Ministério de Minas e Energia
Brasilia – DF – Brazil

Rio Madeira Project

Hydraulic and Sediment Management Studies

Daft Report

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January 2007

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PowerPoint Presentation : Rio Madreira – Santo Antônio Hydro Project, January 12, 2007.

PowerPoint Presentation: Site Visit at Porto Velho December 15, 16 & 17, 2006.

Introduction

Purpose of the report

The purpose of the report is to assess the sediment management aspects of the AHE Santo Antônio Low Head Hydro Project as designed by PCE – FURNAS and ODEBRCHT, draw conclusions and make adequate recommendations to achieve the required objectives.

The main concern of the Engineers of the Ministry of Mines and Energy (MME) is to be sure that the proposed project structural arrangements would ensure adequate sediment management in the reservoir in particular the following points shall be assessed:

A. The sediment management plan for the reservoir and the adequacy of the hydraulic structures designed to address this issue satisfactorily while guaranteeing reliable operation of the power plant.
B. Pay special attention on the quantity and mineral composition of the sediments carried by the Madeira River.
C. The process of sediment transit through the reservoir, the impact that retained sediments may have on the flooded area, possible alterations in the backwater curves of reservoir during the life of the project on the operation and maintenance and the economic life of the power plants.

These review items imply that it will be necessary to analyse:

- The operating conditions of the individual structures such as the power house and the spillway and their impact on the bedload transport patterns.
- The details of the sediment transport mechanism along the entire 125 km of the river length between Girao and Sant Antônio for the various representative river discharges between 5,000m^3/s and 84,000m^3/s.
- Apart from detecting the areas of sedimentation and their eventual effect on water flow profiles the review should comprise checking the sand transit patterns at the power house and the spillway. This would imply that the project layout and the structural arrangements of the power station and the spillway should also be reviewed in an attempt to reduce as much as possible intrusion of sands and fine gravels into the bulb turbine intakes.

General description of the project

The Santo Antônio low head hydro project is located in the rapids of the same name on Rio Madeira just upstream of the city of Porto Velho, in the State of Rondônia. It would have a power plant with an installed capacity of 3,150 MW comprising 44 Bulb Units, total plant discharge 24,000 m^3/s.The project would also have a spillway equipped with 21 Radial Gates 21.83 m high and 20 m wide maximum discharge capacity 84,000 m/s. The normal operating pool level is 70.00 m with exceptional pool Level of 72.00 m for the maximum flood discharge of 84,000 m^3/s. For low water period the upper pool extends about 125 km up to Girau water falls. Beyond the flood discharge of 39, 100m^3/s the maximum pool levels are the same as that of the natural flood water levels from about 60 km upstream of the Santo Antônio project.

Site visit and main findings

A site visit was organized between December 15 and 17, 2006 by the Engineers of the Ministry of Mines and Energy (MME). The team included Dr. John Denys Cadman, Consultant MME, and Mrs. Jennifer Sara Regional coordinator of the World Bank and S. Alam, Consultant. The weather in the project area at Porto Velho was good and we were able to see the project site and the river immediately downstream of the site and upstream up to the Teotonio rapids 17 km from the site. We are thankful to “Electro-Norte” specially Mr. Lima for having provided us with the technical and material assistance without which it would have been impossible to achieve what we accomplished.

Main findings:

- The river discharge was about10, 000 m^3/s, rather low for this time of the year.
- The river water level upstream and downstream of the Saint Antônio rapids should have been respectively 50.50m and 49.50 m. according to the rating curves (water level gauge reading adjacent to the pumping station indicated the water level at ----).
- Attempts were made to collect river bed material samples at 20 different locations. Due to the mal functioning of the Grab-Sampler and also due to the presence of bed rock only 6 samples were collected. (Ref: Fig ---). Visual examination and by rubbing between the fingers indicated that most of the samples contained very fine to fine sands (0.062 mm to 0.50 mm) with a slight trace of coarse silt. Only the sample No. 19 contained very coarse sand to fine gravel (1.00 mm to 10.00 mm). The shape of the courser sand and majority of the fine gravel particles seemed partially rounded, only the very large ones were sharp angular most probably coming from the Tetonia rapids. An important fraction of the sample seemed to be of quartzite origin (Photo 7).
- We saw some minor local bank cave ins (Photo 10) on both banks indicating that some amount of alluvial sediment materials was being added all along the river.
- The sample 19 collected at the underwater point bar confirms that under the existing river channel layout and annual maximum flood flows (about 45,000 m^3/s) the maximum representative particle sizes that is being transported in this part of the river is probably between 4.00 to 5.00 mm (Photo 7) and almost all of these coarse sands and fine gravels are being entrained strictly as bed load, i.e., they are either moving in suspension very close to the river bed or in saltation (intermittent hopping movement along the river bed) or in entrainment (creeping along the river bed without lifting from the bed). Also they are transiting only along the right bank of the river bend immediately up stream of the project axis.
- This confirms that the spillway location at the right bank is a good choice. However, the project layout and bed and bank excavations, and increase in water depths might modify significantly the flow velocity distributions and secondary current patterns in this area. This might eventually influence the direction of the bed load movement. Only a properly designed hydraulic scale model would enable us to determine with certainty the bed load movement patterns with the power plant and the spillway in operation and that the bulk of the bed load would indeed pass through the spillway.
- During our travel on the river both downstream and upstream we were impressed by the quantity and size of the floating debris transiting at the water surface. The trees were 5m to 10m long with diameters around 0.3m to 0.5m and it may be assumed that the submerged debris could be equally big (According to the information available, at Sidney A. Murray hydro station off the Mississippi River submerged debris is about 20 to 30% . Indeed the reckless flat bottom boat pilot broke 4 times the propeller coupling pin by trying to speed through the debris and finally the 45 HP outboard motor of the boat. Fortunately we were near the bank and away from the rapids so we were able to get ashore and returned to the boat station with another boat without any harm.
- So if we consider the length of the power plant is about 1,050 m, the Spillway 500 m and the rock fill dam 900 m long the debris would have plenty of areas to accumulate and conventional trash rack rakes which are designed for much lighter debris will not be able to handle the kind of debris transported by Rio Madeira.[3)]. I do not know the type debris handling equipment has been foreseen at Santo Antônio but based on my experience at Sidney A. Murray Hydro Station off the Mississippi River in Louisiana I can say that it has to be a site specific design.

Conclusions

The proposed project arrangement: Power house at the left bank or middle of the river and Spillway on the right bank would fit in well with the existing sediment transport pattern immediately upstream of the project site. However, the proposed project layout along with the required bed and bank excavations, and water depth increase might modify the flow velocities and especially the secondary currents influencing the bed load movement patterns. The impact of such modifications in conjunction with the river flows and the project operation modes would have to be analyzed preferably by using a hydraulic scale model.

Based on the observations of the river and the project site we can conclude that if not properly managed the floating and submerged debris may become a source of operational difficulties immediately after the project is put into service. Along with the issue of sediment management the debris handling should also get equal attention.

Conclusions and recommendations

Compared to the actual maximum sediment concentration measured (3,500 PPM future concentrations will be much more (10,000 to 20,000 PPM) due to the sand accumulation during the low flows up to 18,000 m^3/s and the annual flushing out during the high flows 30,000 m^3/s or more. The shape of the annual hydrograph is such that at least 4 months of discharge at 30,000 m^3/s or more is guaranteed.

Segregation and accumulation of coarse sands and fine gravels have been observed about 2,000 m upstream of the project are care must be taken in selecting the structural locations so that these sands and fine gravels are evacuated through the spillways. The quartz content of the bedrock material is 40% we do not know yet the exact mineralogical composition of the sample collected during the site visit.

No appreciable impact on the backwater curves are anticipated because the flow through velocities and resulting sand transport capacity in suspension within the pool are sufficient to ensure against massive deposits of sand in the pool impacting backwater or plant operation. This project like all well designed run-of-river projects should perform normally and have a long life.

Floating and submerged debris could create serious operational difficulties. Debris removal equipment should be adapted to the site. Special conception and design assuring required performance should be sought

Important changes in the project layout and concept is possible and a review enabling improvement of project concept, cost savings and construction time reduction is strongly recommended

A state-of-the-art physical scale model should be used to optimize

- **The project layout concept assuring proper sand transit patterns**
- **The handling of the floating and submerged debris against formation of log jams**
- **Prevention of formation of stable air entraining vortices**
- **The performance of the hydraulic design of the structures**

1. General review of the river hydrology and sediment transport data.

1.1 <u>Annual river discharge hydrographs, flow duration curve and sediment discharge data.</u>
Figures 7.23, 7.24 and 7.25 show the annual hydrographs of Rio Madeira at Guajara-Mirim, Abunã, Abunã – Guajara - Mirim and Porto Velho for the years 1982, 1984 and 1986. The river discharge generally increases from October November to April May and goes down between April May and October November. Figure 7.35 shows the average monthly flow duration curve. The maximum plant discharge is 24,000m^3/s (?) is exceeded 30% of the time and the maximum annual flood discharge is 45,000m^3/s (?). (Maximum recorded daily discharge is 48,570 m^3/s, occurred on 14 April 1984)
Total annual sediment discharge of Rio Madeira at its confluence with the Amazon is estimated at 500 Million tons per year by Robert Meade of USGS (Figure 7.70).
Maximum suspended sediment concentration measured at Porto Velho by FURNAS is 3,500 PPM or 3,500 mg/l and the corresponding river discharge was 30,000m^3/s. This is probably the representative discharge around which the rate of rise in the water level is the fastest, producing the steepest water surface slope causing a sudden flux in the sediment concentration. [3)] The maximum daily total sediment load measured at that time on 16/02/2004 was 9,210,329 tons and the corresponding suspended load was 8,889,566 tons (ref: Table 7.69). On the average the bed load is about 6% of the total sediment load (ref: Table 7.74).

The average composition of the suspended load in Rio Madeira at Santo Antônio is shown in the following table (ref: Table 7.75):

Clay	Silt	Sand
26.5	63.7	9.8

The average composition of the river bed material is shown in the following table (ref: Table 7.76):

Clay	Silt	Sand
1.2	7.8	91.0

The report concludes that representative total sediment composition at Porto Velho would be as follows (ref: Table 7.77):

Sediment material	% of Clay	% of Silt	% of Sand
In suspension	25.0	60.1	9.3
Bed load	0.1	0.4	5.2
Total	25.0	60.6	14.4*

* Rounded at 15% for the purpose of evaluations in this report.

The bed material samples collected during the site visit confirm qualitatively some of the particle size distribution curves shown in the Figure 4.17 of the PCE – FURNAS – ODEBRECHT Reports where the characteristic particle size distributions found at three distinct locations are as shown in following tables:

TARUMÃ

% <	10	30	50	60	90	100
d (mm)	0.15 - 0.20	0.20 – 0.31	0.22 – 0.39	0.25 – 0.40	0.35 – 0.82	0.50 - 2.00
Av. d (mm)	0.17	0.25	0.30	0.32	0.58	1.25

CAMALEÃO

% <	10	30	50	60	90	100
d (mm)	0.18 – 0.22	0.25 – 0.35	0.38 – 0.46	0.38 – 0.52	0.70 – 1.20	3.00 – 5.00
Av. d (mm)	0.20	0.30	0.42	0.45	0.95	4.00

PAULINO

% <	10	30	50	60	90	100
d (mm)	0.38 – 0.42	0.52 – 0.62	0.77 -1.30	0.92 -1.50	2.20 – 2.70	4.00 – 5.00
Av. d (mm)	0.40	0.57	1.03	1.21	2.45	4.50

Coarser sediment particles found at Camaleão Island is of lighter material (probably not granite) as can be seen on the Photo No. --- Sediment samples collected at Paolino is located at the interior of a semi circular bend about 2,000m upstream of the project axis and it contains a well segregated sample of coarse sand and fine gravels this in our opinion is a segregated accumulation of coarser sand and fine gravels fraction and not representative of the average sand load of Rio Madeira. Understanding of the particle segregation process and the migratin of coarser sands in this area would be of interest in determining the layout of the spillway and the power station.

Bulk of the sand load is less than 1.00 mm diameter and depending on the river reaches particle sizes up to 2 – 3 mm may also be transported in suspension or in saltation during the peak annual flood discharges of 40,000 to 45,000m^3/s with the existing river channel conditions.

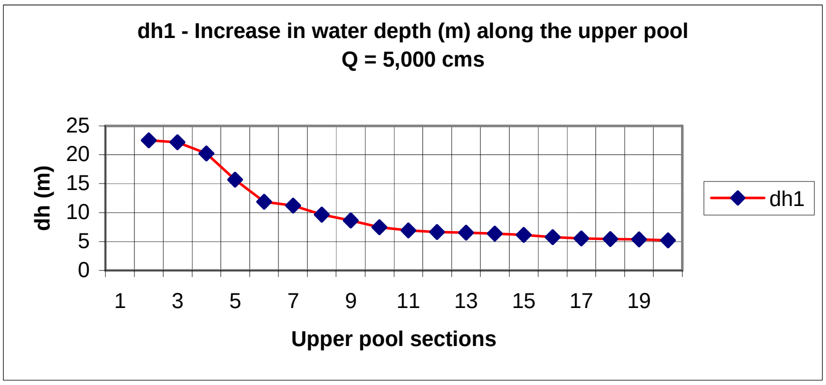
Considering that about 15% of the total suspended sediment discharge is sand (ref: Table 7.77), the total annual sand load could therefore be: 0.15 x 500,000,000 tons = 75,000,000 tons. Of this 95%, i.e., 71,250,000 tons could be between 0.10 to 1.00 mm and 5%,i.e., 3,750,000 tons could be between 1 to 3 mm (ref: Figure 7.17).

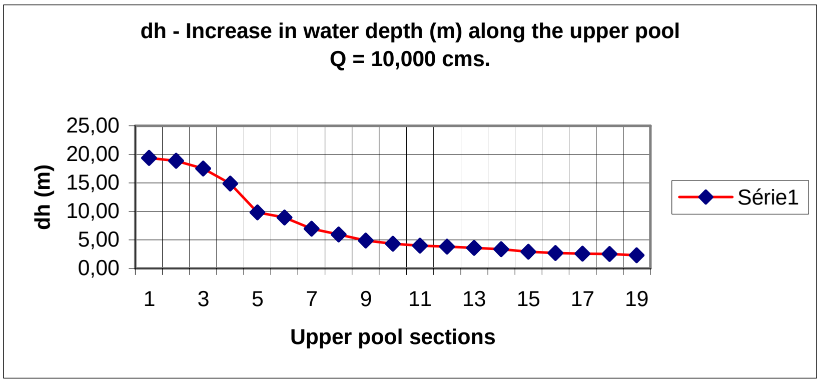
After the construction of the dam the sediment transport conditions would be modified significantly over the entire pool length for smaller discharges (5,000 to 10,000m^3/s) and over about 48% of the pool length created by the storage for higher discharges. The present review should enable us to determine how the sand particles up to 1.00 mm and those between 1.00 to 3.00 mm are going to move along the river between Girau and Santo Antônio with the storage level at 70.00m and for the various annual river discharge conditions.

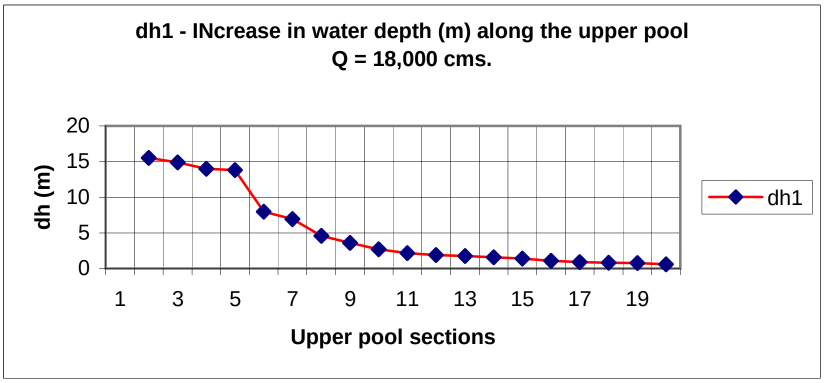
1.2 Increase in water depth along the upper pool for various discharges for a pool elevation at 70.00 m.

The river bed between Santo Antônio and Girao is not a uniform alluvial sandy bed it has several rock outcrops in the shape of islands, sills and rapids (we have not visited the entire length of the upper pool). The longitudinal water surface slope is not a continuous one as a consequence the comparative increase in water depths due to the construction of the Santo Antônio hydro project would be limited towards downstream end of the pool over about 48% of its total length, decreasing gradually with increasing discharges, greater than 39,000m^3/s.

The Figures 1.2.1 and Figures 1.2.2 show the depth variations for the river discharges of: 5,000; 10,000; 18,000; 39,100; 48,600; 61,200; 72,600 and 84,000m^3/s.


dh1 - Increase in water depth (m) along the upper pool
Q = 5,000 cms
dh (m)
25
20
15
10
5
0
1
3
5
7
9
11
13
15
17
19
Upper pool sections
dh1


dh - Increase in water depth (m) along the upper pool
Q = 10,000 cms.
dh (m)
25,00
20,00
15,00
10,00
5,00
0,00
1
3
5
7
9
11
13
15
17
19
Upper pool sections
Série1


dh1 - INcrease in water depth (m) along the upper pool
Q = 18,000 cms.
dh (m)
20
15
10
5
0
1
3
5
7
9
11
13
15
17
19
Upper pool sections
dh1

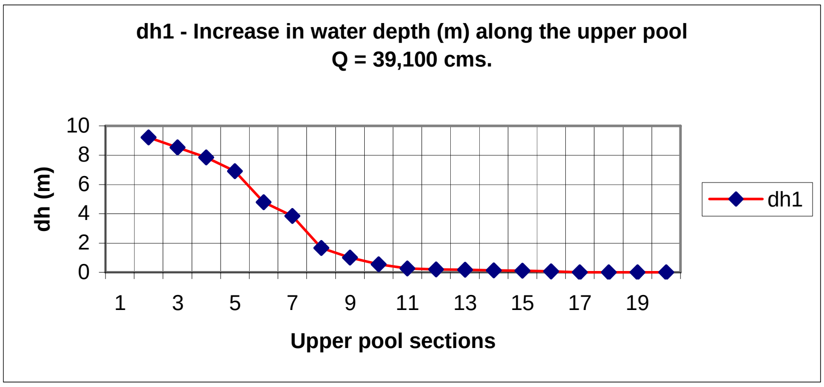
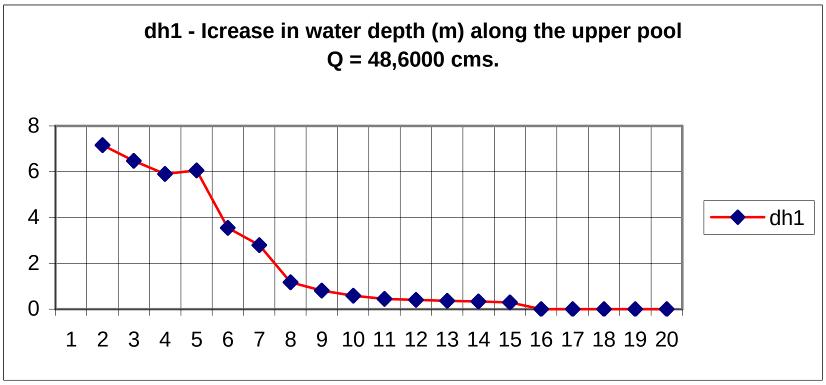
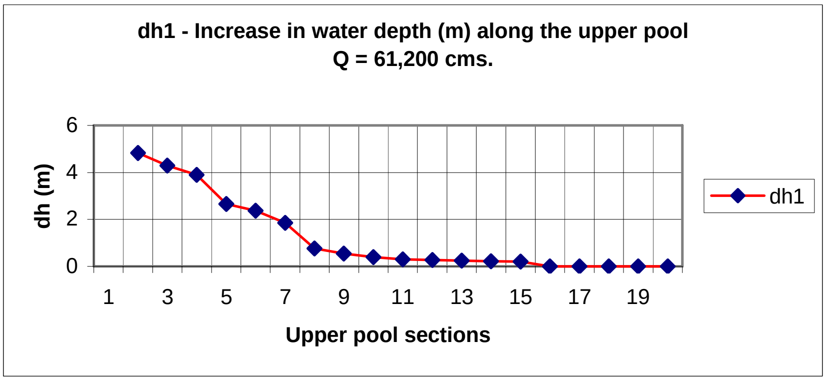
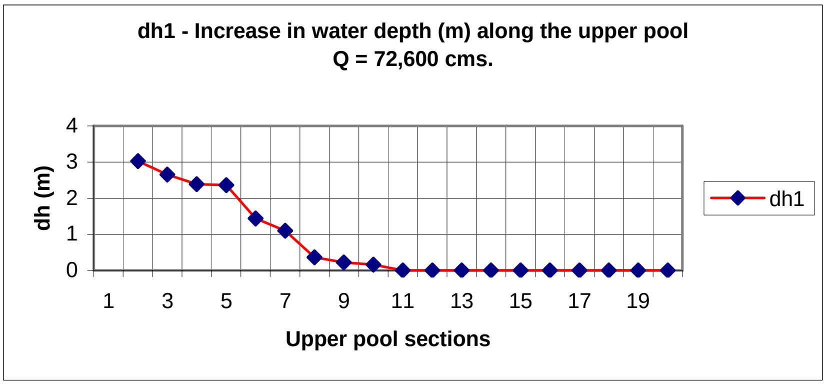

dh1 - Increase in water depth (m) along the upper pool
Q = 39,100 cms.
dh (m)
10
8
6
4
2
0
1
3
5
7
9
11
13
15
17
19
Upper pool sections
dh1

Figure – 1.2.1


dh1 - Icrease in water depth (m) along the upper pool
Q = 48,6000 cms.
8
6
4
2
0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
dh1


dh1 - Increase in water depth (m) along the upper pool
Q = 61,200 cms.
dh (m)
6
4
2
0
1 3 5 7 9 11 13 15 17 19
Upper pool sections
dh1


dh1 - Increase in water depth (m) along the upper pool
Q = 72,600 cms.
dh (m)
4
3
2
1
0
1 3 5 7 9 11 13 15 17 19
Upper pool sections
dh1

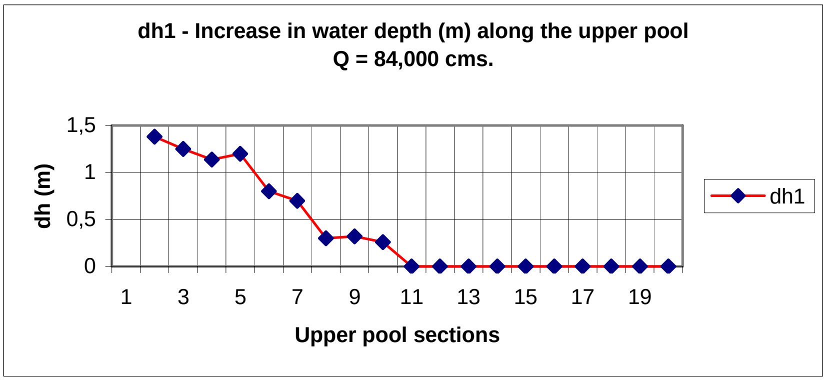

dh1 - Increase in water depth (m) along the upper pool
Q = 84,000 cms.
dh (m)
1,5
1
0,5
0
1 3 5 7 9 11 13 15 17 19
Upper pool sections
dh1

Figure – 1.2.2

It is observed that the maximum increase in depth at the project site is 22.49m for the river discharge of 5,000m^3/s and the minimum is 1.38m for the maximum project discharge of 84,000m^3/s.The Tables showing the details related to these figures are appended in APPENDIX - A1.

It is evident that with the increase in the river discharge the water levels upstream of section 10 approaches almost the natural water levels. For the annual flood discharges of 39,100m^3/s and 48,600m^3/s the increase in water depths are respectively only 1.66m and 1.18m. So annually some of the sediments which will be deposited in the pool during the low flows should start to move downstream during the high flows especially for the river discharge of 39,100m^3/s or more. The average suspended sediment concentrations at this time can be much higher than the maximum recorded (3,500 PPM), and may be as much as 10,000 to 20,000 PPM or 10,000 to 20,000 mg/l or 10 to 20 kg/m^3 (often observed in reservoirs full of sediments). The impact of such heavy sediment concentration of which an important fraction, more than 15% currently observed would be sand may pass through the turbines over a certain period of time unless the spillway which should be in operation at this discharge is capable of attracting the bulk of the heavier sediment concentrations through the spillway bays? An operating procedure to achieve this may eventually be developed with the help of an adequate hydraulic model study.

1.3 Average local flow velocities for various discharges along the upper pool

Average local flow velocity is a good indicator of the sediment transport capacity at that location. We have therefore compared the flow velocities with the pool at 70.00 m to those for the natural conditions and their differences. The Tables and Figures (1 to 8) summarizes the various cases.

Average local flow velocities Q = 5,000 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	0,15	0,30	0,15
S-6	0,19	0,73	0,54
S-7	0,11	0,45	0,34
S-8	0,25	1,18	0,93
S-9	0,19	0,44	0,25
S-10	0,27	0,58	0,31
S-11	0,24	0,44	0,20
S-12	0,14	0,25	0,11
S-13	0,25	0,40	0,15
S-14	0,20	0,30	0,10
S-15	0,17	0,25	0,08
S-16	0,17	0,21	0,04
S-17	0,20	0,29	0,09
S-18	0,26	0,35	0,09
S-19	0,17	0,25	0,08
S-20	0,22	0,31	0,10
S-21	0,16	0,19	0,03
S-22	0,23	0,28	0,05
S-23	0,17	0,20	0,03

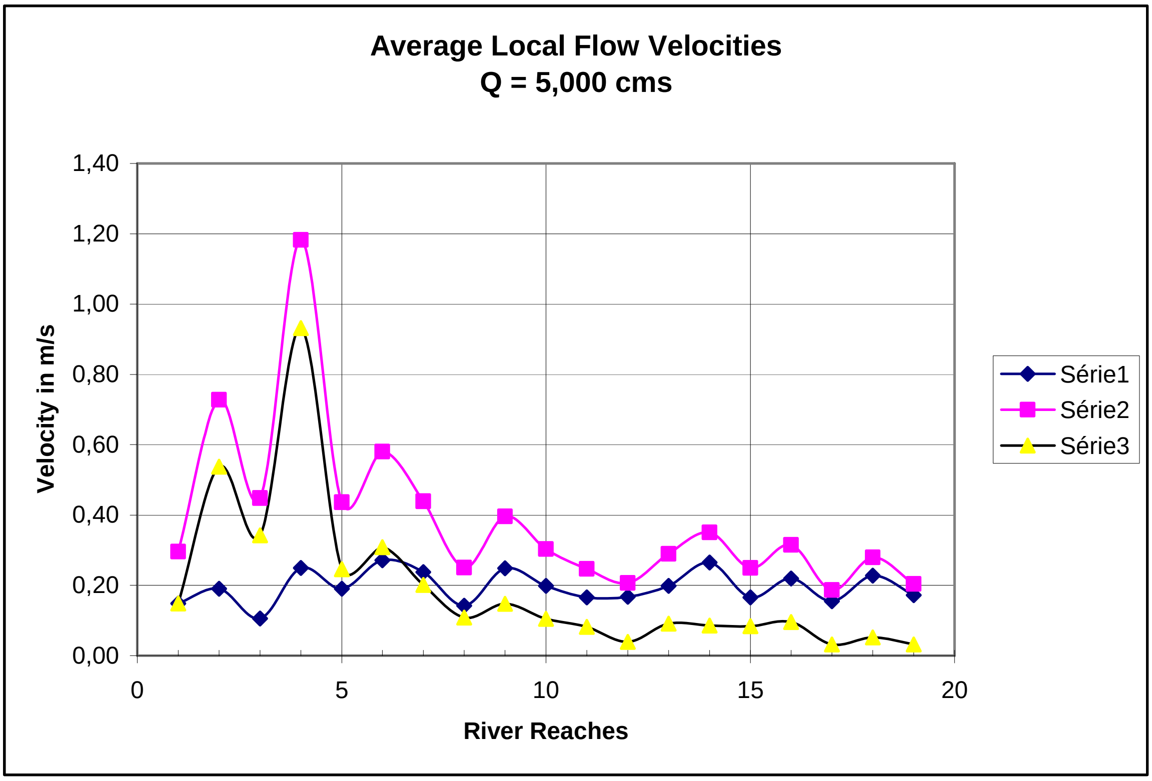


Table & Figure - 1

Average local flow velocities Q =10,000 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	0,30	0,52	0,22
S-6	0,38	1,02	0,64
S-7	0,21	0,60	0,39
S-8	0,50	1,94	1,44
S-9	0,38	0,71	0,33
S-10	0,54	0,94	0,40
S-11	0,47	0,70	0,23
S-12	0,28	0,40	0,12
S-13	0,49	0,65	0,16
S-14	0,39	0,50	0,10
S-15	0,32	0,40	0,08
S-16	0,33	0,37	0,04
S-17	0,39	0,47	0,08
S-18	0,52	0,60	0,08
S-19	0,32	0,38	0,06
S-20	0,42	0,49	0,07
S-21	0,30	0,33	0,03
S-22	0,45	0,49	0,04
S-23	0,34	0,36	0,02

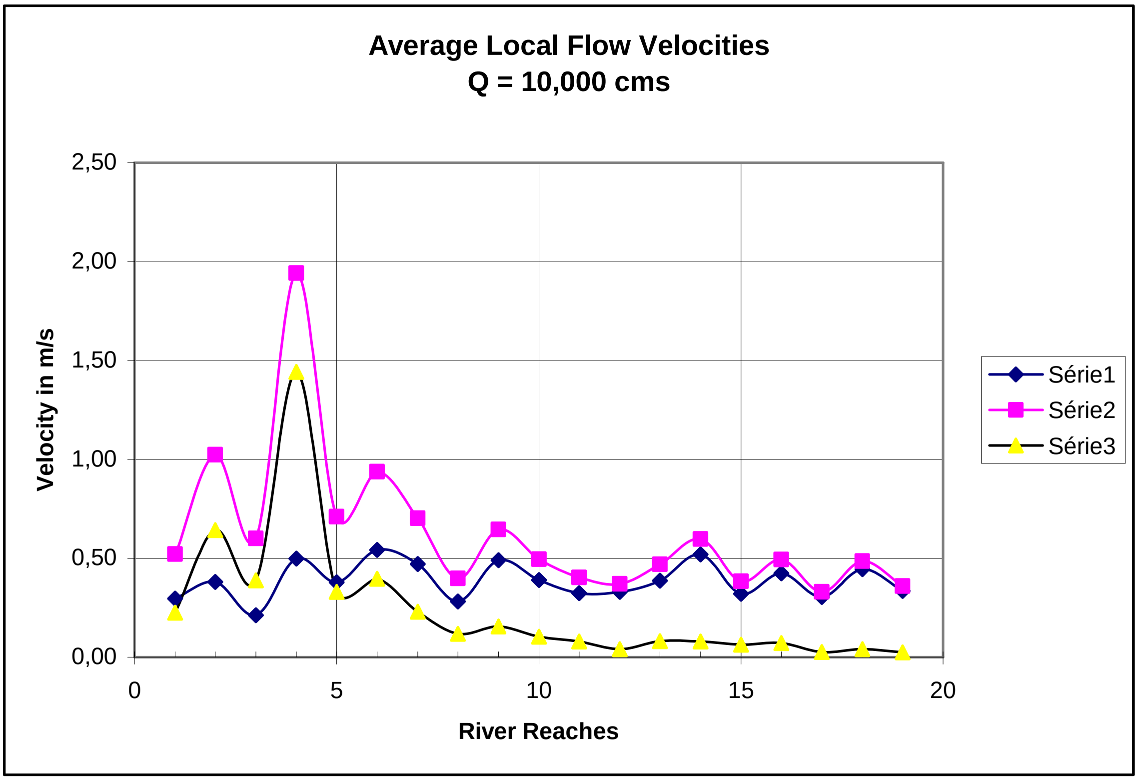


Table & Figure -2

Average local flow velocities Q =18,000 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	0,53	0,81	0,28
S-6	0,68	1,36	0,67
S-7	0,38	0,78	0,40
S-8	0,90	2,86	1,96
S-9	0,68	1,09	0,41
S-10	0,97	1,43	0,47
S-11	0,83	1,06	0,22
S-12	0,49	0,59	0,10
S-13	0,85	0,97	0,12
S-14	0,67	0,75	0,08
S-15	0,56	0,61	0,05
S-16	0,58	0,61	0,03
S-17	0,66	0,71	0,05
S-18	0,89	0,94	0,05
S-19	0,54	0,57	0,03
S-20	0,71	0,74	0,03
S-21	0,52	0,54	0,01
S-22	0,76	0,78	0,02
S-23	0,58	0,59	0,01

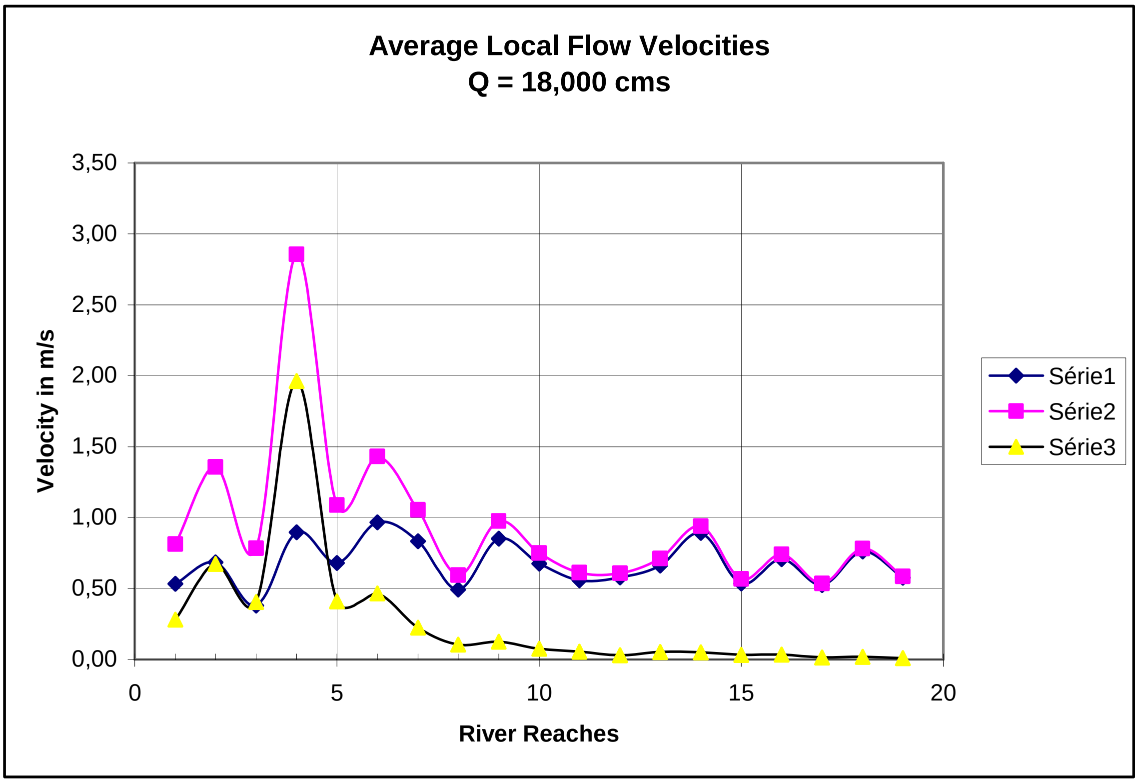


Table & Figure - 3

Average local flow velocities Q =39,100 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	1,16	1,46	0,30
S-6	1,48	2,06	0,58
S-7	0,82	1,15	0,33
S-8	1,91	2,90	0,99
S-9	1,43	1,82	0,40
S-10	2,01	2,44	0,42
S-11	1,66	1,79	0,13
S-12	0,96	1,00	0,04
S-13	1,63	1,67	0,04
S-14	1,29	1,30	0,01
S-15	1,05	1,06	0,01
S-16	1,15	1,15	0,01
S-17	1,23	1,24	0,01
S-18	1,70	1,71	0,01
S-19	0,96	0,96	0,00
S-20	1,27	1,27	0,00
S-21	1,01	1,01	0,00
S-22	1,45	1,45	0,00
S-23	1,10	1,10	0,00

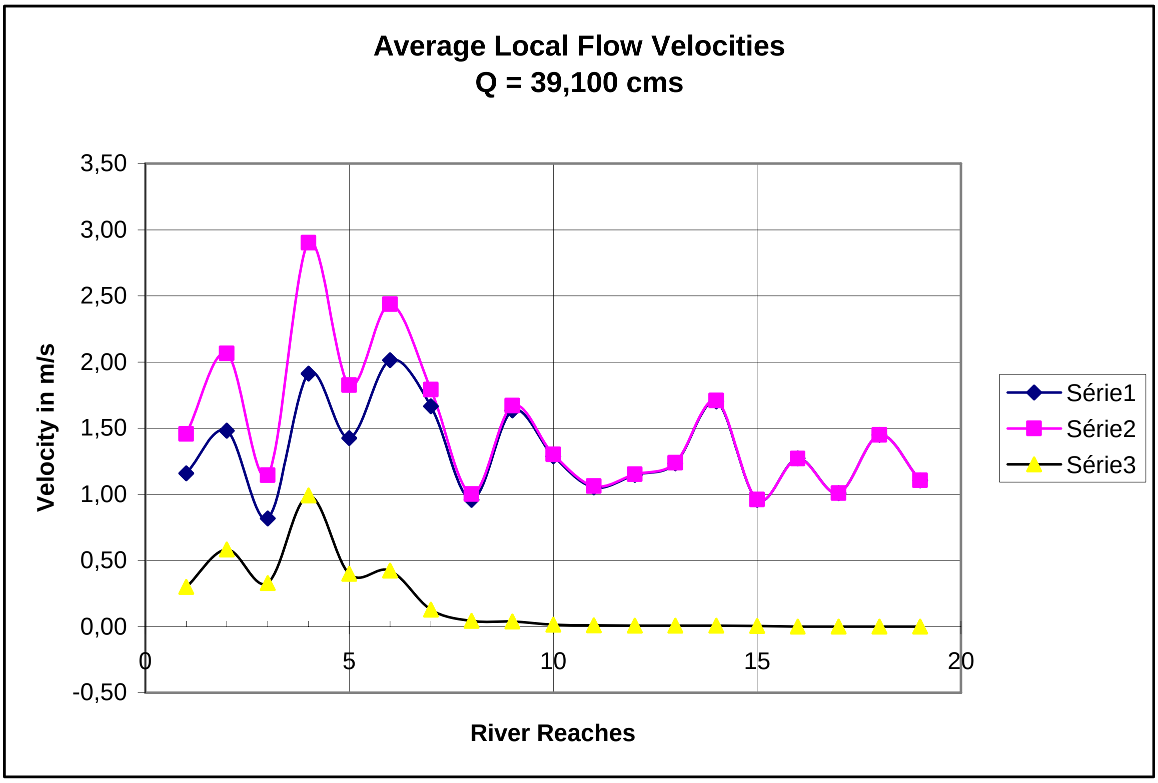


Table & Figure - 4

Average local flow velocities Q =48,600 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	1,44	1,71	0,27
S-6	1,84	2,34	0,50
S-7	1,01	1,29	0,28
S-8	2,35	3,32	0,97
S-9	1,75	2,08	0,33
S-10	2,45	2,80	0,35
S-11	1,98	2,08	0,10
S-12	1,12	1,16	0,04
S-13	1,90	1,95	0,04
S-14	1,49	1,52	0,03
S-15	1,22	1,24	0,02
S-16	1,36	1,38	0,01
S-17	1,43	1,45	0,02
S-18	2,00	2,02	0,02
S-19	1,11	1,11	0,00
S-20	1,48	1,48	0,00
S-21	1,20	1,20	0,00
S-22	1,72	1,72	0,00
S-23	1,32	1,32	0,00

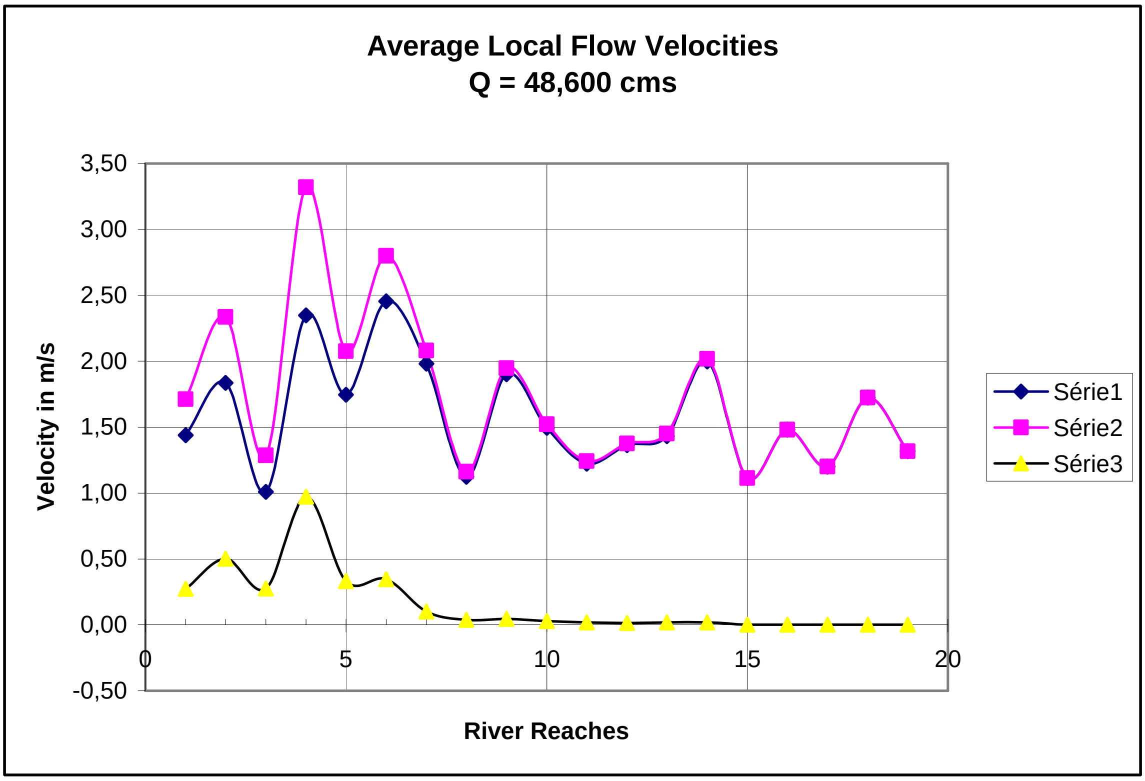


Table & Figure - 5

Average local flow velocities Q =61,200 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	1,81	2,03	0,22
S-6	2,30	2,68	0,38
S-7	1,26	1,46	0,21
S-8	2,84	3,24	0,40
S-9	2,14	2,39	0,25
S-10	3,00	3,26	0,26
S-11	2,39	2,46	0,07
S-12	1,34	1,37	0,03
S-13	2,27	2,30	0,03
S-14	1,79	1,81	0,02
S-15	1,46	1,47	0,01
S-16	1,66	1,67	0,01
S-17	1,71	1,72	0,01
S-18	2,40	2,42	0,01
S-19	1,32	1,32	0,00
S-20	1,75	1,75	0,00
S-21	1,45	1,45	0,00
S-22	2,08	2,08	0,00
S-23	1,59	1,59	0,00

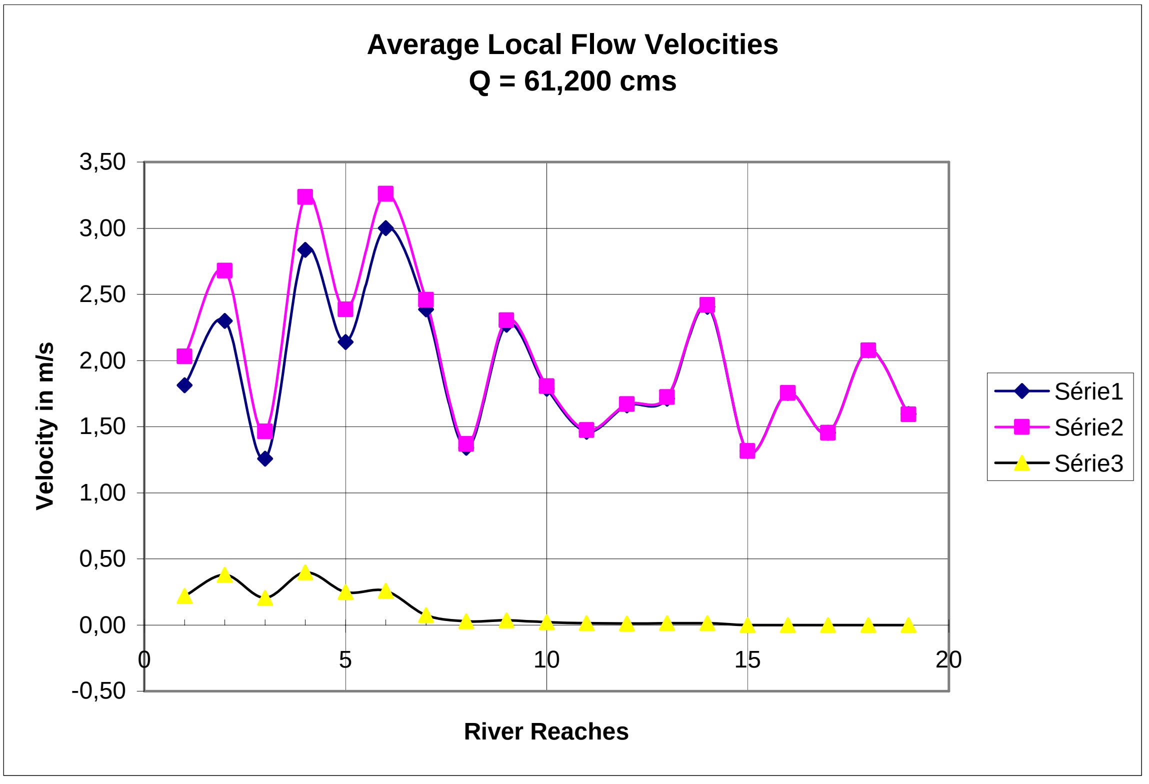


Table & Figure - 6

Average local flow velocities Q =72,600 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	2,15	2,31	0,15
S-6	2,71	2,97	0,26
S-7	1,48	1,61	0,14
S-8	3,33	3,75	0,42
S-9	2,47	2,63	0,16
S-10	3,46	3,63	0,17
S-11	2,70	2,77	0,07
S-12	1,53	1,54	0,01
S-13	2,58	2,60	0,01
S-14	2,04	2,04	0,00
S-15	1,67	1,67	0,00
S-16	1,92	1,92	0,00
S-17	1,95	1,95	0,00
S-18	2,76	2,76	0,00
S-19	1,48	1,48	0,00
S-20	1,98	1,98	0,00
S-21	1,67	1,67	0,00
S-22	2,38	2,38	0,00
S-23	1,83	1,83	0,00

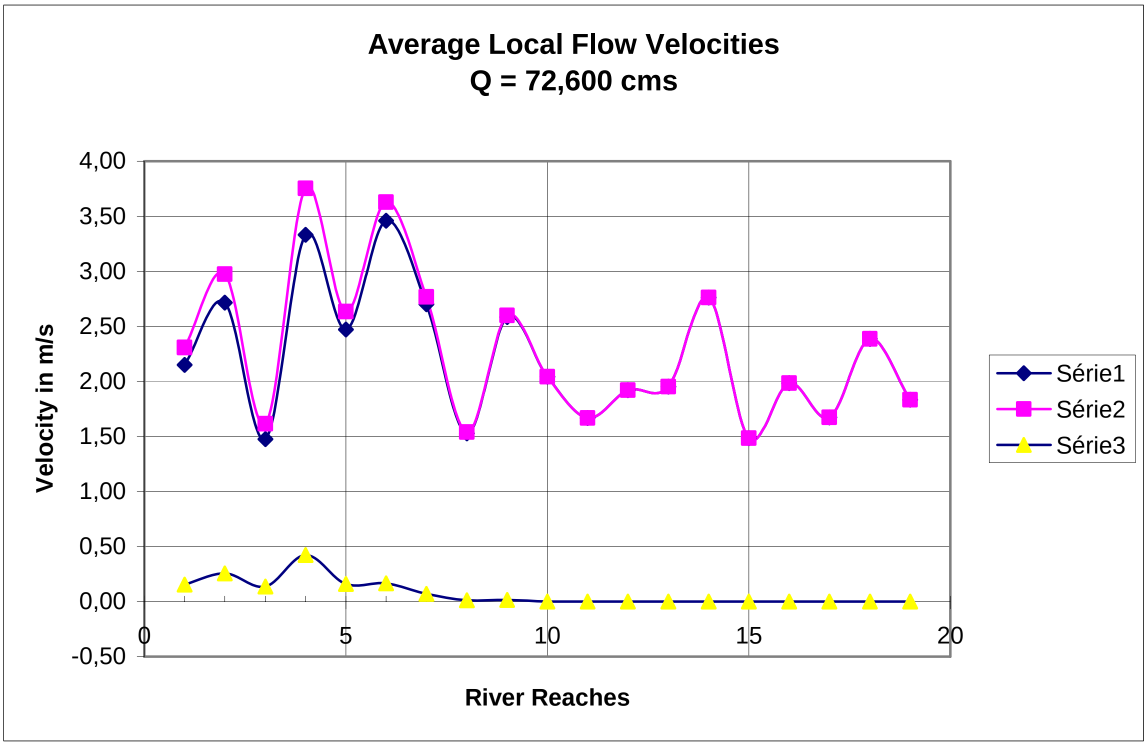


Table & Figure - 7

Average local flow velocities Q =84,000 m3/s			
Sections	With Pool	Nat. Cdn.	Dimunition
	V (m/s)	V (m/s)	(m/s)
S-5	2,49	2,57	0,08
S-6	3,12	3,25	0,13
S-7	1,69	1,76	0,07
S-8	3,81	4,03	0,22
S-9	2,78	2,87	0,09
S-10	3,88	3,99	0,11
S-11	3,04	3,07	0,03
S-12	1,69	1,71	0,02
S-13	2,87	2,90	0,03
S-14	2,28	2,28	0,00
S-15	1,86	1,86	0,00
S-16	2,17	2,17	0,00
S-17	2,18	2,18	0,00
S-18	3,10	3,10	0,00
S-19	1,65	1,65	0,00
S-20	2,21	2,21	0,00
S-21	1,88	1,88	0,00
S-22	2,68	2,68	0,00
S-23	2,06	2,06	0,00

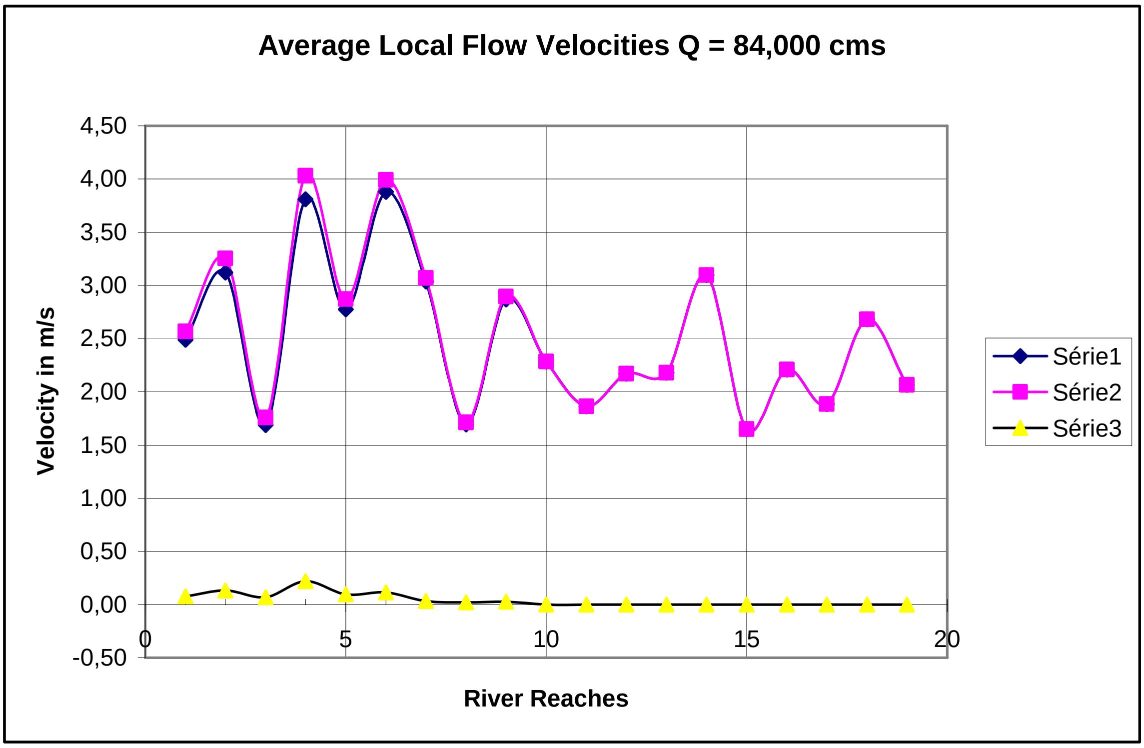


Table & Figure - 8

2. Analysis of sediment transport characteristics between Girao and Santo Antônio

The review of changes in water depths and flow velocities in the Rio Madeira between Girao and Santo Antônio indicate that only about 48% of the pool length towards its downstream end would be subjected to significant modifications in its capacity to transport sand in suspension for discharges less than 18,0000 m^3/s.

The total amount of sediment materials transported by the Rio Madeira is about 500 million tons (Figure 7.70.) of which about 15 % is composed of sands and fine gravels (ref: Table 7.77).

The Report has estimated long term upper pool sedimentation by using the empirical relationship developed by Brune in 1953. His curves, relating trap efficiency and the ratio between reservoir capacity and mean annual water inflow, (both in acre-feet) are shown in Figure 7.84. The report estimates that the initial trap efficiency of the Santo Antônio reservoir would be 19.50% and after 10 years the river bed near the dam would be silted up to Elevation 59.32m after 50 years the level would be 61.63m and after 100 years it will stabilize at 61.63m. We feel that this conclusion is too conservative, because the flow velocities in the approach areas of the powerhouse and spillway during the annual flood discharge of 40,000m^3/s over a period of month and a half or two would be high enough to remove the sands which might have accumulated during the low flow periods. This aspect has been evaluated in § 3 of this report. Also during the final verification of the project layout and structural dimensions it would be possible to assess the bed aggradations in the pool immediately upstream of the powerhouse intakes and the spillway. If necessary project structural arrangements producing minimum sediment deposition in this area could be developed on the model.

In large reservoirs it may be assumed that the trap efficiency will be 100%, i.e., all the sediments entering the reservoir will remain there.

In small reservoirs sometimes most of the inflowing sediments may be transported through the pool. This may also occur during high inflow periods when a reservoir discharges over the spillway and there is an appreciable velocity of flow through the reservoir. The proportion of the sediment passing through reservoir will depend primarily on two factors: the average velocity of the flow through the pool and character of the sediment. In respect to the latter, fine sediments (the silt and clay sizes) may remain in suspension long enough to pass through the reservoir. Sand sizes will not.

Preliminary verifications (ref: Tables & Figures 1 to 8) indicate that for Santo Antônio hydro project almost all the time the river flow velocities and the turbulence along the upper pool would be high enough to keep the silt and clay fraction of the sediments in suspension and would not settle unless in areas with completely stagnant waters.

During our site visit we noticed (Photos – 1, 2 and 3) that all sorts of vegetations: creepers, shrubs and saplings of some types of willow trees were growing on sands, probably the Island of Taruma. We have no idea as to how long it took for those plants to become 2-3m tall? With ponding it is possible that in certain areas (mostly in the areas of shoaling and along the banks with flow separation) intermittent sand depositions would facilitate the growth of this kind of vegetations. Long term impacts of such growths would cause some reduction in flow sections, increase in bank friction coefficient. Which may over long term period be cancelled by bank erosions. During our site visit we did notice bank cave-in on both banks (Photos - 8) and eroding sand Islands (Photo – 10).

As the morphology of the pool created by the Santo Antônio low head hydro project is mostly contained within its original river bed [1], it is not really a vast reservoir and the flow velocities after ponding all along the pool is fairly high and turbulent almost under all annual river discharge conditions.

1) To assess more precisely the flow velocities and sediment transport characteristics in the submerged areas of the pool it will be necessary to survey the real flow sections in within the pool following ponding.

As Santo Antônio is a low head run of river project the over all storage loss is not a significant parameter, particularly due to the presence of rock outcrops the sand deposition would grow in certain areas where we can see the presence of sand bars and Islands. S it appeared to us that instead of trying to determine the over all trap efficiency using the relationship developed by Brune it might be more representative to analyse sediment transport characteristics along the pool.

To assess the impact of the changes in the hydraulic parameters on the sand transport characteristics over the river length concerned 18 separate River Reaches (RR) were considered between sections 5 to 23 (ref: Figure 7.51) as follows:

RR 1 – Sections 6 to 5. RR 2 – Sections 7 to 6. RR 3 – Sections 8 to 7. RR 4 – Section 9 to 8. RR 5 – Section 10 to 9. RR 6 – Section 11 to 10. RR 7 – Section 12 to 11. RR 8 – Section 13 to 12. RR 9 – Sections 14 to 13. RR 10 – Sections 15 to 14. RR 11 – Sections 16 to 15. RR 12 – Sections 17 to 16. RR 13 – Sections 18 to 17. RR 14 – Sections 19 to 18. RR 15– Sections 20 to 19. RR 16 – Sections 21 to 20. RR 17 – Sections 22 to 21. RR 18 – Sections 23 to 22.

The suspended sediment load distribution graph developed by Hunter Rouse (ref: Sedimentation Engineering-ASCE -Manuals and reports on engineering practice No. 54) was used to assess the sand transport patterns along the reservoir.

Knowing the local shear velocity u* which is function of $(gdi)^{0.5}$
Where: g -The gravitational acceleration;
d - The flow depth
i - The water surface slope or energy gradient of the river flow.
And the fall velocity w of a given sand particle size, it is possible to determine the ratio w/u*, which in turn defines the vertical distribution of the given sand particle moving along the turbulent flow.

The following procedures were used for determining the flow depth and the water surface slope: For a given reach the flow depth d was the one located at the upstream end, and the water surface slope was obtained by dividing the difference in water depths upstream and downstream of the reach by the length of the reach [2].

2 It appears that water levels indicated at section 8 for both natural conditions and with storage is influenced by the high velocity flows over the Teotônio Rapids (ref: Table 7.54 and Table 7.60) creating under estimation of water surface slopes (some times negative) between sections 8 and 7 which impacts the capacity of sand transport locally as often apparent in the values of w/u*.

The figure 2.1 shows the relative distribution of the suspended load developed by Rouse. For w/u* = 0.06 the distribution is almost vertical over the total flow depth, and for w/u* = 2 the particle is still in suspension but only over 30% of the flow depth and for w/u* = 4 we have assumed that the particle is almost inert.

The computations presented in this report are approximate because the data related to the effective river and pool widths, hydraulic depths and water surface slopes derived from information in the report and used in computations both for the existing conditions and with storage are somewhat approximate (ref: Tables: 7.54; 7.60. and Figures: 7.55; 7.56; 7.57 and 7.58). However, the overall results may still be considered valid for the present review purposes. Eventually these computations may be updated with more precise data on: channel widths, flow depths, flow sections at each reference sections and water surface slopes between sections.

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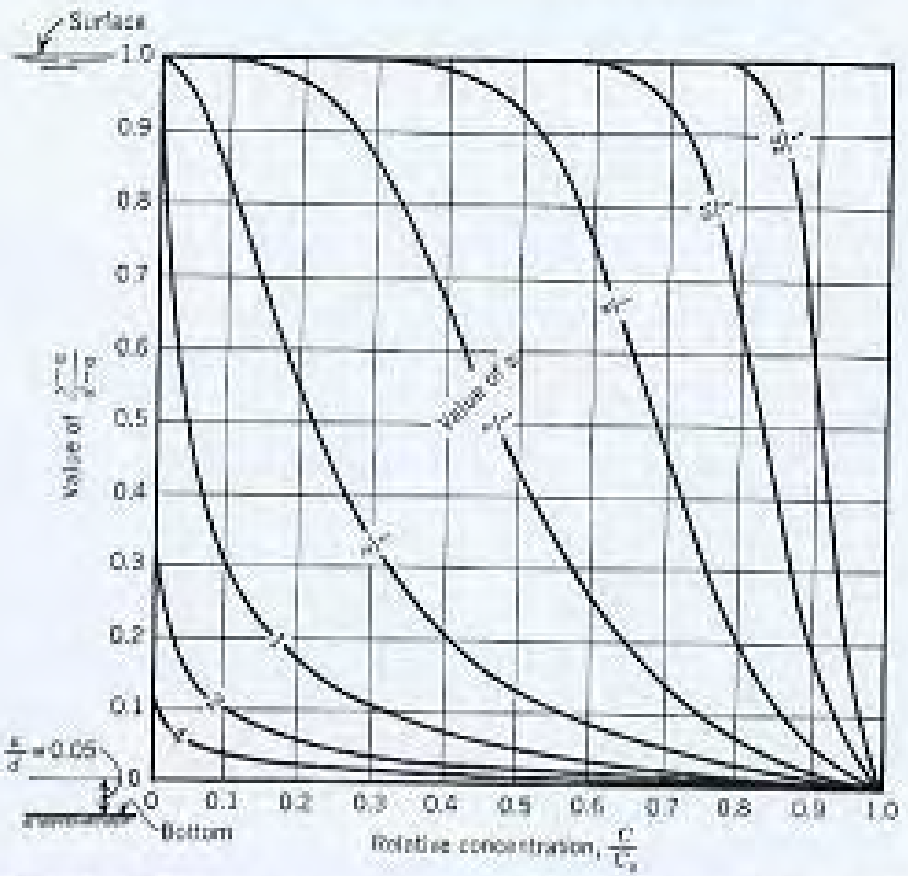


FIG. 2.32.—Graph of Rouse Suspended Load Distribution Equation for $a/d = 0.05$ and Several Values of z

Figure 2.1 –Graph of Rouse Suspended Load Distribution Equation for a/d 0.05 And several values of z

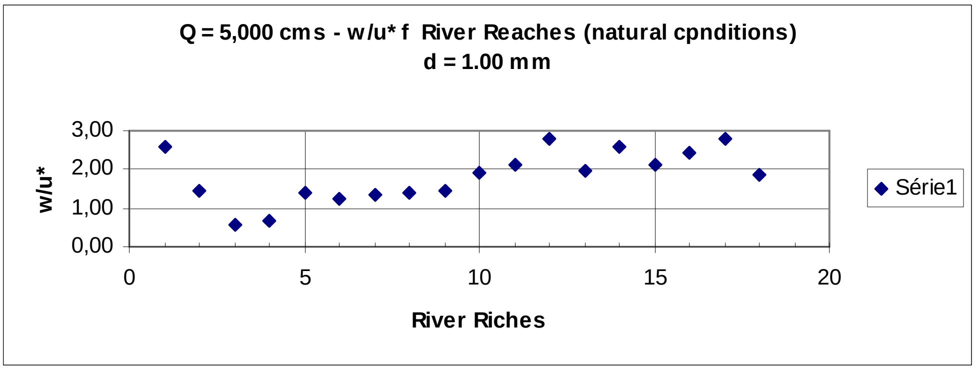
2.1 Sediment transport characteristics under natural conditions for the following river discharges: 5,000; 10,000; 18,000; 39,100; 48,600; 61,200; 72,600 and 84,000 m3/s

The Tables showing the values of w/u* at each river reach and for various sand particle sizes and discharges with the natural river conditions are shown in Appendices I, II, III, IV, V, VI , VII, VIII.

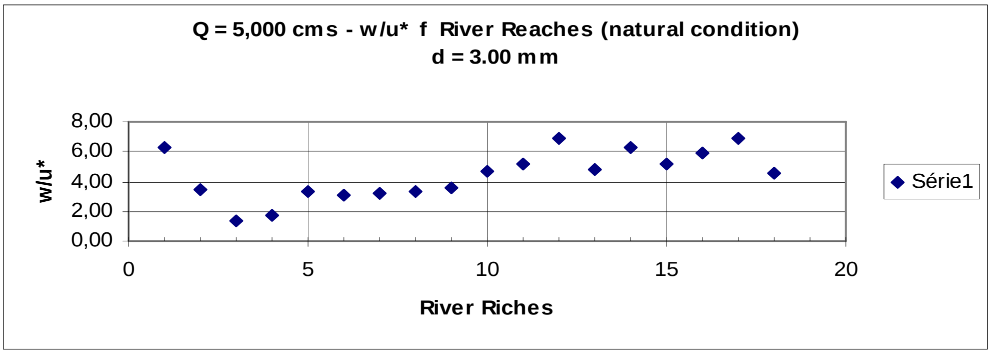
Analyses of transport characteristics for two critical particle sizes 1.00 mm and 3.00 mm and for each reference discharges are given below. The appended Tables give the sand transport characteristics for other particle sizes

I – Q = 5,000 m³/s

1.00 mm particles are transported mostly in suspension downstream of RR 10 and in saltation between RR 11 and RR 18.

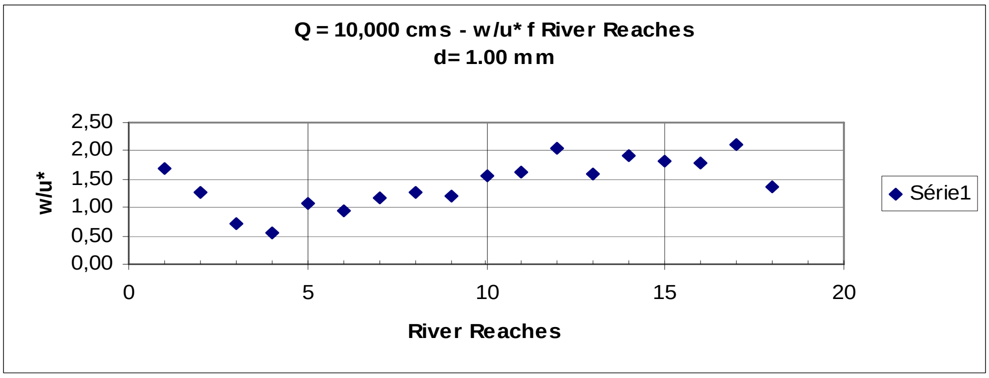


3.00 mm diameter particles are transported in saltation downstream of RR 10 (with the exception of RR 1). For the River Reaches upstream of RR 10, all 3.00mm diameter particles are completely inert.

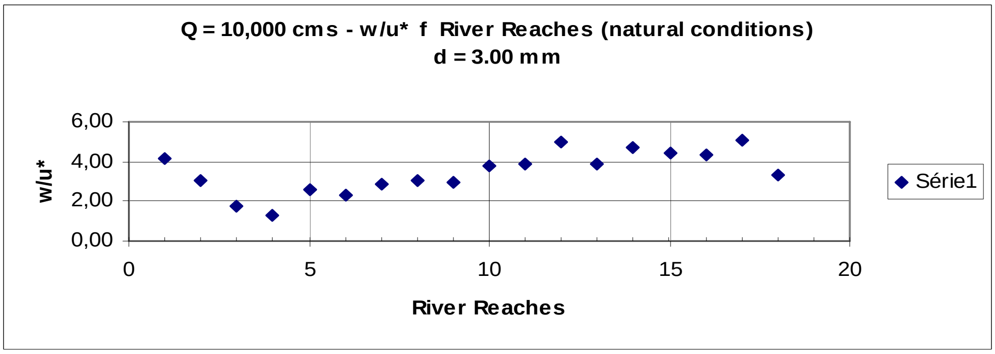


II – Q = 10,000 m³/s

1.00mm particles are transported in suspension over the entire river length between RR 1 and 18.

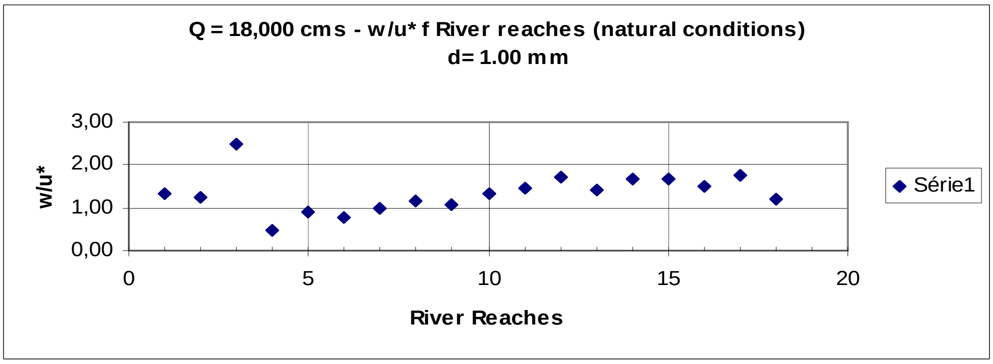


3.00 mm particles between RR 1 and 10 are moving in saltation and suspension, and beyond RR 10 they are inert. At the last RR 18 there is some movement in saltation.



III – Q = 18,000m³/s

1.0 mm particles are transported in suspension over the entire river length between RR 1 and RR 18.



3.00 mm particles are moving in suspension and saltation over the entire length between sections RR 1 to RR 18. Excepting at RR 3 where they are inert.
However, we think that the gauge 8 located just downstream of the Teotõnio Rapids (Photo---) is

strongly affected by the drawdown due to the local velocity head, at least in the order of 1.0 m and is causing the anomaly. This is evident from the gauge readings which are constantly low or even less than the readings at the next downstream gauge 7 (ref: Table 7.54). If we increase the water level by 1m, the value of w/u* becomes 2.18 instead of 6.03. Therefore the values of w/u* in all Tables for RR 3 could at times be much less especially at low river discharges.

Figure corresponding to water level in Table 7.54

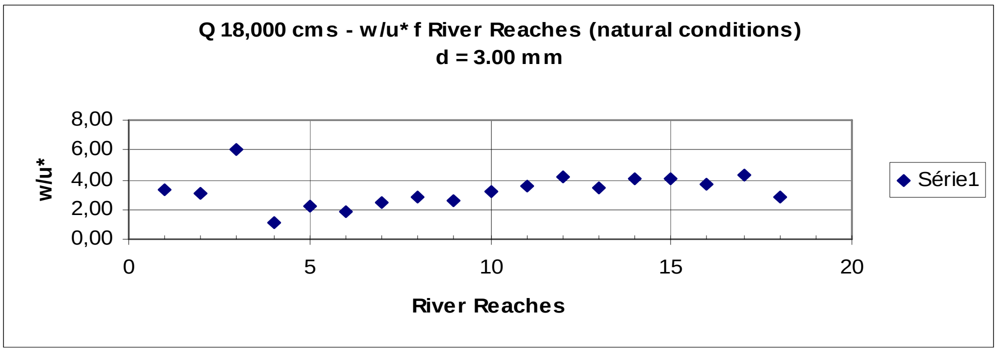
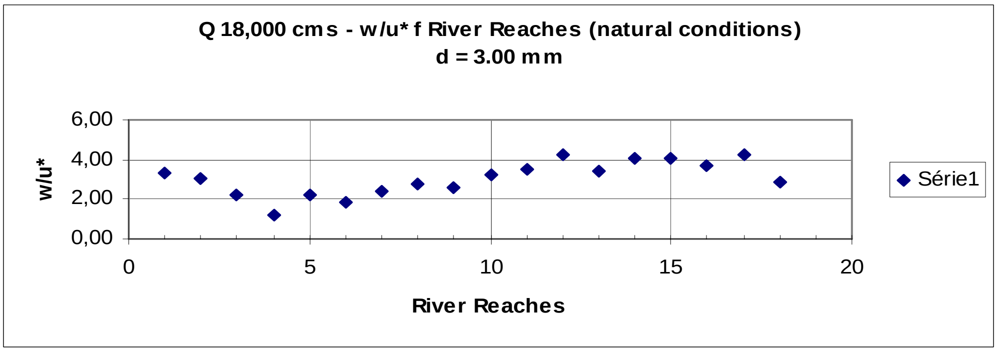
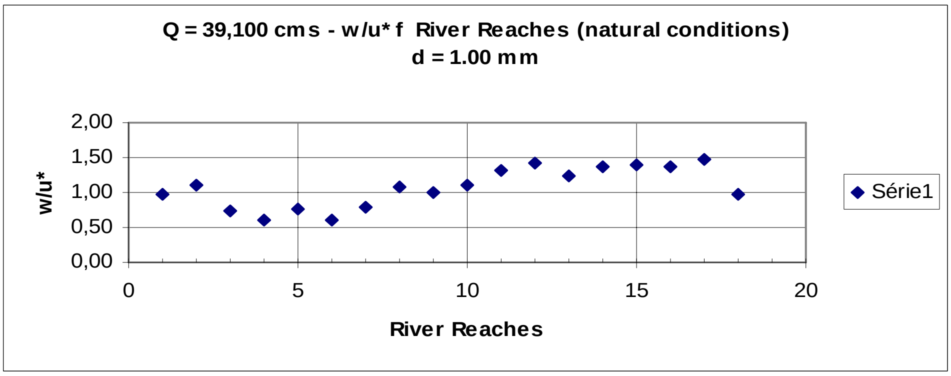


Figure with water level in Table 7.54 + 1.0m

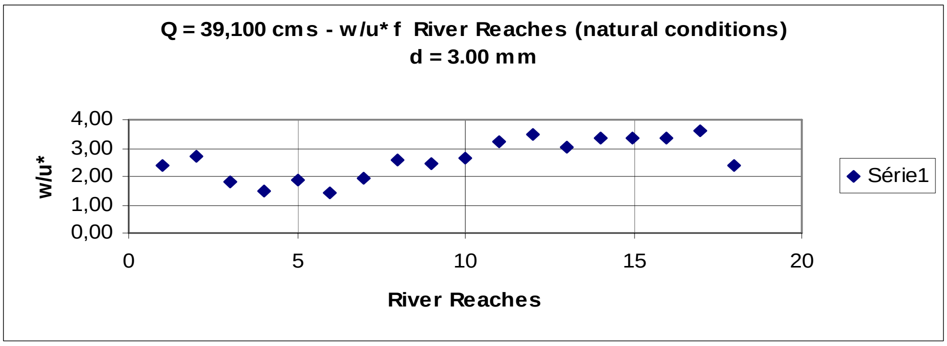


IV – Q = 39,100m^3/s

1.00mm particles are transported in suspension over the entire river length between RR 1 and RR 18.

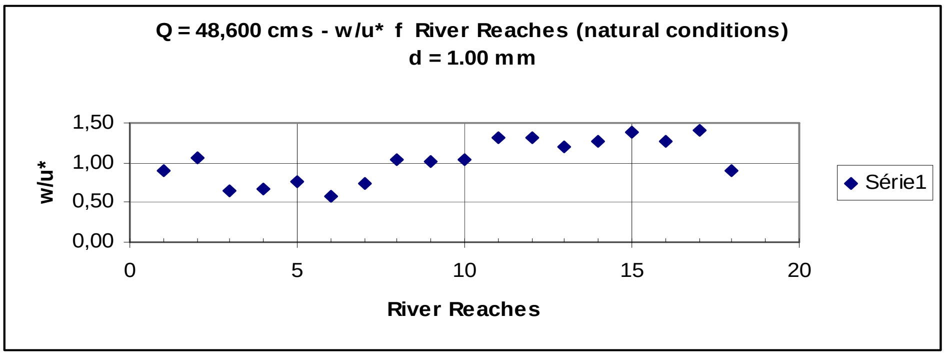


3.00 mm particles are being transported in saltation in all River Reaches excepting between RR 3 and RR 7 where they are moving in suspension.

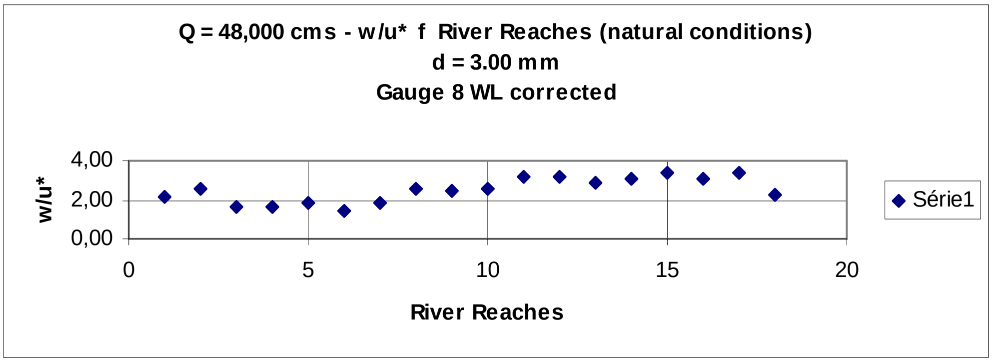


V – Q = 48,600m^3/s

1.00mm particles are transported in suspension over the entire river length between RR 1 and RR 18

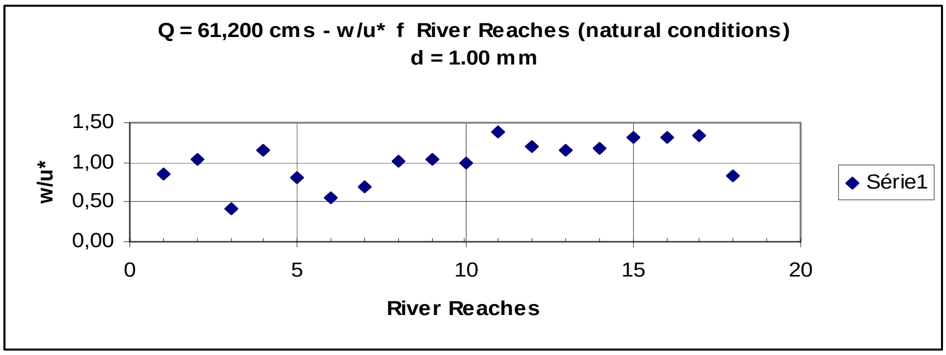


3.00 mm particles are being transported in suspension up to RR 7 and then in saltation to RR 18.

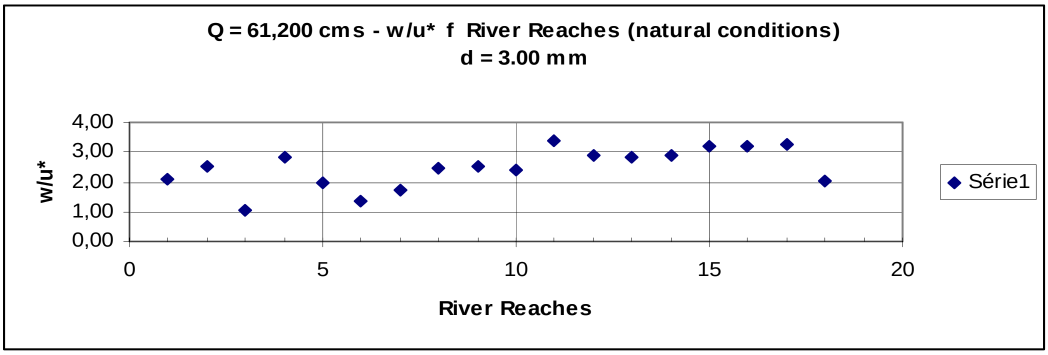


VI – Q = 61,200m^3/s

1.00mm particles are transported in suspension over the entire river length between RR 1 and RR 18.

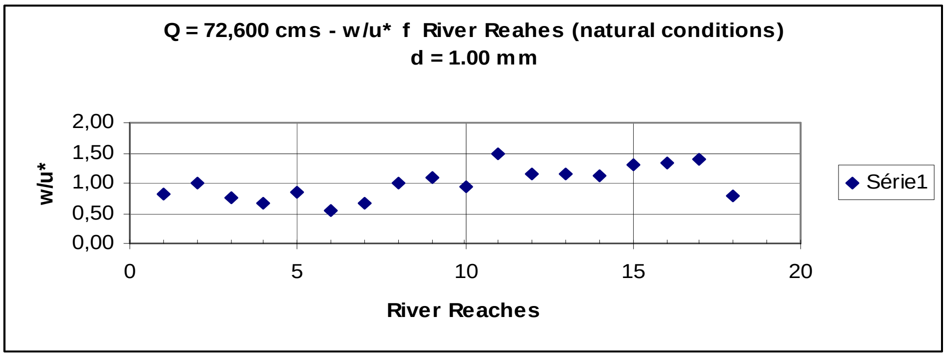


3.00 mm particles are being transported in saltation in all River Reaches excepting between RR 3 and RR 7 and then at RR 18 where they are moving in suspension.

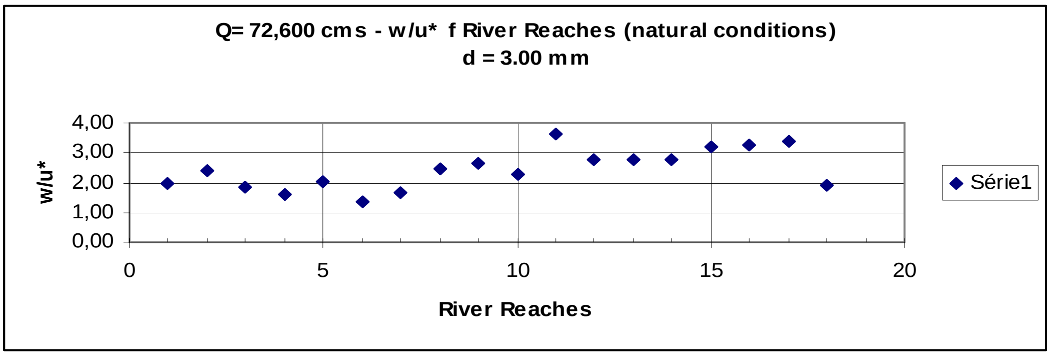


VII – For 72,600m^3/s

1.00mm particles are transported in suspension over the entire river length between RR 1 and RR 18.

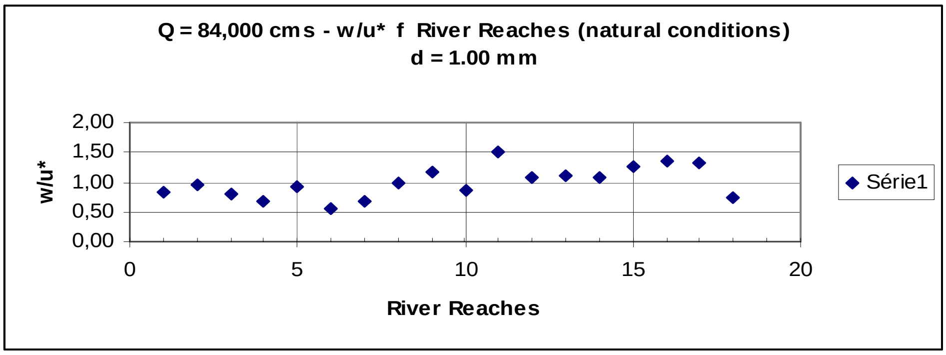


3.00 mm particles are being transported in saltation in all River Reaches excepting between RR 1 and RR 7 and then at RR 18 where they are moving in suspension.

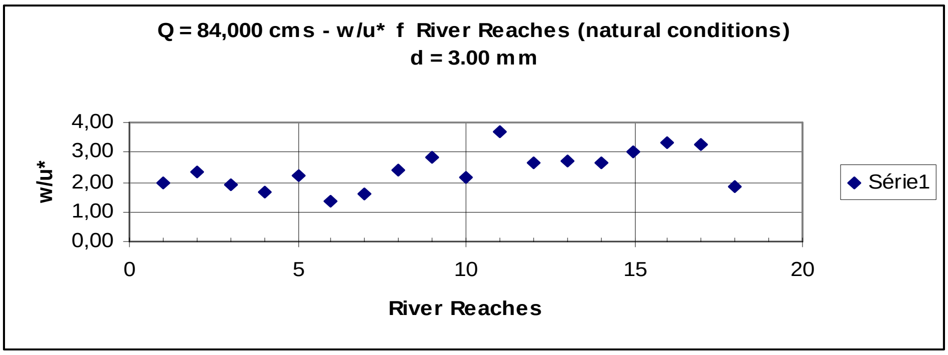


VIII – Q = 84,000m^3/s

1.00mm particles are transported in suspension over the entire river length between RR 1 and RR 18.



3.00 mm particles are being transported in saltation in all River Reaches excepting between RR 3 and RR 7 and then at RR 18 where they are moving in suspension.

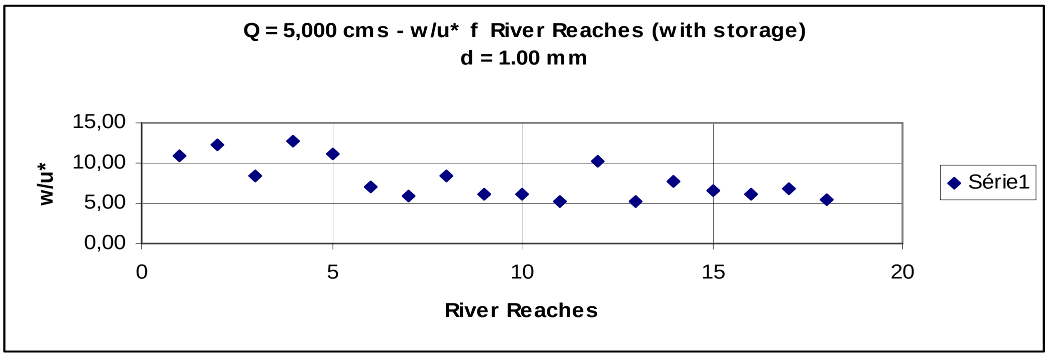


2.2 Sediment transport characteristics with Storage (Ponding) at AHE Santo Antônio for the following river discharges: 5,000; 10,000; 18,000; 39,100; 48,600; 61,200; 2,600 and 84,000 m^3/s

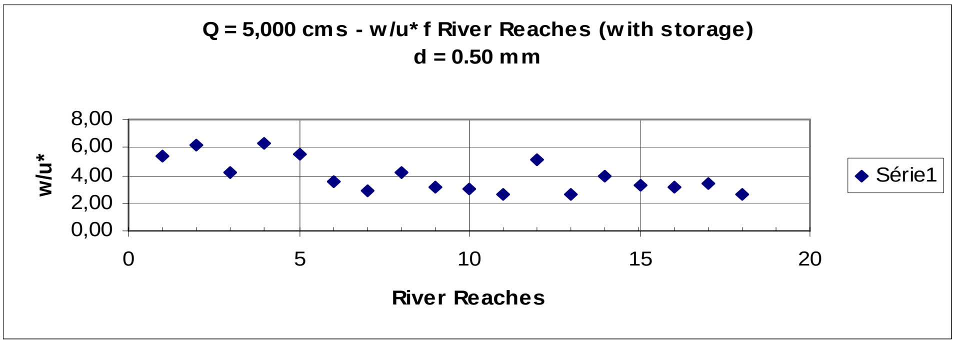
As mentioned earlier with storage the effect of backwater would extend up to Girao for river discharges of 5,000 and 10,000m^3/s and with increasing river discharge the pool would gradually recede and at 39,100m^3/s the ponding would extend to about 60 km out of a total of 125 km i.e., 48% of the pool length. The increased water depth and reduced flow velocities in this portion of the pool will impact its sediment transport capacity. And in certain areas the existing Islands and river channels would be significantly aggraded for discharges up to18,000 m^3/s. For discharges of 39,000 m^3/s or more generalized sand transport of all particle sizes will commence and a part of the earlier deposits would be eroded. The Tables showing the values of w/u* at each River Reach and for various sand particle sizes and discharges with Storage (ponding) are shown in Appendices IA, IIA, IIIA, IVA, VA, VIA , VIIA, VIIIA.
Analyses of transport characteristics for critical particle sizes for each reference discharges are given below.

IA – Q = 5,000m^3/s

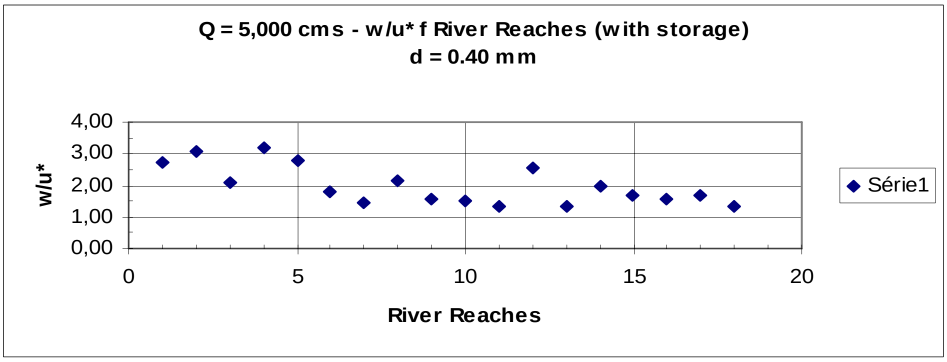
1.00mm particles are completely inert over the entire length of the reservoir RR1 to RR 18



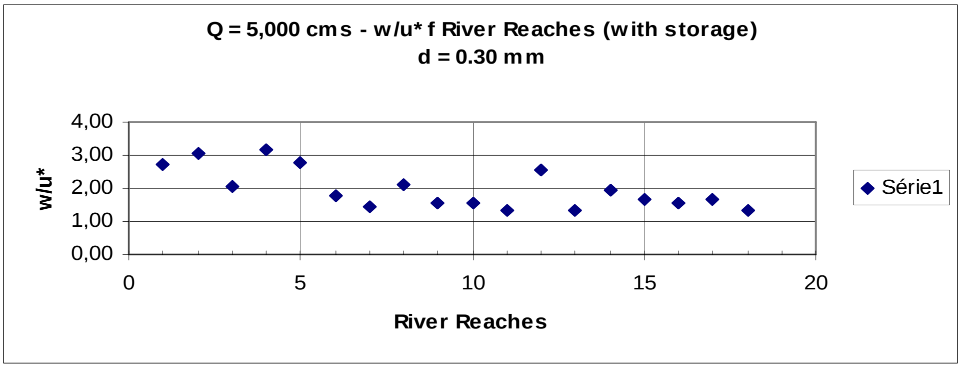
0.50mm particles are completely inert between RR 1 and RR 5 and then moving mostly in saltation upstream of RR 6.



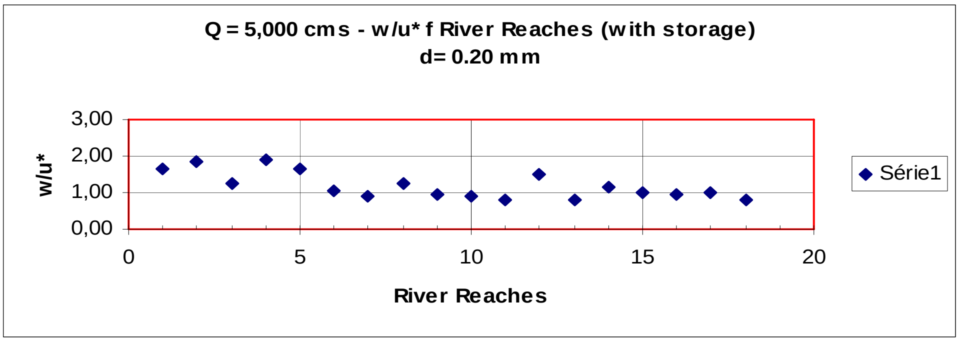
0.40 mm particles are moving in saltation between RR 1 and RR 5 and further upstream they are being transported in suspension.



0.30 mm particles are moving in saltation up to RR 5 further upstream they are moving in suspension over the remaining length of the pool.

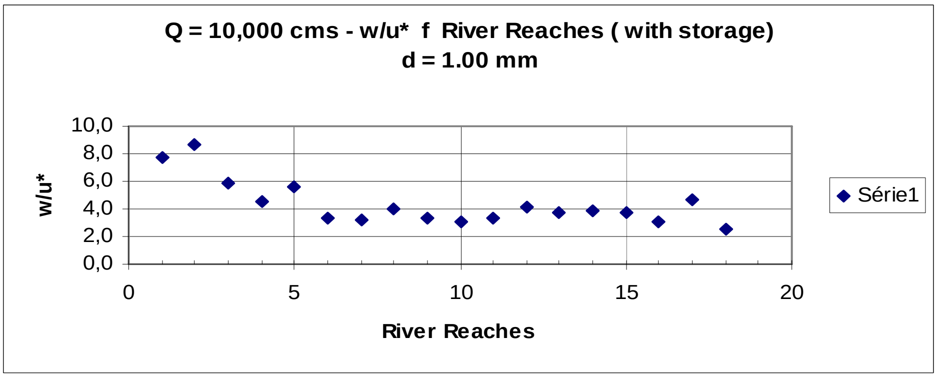


0.20 mm particles are moving in suspension over the entire length of the pool.

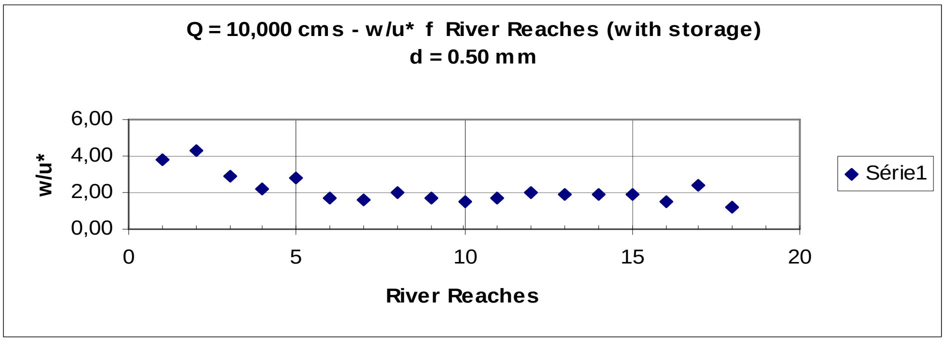


IIA - Q = 10,000 m^3/s

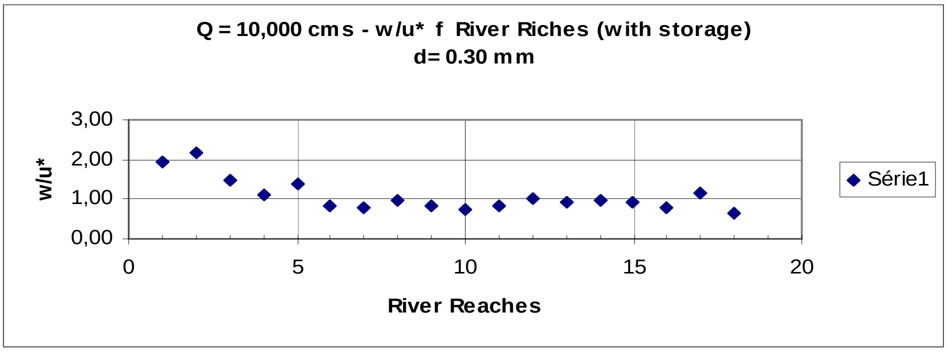
1.00 mm particles are completely inert between RR 1 and RR 5 further upstream they are moving in saltation.



0.50 mm particles are moving in saltation up to section 10 further upstream they are being transported in suspension.

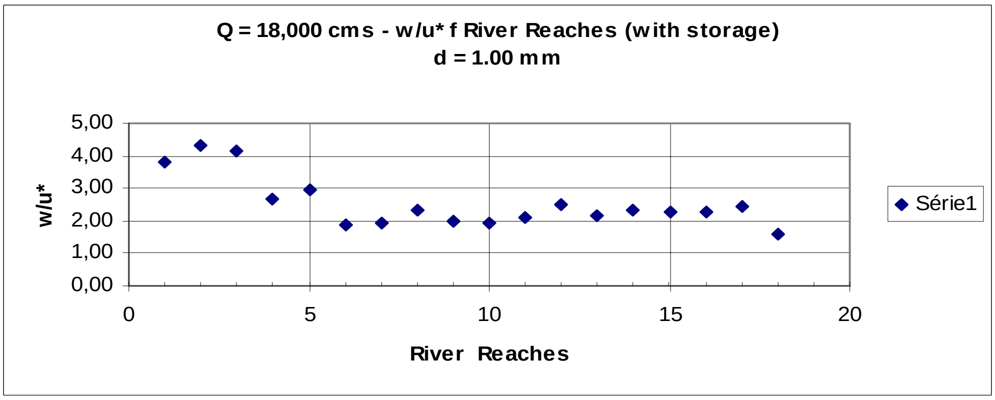


0.03 mm particles are being transported in suspension over the entire length of the pool

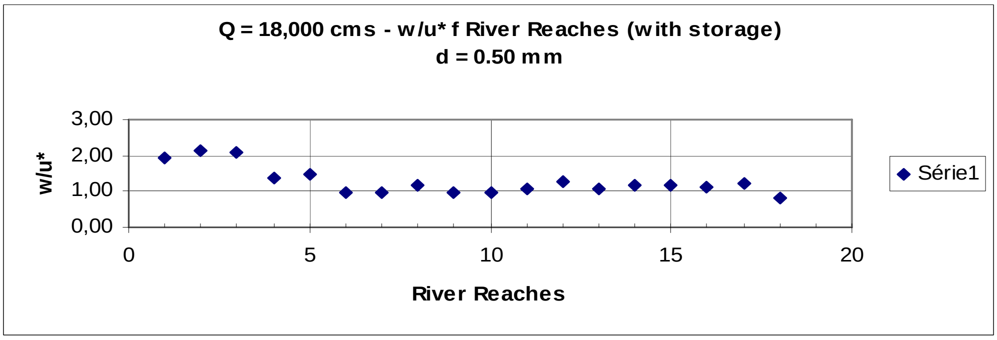


IIIA - Q = 18,000 m^3/s

1.00 mm particles are moving in saltation between Section 6 and 10 further up stream they are being transported in suspension.

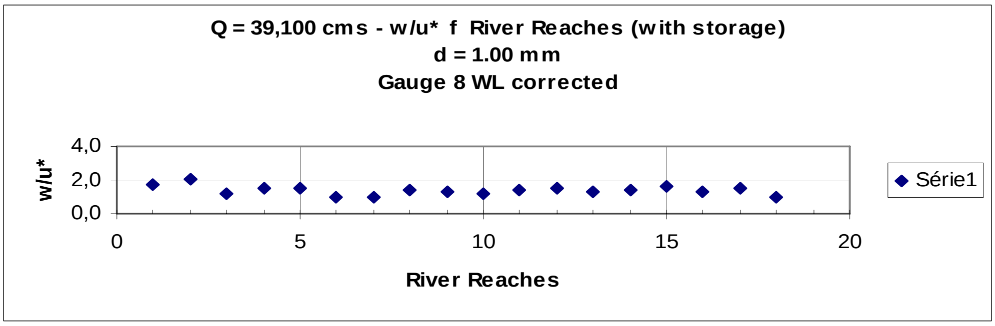


0.50 mm particles are being transported in suspension over the entire length of the pool.

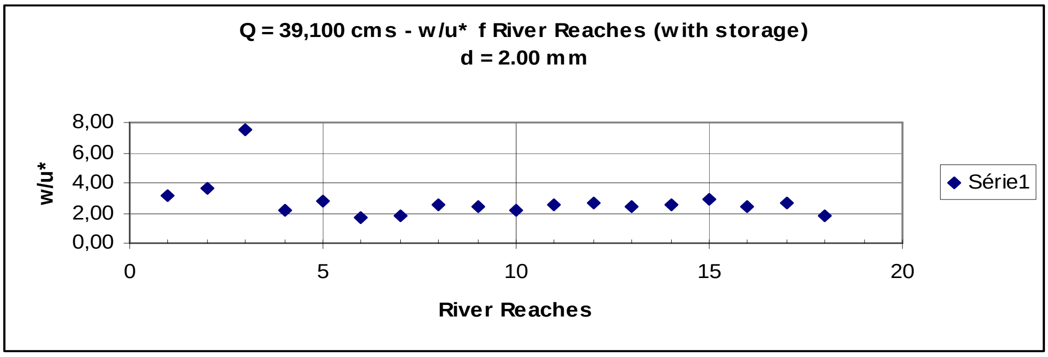


IVA - Q = 39,100 m/s

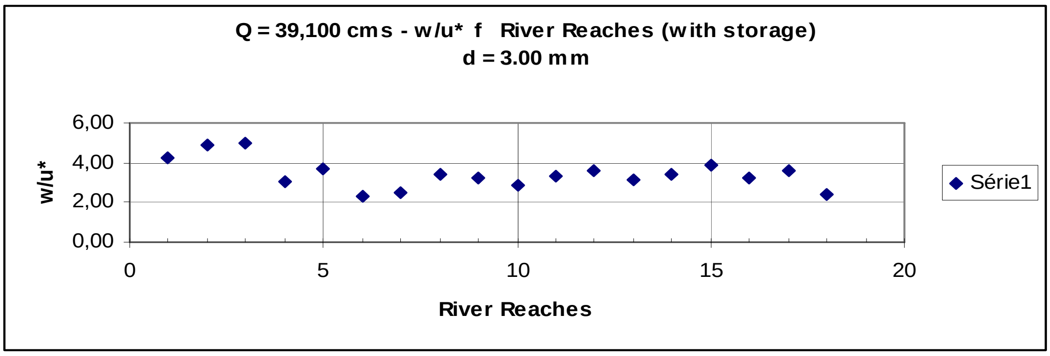
1.00 mm particles are being transported in suspension over the entire length of the pool.



2.00 mm particles are moving almost in suspension over the entire length of the pool

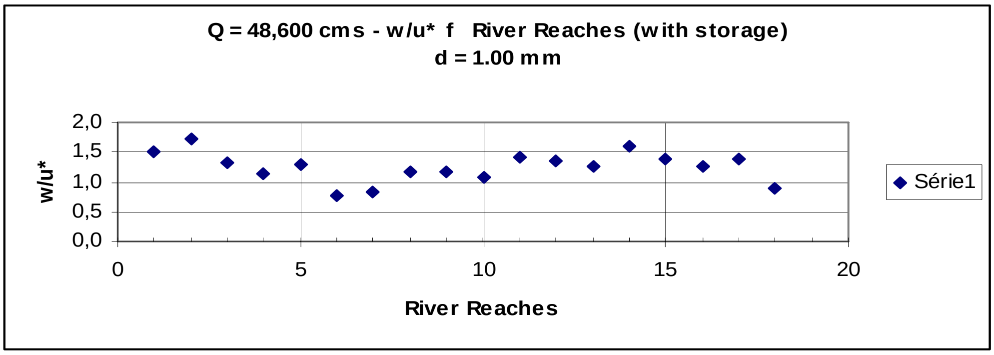


3.00 mm particles are inert up to RR 3 further upstream they are moving in saltation.

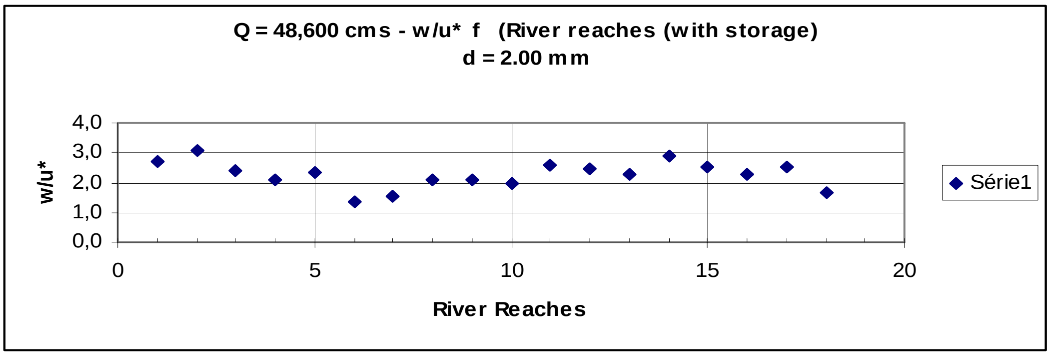


VA - Q = 48,600 m^3/s

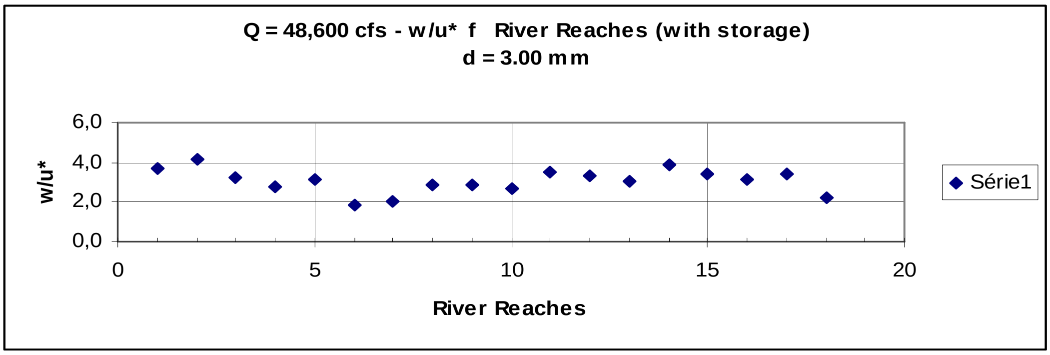
1.00 mm particles are transported in suspension over the entire length of the pool.



2.00 mm particles are mostly moving in saltation and between RR 1and RR 5 and then in suspension between RR 6 and RR 10 followed by RR 11 and RR 17 in saltation and at RR18 in suspension. This underlines the very complex and intermittent nature of the sand movement.

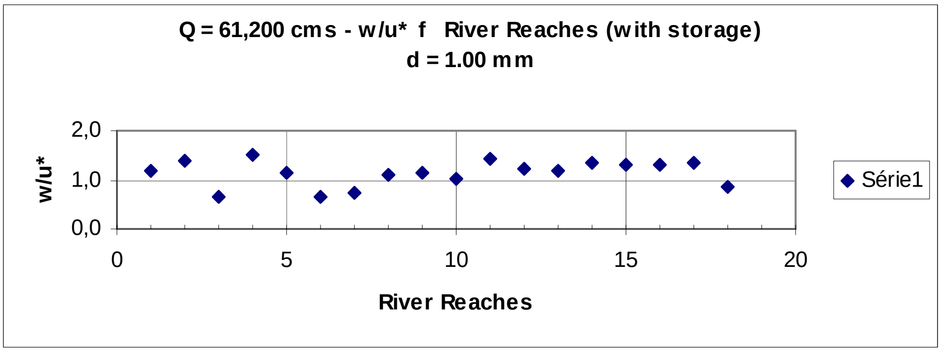


3.00 mm particles are moving generally in saltation over the entire length of the pool

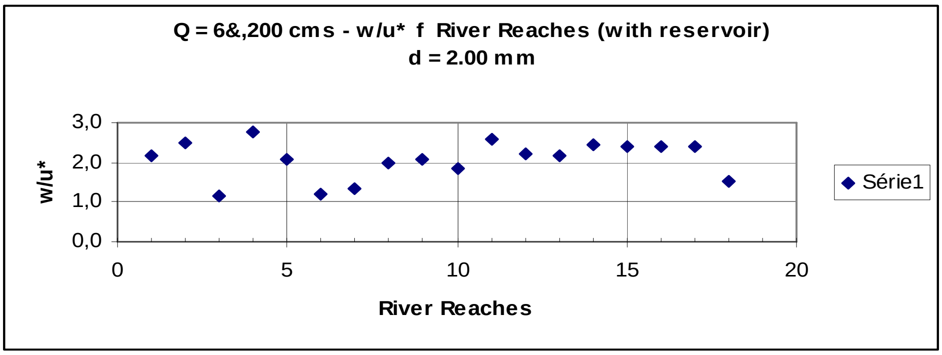


VIA – Q = 61,200 m/s

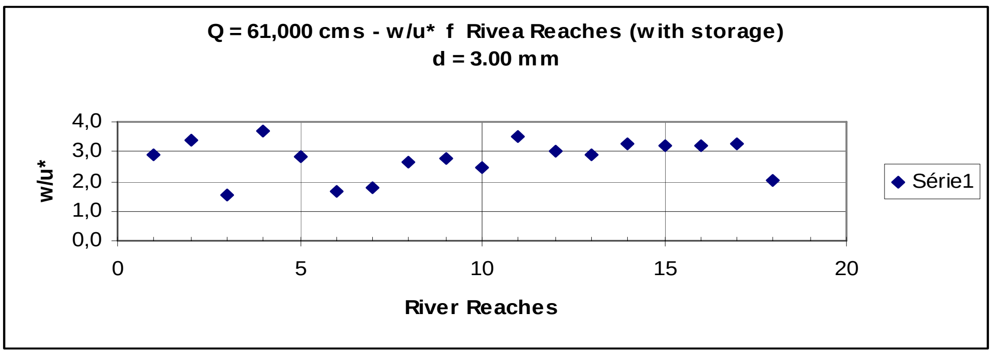
1.0 mm particles are being transported in suspension over the entire length of the pool.



2.00 mm particles are moving generally in saltation and in certain areas is being transported in suspension

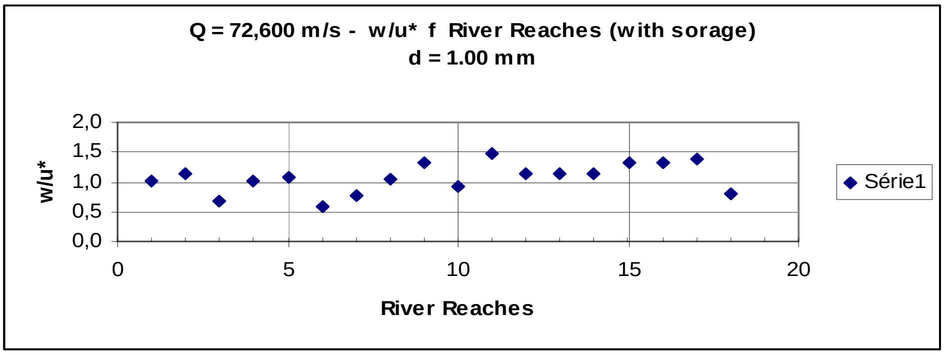


3.00 mm particles are moving in saltation and in certain areas being transported in suspension

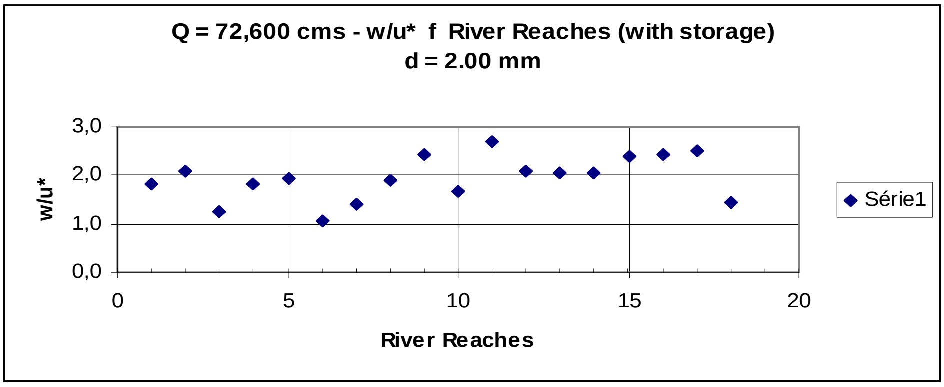


VIIA – Q = 72,600 m/s

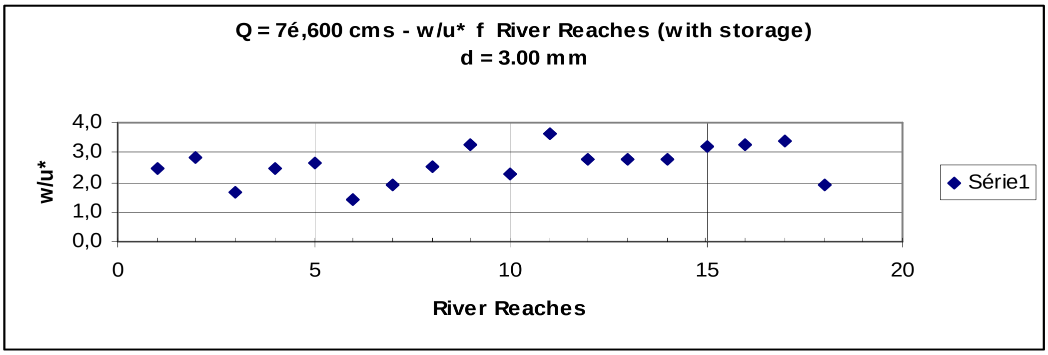
1.0 mm particles are being transported in suspension over the entire length of the pool



2.00 mm particles are mostly being transported in suspension and at certain reaches in saltation

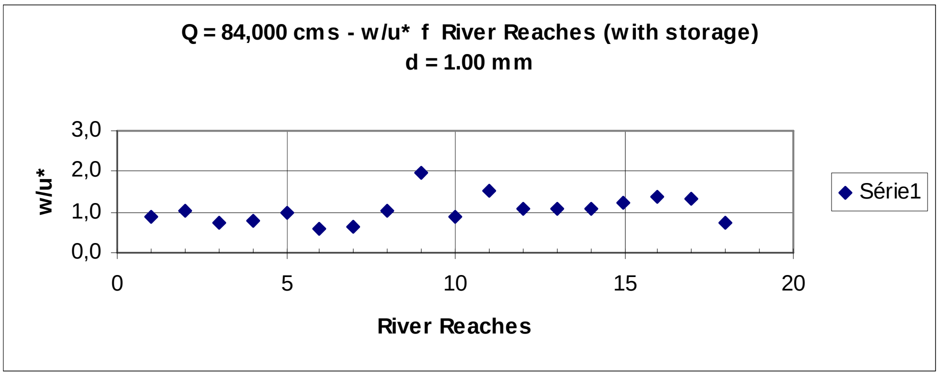


3.00 mm particles are mostly moving in saltation and in certain areas are being transported in suspension.

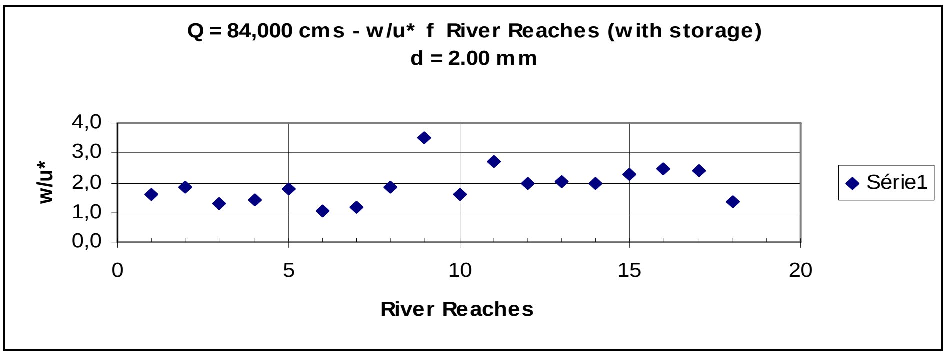


VIIIA – 84,000 m^3/s

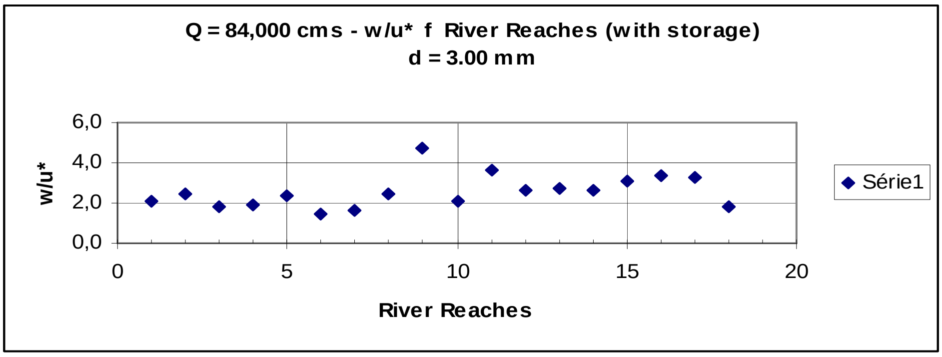
1.00 mm particles are being transported in suspension over the entire length of the pool.



2.00 mm particles are being transported mostly in suspension and in certain areas they are moving in saltation



3.00 mm particles are mostly moving in saltation and in certain areas being transported in suspension.



3. Main Findings and conclusions

Main findings based on the:

- Review of the various reports,
- Visit of the river and the project site, and
- Analyses of the sediment transport characteristics under natural conditions and with storage

Are the following:

3.1 River flow conditions between Girao and Santo Antônio

The Madeira River between Girao and Santo Antônio under its present conditions has several rock outcrops in the shape of rapids and isolated islands. These rock outcrops has stabilized the river bed profile and controls the water surface slopes between reaches. They also create locally very high flow velocities (Photos – 4 and 5) and create immediately downstream sand bars or islands Photos – 8 and 9. Some of the Islands have fairly large trees and other types of vegetal cover. The main Islands are:

a) Ilha do Padre. b) Ilha Santana. c) Ilha Niteroi. d) Ilha Liverpool. e)Ilha Sao Patricio. f) IlhaTaruma. g) Ilha Camaleão and under water fine gravel bar at Paulino are good examples and may be viewed using Google Earth.

Compared to the existing conditions all the Islands downstream of the rapids at section 12 and the river at section 23 will be annually submerged. The increase in submergence depths would be as follows (ref: Figures 1.2.1 &nd 1.2.2):

- 22.00 m to 5.00 m for Q = 5,000 m^3/s.
- 19.00 m to 2.50 m for Q = 10,000 m^3/s.
- 15.00 m to 0.50 m for Q = 18,000 m^3/s.
- 7.00 m to 0.00 m for Q = 48,600 m^3/s.

The corresponding minimum and maximum reduction in flow velocities are:

- 0.03m/s to 0.93 m/s for Q = 5,000 m^3/s.
- 0.02 m/s to 1.44 m/s for Q = 10,000 m^3/s
- 0.01 m/s to 1.96 m/s for Q = 18,000 m^3/s.
- 0.00 m/s to 0.99 m/s for Q = 39,100 m^3/s.
- 0.00 m/s to 0.97 m/s for Q = 48,600 m^3/s.

Although the reductions of flow velocities are significant within the pool, the remaining flow-through velocities are still relatively high preventing deposition of silt and clay which is 85% of the total sediment content. Residual flow-through velocities in the upper pool are as follows (ref: Table and Figures 1 to 8):

- 0.10 m/s to 0.25 m/s for Q = 5,000 m^3/s.
- 0.20 m/s to 0.55 m/s for Q = 10,000 m^3/s
- 0.38 m/s to 0.95 m/s for Q = 18,000 m^3/s.
- 0.80 m/s to 2.00 m/s for Q = 39,100 m^3/s.
- 1.10 m/s to 2.45 m/s for Q = 48,600 m^3/s.

Especially for the annual flood discharges of 39,100m^3/s to 45,000m^3/s the velocities are sufficiently high to ensure transport of sand particles. Considering the vegetal covers of some of the existing Islands it may be assumed that these Islands will grow in size and the channel beds will fill up with sands. However, with time gradually the flow velocities will increase. In the long run a new equilibrium channel cross section as well as the seasonal sand transport pattern through the reservoir will be established.

3.2 Sediment transport conditions between Girao and Santo Antônio

Under the natural conditions the Rio Madeira is capable of transporting sands and fine gravels of particle sizes as shown in Figure 4.17. With storage the flow velocities over the entire length of the pool will be reduced and for discharges up to 18,000m^3/s its sand transport capacity would be considerably reduced. Sand transport capacity of Rio Madeira with natural conditions and with reservoir is summarized below:

Q = 5,000 m^3/s

	d^{max} transported in suspension	d^{max} transported in saltation	d^{max} transported in entrainment	d inert
Natural conditions	1.00 mm	1.00 to 3.00 mm in certain areas	3.00 mm in certain areas	3.00 mm in certain areas
With Reservoir	0.40 mm in certain areas 0.30 mm in certain areas 0.20 mm over the entire length	0.50 mm in certain areas. 0.40 mm in certain areas. 0.30 mm in certain areas	0.50 mm in certain areas.	1.00 mm over the entire length

10,000 m^3/s

	d^{max} transported in suspension	d^{max} transported in saltation	d^{max} transported in entrainment	d inert
Natural conditions	1.00 mm over the entire length 3.00 mm in certain areas	3.00 mm in certain areas	3.00 mm in certain areas	3.00 mm in certain areas
With Reservoir	0.30 mm over the entire length.	0.50 over the entire length	1.00 mm in certain areas	1.00 mm in certain areas

Q = 18,000 m^3/s

	d^{max} transported in suspension	d^{max} transported in saltation	d^{max} transported in entrainment	d inert
Natural conditions	1.00 mm over the entire length 3.00 mmin certain areas	3.00 mm in certain areas	3.00 mm in certain areas	3.00 in certain areas
With Reservoir	1.00 mm in certain areas 0.50 mm over the entire length	1.00 mm in certain areas	1.00 mm in certain areas	

Q = 39,100 m^3/s

	d ^{max} transported in suspension	d ^{max} transported in saltation	d ^{max} transported in entrainment	d inert
Natural conditions	1.00 mm over the entire length	3.00 mm over the entire length.	3.00 mm in certain areas	3.00 mm no where
With Reservoir	1.0 mm over the entire length	2.00 mm over the entire length 3.00 over most of the length	3.00 mm over most of the length	3.00 mm no where

Q = 48,600 m^3/s

	d ^{max} transported in suspension	d ^{max} transported in saltation	d ^{max} transported in entrainment	d inert
Natural conditions	1.00 mm over the entire length 3.00 mm in certain areas	3.00 mm over the entire length	3.00 mm no where	3.00 mm no where
With Reservoir	1.00 mm over the entire length 2.00 mm in certain areas	3.00 mm over the entire length	3.00 mm no where	3.00 mm no where

It can be concluded that that although at low flows (up to 18,000m^3/s) the coarse sand movement are not generalized from 39,100m^3/s however, all sands are transported in suspension and fine gravels are moving in saltation over the entire length.

Hence, accumulation of coarse sands and fine gravels would be a very slow and intermittent process and limited to specific areas. After many years of operation with saturation deposits generalized transport of all bed load material will be re-established

3.3 Need for improving coarse sands and fine gravels evacuation process through the spillway by changing the project layout.

The actual bed load transport in the project area is clearly along the right bank. The spillway layout and its location reproduced here in Figure 3.3.1 are not completely satisfactory. The widespread

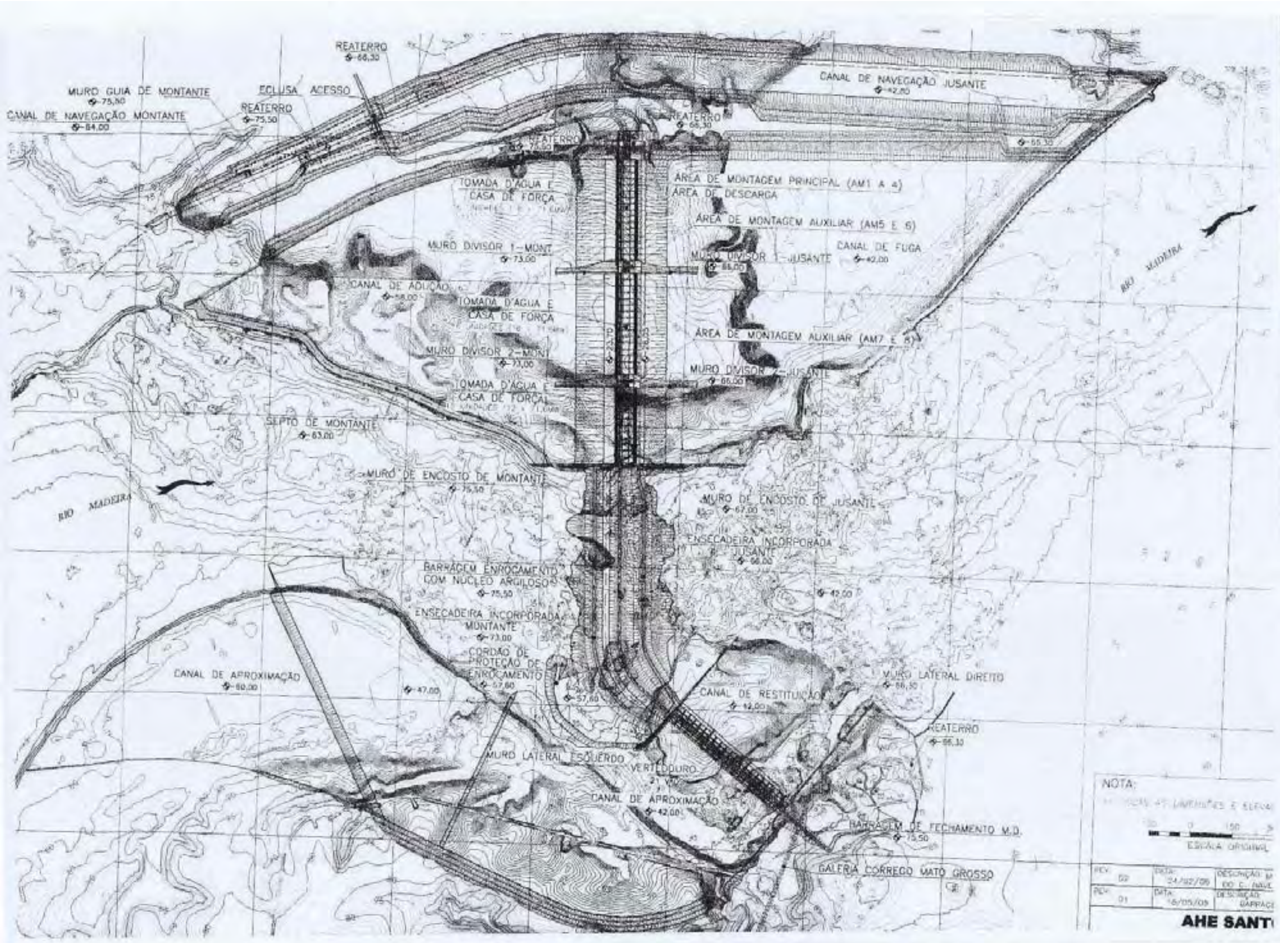


Figure 3.3.1 – The general Layout of the Project proposed by PCE-FOURNAS-ODEBRECHT -ref: Drawing PJ-0532-V3-GR-DE-0021) proposed by PCE – FURNAS and ODEBRECHT

dispersion of the structures has resulted in an increase of the flow surface width from its present 1,100 m to 2,700 m and this combined with the increase in flow depth will modify the flow velocities and secondary current patterns in this area and impact the bed load transport pattern in general.

Also the spillway layout in conjunction with the left abutment wall will create very bad approach flow conditions to some of the left side gates.

During the site visit it became evident that the floating and submerged debris at the power intake trash racks could be a serious threat to uninterrupted power generation. The project layout and structural locations are important from the stand point of operational conditions. So a more compact layout is proposed. So that the actual transit conditions of course sands and fine gravels remain the same.

The Drawing of an Alternative project layout that seems to be more in line with such concept is shown in Figure 3.3.2. This arrangement also proposes several major changes in the project concept which in our opinion will significantly enhance the project. The suggestions are as follows:

- Reduce the number of Radial gates from 21 to 15 and move the spillway upstream close to the area with deposits of coarse sands and fine gravels. The new discharge capacity of the spillway is 60,000 m^3/s

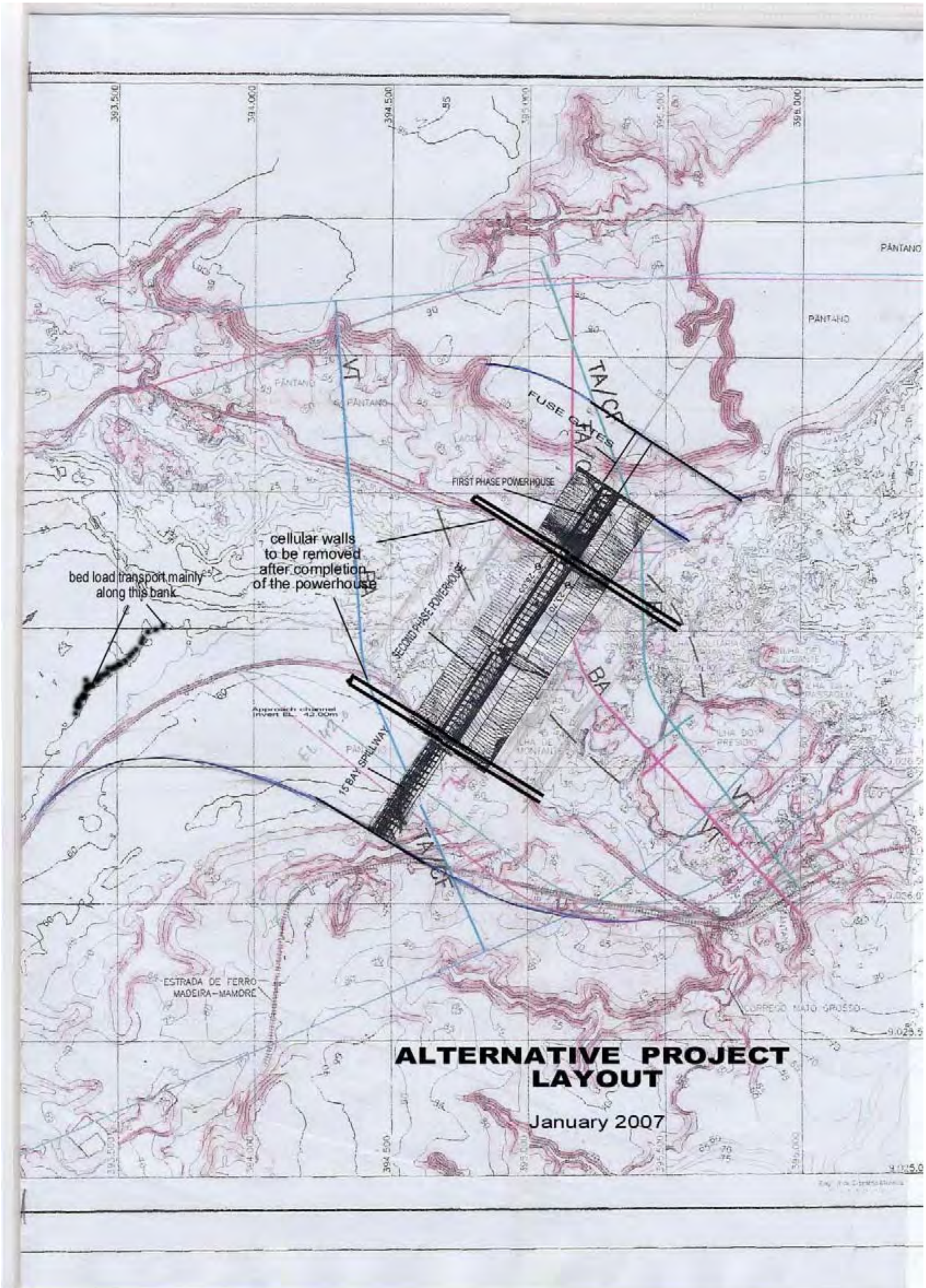


Figure 3.3.2 Alternative project layout

- Replace the 6 Radial gates by Fuse gates with a capacity of about 15,000 m^3/s on the left bank adjacent to the power plant

- A prefabricated float-in type power plant placed in the middle of the river. This will radically change the project construction procedure, construction time and probably the cost. This will allow eliminate the rock-fill dam and reduce significantly the total volume of excavation. The total width at the water surface in the pool would be about 1,700 m instead of 2,700, i.e., a reduction of 1,000 m.
- The possibility of powerhouse sluicing will also add to the capacity of the project flood discharge evacuation in case of emergency. And reduce major variations of the downstream and upstream water level in case of accidental plant shutdown.

ANNEX - I

Need for a comprehensive hydraulic scale model study.

The need for a hydraulic scale model study is evident and we would strongly recommend that a state-of-the-art physical hydraulic scale model be built and a comprehensive study program be carried out as soon as possible.

Optimization of the project layout and hydraulic design

The hydraulic scale model would enable the project designers to verify the performances of the proposed project and if necessary to optimise the various structural arrangements. Such a model would also enable to optimise the total volume of rock excavation and construction phasing. The model may be used particularly to study the following aspects:

Safe sand evacuation

The sand transport characteristics at the AHE Santo Antônio might not be a real problem as far as bed aggradations at the upstream face of the power plant or the spillway, as apprehended by the project designers. However, because of the extremely high annual sediment quantities and at times sand concentrations are very high. The structural design should try to reduce as much as possible the quantity of sand passing through the turbines. And this is where the hydraulic scale model could play an important role by:

- Verifying the existing sand transport patterns through the project site including the segregation of the sand particle sizes observed along the river bed immediately upstream and downstream of the proposed structures.
- Define the optimum structural layout which would ensure a safe sand transport pattern through the project, i.e., most of the sand through the gated spillways when the gates are opened for passing the discharge in excess of the of the power plant discharge.
- Best spillway gate operating procedures for achieving the required sand evacuation route from the upper pool area to the river downstream.

- **Floating and submerged debris transport** [3)]

We feel that at AHE Santo Antônio the problem of managing floating debris may be a major problem and would require innovative design of trash handling equipment and the project layout and concept preventing formation of log jams against the trash racks.

A hydraulic scale model may be used to better understand the process of floating debris approach conditions and accumulation in order to develop practical solutions by:

- Simulating as best as possible on the physical model the observed prototype floating and submerged debris transport patterns, debris characteristics, rates and daily maximum quantities.
- Trying to develop structural arrangements and approach flow distributions such that the debris would not create a massive logjam against the powerhouse trash racks and/or the spillway gates during the rising flood hydrographs.
- Preventing the bulk of the floating debris from stacking against the trash racks. If possible keeping the large trees in circulation in the upper pool away from the trash racks or the spillway gates. This would then allow gradual removal of the debris with the help of adequately equipped cranes placed at convenient locations. The hydraulic scale model might indicate the best structural layout to achieve this

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3) It would be useful to observe and document the general floating and submerged debris approach conditions to the project area under various river flow conditions and also information about their composition and approximate annual volume.

At Sidney A. Murray Low Head hydro Station off the Mississippi River in Louisiana which takes only 15% of the Mississippi River flow, plant discharge 4,500 m^3/s and unit discharge 562 m^3/s, has eight 8.2 m dia. turbines. The annual volume of floating debris is about 115,000m^3 of which about 20 to 30% is submerged debris

- **Stable air entraining vortex formation at the Bulb turbine water intakes**

For low head hydro projects, often such vortices are flow induced, i.e., formed by flow Separation caused by a combination of the approach flow directions and the structural configurations in the vicinity of the water intake. The swirling surface velocities gradually organize and transform into a stable air entraining vortex.

For a low head hydro project the entrained air mesh goes directly into the runner chamber causing violent pressure fluctuation resulting in severe vibration of the turbines and the plant structures. For this reason it is recommended that adequate structural modifications which, would eliminate or attenuate such formation of vortices be developed by using adequate 3D physical model.

- **Power plant intake and the tailrace outlet head-losses**

By definition in low head hydro projects every centimetres count. Therefore the hydraulic model could help to improve the performance by:

- Improving the intake approach flow and draft tube outflow conditions so as to reduce head-losses and recovery of a part of the velocity head in tailrace areas.

- **Surge propagation (upstream and downstream) due to total or partial plant shutdown or start up**

  The hydraulic scale model could fairly easily simulate and assess the impacts of the transient surges at the project site by:

- Reproducing surges propagations upstream and downstream due to load rejection and/or load pickup of all the units or a certain number of units.
- Reproducing the mitigating effects of “Sluicing” in such events by partial closure of the draft tube gates.

- **Construction phasing**

  The general structural arrangements of the project will impact the phasing of the construction works and which in turn would impact factors such as:

- Access to various construction sites
- Need for construction of bridges
- Volume of rock excavations
- Total civil engineering construction time, etc.,

A comprehensive physical hydraulic model could thus be of great help to optimize the project layout and structural concepts assuring better overall performance, reduce costs and some of the risks stemming from hydrology of the watershed particularly during the construction phasing.

Figures