RMD CONSULT

Tiger Ltd.

Initial Assessment of

SHPP Agatobwe, Rwanda

Findings and Recommendations

SHPP Agatobwe, RW, penstock and powerhouse

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1 OBJECTIVE

Objective of the services described in this report is the first assessment of Agatobwe HPP conditions and recommendation of measures for rehabilitation and upgrade of the power plant.

The following tasks have been agreed on:

- Geodetic survey (elevation) along pipeline in order to verify gross head
- Geodetic survey (elevation) of powerhouse and tailwater as a basis for further considerations for turbine placement, reconstruction of tailwater and final gross head
- Survey of powerhouse and analysis of existing structure as basis for recommendations for civil works (adjusting structures for new turbine and tailwater)
- Analysis of penstock and intake structure, considering potential increase of discharge
- General analysis and recommendations for refurbishment of structures
- Evaluation of sediment treating, recommendations for adjustment
- Summarizing recommendations and design parameters as basis for upgrade and refurbishment of HPP Agatobwe
- Outlining of next steps
- Brief report about findings and recommendations

2 SITE VISIT

A site visit took place on 21st and 22nd October 2017. On the site visit RMD CONSULT was accompanied by local staff.

A photo documentation of site visit is attached as annex 1 to this report.

A detailed leveling and survey of power plant facilities and alignment was carried out. Elevations are presented in annex 2, sketches of buildings are attached in annex 3.

3 SITUATION OF POWERPLANT

The Small Hydropower Plant (SHPP) Agatobwe is located at river Agatobwe, Nyaruguru district, near the southern border of Rwanda towards Burundi.

The HPP has been erected from 2006 to 2009, funded by the United Nations. The Power Plant is feeding a 6.6 kV minigrid connecting 2 small villages and the border station towards Burundi.

Main coordinates:

- Powerhouse: S2° 45.836' E29° 40.207'
- Weir/Diversion Intake: S2° 45.584' E29° 40.274'
- Forebay: S2° 45.802' E29° 40.192'
- Main road junction to powerhouse: S2° 45.986' E29° 40.336'
- Main road junction to intake: S2° 45.817' E29° 40.268'

At present the electrical production can only follow the demand in the existing 6.6-kV minigrid. Therefore the power output varies from 10 to 50 kW.

Due to the low demand and type of consumers the grid load is asymmetric. Consequently a high output of reactive power can be assumed.

A 30-kV overhead line for connection to public grid has been erected earlier in 2017. The first pile is located close to the existing transformer. The connection from the grid to the power plant has not been implemented yet.

There are neither any drawings nor reports of the project development and construction phase. There is not any documentation of operation of the power plant existing.

The total staff of the power plant is six persons. Three persons are working as operator of the power plant in 24-hour shifts. Another three persons are maintaining the intake, are cleaning the trash rack, etc.

4 CONDITIONS

The civil structures of the power plant are of simple style but functional and without obvious damages. A remaining lifetime of structures >10 years can be assumed underlying frequent maintenance and absence of disastrous events.

The Quality, performance and maintenance of the electromechanical equipment are poor. The electrical installation and mechanical protection has to be considered unsafe in terms of health and safety. The electromechanical equipment is neither applying automation nor any monitoring devices.

The constitution of the electrical equipment (transformer, grid connection) is poor in terms of safety and environmental issues. Switches, protection and oils sump are missing.

The performance of operation and maintenance is poor. The knowledge of operational staff about equipment and maintenance requirements seems to be low. An operational manual does not exist.

E.g. the last flushing of sediments out of the forebay took place 4 month ago in June 2017 while the frequency of flushing should take place every 2 weeks.

The elevations of existing facilities and alignment were determined by geodetic leveling. The results are shown in annex 2. Main values are shown in Table 1. The elevations are given relative to powerhouse floor (+/- 0.0 m)

Table 1: Relative elevations

Location	**parameter**	**Elevation (relative)**
Weir, spillway crest	Nominal upstream water level	+ 25. 27 m
Diversion canal intake spillway	Nominal intake water level	+ 24.86 m
Forebay, spillway crest	Headwater level	+ 22.79 m
Turbine/Generator axis	Turbine axis	+ 1.22 m

Powerhouse floor	PH reference	+/- 0.00 m
Turbine outlet control weir	Minimum tailwater level	- 1.11 m
Tailwater river	Downstream water level	- 4.75 m

Accordingly the hydraulic system can be described as follows:

Table 2: Elevations

Head losses at headrace canal	2.07 m
Gross head (forebay – powerhouse):	23.90 m
Remaining height to tailwater	> 3 m

The alignment of the hydraulic system is as follows:

Table 3: Alignment length

Open diversion canal at intake	6.85 m
Covered canal	439.7 m
Open canal	17.90 m
Forebay/desander	14.80 m
Penstock DA 885 mm (anchor block to mid of reducer)	14.15 m
Penstock DA 720 mm (up to main intake valve)	22.07 m

The alignment of the headrace canal is shown bellow.

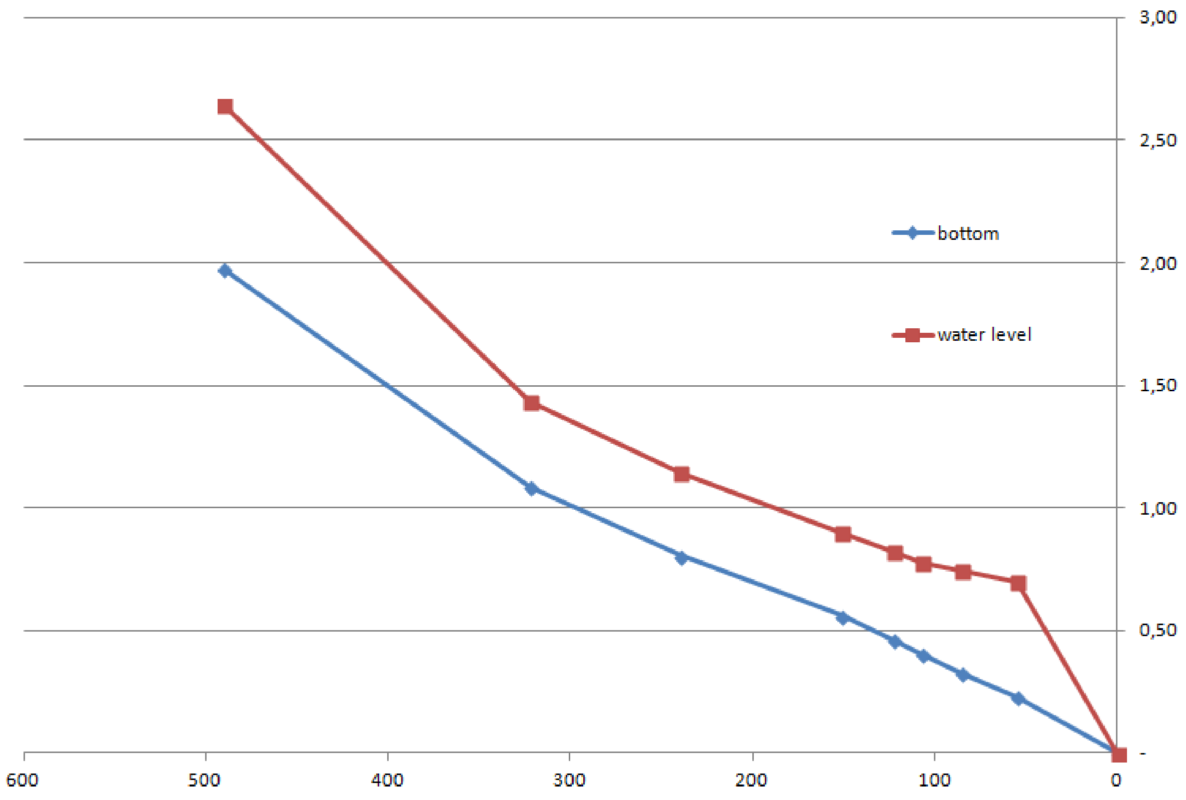


Figure 1: Alignment of the headrace canal

The flow conditions in the canal are varying due to bottom inclination, width, sediment and plant growth (maintenance conditions). In the middle section the flow conditions seem to be quite uniform. Therefore this section is taken into account for the recalculation of the discharge.

Discharge during site visit (22.10.2017) can be estimated as follows:

- Weir discharge: varying from 0 to 100 l/sec (estimation)
- Canal discharge (defined by intake gate): approx. 600 – 650 l/sec, according to flow conditions at manhole #6 and manhole #7
- Spillway flow at forebay: approx 80 l/sec according to leveling of spillway crest and water level, see Table 4

Table 4: Calculation of spillway flow at forebay (22.10.2017)

Spillway discharge			
	left	rigth	
Length [cm]	310	475	
elevation [m, relative]	22,79	22,8	
water level (22.10.2017)	22,88		m, relative
µ	0,62	0,62	-
length, b	3,1	4,75	m
$h_{ü}$	0,09	0,08	m
discharge, Q	35	44	lit/sec
total discharge, Q	79		lit/sec

Therefore the turbine discharge can be estimated to 520 – 570 l/s, which equals approx. 50% of the design flow.

Accordingly the mechanical capacity is calculated:

P = 9.81 x (24 – 1) m x 0.52 m³/s = approx. 117 kW.

The active power feeded into the grid has been 10 kW. Therefore the total efficiency of the system is less than 10% under these conditions.

The following reasons can be assumed for the poor performance of the system:

- Low mechanical and electrical efficiency of turbine and generator due to quality reasons, especially poor performance in part load conditions
- Power consumption in the minigrid far lower than capacity of power plant. Electricity used to load ballast tank
- Asymmetric load in minigrid due to consumer requirements. Eventually large amount of reactive power fed into grid.

5 STEPWISE APPROACH

In order to utilize as soon as possible the full hydraulic potential of the site for hydropower production a stepwise approach is recommended:

Step	Action	Measures / Equipment
1	Connect minigrid to public grid as precondition for refurbishment and upgrade of power plant as well as unbundling of power consumption and production	New transformer 6.6/30-kV, 200 kVA and 30 kV switchgears
2	Feed into public grid with existing power plant in order to utilize full hydraulic potential of existing turbine Note: Isolated network is not possible any more.	Expansion of 30 kV switchgear: generator switch, auxiliary power switch, possibly metering New main transformer 0,4/30 kV, 450 kVA. New auxiliary transformer 0,4/30 kV. Synchronization equipment, protection, automation, headwater level probe. Detailed analysis of grid code and existing equipment required.
3	Upgrade of power plant to 400 kW installed capacity, feeding into public grid	Replace electromechanical equipment (turbine, generator, control equipment) Extend headrace canal or replacing by closed pipe system. New tailwater outlet in order to increase suction head of turbine

Step No. 1 can be considered as a precondition for any further step, as there is no alternative for power production in order to supply the minigrid and border facilities with power. Therefore, before dismantling of equipment and upgrade measures of the power plant the minigrid has to be connected to the public grid. Besides this circumstance, the connection to the public grid is the only way to separate the power production in the power plant from the power consumption in the minigrid.

A detailed analysis of the minigrid and requirements for connection to the public grid shall be carried out by a local electrical engineer. Design of transformer and switchyard is subject to this study.

The costs for step No. 1 arise mainly from delivery of the electrical equipment and can estimated roughly from 30 to 50 kEUR.

In step No. 2 the connection of power plant to the public grid has to be implemented. It is recommended to connect the power plant directly to the public 30 kV grid and not to the 6.6 kV minigrid, as there would by higher requirements in terms of electrical equipment for operation in isolated minigrid. It has to be crosschecked, whether isolated operation is an obligation of the PPA or not.

Similar to the proposed analysis of step 1 requirements of switchyard shall be determined by a local electrical engineer based on the knowledge of grid code.

As an option a temporary connection of the existing power plant to the public grid can be considered in this phase to cover the period of preparation for upgrade until start of dismantling. In this case step 2 can provide also additional knowledge about river flow. A detailed analysis of the grid code and existing electrical equipment is required to determine required measures for synchronization an protection during this temporary operation. Eventually the electrical equipment has to be renewed for this step in order to be able to adjust power production to grid requirements as well as to have the opportunity of disconnecting from the grid and decommissioning of the turbine for step No. 3.

The costs for step No. 2 cannot be forecasted, as measures are subject to detailed investigation of the electrical equipment and grid code. The costs for the switchyard, which is even required for step 3, may vary from 60 to 100 kEUR.

Optional refurbishment of existing electrical equipment for temporary operation is estimated 30 to 50 kEUR. Decision has to be based on economic analysis of measured and power production in the period of project preparation, assuming a average power output of 100 kW with existing equipment feeding to public grid.

For design, dismantling of equipment, construction, delivery and installation of new equipment and relating civil works a period of 15 to 18 month has to be assumed in step No. 3. The new equipment shall feed into the public grid directly only, because opportunity of feeding to isolated minigrid will arise in significant additional cost as mentioned before.

The costs for step No. 3 are subject to further detailed upgrade studies. Even a replacement of powerhouse has to be considered as there are major construction works at substructure and superstructure required to place new equipment into powerhouse. New powerhouse would also provide mere flexibility in terms of turbine layout.

A rough estimate of civil work cost is given in the following chapters.

Due to the conditions of existing structures additional measures have to be considered, as:

- Refurbishment or replacing of powerhouse roof
- Sealing of powerhouse, e.g. electrical room in order to prevent new equipment from dust and insects
- Improvement of access road: 3 culverts, lateritic top cover
- Reducing slope at access to power plant and placing ditch ahead in order to prevent sediments being washed into the powerhouse area
- Preparation of operational manual and training of staff

The cost for additional measures can be estimated roughly from 50 to 100 kUSD.

6 UPGRADE POTENTIAL

6.1 Estimation of flow duration curve

The catchment area has been estimated using the specialized software Whitebox GAT 3.4, developed by the University of Guelph, Canada.

As input for the software ASTER (Advanced Spaceborne Thermal Emission and Reflection) global digital elevation models were used. This data is provided jointly by the U.S. National Aeronautics and Space Administration (NASA) and the U.S. Geological Survey (USGS).

According to the results of the simulation, the catchment area for the hydropower development is around 143 km². The catchment area is represented in Figure 2.

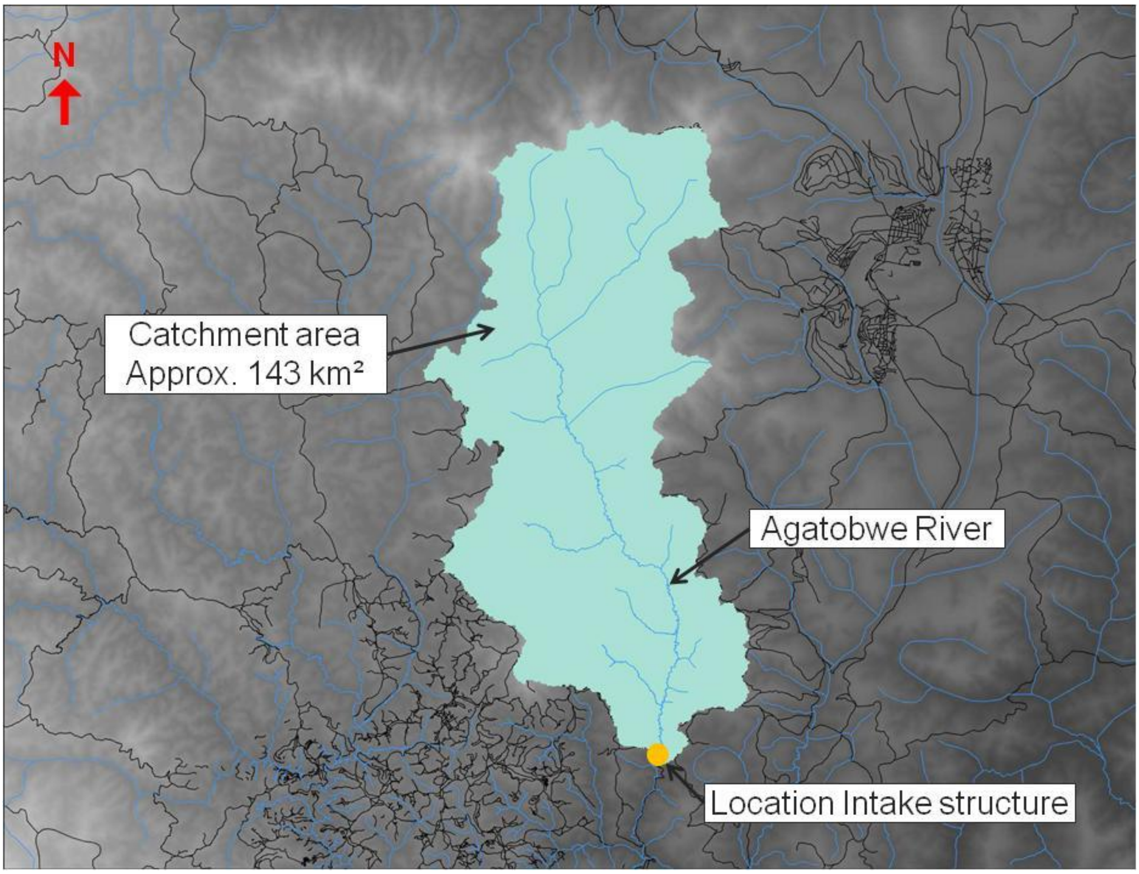


Figure 2: Catchment area Agatobwe HPP

Due to the lack of reliable information regarding the discharge of the river, a flow duration curve has been estimated based on a hydrological study carried out in the year 2014 for the planning of the hydropower plant Rukarara V, located around 40 km north-west of the hydropower plant Agatobwe.

The flow duration curve for this project has been regionalized using two coefficients, namely, a catchment area coefficient (C_A) and a precipitation coefficient (C_P). In the following a short description of the coefficients is given:

- Coefficient area (C_A): the ratio between the catchment areas of the two hydropower developments has been used to determine this coefficient

$$C_A = \frac{Area_{Agatowbe}}{Area_{Rukarara\ V}} = \frac{143\ km^2}{287\ km^2} = 0{,}5$$

- Coefficient precipitation (C_P): the ratio between the average yearly precipitation for the stations Kigembe (located 5 km north-east from the intake of the power plant Agatobwe) and Mushubi (located 10.5 km north from the intake of the power plant Rukarara V) has been used to determine this coefficient

$$C_P = \frac{Average\ yearly\ Precipitation_{Kigembe}}{Average\ yearly\ Precipitation_{Mushubi}} = \frac{1198\ mm}{1501\ mm} = 0{,}8$$

In Table 1 and Figure 3 are the values and the graphic representations of the flow duration curves for Rukarara V and Agatobwe to be found.

Table 1: Flow duration curves Rukarara V and Agatobwe

% time	Flow [m³/s]		% time	Flow [m³/s]		% time	Flow [m³/s]	
	Rukarara V	Agatowbe		Rukarara V	Agatowbe		Rukarara V	Agatowbe
2.5%	16.38	6.55	35.0%	4.94	1.98	67.5%	3.42	1.37
5.0%	12.56	5.02	37.5%	4.86	1.94	70.0%	3.29	1.32
7.5%	10.04	4.02	40.0%	4.65	1.86	72.5%	3.18	1.27
10.0%	8.81	3.52	42.5%	4.54	1.82	75.0%	3.17	1.27
12.5%	8	3.2	45.0%	4.39	1.76	77.5%	3.04	1.22
15.0%	7.4	2.96	47.5%	4.24	1.7	80.0%	3.03	1.21

17.5%	6.89	2.76	50.0%	4.13	1.65	82.5%	2.93	1.17
20.0%	6.45	2.58	52.5%	4.1	1.64	85.0%	2.88	1.15
22.5%	6.07	2.43	55.0%	3.95	1.58	87.5%	2.81	1.12
25.0%	5.71	2.28	57.5%	3.86	1.54	90.0%	2.81	1.12
27.5%	5.53	2.21	60.0%	3.73	1.49	92.5%	2.59	1.04
30.0%	5.34	2.14	62.5%	3.68	1.47	95.0%	2.53	1.01
32.5%	5.19	2.08	65.0%	3.55	1.42	97.5%	2.38	0.95

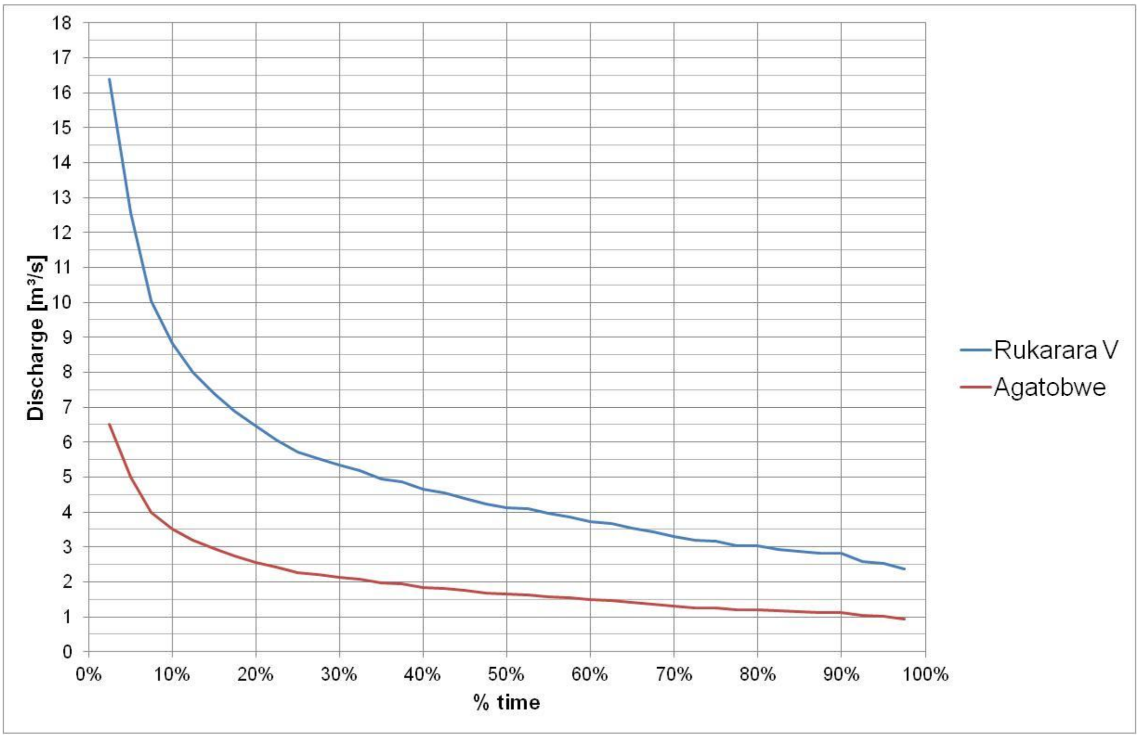


Figure 3: Flow duration curves for Rukarara V and Agatobwe

The flow duration curve has to be considered as a rough estimate in this stage of project development.

During site visit a discharge of approx. 0.7 to 0.8 m³/s has been observed which would be near minimum flow according to this duration curve. Therefore a detailed monitoring of discharge and eventually installation of a gauging station is recommended.

6.2 Measures for the optimization of the power plant

In order to increase the power output of the power plant either the discharge Q or the net head H have to be increased or a combination of both, following the equation

$P = \eta * g * Q * H$

Q an H are linear factors for the Power output P. Therefore in order to meet the proposed power output of 390 kW given in the PPA the product of Q and H have to be double of the present design parameters.

Several measures have been taken into consideration, in order to optimize the output of the existing hydropower plant. The measures have impacts on the different elements of the hydropower complex (see Figure 4).

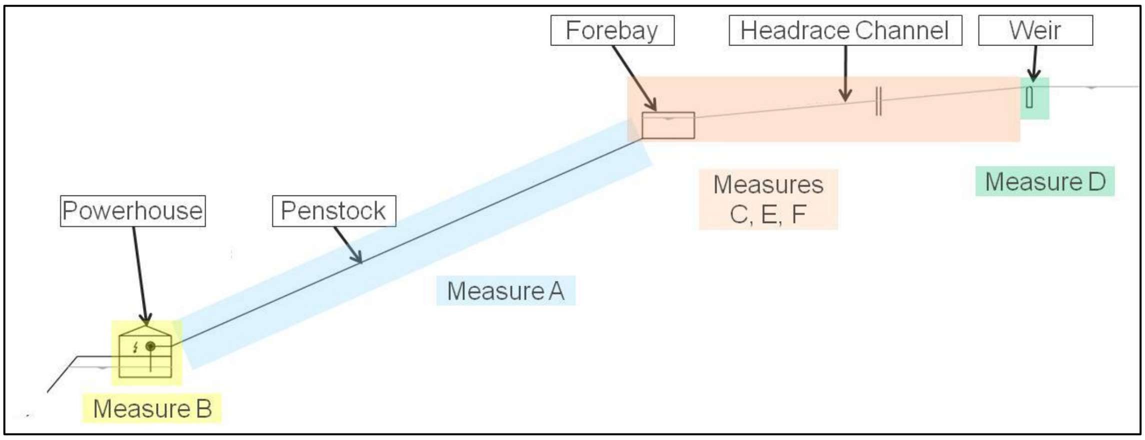


Figure 4: Schematic overview of the optimization's measures

In the following points a description of each measure will be given.

Measure A – Reducing pressure losses in the existing penstock:

Measure A consists of reducing the pressure losses in the existing penstock, by replacing it with a pipeline with a larger diameter.

The total replacement of the existing structures would be necessary, if carrying out this measure, including the new construction of the intake, anchorage blocks, stilts and connection to the powerhouse.

According to estimation, the losses in this short segment of the penstock are less than 0.6 m and approx. 1.5 m for a discharge of 1.9 m³/s (see Figure 5), which doesn't give much way for improvement.

Q	d	L	k	ΔH
m³/s	m	m	mm	m
1,15	0,87	14,20	0,05	0,133
1,15	0,70	22,10	0,05	0,407

Q	d	L	k	ΔH
m³/s	m	m	mm	m
1,90	0,87	14,20	0,05	0,371
1,90	0,70	22,10	0,05	1,094

Figure 5: Estimation of losses along the penstock

Due to the constructive complications and the very little potential for improvement, it is recommended to reject this solution.

Measure B – Lowering the tailwater level:

According to the leveling carried out during the site visit, the existing tailwater level is located 2.3 m below the level of the axis of the turbine. In order to avoid cavitation in the turbine, it has been estimated by manufacturer that the suction head can be maximum 4 m considering the foreseen operational values.

The proposed measure is to lower the tailwater level by 1.5 m by excavating underneath the existing powerhouse to expand the tailwater.

Therefore a new substructure has to be implemented, exceeding boundaries of the existing outlet (see schematic sketch in Annex 4).

The costs for civil works can be estimated to approx. 30 to 50 kEUR.

Replacing the powerhouse would provide opportunity to have a turbine with vertical shaft, which can arise in additional suction head of approx. 0.5 to 0.8 m.

Measure C and F – Reducing pressure losses in the headrace canal:

According to the leveling carried out during the site visit in October 2017, there is a difference of approximately 2 m between the water level at the beginning of the headrace canal, and the water level at the intake of the penstock.

The losses in the headrace canal can only be reduced by changing the system from an open-canal to a low pressure pipeline.

Measure C is dealing with the existing discharge, whereas measure F is considering an extended discharge of 1.9 m³/s.

Following basically the same alignment of the existing canal, a pipeline with a diameter of 1.2 m (C) to 1.4 m (F) can be installed in the same position of the current canal. An estimation of the losses using steel as material for the pipeline has been carried out. The result shows that the losses would be around 0.3 to 0.8 m (see Figure 6), which would bring an increase of approx. 1.2 to 1.7 m of net head.

Q	d	L	k	ΔH
m³/s	m	m	mm	m
1,15	1,20	450,00	0,01	0,312
1,90	1,40	450,00	0,01	0,393
1,90	1,20	450,00	0,01	0,799

Figure 6: Estimation of losses along the pipeline

Additionally it can be necessary to build a sandtrap at the intake of the pipeline, in order to reduce the amount of sediments entering the pipeline. A surge chamber between the low pressure and the high pressure segments of the penstock is necessary.

The costs can be roughly estimated as follows:

- Sandtrap and new intake at intake: 50 to 80 kEUR
- Replacing forebay by surge tank: 20 to 30 kEUR
- Replacing canal by pipeline: 300 to 400 kEUR

Even so changing the open canal to closed pressure pipeline is a precondition for increasing the Headwater Level of the turbine (Measure D).

Measure D - Increasing water elevation at the weir:

An increase of the crest of the weir should be possible with relatively simple modifications of the existing structure.

When increasing the elevation of the weir, it has to be taken into consideration, that the water level during a flood event must be low enough in order to assure that the bridge located upstream of the weir won't be damaged.

Currently, during normal operation, the lower edge of the bridge is located approximately 2.7 m above the water level.

Assuming an overall height of 1 m above the crest of the weir during floods and allowing a free board of 0.5 meters below the bridge, the operational water level can be increased by a maximum of 1.2 m (see Figure 7).

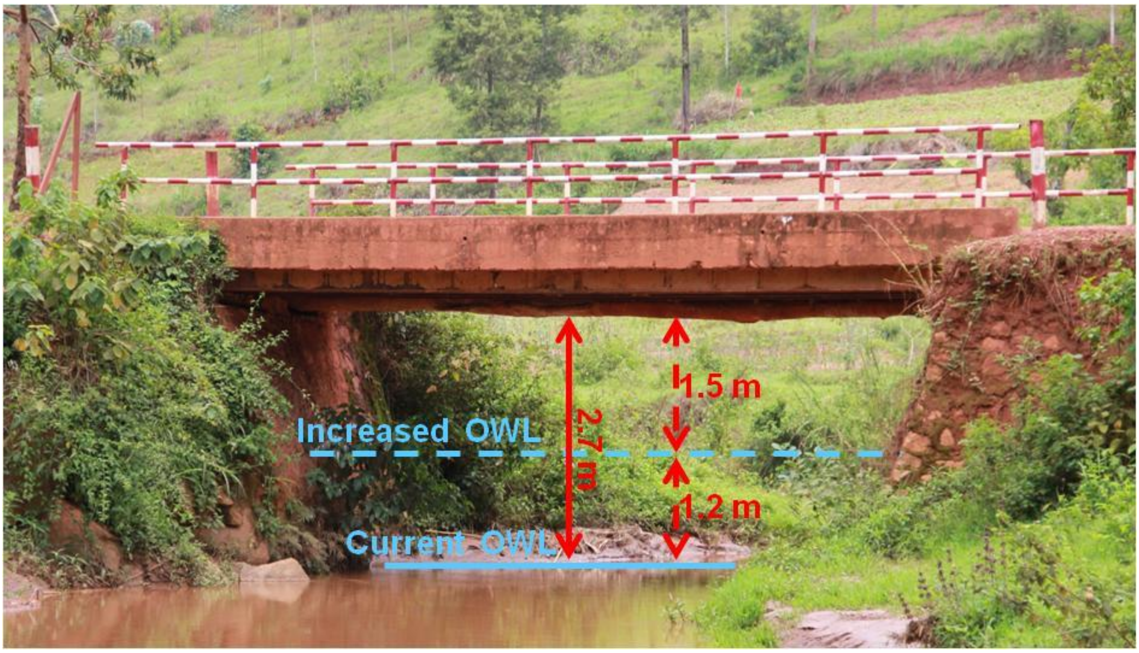


Figure 7: Increase of the operational water level

This measure is just feasible if carried out together with Measure C (changing the headrace canal by a low pressure pipeline), as changing the open canal to a closed pressure pipeline (Measure C) is a precondition for increasing the Headwater Level.

Extending height of canal sidewalls seems not to be feasible according to higher friction head loss and tightness of existing structure.

Alternatives shall be subject to further studies.

The costs for heightening the weir can be estimated rightly from 50 to 80 kEUR.

Measure E – Increasing the discharge:

According to the estimated hydrological data, there is still potential to increase the discharge of the system. Nevertheless the existing canal has a capacity of 1.15 m^3/s with a calculated flow depth of 0.9 m. In order to be able to increase the discharge flowing to the turbines an enlargement of the canal is required.

Therefore the canal has to be dismantled completely and has to be replaced by a new canal in the same alignment providing larger cross-section.

As shown in the flow duration curve in Figure 8, a discharge of 1.9 m^3/s is available around 38 % of the time, whereas the existing design discharge is exceeded around 84% of time. Even if the duration curve may be overestimated by 25% the proposed design discharge of 1.9 m^3/s would be still in a common range of more than 20% exceeding time.

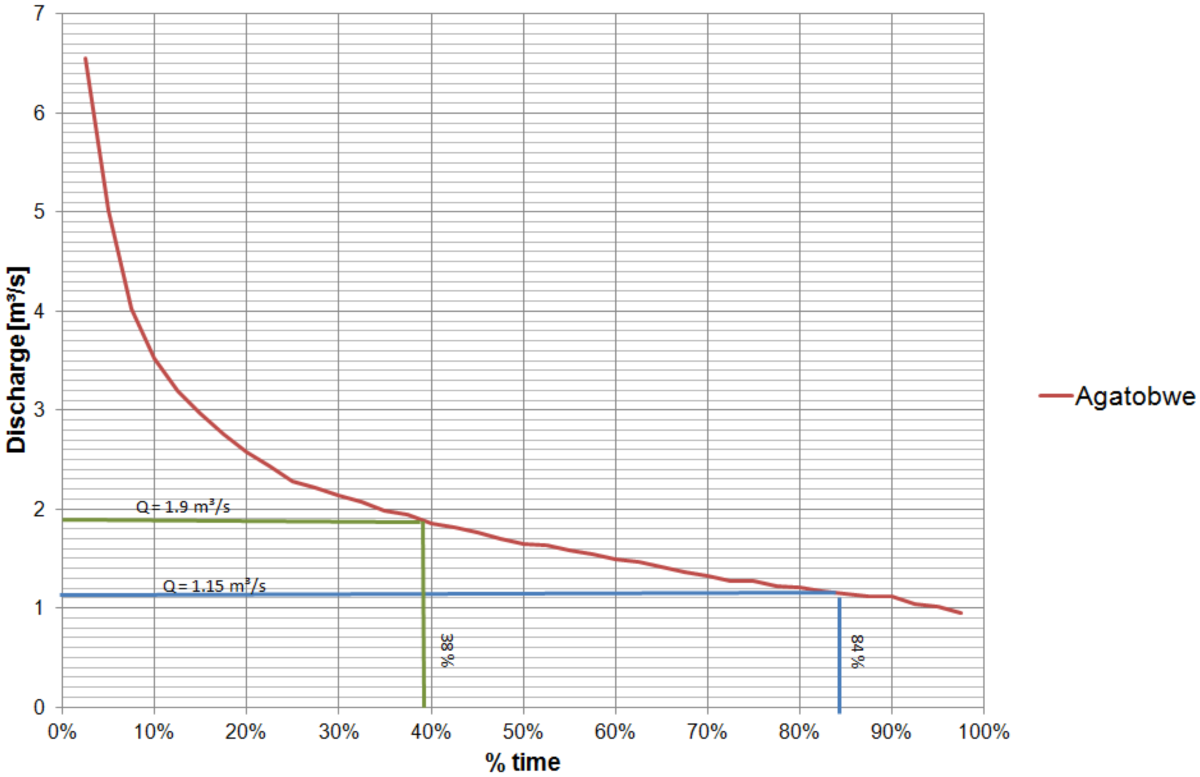


Figure 8: Flow duration curve Agatobwe

As mentioned before, the existing canal doesn't have enough capacity for an increase of the discharge. Therefore an expansion of the canal is necessary. With a width of 1.5 m and the same longitudinal slope as the existing canal, the transport capacity for a discharge of 1.9 m³/s will be assured (see Figure 9).

Calculation of flow parameters shows a required width of 1.4 to 1.5 m for the new canal, assuming same slope and flow depth as the existing once.

Q	h	I	k_{st}	b_s
m³/s	m	$^0/_{00}$	$m^{1/3}/s$	m
1,99	0,90	3,5	50	1,40
2,20	0,90	3,50	50	1,50

Figure 9: Estimation of normal flow conditions

Nevertheless it must be considered, that increasing the discharge will also increase the energy losses in the existing penstock.

Due to higher discharge the existing desander at the forebay may be assumed not sufficient in order to protect turbine from sediments. Therefore in case of increasing discharge also increasing of the forebay has to be considered.

The costs can be roughly estimated as follows:

- Replacing canal: 200 to 300 kEUR
- Increasing forebay: 30 to 50 kEUR

Measure F - Increasing discharge and reducing losses in the headrace canal:

See measure C.

New electromechanical equipment

For upgrade of power plant the complete electromechanical equipment (turbine, generator, control equipment, transformer, switches) has to be replaced.

Referring to survey of existing powerhouse and example of turbine with proposed design parameters the replacement of the turbine seems feasible while the existing building is quite narrow and doesn’t provide any place for maintenance.

In the sketches in annex 4 boundary conditions of existing powerhouse are shown in relation to turbine of proposed size.

Detailed arrangement shall be subject to further studies and design phases.

Even so due to major works required for the replacing of the turbine and refurbishment of the building a complete replacement of powerhouse can be considered.

7 SUMMARY

For the upgrade of the HPP Agatobwe a 3-steps-approach has been described, whereas step 1 (connecting minigrid to public grid) is a precondition for any further activity as there is no other available energy supplier for consumers in case of temporary stop of operation of the existing power plant for upgrade.

As a second step a connection of the power plant to the public grid has to be prepared. Connecting to the 6.6 kV minigrid is not recommended for future operation as there are higher requirements for electrical equipment in case of both modes of operation (public grid and isolated minigrid). Obligations given in the PPA have to be crosschecked accordingly.

In the period of preparation for upgrade measures temporary operation of existing power plant and feeding to public grid can be considered in step 2. Requirements and measures shall be subject to detailed investigation by electrical experts, e.g. manufacturer of equipment. Decision shall be based on required expenses and assumed average power output of 100 kW for temporary period.

Upgrading the power plant to the proposed capacity of 400 kW is feasible. The higher power output arises mainly from higher discharge and partly from slightly increase of gross head.

For upgrade of power plant two major alternatives can be considered:

<u>Alternative 1:</u> existing hydraulic system (canal, open surface flow)

- Increasing discharge from 1.15 to 1.9 m^3/s
- Extending width of canal, replacing by new canal
- Extending forebay due to desanding requirements
- Replacing electromechanical equipment
- Replacing turbine outlet to increase suction head by 1.5 m
- Total gross head is increased from 24 to 25.5 m
- Considering head losses, the net head will be increased from approx. 23 to 24 m

Alternative 2: changing hydraulic system to closed pipe (pressurized flow)

- Increasing discharge from 1.15 to 1.9 m³/s
- Increasing weir height by approx. 1 m
- New sand trap and intake at weir
- Replacing canal by closed pipe, diameter 1.4 m
- Replacing forebay by surge tank
- Replacing electromechanical equipment
- Replacing turbine outlet to increase suction head by 1.5 m
- Total gross head is increased from 24 to 28.5 m
- Considering head losses by friction, the net head is increased from approx. 23 to 25.5 m

Taking cost into account (see Chapter 8) alternative 1 seems to be the more economic solution.

8 COST

For replacement of electromechanical equipment (turbine, generator, control equipment, shipping, installation, commissioning, training etc.) an indicative value of 1 156 kEUR has been given by Kochendörfer & F.EE Hydropower GmbH on request.

The cost estimation can be summarized as follows:

Step 1		
switchyard 6.6/30 kVA	30 to 50 kEUR	
Step 2		
switchyard 0.4/30 kVa	60 to 100 kEUR (switchyard)	
option: replacing existing control equipment for temporary operation	(30 to 50 kEUR)	
Step 3		
	Alternative 1	Alternative 2
Tailwater, civil	30 to 50 kEUR	
Headrace, civil	230 to 350 kEUR	370 to 510 kEUR
Powerhouse, civil	50 to 80 kEUR	
Option: replacing power house, additional cost	(100 to 150 kEUR)	
Electromechanical equipment	1 156 kEUR	
Contingencies		
Additional measures	50 to 100 kEUR	
Ancillary, unforeseen	20 %	20 %
Total, net	**1 930 to 2 400 kEUR**	**2 100 to 2 600 kEUR**

9 RECOMMENDATIONS

A stepwise approach is recommended to connect local minigrid and power plant separately to the public grid. For future operation of the power plant a direct connection to public 30 kV grid is recommended. The required equipment and construction works for switchyards have to be evaluated by an electrical expert based on the knowledge of local grid and grid code.

The existing control equipment of the power plant can be analyzed in order to determine required measures for temporary feeding to grid with existing turbine. In case there are just minor activities of temporary refurbishment of electrical equipment, the preparation phase for upgrade can be used for power production with the existing turbine while gaining more information about range of discharge. Decision shall be based on economical analysis of this option.

According to the estimations of the size of the catchment area, an increase of the design discharge, in order to achieve an upgrading of the installed capacity to 400 kW, seems feasible. Due to minor effects on power output and high cost a change of hydraulic system to closed pipe seems not to be economically advantageous. Therefore the existing canal system shall be preserved. The width of the canal has to be extended according to increased discharge requirements.

Detailed analysis of the measures and cost shall be subject to further project development and design phases.

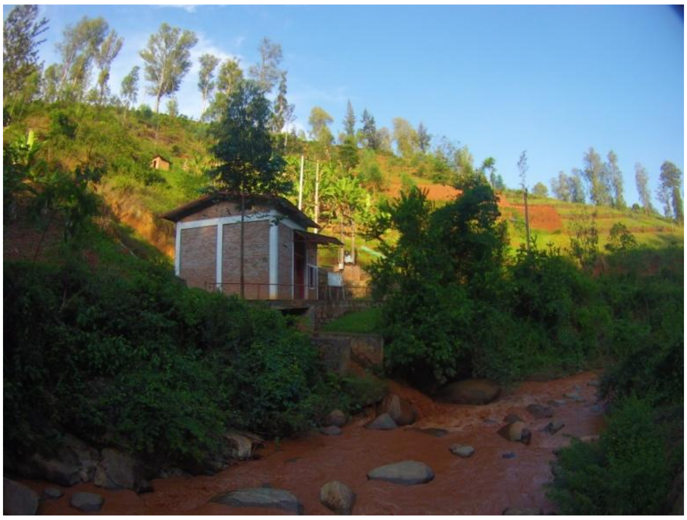
ANNEX

Annex 1	Photo documentation
Annex 2	Geodetic leveling
Annex 3	Sketch of powerhouse
Annex 4	Sketches of new turbine placement in existing building boundaries
Annex 5	Single line Diagram for Stage 2 and Stage 3

Annex 1 Photo documentation

penstock, powerhouse, forebay, *spillway*

penstock, powerhouse, tailwater



powerhouse, tailwater

turbine outlet

tailwater, outlet and river

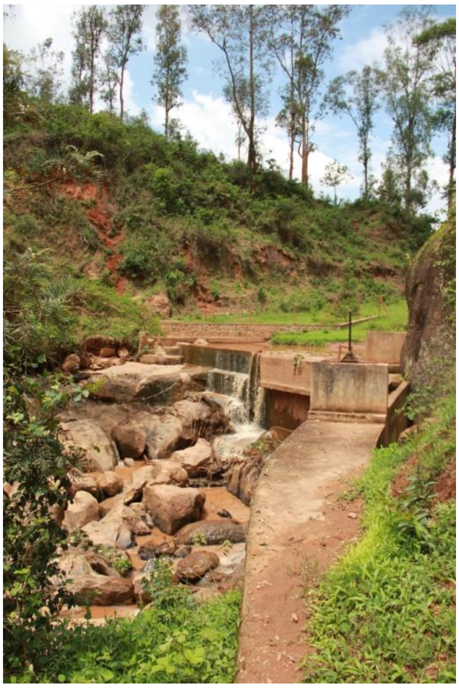
Powerhouse and transformer, south

powerhouse area, west

penstock, anchor block, power-house north

weir, downstream view

weir, diversion intake, spillway, main gate



weir, diversion canal, main gate

basin upstream of weir

road bridge upstream of weir

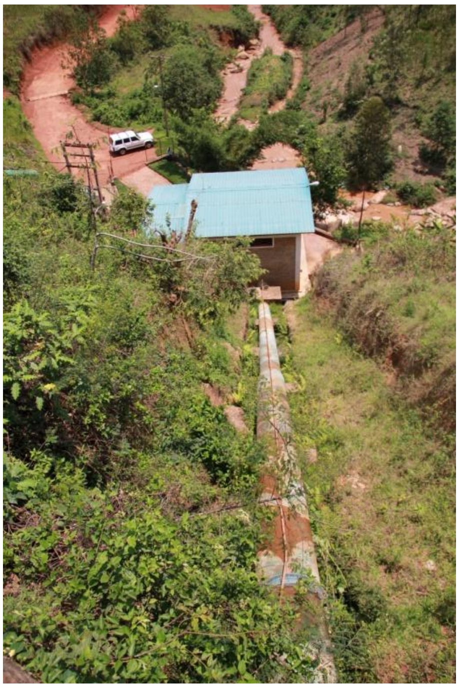
headrace canal

headrace canal alignment, manhole #1

headrace canal, manhole #1, plants, width 1 m, flow depth approx. 0.7 m

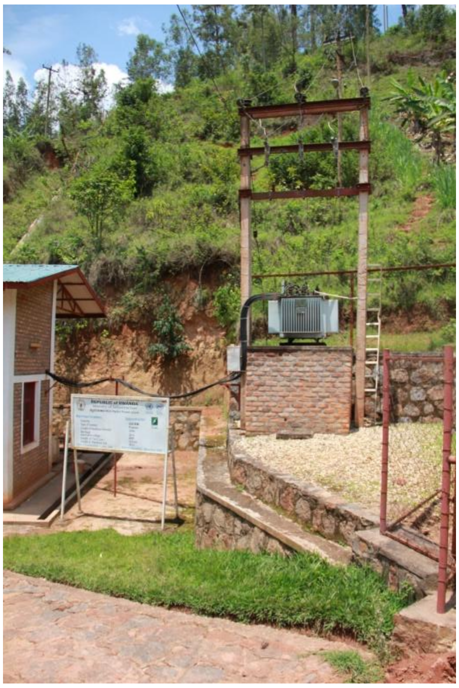
forebay spillway left (3. 1 m length)

forebay spillway right (4.75 m lenth)



penstock, powerhouse, DA 720/885 mm

powerhouse, south



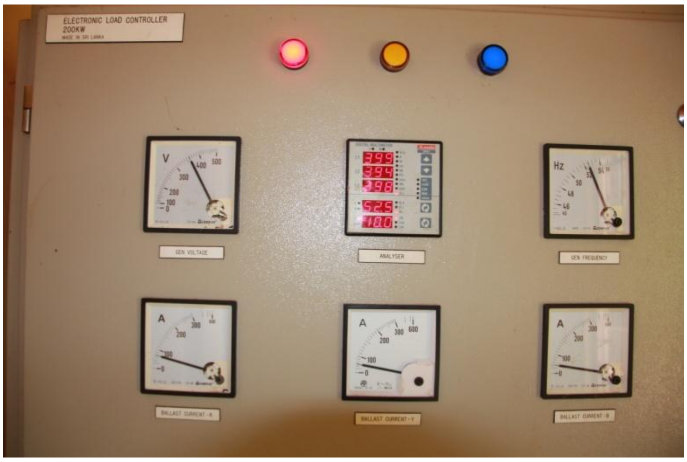
transformer, 6.6-kV minigrid

penstock inside ph, DA 720 mm

turbine and generator

turbine, generator

surge pipe



Instruments, performance
(21.10.2017)

electrical cabinet

load ballast tank, powerhouse noth-east

culvert, ph access road

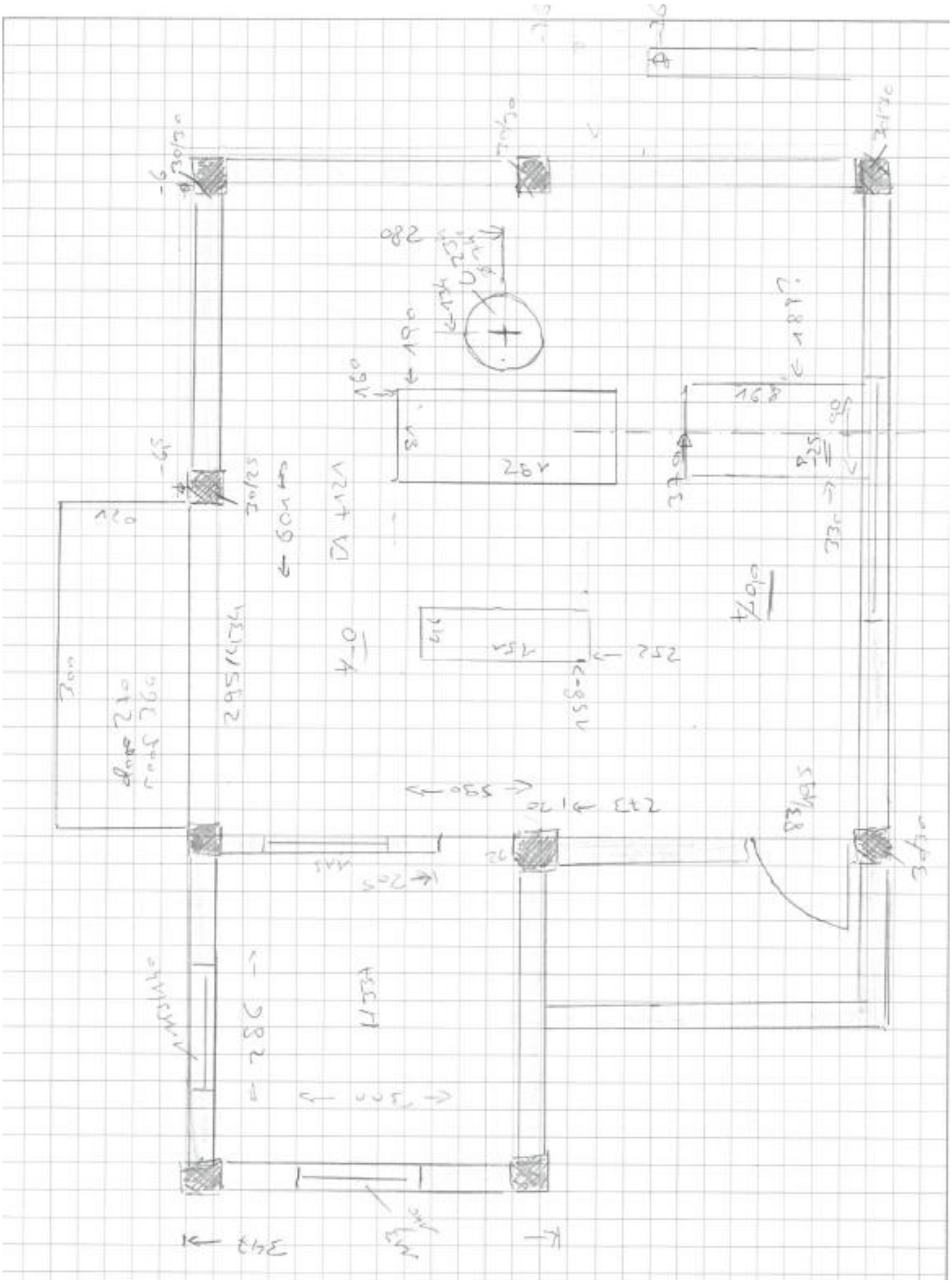
ditch, ph access road

Annex 2 Geodetic leveling

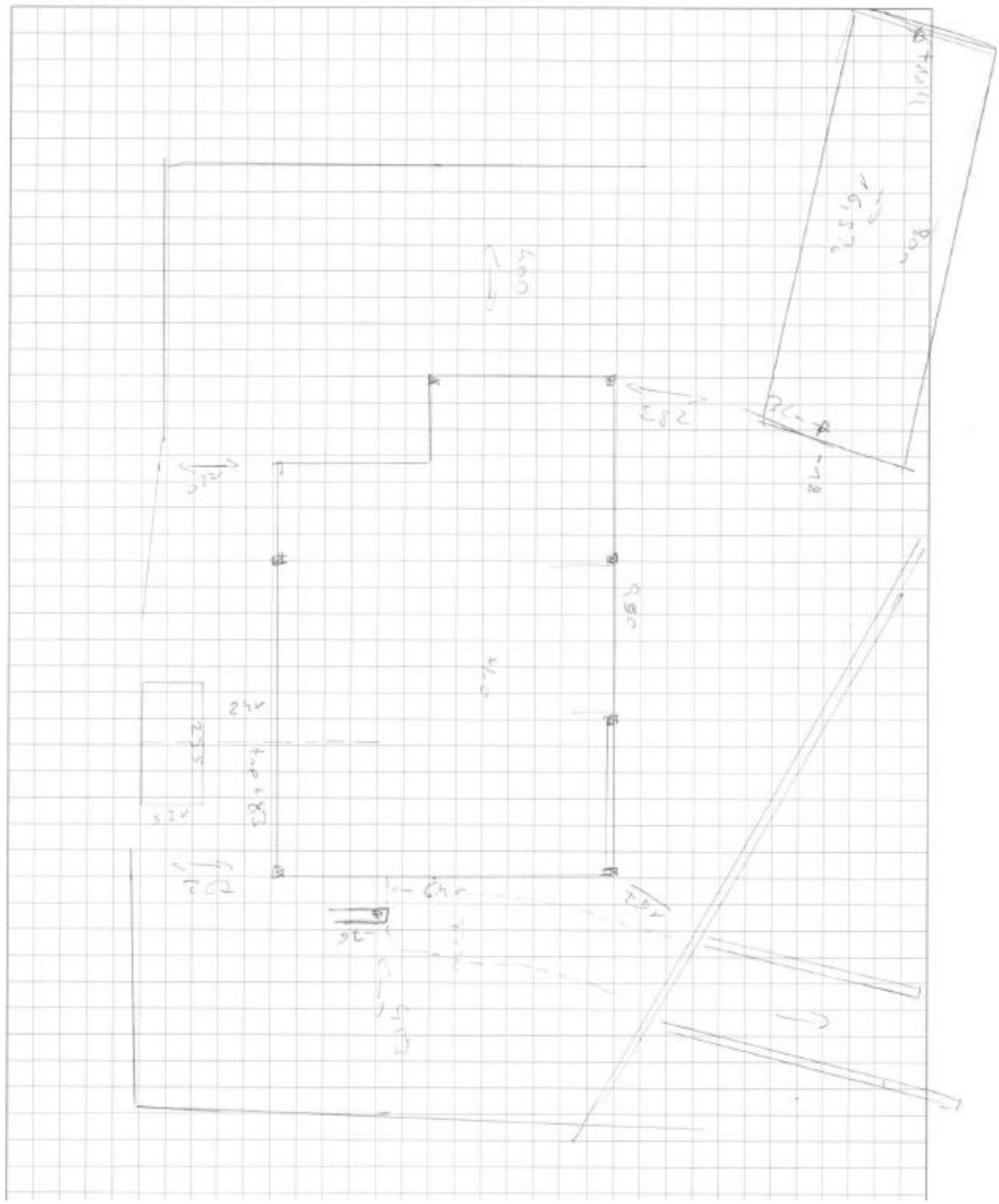
N°	view	location #1	location #2	value reading	relative elevation [cm]	absolute elevation	comment
1	**back**	**powerhouse**	**floor**	**133**	**0**	**-**	**powerhouse floor "KW +/- 0,00"**
	foreward	socle door		139,5	-6,5	- 0,07	
	foreward	turbine axis		12	121	1,21	
	foreward	outlet wall	left, top	300	-167	- 1,67	
	foreward	outlet wall	left, top	301	-168	- 1,68	
	foreward	outlet	bottom	380	-247	- 2,47	
	foreward	outlet	bottom	540	-407	- 4,07	
	foreward	outlet wall	rigth, lower poart	462	-329	- 3,29	
	foreward	**tailwater level**	**upstream outlet**	**608**	**-475**	**- 4,75**	**TWL river**
	foreward	socle south-west		139	-6	- 0,06	
2	back	socle south-west		131		- 0,06	
	foreward	ditch west	bottom	201	-70	- 0,76	
		turbine outlet	**water level**		**-105**	**- 1,11**	**TWL turbine, from picture**
	foreward	pipe outside	top	42	89	0,83	
3	back	socle south-west		120		- 0,06	
	foreward	access ramp	down	132	-12	- 0,18	
	foreward	access ramp	upstream outlet	0	120	1,14	
4	back	socle south-west		295,3		- 0,06	
	foreward	pipe outside	top	206,3	89	0,83	
	foreward	anc hor block	top	137	158,3	1,52	
5	back	anchor block	top	579,6		1,52	
	foreward	pipe us anchbl.	top	599	-19,4	1,33	
	foreward	support #1	top	255	324,6	4,77	
6	back	support #1	top	710,5		4,77	
	foreward	support #2	top	285,5	425	9,02	
	foreward	support #3	top	-19,5	730	12,07	
7	back	support #3	top	617		12,07	
	foreward	support #4	top	303	314	15,21	
8	back	support #4	top	689		15,21	
	foreward	pipe, upper end	top	235	454	19,75	
	foreward	anchor block	top	73,5	615,5	21,36	
9	back	anchor block	top	312		21,36	
	foreward	forebay	bottom	406,5	-94,5	20,42	
	foreward	**forebay**	**water level**	**161**	**151**	**22,87**	**WL forebay (22.10.2017)**
	foreward	forebay	concrete wall	121	191	23,27	
	foreward	spillway canal	bottom	225	87	22,23	
	foreward	spillway canal	water level	191	121	22,57	
	foreward	spillway canal	concrete wall	123	189	23,25	
	foreward	**spillway left**	**crest**	**169**	**143**	**22,79**	**NWL, korrigiert v. 159**
	foreward	forebay	bottom at beam	563	-251	18,85	
	foreward	canal end	bottom	261	51	21,87	
	foreward	canal end	water level	160,5	151,5	22,88	
	foreward	canal end	wall	119,5	192,5	23,29	
	foreward	**spillway rigth**	**crest**	**168,5**	**143,5**	**22,80**	**NWL**
	foreward	canel start	bottom	250	62	21,98	0 m
	foreward	canel start	water level	160,5	151,5	22,88	
	foreward	canel start	concrete wall	143	169	23,05	
	foreward	canel start	top	136	176	23,12	
10	back	canel start	top	144	168	23,12	
	foreward	man hole #1	bottom	235	-91	22,21	55 m
	foreward	man hole #1	water level	165	-21	22,91	
	foreward	man hole #1	concrete cover	131	13	23,25	
	foreward	man hole #1	top	121	23	23,35	
	foreward	man hole #2	bottom	225,5	-81,5	22,31	85,7 m
	foreward	man hole #2	water level	160,5	-16,5	22,96	
	foreward	man hole #2	top	108	36	23,48	
11	back	man hole #2	top	151	-7	23,48	
	foreward	man hole #3	bottom	260,5	-109,5	22,39	107,3 m
	foreward	man hole #3	water level	200,5	-49,5	22,99	
	foreward	man hole #3	top	142,5	8,5	23,57	
	foreward	man hole #4	bottom	255	-104	22,44	122,7 m
	foreward	man hole #4	water level	196	-45	23,03	
	foreward	man hole #4	top	138	13	23,61	
	foreward	man hole #5	bottom	245	-94	22,54	151,1 m
	foreward	man hole #5	water level	188	-37	23,11	
	foreward	man hole #5	top	128	23	23,71	
12	back	man hole #5	top	144,5	6,5	23,71	151,1 m
	foreward	man hole #6	bottom	237	-92,5	22,79	239,9 m
	foreward	man hole #6	water level	180	-35,5	23,36	
	foreward	man hole #6	top	117	27,5	23,99	
13	back	man hole #6	top	146,5		23,99	239,9 m
	foreward	rock		94,5	52	24,51	
14	back	rock		132		24,51	
	foreward	man hole #7	bottom	276	-144	23,07	321,8 m
	foreward	man hole #7	water level	218	-86	23,65	
	foreward	man hole #7	top	157	-25	24,26	
		culvert crossing					333 m
	foreward	rock	top	131	1	24,52	
15	back	rock		203	-71	24,52	
	foreward	soil		162	41	24,93	
16	back	soil		124	8	24,93	
	foreward	intake gate	fixe point	98,5	25,5	25,18	
17	back	intake gate	fixe point	127,7		25,18	439,7 m
	foreward	canal us intake gate	bottom	250,5	-122,8	23,96	
	foreward	canal us intake gate	water level	160,5	-32,8	24,86	
	foreward	spillway ds	ds	159	-31,3	24,87	
	foreward	**spillway ds**	**us**	**160**	**-32,3**	**24,86**	**NWL canal intake**
	foreward	spillway middle	crest	160,5	-32,8	24,86	
	foreward	intake structure	top	111	16,7	25,35	
	foreward	trashrack	top	132	-4,3	25,14	
	foreward	**weir**	**crest, left**	**119**	**8,7**	**25,27**	**NWL basin**
	foreward	weir	crest, rigth	115,5	12,2	25,31	
	foreward	wingwall rigth	top	5	122,7	26,41	
	foreward	bottom outlet rigth	bottom	435	-307,3	22,11	
	foreward	bottom outlet left	bottom	485	-357,3	21,61	

Annex 3 Sketch of powerhouse

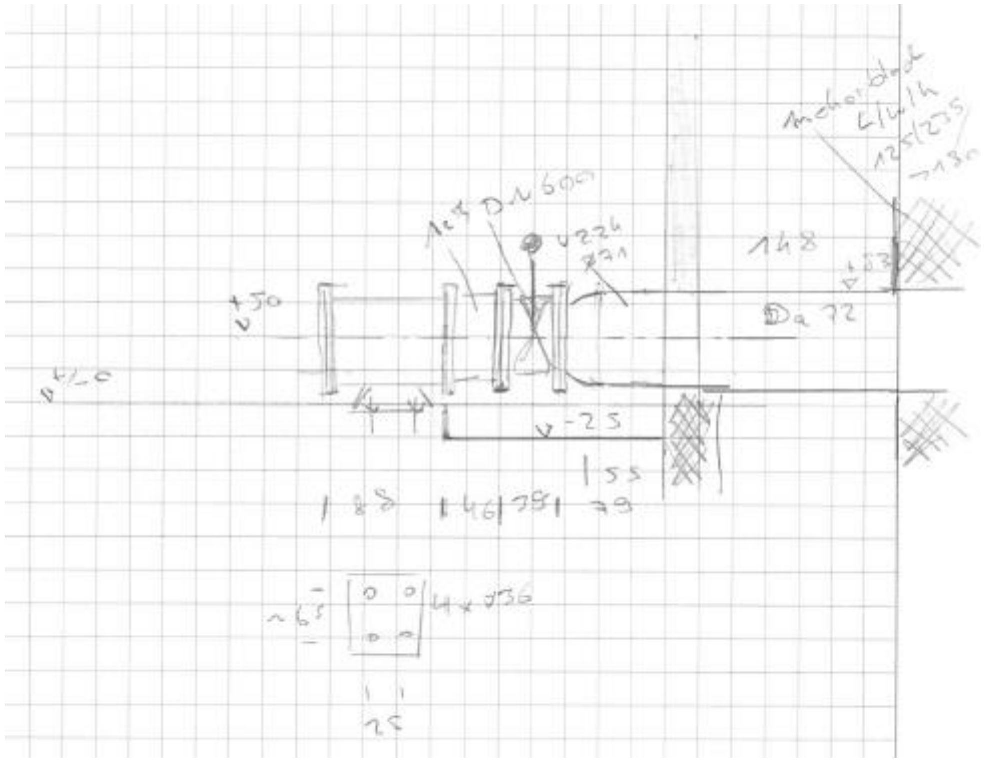
Powerhouse layout



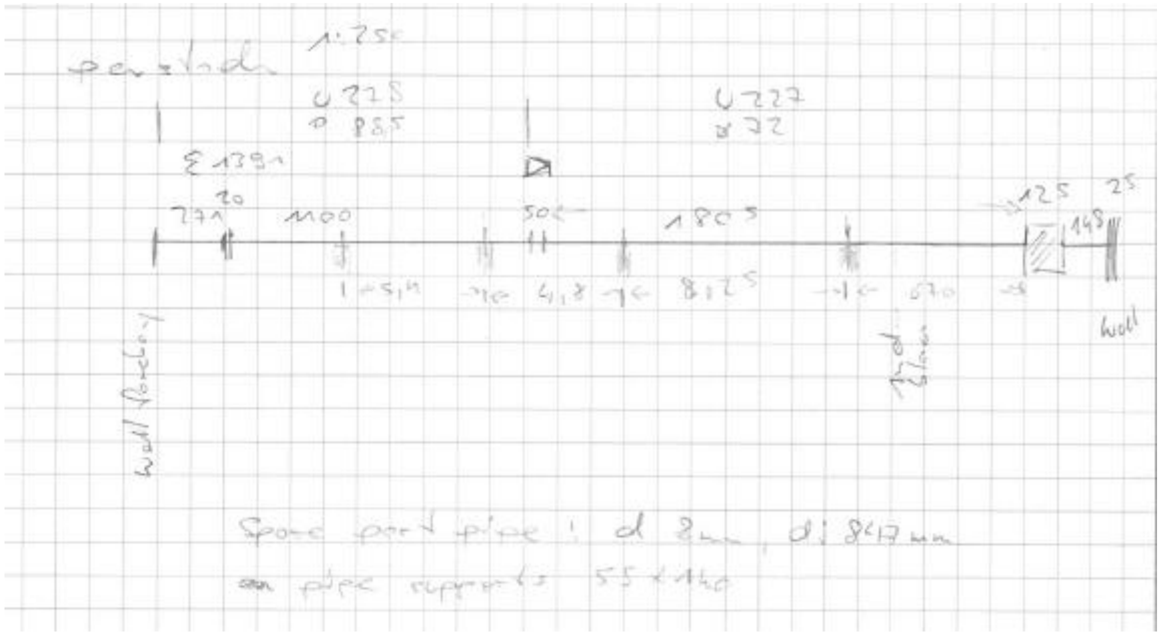
Powerhouse surrounding area



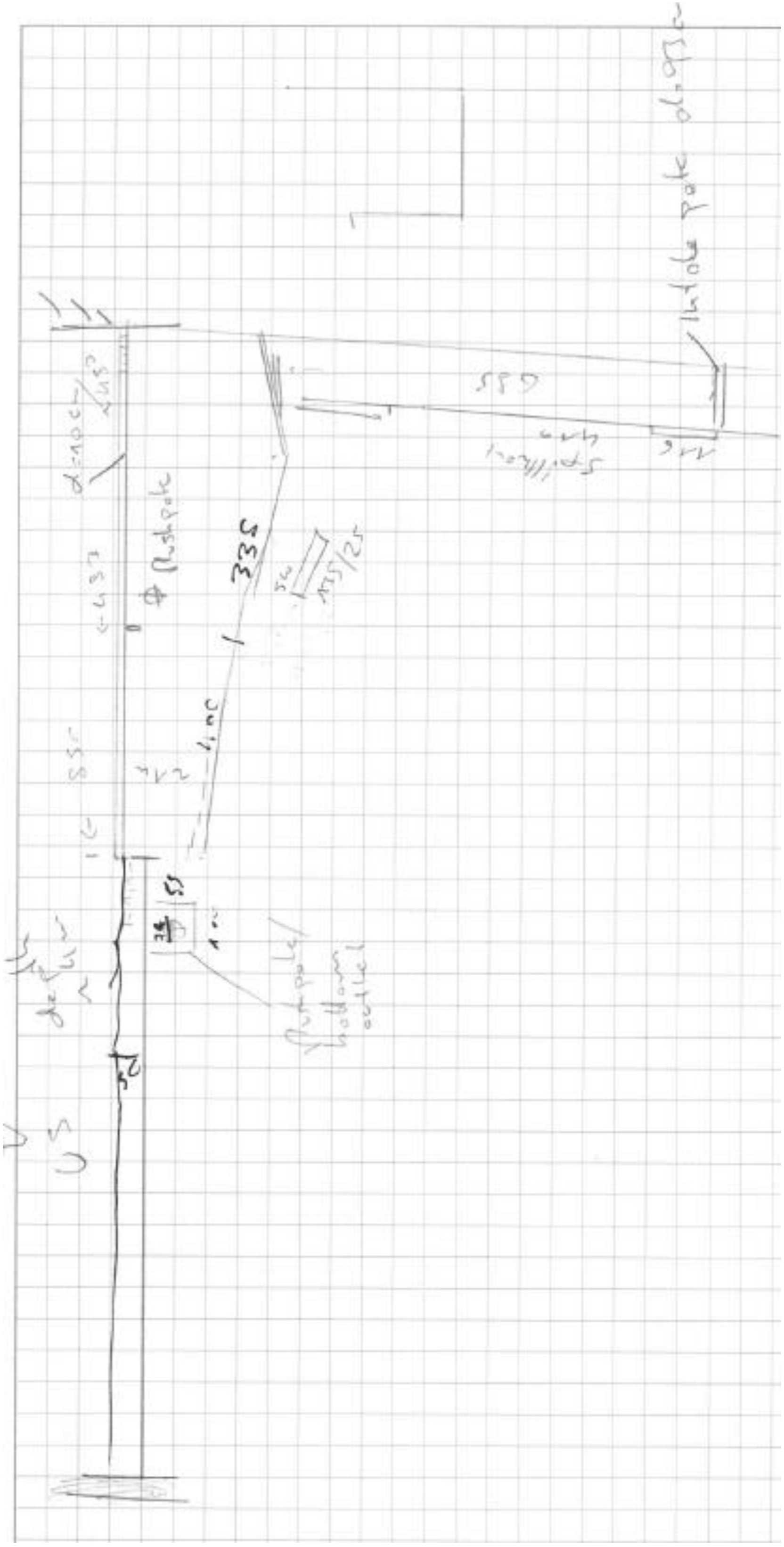
Penstock alignment inside powerhouse



Penstock alignment outside powerhouse

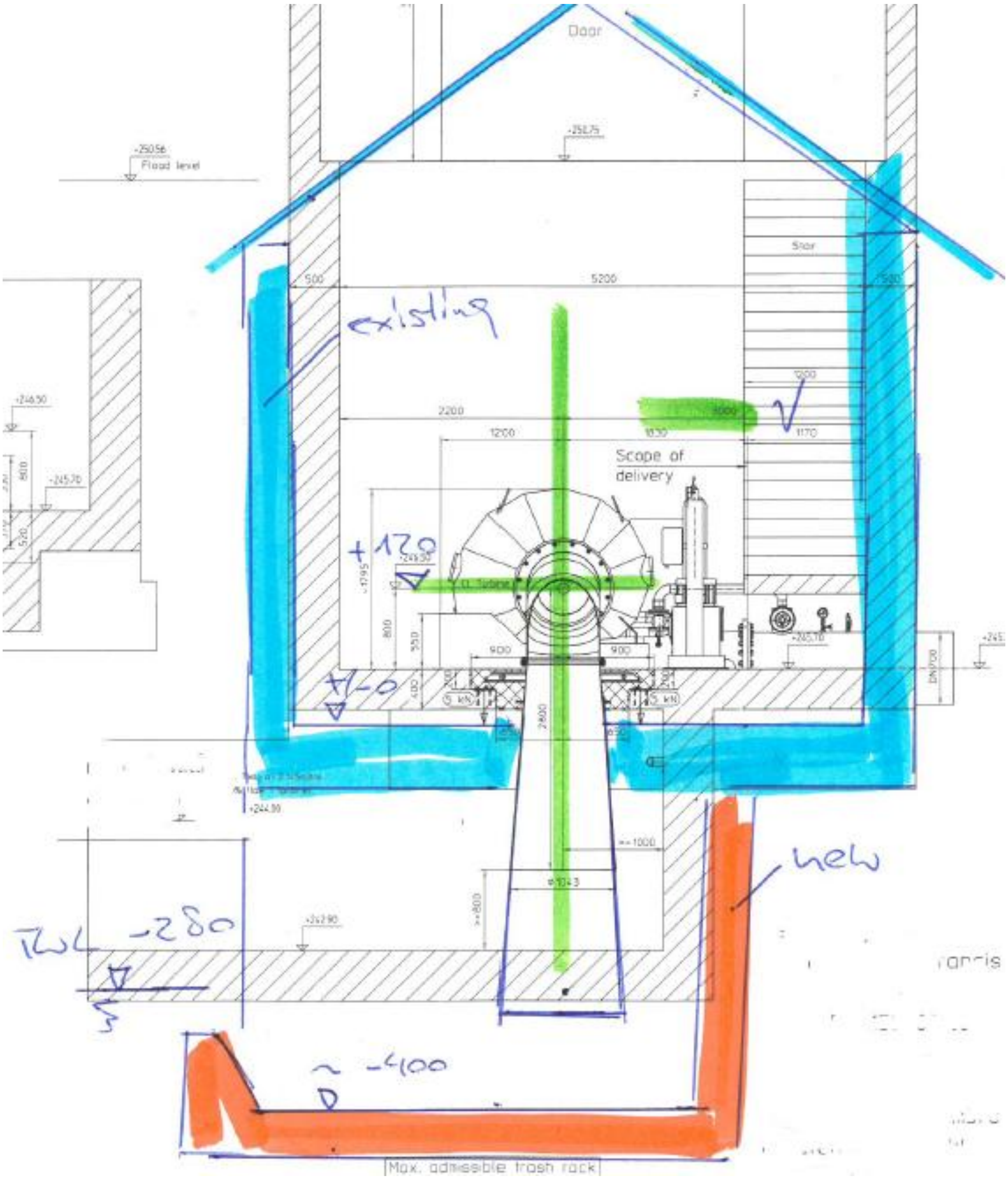


Weir and diversion canal intake

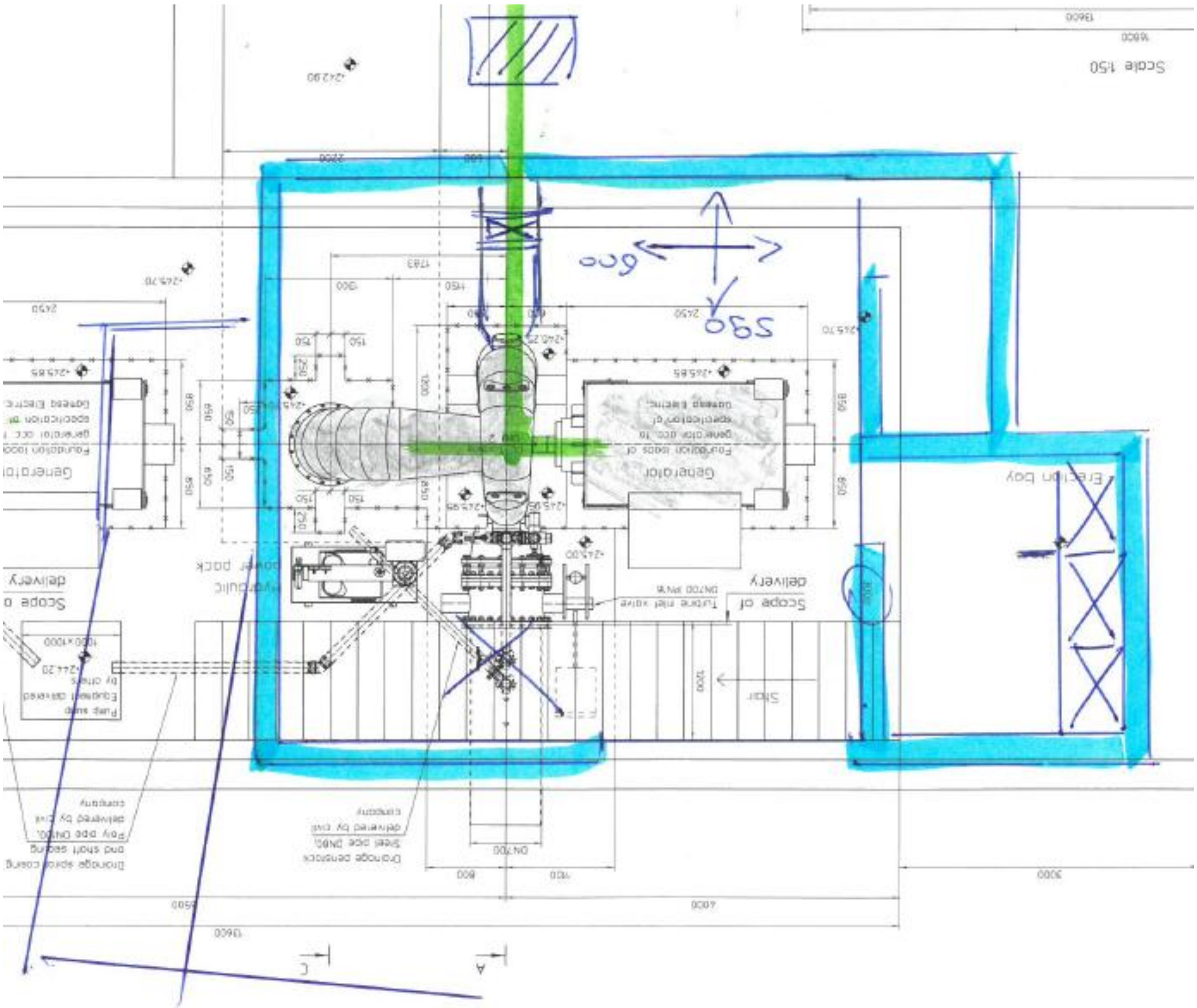


Annex 4 Sketches of new turbine placement in existing building boundaries

Cross-section



Layout



Annex 5 Single line Diagram for Stage 2 and Stage 3