ANNEX 6

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Letter of Prof. A Schleiss and Dr G. de Cesare, December 22, 2006

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V/réf. N/réf. LCH-331/AS-GDC Lausanne, 22nd December 2006

Re: Baglihar Hydroelectric Plant - Assessment of reservoir sedimentation and its effects on the power intake, the pondage, and the upstream flood safety.

Dear Professor Lafitte,

On 3 November 2006 you requested our point of view on the design of the appurtenant works for the dam of the Baglihar Hydroelectric scheme in India, in relation to the sediment transport of the Chenab River. In particular, we were asked to respond to the following questions concerning the design of the sluice spillway and of the power intake, done by India.

Is the design compatible with:

a) sustainability of the live storage volume (pondage)?

b) protection of the power intake against the deposition of bed load sediments, and the prevention of suspended load sediments from entering in the power tunnel, which would cause erosion of the turbines?

c) protection of the town of Pul Doda, upstream of the reservoir, from flooding?

The principal data on which the present assessment is based are given as an annex to this letter.

1 Design principles

1.1 Baglihar is a run-of-river plant, the head being created by a dam 134 m high above the river bed; the initial reservoir volume is 400 $M.m^3$. The Chenab River is subjected to an exceptional level of sediment transport (mean annual sediment yield: 30 $M.m^3$) and it is foreseen by the designer that the reservoir will ultimately be filled with sediment, except for a remaining live storage volume of 37.5 $M.m^3$.

1.2 The design of the dam's appurtenant works (spillway, bottom outlet and power intake) should be based on the state of the art and in this respect a major reference is Bulletin 115 of the International Commission on Large Dams (ICOLD): *"Dealing with reservoir sedimentation"*, published in 1999, paragraph 7.1 General, page 79, which reads as follows:
"Bottom outlets may be used for under sluicing floods, emptying of reservoirs, sluicing of sediments and preventing sediment from entering intakes, etc. For the control of reservoir

sedimentation, bottom outlets should be designed (and operated) to preserve reservoir storage in the long term. Hydrological and hydraulic conditions, especially during high flows and high sediment load periods, should be analysed so that suitable outlets can be designed. The bottom outlets should be located low enough to enable draw-down flushing and should have sufficient discharge capacity.

Many existing dams do not have bottom outlets or the outlets have small discharge capacities. Reconstruction of outlets afterwards has been carried out (Sanmenxia, Shuicaozi and Linjiaxia Dams, in China), but it is generally believed to be uneconomic and in many cases technically difficult.

When the outlet capacity is too small, a low reservoir level cannot be maintained and sediment eroded from the upper reaches (delta) will only be deposited closer to the dam (reworking). In recent years the trend is towards larger low level bottom outlets which will ensure as little change as possible in the river sediment balance".

1.3 The Baglihar dam has, in addition to a surface spillway with three bays, a sluice spillway with 5 orifices of 105 m^2 each, with a sill level at 808 m asl., that is to say 32 m below the Full Pond Level (FPL) of 840 m asl. The total spillway capacity is of 16500 m^3/s and the sluice spillway capacity (at FPL) is 11200 m^3/s.
The Sediment Management Plan prepared by India[1], says that any river flow exceeding 430 m^3/s (the maximum discharge in the turbines), will be used for sluicing and under-pressure flushing of sediments, especially during the flood season (monsoon period), and at that time, the pond level will be drawn down to the Dead Storage Level (DSL) of 835 m asl.

1.4 We consider that the design principles for these large sluice gates, located below the DSL, are sound and in accordance with current practice, allowing for maintenance of the reservoir by sluicing and flushing. But it is evident that to be efficient, these maintenance operations require a pond level lower than the DSL of 835 m asl. We should justify this statement by our answers to the three questions mentioned above.

2 Sedimentation of the reservoir, with operation of sluicing without drawdown

2.1 The initial river bed level at the dam axis is at 712 m asl, and at the entrance to the reservoir at 840 m asl. The length of the reservoir is 32 km (Some variations appear in India's documents concerning this length: 26 km and 32 km, the latter being realistic) and the mean width of the river bed is 85 m.

2.2 India presents, in its Rejoinder[2], the numerical analysis with a 1D model (Mike11) of the filling of the entire reservoir for a time period of 100 years. The geometric and hydraulic boundary conditions are as follows:

- Sill of the sluice spillway: 808 m asl.
- Bed level fixed at the entrance of the reservoir: 840 m asl.
- Constant water level at the dam axis - monsoon time: 835, non-monsoon 835-840 m asl.
- Power intake discharge: 430 m^3/s.

The grain size distribution of the sediments transported are in a range of 0.063 to 1.18 mm and the mean grain size, d_m, is 0.22 mm.

[1] Government of India. Planning and Model Test Documents. Volume 5 (ii). Sedimentation of the Reservoir and Sediment Management. p 46. 26/12/2005
[2] Government of India. Rejoinder. Volume II. 20 March 2006

After complete impounding of the reservoir (Figures 4.8 and 4.9 of the Rejoinder, Volume II, attached) the river bed level starts at the dam axis, at 808 m asl (the sill level of the sluice gates), then increases slowly up to 810 m asl, 300 m upstream, reaching about 827 m asl some 500 m from the dam axis. At the entrance to the reservoir, the river bed level remains at the fixed level of 840 m asl. The equilibrium slope of the new river bed formed inside the reservoir, according to Figures 4.8, is very small, almost horizontal; it increases by only a few meters along the whole length of the reservoir.

The result of the analysis in the near field of the dam, the first 300 m, is very questionable. The sluicing with a drawdown which is not lower than 835 m asl will generally not allow for a scouring effect of the sediment for a sufficient length upstream of the dam axis to protect the power intake, and to preserve the pondage.

In this respect we are in total agreement with the Experts of Pakistan and in particular with Prof. A. Rooseboom[3], who wrote: "*Efficient sluicing and flushing generally go hand in hand with drawdown, and the recommendations in ICOLD Bulletin 115 are predicated on that basis*."

With the new river bed level near the dam at 827 m asl, and above 840 m asl at the entrance, the answers to the first two questions a) and b) are negative, and c) is not conclusive:

- The pondage of 37.5 M.m^3 will be reduced; according to India[4] the final sustainable volume will be 12 M.m^3
- The water intake, which has its sill level at 818 m asl, i.e. lower than the new river bed in this zone, (about 827 m asl) will be flooded by the bed load sediments and the suspended sediment will enter the power tunnel.
- As some of the inputs and results of the Indian calculations concerning impounding of the reservoir are questionable, especially the length scale, it is not certain whether the river bed level at the entrance to the reservoir will increase higher than 840 m asl or not, with the risk of submergence of the town of Pul Doda.

2.3 But before the analysis done by India in its Rejoinder of March 2006, Pakistan presented in its Reply[5], in January 2006, a numerical analysis with a 1 D model (HEC6-KC-1D) of the filling of the reservoir. It shows (Figure A-14 attached) that after 60 years of sedimentation, the river bed level 1.5 km upstream of the dam will be at 832 m asl, 850 m asl at a distance of some 28 km, and at 858 m as. at the reservoir entrance. The 100 year flood elevation will rise to 867 m asl at the bridge location and the town of Pul Doda will be inundated[6]. We note that the equilibrium slope of the new river bed for a length of 28 km within the reservoir is about 0.08 %, according to Figure A-14.

3 Sedimentation of the reservoir, with operation of drawdown sluicing

3.1 With the five sluice gates, each 10 m wide and 10.5 m high, with their sill level at 808 m asl, what should be the drawdown level of the reservoir for an efficient sluicing operation and at what frequency the operation have to be performed?

[3] Comments of the Government of Pakistan on the final draft Determination by the NE, October 24 2006.
[4] Government of India. Rejoinder. Volume II. 20 March 2006.
[5] Government of Pakistan. Reply to the Counter Memorial by Government of India . Part I. January 25, 2006.
[6] A large ambiguity appears in the analysis done by Pakistan. In Annex IA of the Reply, p1, the gates sill level is indicated at 808 m asl, as in the Indian design, but in Figure A-14, p 20, the river bed level at the dam axis, after 60 years, is at about 825 m asl, which would be the case for a gate sill at 825.60 m asl of the chute spillway proposed by Pakistan. So the reservoir sedimentation would be as stated in Pakistan's design. A rough correction, for a sill at 808 m asl, will give a river bed level of about the same value as stated by India, i.e., 827m asl. But the important result which remains valid is the river bed equilibrium slope of ~0.08 %.

This operation should be made each year during the monsoon season. The objective is that the floods discharged will create the highest tractive force on the river bed all along the river ($\tau = \gamma\ h\ S$, with γ the specific weight of water, h the water depth and S the energy gradient). Protection of the live storage necessitates not only designing low level sluice gates, but also, if possible, providing the possibility to evacuate the floods with a free surface flow through the orifice spillway. These conditions will determine the size of the gates, the flood discharge and its frequency.

3.2 The estimated maximum discharge through the five fully open sluice gates (with a height of of 10.5 m), with a free surface flow, is about 5800 m^3/s. The frequency of this flood is between 1/10 (5100 m^3/s) and 1/25 (6200 m^3/s)[7].
The total width of the sluice spillway is 50 m and the width of the reservoir just upstream is about 200 m. So, the sluice spillway will create a bottleneck, and in order to evacuate the discharge, the reservoir level should be 16 m above the sluice sill, i.e. at el 824 m asl.

3.3 Some considerations are now developed on the evolution of the morphology of the river bed.

- The mean value of the initial slope of the river bed, between the dam axis and the entrance, is 0.38 %. The final equilibrium bed slope found in the numerical model of Pakistan (~0.08 %) is some five times lower than the initial slope bed slope. This means that in its initial stage, the Chenab River is in a state of degradation; there is a tendency for the bed slope to be reduced by backward erosion, starting at some fixed point on the river bed.
  With the presence of the dam and the free flow over it, the reservoir will fill up starting from the gate sill level as a fixed point, proceeding upstream with its equilibrium bed slope up to a point where it connects again to the initial bottom slope. From that point upwards, one will find the original bottom slope which will not fill up, and where ultimately it will generate backward erosion again.

- For the case of the reservoir being completely full, we can estimate approximately the equilibrium river width and slope according to Yalin and da Silva[8], for discharges ranging from Q_2 ~ 3000 m^3/s to Q_{10} ~ 5000 m^3/s, and a mean grain size, d_m, of 0.22 mm
  - regime width: 305 - 388 m
  - regime slope: 0.004 - 0.006 %
  - normal water depth: 9.6 - 12.5 m

  These very low slopes calculated for two single discharges are 5 to 8 times lower than those of the equilibrium bed calculated by Pakistan using the numerical model. We know that the calculations are based on a discharge series of 60 years and a grain size distribution curve. But we can say that for frequent floods, the river bed will be in state of degradation.

3.4 We consider the free flow surface discharge of 5800 m^3/s evacuated by the five sluice gates totally open. We have seen under 3.2 that the upstream water level was 16 m above the gate sill i.e. 824 m asl. But, for a river width of 200 m, a bed slope of 0.08%, and a Manning coefficient n of 0.033, the normal water depth is only about 8 m. So, from the point near the spillway with a depth of 16 m, a backwater curve will develop (over about 10 km) in order to raise the normal water level. The consequence is that along the zone where the water depth is higher than the normal level, the water velocity is lower and deposits occur. We estimate

[7] Government of India. Planning and Model Test Documents. Volume 5 (i). Hydrology. p22 30/11/2005.
[8] Yalin M.S. and da Silva A.M, "Fluvial processes, solutions manual", IAHR International Association for Hydraulic Research Monograph, Balkemaa, 2001)

roughly that close to the dam the depth of the sediments will be equal to the difference between the water depth at this point (16 m) and the normal water depth (8 m), that is to say 8 m, which has to be added to the sluice elevation at 808 m asl, resulting in a fixed point at an el. 816 m asl. From that point on, the river bed will develop upstream with a maximum slope of approximately 0.08 %. The power intake, extending for a length of 80 m upstream of the dam axis and with its sill at el. 818 m asl, will be free of accumulated sediments in front of it.

3.5 The above calculation was done for a flood of 5800 m^3/s, which is the maximum discharge with a free flow surface through the five sluice gates; its return period is about 20 years. The sluicing operations should be carried out each year. The annual flood is about 2500 m^3/s. Considering the two following floods of relatively high frequencies, the reservoir and river bed levels, near the dam axis, in front of the power intake, will be as follows:

Discharge [m^3/s]	Reservoir level [m asl]	Water level over sill [m]	Normal water depth in river [m]	River bed level [m asl]
Q_2 = 3000	818.45	10.45	5.60	813
Q_{10} = 5000	822.60	14.60	7.60	815

It should be recalled that the sill of the power intake is at el. 818 m asl.

3.6 Larger flood events than 5800 m^3/s will produce pressure flow through the sluices with rising of the water level upstream. That will produce a higher sediment level in front of the dam with a local erosion cone just upstream of the sluices. The deposits formed will be evacuated by drawdown sluicing and flushing by the higher frequency floods.

3.7 As for the protection of the town of Pul Doda against flooding, the numerical analysis conducted by Pakistan[9] can give a good approximation.
The sedimentation of the reservoir over 60 years was calculated for three scenarios of Full Pond Level (FPL) elevations (820, 810, 800 m asl.). For the first, 820 m asl, which is close to the reservoir levels prescribed during the operations of drawdown-sluicing mentioned under 3.5 (818.5 and 822.0 m asl) the result of the analysis[10] (Figure A-15) is that the river bed level at the bridge will increase by about 8 m (from 819 m asl to 827 m asl). For a 100-year flood of about 8100 m^3/s, the water level at the bridge will rise to 854 m asl, while the bridge deck is located at el. 851 m asl. The risk of Pul Doda flooding is real. But if we consider the second scenario, with the reservoir level at 810 m asl, the safety against flooding is approximately fulfilled. This result calls into question the sill level of the gates, which is probably not sufficiently low.

4 Conclusion

4.1 Lowering of the reservoir level during flood events is nowadays common practice. In accordance with the state of the art, it is an operation which is considered necessary to keep the reservoir, power intakes and appurtenant structures operational and safe. Operation of the power plant has to be interrupted during this drawdown.

4.2 In the case of Baglihar, a free flow passage of flood events up to a 20-year return period discharge, representing 5800 m^3/s, is feasible, through the five sluice gates in their fully open

[9] Government of Pakistan. Reply to the Counter Memorial by Government of India. Part I, January 25, 2006.
[10] It is clear that the Pakistan's analysis is performed with an hypothesis of a constant reservoir level throughout the year, and in our case we try to estimate the sedimentation of the reservoir resulting from a reservoir level at 840 m asl and 835 m asl, with drawdown operation at 820 m asl during a very short period of time, for example 10 days per year. But Pakistan's analysis gives a preliminary useful idea of the phenomena.

position. The sluicing and flushing operations for maintenance should be carried out each year, and for that purpose, the reservoir level should be drawn down up to a maximum of about el. 818 m asl, that is to say 17 m below the DSL, corresponding to the free flow passage of the annual flood.

4.3 The sill level of the power intake being at el. 818 m asl, it is evident that the power plant could not operate during this phase of maintenance. Its duration can be limited to the duration of the maximum flood discharge, bed and suspended load transport; it should not be stopped at an early stage when solid material is still arriving at the dam. This operation should be monitored very carefully to ensure its success and moreover to prevent abrasion of the turbines.

4,4 Finally, we can state that with adoption of this operation of drawdown sluicing, the answers to the two first questions are clearly positive: a) the pondage volume will be sustainable, b) the power intake will be protected against the deposition of bed load sediments and against substantial suspended load sediments entering the power tunnel.

As for the question c) concerning the protection of the town of Pul Doda, some calculations indicate that the protection against flooding would not be ensured. We consider that the situation could be safe with the sill level of the sluice gates lower, at about el. 800 m asl.

Yours Sincerely

Laboratory of Hydraulic Constructions, LCH

A. Schleiss

Prof. Dr Anton J. Schleiss Dr Giovanni De Cesare

Annexes:

- A1 - Principal data of the Indian design
- A2 - Principal results from the reservoir sedimentation calculations
- CVs of Prof. Dr Anton J. Schleiss and Dr Giovanni De Cesare

ANNEX 1

Principal data of the Indian design

- Full Pond Level (FPL): 840.0 m asl.
- Dead Storage Level (DSL): 835.0 m asl.
- Total spillway capacity (Probable Maximum Flood-PMF-) : .... 16500 m^3/s
    - 5 orifice spillways : .... 11200 m^3/s
    - 3 surface spillways and one auxiliary spillway : .... 5300 m^3/s
- Sluice spillway: 5 gates, each with dimensions of 10 m (W) x 10.50 m (H).
- Sill level of the gates of the chute spillway: 821.0 m asl.
- Sill level of the power intake: 818.0 m asl.
- Sill level of the gates of the sluice spillway: 808.0 m asl.

- Initial river bed level at dam axis: 712 m asl.
- Initial river bed at the reservoir entrance: 840 m asl.
- Length of the reservoir: 32 km (26 km)
- Mean width of the initial river bed: 85 m
- Mean width of the new river bed for the reservoir completely impounded: 200-300 m
- Initial slope of the river bed: 0.38%

- Mean annual river inflow: 25000 $M.m^3$
- Median annual discharge: 410 m^3/s
- Mean annual sediment yield: 30 $M.m^3$
- Total (initial) reservoir volume: 400 $M.m^3$
- Live storage volume (pondage): 37.5 $M.m^3$
- Approximate grain size distribution of the sediments transported by the Chenab River: 0.009 to 1.18 mm (these values indicate a rather fine bottom material composition for such a steep riverbed); admitted mean grain size: d_m = 0.22 mm

ANNEX 2.1

Principal results from the reservoir sedimentation calculations by India

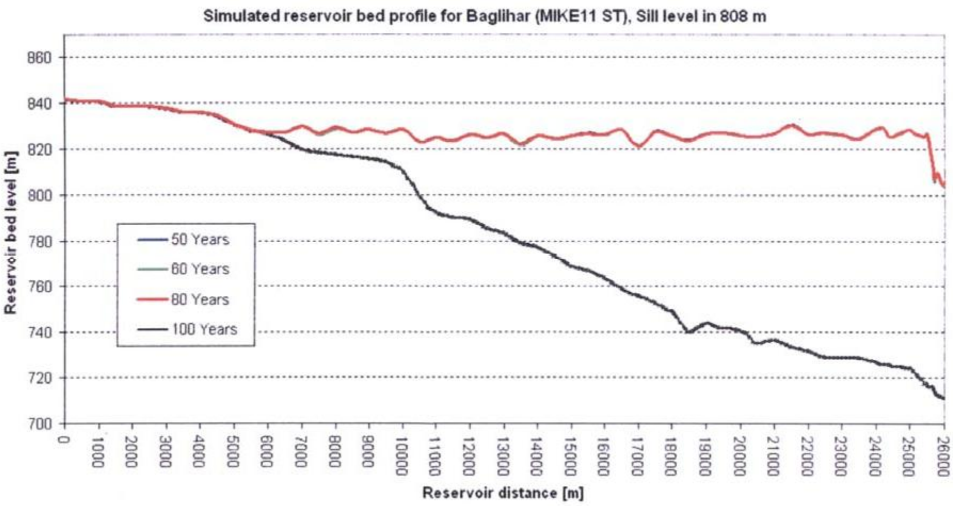

Figure 4.8 Government of India. Rejoinder, Volume II, March 20, 2006, p26

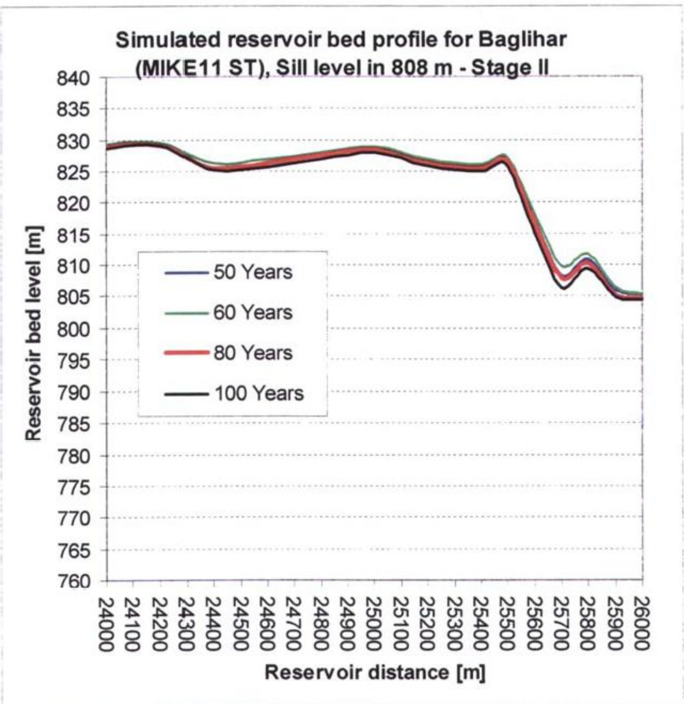

Figure 4.9 Government of India. Rejoinder, Volume II, March 20, 2006, p27.

ANNEX 2.2

Principal results from the reservoir sedimentation calculations by Pakistan

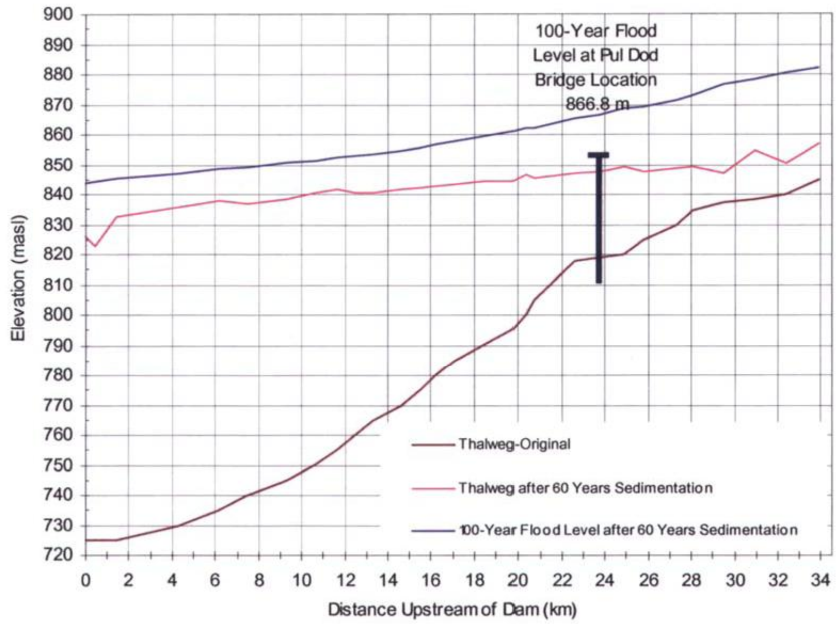

Figure A-14 of the Government of the Pakistani Reply to the Counter Memorial by Government of India, Part I, January 25, 2006, Annex I-A, p20

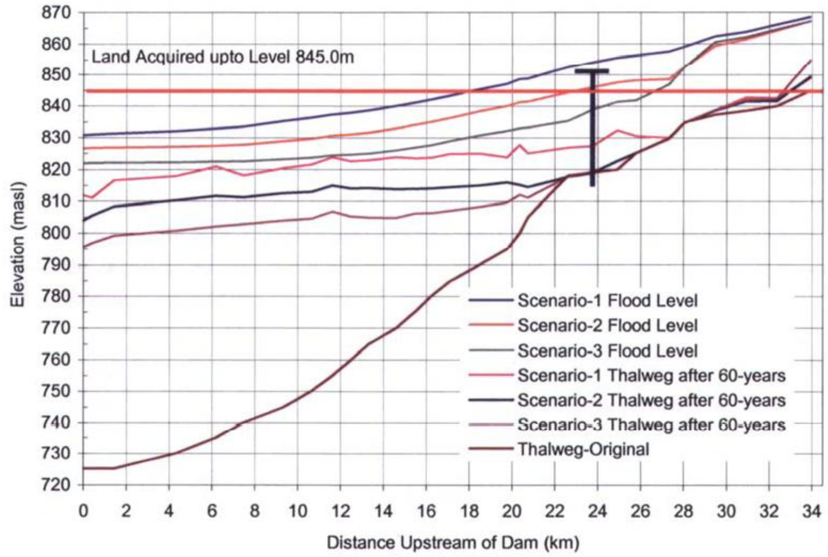

Figure A-15 of the Government of the Pakistani Reply to the Counter Memorial by Government of India, Part I, January 25, 2006, Annex I-A, p21,
Scenario-1: FPL 820 m asl.; Scenario-2: FPL 810 m asl.; Scenario-3: FPL 800 m asl.

CV of Prof. Dr Anton J. Schleiss

Anton J. Schleiss

Marital status : Married

Current position :

Swiss Federal Institute of Technology Lausanne (EPFL)
Laboratory of Hydraulic Constructions (LCH)
Faculty of Architecture, Civil Engineering and Environment

Director and Full Professor since 1.10.1997, responsible for teaching, research and consulting (services as expert) in the field of hydraulic structures, dams and flood protection

Former positions :

1986 - 1996: Electrowatt Engineering Ltd. Zurich
Head of the Hydraulic Structures Section, Hydropower Plants and Water Management Department, Business Unit Energy Swiss Federal Institute of Technology Lausanne (EPFL)

1979 - 1985: Swiss Federal Institute of Technology Zurich; Laboratory of Hydraulics, Hydrology and Glaciology (VAW), Research Associate and Senior Assistant.

Date of Birth : January 11 1953

Nationality : Swiss

Education : Swiss courses for management of enterprises (SKU – Schweizerische Kurse für Unternehmensführung), 1996
Laboratory of Hydraulics, Hydrology and Glaciology, Swiss Federal Institute of Technology (ETH), Zurich, Ph.D. Thesis in Technical Sciences, 1986
Swiss Federal Institute of Technology (ETH), Zurich
Dipl. Ing. ETH, 1978 (equivalent to M.Sc. in Civil Engineering)

Languages : German (mother tongue), English, French, knowledge of Spanish

Publications : Author or co-author of more than 100 scientific publications; reviewer for : J. of Hydraulic Research; J. of Hydraulic Engineering; Advances in Water Resources, J. of Limnology and Oceanography, Aquatic Sciences

Foreign experience : Austria, Georgia, India, Iceland, Islamic Republic of Iran, Korea, Malaysia, Sri Lanka, USA, Tanzania

Professional activities : Hydraulic and hydromechanical analysis of hydraulic structures (steady or unsteady flows and transient phenomena, vibration and stability problems)
Planning and design of hydroelectric and irrigation schemes
Planning and design of high head pressure conduits
(penstocks, steel liners, shafts and tunnels, caverns)
Supervision and interpretation of geotechnical site investigations for pressure tunnels and shafts,
Supervision and interpretation of hydraulic model tests
Computer analysis and simulation of coupled mechanical-hydraulic problems, especially in the field of high head pressure conduits under consideration of seepage phenomena through concrete lining

and rock
Planning and design of river flood protection measures, river hydraulics, sediment transport

Research activities :

Reservoir sedimentation by turbidity currents
Erosion of Alpine catchment areas by overland flow
Hydraulic behaviour and modelling of surface runoff
Dynamic water pressure in rock fissures due to high velocity jets
Roughness effect of outside protection walls on flow and scouring in river beds
Technical solutions for managing reservoir sedimentation by density currents in Alpine reservoirs
High velocity flow on steep slope over macro-roughness
Influence of a side weir on the sediment transport in a channel
Influence of aeration and prototype characteristics of plunge pools and rock joints on the formation of rock scour by high velocity jets
Effects of waves on erosion and efficiency of protection measures on lake bank stability
Effects of river naturalization measures on peaking in rivers due to hydro power plants
Expert system for the managements of floods in the Rhone catchment area
Methodologies and strategies for the evaluation of multi-purpose run-of-river power plants
Design of contractible floating oil retaining reservoir
Influence of geometry of shallow reservoirs on the sedimentation process

Activities in professional organisations :

Swiss Association of Engineers and Architects (SIA)
Institution of Water Management (SWV), member of the board, Switzerland
Swiss Committee on Dams (SCD), president
International Commission on Large Dams (ICOLD), member of the technical committee on Hydraulics for Dams
International Hydropower Association (IHA) member of the technical committee for organisation of conferences
Swiss Committee of Flood Protection (KOHS), president
Swiss Water Pollution Control Association (VSA)
International Association for Hydraulic Research (IAHR), member of the European committee
Society of Water Management and Rural Engineering (DVWK), Germany

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CV of Dr Giovanni De Cesare

Giovanni De Cesare

Marital status : Married, three children

Current positions :

Swiss Federal Institute of Technology Lausanne (EPFL), Laboratory of Hydraulic Constructions (LCH), Faculty of Architecture, Civil Engineering and Environment

Senior research associate, lecturer and administrator of the Master of advanced Studies (MAS) programme in water resources management and engineering, member of the scientific council of the LCH

University of Applied Sciences of Western Switzerland in Yverdon (HEIG-VD)
Lecturer in hydraulics

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Nationality : Italian

Education : PhD in technical sciences EPFL, Lausanne, 1998
Graduate Civil Engineer EPFL, Lausanne, 1992
Undergraduate Student Exchange, Georgia Institute of Technology, Atlanta, 1990

Languages : German, Italian, French, English, basics in Polish and Spanish

Publications : Author of some 24 and co-author of more than 30 scientific publications with about 20 papers related to reservoir sedimentation; reviewer for : J. of Hydraulic Research; J. of Fluids Engineering; Sedimentology, J. of Flow Measurement and Instrumentation; and Hydro Review Worldwide HRW

Service activities : Participation in more than 30 commissioned studies and expert reports in applied hydraulics, hydraulic constructions, physical and numerical modelling, with some 15 related to sedimentation

Research activities : Main research topics: Numerical simulation of hydraulic works and reservoir sedimentation; Turbidity currents; Reservoir sediment management; ultrasonic Doppler flow measurement. Swiss coordinator of the European INTERREG III B ALPRESERV project "Sustainable Sediment Management of Alpine Reservoirs considering ecological and economical aspects"

Activities in professional organisations :

Swiss Association of Biotechnical Engineering (VIB), member of the directorial board

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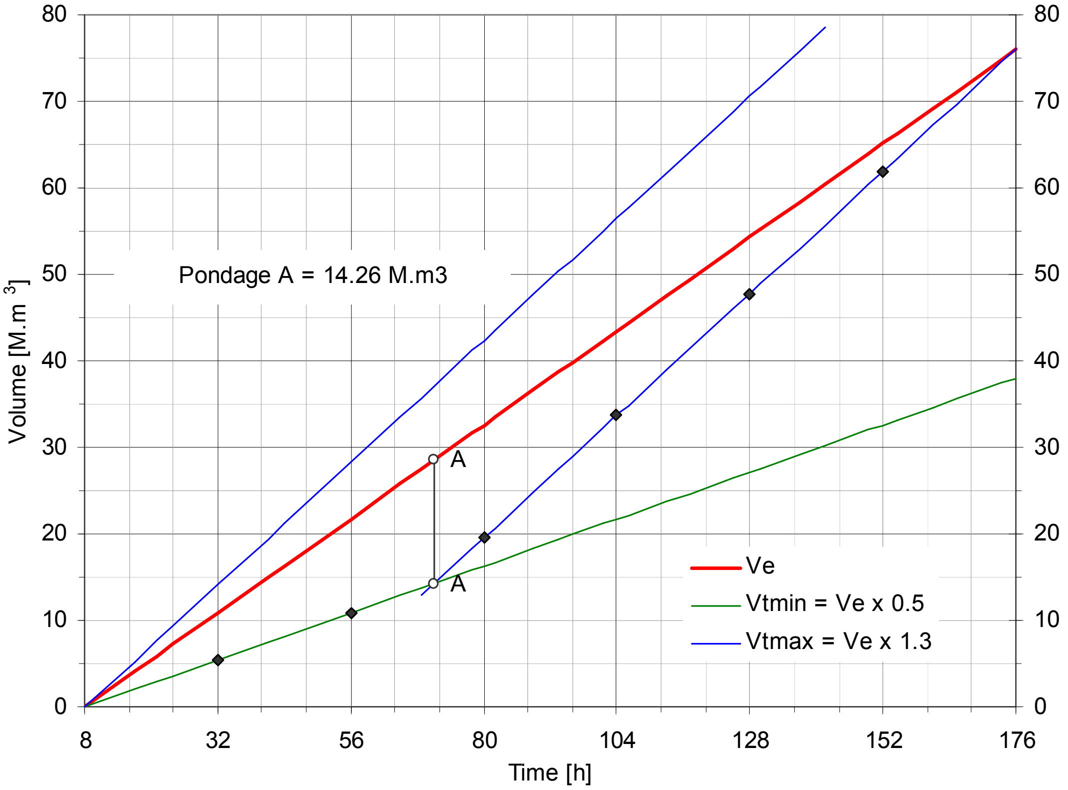
Mass curves of plant operation
according exclusively to the provision of the Treaty, Annexure D, Part 3, Paragraph 15 (ii)



Mean discharge at the site during the week: 125.68 m^3/s

Operation of the plant:

- Weekend: minimum discharge through the turbines: $V_{t,\min}$
- Working day maximum discharge through the turbines: $V_{t,\max}$

Calculation of the volume of Pondage in accordance with the provision of the Treaty

Annexure D, Part 3, Paragraph 15 (ii)

h : time

q_e : Inflow discharge entering the reservoir

q_t : Discharge through the turbine

V_e : Volume entering the reservoir

V_t : Volume flowing through the turbines

$V_e(h) = \int_o^h q_e \cdot dh$ and $V_t(h) = \int_o^h q_t \cdot dh$

Weekly balance: at the end of the week, $h = h_0$, and $V_{t,h_0} = V_{e,h_0}$

Daily conditions: $V_{t,\max} = a \cdot V_e$ and $V_{t,\min} = b \cdot V_e$

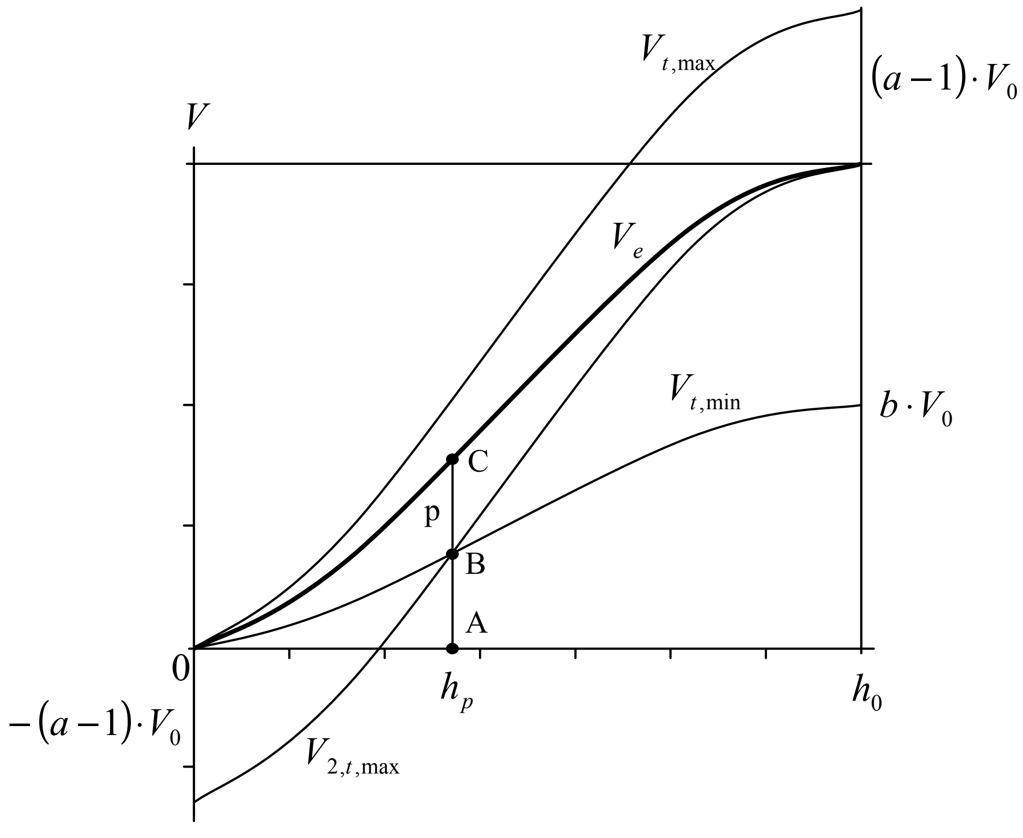


Figure 1: Operation of the plant according to the provision of the Treaty, *Annexure D, Part 3, Paragraph 15(ii)*

Hypothesis for operation of the plant:

- Weekend: minimum discharge through the turbines: $V_{t,\min}$

- Working day: maximum discharge through the turbines: $V_{t,\max}$, which is the transfer by $(a-1)\cdot V_0$ of $V_{t,\max}$

$$V_{2,t,\max} = a \cdot V_e - (a-1)\cdot V_0$$

$\overline{BC} = p$: necessary volume of the operating pool

h_p : time where the volume of operating pool appears

h_p is at the intersection of $V_{t,\min}$ and $V_{2,t,\max}$

$V_{t,\min} = V_{2,t,\max}$ gives $V_{e,p} = \overline{AC}$

$$\boxed{p = V_0 \cdot \frac{a-1}{a-b} \cdot (1-b)} \text{ (1) and } \boxed{P = 2 \cdot p} \text{ (2)}$$

***P* is independent of the function $V_e = f(h)$ and depends only of the values of V_e, a and b.**

PRACTICAL APPLICATION, WITH A CONSTANT INFLOW ALONG THE WEEK

$V_e = f(h)$ is linear, $V_e = \frac{V_0}{h_0} \cdot h$

$$\boxed{h_p = \frac{a-1}{a-b} \cdot h_0} \text{ (3)}$$

Considering $a = 1.3$ and $b = 0.5$, and $h_0 = 168\,[h]$, we obtain $V_0 = 76.01\,[Mm^3]$

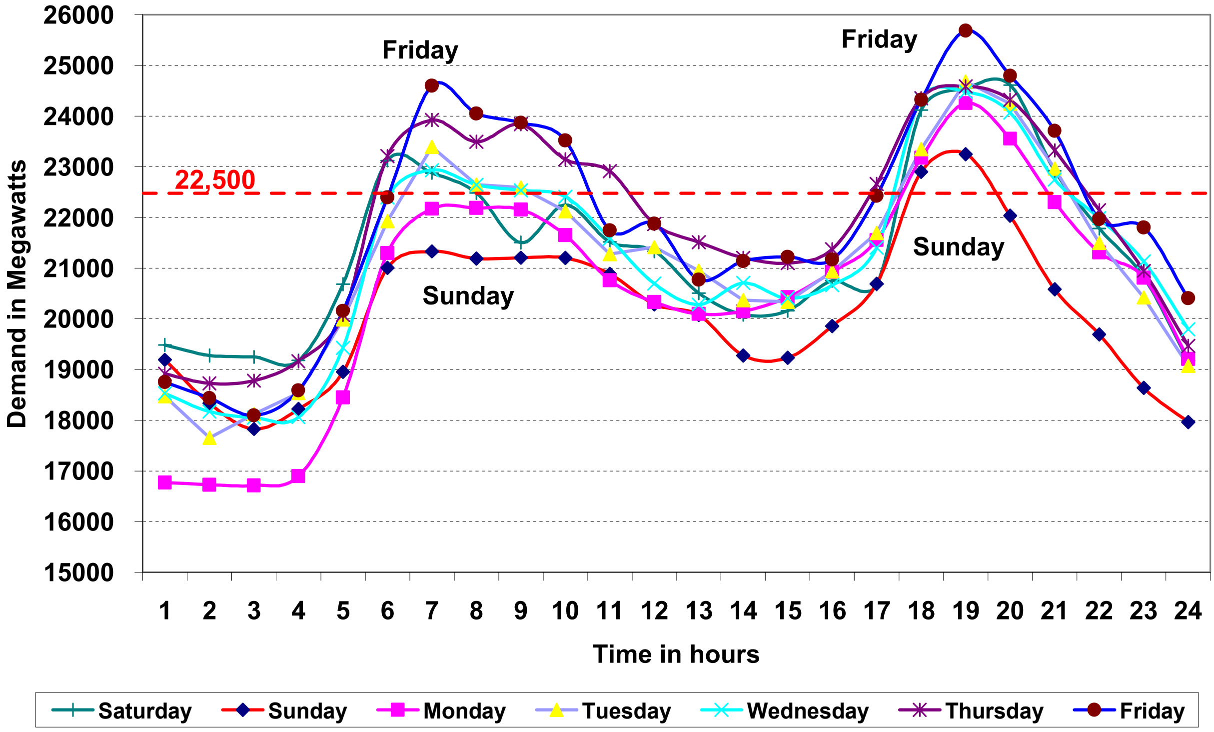
(1) gives $p = 0.187 \cdot V_0$

$$\boxed{p = \overline{BC} = 14.25\,[M \cdot m^3]} \text{ and}$$

$$\boxed{P = 2 \cdot p = 28.50\,[M \cdot m^3]}$$

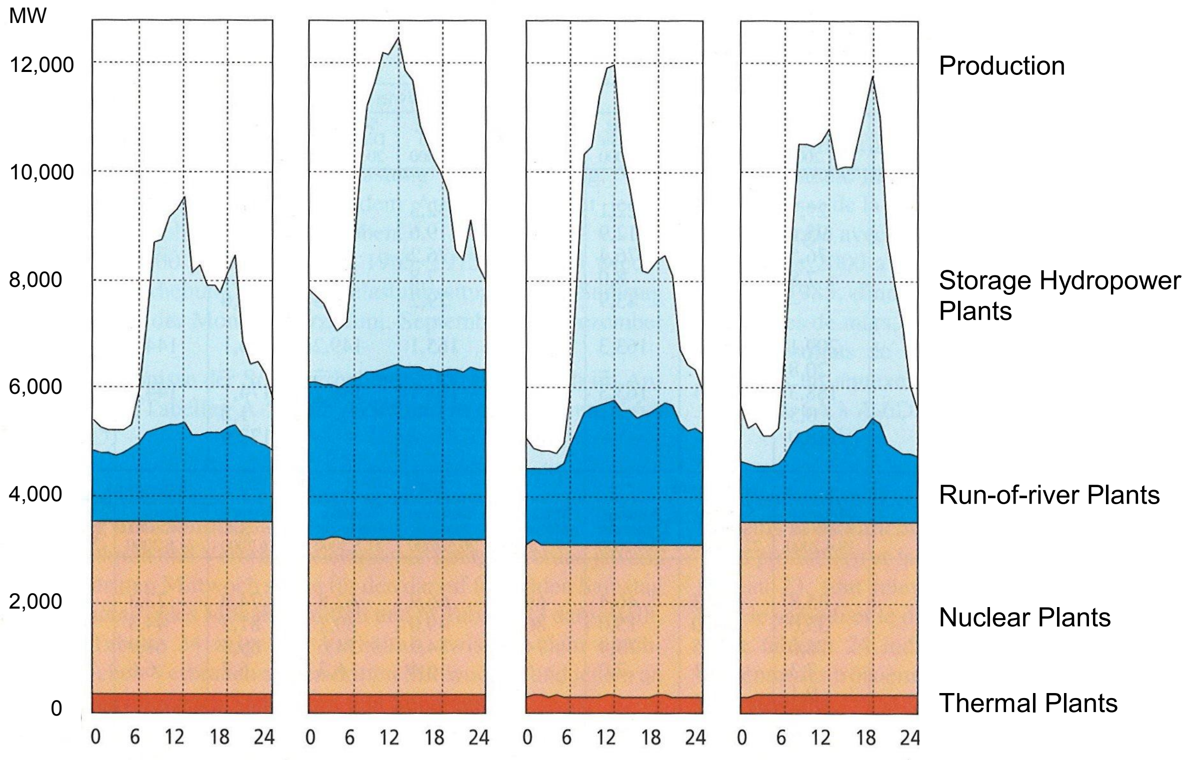
$$\boxed{h_p = 63\,[h] = 2\ days\ and\ 15\ hours}$$

Daily Load Curves in Northern Region
representative week in December 2004



Source: India's Planning and Model Test Document - Volume 5(v): Reply to Questions posed by the Neutral Expert, pp. 10-18
Figure: Extract from India's presentation on Pondag e, 20th October 2005

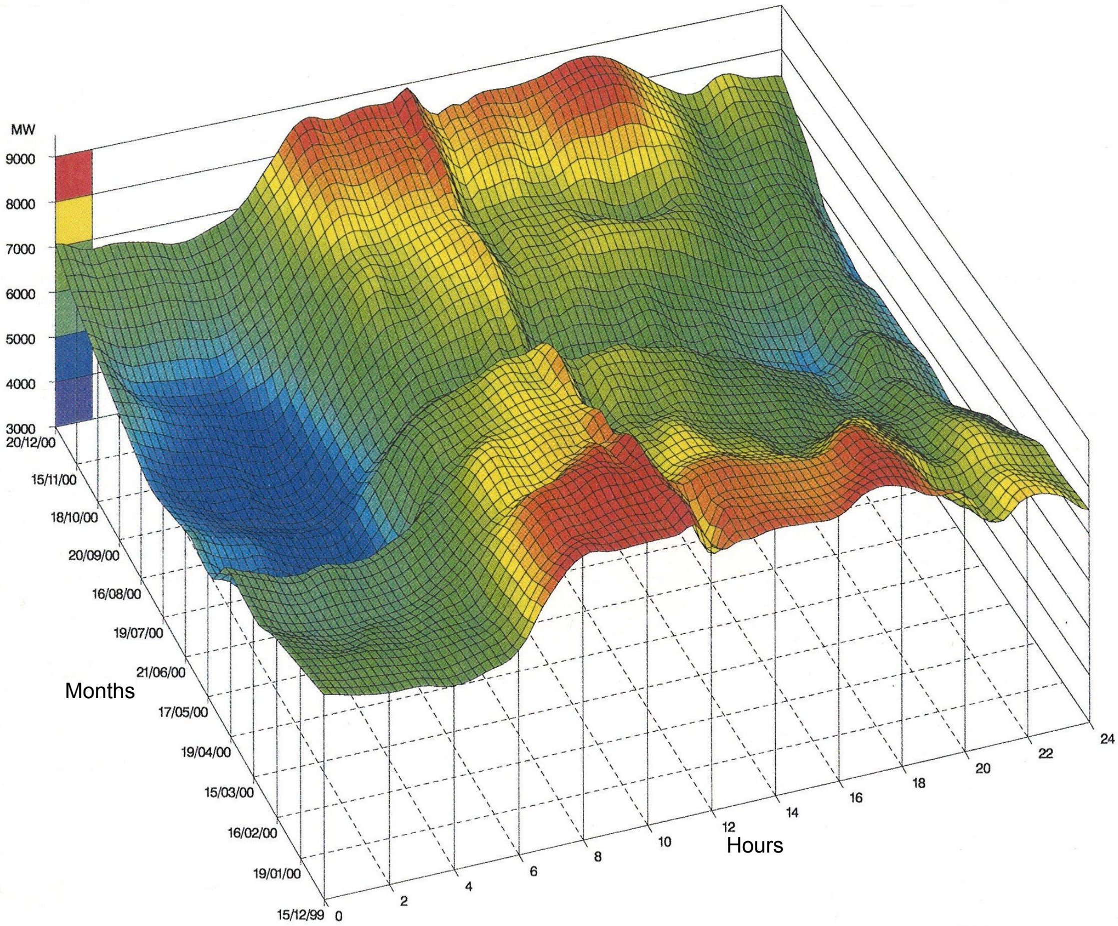
Load curves in Switzerland, 2000
Characteristic working day load curves (Wednesday)



Source: Bulletin of the VSE - Society of Swiss Electricity Companies, 12/2001, *Schweizerische Elektrizitätsstatistik 2000*

Load curve in Switzerland, 2000

Daily and monthly load curve of the Swiss power plants, 2000



Source: Bulletin of the VSE - Society of Swiss Electricity Companies, 12/2001, *Schweizerische Elektrizitätsstatistik 2000*

Pondage calculation done by the NE
Numerical values

Day			Cumulated volume		Difference
Starting on Saturday at 8 A.M., according to the Treaty		Time [hour]	Inflow V_e [M.m^3]	Turbined Outflow V_t [M.m^3]	V_e-V_t [M.m^3]
1	Saturday - Sunday	8	0.00	0.00	0.00
		17	4.07	0.00	4.07
		21	5.88	6.19	-0.31
		24	7.24	6.19	1.05
		32	10.86	6.19	4.67
2	Sunday - Monday	41	14.93	6.19	8.74
		44	16.29	10.86	5.43
		56	21.72	10.86	10.86
3	Monday - Tuesday	65	25.79	10.86	14.93
		68.62	27.43	16.44	10.99
		78	31.67	16.44	**15.23**
		80	32.58	19.54	13.04
4	Tuesday - Wednesday	82	33.48	22.63	10.85
		89	36.65	22.63	14.02
		93.62	38.74	29.79	8.95
		96	39.82	29.79	10.03
		101.5	42.30	29.79	12.51
		104	43.44	33.66	9.78
5	Wednesday - Thursday	106	44.34	36.76	7.58
		113	47.51	36.76	10.75
		117.12	49.37	43.14	6.23
		125	52.94	43.14	9.80
		128	54.29	47.78	6.51
6	Thursday - Friday	130	55.20	50.87	4.33
		136.88	58.31	50.87	7.44
		141.5	60.40	58.03	2.37
		149.5	64.02	58.03	5.99
		152	65.15	61.9	3.25
7	Friday - Saturday	154.5	66.28	65.77	0.51
		161	69.22	65.77	3.45
		165.12	71.09	72.14	**-1.05**
		173.5	74.88	72.14	2.74
		176	76.01	76.01	0.00
			Pondage : (15.23 + 1.05) =		16.28

Pondage calculation done by the NE
graphical presentation

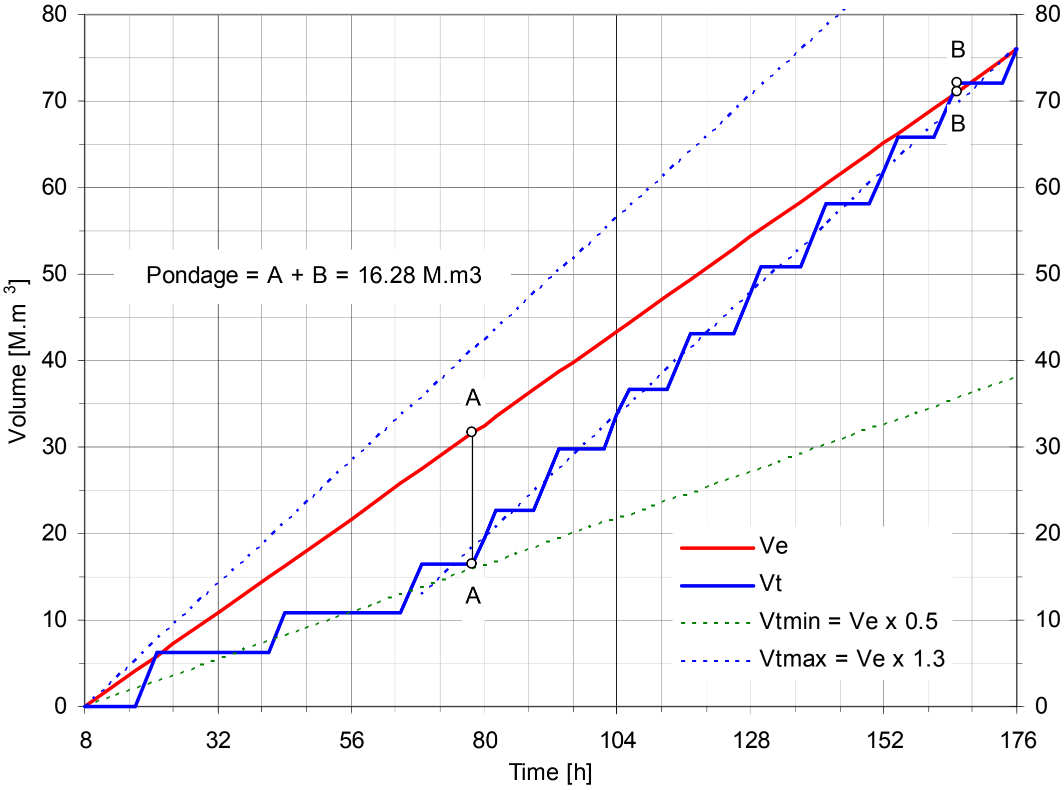


Figure 1: Mass Curve of river inflow and discharge through the turbines