debris a trashrack is placed at the entry to the intake. For small hydro plants the trashrack overall size is determined based on an approach velocity of 0. 75 m/s to 1. 0m/s to facilitate manual raking. Trashracks may be designed in panels that can be lowered into place in grooves provided in the intake walls or permanently attacked to anchors in the intake face. The trashracks should to sloped at 14° from the vertical (4V: 1H) to facilitate raking. The spacing between bars is determined as a function of the spacing between turbine runner blades.

IS 11388 Recommendations for Design of Trashracks for Intakes should be consulted for information about spacing between trashracks bars, structural design and vibration problems. Also, see Section 2. 2. 1/5 of this Standard. 1. 4. 2 Control of Vortices First of all the direction of approach velocity should be axial withrespectthe intake if at all possible. If flow approaches at a significant angle (greater than 45o) AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 16 from axial these will be significant risk of vortex problems.

In such a situation an experienced hydraulic engineer should be consulted and for important projects hydraulic model studies may be required. For normal approach flow the submergence can be determined from the following formulae: S = 0. 725VD0. 5 S D V = submergence to the roof of the gate section (m) = diameter of penstock and height of gate (m) = velocity at gate for design flow. (m/s) Where: A recent paper by Raghavan and

Ramachandran discusses the merits of various formulae for determining submergence (S). 1. 4. 3 Minimization of Head losses

Head losses are minimized by providing a streamlined transition between the entry section and gate section. Minimum losses will be produced when a streamlined bellmouth intake is used. For a bellmouth intake the transition section is formed with quadrants of ellipses as shown in the following sketch. The bellmouth type intake is preferred when ever the additional costs are economically justified. For smaller, mainly mini hydropower stations, simpler designs are often optimal as the cost of construction of curved concrete surfaces may not be offset by the value of reduction in head losses.

Details on the geometry of both types are given • Bellmouth Intake Geometry Geometries for typical run-of-river intakes are shown below: A gate width to height of 0. 785 (D): 1. 00 (H) with H = D is recommended. This permits some reduction in the cost of gates without a significant sacrifice in hydraulic efficiency. There is a second transition between the gate and penstock, rectangular to circular. For a gate having H = D and W= 0. 785D the flow velocity at the gate will be equal to the velocity in the penstock so no further flow acceleration is produced in this section. A length for this transition of 1. x D should be satisfactory. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 17 The head loss co-efficient for this arrangement in Ki = 0. 10 Details for layout of bell mouth transitions connecting to a sloping penstock are given in IS9761. • Simplified layout (Mini-Hydro): For smaller/mini hydro projects intake design can be simplified by forming the transition in plane surfaces as shown below: The head loss for this design (Ki)

= 0. 19V2/2g. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 18 . 4. 4. AIR VENT An air vent should be placed downstream of the head gate to facilitate air exchange between atmosphere and the penstock for the following conditions: • Penstock filling when air will be expelled from the penstock as water enters. • Penstock draining when air will enter the penstock to occupy the space previously filled by water. The air vent (pipe) must have an adequate cross section area to effectively handle these exchanges of air. The following design rules are recommended: Air vent area should the greater of the following values Where: (m3/s) AV = 0. 0 Ap or QT AV = 25. 0 (m2) AV = cross-section area of air vent pipe AP = cross-section area of penstock (m2) QP = turbine rated flow (? QT of more than one turbine on the penstock) The air vent should exhaust to a safe location unoccupied by power company employees on the general public. 1. 4. 5 PENSTOCK FILLING A penstock should be filled slowly to avoid excessive and dangerous “ blowback”. The recommended practice is to control filling rate via the head gate. The AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 19 ead gate should not be opened more than 50 mm until the penstock is completely full. (This is sometime referred to as “ cracking” the gate.) 1. 4. 6 REFERENCES ON PENSTOCK INTAKES: • 1. 4. 7 Indian Standard Cited. IS 9761: Hydropower Intakes – Criteria for Hydraulic Design OTHER REFERENCES • Guidelines for Design of Intakes for Hydroelectric Plants ASCE, New York (1995) • Validating the Design of an Intake Structure : By Narasimham Raghavan and M. K. Ramachandran, HRW – September 2007. • Layman’s Guidebook European Small Hydro Association Brussels, Belgium (June 1998)

Available on the internet. • Vortices at Intakes By J. L. Gordon Water Power & Dam Construction April 1970 1. 5. TRASHRACKS AND SAFETY RACKS 1. 5. 1 Trashracks: Trashracks at penstock intakes for small hydro plants should be sloped at 4 V: 1H to facilitate manual raking and the approach velocity to the trashracks limited to 1. 0 m/s or less. Use of rectangular bars is normal practice for SHP's. Support beams should be alignment with the flow direction to minimize hydraulic losses. Detailed trashrack design should be done in accordance with IS 11388. 1. 5. 2

Safety Racks: Safety racks are required at tunnel and inverted siphon entries to prevent animals or people who may have fallen into the canal from being pulled into these submerged water ways. A clear spacing of 200 mm between bars is recommended. Other aspects of design should be in accordance with IS 11388. 1. 5. 3 References on Trashracks IS11388 – " Recommendations for Design of Trashracks for Intakes". ASCE (1995) --" Guidelines for Design of Intakes for Hydroelectric Plants". AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 20 DRAWINGS:

AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 21 AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 22 2. HYDRAULIC DESIGN OF WATERWAYS The waterways or water conduction system is the system of canals, aqueducts, tunnels, inverted siphons and pipelines connecting the head works with the forebay tank. This Section provides guidelines and norms for the hydraulic design of these structures.

2. 1 2. 1. 1 CANALS Canals for small hydro plants are typically constructed in masonry or reinforced concrete.

Several typical cross section designs are shown below: AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 23 Lined canals in earth, if required, should be designed in accordance with Indian Standard: IS 10430. A further division of canal types is based on function: - Feeder canal to connect the head regulator (intake) to the desilter - Power canal to connect the desilter to the Forebay tank. 2. 1. 2 Feeder Canals 2. 1. 2. 1 Feeder canal hydraulic design shall be based on the following criteria: = Turbine flow (QT) + Desilter flushing flow (QF).

Design flow (Qd) AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 24 2. 1. 2. 2 Scouring velocity: A sufficiently high velocity must be provided to prevent deposition of sediment within the canal. This (scouring) velocity can be determined from the following formulae: d 9/7 S C = 0. 66 6 / 7 n = 0. 015 q 1 1 ? VS = . R 2 / 3 . S C/ 2 n Where: Sc = Scouring slope d = Target sediment size (m) q = Flow per unit width (Q/W) (m/s/m) R = hydraulic radius (m) Vs = scouring velocity (m/s) n = Manning’s roughness coefficient 2. 1. 2. 3 Optimization:

The optimum cross section dimensions, slope and velocity should be determined by economic analysis so as to minimize the total life time costs of capital, O&M and head losses (as capitalized value). The economic parameters for this analysis should be chosen in consultation with the appropriate regional, state or central power authorities these parameters include: - Discount rate (i) - Escalation rate(e) - Plant load factor - Service life in years (n) - Annual O+M for canal (% of capital cost) - Value of energy losses (Rs/kWh). Also see Section 1. 7 of this Standard. The selected design would be based on the highest of Vs or Voptimum. . 1. 2. 4 Freeboard: A freeboard allowance above the steady state design water level is required to contain water safely within the canal in event of power outages or floods. A minimum of 0. 5 m is recommended. 2. 1. 3 Power Canals: Power canal design shall be based on the following criteria a) Design flow = total turbine flow (QT) b) Power canal design should be based on optimization of dimensions, slope and velocity, as explained in the previous section. For mini-hydro plants Q < 2. 0 m3/s optimal geometric design dimensions for Type 1 (masonry construction) can be estimated by assuming a longitudinal slope of 0. 04 and a Manning's n value of 0. 018. Masonry construction would normally be preferred for canals with widths (W) less than 2. 0 m (flow area = AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 25 2. 0 m2). For larger canals with flow areas greater than 2. 0m2, a Type 3, box culvert design would be preferred – based on economic analysis. c) Freeboard: A freeboard allowance above the steady state design level is required to contain water safety within the canal in event of power outages. The waterway in most SHP's terminates in a Forebay tank.

This tank is normally equipped with an escape weir to discharge surplus water or an escape weir is provided near to the forebay tank. For mini-hydro plants a minimum freeboard of 0. 50 m is recommended. The adequacy of the above minimum freeboard should be verified for the following conditions: • Maximum flow in the power canal co-incident with sudden outage of the plant. • Design flow plus margins for leakage losses (+0. 02 to +0. 05 QT) and above rated operation (+ 0. 1QT). • Characteristics of head regulator flow control. The freeboard allowance may be reduced to 0. 5 m after taking these factors into consideration. The maximum water level occurring in the forebay tank can be determined from the weir equation governing flow in the escape weir. 2. 1. 4 Rejection Surge Designs which do not incorporate downstream escape weirs would be subject to the occurrence of a rejection surge in the canal on sudden turbine shutdown, giving above static water levels at the downstream end, reducing to the static level at the upstream (entry) end of the water way. Methods for evaluating water level changes due to a rejection surge are explained in Section 2. 2. 2 / 7. 0 of this Standard. . 2 AQUEDUCTS Aqueducts are typically required where feeder or power canals pass over a gully or side stream valley. If the length of the aqueduct is relatively short the same channel dimensions as for the canal can be retained and there would be no change in hydraulic design. For longer aqueducts design would be based on economic analysis subject to the proviso that flow remains sub critical with NF ? 0. 8 in the flume sections. The following sketch shows the principal dimension of aqueduct entry and exit transitions and flume section. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of

Small Hydro Plants /May 2011 26 The changes in invert elevation across the entry and exit structures can be calculated by Bernouli's equation as below: • Entry transition – consider cross – section (1) and (2); V2 V2 Z 1 + D + 1 = Z 2 + d + 2 + hL 2g 2g and 2 • b? V ? hL = 0. 10 ? 1 ? ?. 2 ? B ? 2g Z2 can be determined from the above equations, since all geometrical parameters are known. Flume – Sections (2) to (3) The slope of the flume section is determined from Manning's equation 2 • ? Vn ? (S) = ? 2 / 3 ? . A Manning's n = 0. 018 is suggested for concrete channels. ? R ?

Some designers increase this slope by 10% to provide a margin of safety on flow capacity of the flume. Exit transition – consider cross section (3) and (4): V2 V2 Z 3 + d + 3 = Z 4 + D + 4 + hL 2g 2g AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 27 and 2 b? V ? hL = 0. 20 ? 1 ? ?. 3 ? B ? 2g Z4 can be determined from the above equations, since all geometrical parameters are known. The same basic geometry can be adapted for transition between trapezoidal canals sections and rectangular flume section, using mean flow width (B) = A/D. . 3. INVERTED SYPHONS 2. 3. 1 Inverted syphons are used where it is more economical to route the waterway underneath an obstacle. The inverted syphon is made up of the following components: • Entry structure • Syphon barrels • Exit structure • Entry Structure: Hydraulic design of the entry structure is similar to the design of reservoir, canal and penstock intakes. Follow the guidelines given in Section 2. 2. 2/2. of this Standard. • Syphon barrels: The syphon barrel dimensions are normally determined by optimization ? V? ? does not tudies, with the proviso that the Froude Number ? N F = ? gd ? ? ? exceed 0. 8. Invert elevations are determined by accounting for head losses from entry to exit of the structure using Bernouli's equation. For reinforced concrete channels a Manning's " n" value of 0. 018 is recommended. The head loss coefficients for mitre bends can be determined from USACE HDC 228. 2. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 28 AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of

Small Hydro Plants /May 2011 29 Exit structure: The exit structure is designed as a diverging transition to minimize head losses; the design is similar to the outlet transition from flume to canal as discussed in Subsection 2. 2. 2/2 of this Standard. The following sketches show the layout of a typical inverted siphon. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 30 2. 3. 2 Reference on Aqueducts and Inverted Syphons “ Hydraulic Structures” By C. D. Smith University of Saskatchewan Saskatoon (SK) Canada 2. 4. LOW PRESSURE PIPELINES

Low pressure pipelines may be employed as an alternative to pressurized box culverts, aqueducts or inverted syphons. Concrete, plastic and steel pipes are suitable depending on site conditions and economics. Steel pipe is often an attractive alternative in place of concrete aqueducts in the form of pipe bridges, since relatively large diameter pipe possesses significant inherent structural strength. Steel pipe (with stiffening rings, as necessary), concrete and plastic pipe also have significant resistance against external pressure, if buried, and offer alternatives to inverted syphons of reinforced concrete construction.

Generally pressurized flow is preferred. The pipe profile should be chosen so that pressure is positive through out. If there is a high point in the line that could trap air on filling an air bleeder valve should be provided. Otherwise, hydraulic design for low pressure pipelines is similar to the requirements for inverted syphons. The choice of type of design; low pressure pipeline land pipeline material), inverted syphon or aqueduct, depends on economic and constructability considerations, in the context of a given SHP. Manning’s “ n”

Values for selected Pipe Materials Material Welded Steel Polyethylene (HDPE) Poly Vinyl Chloride (PVC)

Asbestos Cement Cast iron Ductile iron Precast concrete pipe Manning's " n" 0. 012 0. 009 0. 009 0. 011 0. 014 0. 015 0. 013(2) Note: (1) From Table 5. 4 Layman's Guide Book – ESHA (2) From Ven T. Chow – Open Channel Hydraulics AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 31 2. 5. TUNNELS 2. 5. 1 Tunnels often provide an appropriate solution for water conveyance in mountainous areas. Tunnels for SHP are generally of two types. • Unlined tunnels • Concrete lined tunnels On SHP tunnels are usually used as part of the water ways system and not subject to high pressures. . 5. 2 Unlined tunnels: Unlined water tunnels can be used in areas of favourable geology where the following criteria are satisfied: a) Rock mass is adequately water tight. Rock surfaces are sound and not vulnerable to erosion (or erodible zones b) are suitably protected. The static water pressure does not exceed the magnitude of the minor field c) rock stress. Controlled perimeter blasting is recommended in order to minimize over break and produce sound rock surfaces. Additionally, this construction approach tends to produce relatively uniform surfaces and minimizes the hydraulic roughness of the completed tunnel surfaces.

Design velocities of 1. 5 to 2. 0 m/s on the mean AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 32 cross section area give optimal cross section design. It is normal practice to provide a 100mm thick reinforced concrete pavement over leveled and compacted tunnel muck in the invent of the tunnel. IS 4880: Part

3 provides additional guidance on the hydraulic design of tunnels and on the selection of appropriate Manning's " n" values. 2. 5. 3 Lined Tunnels Where geological are unfavourable it is often necessary to provide concrete linings for support of rock surfaces.

IS4880: Parts 1-7 give comprehensive guidelines on the design of lined tunnels. 2. 5. 4 High Pressure Tunnels Design of high pressure tunnels is not covered in this standard. For high pressure design, if required, the designer should consult an experienced geotechnical engineer or engineering geologist. For the purpose of this standard, high pressure design is defined as tunnels subject to water pressures in excess of 10m relative to the crown of the tunnels. 2. 5. 5 Reference on Tunnels IS Standards: IS 4880 " Code of Practice for the Design of Tunnels Conveying Water". Other References: Norwegian Hydropower Tunnelling" (Third volume of collected papers) Norwegian Tunneling Society Trondheim, Norway. www. tunnel. no Notably: Development of Unlined Pressure Shafts and Tunnels in Norway, by Einar Broch. 2. 6. CULVERTS AND CROSS-DRAINAGE WORKS Small hydro projects constructed in hilly areas usually include a lengthy power canal routed along a hillside contour. Lateral inflows from streams and gullies intercepted by SHP canals often transport large sediments loads which must be prevented from entering the canal. The first line of defense is the canal upstream ditch which intercepts local lateral runoff.

The flow in these chains must be periodically discharged or the drain capacity will be exceeded. Flow from these drains is usually evacuated via culverts passing underneath the canal. These culverts would normally be located where gullies or streams cross the canal alignment. The capacity of canal ditches should be decided taking into consideration the average distance between culverts. In the rare cases when distance between culverts is excessive, consideration should be given to diverting AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 33 itch flows across the canal in flumes or half round pipes to discharge over the downhill side of the canal at suitable locations. Culverts are usually required where the canal route crosses gullies or streams. Culverts at these points provide for flow separation between lateral inflows and canal inflows and often present the most economical solution for crossing small but steep valley locations. It is recommended that culverts design be based on the following hydrological criteria. • For mini hydro projects, 1 in 10 year flood (Q10) • For small hydro projects, 1 in 25 year flood (Q25)

Where it is practical to extract the necessary basin parameters, the procedures given in Section 1. 4 should be applied. Otherwise design flows should be estimated from field measurements of cross section area and longitudinal slope at representative cross section of the gully or side stream. A survivable design approach is further recommended with canal walls strengthened to allow local over topping without damage to the canal integrity when floods exceed the design flood values. Detailed hydraulic design should be based on information from reliable texts or design guidelines – such as: “ Design of Small Bridges and Culverts” Goverdhanlal • • 2. 7 2. 7. 1 “ Engineering and Design – Drainage and Erosion Control”. Engineering Manual EM 1110-3-136 U. S. Army Corps of Engineers (1984) www. usace. army. mil/publications/eng-manuals Manufacturer’s guides, notably: - American Concrete Pipe Association www. concrete-pipe. org - Corrugated Steel Pipe Institute www. cspi. ca Power Canal Surges Power canals that are not provided with escape weirs near their downstream end will be subject to canal surges on rapid load rejections or load additions.

The rejection surge will typically cause the downstream water level to rise above static level and may control the design of canal freeboard. For load additions there is a risk that the level will fall to critical at the downstream end and restrict the rate at which load can be taken on by the unit. The following formulae taken from IS 7916: 1992 can be used to estimate the magnitude of canal surges. AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 34 Maximum surge height in a power channel due to load rejection may be calculated from the empirical formulae given below:

For abrupt closure hmax = K 2 + 2 Kh For gradual closure within the period required for the first wave to travel twice the length of the channel: K hmax = + V . h / g 2 Where: hmax = maximum surge wave height, K = V2/2g = velocity head, V = mean velocity of flow, and area of cross sec tion h = effective depth = top width • Maximum water level resulting from a rejection surge at the downstream of a canal: Maximum W. L. = Yo + hmax • Minimum water level resulting from by a start up surge at the downstream end of a canal: Minimum W. L. = YS - hmax Where: Yo YS = steady state downstream water level static downstream water level. The maximum water level profile can be approximated by a straight line joining the maximum downstream water level to the reservoir level. 2. 7. 2 Canal Surges on Complex Waterways: For waterway systems comprising several different water conductor types, the above equations are not applicable. In such cases a more detailed type of analysis will be required. The U. S. National Weather Service FLDWAV computer program can be used to solved for the transient flow conditions in such cases (Helwig, 2002). 2. 7. 3 References IS Standards cited:

IS 7916: 1992 “ Open Channel – Code of Practice”. Other References “ Application of FLDWAV(Floodwave) Computer Model to Solve for Power Canal Rejection Wave for Simple and Complex Cases”. P. C. Helwig Canadian Society for Civil Engineering Proceedings, Annual Conference Montreal, Canada (2002). AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 35 3. HYDRAULIC DESIGN OF DESILTERS 3. 1 BACKGROUND Sediment transported in the flow, especially particles of hard materials such as quartz, can be harmful to turbine components.

The severity of damage to equipment is a function of several variables, notably: sediment size, sediment hardness, particle shape, sediment concentration and plant head. The control of turbine wear problems due to silt erosion requires a comprehensive design approach in which sediment properties, turbine mechanical and hydraulic design, material selection and features to facilitate equipment maintenance are all considered (Naidu, 2004). Accordingly the design parameters for desilter design should be made in consultation with the mechanical designers and turbine manufacturer.

Where the risk of damage is judged to be high a settling basin (or desilter) should be constructed in the plant waterway to remove particles, greater than a selected target size. 3. 1. 1 Need The first design decision is to determine whether the sediment load in the river of interest is sufficiently high to merit construction of a desilter. There is little guidance available on this topic; however, the following limits are suggested by Naidu (2004): Table 2. 2. 3/1. 0 Concentration Suggested Maximum Allowable Sediment versus Plant Head. Parameter Head Maximum allowable sediment concentration

Low and Medium Head Turbines ? 150 m High Head Turbines > 150 m 200 ppm 150 ppm 3. 1. 2 Removal Size There are also considerable divergences of opinion on the selection of design size for sediment removal. Nozaki (1985) suggests a size range of between 0. 3 mm to 0. 6 mm for plant heads ranging from 100 m to 300 m. Indian practice is to design for a particles size of 0. 20 m regardless of head. Some authors suggest that removal of particles smaller than 0. 20 mm is not practical. The adoption of 0. 20 mm is the design (target) sediment size is recommended for Indian SHP designs.

AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 36 3. 1. 3 Types of Desilters There are two basic types of desilters: Continuous flushing type Intermittent flushing type Guidelines for design of both types are given in this section. 3. 2. DESIGN CONSIDERATIONS 3. 2. 1 Data Requirements (Small Hydro Plants) It is recommended that a program of suspended sediment sampling be initiated near the intake site from an early stage during site investigations to ensure that sufficient data is available for design.

The sampling program should extend through the entire rainy season and should comprise at least two readings daily. On glacier fed rivers where diurnal flow variations may exist, the schedule of sampling should be adjusted to take this phenomenon into account and the scheduled sampling times be adjusted to coincide with the hour of peak daily flow with another sample taken about twelve hours later. While it is often assumed that sediment load is directly related to flow, this is only true on the average, in a statistical sense.

In fact it is quite likely, that the peak sediment event of a year may be associated with a unique upstream event such as a major landslide into the river. Such events often account for a disproportionately large proportion of the annual sediment flow. Therefore, it would also be desirable to design the sediment measurement program to provide more detailed information about such events, basically to increase the sampling frequency to one sample per 1 or 2 hours at these times. A five year long sediment collecting program would be ideal. Less than one monsoon season of data is considered unsatisfactory.

Some authors suggest that the vertical variation of sediment concentration and variations horizontally across the river be measured. However, on fast flowing rivers inherent turbulence should ensure uniform mixing and sampling at one representative point should be sufficient. The data collected in a sediment sampling program should include: • Mean daily concentration of suspended sediment (average of two readings twelve hours apart) • Water temperature • Flow (from a related flow gauging program) The following additional information can then be derived from collected samples.

AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro Plants /May 2011 37 • • • A sediment rating curve (sediment concentration versus flow – where possible) Particle size gradation curve on combined sample Specific gravity of particles. It is also recommended that a petrographic analysis be carried out to identify the component minerals of the sediment mix. It is likewise recommended that experiments be made on selected ranges of particles sizes to determine settling velocities. A further discussion on the subject of sediment sampling is given in Avery (1989)

The characteristics of the sediment on a given river as obtained from a data collection program will assist in selection of appropriate design criteria. 3. 2. 2 Data Requirements (Mini Hydro Plants) On mini hydro projects where resources and time may not be available to undertake a comprehensive sampling program, selection of design parameters will depend to a great extent on engineering judgment, supplemented by observations on site and local information. The following regional formula by Garde and Kothyari (1985) can be used to support engineering decision making. 0. 19 ? P ? 0 Vs = 530. 0 P0. 6. Fe1. . S0. 25 Dd . 10 .? max ? ? P? Where Vs = mean sediment load in (tonnes/km2/year) s = average slope (m/m) Dd = drainage density, as total length of streams divided by catchment area (km/km2) P = mean annual precipitation (cm) Pmax = average precipitation for wettest month (cm) Fe = ground cover factor, as below: 1 Fe = [0. 80 AA + 0. 60 AG + 0. 30 AF + 0. 10 AW] ? Ai = arable land area AA = grass land area (all in km2) AG AF = forested area AW = waste land area (bare rock) 3. 2. 3 Design Criteria The principle design criteria are: 1. The target size for removal (d): d = 0. 20 mm is recommended 2.

Flushing flow: QF = 0. 2 QP is recommended 3. Total (design) flow: QT = QP + QF = 1. 2 QP. Where QP is plant flow capacity in (m3/s). AHEC/MNRE/SHP Standards/ Civil Works - Guidelines For Hydraulic Design Of Small Hydro

Plants /May 2011 38 3. 2. 4 Siting The following factors control site selection 1. A site along the water way of appropriate size and relatively level with respect to cross section topography 2. A site high enough above river level to provide adequate head for flushing. For preliminary layout a reference river level corresponding to the mean annual flood and minimum flushing head of 1. 0 m is recommended. In principle a desilting tank can be located anywhere along the water conductor system, upstream of the penstock intake. Sometimes it is convenient to locate the desilting basin at the downstream end of the waterway system where the desilter can also provide the functions of a forebay tank. However, a location as close to the head works is normally preferred, site topography permitting. 3. 3 Hydraulic Design A desilter is made up of the following elements: • Inlet section Settling tank • Outlet section • • Flushing system 3. 3. 1