RISK-BASED STABILITY EVALUATION OF THE UPPER SPILLWAY MIDDLE PIER – ISIMBA HEPP UGANDA

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ABSTRACT

The 183 MW Isimba HEPP is a runoff river hydropower plant located on the White Nile in Uganda commissioned on 21st March 2019 and currently operational under the defects liability period. During the initial visual inspections, the middle pier of the upper spillway was observed to be abnormally oscillating due to immense spillway discharge induced vibrations during spillway operations. This was further checked with the erratic vertical displacement readings of the strain gauge installed in the mass concrete of the bottom slab. This paper discusses the potential failure modes of uplift of the stilling basin and flexural failure of the pier which prompted a vibration test. A vibration test was carried out on the pier to collect field parameters for structural analysis using fluid-solid coupling numerical analysis methods. However, this test came with public safety concerns since opening of the gates involved release of a 20-year flood which posed high potential risks to the downstream activities. The paper further discusses the pier stability analysis process and proposes rational vibration reduction measures without compromising dam safety, public safety and health concerns.

1. INTRODUCTION

1.1 Background

Isimba Hydroelectric power plant is a runoff river station located on the White Nile approximately 50km downstream of Lake Victoria in Southern Uganda. The plant was commissioned on 21st March 2019 and is currently operated by Uganda Electricity Generation Company Limited (UEGCL) under the defects liability period (DLP). The plant undertakes part of the function of peak-regulation of the power grid among others. It is equipped with four vertical Kaplan turbine – generator units with a unit capacity of 45.8MW, thereby giving a 183.2MW total installed capacity.

The main civil structures of the plant include; The Left and Right earth-rock fill embankment dams with a total length of 1424m and maximum dam height of 26.5m, concrete gravity dams GD1 & GD2 with a length of 140m (including wing walls) and a maximum height of 26.5m, overflow ogee spillway (SP1 & SP2), powerhouse and a 132kV switchyard. The main powerhouse is laid in a riverbed and designed to accommodate four generating units and an erection bay. See Figure *1* for a project overview.

The design inflow discharge from hydraulic computations is 1375m^3/s which translates into about 344m3/s for a single generating unit. Isimba HEPP has a reservoir capacity of 170,680,000m^3 and a rated head of 15.4m. The reservoir full supply level (FSL) of the plant is 1054.5m.asl (meters above sea level) under which the reservoir capacity is 160,800,000m^3. The probable maximum flood (PMF) water level and minimum operating water levels are 1055m.asl and 1052.5m.asl respectively.

1.2 Spillway description

The main flood release structure adopts a spillway designed for a standard 1,000-year flood of 3,500m^3/s and checked with a 10,000year flood of 4,500m^3/s for which the lower spillway SP1 (bottom outlets) with a capacity of 3600m^3/s is designed according to the requirement of flood control and additionally flushing of sediment during normal operating conditions. The upper spillway SP2 (surface outlets) with a combined discharge capacity of 1650m^3/s is for mainly regulation of the reservoir water level, limitation of the fluctuations in the upstream and downstream water levels, flushing of float and also flood discharge. The (n-1) principle was adopted in the design of spillway gates with a total installed flow capacity of 5250m^3/s which is greater than 10,000-year flood of 4,500m^3/s as a safety measure in case a flood occurred during spillway maintenance. The main technical parameters of the spillway are as summarized in Table 1.

Figure 1 : Layout of the Civil structures of Isimba HEPP

Lower spillway SP1 is set on the right side of the powerhouse with its left wall closely connected to the powerhouse. It is provided with three outlets divided by two middle piers of a thickness of 3.0m with each of the side piers 2.5m thick.

Table 1 : Technical parameter of the Isimba HEPP spillway

Item	**Unit**	**Lower Spillway SP1**	**Upper Spillway SP2**
Opening width	m	9.50	14.00
Opening height	m	10.5	Open overflow weir
Bottom elevation	-	1029.00	1044.50
Top elevation	m	1039.50	Open end
Single opening discharge	m^3/s	1239.25	945.82
Number of outlets	-	3	2

Upper spillway SP2 set at the right side of SP1 is provided with two outlets for flood discharging purposes which are divided by one 3.0m middle concrete pier with two side walls of 2.5m thickness each. The total length of the upper spillway along the flow direction is 119.52m and operates by means of an open-type overflow weir. The energy dissipation is conducted by bottom flow through a stilling basin which is provided with baffle blocks to break the velocity of the flood which would otherwise erode and wash away the downstream river bed. Drainage openings of 150mm are set in the stilling basin for anti-floating purposes during the discharge of huge floods. The stilling basin is anchored to the foundation bedrock by 6m anchor rods with anti-wear concrete set at the top of the spillway baseboard to control abrasion of the spillway surface.

1.3 Motivation of Evaluation

As per the Dam Monitoring and Surveillance plan, visual inspections are aimed to detect any changes that cannot be recognized with measuring instruments. During the initial reservoir impoundment visual inspections, it was observed that the upper spillway appeared to be oscillating in an abnormal way due to immense vibrations induced by large spillway discharges. This was later checked with the erratic vertical movement and rapid increase of the strain gauge (S8) installed in the mass concrete weir under the spillway gate aimed to monitor vertical strains of the concrete in response to external loads such as the hydrostatic loads from impounding. Instrumentation data processing also showed that the maximum vertical strain remained constant despite the stabilization of the reservoir water level after impounding as shown in Figure 2. Potential failure mode as defined by the Federal Energy Regulatory Commission (FERC) is, “the chain of events leading to unsatisfactory performance of the dam or a portion thereof. The dam does not have to completely fail in the sense of a complete release of the impounded water.” (Schweiger, Kline and Burch, 2019)

In reference to the above mentioned observations, potential failure modes of; possible Uplift of the stilling basin and flexural failure of the pier were identified in a site risk workshop, quantitatively assessed and subsequently ranked as extreme risks (E) in the Isimba Dam safety risk register due to the inherent consequences they posed to the owner. This called for a need for further investigation and analysis of the looming state of the spillway for reevaluation of the posed risks. It is well known that a high percentage of dam safety incidents occur within the first five years of operation; the initial impounding and the first years of operation are the most dangerous and therefore dam failure prevention must be efficient during the life cycle of the dam from impoundment to decommission (Mean et al. 2019)

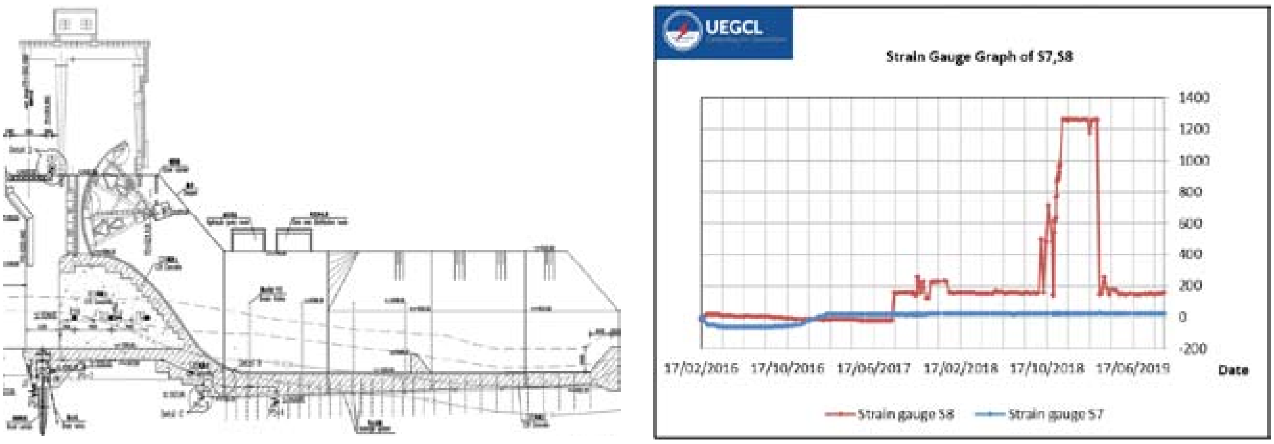

Figure 2 : Location of the Strain gauge S8 and the strain gauge response due to initial impounding

2. PIER ANALYSES

2.1 Pier preliminary analysis

Following the advisory and recommendation for further investigation of the Upper spillway middle pier, the Owner's Engineer carried out an in-depth analysis in order to;

- Verify the stability of the top of the weir considering a horizontal crack in the flow direction.
- Determine the perennity of the pier under the vibrations.

In the analysis, a geometrical model of the weir with an assumed horizontal crack at elevation 1034.00m.asl was is in the stability analysis. The friction angle Φ is taken to be 40 degrees at zero cohesion. The model is tested under the most unfavorable operating condition considering the Full supply level (FSL) at 1054m and the downstream water level at 1039.08m. However, the friction and shear resistance of the traversing reinforcement of the walls on each side are not considered (no coefficient is applied on the cohesion and friction). Under the above considered hypothesis, the weir is found to be stable against sliding under the assumed unfavourable situation if a crack developed horizontally along the longitudinal direction with a factor of safety 0.93 which is close to 1. The calculations are run using SOURIS, an intern software by ARTELIA.

In the structural analysis of the long-term resistance of the pier under the vibrations, a 3D model composed of two 2.5m thick walls and a slab is modelled as anchored (pinned supports) see Figure *3*. The set-in-place concrete reinforcement (Concrete C20, Steel rebars fyk = 420 MPa with a cover of the 0.15m) is found to have a maximum bending moment of about 1600kN.m/ml at the bottom of the wall (El + 1025.00) giving stress in concrete as; 2.14MPa (OK) and stress in steel rebars as 58MPa<336 MPa (OK) and a natural frequency of 3 Hz.

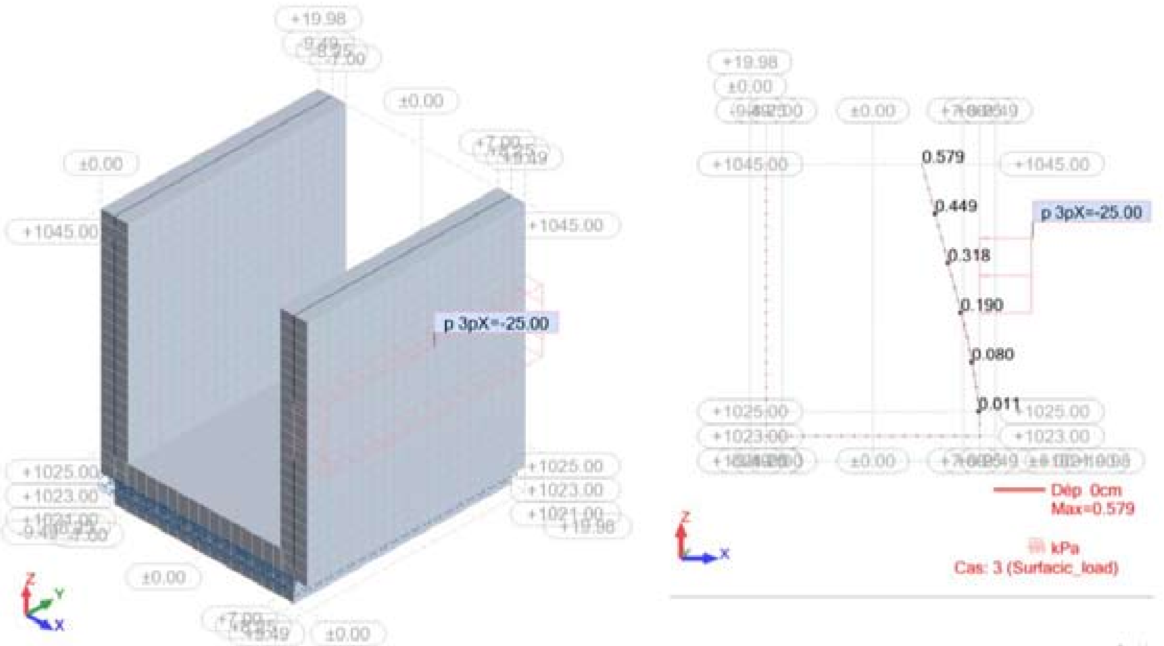

Figure 3 : 3D model adopted in the verification of the spillway pier

The fatigue of the reinforced concrete is then evaluated in order to estimate the maximum number of cycles that the pier can withstand according to Eurocode 2. Calculation of the fatigue for concrete and the fatigue for the steel rebars provides that the reinforced pier has a resistance of 42years under conditions of continuous cyclic vibrations from the discharge of the upper spillway

However, the spillway was designed to safely release flows of about 4500m3/s (10000-year flood) which requires the use actual flow and instrumentation data to carry out a more informed analysis which takes into account the actual conditions of the structure. Therefore, it was proposed that a stability analysis be carried out by the Contractor using site conditions which had to be done through a vibration test as explained below.

2.2 Pier Vibration Test

The vibration test was carried out on the middle pier of the upper spillway in order to measure pressure pulsation, vibration frequency and the horizontal and vertical vibration of the pier under abnormal gate operating conditions. A total of 34 sensors were installed for data collection which included; 14 horizontal and vertical displacement type vibration sensors, 12 vibration acceleration sensors and 8 pressure sensor. See figure for the experimental layout.

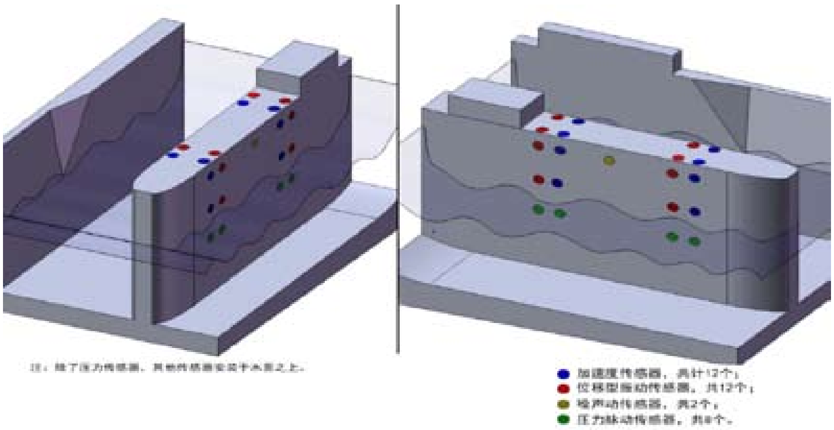

Figure 4 : Layout of the data collection sensors of the vibration test

2.2.1 *Test Procedure*

The Upper spillway outlets are first opened simultaneously from zero to full opening at increasing intervals of 15% for a time duration of 3.5 minutes between each interval. The reading from the data collection sensors are recorded by the data collection station set up in the spillway room. The procedure is then repeated for each individual left and right opening until the all the required data is collected as shown in Table 2. Analysis of the results is then made to derive relevant conclusions.

Table 2 : Vibration test procedure for the gate operation

S.N	Left Outlet Opening (%)	Right Outlet Opening (%)	Duration of test Time (minutes)
1	15	15	3.5
2	30	30	3.5
3	45	45	3.5
4	60	60	3.5
5	75	75	3.5
6	0	15	3.5
7	0	30	3.5
8	0	45	3.5
9	0	60	3.5
10	0	70	3.5
11	15	0	3.5
12	30	0	3.5
13	45	0	3.5
14	60	0	3.5
15	70	0	3.5

However, from the pre-test hydraulic computations, a total plant discharge of over 2,200m^3/s was obtained to be released from the combined spillage and discharge through the turbine units. This is approximate to the plant's 20-year flood of 2,000m^3/s which corresponds to an anticipated 2m rise in the downstream water levels. From the hydrological records, this flow was last experienced in 1970 rendering this one of the most sensitive experiment since it involved crosscutting risks to the client and the public safety. Potential failure mode analysis was further cascaded in the mitigation of the newly posed potential risks induced by the vibration test as discussed below.

2.2.2 *Risk Management*

The dam owner is responsible for managing the public safety risks caused by a dam, as far upstream and downstream as the owner has property rights and an additional responsibility to assess specific locations where the hazards are known by the owner to result directly from the dam or its operations and to inform the public and other affected property owners of these hazards (CDA Guidelines, 2019). With safety as one of its core values, UEGCL used a risk-informed approach to ensure safe execution of the vibration field test where the risks are first assessed (risk identification, risk analysis and evaluation) and then mitigation measures are proposed. The vibration test was first divided into three parts; preparation and installation of the test sensors, execution of the vibration test and finally decommissioning of the test and reinstatement. Different permits for work were issued for the three sections of the test in which risk assessment was carried out all the identified risks. The major risks identified included; structural failure of the pier due to prolonged release of high discharges, loss of power production business due to reduced water levels, possible failure of the spillway gates, potential harm to the downstream activities (ferry operations about 5 km downstream of the dam, fishing and sand mining activities and bridge construction works about 400 km downstream of Isimba).

As a precursor to the vibration test commencement, check sheets were developed to address compliance and adherence to the proposed mitigation measures. Risk treatment of the main identified risks was as follows; the risk of structural failure was mitigated by adopting a threshold displacement of 2mm as obtained from the preliminary analysis beyond which the test would have to be terminated to preserve the integrity of the structure. See for the vibration monitoring team. Loss of the power production was mitigated by coordinating with the upstream cascade dams to avail sufficient inflows to Isimba HEPP so as to maintain the reservoir at FSL which was sufficient to sustain both the vibration test and power production. For the downstream activities, sensitization was carried prior to the vibration test and all the potentially affected communities evacuated and further temporarily suspending the bridge works and ferry operations. Public safety signage and surveillance teams were deployed both upstream and downstream to ensure communities stayed away during test execution.

Figure 5 : The data collection station and the vibration test execution team

2.2.3 *Numerical Simulation*

Study on spillway unsteady flow, static and vibration characteristics based on fluid-solid coupling. The large discharge energy of the Isimba HEPP upper spillway generates complex flow states in the spillway such as overturning, recirculating, oscillating, vortex and other unfavourable hydraulic flow states. These water pressure pulsations generate huge water pressure which causes a large hydraulic shock on the flood discharge chamber middle pier. To obtain the flow characteristics of the water flow, it is necessary to obtain the magnitude and position distribution of the water flow excitation force to understand its periodicity. The simulation is mainly for the impact of the unsteady flow on the upper spillway middle pier. The fluid under unsteady conditions impacts the solid so that the vibration generated by the solid (the flow-induced vibration) can be numerically simulated by unidirectional fluid-solid coupling data transfer. The fluid solver is mainly responsible for the calculation of the physical quantities such as pressure, temperature, velocity and composition of the flow field, while the solid structure solver is responsible for the calculation of displacement, strain and stress. Among these solution variables, the physical quantities used for two-phase exchange and present in both solid and fluid solutions are pressure and displacement. In the fluid solver, the pressure is the direct solution amount and in the solid solver, the pressure can be used as the load. In the solid solver, the displacement is the direct solution amount, while in the fluid solver, the displacement can be used as the load, which is expressed as the calculation domain displacement or deformation. Therefore, the fluid-solid coupling problem depends critically on the transfer problem of the common variable parameter solution calculation.

Main method for the Upper spillway flow characteristics.

Navier-Stokes equation: Assupmtion made; (1) the fluid is continuous, (2) homogenous and incompressible, (3) Isotropic Newtonian fluid.

$$\frac{uv}{\partial} = F_e - \frac{1}{\rho}\nabla P + k\nabla^2 v \qquad (1)$$

Where: V – fluid velocity; P – fluid pressure; F_e – external pressure on the fluid; k – fluid motion viscosity coefficient.

Turbulence model

The turbulence model commonly used in engineering is adopted: the standard k-ε model. This model can be used when the model has a high Reynolds number. The model was proposed by LAUNDER and others in 1972. Since the flow rate of this experimental model is set higher, the Reynolds number is much higher than the laminar Reynolds number when flowing in the pipeline, so it can be assumed that the flow is completely turbulent, and it is appropriate to use the $\mathbf{k} - \mathcal{E}$ model here.

Main method for calculation of SP2 vibration characteristics

$$[K]\{\delta\} + [C]\{\dot{\delta}\} + [M]\{\ddot{\delta}\} = [M]\{\ddot{\delta}_a \qquad (2)$$

Where: [K], [C], and [M] are the overall stiffness matrix, damping matrix and mass matrix of the structure respectively.

The digital-to-analog calculation involves the simulation of the flow state of the plenum. The water pressure at different positions of SP2 is obtained to transfer the water pressure to SP2. The structural statics and dynamics of SP2 structure are analysed to obtain the self-generated vibration characteristics of SP2 structure, and the stress, strain and vibration characteristics of the force device.

3. RESULTS AND DISCUSSIONS

Figure 6 : Showing the discharge hydrograph of the spill way during the vibration test

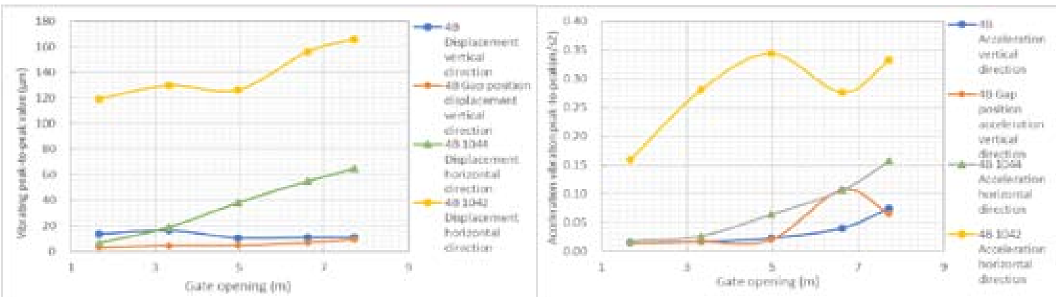

Figure 7 : Showing the acceleration response of the sensors during the vibration test

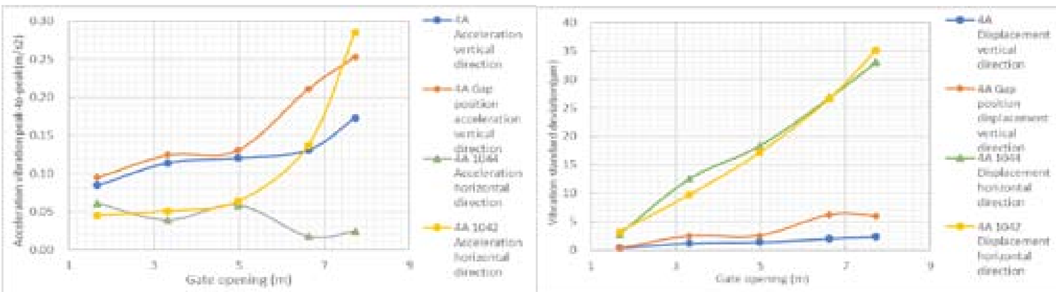

Figure 8 : Showing the displacement vibration of the sensors during the vibration

Figure 9 : Showing the right opening at 70% opening with the corresponding downstream water level rise

From the Preliminary analysis using SOURIS an intern software from ARTELIA the owner's Engineer (OE), the hypothesis adopted provides that the spillway weir is stable against siding under the most unfavorable situation assuming a crack develops horizontally along the longitudinal direction of the weir at +1034.00. The results also show that the maximum bending moment of about 1600kN at the bottom of the wall is in range and the additional stress coming from the vibration acceptable. The resistance of the reinforced pier is evaluated as 42years under conditions of continuous spillage which is a very unlikely situation since opening of the gates is rare during normal conditions.

The vibration is maximum under the operating conditions in which the two outlets of the Upper spillway are opened at the same time see Figure 6 for discharge hydrograph, and the vibration value becomes larger as the gate opening degree increases; when the gate opening reaches the maximum 75%, the vertical vibration peak-to-peak value is between 0.37 μm and 29.2 μm. The "4A gap displacement vertical" value is the largest, which is 29.2μm (Figure 8 above); the horizontal vibration peak-to-peak value is between 2.76μm and 174.86μm, wherein the "4B 1042 displacement level" has the largest value of 174.86μm; the vertical acceleration vibration peak-to-peak value is 0.061m. Between /s2~0.5209m/s2, the value of "4B gap acceleration vertical" is the largest, 0.5209m/s2; the horizontal acceleration vibration peak-to-peak value is between 0.2301m/s2~0.6844m/s2, of which "4B1042 acceleration level" The maximum value is 0.6844m/s2 (Figure 7 above); the standard deviation of vertical vibration is between 0.14μm and 11.88μm, among which the value of "1A displacement vertical" is the largest, which is 11.88μm; the standard deviation of horizontal vibration is between 0.86μm and 53.08μm, among which " The 4B 1042 displacement level has the largest value of 53.08μm; the vertical acceleration vibration standard deviation is between 0.0143m/s2 and 0.1962 m/s2, of which the "4A gap acceleration vertical" value is the largest, 0.1962m/s2; horizontal acceleration vibration standard The difference is 0. Between 0143m/s2 and 0.1962 m/s2, the value of "4A gap acceleration vertical" is the largest, which is 0.1962m/s2. In the case of the same opening degree, the noise values are basically the same, and there is no abnormal state. At 75% opening of the gates, free fall of the water was achieved (the bottom of the gate left the water surface) see Figure 9 above.

The risk informed approach used during the test was effective since there was no harm caused to the downstream residents, the ferry operation and the bridge works as verified by the post-test inspection. The concrete structure and the mechanical equipment of the spillway were not significantly affected either. Coordination with the cascade upstream dams also ensured that there was enough inflow to allow for the vibration test as well as power production. Water levels during the test and after reinstatement. After successful completion of the vibration test, UEGCL embarked on a reinstatement process by operating the gates to normalize the downstream water level back to elevation 1038.7m.asl so as to allow for ferry operation and bridge construction works. (Fig.10)

3.1 Interim vibration reduction measures

In the meantime, unnecessary spillway gate operations ought to be avoided by the operator in order to prevent development of structural fatigue in the spillway structure due to prolonged spillage. The upper spillway radial gates may be operated at less than 70% opening as per the vibration test data results until the final stability analysis of the pier is done and correctly verified.

During flood conditions, sharing of the discharge among the two upper spillway outlets is advised as compared to individual opening of the outlets for the same quantity of flow. From the vibration test data results, it is observed that using two outlets is preferable to one when discharging a flood of the same magnitude due to the reduced vibration results.

Figure 10 : Showing the downstream water levels during the test and after reinstatement process

4. CONCLUSION

Regular visual inspections of the spillway structures need to be carried as per the Dam surveillance and operation plan in order to capture any defects and deviations that may arise due to future spillway operations. These include spillway inspections during the operation and also immediately after termination of each spillway operation so as that the effects of each spillage operation can be documented and for future trending and analysis. Annual diving inspections of the spillway stilling basin should also be incorporated in the annual Dam monitoring routine to check on the status of the spillway baseboard surface and the drain holes.

Additional instrumentation should be considered in order to record vibration characteristics for continuous data collection and analysis by the operator per the Dam instrumentation plan in order to allow early detection of faults and defects for timely intervention by the level III Dam experts. Continuous analysis ought to be carried by the Operator in order to assess the structural behavior of the pier as it ages every after five years using the existing parameters at that specific time.

A stability analysis should be carried out by the Contractor and submitted to be OE for review and any remedial and further mitigations for the inherent Potential failure modes implemented before the end of the 2 year defects liability period in order to avoid transfer of the liability risks to the Operator or client. This will facilitate firm ground for a risk workshop to reevaluate the initially identified extreme risks to be reduced to as low as reasonably practical (ALARP). Dam operators and designers need to adopt a risk-based evaluation of all post-construction defects and structural observations in order to have Risk-Informed decision making which reduces the residual risk to the Dam Owner.

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