Guyzance – Intake and Discharge Velocity Analysis

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V02

Table of Contents

Table of Figures

1 Summary

This paper presents the analyses to determine the average water velocities as a function of river flow and system intake flow. The system intake flow, as used here, is governed by a Proportional Take regime, in which there is a specified Hands-Off-Flow (HOF) and a sharing of river flows above that HOF. Average velocities are determined for the designed intake structure, as well as the discharge region of the hydropower system, and the river cross section just upstream of the hydropower discharge.

At the intake, the goal is to keep the average velocity through the intake screening ≤ 300 mm/s. This helps ensure that fish are able to swim away from the intake screen if they opt not to pass through. If a fine screen ends up being required, the designed intake should not cause impingement (trapping) of fish or eel. The analysis shows that this average velocity goal is met by the design.

At the discharge of the hydropower system, we do not want to create an attractive velocity for fish that is greater than the river flow running past the discharge point. The deepest point of the discharge area is at the excavation for the tail of the screw, which is designed as being set back a few meters from the existing river bank. This discharge pool then tapers up to meet the existing natural river bed at a little distance into the river. Our primary interest was at the transition from the tapered discharge pool to the existing natural river bed level. At all river flows investigated (Q10-Q70), the main river flow velocity was at least 4.4x the velocity of the water exiting from this discharge region.

Even when measured at the screw discharge face, the velocity of the screw flow is found always to be less than the velocity of flows in the main river.

2 Problem Statement

2.1 System Intake

It is necessary to determine what the average intake velocity of water is through the intake screen. The goal is to make sure that the velocity is not too high, and, in the event that adding a fine exclusion screen is found desirable, that the velocity is still not such that it causes impingement/trapping problems.

The target for the intake is to maintain the average velocity less than or equal to 300 mm/sec over the full range of water levels when the system is operating. The analysis presented here investigates this question. It is based on the proposed Q75 hands-off-flow (HOF) with a 50/50 split (or “Proportional Take”) of river flows above this level. (NB: Because of the necessary “start-up” flow for the system, under this regime the system in fact does not actually start up until approximately Q70 river flow.)

Figure 1 Intake Structure

Figure 1 above is a perspective view of the intake structure with 100 mm clear spacing between vertical bars. The intake is in-line with the river bank. The screening (in this design) is intended to keep out large debris and people only. It is not intended to exclude fish or eels.

Beyond the intake screen, the intake funnels down into a 1.8m-diameter pipe that leads to the screw system.

The width of the screen opening is 8.66 m and the invert of the screen opening is 15.57 mAOD. The area for the water flow is based on the clear opening for the water and the upper water level (UWL) under varying river flow and operating conditions.

If the EA ultimately finds grounds to determine that the screening at this system needs to be narrowed to 12.5 mm (clear openings), the intake civil structure as depicted will support this without modification, in that the finer screen would still remain within the water velocity guideline noted above ($\leq$ 300 mm /s).

2.2 System Discharge

There is a concern that the discharge of a screw system could attract fish from the main river flow and thereby delay their upstream migration. The analysis here looks at the river velocity at a cross section directly upstream of the discharge, and compares this with the average velocity of discharged water as it exits the excavated pool and enters the natural bed level of the river.

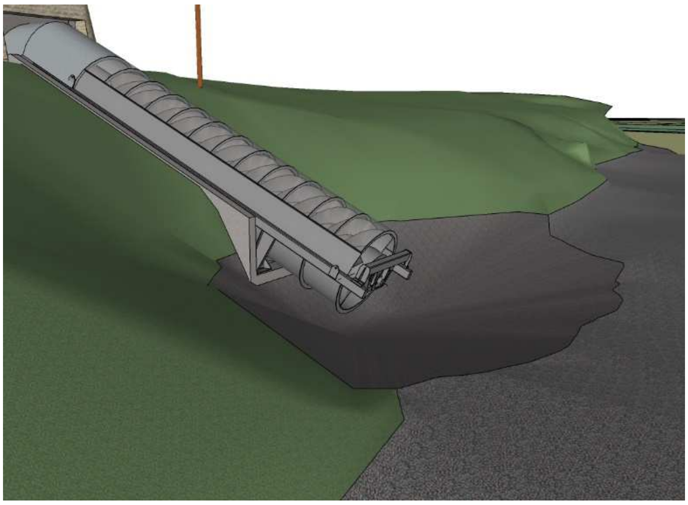

Figure 2 System Discharge - no water shown

Figure 2 above is a perspective view of the discharge region of the hydropower system. The screw system is set back into the bank a short distance, but it does extend down below the level of the current river bed. The bed is dug out just below the discharge point of the screw and is then tapered gradually back to the natural bed level. This sloped transition from the dug-out and/or dredged area extends a maximum of approximately 2.5 m (in plan) into the existing river bed, and tapers or slopes up towards all sides of the discharge region.

The analysis here looks at the average velocity of the water exiting this dug-out region – i.e. the section defined by the transition from dug-out to natural bed.

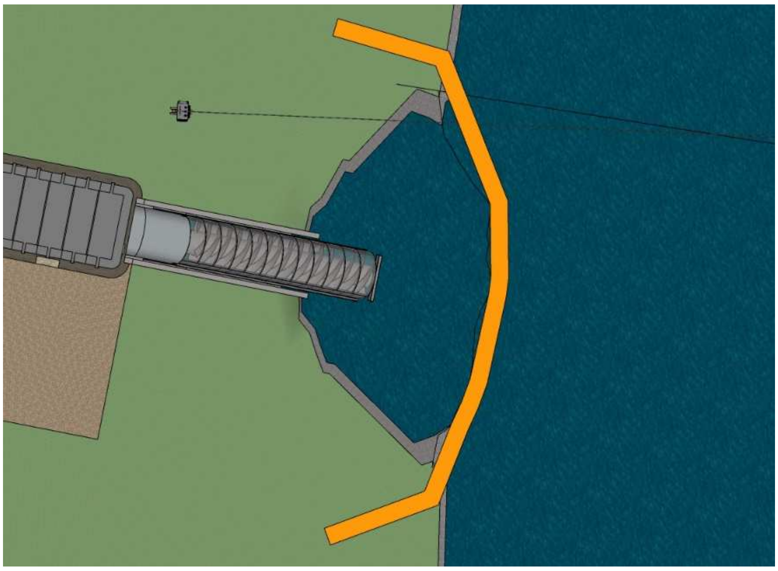

Figure 3 Plan View - coffer dam (likely using dumpy bags)

Figure 3 above is a plan view showing the extent of the coffer dam in orange. (The coffer-damming method here is likely to be dumpy bags and membrane.) All of the excavation of the sloped discharge pool (deepest at the screw, tapering up to the natural bed level) takes place within the proposed coffer dam.

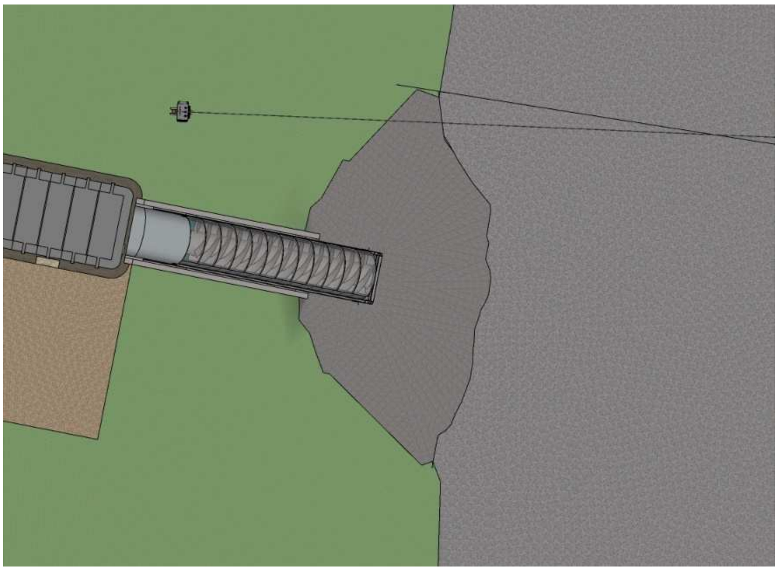

Figure 4 Plan View - Extent of Shaped Riverbed

Figure 4 above shows the extent of the dredged or profiled region in plan view.

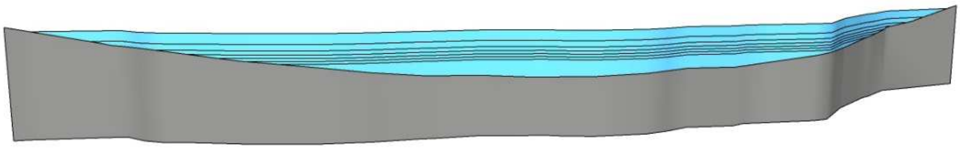

Figure 5 Cross Sectional Areas Discharge Passes Vs River Flow

Figure 5 is a perspective view of the calculated cross sections that the discharge water passes through (the transition line from dredged to natural bed level). The bottom of the figure represents the river bed. The blue regions represent the surface area at various river flows (Q10 to Q70), as determined by recent water-level monitoring at this location across this range of flows.

Figure 6 below is a similar plan, widened out to show the width of the main river channel. The discharge region is on the left (end of the screw). A straight line (up and down) shows the approximate position of the existing river bank. Just upstream of the discharge, the heavy diagonal line marks a section across the river. (NB: Ignore the lighter line, which is the route of an overhead cable.)

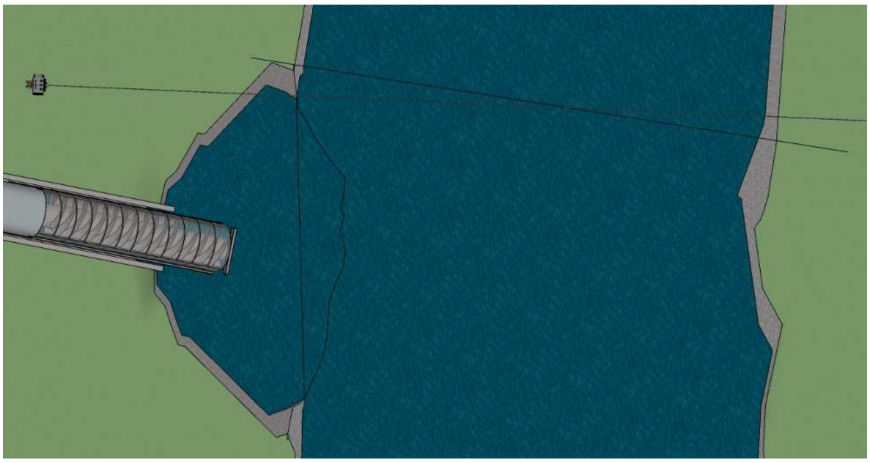

Figure 6 River Section and Discharge Area-Plan View

Figure 7 below, similar to Figure 5 above, is another section view. This is the section just upstream of the discharge region as marked in Figure 6. Again here the “blue” area is the water level at various river flows (Q10, 20, 30…70).

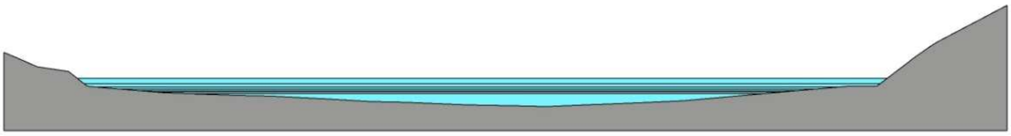

Figure 7 River Section

3 Analysis and Results

The analyses shown here are based on:

- Water level measurements made at the intake and discharge location in October of 2018. This data was referenced to AOD.
- Matching river flow data (adjusted for catchment difference from Morwick Gauge to hydropower site) from the EA
- Hydropower system proportional-take regime

The proportional take included a Hands-Off-Flow (HOF) equivalent to Q75 river flow (which was determined from a catchment adjusted, long record of river flows). Once the river reaches this HOF level, the flow must rise just a bit more to provide enough for the minimum hydropower start-up, which we've assumed to be 5% of the maximum system flow. Effectively, this means that the hydropower system will not start until approximately Q70.

At flows above this HOF, the extra flow is shared 50/50 between the depleted reach and the hydropower system; until the hydropower system is at its full flow capacity. Above this point, the hydro takes no more, so all of any additional river flow continues in the depleted reach.

The table below gives a summary of site and system flows over the range of river flows (based on the "proportional take" described above).

Qn	**Site flow m^3/s**	**System Flow m^3/s**	**% of Total thru System**
1	60.302	2.90	4.8%
5	28.282	2.90	10.3%
10	18.243	2.90	15.9%
15	13.707	2.90	21.2%
20	11.004	2.90	26.4%
25	9.228	2.90	31.4%
30	7.818	2.82	36.1%
35	6.815	2.32	34.0%

Qn	Site flow m^3/s	System Flow m^3/s	% of Total thru System
40	5.965	1.89	31.7%
45	5.193	1.51	29.0%
50	4.566	1.19	26.1%
55	3.996	0.91	22.7%
60	3.475	0.65	18.6%
65	2.983	0.40	13.5%
70	2.539	0.18	7.1%
75	2.181	0	0.0%
80	1.902	0	0.0%
85	1.660	0	0.0%
90	1.429	0	0.0%
95	1.178	0	0.0%

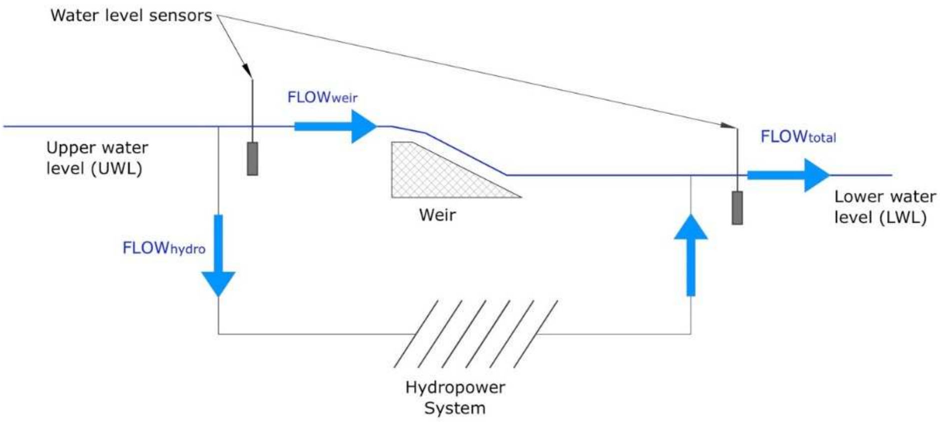

Figure 8 Proportional Take Schematic

Figure 8 shows the site schematically and how the proportional take functions. The "Depleted" reach of the river is the flow passing over the weir, i.e. flow staying in the river from the intake to the discharge point. The figure also includes the water level sensors used for system control, including that at the upstream intake which would be used to maintain the proportional take.

3.1 Intake Analysis

The table below is the summary of the results of the "intake" velocity analysis.

For each value of Qn, the upper water level (UWL) varies.

- When the hydro is NOT operating, the UWL at the intake represents the TOTAL river flow for that given Qn. But the screen velocity is zero (no flow into the intake).
- When the hydro is operating, the UWL no longer represents the total river flow for that Qn, but instead it represents what the Qn value would be if the flow left in the depleted reach by the hydro were the total flow (table above).

			12.5 mm open spacing		100 mm open spacing	
Qn	**UWL mAOD**	**System Flow m^3/s**	**Open Area m^2**	**Vel Ave m/s**	**Open Area m^2**	**Vel Ave m/s**
10	17.399	2.900	11.314	0.256	12.671	0.229
20	17.199	2.900	10.078	0.288	11.287	0.257
30	17.099	2.819	9.456	0.298	10.591	0.266
40	17.066	1.893	9.257	0.204	10.367	0.183
50	17.040	1.193	9.091	0.131	10.182	0.117
60	17.016	0.647	8.948	0.072	10.021	0.065
70	16.994	0.179	8.811	0.020	9.868	0.018

One can see from the table above that, over a wide range of river flows, the average velocity through the intake screen remains below 300 mm/s.

Below Q70, the system is not operating, so the intake velocity at that point would be zero.

At flows greater than Q10, the flow into the system does not increase and therefore the average intake velocity will not increase. Of note, above Q10, the intake opening begins to be completely covered by the UWL.

3.2 Discharge Analysis: Comparison of Discharge and River Velocity

The lower water level (LWL) at the discharge point is governed by the TOTAL river flow; meaning the sum of the flow in the depleted reach and the hydropower system flow whenever operating.

The system discharge velocities (at the boundary between the dredged and natural river bed) were determined via the areas shown in Figure 5 above and the system flows. The results are shown in the table below.

Qn	LWL mAOD	Discharge Area m^2	System Flow m^3/s	Discharge Vel Ave m/s
10	12.273	12.319	2.900	0.235
20	12.089	8.421	2.900	0.344
30	11.980	6.285	2.819	0.448
40	11.910	5.044	1.893	0.375
50	11.853	4.090	1.193	0.292
60	11.802	3.284	0.647	0.197
70	11.752	2.553	0.179	0.070

The average river velocity was taken at a point JUST upstream of the discharge zone. The cross-sectional areas were determined from Figure 7 above. The FLOW used was the flow in the depleted reach. While the depleted and system flows combine in this location, the water velocity just upstream is governed by the depleted flow. This is what creates the “attraction” for upstream migration. The goal is that the system discharge velocity is low enough not to unduly compete with the attractive velocity from the main channel directly upstream.

Qn	LWL mAOD	Depleted Flow m^3/s	River Section m^2	River Section Ave velocity m/s
10	12.273	15.343	17.730	1.156
20	12.089	8.104	12.733	1.571
30	11.980	4.999	9.831	1.967
40	11.910	4.073	8.069	1.981
50	11.853	3.373	6.711	1.990
60	11.802	2.827	5.567	1.969
70	11.752	2.359	4.525	1.918

Comparing these two tables, one can see that the river velocity is always at least 4.4x that of the average velocity of water from the discharge region of the hydropower system.

3.3 Discharge Analysis: Comparison Screw Face and River Velocity

If fish cross from the natural river bed and into the deeper water directly downstream of the hydropower system, they will encounter higher velocity flow (up close to the screw discharge, at what we will call "the screw face"), but the velocity out of the screw is still less than the average velocity in the main river.

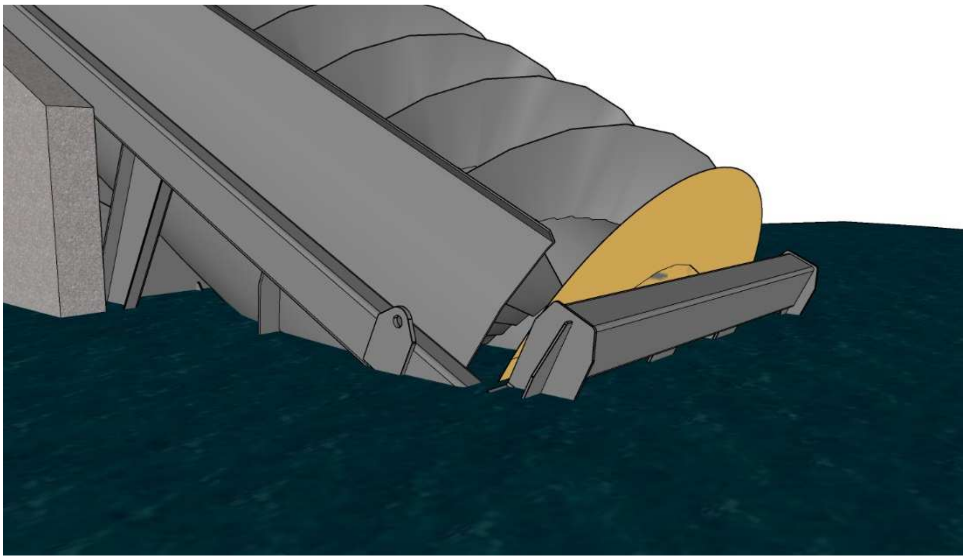

Figure 9 Determining Screw Discharge Area Vs LWL

Figure 9 is a perspective view showing the screw with the LWL at Q30 total river flow. The yellow disk is a hypothetical surface used to graphically determine the discharge area in the various LWL conditions (between Q10 and Q70).

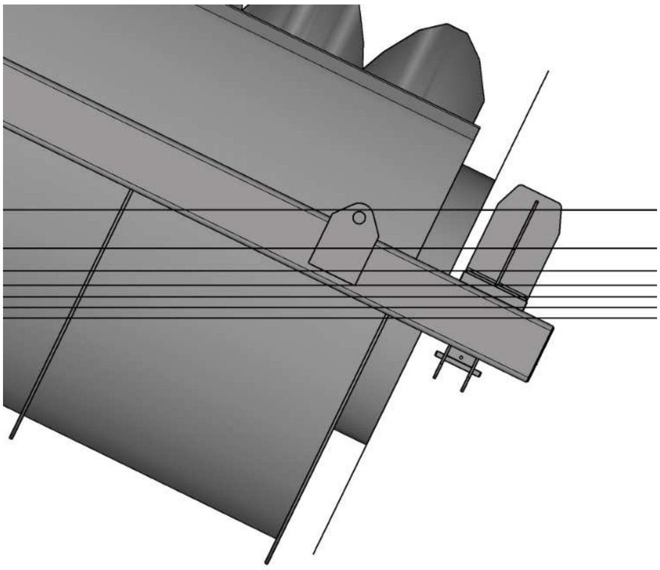

Figure 10 Side View-Screw Discharge, LWL Q70 to Q10

Figure 10 is a side view showing the disk used for determining discharge area and the LWL at Q10, 20, 30…70

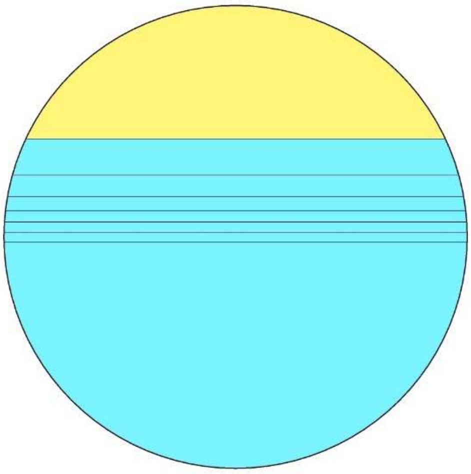

Figure 11 Screw Discharge Area (Blue is flow region)

Figure 11 shows the discharge areas for various cases of LWL, as used below.

Note – the screw diameter is 2.6 m

Qn	System Flow m^3/s	Area m^2	Vel Ave m/s
10	2.900	4.041	0.72
20	2.900	3.542	0.82
30	2.819	3.235	0.87
40	1.893	3.034	0.62
50	1.193	2.870	0.42
60	0.647	2.723	0.24
70	0.179	2.578	0.07

The table above tabulates the average screw discharge velocity for the various river conditions (for hydropower system flow in those conditions, under the proposed regime). If one compares these velocities with the average river velocity (just upstream of the discharge point), one will note that the river velocity is always at least 1.6x the velocity of the water at the screw discharge. The 1.6 ratio occurs at Q10 and increases to 8.3 by Q60 (27.6 at river flow Q70)

Appendix A References

Drawing: Guyzance-113-V01-20181119-Velocity-Discharge.pdf

Document: 9_SDM 2018XXXX Guyzance Meander 829.docx