1.0 INTRODUCTION

Safety is the primary and most important reason for monitoring (measurement of change over time) the deformations of dams. A secondary reason is the need for improving our knowledge of the mechanical behavior of the dams (*Gikas and Sakellariou, 2008*). The safety of large man-made structures, the environmental protection and the development of associated mitigating measures in the case of natural disasters, require a good understanding of the causes and mechanism of the structural process (*Gikas and Sakellariou, 2008*). To ensure safety and integrity of a dam or any structure, a deformation monitoring system on a dam must detect change, rate of change and rate of increase of change over time, effectively. In order to obtain these dynamics reliably, about the mechanical behavior of a structure being monitored, there is need for an automated geodetic monitoring system in-situ *(Trimble, 2015).*

Deformation monitoring systems are designed to measure the deflection or distortion of a structure, under normal and extreme conditions. Dams, bridges, large and tall buildings are examples of structures that are routinely surveyed and monitored (*Choudhury and Rizos, 2011*). Such man-made structures and slopes undergo various forms of deformations. Precise and accurate monitoring of the progressive deformations of these structures can often provide vital information on the stability and safety status. Deformation monitoring can be carried out by manual deformation monitoring and/or automatic deformation monitoring system (*Gikas et al, 2005*).

An understanding of the mechanical behaviour of concrete dams under the influence of natural, and human activities is a valuable input into the improved future design code of concrete dams *(Gikas and Sakellariou, 2008).*

A lot of reports have been written describing how accurate the monitoring instruments are (which they indeed are) in deformation monitoring of the Kariba Dam by ZRA, e.g. in 2008, 2010 and 2012 reports. No research before this one had been done to assess the procedures followed by ZRA and how they visualize, analyze and interpret measurement results from the deformation monitoring using classical instruments. Incorrect procedures in measurements, analysis, visualization and interpretation of deformation may lead to misjudgments of the dam deformations and stability, despite using accurate instruments. Analysis and interpretation of results in this study were based on underlying theories and concepts.

Participation in the field measurements for February 2015 on Kariba Dam was part of this research in order to understand and assess adequacy of the current method to address the deformations of the Kariba Dam and justify the upgrade of the monitoring system. The past deformation results were compared to the current deformation results to derive dam behaviour as well as evaluating the quality of results from classical monitoring methods. The data analysis yielded the physical meaning of the past dam deformation results obtained by ZRA over the years.

However, this study focused mainly on interpreting dam deformations by analyzing past deformation data and visualizing it to bring out correlations between the Kariba Dam behaviour and the surrounding geodynamics, in view of the current hazardous geological environment around the dam. The deformations were analyzed based on Albert Einstein's special relativity theory and Sir Isaac Newton's laws of motion.

This research is founded on ensuring safety of the Kariba Dam by providing easy-to-understand scientific illustrations or visualizations of Kariba Dam deformations measured by Zambezi River Authority (ZRA) since 1989 to date, but mainly from 2008 to 2015. Some leveling data obtained since 1963 was also referred to. The interpreted deformation information is meant for the decision makers in the procurement of deformation monitoring equipment for Kariba Dam, thus ensuring safety. The interpreted deformation information includes landslide direction, its effects on the deformation monitoring system and the current geometric status of the dam which may have an adverse impact on the dam stability, if no corrective measures are put in place.

The analysis, visualizations and data interpretations in this research provided a basis for the theoretical designs and proposals to upgrade the classical geodetic monitoring systems of the Kariba Dam to modern ones. The proposals included the introduction of the prism array which would supplement the existing monitoring networks.

1.1 Problem Statement

The current classical geodetic deformation monitoring method for the Kariba Dam was designed to detect normal stresses and strains caused by: seasonal variations in Temperature and Hydrostatic Pressure, Corrosion and Aging. However, more unexpected factors have set in, over time. These unexpected factors are: Alkali Aggregate Reaction (AAR), deteriorating south bank geology, plunge pool scouring, Reservoir Induced Seismicity (RIS) and blasting vibrations (*International Rivers, 2009*).

Kariba Dam has since suffered from unexpected dam stability threats which emerged years after the dam was constructed. It must be borne in mind that Kariba dam already suffers effects from temperature changes, hydrostatic pressure, corrosion, aging, etc. These unexpected factors have potential adverse effects on the dam's integrity, especially under combined-effect conditions.

However, the deformation monitoring of these threats on the Kariba Dam has remained traditional, despite rapid advancements in geodetic deformation monitoring technology. Lack of the deformation monitoring system upgrade exposes the dam to a risk of disaster occurrence without detecting it beforehand. Making a new measurement cycle through the traditional geodetic methods involves a series of stages that cannot be avoided: moving specialists to the structure to be monitored, actual performance of measurements, processing them etc. This automatically implies a certain period of time required to prepare technical documentation and reach some relevant conclusions, which, in emergency situations e.g. a RIS occurrence of a significant magnitude or a landslide, could be much too late. This situation could be avoided by, alternatively, the use of spatial automatic deformation monitoring systems of land and structural deformations installed in-situ (*Setan and Idris, 2008*).

1.2 Research Questions

1. How would GNSS supplement deformation monitoring system at Kariba Dam?

2. How could the expected and unexpected factors be analyzed and visualized to better understand their effects on the Dam?

1.3 Project Objective

To design a real time and automated Robotic Total Station-GNSS integrated system for Kariba dam deformation monitoring system to complement the classical monitoring systems

1.3.1 Specific Objectives:

1. To analyze whether or not the unexpected factors have effect on dam deformations

2. To analyze the current extent of dam deformation and trend compared to the previous

3. To evaluate adequacy of the current classical method for early detection of deformation changes in the Kariba Dam behaviour patterns

4. To analyze the effects of the human errors to Kariba Dam stability

5. To substantiate how GNSS integration could form part of the solution to problems facing the Kariba Dam monitoring system

1.4 Rationale

Ensuring the uprightness of a dam, bridge or any other concrete structure is critical to the success of a business and safety of a project *(Trimble, 2015)*. The unexpected factors may cause failure of the Kariba Dam which would destroy a lot of property and people living downstream with a Tsunami, and disruption of hydroelectric power generation would hamper economic activities both in Zambia and Zimbabwe, thereby causing loss of employment and revenue to the countries concerned. Therefore, there is need to know effects and the extent of the effects induced by the unexpected factors at the Kariba Dam. This, therefore, requires improved geodetic monitoring, analysis and visualization methods if the dam safety is to be ensured. Ensuring safety of the Kariba Dam lies in the improved deformation monitoring system which is capable of measuring accurately and reporting on the dam status in real time via text messages and website. The quick alert issuance of dam deformations to classified users may help in timely implementation of mitigation measures in case of a disaster.

Thus, it was important to evaluate the dam's current deformation status for monitoring and planning purposes.

2.0 BACKGROUND

2.1 General

The Kariba Dam is located (16^{o}31’18” S, 28^{o}45’41” E) almost midway down the Kariba Gorge of the Zambezi river basin on the border between Zambia and Zimbabwe. It is a double curvature concrete arch designed by a French firm called Tractebel Engineering, and was constructed between 1956 and 1959. It has a 128-meter high wall, crest width of 13meters, 24meters base width and 618 meters long wall. It created then the world’s biggest man-made reservoir which impounds about 181 billion cubic meters of water. Its owner, Zambezi River Authority (ZRA) was established as a corporate body through parallel parliament legislations of Zambia and Zimbabwe in 1987. It feeds water to two underground hydropower stations located on the North bank in Zambia and on the South bank in Zimbabwe with a combined capacity recently increased to 1470MegaWatts (*Coyne and Bellier, 2011*).

Figure 2.1: *Kariba Dam location (left) and general view (right); courtesy of ZRA*

2.1.1 Kariba Dam design loads

The Dam has six inspection Galleries (A, B, C, D, E & F) which run from North to South Bank, as shown in figure 2.2 below. Gallery E is discontinuous at the spill way section due to the flood gates, hence not shown in figure 2.2. The galleries are also used for traversing and levelling inside the dam arches. Kariba Dam was designed to withstand effects from foreseen loads such as hydrostatic pressure, temperature variations and the uplift pressure from ground water (*ZRA, 2012*).

The resulting stresses and strains from these loads on the dam are measured against fixed points such as North Bank (NB4) with survey instruments (see figure 2.2 below). The strains and stresses are detected by monitoring dam deformations on the object monitoring points such as NB20, CD2 and K203 located on the dam. The tilts of the dam are measured also with pendulums, for example, the ones shown hanging at point labelled K203 in figure 2.2 below (*ZRA, 2012*). All the forces acting upon the dam cancel each other such that the resulting net force is zero, thus the dam is said to be stable. The cancellation of the forces at play is aided by anchorage of the dam on the hill sides through the dam abutments, foundation footing and design mechanisms such as the footing drainages.

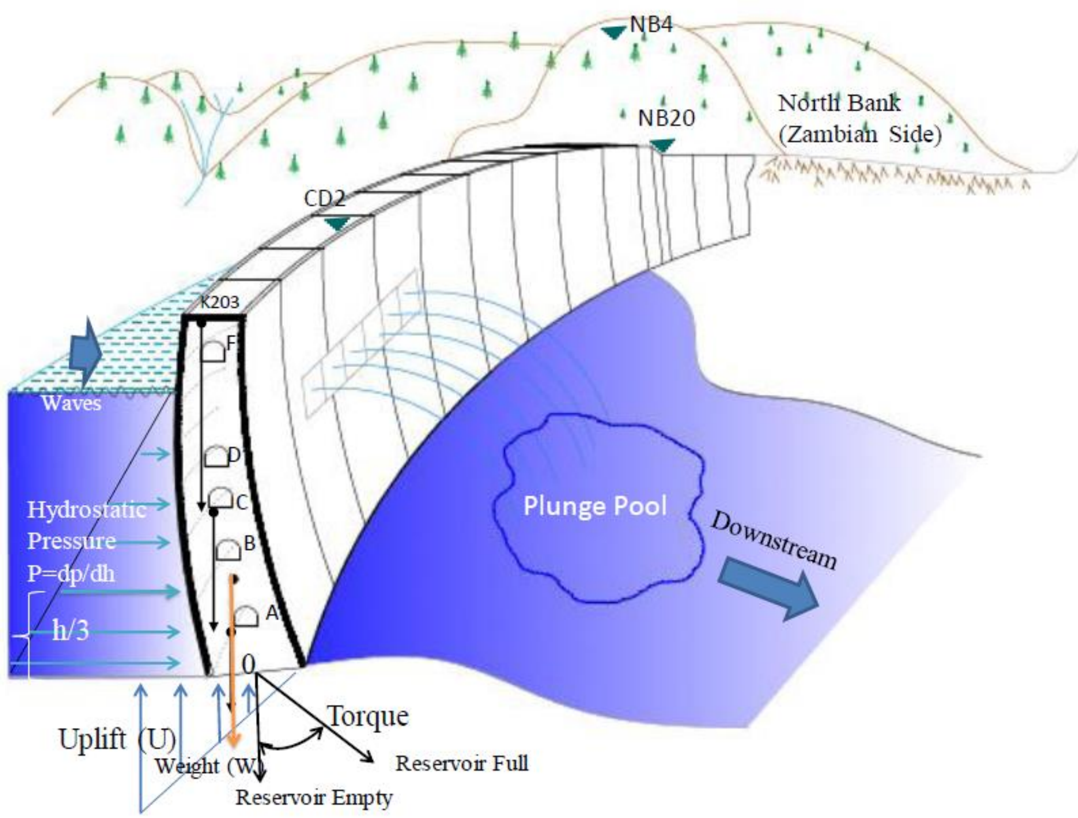

Figure 2.2: *Cross section of Kariba Dam, viewed from Zimbabwean side (South Bank)-not to scale*

The following are some of the major design loads of the double curvature concrete arch dam (*Simscience, 2015*):

- Temperature variations
- Hydrostatic pressure (P)
- Uplift (U) pressure
- Dam weight (W) or dead load

2.1.1.1 Temperature Variations

Concrete expands under high temperature and contracts under cool conditions. The repeated expansion and contraction causes concrete to crack superficially due to stresses and strains build-up inside the concrete (*Whitelow, 1991).* At Kariba Dam, temperature is known to cause the upstream dam drift in the hot season (from August to February) and downstream in cold season, from February to August *(Coyne and Bellier, 2011)*. It has also contributed to cracks on the concrete surface which ZRA Engineers have assessed to be superficial and that the phenomenon is normal as surface cracks on concrete dams are expected. Kariba Dam was designed to withstand the seasonal dam drifts, stresses and strains caused by temperature variations (*ZRA 2000*). However, a 5-Yearly Inspection Report of 2011 by Coyne and Bellier stated that there were deeper cracks and leakages in some parts of the dam, without attributing the problem to temperature variations.

2.1.1.2 Hydrostatic Pressure

The Kariba Dam impounds about 181 billion cubic meters of water in its reservoir at its maximum capacity (*Coyne and Bellier, 2011*). This water exerts horizontal pressure (P) against the dam in the shape of a triangle as shown in figure 2.2 above. Hydrostatic pressure increases with depth, with its resultant force being applied at one third of its height (h/3)-center of gravity of the pressure triangle *(Whitlow, 1991*) as shown in figure 2.2 above. So at the reservoir surface, static reservoir water exerts less pressure against the arch dam compared to the bottom where it exerts maximum pressure. Thus, when the reservoir is full, the hydrostatic pressure tends to drift the dam downstream. It does not only push the dam downstream but also creates a torque about '0', in figure 2.2 above, which swings the dam footing downstream as shown in the same figure. It (Hydrostatic Pressure) has an opposite effect when water level reduces (*Coyne and Bellier, 2011*). Hydrostatic pressure has the same deformation effect as that of temperature variations on concrete (*Coyne and Bellier, 2011*). This means that between August and February, the dam drifts upstream because the temperature is high and water level is low; the dam drifts downstream between February and August due to increased water level in the reservoir and reduced temperature (*Coyne and Bellier, 2011*).

2.1.1.3 Uplift pressure

Just like any other concrete dam, Kariba Dam faces a force being exerted right from its bottom by ground water when the aquifer becomes saturated. The Uplift or pore pressure is minimized through water drainages in its footing (*ZRA, 2010*).

2.1.1.4 Dam weight

The dam weight of the Kariba Dam (W in figure 2.2 above) is designed to act downwards along its center of gravity to the foundation where the loads are distributed. This weight provides a counter balance force to Uplift and Hydrostatic Pressure such that the resulting effect is dam stability (*Coyne and Bellier, 2005*).

In addition to its weight, Kariba Dam achieves its stability through its double curvature; horizontal and vertical curvatures as shown in figure 2.2 above (*Coyne and Bellier, 2005*). The horizontal curvature is responsible for distributing the hydrostatic pressures horizontally through the arch to the abutments, while the vertical cantilever curvature pushes the head-on hydrostatic and hydrodynamic pressures backwards. The stability action described above is what is referred to as Egg-shell effect of the Kariba Dam. Thus, for as long as the dam faces no net force, stability is guaranteed, and hence its (Kariba Dam) safety.

However, dam stability should not be taken for granted or assumed on account that the dam is designed to withstand effects from the above loads; regular deformation monitoring by precise instruments is necessary. For this reason, Kariba Dam is regularly monitored to ascertain its stability.

2.1.2 Kariba Dam deformation monitoring

To ensure Kariba Dam deformations (caused by the above design loads) are within its expected limits, monitoring the dam deformations accurately was made mandatory. It is for this reason that Zambezi River Authority (a successor to the Central African Power Corporation, CAPCO, which formerly managed the Dam) was given the mandate to manage the infrastructure by both Zambian and Zimbabwean governments under the Zambezi River Authority Acts No.17 and 19 of 1987, respectively, pursuant to the Inter-Government Agreement of 1986 between the two countries (*Munyaradzi and Mukosa, 2010*).

Article 9 of 1987 Acts outlines the functions of the Authority as follows:

1. operate, monitor and maintain Kariba Complex (Kariba Dam and Reservoir, as defined in Article 1 of the same Acts)

2. collect, accumulate and process hydrological and environmental data of the Zambezi River for the better performance of its functions and for any other purposes beneficial to the contracting states

3. Make recommendations as will ensure the effective and efficient use of the waters and other resources of the Zambezi River.

2.1.2.1 Geotechnical installations

Thus, ZRA installed an array of geotechnical instruments on the dam (pursuant to its outlined functions in the 1987Acts) such as crackmeters, piezometers, plumb bobs or pendulums, extensometers, strain-meters, etc., in addition to the geotechnical instruments installed during construction. The geotechnical instruments are read every two weeks, mainly through the inspection Galleries (*Coyne and Bellier, 2005*). These instruments check amount of tilt, strains and stresses within the concrete structure (*Coyne and Bellier, 2006*).

2.1.2.2 Anchor Bars

Due to the poor south bank geology, one-hundred-ton anchor cables were installed in the unstable side of the south bank. This was done in order to anchor the sliding mass to the stable gneiss rock and limit its movement towards the river. Extensometers were used to control the final dimensions of the fastening bar. Measurements are taken of the toggle bar to detect any dimensional changes that might indicate a loss of force in the cables, hence the need for re-tensioning (*ZRA*, *2004*).

However, the geotechnical instruments give measurements which are not geo-referenced and this makes it difficult to know the dam deformation trends or direction of deformations over time (*ZRA*, *2005*).

2.1.2.3 Geodetic deformation monitoring system

To avoid getting unreferenced results only, ZRA supplemented the geotechnical method with the geodetic deformation monitoring techniques which give geo-referenced (earth-related) results. Thus deformation results are related to local directions (North, South, Upstream or Downstream). The relative description helps determine whether or not the dam drifts are in the expected directions and amounts. Geodetic deformation monitoring of the Kariba Dam is done by taking periodic measurements (February and August) on the Dam Crest and Galleries using a Total Station for direction and distance measurements to the prisms during triangulation and traversing, and digital level for crest and gallery surface levels during leveling (*ZRA,2012*).

The frequency of measurements (twice a year) using this method was chosen by ZRA because major water levels and temperature variations, the envisaged main causes of dam deformations, are felt in February and August. The effect of reduced winter temperatures and increased water level on the dam is checked geodetically in August when the dam is expected to deform downstream due to increased hydrostatic pressure in the reservoir. The effect of increased summer temperature and reduced water level is checked in February when the dam is expected to deform upstream (*ZRA*, *2012*). The intermediate measurements on the dam are done with geotechnical instruments and the results of which are confirmed by geodetic measurements twice a year.

The deformation monitoring of the Kariba Dam started in 1963, but was not done as frequently and as accurately as it is now (ZRA, 2012). Frequent and accurate monitoring of the dam started in 1989 which provided results which are being used as reference to date (*ZRA, 2012*). Improvements to the monitoring system of the Kariba Dam was being made over time with new installation of geotechnical instruments and acquisition of accurate survey instruments in 2012 (*ZRA,2012*).

The Geodetic Surveys on the Kariba Dam which are carried out periodically comprise three exercises: Traversing for underground deformation measurements in tunnels and dam galleries, Triangulation and Levelling for dam crest measurements (*ZRA, 2005*).

2.1.2.4 Principle behind Kariba Dam monitoring

Geodetic deformation monitoring of the Kariba Dam, like any other deformation monitoring system, is based on Albert Einstein's theory on special relativity of motion which states that the laws of physics are same everywhere but are provincial or local. This means motion is viewed differently from different locations (*Nola, 2015*). For example, a person aboard an aircraft does not feel the speed of an Aircraft because the eyes (sensors) have no stationary (fixed) objects to relate the speed to, while an observer on the ground sees an Aircraft flyby at a super-fast speed because he/she is stationary and can relate the speed to stationary objects on the ground. Another example is when two cars are moving in the same direction; a car in front is going at 120km/h while the car behind is going at 130km/h. The drivers in the two cars will only observe 10km/h as an overtaking speed of the behind car while an observer standing by the road side will observe the speeding cars at their actual speeds. For a stationary observer to judge the direction of the cars, he/she must know which direction is North or East as reference.

The concept from this theory is that in order to detect the actual relative motion, the measuring sensors, eyes inclusive, must be in fixed positions. The sensors must take reference to fixed points in certain directions as reference in order to measure direction of movement.

2.1.2.5 Kariba Dam reference networks

In order to adhere to Albert Einstein's theory stated above, fixed points called Geodetic Control Points (GCPs) were monumented on and around the dam area where the sensors such as Total Station and Digital Levels are referenced to in order to measure dam deformations. The directions of deformations are referenced to certain directions between two or more fixed control points assumed to be outside the deforming zone of the dam (*ZRA*, *2012*). The reference coordinate system for Kariba Dam deformation monitoring (see figures 2.3 and 2.4 below) is a local Cartesian where the X-Axis is positive in the downstream direction, Y-Axis positive in the North Bank direction and Z-Axis positive in the vertical. For example, a negative measurement in the X-axis means dam deformation is upstream while a negative in the Z-axis means the point is sinking. See figure 2.3 below.

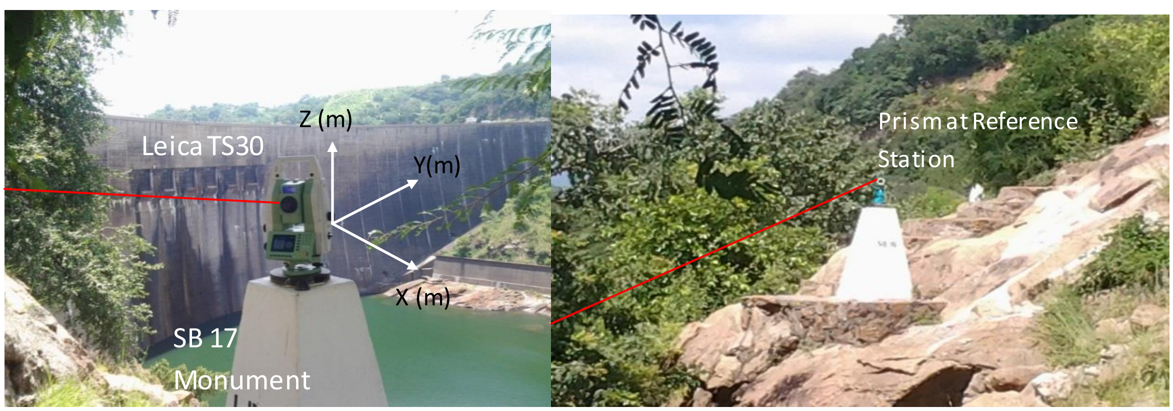

Figure 2.3: *Total Station (right) measuring to Prism (left); courtesy of ZRA*

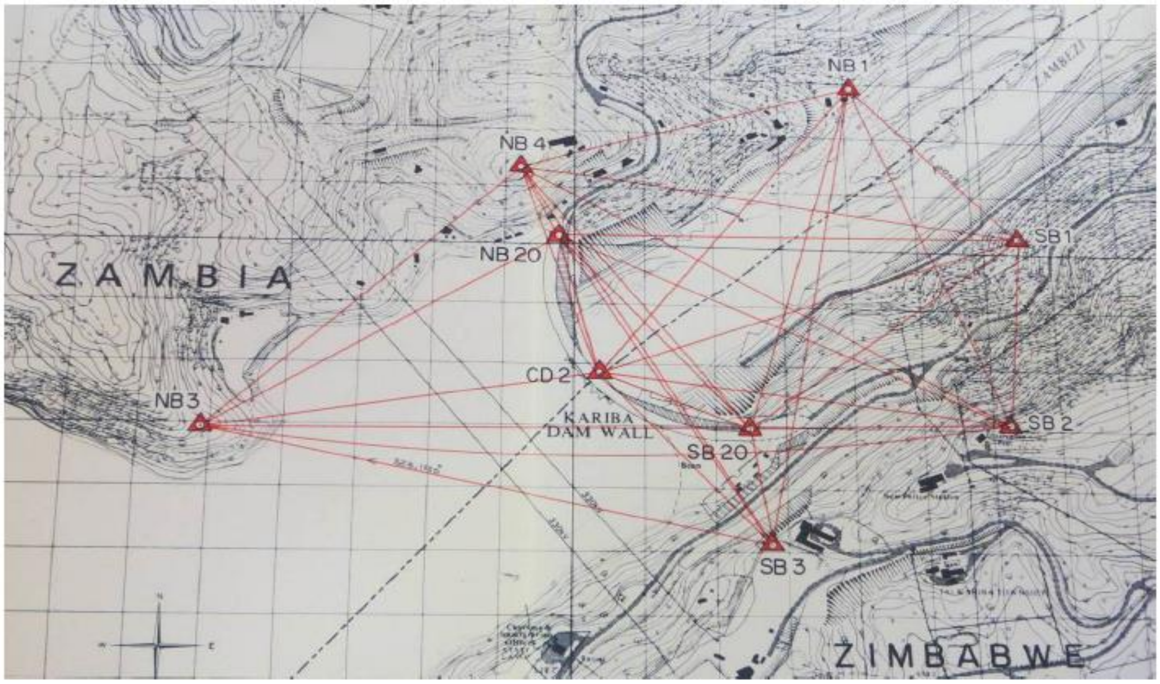

Figure 2.4: *Primary Geodetic Control* Points *against which deformations are measured; courtesy of ZRA*

The control points shown in figure 2.4 above constitute what is called a primary network, and are assumed to be in fixed positions, except those on the concrete dam which define the geometry of the object being monitored. Those placed on the dam are, however, considered primary for only a particular session once their coordinates are checked and updated with reference to NB4 and NB3 (monuments assumed to be fixed and therefore used as baseline for reference bearing). The assumption, when adopting dam controls as primary, is that the deformation is slow such that the coordinates of the dam-placed primary controls remain as accurate as the rest of primary points for a particular measurement session, as long as the measurements are done within a day during which deformations are not significant enough to cause errors in the measurements (*ZRA*, *2009*). The primary controls form a fixed reference against which Kariba Dam deformations are measured.

The other network is called secondary which is assumed to be in deforming zones. This constitutes monuments on the South Bank (unstable ground due to poor geology), as well as some on the dam. The secondary points form objective points where deformations are detected with reference to the primary network. This way, ZRA Engineers have been able to detect dam deformations and landslide on the South Bank.

As a precaution, ZRA monitoring team assesses the primary controls by taking measurements and give description of their respective stability status before using them as reference for deformation monitoring of the dam (*ZRA, 2008*).

2.1.3 Unexpected dam stability threats

The traditional geodetic deformation monitoring method has worked well so far. However, Kariba Dam is faced with stability-threating effects from the following factors which may not adequately be monitored under the current classical methods as they were not catered for in the current instrumentation design. This is evident by the fact that the real cause of vertical cracks and water leakages in some parts of the dam has not been established (*Coyne and Bellier, 2011*).

2.1.3.1 Plunge Pool erosion

Kariba Dam has a plunge pool which has scoured to a depth of more than 90m by the discharged water, beyond the expected depth of 45m to 50m (*George et al, 2015*). The major problem is the back cutting of the plunge pool towards the dam, threatening its foundation stability (*George et al, 2015*). ZRA has since engaged consultants who have proposed options to address the problem. One of the options is that of blasting and widening the plunge pool and removing about 300,000m^3 of rock, thus reducing excessive turbulence which scours the plunge pool *(International Rivers, 2012)*.

According to International Rivers Report of 2009, questions have been asked as to whether vibrations from the maintenance earth works will not threaten dam stability; even after the blasting of the plunge pool wide, will the bedrock remain as effective as it always has been in providing anchorage to the dam footing? It must be borne in mind that the hydrostatic pressure increases with dam depth (*Coyne and Bellier, 2011)*. The major risk is that any weakness in the foundation may result in dam failure as it may not withstand the torque exerted by the hydrostatic pressure at full reservoir capacity (*International Rivers, 2012*).

In early 2015, a Panel of experts on dam safety discovered that the plunge pool is progressively eroding towards the dam toe along a fault line which runs upstream/downstream, and was unforeseen before the dam construction (*George et al, 2015*). See the current plunge pool status in figure 2.5 and 2.6 below.

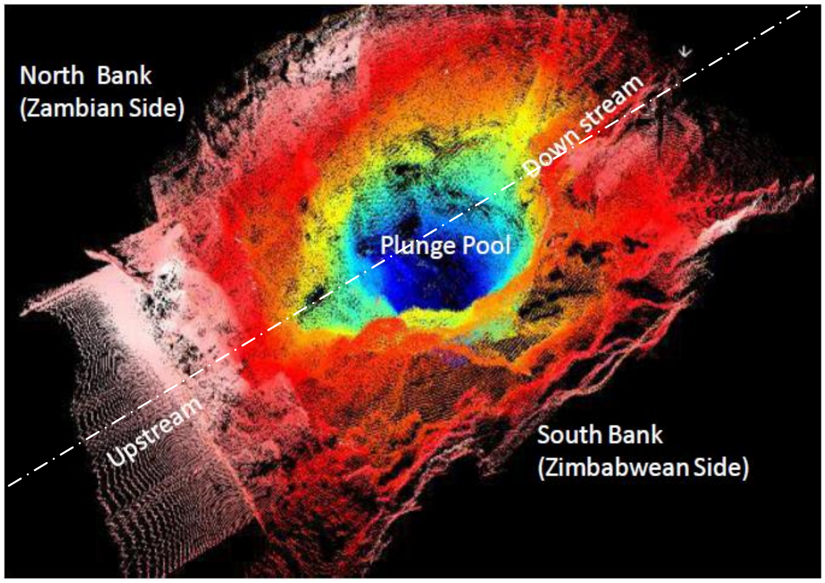

Figure 2.5: *Current status of the Plunge Pool; courtesy of ZRA*

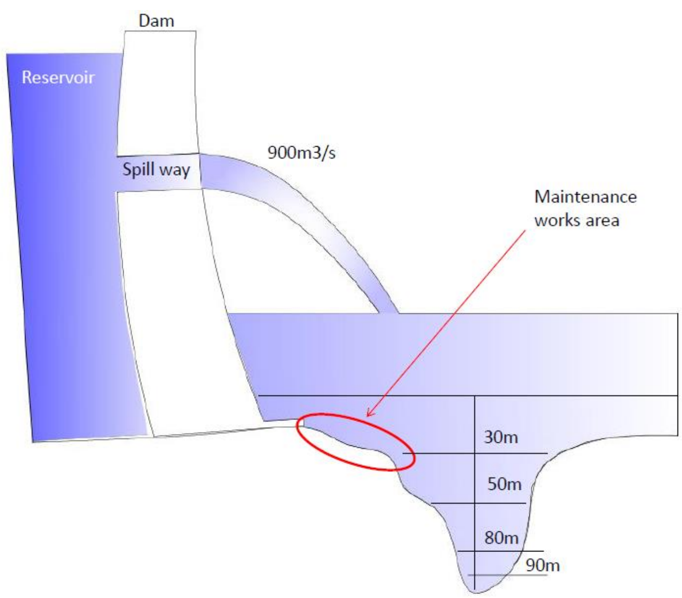

Figure 2.6: *Plunge Pool depths, longitudinal section (upstream-downstream) of figure 2.5*

2.1.3.2 South Bank Landslide

The geology of the Kariba Dam south bank has deteriorated such that the land mass above hydroelectric power tunnels is sliding towards the river (*ZRA, 2005*). The geological formation of the south bank consists of fractured gneiss and quartzite with micaceous bands. Infiltration of water and poor geological formation are major contributing factors to the drifts of the south bank towards the river (*ZRA, 2005*). The fracturing of rocks might have been caused by human activities, physical weathering through temperature variations which cause the rock to expand/contract and tree root growth which penetrated the bedrock some years after the first geological research before constructing the Dam *(ZRA, 2005)*. The fractured rocks caused by physical weathering and more water infiltration into the ground resulted in the landslide which has been described by ZRA in a number of reports in the last few years.

This land slide constitutes two slides:

- A Shallow slide (above hydropower caverns), occurring along the gneiss/ quartzite contact interface starting at elevation 490m at the top to elevation 415m towards the base of the south bank slope as shown in figure 2.7 below.

- A deeper slide, more or less parallel to the slope at about 50m depth below the gneiss/quartzite contact zone (*ZRA, 2005*). The quartzite layer being the outside slope layer, shaded gray and the gneiss layer being the unshaded layer between the deep slide and the quartzite layer. See figure 2.7 below.

Note: *It must be borne is mind that no land mass is truly stable around dam areas, but the degree of instability differs. Hence, the two zones of both South Bank are said to be sliding zones. However, the deeper zone is said to drift an acceptable rate, and thus described as stable. The shallow zone is said to be unstable due to unacceptable rates of the slide.*

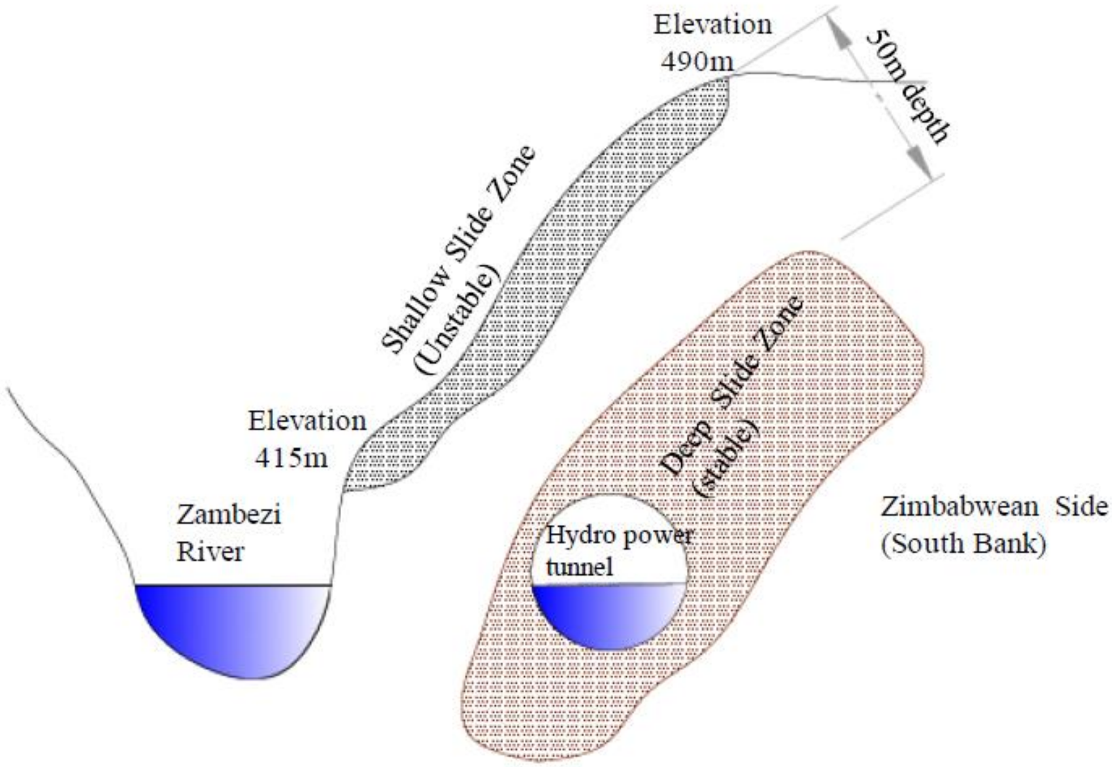

Figure 2.7: *Cross-Sectional view of the Landslide zones of the South Bank (not to scale)*

2.1.3.3 Alkali Aggregate Reaction (AAR)

Kariba Dam is also faced with a chemical reaction called Alkali Aggregate Reaction (*Harad et al, 2015*). This reaction comes about because, alkali, a chemical in cement reacts in moist conditions to produce a gel which hardens and ultimately increases the dam volume (*Harad et al, 2015*). The effect is the heaving of the concrete in upstream and upward directions, which is an undesirable deformation (*ZRA, 2010*). This was detected in the 1970s through the use of pendulums, for example, those dangled as shown at K203 in figure 2.2 of the dam design; evidenced by a slow upward trend in the crest levelling data, confirmed in late 90s by the strain-meters embedded within the concrete, precise levelling within the galleries and by laboratory testing. The annual swelling rates have been assessed at 5-10.10^{-6}mm/year in the dam and at 15-20.10^{-6}mm/year around the spillway; with a reduction tendency by the year 2000 and up to 2011(*Coyne and Bellier, 2011*). Significant reduction in the AAR effects was observed in the recent years' deformation results, but still has effects on the general dam behavior and its overall safety (*Harad et al, 2015*). Particularly, AAR negatively affects the normal mechanical operations of flood gates by jamming them (*Harad et al, 2015*).

The following figure 2.8 shows the dam swelling under the effluence of the AAR.

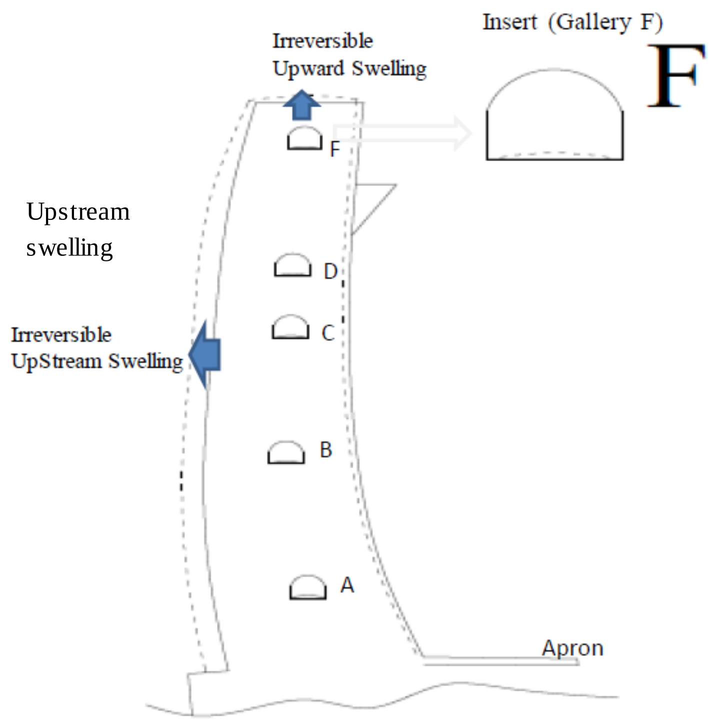

Figure 2.8: *Effects of AAR on the concrete arched dam; section view parallel to main dam Axis*

The irreversible radial upstream deflection of the South end arch of the dam was assessed to be 13.7mm and 8mm on the North and South arch end, respectively, over a period of 20 years (*Coyne and Bellier, 2011*). These measurements were observed from pendulums in Gallery F. The upward swelling rate was assessed as 1.5^{-6}mm/ year over a period of ten years (*Coyne and Bellier, 2011*).

2.1.3.4 Unusual Dam behaviour

2.1.3.4.1 Tangential movement

Both South and North ends are slightly moving along their tangential direction to the north: by 7.2mm on the Southern side and 3.4mm on the Northern side, over a period of 20 years. These movements were not confirmed by pendulums as they are located too far from the arch ends (*Coyne and Bellier, 2011*). These, however, movements (see figure 2.9 below) were described as being of less concern by Tractebel consultants.

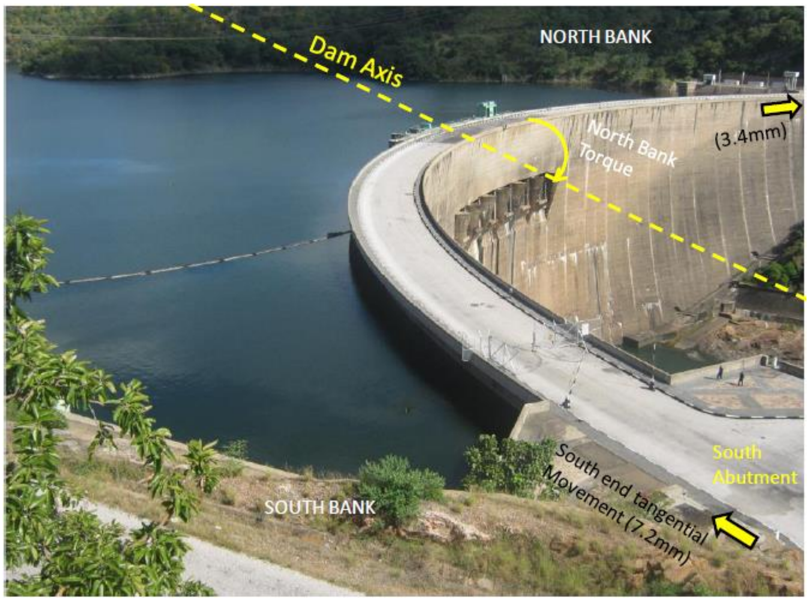

Figure 2.9: *Northward movement of the dam*

2.1.3.4.2 Rotational movement

Another assessment on the dam behaviour towards its foundation was also conducted in Gallery B which is situated way below the spillway gates (*Coyne and Bellier, 2011*).

The results from geodetic monitoring system were evaluated by Coyne and Bellier and showed that central/north section tends to deflect slowly downstream. This deflection was assessed to be 1mm from 1989 to 1999, 1.7mm from 1999 to 2009, and 1.4mm from 2009 to 2010. These, deflections were not confirmed by pendulums either, because the north end arch is far from the pendulum positions.

Particularly, the 2009 to 2010 downstream deflection of the north arch end attracted the attention of the consultants (Coyne and Bellier). Coyne and Bellier noted that this deflection was abnormal as it represented between 21 to 33 percent of the F-gallery radial movements, a phenomenon which contradicted the long known trend; that upper sections of the dam deflect more than the lower ones. This deflection was attributed to the over-spillage of the water which cooled the downstream face in 2010 (*Coyne and Bellier, 2011*). Figure 2.10 below illustrates the rotational deflection described above by Coyne and Bellier.

The real cause, however, of this unusual dam deflection was **unknown** (*Coyne and Bellier, 2011*). Thus, Coyne and Bellier in their 2011 report recommended more inquiry into this phenomenon.

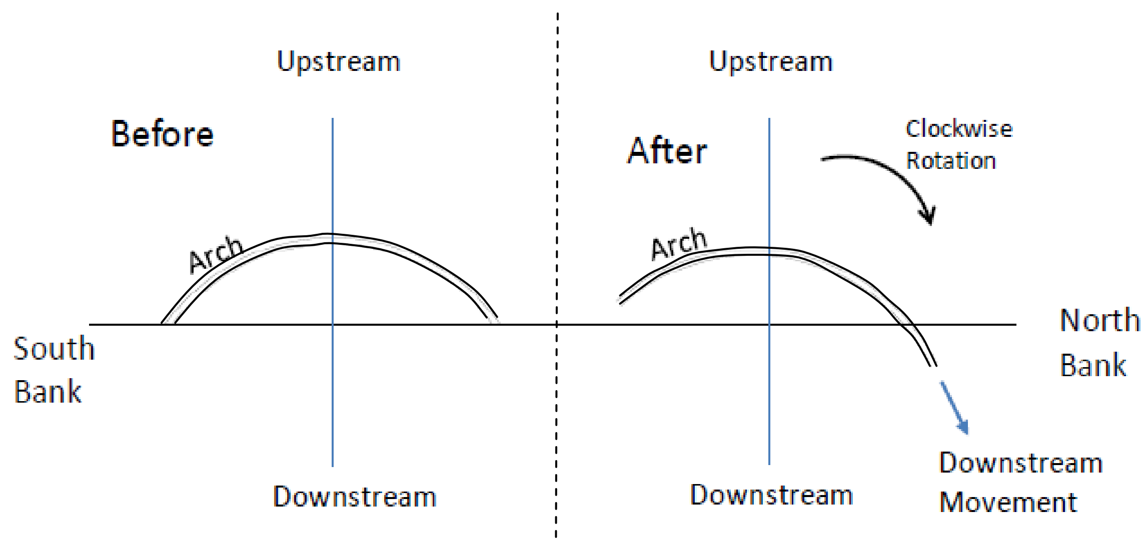

Figure 2.10: *Kariba Dam misalignment problem about its upstream-downstream Axis (Coyne and Bellier, 2011)*

How could the rotational movements, assessed by the dam consultants, be fully detected and controlled?

The answer to this question lies in the proposed deformation monitoring system design which is discussed in Chapter 5.

2.1.3.5 Reservoir Induced Seismicity (RIS)

New scientific studies have proven that dam walls more than 100m high can induce regional earthquakes called RIS, according to International Rivers' 2009 report about the main causes of dam disasters; the 128m-high Kariba Dam falls in this category.

The impounding of Kariba Dam reservoir was characterized by earthquake activity. This was due to the weight of the water which translates into a mass of about 180 billion metric tons pressed upon the valley. As water filled in, some micro-cracks became lubricated with water which caused landmasses to slide over each other (*Gahalaut et al, 2007*).This happened because Kariba Dam reservoir is located in a seismological area, at the southern end of the African Rift Valley(*International Rivers, 2009*). Since its filling in the early 1960s, Kariba Dam has induced numerous earth tremors in the area, 20 of them larger than magnitude 5 on the Richter scale; the largest being magnitude 6.3 in 1963, although the trend has notably subsided now (*International Rivers, 2009*).

Although RIS occurrence at Kariba has reduced over the recent years, it does not mean the threat should be overlooked (*International Rivers, 2009*). The Itezhi Tezhi Dam, for example, experienced an earth tremor of more than magnitude 5.0 in 2011, and was only detected days after by the Zimbabwean Seismography, Itezhi Tezhi dam safety experts said. So the same may happen at Kariba dam. Therefore, the Kariba Dam still has the potential to trigger a regional earthquake which can potentially cause it to fail.

The possibility of Reservoir Induced seismicity was not taken into account in the design of Kariba Dam, hence seismic activity's effect on the dam's safety is unknown (international rivers,2009). RIS can potentially amplify the hydrostatic pressure by triggering bigger waves in the reservoir which may push the dam off-stability. Given the possible compromised foundation from plunge pool earthworks (about 300,000 cubic meters of material removal), failure may result (International Rivers, 2009).

In response to the above stability-threating factors, Zambezi River Authority has done a number of corrective measures to abate possible dam failure as discussed below.

2.1.4 Landslide remedial measures

2.1.4.1 Slope maintenance

Several remedial engineering works were carried out over the years to improve safety of the south bank with technical assistance from Tractebel Engineering, a firm that designed the Kariba Dam (*ZRA, 2005*). The following were some of the remedial works undertaken by Zambezi River Authority:

1. Drilling of shafts and adits to improve the underground water drainage system
2. Removal of surface micaceous bands and weaker material and replacing the same with concrete as shown in figure 2.11 below to reduce water ingress into the slope which would trigger more landslide.

Figure 2.11*: Stone-pitched slope of the South Bank at Kariba Dam;Courtesy of ZRA*

The above mentioned maintenance works have only reduced the danger by about 53 percent. The annual landslide speed before the remedial works was reported by ZRA as ~3mm while after the stabilization works were done, the annual landslide rate reduced to 1.6mm/year *(ZRA, 2012).*

However, the drill and blast operations at the Kariba South bank for the hydropower cavern extensions which cause vibrations and shock waves are a danger to the integrity of the dam as they have potential to also accelerate the landslide towards the river, thus blast and drill vibrations are monitored to keep vibrations within the specified safety limits (*Coyne and Bellier, 2011)*.

2.1.4.2 South bank network expansion

The separate rigid secondary geodetic network was created to monitor the south bank slope stability (*ZRA, 2012*). This network was designed by Walter Schneider Ltd (now Schneider Ingenieure AG) with joint cooperation from Coyne and Bellier and Zambezi River Authority (*ZRA, 2012*). Introduced in 1991, the boundary of this network starts from the main network beacon SB3 on the south bank hillside, to the crest beacon CD2, PS2 on the downstream slope, to SB1 and south bank pendulum shaft P4.The shallow slide zone is monitored through surface beacons SB17, SB19, SB21, PS1 and PS2. The upstream midsection is monitored through SB16 and IB3. SB5 and SB8 monitor areas between upper and lower sections of the international road where there is a mica layer complex. The deeper zone are monitored through the traverse network which starts from SB11, connecting to SB12 and adit-1440 beacons (SB13 and SB14). This traverse exits at SB 15 which is to the eastern side of the underground adit 1440 (*ZRA, 2012*). See drawing 5, south bank network, in Appendix C.

However, in the ZRA 2012 south bank report, it was mentioned that two years of landslide measurements were not adequate to characterize the south bank drifts; the measurements done in 2012 of the south bank was just to confirm whether or not the landslide had reduced. This was after precautionary measures were taken, following reports that the south bank was moving towards the river. It was also recommended in the ZRA 2012 report that there was need for more years of measurements on the south bank in order to obtain full reflective information about the landslide.

The remedial measures, however, have not stopped public concerns about the Kariba Dam safety.

2.1.5 Kariba Dam stability concerns

Despite the remedial measures on the dam, the members of the general public have noticed cracks on the dam and have raised concerns about the Kariba Dam stability. Leaks and cracks behind stop beams were confirmed in 2011 by visual dam inspections, done every 5 years by Tractebel Engineering team. Amongst the type of cracks observed were Y-cracks, horizontal and vertical cracks. The Y-cracks were observed in a number of galleries, including Gallery B and F. The vertical cracks and honeycomb (concrete-surface-pilling off) were observed in the air-vent shafts as well (*Coyne and Bellier, 2011*).

Some of the signs have been described as being normal in a number of reports about the Kariba Dam status. Only a mention of few contradicting findings has been reported, findings of which have been attributed to faulty instruments and unusual spillages (*Coyne and Bellier, 2011*).

On 3 October 2014, BBC News reported that Kariba Dam is in a dangerous state. The media pointed out that the dam was built on a seemingly solid basalt rock but the erosion of the crater from the spillway torrents has reached its foundations in 50 years' time, and this could potentially cause the dam to fail if no repairs were done (*IRMSA, 2015*).

Thus, it was vital in this study to look at how some of the bad choices made in dam management by those responsible may lead to dam failure. Hence two dam failures were looked at: Vajont and Teton dam failures.

2.1.5.1 Dam failure attributed to human errors

Human errors played a major role in the failure of Teton Dam in the United States and Vajont Dam in the Italian Alps, back in 1976 and 1963, respectively.

2.1.5.1.1 Vajont Dam failure

Failure of the Vajont Dam occurred, in its early filling stages, in the Italian Alps after a landslide triggered the Vajont dam failure in 1963. Before the landslide, the engineers claimed the dam was stable, and ignored the signs, and now Vajont has been called one of the "greatest failures of science and engineering" (*Andrew et al, 2015*).

The following were some of the bad choices which contributed towards the mighty Vajont Dam failure (*Andrew et al, 2015*):

1. Increasing the dam height by 66 meters to impound about 200% more reservoir volume by the Italian Government. The aggressive move by the government came after the geological study and engineering studies on the capacities of the dam (*Human factor 1-changing the dam in a more ambitious and risky way without thorough technical ground work as studies conducted had flaws*)

2. After the design modification, one expert warned the designer that the modification might cause geological problems, but the authorities refused to conduct an expert review. Later, three independent experts also concluded that the mountainside was dangerously unstable. Their reports were ignored (*Human factor 2 & 3-avoiding critical inquiry and disregarding conflicting information*). Thus the expansion project proceeded to advanced stages without thorough slope assessment

3. Hiding of non-linear multiplication of stresses resulting from the modification played a major role in the failure of the Vajont Dam. The authorities did not appreciate this change in the stresses (*Human factor 4-lack of technical understanding*)

4. In 1959, a journalist who predicted disaster was publicly denounced; the decision-makers asserted that the geology and stability of the area was well understood and that the dam was stable (*Human Factor 5 & 6-"shooting the messenger" and overestimating one's own knowledge*)

5. In 1960, Landslide in another Alps reservoir at Pontesei created a Tsumani which overtopped that dam, and this caused several geological studies to be conducted at the Vajont dam. One of the geologists concluded that there was a prehistoric landslide at Vajont hillsides which threatened Vajont Dam stability. Other consultants on the project, and the general scientific community, did not accept the geologist's theory that the landslide existed because the slopes extending into the reservoir composed of intact geological layers which gave a superficial illusion that there was no landslide. A possible landslide was detected during one of the reservoir filling sessions, through measurements, but it was very slow. Later, after the filling was stopped, measurements proved the landslide had stopped. Thus, the general consensus on the project was that the landslide was not a threat because it could only generate slight motions under gradual filling of the reservoir (*Human Factor 7-optimistic misunderstanding of the risk*)

Finally in 1963, during some filling stage of the reservoir, the landslide was detected again, and the water in the reservoir had dropped. It was surprising that the landslide accelerated during the low water levels in the reservoir. In no time, a 2km wide, 250meter thick of rock plunged into the reservoir, causing a water wave which triggered an abnormal hydrodynamic pressure which pushed Vajont Dam off equilibrium, killing 2,000 people, who lived downstream, in just 7 minutes (*Andrew et al, 2015)*.

2.1.5.1.2 Teton Dam failure

Another dam disaster was Teton in the United States of America in 1976. The failure was attributed to the wrong analysis and interpretation of dam deformations (*Andrew et al, 2015)*.The following were some of the human errors that led to the failure of the dam (*Andrew et al, 2015*):

1. In its early construction stages, an independent team of geologists raised a concern that the dam would be in danger of imminent seismic collapse, and drafted a memo about it. The geologists' supervisors objected to the strong concerns in the memo, and it was redrafted many times until the memo was void of urgency and maybe absolute unclear (*Human Factor 1-diluting communications content for social reasons*). Thus the memo had no apparent impact on the Bureau of Reclamation (BoR) which was constructing the dam

2. A surviving negligible note written by BoR geologist showed that the only recorded reaction to the geologists' memo was: intention to prepare some "constructive criticism" (*Human Factor 2-dismissing contradicting opinions and evidence*)

3. At the time when the concerns were raised, the BoR had already spend $4.6 million on the project which was going to cost $100 million to complete, and the dam failure was going to cost the government about $200 billion. The BoR disliked the idea of having to heavily alter the project, owing to large investments (*Human Factor 3-reluctance to abandon sunk costs*)

4. The earthquake, however did not cause collapse; cracks in the bedrock did: During construction, cracks were underestimated as being less than 2mm, when actually the 'cracks' were caves large enough to walk into (*Human Factor 4-incomplete investigations which might have led to inappropriate assumptions*)

5. The perceived cracks led to grouting (crack filling), and the cement used was twice the estimated in the Bill of Quantities (BoQs). After the grouting, the reservoir started filling, and the Engineers doubted the filling rate twice i.e. the actual filling rate was two times the expected (*Human Factor 5 & 6-misconception and doubting actual measurements*)

6. After the two doublings in 5 above , the Bill of Counting (BoC) memo showed that functioning monitors (14 of 17) indicated a ground water flow 1,000 times what was expected earlier; the memo concluded that the monitoring system was

faulty (*Human Factor 7-wrongheaded conclusion which fits expectations, not evidence*)

7. Two weeks later, two leaks appeared over two days; the project Engineer did not get worried and ignored the leak, since some degree of leaking is normal for such big dams (*Human Factor 8-reasonable-seeming general interpretation, but maybe not-in- line with a current situation*)

Finally, three more leaks appeared, this time in rapid succession, which now worried the engineer but it was too late as billions of gallons of water soon gushed out downstream, washing away the Teton Dam in 1976 (*Andrew et al, 2015)*.

Unfortunately, none of the above numbered items, in current dam failures, are addressed by improving the design code of dams. They all appear to emanate from such elements as faulty investigations, disregarding of conflicting information, assumptions about evidence, and pressures of financial commitments (*Andrew et al, 2015)*. This means that human errors are the largest causative factors in dam failures.

This does not mean humans are to blame entirely for dam failures; some dam stability situations may just be outside human control (*Andrew et al, 2015)*.

Having gathered the knowledge and lessons of how concretes dams fail due to bad choices, it was important to look at the underlying principles in dam stability, monitoring, analysis, visualization and interpretation in order to facilitate good practices in dam monitoring.

Dam stability obeys Isaac Newtonian laws of motion. Thus, pages 30 to 33 discuss Newton's laws of motion and how they apply in dam deformation monitoring and analysis.

2.2.0 Theoretical Background

2.2.1 Isaac Newton's laws of motion

2.2.1.1 First law

Isaac newton's first law of motion states that every object, Kariba Dam inclusive, will remain at rest or in constant motion in a straight line unless compelled to change its state by an external force. The key point here is that if there is no net force acting on an object (if all external forces cancel each other) then the object will maintain its status of motion. A change in the object motion status will only happen if an external force is applied on an object (*Glenn Research Center, 2015*). The first law of motion answered questions as to whether or not Kariba Dam has net force acting upon it, by analyzing its detected deflections or distortions or motions measured since 1963 to date.

2.2.1.2 Second law

The second law explains how the velocity of an object changes when it is subjected to an external force. It states that the force **F** (for an object with a constant mass) is a product of object mass **m** and its acceleration **a.** It is expressed as **F=m*a**. The law defines the force to be equal to change in momentum (mass*velocity) per change in time. For an external applied force, the change in velocity depends on the mass of the object. A force will cause a change in velocity; and likewise, a change in velocity of an object will exert a force (*Glenn Research Center, 2015*). In dam deformation monitoring, rate of change of deformations are calculated using this second law.

2.2.1.2.1 Velocity

Velocity is a vector measurement of the rate and the direction of motion or the rate and direction of the change in the position of an object. The scalar (absolute value) magnitude of the velocity vector is the speed of the motion (*Steven, 2015*). It can be expressed as;

$$v = \frac{d}{\Delta t} \tag{1}$$

Where **d** is displacement and **Δt** is a period over which the displacement of an object has taken place.

2.2.1.2.2 Acceleration

In physics terms, acceleration, **a,** is the amount by which velocity changes in a given amount of time (*Steven, 2015*). Given the initial and final velocities, v_i and v_f, the initial and final times over which speed changes, t_i and t_f, acceleration can be expressed as (distance/time/time= distance/time^2):

$$a = \frac{vf - vi}{\Delta t} \qquad (2)$$

2.2.1.3 Third law

The third law states that for every action (force) in nature, there is an equal and opposite reaction. This means that if object A exerts a force on object B, then object B will exert equal and opposite force on object A (*Glenn Research Center, 2015*). This law explains how dams achieve their stability. A net force in the free body of an object or dam may spell failure if not monitored and controlled appropriately. Forces acting upon a body or a dam are not always perpendicular to the surface of application; they may be an angle.

2.2.2 Force acting at an angle

The force **F** acting at an angle can be resolved in two perpendicular components, **Fsinθ** and **Fcosθ** (*Graham et al, 2009)* as shown in the figure 2.12 below.

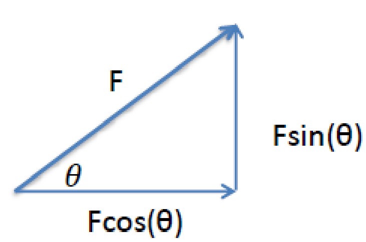

Figure 2.12*: Force resolutions into components*

The Isaac Newton's laws of motion can be applied in each component direction (*Graham et al, 2009*).

2.2.3 Moment or Torque of a force

A force may be thought of as a push or a pull in a certain direction (*Glenn Research Center, 2015*). When a force is applied on an object (dam or landmass) the resulting motion of the object will depend on: where the force is applied, how the object is confined or constrained and its mechanical properties. Kariba Dam and the surrounding landmasses are constrained objects and their deformations are a function of how they are constrained. If the object is unconstrained and the force is applied through the center of gravity, the object moves in pure translation or a tangential movement, as described by Newton's laws of motion. If the object is constrained or pinned at some point called pivot, the object rotates about the pivot, but does not translate the pivot. If the object is unconfined and the force is applied at some distance from the center of gravity, the object both translates and rotates (*Glenn Research Center, 2015*). These movements may be applied in analyzing the kinds of deflections expected on the constrained landmasses around the Kariba Dam where Anchors bars are acting as pivots. Thus, moment or torque is vital in analyzing the south bank landslides due to the poor geological formations.

The product of the force and the perpendicular distance to the center of gravity for an unconstrained object, or to the pivot for a confined object, is called the **torque** or the **moment of forces,** a measure of a force to cause a rotation (*Whitelow, 1991*)**.** A torque or moment of forces is also a vector quantity and produces a rotation in the same way that a force produces translation. The first Isaac Newton's law of motion also applies in the case of a moment. A torque produces a change in angular velocity which is called an angular acceleration (*Glenn Research Center, 2015*). This concept applies in the angular movements noted by Coyne and Bellier in their 2011 analysis, where the dam arch was reported to have translated northwards and downstream. The torque concept described above can be summarized in figure 2.13 below.

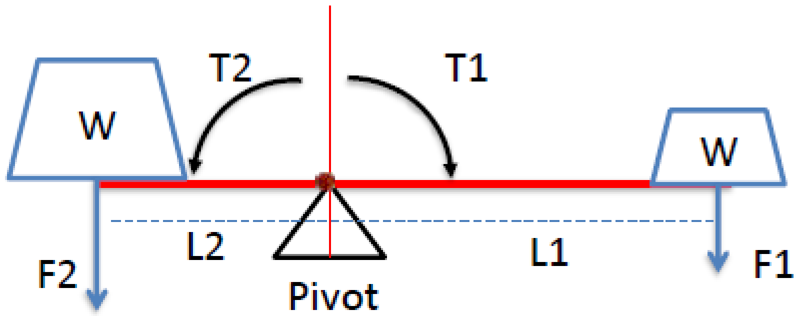

Figure 2.13: *Moment of a force*

For a system of forces creating a torque **T** to balance, both clockwise **T1** and anticlockwise **T2** must be equal; when not in equilibrium, the rotation will be in the direction of the higher torque (*Glenn Research Center, 2015*). This explains how landmasses around the Kariba Dam would remain stable in an ideal situation.

The forces acting on the Kariba Dam and its surrounding areas can only be detected by taking measurements of the dam deflections. The magnitudes of the deflections or distortions are the tell-tale of whether or not the dam or indeed the landmasses are stable. The deflections can be so small (in millimeters) that adjustments or corrections for errors in the results are cardinal.

2.2.4 Least Squares Network adjustment

In deformation monitoring of the Kariba Dam, measurements are always without random errors, hence it becomes important to adjust the measurements. The method used to adjust measurements in deformation monitoring is least squares adjustment methods.

The method of least squares is a mathematical model which uses linear algebra and calculus to determine the best fit line to the data. This model applies in classical monitoring systems also (*Larson and Farber, 2003)*.

2.2.4.1 Mathematical model

Least Squares Model: Given the data set (X1, Y1,….Xn, Yn) which consists of measured points, as shown in figure 2.14 below.

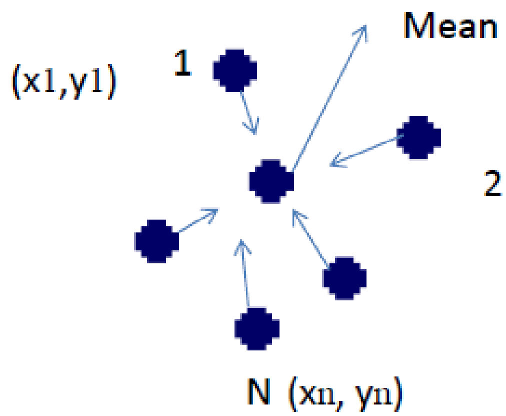

Figure 2.14*: Data-set of measured points of an object point (Larson and Farber, 2003)*

The model calculates the data set mean, **y=ax+b**, such that the standard deviation (E) is:

$$E(a,b) = \sum_{n=1}^{N} (y_n - (ax_n + b))^2 \tag{3}$$

$$\frac{\partial E}{\partial a} = 0, \quad \frac{\partial E}{\partial b} = 0 \tag{4}$$

(Larson and Farbe, 2003).

The assumption in this model is that the standard deviation from the average value must be infinitely small to Zero as shown in the derivatives above. Thus this method works only for medium (depending on accuracy required) random errors in measurements (*Larson and Farbe, 2003).*

The deformation measurements on the Kariba dam are based on a local coordinate system, a reference for detection of movements, pursuant to Albert Einstein's principle on relative motion. Hence, the next sub-topic talk about the surveyor's coordinate used for the Kariba Dam geodetic deformation monitoring.

2.2.4.2 Surveyor's Reference System

A two dimensional plane Cartesian local coordinate system is used for surveying computations of distances and bearings (*Earl, 1986*).

This reference system has a set of mutually perpendicular axes consisting of:

a) The abscissa, a horizontal line along which the X distance is measure and,

b) The ordinate, vertical line along which Y distance is measured.

2.2.4.2.1 Bearing and distance computations

Given two points, 1 and 2, with plane coordinates 1(x_1, y_1,) and 2(x_2, y_2) respectively; the bearing α between the points can be computed as:

$$\alpha = \tan^{-1}[\frac{(x_2 - x_1)}{(y_2 - y_1)}] \tag{5}$$

$$\alpha = \tan^{-1}(\frac{\Delta x}{\Delta y}) \tag{6}$$

where Δ denotes change in x or y.

The horizontal distance **d** between the same points is computed using the formula,

$$d = \sqrt{(\Delta x^2 + \Delta y^2)} \tag{7}$$

(*Earl, 1986*).

2.2.4.2.2 Labelling and use of Map directions

The following are the standard labels of the reference system in surveying (*Earl, 1986*):

1. North (Zambian side) the positive Y axis direction;

2. East (downstream) the positive X axis direction;

3. West (Upstream), the negative X axis direction;
4. South (Zimbabwean side), the negative Y axis direction.

The bearings are taken clockwise with reference to the North, the positive Y direction (*Earl, 1986*).

To enable vertical deformation detection, the local coordinate system used for the Kariba Dam monitoring incorporates the Z axis which is positive in the upwards direction. This axis is used to detect sinking or rising tendencies on the dam and surrounding areas.

Deformation monitoring of dams, like any other scientific field, is dynamic due to the dynamic nature of dam deformations. The Kariba Dam deformation problems are dynamic due to some natural and human activities on and around the dam. Hence, it was vital to look at some of the technological advancements in the geodetic deformation monitoring systems currently available. This helped in designing a suitable monitoring system to detect, analyze, visualize and report on the Kariba Dam deformations in time.

2.3.0 Modern Geodetic Deformation Monitoring

Kariba Dam deformation monitoring is an exercise which involves monitoring of both landslide and dam deformations.

The landslide is due to the poor geology of the south bank which rests on top of the hydroelectric power tunnels.

Its (landslide) stability must be monitored on a daily basis to ensure safety of the dam, which may be overstressed from the south bank landslide. It is also important to ensure the dam is measured in its entirety accurately to check for any adverse effects from the landslide or any other activity.

For this reason, modern technology in dam deformation monitoring was looked at in this study, in a bid to design a suitable geodetic monitoring system for the Kariba Dam.

2.3.1 GOCA System

An Automated Deformation Monitoring System is an integrated high-tech hardware and software installation called GNSS/LPS-based, Online Control and Alarm (GOCA) System. Classical Local Positioning Sensors (LPS) include instruments such as Robotic Total Stations (RTS), Prisms and Geotechnical instruments *(Jäger and González, 2015*). A GOCA system may be installed as an early-warning system for natural hazards such as landslides, earth quakes, volcanoes, etc.

The GOCA system was developed based on a research which was conducted at the Survey and Geomatic Department and the Institute of Applied Research (IAF) by the University of Applied Sciences Karlsruhe (*Jäger and González, 2015*).

2.3.1.1 Components of the GOCA System

The main components of the automated, real-time geodetic monitoring system consist of area being monitored and computer center system (*Jäger and González, 2015*).

The area being monitored comprises:

a) Dam Deformation Monitoring System (DDMS) which includes a number of monitored object points on the dam and a network of local reference stations with respect to which displacements of the object points are to be determined (Albert Einstein's principle of relative motion). The reference stations are assumed fixed while object points are potential moving portions for which vector positions are monitored.

b) On-site permanently installed GNSS Continuous Operating Reference System (CORS) and LPS sensors which are used to monitor:

   1. the dam displacements under the influence of both expected and unexpected forces
   2. the stability of the area surrounding the reservoir (ridge lines)
   3. seismic activities and prediction of their occurrences and

4. Positions of Dam Deformation Monitoring (DDM) reference stations, especially in unstable areas, and calibrating them to ensure correct reference coordinates all the time.

The computer center system consists of a server and the communication system from the remote area (dam being monitored) to computer center (*Jäger and González, 2015*). The computer center has two software packages running, namely: the GNSS/LPS-based, Online Control and Alarm system (GOCA) sensor communication software and the GOCA deformation analysis software. The GOCA hardware-control and communication software modules collect, in different kind of communication modes, the GNSS/LPS data at a user defined sampling rate.

The GOCA deformation-analysis software is responsible for a further processing of that data in a three-step sequential adjustment procedure (initialization, Geo-referencing and reference point testing). Both least squares and robust estimation methods are applied. The first focus is set on the robust online displacement estimation, statistical testing and alarm setting. Further the algorithmic scheme of a Kalman filter is treated, which is applied in the GOCA system for the estimation of the object-point state vector of displacements, velocities and accelerations. The further concern of the deformation analysis is the integration of further parameters such as material parameters and damage models as well as the addition of geotechnical sensors such as strain-meters and lead to system based analysis approaches (*Kälber and Jäger, 2000*).

The GOCA software is responsible for sounding an alarm if a user-defined displacement threshold is exceeded during an online monitoring. The complete deformation analysis functionality is provided online, near online and in post-processing mode (*Kälber and Jäger, 2000*).

GOCA software is designed for the classical type of monitoring geodetic network, where the objects geometry is described in a system of coordinates which is not influenced by deformation process of the studied area, the coordinates being represented by the stable reference points located in the area, as well as by the object-points, that are located in areas with possible deformations of the monitored objective (*Kälber and Jäger, 2000*). To ensure a stability of the reference system for each observation epoch/cycle, the GOCA system concept includes detecting reference network points that become unstable, which is possible by using deformation-analysis application.

The GOCA array may consist of different instrument makes with a possibility of adding more to the array, as shown in the figure 2.15 below. The brand names in the diagram are just examples; thus the users may choose to acquire any brand name of their choice.

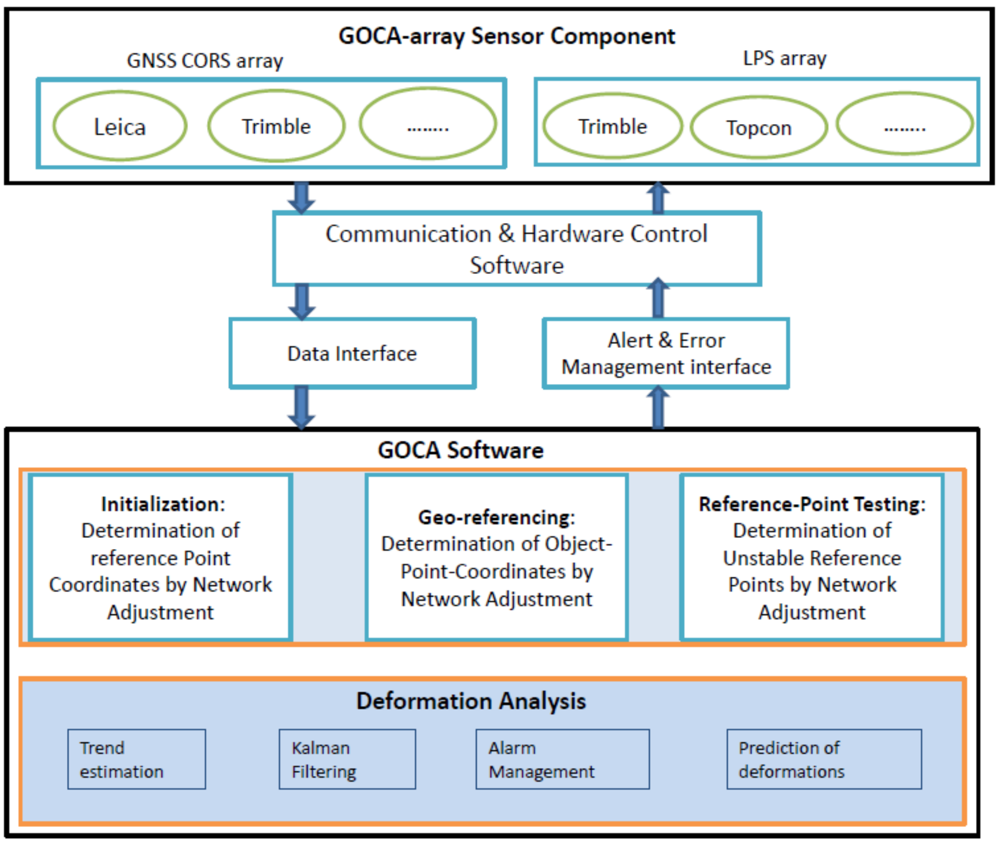

Figure 2.15: *Schematic diagram of the GOCA system*

After adjustments, the GOCA monitoring system software automatically generates a series of graphs that show the evolution of the horizontal and vertical displacements of mobile points (mobile sensors) installed on the monitored objective.

A GOCA system comprises a Continuous Operating Reference Station (CORS), which is a set of continuously operating GNSS geodetic receiver, usually installed outside the deformation zone of the object points or a dam being monitored (*Kälber and Jäger, 2000*).

2.3.2 GNSS Continuous Operating Reference Station (CORS)

High-precision geodetic GNSS receivers, strict surveying procedures and data processing methods, the continuous or campaign GNSS surveying can be used to detect ground deformation with a horizontal accuracy of 0.1cm per year and a vertical accuracy of 0.3cm per year (*Cai et al,2008*). Tasci (2010) in his research concluded that maximum horizontal dam movements caused by water load effect could occur in the middle of the dam's crest in arch dams, and was proved by applied GNSS measurements and deformations analysis methods. It may offer advantages over traditional surveying methods that measure relative geometric quantities between selected points, and also over techniques that employ geo-technical instrumentation (Ali et al., 2005). Surface displacements are an important indicator of structural stability through inclusion of surface monuments in the overall monitoring management.

As a supplement to the existing geotechnical instrumentation on the dam wall and conventional surveying methods, the use of continuous Real-Time Deformation Monitoring System which comprises Global Navigation Satellite System (GNSS) and Total Stations for the dam monitoring may offer more reliable and efficient method for a three-dimensional monitoring (*Rizos et al., 2010*).

Global Navigation Satellite Systems (GNSS) are a very popular technology for geodetic monitoring of both "fast" and "slow" structural deformation (*Choudhury and Rizos, 2011*). GNSS technology is being extensively used for monitoring the displacements of engineering structures such as high- rise buildings, dams, bridges, etc. Large structures increasingly have one or more GNSS receivers installed on them, a trend likely to continue unceasingly (*Rizos et al., 2010*).

GNSS in Continuous Operating Reference Station (CORS) not only provides navigation services based on pseudo range observations, but also offers carrier phase observations to develop real-time and near-real-time deformation monitoring techniques (*Jiang-Xiang and Hong, 2009*). The development of prediction methods, which allow a determination of deformations and stress distribution and comparison of predicted values with the observed (Kalman Filtering), constitutes to be valid tools to control safety (*Rizos et al., 2008*). Several other trends of GNSS CORS are also emerging (*Rizos et al., 2010*): integrated deformation monitoring systems, real-time kinematic (RTK) being almost exclusively the GNSS technique that is used, the use of low cost L1-only GNSS sensors on the deforming structure, system control platforms that link to and manage the data logging and control of many sensors (including GNSS), increased use of sophisticated time series analysis to characterize the movements structures, and use of installed permanent GNSS reference station infrastructure called GNSS CORS Stations (*Rizos et al., 2010*).

The dynamics of the structure typically defines the nature of the coordinate analysis (*Rizos et al., 2010*). Three dimensional monitoring using manual or Robotic Total Stations (RTS) and reflective targets is a useful and practical method to determine whether or not structures are undergoing any deformation due to external influences. Continuous geodetic deformation monitoring applications typically rely on positioning or coordinate solutions (provided by GNSS, Total Station or other manual or automatic survey technology) and a subsequent (or near-real-time) statistical analysis procedure to unambiguously identify movement of target or monitoring instrument that is interpreted as structural deformation (*Choudhury et Rizos, 2011*). Robotic or Motorized Total Stations with automatic reflector recognition and high accuracy GNSS CORS are now widely accepted as ideal sensors to be deployed in such applications (*Cranenbroeck et al., 2004*).

Equipped with the background knowledge of the latest monitoring methods, the analysis of the Kariba Dam deformations was done as described in chapter 3 below, a step just before the design of the suitable deformation monitoring system for the dam.

3.0 MATERIALS AND METHODS

The methodology employed in this research is analogous to building a house; The Author takes the position of a builder,while the blocks are represented by bits and pieces of data provided by ZRA and Kariba Dam consultants who represent block suppliers in this example. Without building a house, blocks remain useless. The house in this research is represented by the physical meaning of the deformation data analysis, using the deformation results (blocks) which ZRA,as owners, have been measuring (moulding) over several years. The users of the house (research results), in the example above, are the dam managers and other stakeholders who must use the physical meaning of dam deformations as a tool in their decision making processes, in a bid to ensure uprightness of the dam and avoid economic-activity disruptions which may otherwise result from the dam failure.

After the data analysis, a Global Navigation Satellite (GNSS)-based System was designed to address the current geodetic monitoring system challenges. The design is an automated, Robotic Total Station (RTS) integrated system operated by Web-based computer software specifically programmed for deformation monitoring.

The monitoring system design was done after substantiating that South Bank landslide is not localized, and that it has an adverse effect on the dam stability, in addition to subsiding Alkali Aggregate Reactions (AAR), seasonal temperature and hydrostatic pressure variations. Further, a prism sensor array was designed and proposed to be used for both classical and modern instruments, and recommended that the prism array be purchased and installed as soon as possible, in order to increase deformation detection even before the old monitoring system is upgraded because prism sensors are relatively cheap to buy and install.

An automated geodetic deformation monitoring system was mainly designed for improved measurements on the dam, visualization, data analysis and alert issuance to classified users. This is in a bid to increase knowledge about the mechanical behaviour of the dam for its maintenance and water usage to ensure the dam usefulness all the time.

The methodology is summarized in the flow chart in figure 3.1 below.

3.1 Flow Chart

The following flow chart is a summary of the various stages of this research.

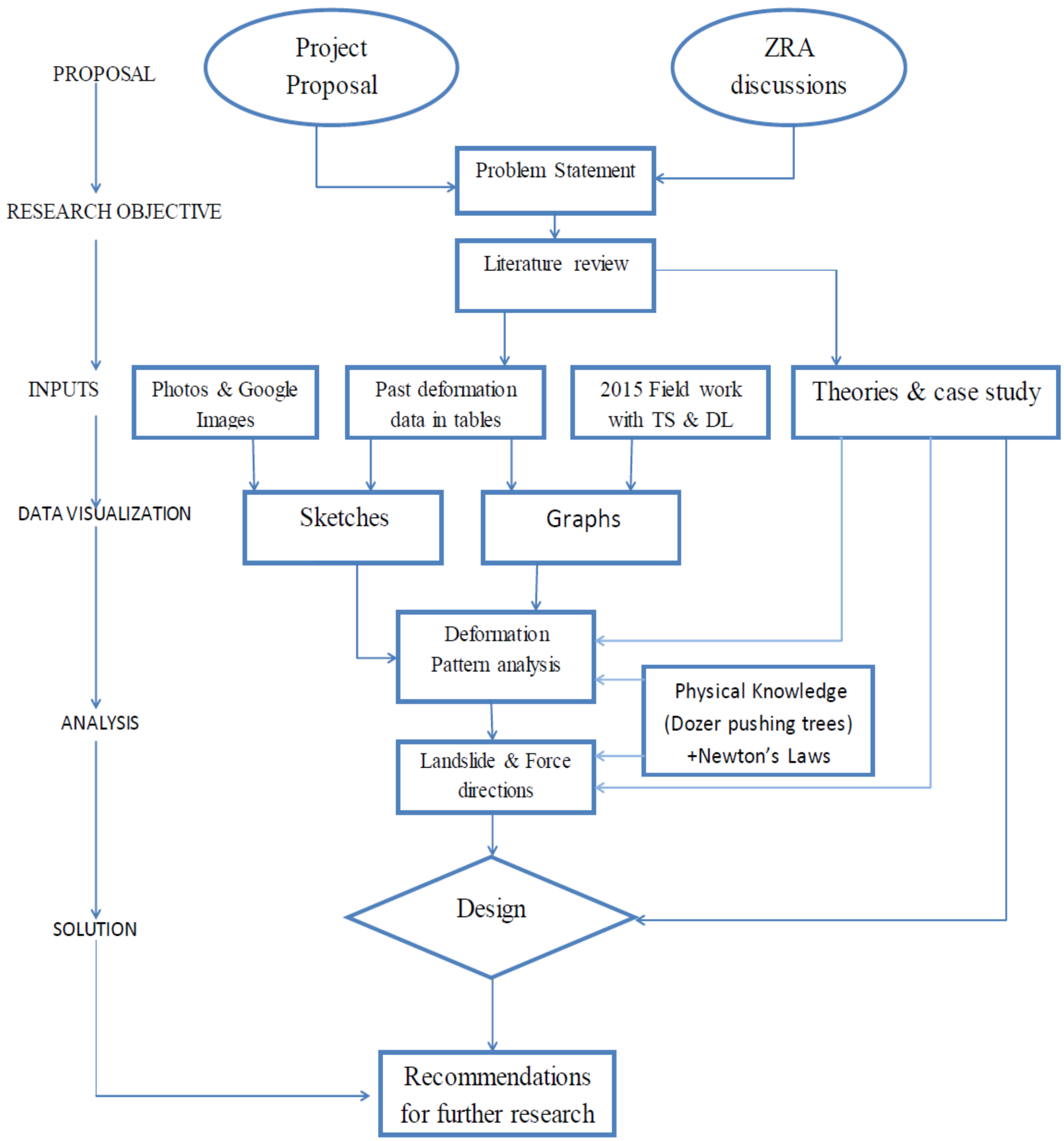

Figure 3.1*: Flow chart of the research methodology*

3.2 Field work

3.2.1 Reconnaissance Survey

Prior to the field survey, familiarization of the site was conducted by walking around the study area including inside the galleries where various geotechnical instruments are installed. Visual inspection of the cracks on the inside and outside walls was conducted so as to have an idea about the type of deformation the dam was undergoing.

Preliminary data (past dam deformation reports) was already collected during this exercise in order to understand how the deformation monitoring of the Kariba dam was done. An insight into the kind of deformations the dam faced was gained during the first field visit.

3.2.2 Instruments

During the geodetic deformation monitoring of the Kariba Dam, traverse distances, horizontal and vertical angles were all measured using the LEICA TS30 Total Station and the three altrimetric wires of known vertical height in gallery F which were used for transferring measurements to the gallery tunnels. KERN ME5000 reflectors were used for measuring triangulation distances and angles on the outside while special and customized veneer scales were used to measure horizontal angles with respect to the altrimetric wires from gallery F directions and verticals angles at RD1, RG1, RG3 and SB17 were measured using WILD targets (see points in drawing 1 in the Appendix C).

The measurements were electronically logged and later downloaded into LEICA Geo Office software.

During the field measurements, observations mean errors were preset in the LEICA TS30 Total Station to mean error of 0.0090 or 32.4" and 0.0250 or 0^{o} 1'30" degrees for horizontal and vertical angles respectively. This was done as compensation for angular errors resulting from tilting controls in the horizontal and vertical angles.

These mean errors maybe increased, depending on the amount of deformation on Trig. Controls.

The LEICA TS30 Total Station used in the field measurements has the following specifications:

a. Angular accuracy of 0.5”
b. Pinpoint Electronic Distance Measurement (EDM) Accuracy of 0.6mm+1ppm to prism and 2mm+2ppm to any surface
c. Automatic Target Recognition (ATR) accuracy of 1”.

Precision leveling on the dam crest was carried out using the LEICA DNA03 digital level by measuring to Invar staves with all the measurements being recorded electronically and later downloaded into LEICA Geo Office software. The WILD NA2 automatic level, in combination with the WILD GPM3 parallel plate micrometer, was used for galleries F, D and B measurements as the digital level was limited to crest measurements due to space limitation (in galleries) for the Invar staves. All NA2 automatic level readings were manually booked and processed soon after field work.

During triangulation, traversing and leveling, the OAKON RTD digital thermometer was used for measuring temperatures whilst atmospheric pressure at instrument station was measured using Kestrel 4000NV digital weather meter. Local pressures at target stations were measured with handheld Thommen barometers which were calibrated prior to measurements.

The LEICA DNA03 used for crest leveling has the following specifications:

Table 3.1: *Digital Level accuracies*

Electronic Measurements	**Accuracy**
With invar staffs	0.3mm
With standard staffs	1.0mm
Optical Measurements	2.0mm
Standard deviation distance(Electronic)	1cm/20m (500ppm)
Range in Electronic measurement	1.8m-110m
Range in Optical measurement	From 0.6m

3.3 Field Data Processing

Assessment of field measurements was done using the Helmert99 software (version of April 2004) observation files and its output data. The evaluation results were referenced to the August 1989 measurements for current deformation trend analysis and visualization.

In the Helmert99 Transformation, three beacons (SB1, SB2 & SB3) on the South Bank and another three (NB1, NB4 & NB3) on the North Bank were used as control points (see drawing 5 in the Appendix C). The benchmark (BMA) level used was fixed at 497.17960m, a reference datum of 1989.

The distances measured in the field were corrected for temperature and atmospheric pressure before they were used to calculate the control point coordinates which were evaluated by calculating displacement vectors or residuals of movements (See displacement tables in Appendix B). All control-point residuals computed were presented in form of tables with remarks about their respective displacements. The displacements of the Primary control points were also compared to displacements obtained from pendulum readings. Once the two sets of displacements matched within acceptable limits of about 0-10 mm, final residuals in X, Y & Z were presented in form of tables. It is these tables that were visualized in different ways to extract patterns in the control-point deformations, and hence the slope and dam deformations because these points are anchored on the dam and on the banks.

3.4 Deformation Analysis and Results

3.4.1 Past reports used

The analysis in this research made use of the following reports from the ZRA library:

1) The 5-Yearly Inspection Reports of 2006 and 2011 done by Tractebel Engineering, the Kariba Dam designers which gave insight into the dam status, behaviour and maintenance recommendations

2) The Geodetic Deformation Monitoring (GDM) reports of 2008 to 2013 which provided data for deformation analysis of the control pillars. The 2012 deformations of the control pillars where visualized to derive pattern of deformation amongst the control points. The other control pillar deformations from other years were just used to confirm the 2012 deformation trend. The 2012 deformations where particularly chosen because that was when reliable and accurate data was expected following the introduction of the new Leica Total Station for the deformation monitoring measurements (see LEICA TS30 and LEICA DNA03 specifications in table 3.1 above)

3) The instrumentation reports which described the instruments and methods used to monitor the dam deformations

4) Status of the Kariba Dam reports done by ZRA and other consultants

5) Maintenance work reports which described the plunge pool status and suggestions to curb the scouring

6) Reports on control of blasting and vibrations from the earth works

7) Dam Crest Levelling graphs which were given in soft copy during the early stages of the study. These graphs were used to analyze the vertical swelling behaviour of the dam from the 1989 to 2015

8) Coordinate lists of control points measured in 2008, 2009,2010,2012,2013 and 2015 were used for control-displacement computations using 1989 as reference coordinates (see Appendix B).

3.4.2 Landslide and AAR effects on the dam

In order to design a system which could effectively characterize the landslide on the southern bank, it was vital to substantiate landslide effects on the dam. The landslide of the South Bank, if not well detected and controlled, has a compressional effect on the dam through an axial loading on the South Bank abutment. This effectively means the dam would heave upwards and upstream under its influence; an effect similar to the Alkali Aggregate Reaction (AAR) on the dam, as illustrated in figure 3.2 below. This is a problem because it means that upward and upstream dam displacements resulting from the landslide would mistakenly be attributed to the chemical reaction, an effect of which was established as early as 1975. Thus the intermixed effects of landslide and AAR had to be differentiated in this research by isolating them in an investigative-like analysis.

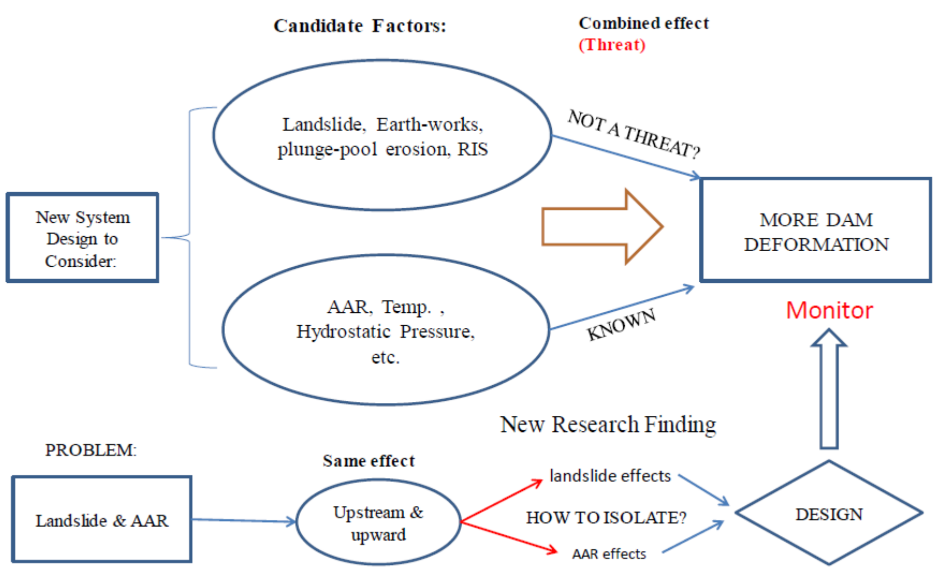

Figure 3.2: separation process of AAR and landslide effects

3.4.2.1 Investigative AAR-landslide effects analysis

Following the interweaved effects of AAR and landslide on the dam, as illustrated in figure 3.2 above, the first focus in the analysis was to separate the two effects. The separation called for geodynamic analysis outside the dam structure, where AAR was not occuring. Certainly, if the ground in question was unstable, some features anchored on it would not remain in one position through out. The features in this case which reflected the effects of the landslide are the control pillars,about 1.5m tall, on the dam hillsides.

The control-pillar deformation analysis proved that landslide at Kariba Dam exists and that its effects extends to many places, including on the dam and the North Bank.

After this process, it was just a question of looking back into their (AAR and landslide) individual histories; AAR effect was traced back to when it was first reported and how the dam responded to it over the years, and the result was the 'signature' of the AAR in the way it responded. Any other deformation pattern not corresponding with the AAR 'signature' on the dam was attributed to the landslide. The directions of dam deformations only typical of landslide compressional forces were also drawn from the past reports. At this point, the dam revealed the landslide 'signature'.

Having done this, it was now a question of relating the existing theories and the dam behaviour under landslide forces and AAR, in isolation. Now the culprit to new dam deformations was found, as proved in the following analysis.

3.4.3 Landslide deformation analysis on control pillars

The 2012 status vectors used in the analysis of these monuments were confirmed using the control displacements of 2013 and 2015. Their (control points) stability descriptions were meant to assess their usability in the subsquent geodetic measurements of the Kariba Dam. But in this research, their status vectors, described by the ZRA monitoring engineers after measurements, formed a basis for the geodynamic analyis which linked the landslide to the deformation patterns around the Kariba Dam.

Hint: *Imagine standing at some vantage point, overlooking a canopy of a thick forest, and watching trees falling in certain directions. While looking at the falling trees in the forest, one realizes that the trees are being knocked down by something, otherwise they would not be falling down-Isaac Newton's first law of motion. Then one gets a pencil and starts sketching the falling directions of the trees and deriving a force direction of the hidden Machine which is knocking down the trees. Suddenly the sketch reviews one general direction of the force being applied on the falling trees. Judging from the knock-off effects on the trees, ones realizes that there is a Bull Dozer which is pushing down the trees in the forest in that direction.*

Analogous to the falling trees above, control pillars on the slopes would only deform under a force being applied on them from something. The deformation amounts of the control monuments are measured every year and presented in a table form by ZRA (see table 3.2 below). The direction of deformation of an individual pillar is represented by its residuals.

In the example of falling trees above, a mere description of the falling directions without linking them to what is causing them to fall would be an incomplete work; descriptions like, "that tree is falling west, while being lifted up, the second one seems to be falling East;and the third one seems to tilt towards the North". Then the observer concludes that since the trees are not falling in the same direction and thus the tree-falls are due to the root-anchorage problems of the individual trees. Such a non-holistic analysis on the falling trees would not yield any meaningful pattern, and thus may not establish the real cause of the tree-falls.

Thus, the Author in this research looked at the tilting pillars from a different viewpoint in order to find the root cause of the control-pillar deformations. The first approach was to find a parttern in the deforming pillars.

To do this, the data in table 3.2 below (from ZRA 2012 report) merely describing the amount of tilts of the individual pillars were analyzed further as shown in the following paragraphs.

Table 3.2: Control Pillar Residuals(mm) of 2012; Courtesy of ZRA

Station	Mean Error Ellipses			Residuals of the Helmert Transformation			Remarks
	Δx	δy	δh	δx	δy	δh	
SB1	0.34	0.28	1.20	0.20	0.82	-4.97	Deformation is normal for this station.
SB2	0.41	0.35	1.27	-0.41	-6.40	-13.19	Deformation is normal in the x-axis and significant in the y-axis. This station is sinking and moving away from the river.
SB3	0.39	0.34	1.08	1.20	7.59	-4.48	Deformation is normal in the x-axis and in the height but significant in the y-axis.
NB1	0.39	0.32	1.33	0.76	1.75	-6.44	Deformation is normal in the x-axis and significant in the y-axis. The station is also sinking.
NB3	0.39	0.23	1.54	-1.44	-1.07	-2.25	This station is stable.
NB4	0.35	0.30	0.86	-0.31	-2.68	-16.21	Deformation is significant in the height. This station is sinking
SB20	0.34	0.27	0.84	3.15	3.84	11.62	Deformation is significant in the entire three axis.
CD2	0.34	0.24	0.74	-1.69	-0.11	23.86	Deformation is normal in the y-axis and significant in the x-axis and in the height.
P434	0.45	0.36	0.99	-9.31	-7.64	26.35	Deformation is significant in the entire three axis.
NB20	0.36	0.31	0.58	-0.05	4.81	8.83	Deformation is normal in the x-axis but significant in the y-axis and in the height.
RG1	2.46	1.09	1.77	-1.75	-9.99	-0.60	Deformation is normal in the x-axis and height but significant in the y-axis as station is sliding towards the river.
RG3	1.44	0.86	1.30	-3.10	-6.71	-4.18	Deformation is normal in the x-axis and height but significant in the y-axis.
RD1	1.09	0.46	1.21	-0.90	6.17	-3.80	Deformation is normal in the x-axis and height but significant in the y-axis.
SB17	1.47	0.71	1.33	-1.51	26.49	-11.30	This station is sinking and also sliding towards the river. Deformation is significant in the y-axis and height.

This trend in the deformation pattern of the trig pillars was recorded in the 2008, 2009, 2013 and 2015 measurements and results showed them as being progressive. See more in Appendix A.

3.4.3.1 Landslide effect on control pillars

The vector displacements of the control points summarized in table 3.2 above were visualized by plotting and sketching them in order to find out whether or not their respective tilts or deformations had a link to the south bank landslide.

In figure 3.4 below, the direction of pillar movement is denoted by t_1 and t_2, where t_1 is time before and t_2 is time after displacement.

In order to understand the sketch, remember the local coordinate system used for Kariba Dam monitoring which is shown in figure 3.3 below.

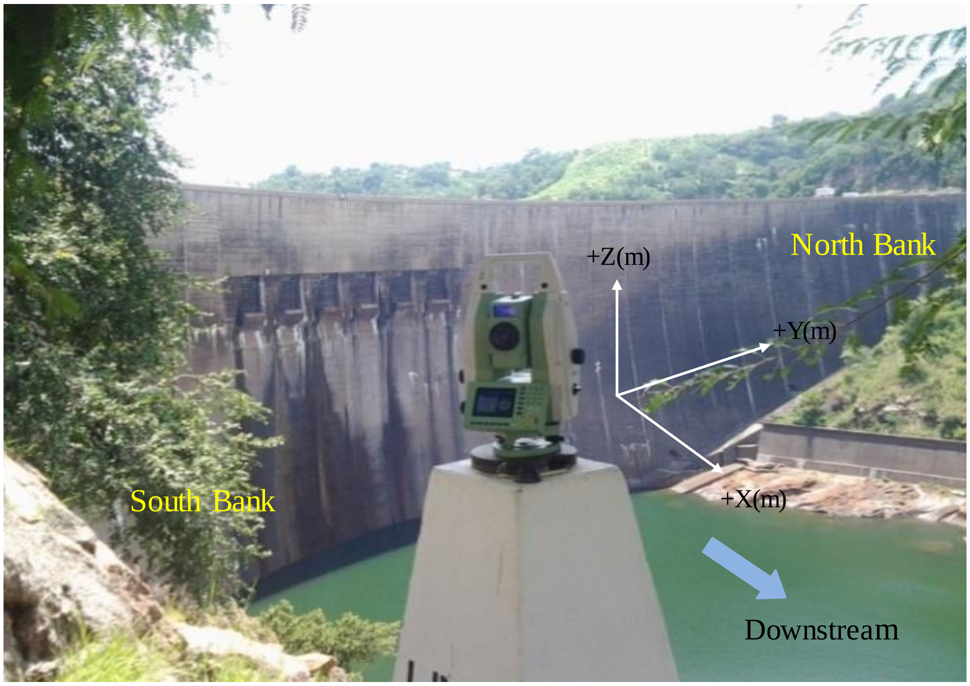

Figure 3.3: *Kariba Dam Local Coordinate System orientation: Courtesy of ZRA*

In connection with table 3.2 above and the local coordinate system in figure 3.3: positive in the Y-Axis means tilt towards the North Bank (NB) or Zambian side, while negative is towards the South Bank (SB) or Zimbabwean direction; positive towards the X-Axis means tilt in the downstream direction, while negative means upstream tilt. And positive in the Z-Axis means upward displacement while negative means sinking of the monuments.

To understand the effect of the landslide, remember newton's first law of motion, "An object remains in its state of rest or motion until a force is applied on it, and an object in motion continues to move in a straight line unless another force is applied to change its direction".

The residuals of the helmet transformation in Table 3.2 were sketched as shown in figure 3.4 below to clearly show the tilting relationship or pattern amongst the monuments. The causative force was derived and drawn in relation to the resulting tilting direction as shown in figure 3.4 below.

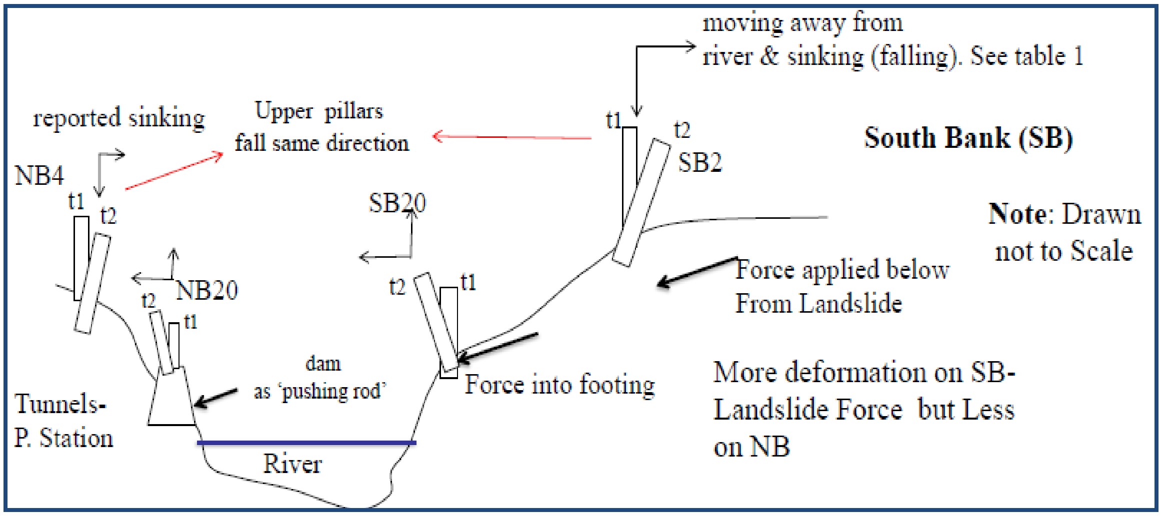

Figure 3.4: *Sketch of table 1 control point deformations-cross section from SB2 to NB4*

When the deformations of some of the north bank pillars where compared to the deformations of the corresponding pillars on the South Bank, the following were the derived patterns:

- NB4 and SB2 tilt south wards with sinking a tendency, but SB2 showing more deformation. This is due to the undercutting force being applied by a landslide way below the hill tops

- NB20 and SB20 (located on the dam crest) tilt north wards due to the pushing tendency of the force at this point. They are also heaving up along with the dam crest but SB20 heaving more-a direct result of the compressional force on the south abutment which is causing the dam crest to buckle upwards

- NB1 and SB1 (see table 3.4 above) both tilt in the related directions (northwards and sinking). Their positive x, y residuals and negative z values show that a force is pushing them to the north while pressing them downwards, hence sinking. This indicated the direction of the force from the south bank as shown in figure 3.4 above.

Other landslide-related deformations reviewed are as follows:

- CD2 and P434, located at the central dam crest drift upstream, in the direction of the horizontal upstream buckling of the dam crest under compressional forces of the south bank. They are also drifting upwards along with the crest upward-buckling, also a direct compressional effect of the south bank force. K204 and K202 towards the arch ends have buckled more than CD2 and P434 due to lack of constraints of the spillways towards the arch ends (see a drawing 1and 2 in Appendix C)

- SB17 drifts northwards, a direct response to the south bank force. Its sinking behaviour may be due to a downward -pressing force owing to its location on the slope (relatively near the water edge)

- RG1 and RG3, located on the north bank, just to the east of north arch end, are being curved inwards towards the spillways due to the north–arch-end rotation tendency in its response to the south bank push (see drawing 1and 4 in Appendix C)

- RD1 (located on the south bank) is swinging downstream, in the direction of the folding south arch end which is closing in towards the plunge pool, a direct response of the arch to the compression of the landslide(see figure 3.7, and drawing 1 in Appendix C)

The control pillar deformation directions were found to be determined by dam arch movements in its response to the compressional force being applied on it or directly by a landslide force, literally for all the points outside the dam area (see drawing 1 and 4 in Appendix C). The slope also played a role by affecting point of application of the landslide force on the control pillars as explained in the tree-dozer concept below.

However, all the resulting deformations either on the dam or on the hillside obeyed the south bank landslide. Hence, it was assessed that the drifts were not random but systematic as all drifts ‘danced’ to the tune of the south bank landslide (see drawing 1, 2 and 4 in the Appendix C).

The above sketch and the force directions were further evaluated by calculating displacements from coordinates measured in 2008, 2009, 2010, 2013 and 2015 (see coordinates and displacements in the Appendix B). The 1989 coordinates were used as reference against which displacements were measured. Then the displacements for 2015 were plotted as shown on a Vector Map (drawing 1) in the Appendix C. The Vector Map reviewed a similar pattern which reviewed the force direction described above.

3.4.4 Tree-dozer concept (force & its knock-off effect)

The control-pillar deformation pattern revealed the fact that there is a force being exerted from the South Bank against the dam, and transmitted to the North Bank. The landslide force and its knock-off effect on the controls are similar to one being applied by a Bull Dozer when knocking down trees (see following illustrations in figure 3.5 below).

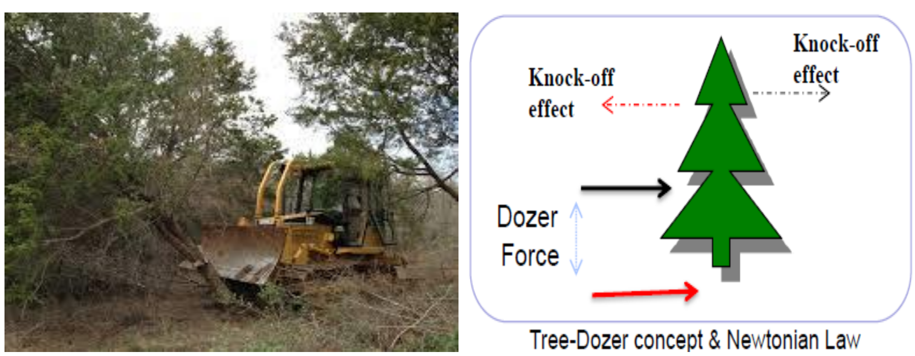

***Figure 3.5**: Hint; Bull Dozer knocking down trees (left) and its knock-off effect (right)*

In the case of landslide, the slope changes on the banks are analogous to lifting up and down of the Bull Dozer blade, thereby causing the monuments to fall either in the direction of the force or in the opposite direction (relate figure 3.4 & 3.5 above). The force of the landslide on the higher-altitude points applies way below the controls (similar to Dozer blade shoveling below tree roots), thus the knock-effect is in the opposite direction (see figure 3.4 also). As one goes towards the river, the force is guided in the control pillar footings by the slope change which is similar to lifting up of the Dozer blade to the tree stem as shown in figure 3.5 above, thus pushing a control pillar in the direction of the force-towards the river (compare with figure 3.4).

4.4.5 Landslide Direction

After visualizing all the deforming control points in the network by sketching and plotting their displacements, the following layout (see figure 3.6 below) showing the landslide directions was derived using the tree-dozer concept. The analysis was done in retrograde, whereby given the direction of deformation of a control pillar, a corresponding direction of the force responsible for that displacement was derived. All residuals of the deforming pillars resulted in one general direction of the cause of deformation-landslide (See drawing 1, Appendix C).

Note: not all deformation directions of the control pillars are the same as the direction of the landslide, because the resulting residuals depend on the geographical location of the control pillar (with respect to the slope) which determines a point of force application. Thus a landslide Force has different knock-off effects on different control pillars. However, some control pillars on corresponding slopes on either side of the hillsides of north and south banks have the same knock-off directions-*a pattern that no one else had ever discovered before this research*. This discovery in the deformation trend of control pillars (in relation to the landslide) substantiated the devastating effects of the landslide on the dam.

The derived directions showed that the general movement of the landslide is northwards. This pointed to the fact that the dam may be *suffering compressional load, applied axially from the south arch end,* which resulted in the new dam deformations observed by Coyne and Bellier. The resulting deformation tendencies included the tangential and rotational deflections detected in 2010 measurements (see drawing 2 &3 in Appendix C).

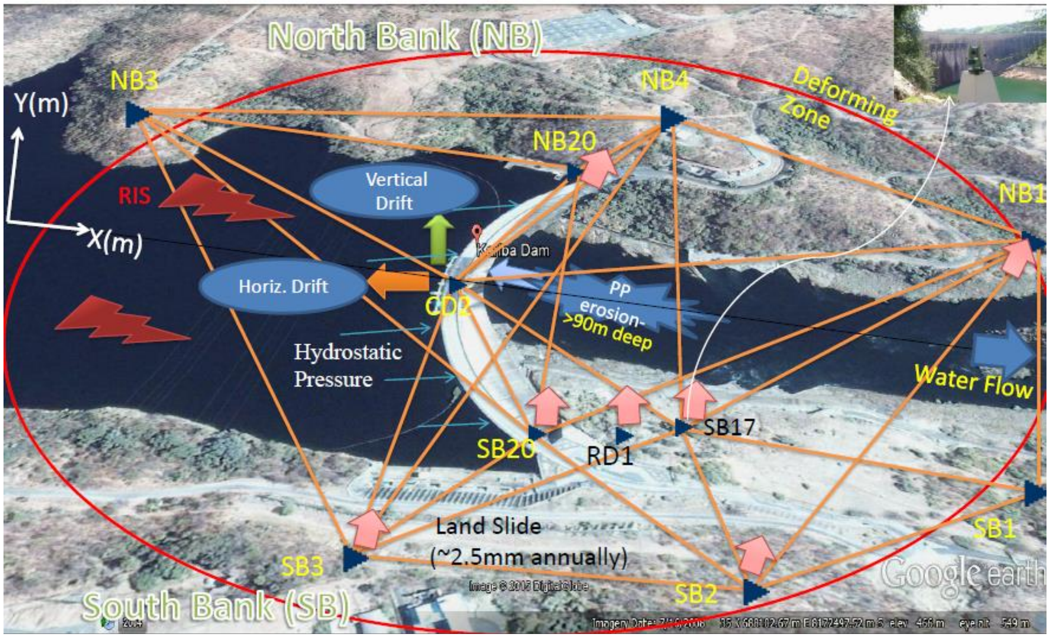

Figure 3.6: *Sketch of Control points linked to landslide force directions (in pink arrows)*

3.4.6 System of Forces on the Kariba Dam

The landslide might have triggered the non-equilibrium status of the dam. The system of major forces responsible for the reported new dam behaviour may be resolved as in the following sketch (figure 3.7); not drawn to scale as it is for illustration purposes only.

Note: *The thrust here is focused on explaining the source of the northward translational and clockwise rotational movements of the dam arch. It is already known that hydrostatic and hydrodynamic forces were taken into account at design stage; hence they are not a threat in this case to* the *dam stability. As such, it can be assumed, from the geometric distortions of the dam, that the strange dam behavior cannot be attributed to the static water pressure. Thus stressing on the hydrostatic and hydrodynamic was not necessary; just showing them on the free body diagram sufficed.*

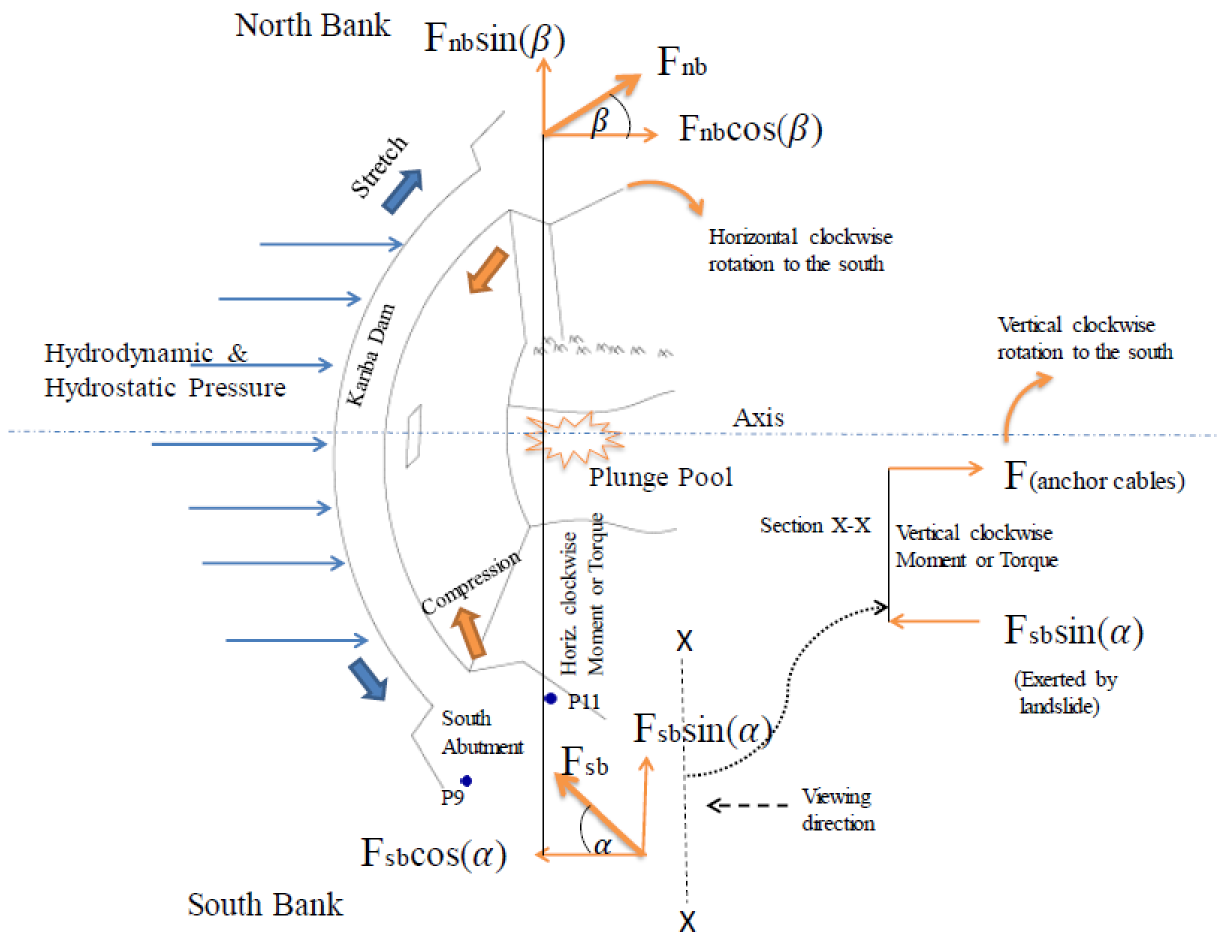

Figure 3.7: *Illustrative Free-body Diagram of the Kariba Dam arch (not to scale)*

The south bank landslide Force (**F_{sb}**) can be resolved into two components; **F_{sb}sin(α)** and **F_{sb}cos(α).** The **F_{sb}sin(α)** component is responsible for the vertical clockwise rotation because it creates a torque with the opposite forces exerted by the anchor cables meant to minimize effects of landslide on the dam (see figure 3.7 above). This phenomenon was evidenced in the past results of pendulum (P11) and (P9) situated in the south abutment. Force component **F_{sb}cos(α)** has the same effect as force component **F_{sb}sin(α)** on the pendulums.

Due to the fact that the south bank is compressing the arch, a phenomenon analogous to pushing the northern side using the dam as quasi-rigid lever, the northern side experiences a force ($\mathbf{F_{nb}}$). This force can also be resolved into components as shown in the sketch above.

Note that force component $\mathbf{F_{sb}cos(\alpha)}$ of the south bank pushes the dam upstream while the component $\mathbf{F_{nb}cos(\beta)}$ of the north bank pushes the dam downstream. This phenomenon is a typical torque which tends to rotate the dam clockwise, with the north arch end moving downstream and towards the river; pushing RG1 and RG3 inwards (see drawings 1, 2 & 4 in the Appendix C). This was assessed by the Tractebel Engineering consultants, Coyne and Bellier in 2011 also; although they could not understand why. The other force component, $\mathbf{F_{nb}sin(\beta)}$, and directed into the NB4 hill shown in figure 3.6 above, may be responsible for the sinking tendencies of the hill reported in a number of ZRA geodetic measurements reports. The sinking may be due to the collapsing tendency promoted by the tunnel voids under the hill.

The rotational movements of the arch in the horizontal may be an indication that the dam is slowly losing its alignment with respect to its upstream-downstream Axis (see drawing 2 in Appendix C).

The resultant Force ($\mathbf{F_{sb}}$) of the south bank may be responsible for the tangential movements of the south and the north arch ends. This tangential movement to the north was reported by Coyne and Bellier in 2011 5-Yearly Inspection Report, although they attributed the movement to the unusual spillage in 2010. But they recommended further probe into the causative factor as this problem did not start in 2010 but might have just been worsened by water spillage from the gates.

The extension of the south bank is still on-going; hence more landslide forces against the south abutment are expected. Even after the earth works are completed, there is still uncertainty about the weakness of the south bank which is now more prone to collapsing, especially that it now houses more tunnel voids than before (*Human causative Factor to dam failure-over ambitious and risky project expansions without thorough scientific researches*).

3.4.7 Velocity and Acceleration of the north arch-end movements

The velocities and accelerations were computed from the deflection magnitudes assessed by Coyne and Bellier in order to compute the net Force responsible for the new dam deflections (see drawing 2 in the Appendix C).

The following Table shows the magnitudes of rotational movements assessed by Coyne and Bellier in 2011. From the downstream movements of the north-arch ends, velocities **v** and accelerations **a** were computed using the formulae (equations 1 and 2):

$$v = \frac{d}{\Delta t} \tag{1}$$

$$a = \frac{vf - vi}{\Delta t} \tag{2}$$

Table 3.3*: Deformation Magnitudes and the derived Velocities & Accelerations*

Time-Year (yr) Δt	1989-1999 10 year period	1999-2009 10 year period	2009-2010 1 year period
Rotational Movements **d** (mm)	1	1.7	1.4
Velocities (mm/yr) **v**	0.1	0.17	1.4
Accelerations **a** (mm/yr^2)	0.00	0.007	1.23

From the above, the velocity increased about **8 times** the normal velocity (0.17 to 1.4mm/year) between 2009 and 2010; and **14 times** compared to 1989-1999 velocity. This increase in velocity was what Coyne et Bellier referred to as being **abnormal** as it meant that there was a net force in the system of Kariba Dam equilibrium. The net force between 1989-1999 and 1999-2009 was **Zero** because acceleration was **Zero**, from the formula **F=m*a**. Hence the dam may be said to have been stable from 1989 to 2009 because the net force was zero during this period; but from 2009 to 2010, the dam experienced a net force, hence unstable.

3.4.8 Force exerted on the north bank

Isaac Newton's second law of motion can be applied to calculate the Force **F** exerted against the north bank. Using the formula **F=m*a,** where **m** is the constant mass of the dam and **a** is acceleration between 2009 and 2010 period, the Force **F= (m*1.23)**N**.** If the resilience capacity of the north bank against this force (Isaac Newton's third law of motion) is not checked, failure of the hill may take place. This force is responsible for the dam arch geometric distortions shown in drawing 2 in Appendix C.

For this reason, a geological study must be conducted to ascertain the magnitude of this force in order that its effects and extents may be known fully.

3.4.9 Vertical Pillar displacements

The effects of the landslide on the surrounding Kariba Dam area were also assessed by visualizing the amount of sinking for each monument pillar used as control for deformation monitoring. The bar chart below depicts the upward/downward displacements of the control pillars.

To analyze the sinking behaviour of the control pillars, the vertical displacements of the control pillar residuals in table 3.2 were plotted as shown in the bar chart (figure 3.8) below.

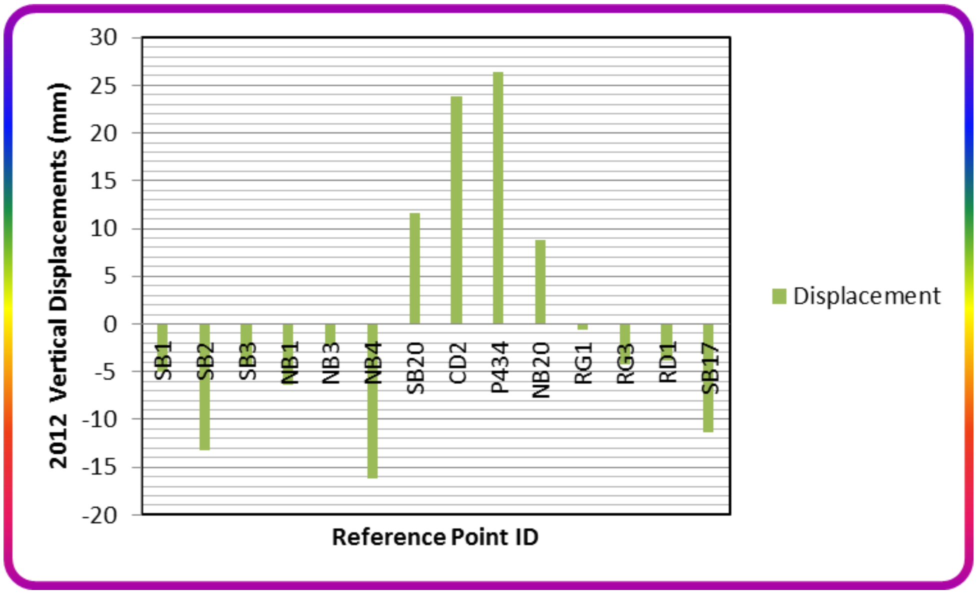

Figure 3.8*: 2012 Vertical displacements of monuments*

Vertical displacements, as shown in figure 3.8 above, show that control points situation above the hills which house the tunnels are sinking due to the hydropower caverns below; with those (SB2, SB17 and NB4) directly above the hydropower tunnels sinking more than those further away from the tunnels. The North Bank which houses the North Bank hydropower station is being affected (as shown by the vertical negative displacement of NB4) by the South Bank through the quasi-rigid dam structure which is being used as 'pushing rod' against the hill, effectively transmitting the force of the landslide across to the Zambian side.

As explained already, the control pillars with positive vertical displacements are located on the dam concrete which displaces upwards due to its structural property. The maximum upward displacement is experienced at P434 and CD2 which are located at the central block above the spillway gates. P434 is on the upper stream side while CD2 on the downstream side of the double-arch concrete dam. Comparing the two, P434 has a bigger permanent deformation which means under the compressional force of the landslide, the dam deforms worst on the upper stream face which is submerged in water. This compressional force may crack the dam, resulting in water leakages into the dam galleries.

The 2012 trend was confirmed in the 2015 measurements except the 2015 displacements are progressively bigger than those of 2012, see 2015 bar chart in 3.9 below from a 2015 displacement Table in the Appendix A.

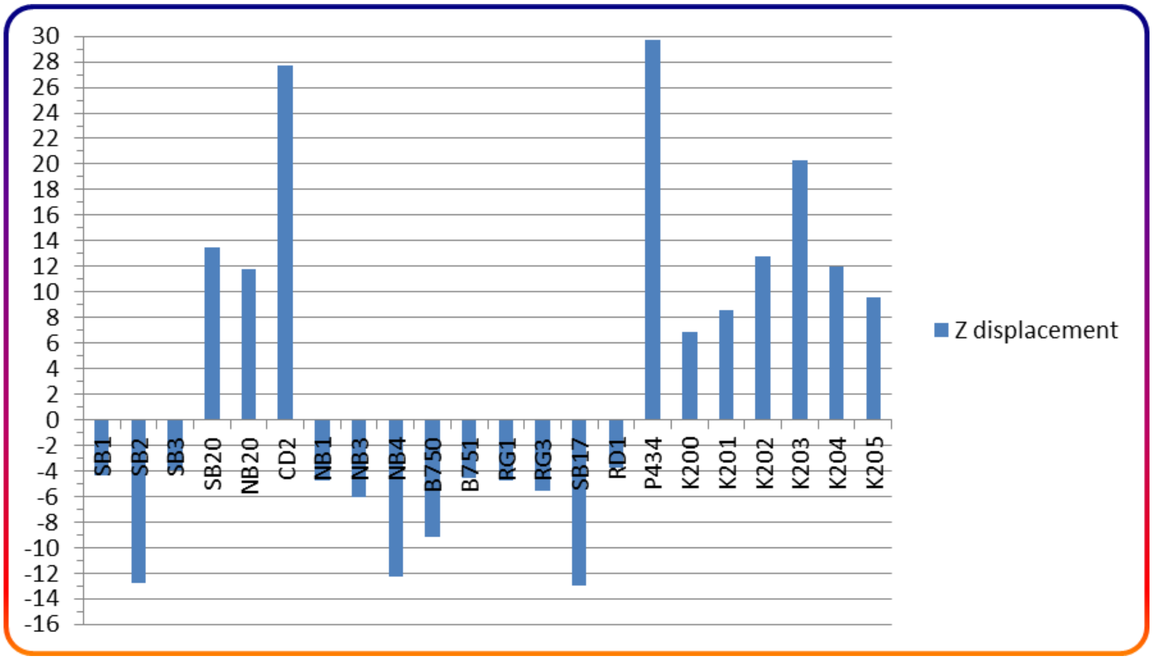

Figure 3.9*: 2015 Vertical displacements of monuments*

3.4.10 Lateral control pillar displacements

The lateral displacements **d** were calculated from Table 3.2 using the formula (eq.7);

$$d = \sqrt{(\Delta x^2 + \Delta y^2)}$$

and values of '**d**' for each control point were entered in the table below before plotting them into the bar chart as shown in figure 3.10. The partials ΔX and ΔY shown represent drifts of the selected control points in X and Y-Axis, respectively. The displacements were measured with respect to the 1989 reference vector positions. See also displacements for 2008, 2009, 2010, 2013 and 2015 in the Appendix B.

__Table 3.4__: Derived lateral displacement __d__ from the 2012 residuals in Table 3.2

ID	ΔX (mm)	ΔY (mm)	d(mm)
SB1	0.2	0.82	0.84
SB2	-0.41	-6.4	6.41
SB3	1.2	7.59	7.68
NB1	0.76	1.75	1.91
NB3	-1.44	-1.07	1.79
NB4	-0.31	-2.68	2.70
SB20	3.15	3.84	4.97
CD2	-1.69	-0.11	1.69
P434	-9.31	-7.64	12.04
NB20	-0.05	4.81	4.81
RG1	-1.75	-9.99	10.14
RG3	-3.1	-6.71	7.39
RD1	-0.9	6.17	6.24
SB17	-1.5	26.49	26.53

The control-pillar lateral displacements of 2015 were also computed and displayed in the lateral displacement table in the Appendix B. The following observations after comparing the 2012 and 2015 lateral displacements were made:

- SB2 in 2012 had a lateral displacement of 6.4mm, yet in 2015 it indicated it had only moved 3.8mm since 1989. This was a contradiction to the normal and established trend of this point which is reported to be problematic in a number of reports
- The same problem was observed in few other control pillars, as indicated on a Vector Map in the appendices.

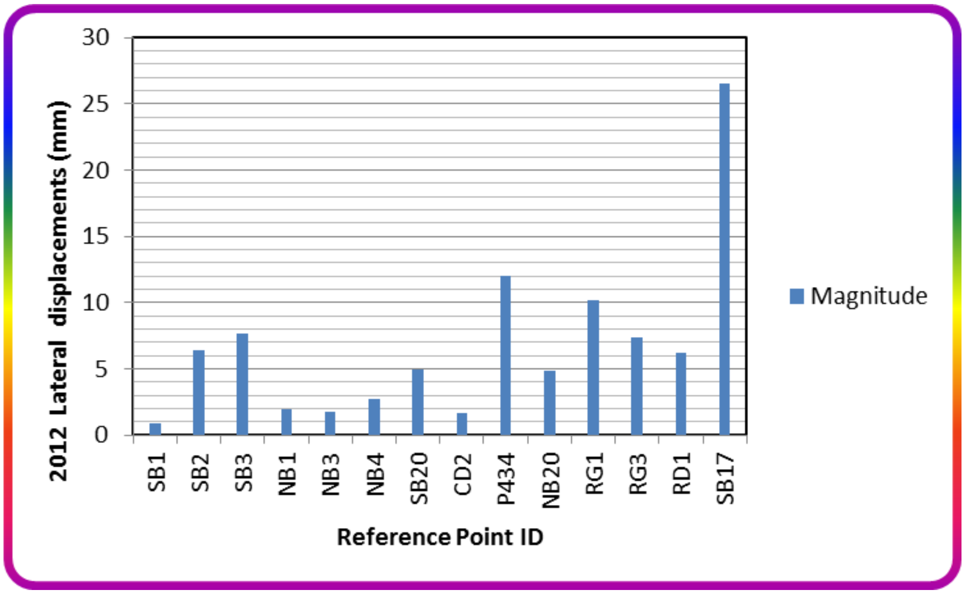

Figure 3.10: *Lateral-displacement bar chart of control points (see direction of drifts in drawing 1 in the appendices)*

From figure 3.10 above, the South Bank (SB) control points have bigger displacement magnitudes compared to those in the North Bank (NB). This is attributed to the poor south bank geology which causes a landslide.

However, after comparing the rates of displacements since 2008 to 2015(see Appendix B), the following were the observations:

- The sinking rates of the control pillars on south bank were generally higher between 2009 and 2010, after which reductions were recorded. For example SB17 which had ~2.6mm/year between 2009 and 2010 now recorded about zero sinking rate from 2013 to 2015

- P434, located on the dam crest indicated that the swelling rate between 2008 and 2010 was about 2.2mm /year, and a reduction was seen between 2010 to 2015 with an annual rate of about 1.3mm

- Contradicting sinking behaviour to known trends; for example, NB4 which is known to be sinking, and by 2012, it had sank by -16.21mm (see table 3.2 above), and yet in 2013 and 2015, it was observed at -15.4mm and -12.3mm, respectively (see yearly displacement tables in the Appendix B).

3.4.11 Isolating AAR from landslide effects

3.4.11.1 AAR effects on the dam

The leveling results show an increase in the dam swelling rates. It should be mentioned here that the leveling measurements are referenced to the Benchmarks and not the tilting pillars. The current dam deformations (permanent upward and upstream drifts) can only be attributed to the landslide of the south bank. This is because the chemical reaction was proved to be subsiding starting from 2001 onwards; and by 2012, the reaction showed significant reduction as confirmed in the 2012 crest levels measured on each block numbered from 0 to 35 of the North bank, and 0 to 40 of the South bank. The same reduction in AAR effects was reported by Coyne and Bellier in the 2011 5-Yearly Inspection Report.

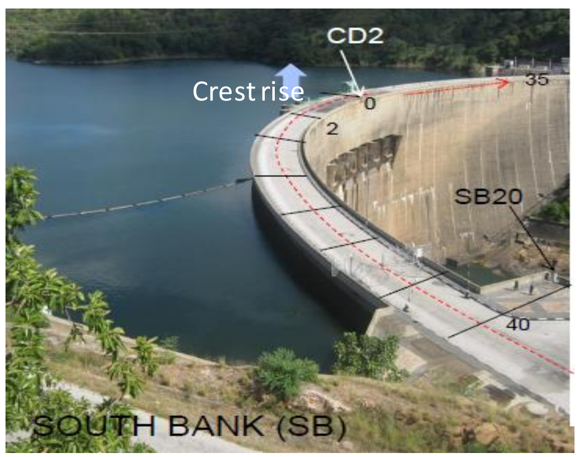

Figure 3.11*: Photo showing dam crest sections with CD2 at 0 as center*

The annual levels picked along the broken red line in the above from south to north bank were plotted as shown in figure 3.12 below.

From displacements **d (**over 10 years' period) between the graphs in figure3.12 below, between 1972 and 1982, the rate of decrease was 20mm/10=2mm/year; while between 2002 and 2012, the rate of decrease was 8mm/10=0.8mm/year.

Hence dis-acceleration effect of AAR can be computed as (0.8-2)/10 which is equal to -0.12mm/yr^2. Using this dis-acceleration, the final velocity v_f for 2015 since 2012 (3 years' time) can be computed as (using acceleration formula) -0.12*3=v_f-0.8; giving v_f=0.44mm/year. At this rate of decrease of the AAR effect, one expects its effect to end in the next 4 years (in 2019), calculated from -0.12*yrs+0.44=0; re-arranging the equation, we get ~ 4years.

The reduction in the AAR effect (AAR signature) on the dam drifts means that the chemical reaction is no longer a dam-stability threat; and thus cannot be attributed to the change in the dam behaviour (rotations and translations) assessed by Coyne and Bellier.

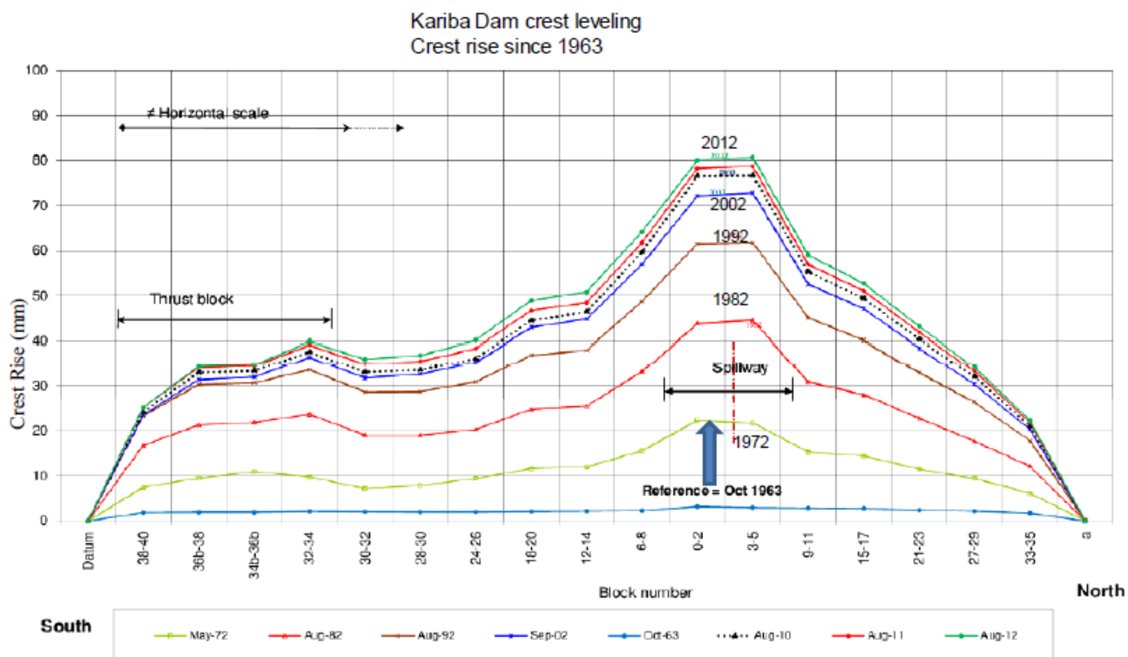

Figure 3.12: *Graphs showing AAR effect reduction; courtesy of ZRA*

3.4.11.2 Increased dam crest swelling

However, the levelling measurements from 2013 to 2015 detected an increase in the permanent crest rise of about 5mm, see figure 3.13 and 3.14 below, both taken along the same broken red line of figure 3.11. Each crest block numbered in figure 3.11 is represented by its respective annual rise graph. For example, block 0 to 2 in figure 3.11

was plotted and represented by the graph labeled block 0-2 in figure 3.14. The oscillations of about 2-3 mm amplitude superimposed on each graph represent the cyclic dam drifts resulting from seasonal temperature and water level variations between two survey sessions. The portions on the graphs where the cyclic dam drifts were not plotted indicate data gaps in the corresponding years due to reported lack of consistent data capture then.

The graphs representing both south and north bank blocks show that in 2007 to 2012, AAR effect dampened significantly ; thus the dam drifted back-forth at the same vertical level of about 80mm, above reference level (0mm) of 1963 . This trend only lasted up to the year 2012, as evidenced in the zero rise of all the graphs between the years 2007 to 2012 (see figures 3.13 and 3.14 below). After 2012, the graphs representing annual block rise started rising again, beyond 80mm; and the 2015's crest measurement results confirmed this trend. This trend is opposite of AAR effect on the dam

This dam behaviour coincided with hydropower-extension-work activities on the south bank which began after the year 2012.

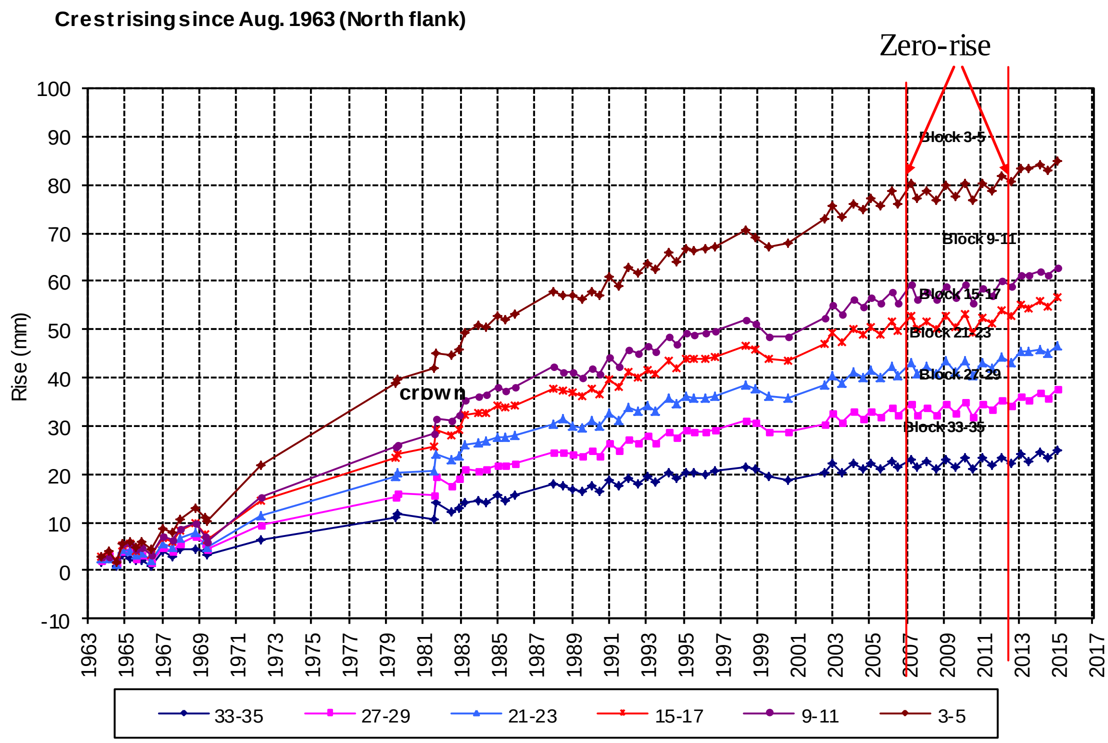

Figure 3.13*: Graphs of annual rise rate of individual crest block; Courtesy of ZRA*

The two graphs show that crest heaving is more towards the central blocks, such as block 3-5 and block 0-2, than towards the end blocks such as block 33-35 of the north bank and block 38-40 of the south bank.

The total crest rise from 1989 to 2015 stands at about 27mm; calculated from either of the two graphs as follows:

- The rise in 1989 was at about 58mm above zero of 1963, and
- In 2015, its stands about 85mm above zero level of 1963.

Subtracting the two vertical levels give about 27mm.

The two graphs in figures 3.13 and 3.14 show that the deformation tendency of the crest is not a linear problem but an exponential one.

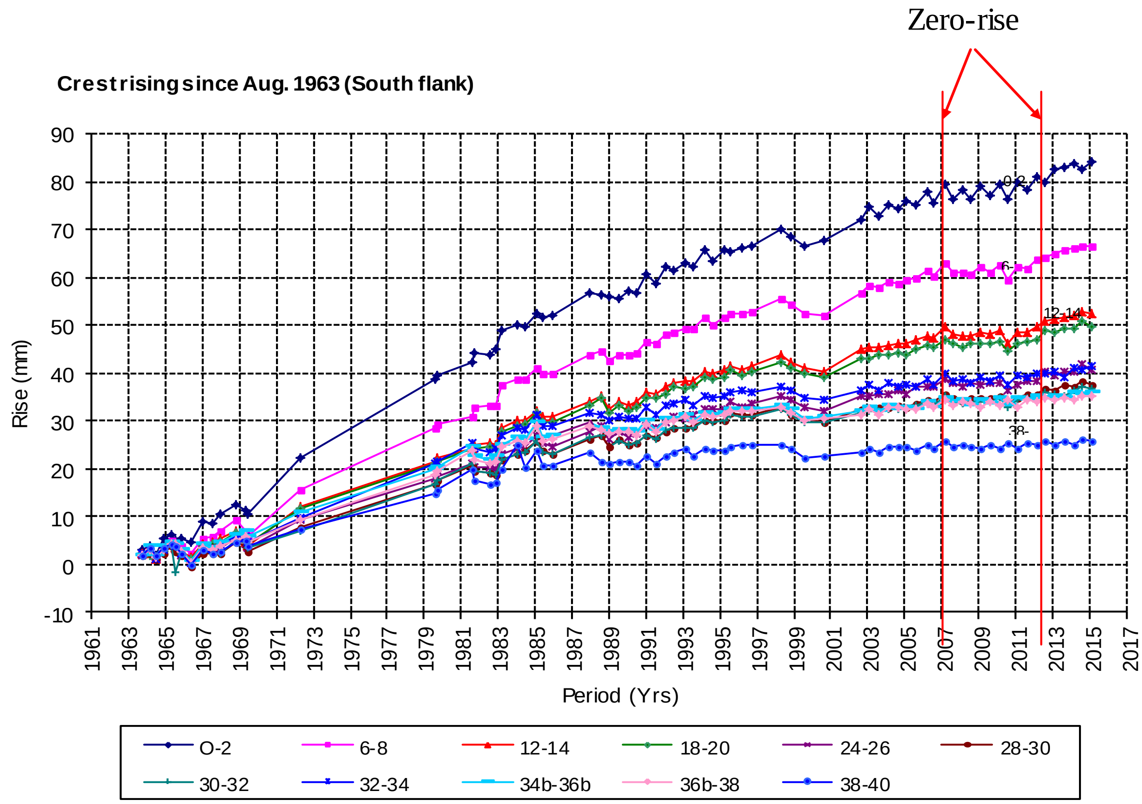

***Figure 3.14**: Graphs of annual rise rate of individual crest blocks for south bank; courtesy of ZRA*

3.4.11.3 New dam behaviour

As evidenced by several past reports from ZRA, the AAR effect showed reduction tendencies by the year 2001 to 2012, after which the dam-crest-rise rate increased again to about 5mm over the period of 3 years (1.7mm/yr), compared to the expected decrease rate of 0.44mm/yr between 2012 and 2015. This represents an annual rate increase of about 4 times the expected in 2015 (see figure 3.15 below).

The above analysis led to the isolation of AAR from landslide effects, giving two distinct signatures as shown in figure 3.15 below. The AAR effect reduction over the years, as measured geodetically, has been also proved using laboratory tests.

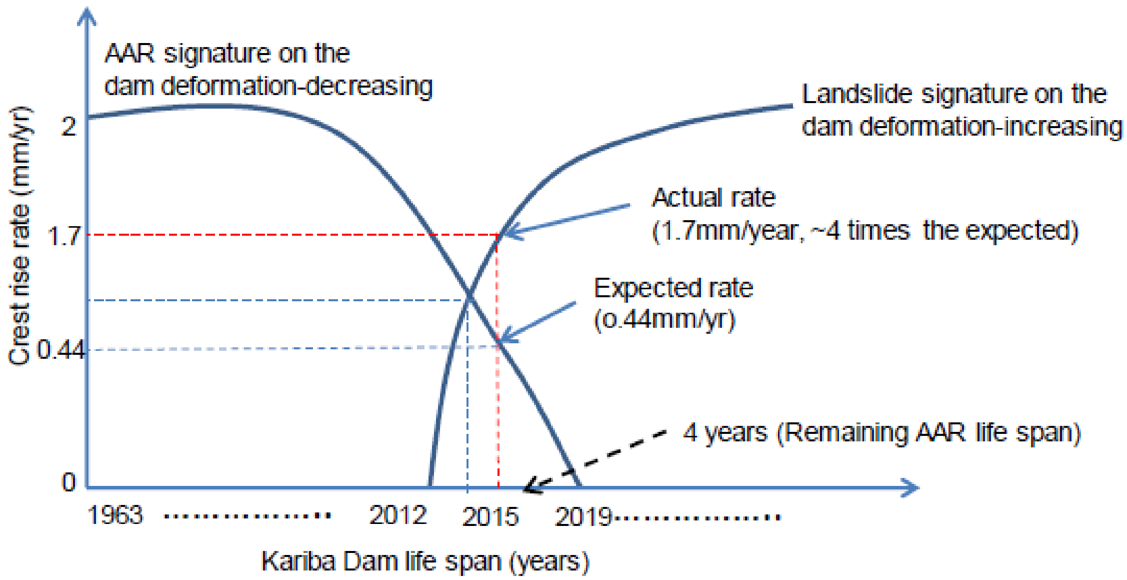

Figure 3.15: *AAR-Landslide effects on the dam*

From the two effects shown in figure 3.15, it was concluded that landslide is responsible for an annual rate increase of the dam deformations, evidenced in figure 3.13 and 3.14 above.

The south bank landslide effects manifested above may be attributed to the south bank extension earth-works which started in 2012. If this effect is not checked, the elasticity of the dam reinforcements may be exceeded, and this may spell dam failure.

Note: *Figure 3.15 above does not show that landslide started in 2012, but that its effect on the crest rise is what manifested in the period between 2012 and 2015. The landslide itself existed before 2012 but did not cause significant crest rise, thus the rise rates were detected as reducing before 2012.*

The time lag between the existence of the landslide and the dam crest rise is due to the fact that the dam is a reinforced concrete structure which has a significant resilience capacity to withstand compressional landslide load within a certain limit, for a given period of time; after which it starts to buckle.

The manifestation of landslide effect on the crest rise after 2012 is an indication that the dam reinforcements are now buckling under the landslide load (see drawing 2 and 3 in the Appendix C). *How long this buckling will take before breaking depends on the elasticity of the concrete reinforcements, and how well the landslide is monitored and controlled.*

3.4.11.4 Other effects of the landslide on the dam

The south bank landslide is associated with a force being applied at the south abutment which has resulted into the following (see drawing 1, 2 and 3 in the Appendix C):

1) Horizontal clockwise rotation of the north arch end and vertical south bank vertical rotation southwards. This was evidenced by the north arch end movements and the negative pendulum readings reported by Tractebel engineers in 2011

2) Northward tangential movement of the arch (dam as a whole), also as indicated by Tractebel engineers in 2011

3.5 Current deformation monitoring system errors

The current classical geodetic survey method relies on the established control points which are monumented on and around the dam. However, the current deformation monitoring system may be prone to errors due to some control point instabilities.

Figure 3.16 below illustrates how errors are introduced in the measurements owing to unstable control points.

In the illustration below, assume that two points in the reference network have deformed, as shown in figure 3.16 below, at time t_2. Let SB 17 and NB1 represent the two points. Let their coordinates at t_1 be denoted (x_{sb17}, y_{sb17}, z_{sb17}) and (x_{nb1}, y_{nb1}, z_{nb1}), respectively.

Due to ground instabilities, at time t_2, both points have drifted and assumed coordinates;

($x_{sb17}+\sigma_x$, $y_{sb17}+\sigma_y$, $z_{sb17}+\sigma_z$) and ($x_{nb1}+\sigma_x$, $y_{nb1}+\sigma_y$, $z_{nb1}+\sigma_z$), where σ is a drift in x, y and z; **Note**: The drifts of SB17 are different from those of NB1 at time t_2, and from the rest of the control points in the network, thus the *positional error inherited is not one constant*.

Figure 3.16*: Total Station (left) mounted on unstable control point, SB17, (right); courtesy of ZRA*

3.5.1 Bearing and distance errors

The resulting bearing **α** and distance **d** between SB17 and NB1 due to the drifts at t_2 are;

$$\alpha = \tan^{-1}[\frac{(\Delta x + (\sigma_{nb1} - \sigma_{sb17}))}{(\Delta y + (\sigma_{nb1} - \sigma_{sb17}))}] = \tan^{-1}[\frac{(\Delta x + \Delta\sigma_x)}{(\Delta y + \Delta\sigma_y)}] \tag{8}$$

$$d = \sqrt{(\Delta x + \Delta\sigma_x)^2 + (\Delta y + \Delta\sigma_y)^2} \tag{9}$$

$$\sigma_{nb1} \neq \sigma_{sb17}$$

Note: because the drifts in x, y, z for SB17 are not the same as those of NB1 or any other point in the network, the *errors do not cancel out and thus are passed on* in the computations and results. Hence the traditional monitoring method is erroneous, and is not suitable for Kariba Dam monitoring.

3.5.2 Least squares adjustment failure

In the above illustration, a Total Station set up on a reference point at time t_1 gives coordinates of the observed point as a scatter plot (a) in figure 3.17 below, because the coordinates for the same point will not remain the same due to errors beyond human control.

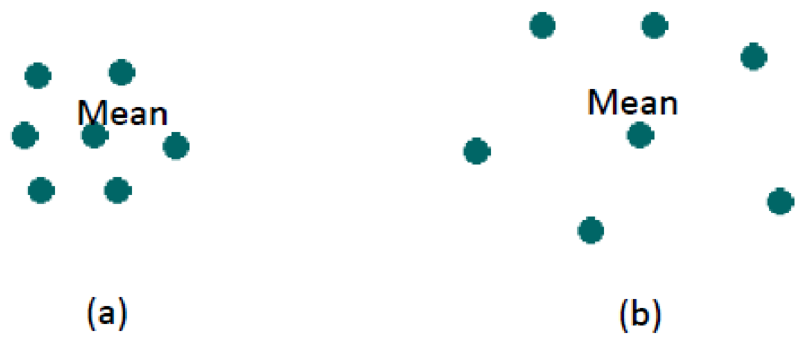

***Figure 3.17**: The scatter plots obtained at t_1 (a) and time t_2 (b)*

During data processing, the measurements are averaged to get the mean position of the measured point which is used in the network adjustment using least squares method.

The next round measurement carried out at time t_2 to the same observed point gives a scatter plot (b) with more scattered points around the mean. The resulting standard deviation obtained from this scatter plot is relatively bigger due to errors, $\Delta\sigma_x$ and $\Delta\sigma_y$, in **x** and **y** respectively passed on in the computations.

Meanwhile, the model assumes that the mean error standard deviation (E) is infinitely small such that it can be equated to zero *(Larson and Farbe, 2003)*. The mean algebra equation 3, however, becomes;

$$y = a(x + \Delta\sigma_x) + b \tag{10}$$

The mean error standard deviation, denoted as E in equation 3, becomes;

$$E(a,b) = \sum_{n=1}^{N} \left((y_n + \Delta\sigma_y) - ((ax_n + \Delta\sigma_x) + b)\right)^2 \tag{11}$$

Due to the inherited errors in x and y, the value of E is not equal to zero, thus the model assumption is violated, and hence the model gives incorrect results;

$$\frac{\partial E}{\partial a} \neq 0 \quad \frac{\partial E}{\partial b} \neq 0 \tag{12}$$

Hence, the results computed in this way may not reflect the actual deformations. It must be mentioned here that these errors result from deforming control pillars despite the accurate instruments and reliable software being used. Thus, on account of the unstable geodetic monitoring networks of the Kariba Dam, the classical method currently being used fails to meet the standards, hence unreliable and needs upgrade.

3.5.3 Inadequacy of the current monitoring method

The traditional deformation survey methods currently being employed for Kariba Dam monitoring has defects also in the data capture as the outside vertical dam surface is not measured. The verticality of the dam wall currently is monitored through crest and gallery control points, and pendulums which measure drifts at (a) and (b), respectively as shown in figure 3.18. This method is only ideal for homogeneous dam deformations where the amount of deformations at the dam bottom is equal to the crest deformations. But the Kariba Dam deformation is not homogeneous because different dam areas deform differently; the dam can twist and tilt at the same time due to unexpected forces being applied on the structure as shown in figure 3.18 below.

The inadequacy in data capturing was reported in the 5-Yearly Inspection Report of 2011 by Tractebel Engineering (Coyne and Bellier). It was noted that the tangential abutment movements towards the north and rotational tendencies of the arch about its downstream-upstream axis was not confirmed by pendulums because arch ends were too far from the pendulums.

Because of the possible weakened foundation from the scouring plunge pool, along a faulty line, the deformations are expected to increase from crest to bottom (foundation) because the hydrostatic pressure increases with water depth. Using the current method, dam displacements may be under reported due to lack of measurements on the vertical downstream surface which would otherwise give a complete dam deformation analysis from top to bottom.

3.5.3.1 Pendulum Flaws

The use of pendulums to confirm the dam behaviour under non-uniform deformations may result in contradicting results, because if the dam bottom drifts downstream more than the upper section, the dam inclines upstream and the pendulum gets a negative reading at section (b) in figure 3.18 below, which may be interpreted as upstream drift.

A pendulum gives a negative reading because of the upstream deflection of the line of gravity relative to the original position, as shown in figure 3.18, as a direct effect of the more deformations at the bottom than towards the crest. The confusion that may arise from this kind of deformation is that, the geodetic survey would indicate that the dam crest has deflected downstream as shown at (a) in the figure 3.18 below, yet the pendulum would indicate an upstream deflection; even when actually the dam has deflected downstream, except that the deformation towards the dam foundation is more than the upper sections.

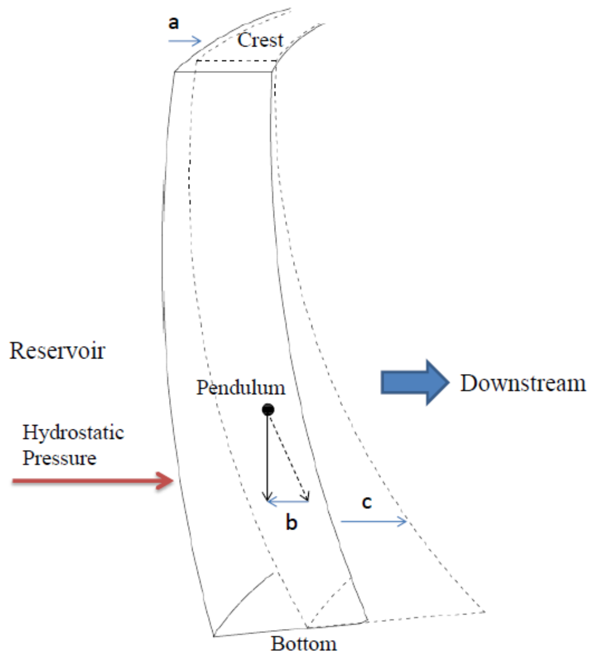

Figure 3.18: *Cross Sectional view; illustration of non-uniform dam deformation*

This kind of heterogeneous dam deformation cannot be reported correctly using the traditional methods because the vertical dam surface is not measured in its entirety. In order to fully capture the dam drifts, there is need for a monitoring system which can measure deformations at (c) as shown in figure 3.18 above. Measurements on the vertical surface are only possible with installation of object points (Prisms) on the dam vertical surface which would define its geometry, thus making it possible to monitor its geometric deformations

3.5.3.1.1 South Bank vertical rotation

A similar problem to the above was reported by Coyne and Bellier in 2011 and by ZRA safety engineers in 2012 that some pendulums (P9 and P11) contradicted the known downstream drifts tendencies by giving negative readings in the y-axis (away from the river); while all other readings from the geodetic measurements indicated drift towards the north. Coyne and Bellier attributed the negative readings to faulty pendulums (*Human Causative Factor to dam failure-wrongheaded conclusion which suit expectations, not evidence*). See table 3.5 below.

***Table 3.5**: Doubted negative pendulum readings*

mm/15 years	North (Y)		Downstream (X)	
	Geodetic	Pendulum	Geodetic	Pendulum
P8	-0.3	1.5	2.3	1.8
P9 (P1 Shaft) P2	-4	-2.2	4.6	2.7
P11	-0.3	-6.3	7	2.1
P4	0.6	2.3	3.1	2.6
P5	2.4	2.7	4	3.6
P7	10.2	N/A	4.7	N/A
PS1	21.4	25.3	3.1	4.3
PS2	23	26.6	-2.7	-3.5

This deformation questioned by Coyne and Bellier is possible under the geological conditions of the South Bank. Pendulum 9 and 11 are situated on the South Bank Abutment where reports of poor geological have been done.

The phenomenon which puzzled Coyne and Bellier of Tractebel Engineering (Kariba Dam designers) is explained and illustrated in figure 3.19 below.

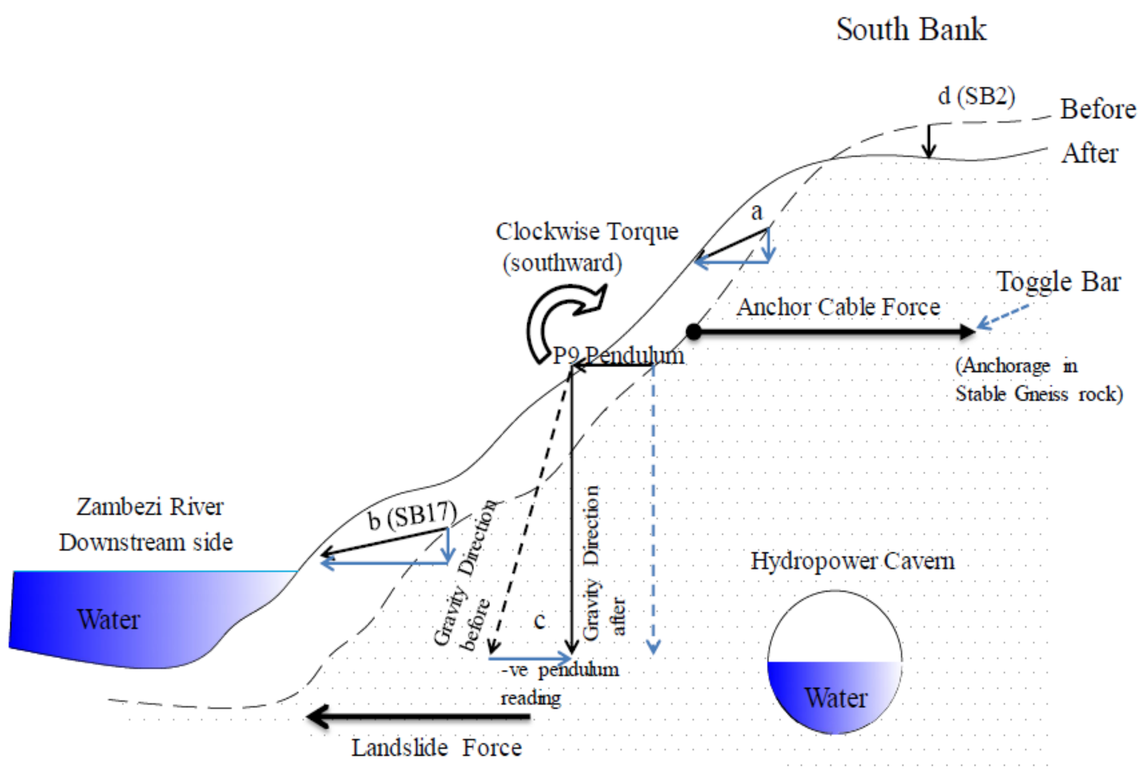

Figure 3.19*.Contradiction of pendulum readings*

From the displacements of lower portions such as (b) compared to the top ones at (a) in the figure above, it can be concluded that the displacements of the south bank bottom are more than the top parts. This phenomenon reflects the fact that the anchor cables and the landslide are creating a clockwise torque which causes the hill and the south abutment to vertically rotate in the clockwise direction about the anchorages. As the hill rotates clockwise, the upper side around SB2 will be detected sinking (as evidenced by the sinking behaviour of SB2 control point), tending to collapse the tunnels below. Points around SB17 will be detected sinking only for a certain period of time after they start to rise again owing to the south ward vertical rotation of the landmass on which they are anchored. The vertical rotation transmitted to the north bank may be responsible for the contradicting-like rising tendency of NB4 (illustrated in drawing 6 in the Appendix C) as the dam tends to lift up the hill; but at the same time causing it to collapse due to the tunnel voids under it.

3.6 Rotational effects on the measurements

As the landmass or abutment rotates vertically clockwise (southwards), the center of gravity deflects to the south, as shown in figure 3.19 above, detected by pendulums which will measure negative readings.

In addition, the lower portions of the landmass will translate towards the river (positive north) which will be detected by geodetic instruments as positive. Thus the result is a contradiction in the readings such as the ones Coyne and Bellier questioned and disputed. The negative readings are a sign of *a new deformation pattern of the dam*. Hence, attributing the negative readings to faulty instruments may be a catastrophic human factor which may lead to wrong decisions on the mitigation measures.

3.7 Other Rotation-related problems

The clockwise rotation (southward) of the anchored landmass and the abutment might cause loss of tension in the Anchor Cables which are pulling to the south, see illustration above. The loss in the tension of the Anchor Cables may be mistaken for the loosening of the toggle bar which may lead to subsequent re-tensioning of the Anchor Cables, hence more clockwise rotation which may end up triggering more rotation, therefore more northward movements of the south abutment. More movements may result in the dam failure.

Therefore, the negative pendulum readings (questioned by Coyne and Bellier in the 2011 5-yearly inspection) were pointing to the real problem. Hence the assertion that the pendulums might have been faulty could be incorrect.

3.8 Earth-work effects on dam stability

As more tunnels are being drilled in the south bank hill, one expects more vertical rotation due to less strain resistance resulting from landmass-to-void ratio problems caused by the hydro power extension works (*Human Factor-over ambitious and risky extension works without thorough feasibility studies*). In addition to the weakening compression strength of the south bank, vibrations from drilling and blasting, or indeed a

Reservoir Induced Seismicity (RIS), may accelerate the landslide which may result in more northward force, hence more rotation, and more rotation may lead to dam failure.

Note that dam failure may result from *compromised strain resistances* of the hills anchoring the dam; despite the fact that the earth-work vibrations are monitored accurately. This effect may not take place during the construction stages, but may occur years after construction.

Hence limiting the vibrations accurately alone is not enough, thus must be supplemented by controlling the amount of voids being created under the hills for hydropower generation. This may mean changing the design of the project. Reluctance by the authorities to this idea due to the amount spent on the project so far is another Human Causative Factor-*unwillingness to forgo sunk costs.*

3.9 Reasons for misinterpretation

The deformation data misinterpretation is because the current traditional method is not adequate as measurements are not taken on both bottom and top along corresponding lines. Thus the torque created due to the bigger displacements at the bottom than at the top is not detected; instead it is explained as erroneous. This problem was attributed to faulty instruments (pendulums P9 and P11 which gave wrong results) by Coyne and Bellier (*Human Causative Factor to dam failure-optimistic misunderstanding of the risk*).

3.9.1 Inadequate landslide data capture

The secondary control network (see drawing 5 in the Appendix C) established to monitor south bank landslide is only adequate to detect existence of the landslide but not enough to report on the speeds and accelerations, and effects of the problem. This is because the control points are not arranged to define the geometry of the slope. In addition to irregular arrangement of control points, the monitoring frequency (two times a year) is not suitable to characterize a landslide. It is for this reason that the effects of the landslide on the dam have been underreported as shown in the data analysis above. The pendulums established to confirm landslide under non-uniform landslide drifts may result in more confusion like was the case in 2012 and earlier when negative readings from the pendulums were a puzzle.

The south bank secondary control network was introduced in 1991 and was designed to detect landslides of the south bank using the classical deformation monitoring method. It was designed based on the old south bank geological status which has since weakened further following the earth work activities in the extensions of the hydropower caverns on both south and north bank. The designers did not foresee the hydropower extension needs which only started around 2012 for the south bank and 2008 for the north bank.

Although piezometers have been put to monitor blast and drill vibrations in addition to geodetic measurements, limits of vibrations could be exceeded on account of inaccurately reported landslide and its rates of movements. This means that a landslide may occur unnoticed.

Chapter 4 talks about the new monitoring system design which seeks to address the above monitoring flaws of the Kariba Dam.

4.0 GEODETIC DEFORMATION MONITORING SYSTEM DESIGN

Having analyzed the Kariba Dam deformations and the current monitoring systems with their associated flaws, a Global Navigation Satellite System, Online Control and Alarm System (GOCA) was designed to address the challenges faced by the Kariba Dam.

4.1 Reasons for the design

The Kariba Dam deformation problem calls for an updated monitoring system to fully characterize the deformations on the dam and its surrounding areas. The current method despite being accurate has limitations in the data capture under current dam deformations. Limitations in the data capture lead to inaccurate results which do not reflect the real deformations on the ground. Inaccurate results would lead to underestimation of the problem by decision makers.

The new monitoring system therefore was designed to measure landslide drifts and its effects on the dam, accurately. The new system was designed to also measure deformations in areas where the traditional method currently does not give deformation results.

From the geodynamic analysis, it was obvious that truly stable ground assumed for primary control network against which displacements are measured is theoretical; as the real-world ground is unstable. This simply means dam monitoring under dynamic geology requires a dynamic system which hardly gets affected by the deteriorating geology. The system was designed to check and update (calibrating) the positions of monitoring control points automatically. This avoids introduction of errors in the results which would otherwise be mistaken as deformations.

In case of a disaster, the system is designed to report on the deformations online and in real time to shorten the time required to mitigate the potential dam failure effects on people and property further downstream.

The system meant to address the Kariba Dam problems is one which employs GNSS which must be installed as Continuous Operating Reference Station, Local positioning sensors (LPS) such as Total Stations, and weather stations for dam deformations. This system is called GOCA, an automated online-based monitoring system. It measures, analyses, visualizes and reports on displacements automatically. It has an option to set deformation thresholds beyond which alerts are sent via mobile text messages and internet to classified users who are mainly decision makers.

The new system is also compatible with geotechnical instruments such as water level gauges, strain-meters, piezometers, etc.

4.2 Conceptual design

The conceptual design in figure 4.1 below is an illustration of how the GOCA system for Kariba Dam is designed to work. The monitoring sensors transmit data signals to the main data transmitter which, in return, sends the data to main server at the control center.

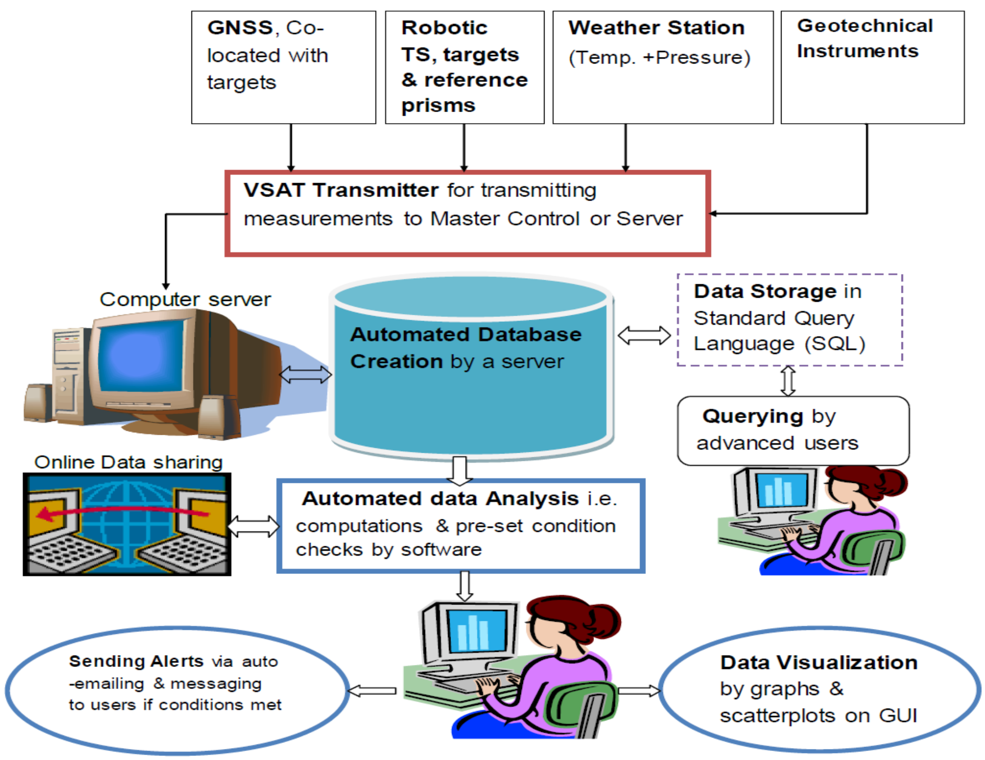

***Figure 4.1**: Schematic GOCA system design*

The server creates a database for storing measured data for subsequent data validation, analysis, visualization and alert issuance should the user pre-set conditions be met. The stored data in the database can also be queried by advanced users.

Data visualization is done online via a Website accessed by classified users with passwords.

The system may be run on either Trimble 4D or Leica GeoMos Software, both of which are Web-based monitoring software.

4.3 GNSS-RTS Physical design

The Automated deformation monitoring system designed in this research is an integration of Robotic Total Stations (RTS) and GNSS CORS. The concept is based on the fact that GNSS accuracy alone is unacceptable for deformation monitoring of the Kariba Dam deformations and Total Station has an acceptable accuracy but it becomes inaccurate in the case of the Kariba Dam geology which does not provide stable ground for reference stations.

Figure 4.2 below illustrates the co-location technique where two Robotic Total Stations (RTS) and GNSS antennae are co-located in order to calibrate the RTS positions between different measurement sessions.

The following are the reasons for the design:

- The base station GNSS CORS outside the deforming zone is meant to calibrate monitoring stations,1 and 2, through a correction signal
- Station 1 is designed to monitor landslide, dam arch drifts and rotations on the downstream side
- Station 2 is designed to monitor upstream arch geometric deformations illustrated on a vector map and arch geometric status drawings (see attached drawings 1, 2, 3 and 4 in the Appendix C).

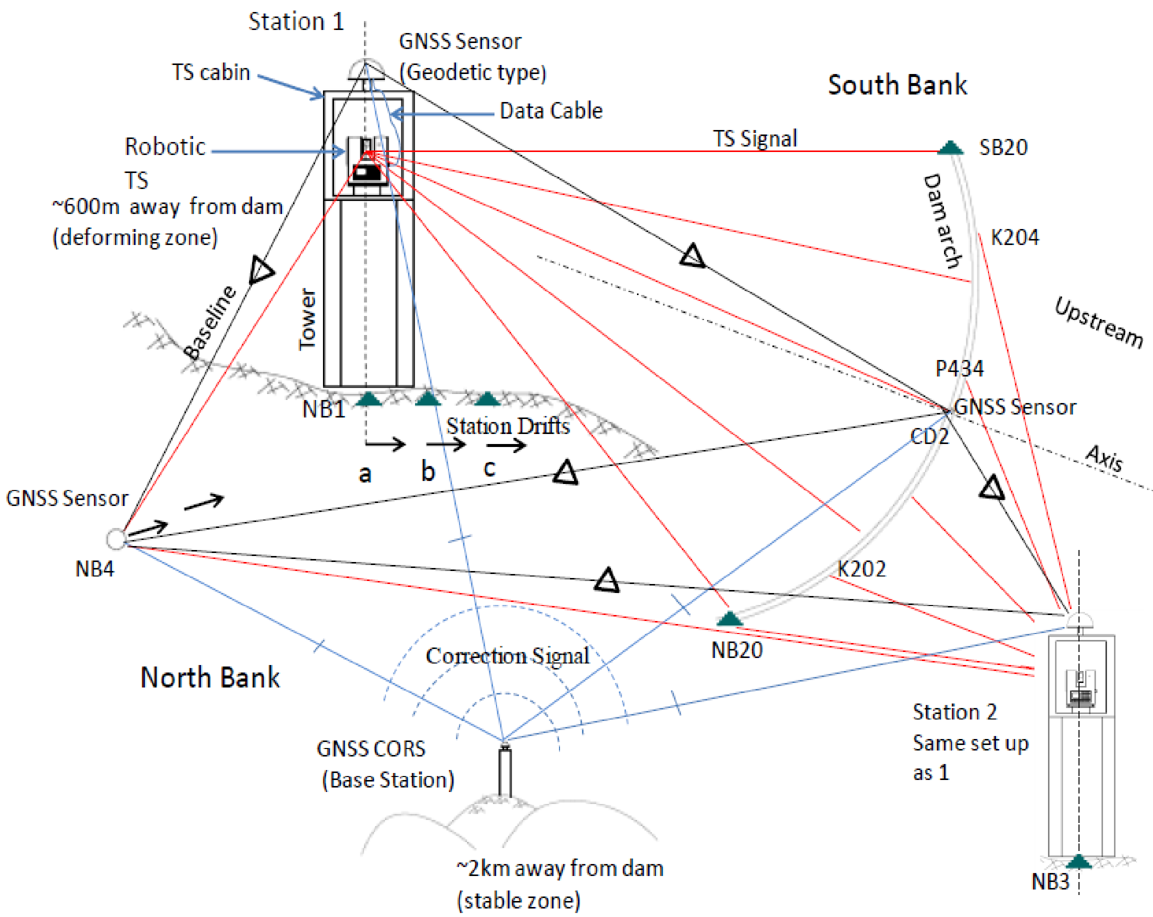

Figure 4.2: *Geodetic deformation monitoring system design for Kariba dam*

4.3.1 Error handling using RTS-GNSS co-location

Owing to the ground deformations around the Kariba Dam, the RTS position (for example NB1) drifts after a certain period of time, and assumes positions a, b, c, etc. as shown in the figure 4.2 above, at times t_1, t_2 and t_3, respectively. Thus NB1 will have coordinates: **a**(x_a,y_a,z_a),**b**(x_b,y_b,z_b) and c(x_c,y_c,z_c) at these respective times.

Note: Under the current deformation monitoring method, the same set of coordinates are adopted for different positions as **a**(x_a,y_a,z_a), **b**(x_a,y_a,z_a) and c(x_a,y_a,z_a) at times t_1, t_2 and t_3, respectively. Remember that the reference station, for example, NB4 is also drifting. Thus reference stations, NB1 and NB4 inherit position errors due to the adoption of the coordinates which only applied at time t_1. Hence the baseline bearing $\boldsymbol{\alpha}$, NB1-NB3, computed as below is not correct due to the inherent errors, $\Delta\sigma_x$ and $\Delta\sigma_y$ in x and y respectively (recall eq.8);

$$\alpha = \tan^{-1}[\frac{(\Delta x + \Delta\sigma_x)}{(\Delta y + \Delta\sigma_y)}]$$

The error incurred in the z affects the vertical angle which is used for height computations. The errors inherited in this manner are as many as the drifting reference points in the network, and are not the same owing to the different residual errors for each control pillar, thus cannot be modelled using a Total Station alone. This renders the *current monitoring method inaccurate.*

The introduction of the GNSS Continuous Operating Reference System (CORS) is meant to model the positional errors, owing to the unstable ground, in the deformation monitoring system of the Kariba Dam.

The GNSS CORS sends correction parameters ($\Delta\sigma_x$, $\Delta\sigma_y$, $\Delta\sigma_z$) to the rover GNSS antennae co-located with the RTS at the observing stations (1 and 2) at times t_1, t_2 and t_3. The rover GNSS antennae then feed the corrections into the RTS through their respective data cables. The GNSS antennae at CD2 and NB4 are also corrected for their position errors. Note: the GNSS antennae at CD2, NB4, NB3 and NB1 form a dynamic reference network free of poor geology-related errors.

Using the algorithm inherent in the RTS, the correction parameters are applied every time the RTS station drift owing to the unstable ground. The RTS and GNSS can be synchronized in such a way that the corrections are applied at the beginning of every measurement cycle.

After corrections have been applied to update coordinates at t_2 or t_3, the only remaining errors are the GNSS manufacturer's errors in the positions. It is known that the errors (σ_{xgnss}, σ_{ygnss}, σ_{zgnss}) from the GNSS are *unacceptable* for Kariba Dam deformation monitoring; it is also known that the *GNSS errors are same and constant* throughout the dynamic reference network during a particular session of measurements.

When the bearing is computed between two points of the same position error;

$$\alpha_{corr.} = \tan^{-1}[\frac{(\Delta x + (\sigma_{gnss} - \sigma_{gnss})}{(\Delta y + (\sigma_{gnss} - \sigma_{gnss})}] \tag{13}$$

The GNSS position errors in **(x, y, z) cancel out** because they remain constant and same, between reference points, for the rest of the measurement cycle.

Hence the bearing becomes;

$$\alpha_{corr.} = \tan^{-1}[\frac{(\Delta x + 0)}{(\Delta y + 0)}] \qquad \tan^{-1}(\frac{\Delta x}{(\Delta y}) \tag{14}$$

Hence the **error-free bearing** is achieved**.**

The same correction procedure is repeated for distances between points.

The error handling procedure is done in real time without loss of lock to the GNSS satellites, thus providing higher accuracy in the deformation monitoring results.

Figure 4.3 below is an illustration of the physical design which combined Google image of the Kariba Dam and photos of the integrated installation.

The photo (real world) deformation monitoring design below shows:

- GNSS antennae co-located with Robotic Total Stations, and prisms
- Visual realism of the whole design set up, except that the GNSS CORS on the photo must be upstream; but for lack of space on the upper side, it was placed downstream merely to give an impression that it must be outside deforming zone
- How and where the weather stations must be located (on top of the RTS cabin)
- Also how prisms must be annexed onto the surfaces (dam and slopes)

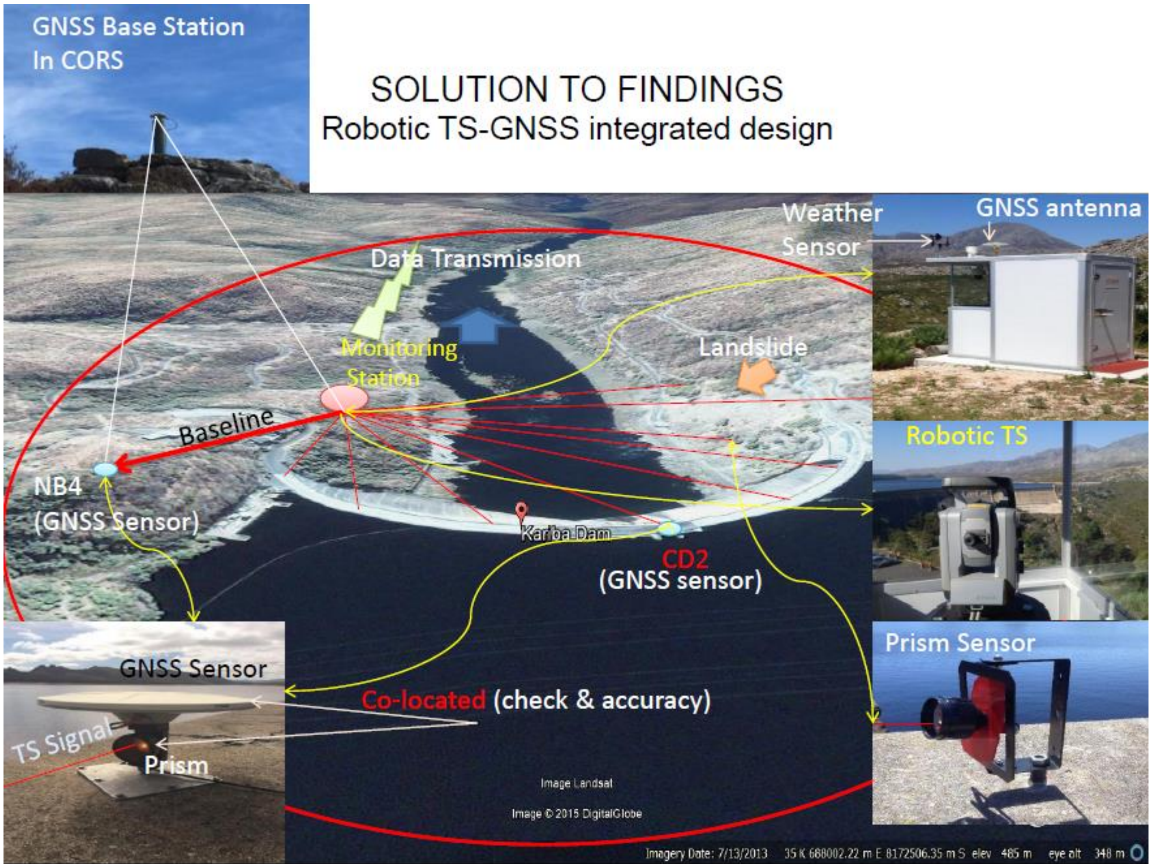

Figure 4.3: *Geodetic deformation monitoring system for Kariba Dam(Photos taken from Steenbras dam-SA)*

4.3.2 Design features

In order to address the deformation monitoring challenges faced at Kariba Dam, the GOCA in-situ system is designed to have the following features:

- One GNSS Base Station as Continuously Operating Reference Station (CORS) permanently installed outside deforming zone (red boundary in shown in figure 4.3) for providing correction parameters, making the dynamic control network which suits the dynamic geology of the Kariba Dam.
- Two Monitoring Stations each comprising one Robotic Total Station (RTS) co-located with one GNSS receiver installed on top of the RTS housing as shown above in figure 4.3. Downstream RTS will be measuring the downstream

deformations while the upstream RTS will be measuring the upstream drifts(see the kinds of drifts in drawings 1,2,3 and 4 in the Appendix C)

Note: due to visibility problems and size of the dam, a minimum of two observation stations are suitable for Kariba Dam monitoring.

- Two Towers (one for each observation station) or high ground, see figure 4.2, for a good view to the dam
- One GNSS sensor on the dam crest (CD2) and another GNSS sensor on NB4 for baseline computations and loop closure adjustments of GNSS solutions. Both GNSS antennae must be prism co-located to enable RTS measurements to them
- An array of prisms to be installed on the dam crest (both upstream and downstream edges) and on the downstream vertical wall of the dam. This is to allow measurement of dam wall from top to bottom
- An automated weather sensor for measuring weather parameters like temperature and atmospheric pressure.
- An automatic water level meter for water level measurements.
- Local Wi-Fi installation for data transmission to a VSAT transmitter which finally sends the measurements to the master station where data analysis, visualization and alert issuance are done (*Trimble, 2015*).

The physical design above would not be effective without an improvement on the monitoring networks around the Kariba Dam; more object points must be introduced with permanently installed prisms in regular array.

Thus, the prism array was design to go alongside the physical deformation monitoring system designed above.

4.4 Prism Array design

Since the top and bottom deformations at Kariba Dam are not the same, a prism array was designed to be permanently installed on the surface in a grid pattern in order that measurements on both top and bottom may be taken. This array was designed to be used for both classical and modern monitoring technology. The prism array will also address the unusual deformation patterns noted by Coyne and Bellier in 2010.

The downstream vertical dam face is designed to have a minimum of 7 prisms in each row fixed at a constant interval from south bank to north bank (see figure 4.4 below). The south bank is designed to have a minimum of 3 prism columns running from top to bottom of the hill towards the river, with each column meant to detect landslide at its slope location.

The north bank prisms should be carefully placed as most parts of the bank are not exposed due to vegetation. Prisms especially on the north bank should not be installed close to vegetation thickets as physical weathering may affect their stability which might be misinterpreted as having been caused by the arch end movements.

The designed prism array in this research will detect the following:

1. South bank landslide-drift directions towards the river and rotation tendencies of the landmass

2. The rotational and tangential deflection tendencies by the arch ends which are not confirmed by pendulums at the moment (see drawings 1,2,3 and 4 in the Appendix C)

3. Any unusual downstream/upstream deflections of the dam from top to bottom. This is a follow up on effects of back-cutting of the plunge pool towards the dam toe

The prism array concept is illustrated in figure 4.4 below, although not all the prisms are shown in the sketch for clarity purposes.

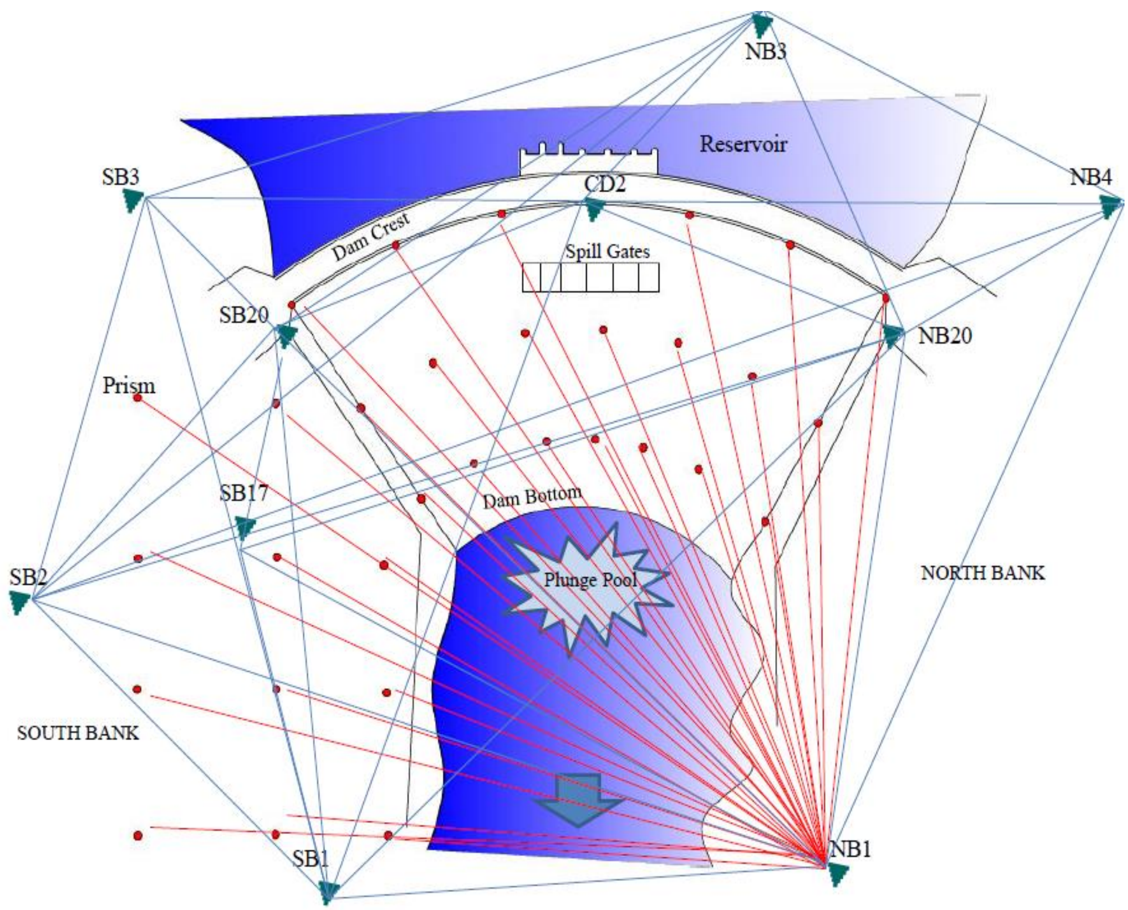

Figure 4.4: Prism *array for Kariba Dam monitoring*

The prism array may be measured from the primary and secondary networks, as shown in figure 4.4 above, using the currently employed geodetic deformation monitoring method.

The prisms on the slope surface must be protected against rock falls in order to ensure their safety all the time.

The prism array is vital as it is designed to supplement both primary and secondary networks of the Kariba Dam deformation monitoring in areas where control pillars where not installed. This will help in defining the geometries of the areas being monitored better, hence providing full geometric deformations of the monitored objects.

The prism array once installed will form a third Kariba Dam network called the *Prism Array Network (PAN).*

4.5 Advantages of the upgraded system

The upgrade of the deformation monitoring System to the systems described above has some of the following merits:

1. Integration of GNSS with Total Station ensures accurate results

2. It is fully automated, thus less human interference

3. Capable of measuring the full dam height to capture full deformation trend of the wall-rotations and translations through the prism array which defines the geometry of the dam and hillsides better than the existing primary and secondary reference networks of the geodetic monitoring system

4. Automated alert issuance forms a basis for quick response in case of a disaster

5. Data visualization is simplified through easy-to-understand graphs and plots viewed in real-time anywhere via a Website

4.6 Disadvantages:

The upgrade will not come without disadvantages. The following are some of the disadvantages:

1. The initial cost required to set up such a system is high (>$500,000)

2. The system requires training in order to run it effectively, hence more expenditure and time required by the company

3. Since the system is permanently installed, it requires a lot of security to guarantee its safety

4. Installation of permanent prisms on vertical dam surfaces may be a challenging and risky task; installers need to be dangled above the plunge pool with ropes anchored onto the dam crest

5. The system is only capable of measuring the outside surfaces only. Traversing inside the tunnels and galleries is not possible. Hence it requires an additional installation of automated geotechnical instruments in order to get inside measurements.

However, the benefits from the automated monitoring system seem to outweigh the disadvantages because the cost which may result from the dam failure is huge compared to the upgrade cost.

The proposed deformation monitoring system design in Chapter 4 above has been implemented in a number of concrete dams around the world. However, for this study, only two case studies were conducted: Sermo and Steenbras Dam, as detailed in chapter 5 below.

5.0 CASE STUDIES

5.1 Automated deformation monitoring: Case study of Sermo Dam, Yogyakarta Province, Indonesia.

Sermo Dam is located in Java Island and has been in operation since 1996. It was suspected that Sermo dam suffered deformations caused by earth quake, which made monitoring it, an urgent need.

A geodetic deformation monitoring system was designed to detect displacements in Sermo dam using Multi sensors permanently installed on the dam.

5.1.1 Sensors installed

The sensors installed are: 3D Robotic Total Station (RTS) sensor, an array of permanently mounted prisms, two units of Global Navigation Satellite System as Continuous Operating Reference System (GNSS CORS) sensors, Automatic Water Level Recording (AWLR) sensor, and Pan, Tilt, and Zoom digital Camera. The GNSS CORS (Leica and Javad) sensors are installed using two towers; the first tower is for the base station (SRM1) and the second tower, in the upstream area, is for a back sight station (SRM2) co-located with one prism (see photos and associated descriptions in the Appendix A). At the first tower, the RTS and GNSS CORS are co-located with a GNSS antenna installed at 2.30 meters above the RTS. The distance between the first and second tower is 2500meters. Also 19 prisms as targets of RTS which comprise 6 prisms at downstream slope and 13 prisms fixed on the dam crest and on the dam slopes. The Automatic Water Level Recording sensor and a digital Camera (CCTV) are installed at the Sermo Dam intake.

Robotic Total Station (RTS) and GNSS are housed in an observation cabin. The windows of the cabin constructed of four large glass panels joined in a faceted arrangement, similar to a control tower window at the airport (see photos in the appendices). Windows have 6mm thick soda lime float glass. This glass has 91% transmission of light due to its low iron content. The window is installed at a slight angle to prevent RTS line of sight perpendicular to the glass panel to avoid glass-reflected light coming directly into the RTS electronic distance measurement unit.

The system is operated on a set of time schedule, automatically turns on at the correct time, starts direction (horizontal and vertical) and distance measurement sets. The RTS is programmed to collect a prescribed number of sets of angles at predetermined time intervals. When the measurements are completed, the data is transmitted to the main office computer, via Wireless communication, for its evaluation, processing and deformation analysis and visualization. The RTS array was installed to measure within a radius of 400meters.

The Sermo Dam deformation monitoring system offers a standard deviation of distance measurement of square root $((1mm)^2+(2ppm)^2)$ and angular standard deviation of 1.5" (*Sunantyo et al, 2012*).

5.1.2 Leica GeoMoS software

The GOCA system installed at sermo dam is run on GeoMoS Leica software created by Leica Geosystems. The main features of the Leica GeoMoS software include great flexibility in choosing monitoring equipment and method, storing information in a Standard Query Language (SQL) type database, parallel use of multi sensor, a wide range of communication technologies for the control of sensors and data acquisition, creating a Total Station capable of measuring accurately up to the distance of 8Km with Leica TCA 1201M. Data can easily be imported and exported in various data exchange formats such as ASCII DXF and standard excel formats to GeoMoS service users *(Leica Geosystems, 2015)*.

5.2 GOCA deformation monitoring: Case study of Steenbras Dam, Cape Town, South Africa

Steenbras dam is situated in the Hottentots-Holland mountains close to the town of Gordon's Bay, South Africa, approximately 50 km from Cape Town. The dam is owned and operated by the City of Cape Town municipality.

Optron (Trimble's SA-based Agent) uses Steenbras dam as a demonstration site for Trimble 4D Control, Trimble's deformation monitoring software.

The monitoring setup consist of 1 NetR5 GNSS base station, 4 NetRS GPS monitoring points, 1 Trimble S8 Robotic Total Station, 3 reference prisms, 17 prism monitoring points and 1 Cambell Scientific weather station.
The GNSS monitoring points are all co-located with prisms as well as GNSS antennas as shown in figure 5.1 below.

Figure 5.1*: Co-located GNSS receiver with Prism; Courtesy of Trimble*

The co-location technique makes it possible to measure to the GNSS antennas as well as with the Robotic Total Station (RTS) to improve the accuracy of the rover GNSS solutions with reference to the base (master reference point).

The data from the GNSS receivers is collected via a local Wi-Fi network to the computer center.

The S8 Robotic Total Station (RTS) is an optical instrument that measures angles and distances to reflective targets (prisms) from a known location which is also being monitored through the GNSS receiver mounted on top of its Cabin roof, with reference to the zero-order GNSS base station. From these measurements coordinates of the targets are calculated to millimeter precision. See set up below of a GNSS receiver, S8 TS and a prism.

Figure 5.2: *S8 RTS (left) in its Cabin and target prism to the right; courtesy of Trimble*

Figure 5.3: *Protective cabin, housing the S8 total station as well as a GNSS receiver on top with the weather station (left) on the cabin roof; courtesy of Trimble*

The NetR5 GNSS and NetRS GPS sensors collect positioning information from a network of GNSS Satellites, and this data is then post processed in the Trimble 3D software to provide sub-centimeter accuracy for the coordinates. The GNSS base station is used as reference by these sensors. The GNSS base station is installed in the stable location free from the dam deformations as shown in figure 5.4 below.

Figure 5.4*: Shows the GNSS Base Station; courtesy of Trimble*

From these coordinates collected from the S8 RTS as well as the GNSS sensors, displacements are calculated for the monitoring points.

All the sensors are connected via a Very Small Aperture Terminal (V-SAT) link, or Satellite Internet link, to a server in the Trignet offices in Mowbray, Cape Town that has the Trimble 4D Control (T4D) software suite installed on it.

A V-SAT is a cost effective satellite communications system over a wide geographical area (*Vsat*, *2015*).

5.2.1 Trimble 4D Control (T4D) Software

The whole set up explained is controlled by Trimble 4D Web which is used to analyze, visualize monitoring projects, and provides access to a monitoring system over a fast, feature rich Web interface. Visualization of the dam deformation is Web-based where classified users log on for data analysis and queries.

5.2.2 Advantages:

Trimble 4D software has the following advantages:

- Gives very intuitive maps with customized views to identify sensors and measurement points. You can use aerial photos or other imagery to provide detail background information

- Fast and easy charting makes it possible to plot the results of individual points or sensors

- It provides Link to project Webcams in real-time and make visual inspection from any location in the world via internet

- The users can set conditions for issuing alerts and messages and manage in a secure environment i.e. controlled access to specified stakeholders by assigning different access levels to ensure information is available only to those who need it

Trimble 4D Control supports various applications to enable an extensive and growing range of capabilities for different monitoring needs. Each application offers a number of modules and functionalities for specific monitoring operations such as instrument controls, data collection, data processing, data analysis and issuance of alerts.

The software remains open to new additional applications to the existing installations any time as need arises. Some of the applications which are supported by Trimble 4D software include (*Trimble, 2015*):

- **Radar**: Pre-processed data from a radar device to calculate displacements based on radar interferograms such as IBIS-FL GeoRadar system

- **Seismogeodetic**: Integrates strong motion and high frequency data for the combined processing of accelerometer and GNSS data using a Kalman filter to produce high rate GNSS displacement data

- **Geotechnical**: Provides continuous high precision measurements and rapid updates to monitor over long distances

- **Integrity manager**: Provides real- time and Post Processing Engines to monitor GNSS reference stations

The 4D Trimble Control Software consist of the following features:

1. **Trimble Pivot Platform (TPP)**, which comprises many different modules, whose main task is to control the collection of data from the various sensors and controlling of the scheduling of cycle measurements of the Trimble S8 RTS. All data collected are stored in Microsoft SQL database. A Database called TPPDB is automatically created in Microsoft Standard Query Language (SQL).

The TPP also has all modules for GNSS data post processing called Integrity Monitor which plots displacement graphs.

2. **Trimble 4D Control Desktop** for selecting which sensors are to view in T4D Web.

T4D Web allows any user on the same network to analyze and view data on his or her own computer using a web browser.

The main function of T4D Desktop is to provide the link between Trimble Pivot Platform, and T4D Web-provides a Graphical User Interface (GUI) for easy interaction with system users.

The user creates a project with the correct coordinate system, and then chooses which sensors to add in the project, all of which will be visible in T4D Web.

There are also different calculation sensors that can be added, that automatically do calculation from the data received from various sensors, i.e. the Virtual Crack meter, which calculates the distance between two static sensors, and reports on the 3D distance between them.

T4D Desktop creates another SQL database called TPP Monitoring, and all the data from the sensors that are added in the project, are copied from the TPPDB database to the TPP Monitoring database, to enable T4D Web to query data easier and quicker in the database.

3. **Trimble 4D Control Web**. Trimble 4D Control Web is used to view and analyze results from data collected by Trimble Pivot Platform. Once the software is installed on the main server, users can access the results of the monitored sensors via a web browser.

   T4D Web queries all the data from the TPP Monitoring database. Displacement graphs, maps and scatter plots can be viewed and printed for individual sensors, and all displacement results can be exported directly to Microsoft Excel. Analysis and alarms can also be created for groups of sensors. Different alarm levels can be configured and automatic email and SMS alarms can be sent once the alarm thresholds have been met. Advanced users can also use SQL to retrieve raw data directly from the database and conduct custom analysis.

5.3 Ultimate objective of the system upgrade

Because Hydrostatic pressure tends to stretch the arched dam while landslide compresses it; there exists a balance or equilibrium between the two forces which minimizes the effects on the dam. This is achieved by a certain amount of water in the reservoir which resists the south bank landslide. This ultimately means that as water level reduces in the reservoir, the hydrostatic pressure reduces while the landslide compressional force increases due to less hydro resistance as shown in the following figure. This explains why the Kariba Dam may be at risk (more south bank landslide) when water levels are low in the reservoir due to poor rainfall pattern. Thus, extreme reduced water levels in the Kariba Dam reservoir are as dangerous as high water levels; dam safety engineers are currently just concerned with high water levels.

The illustration of the water level and landslide relationship is shown in figure 5.5 below.

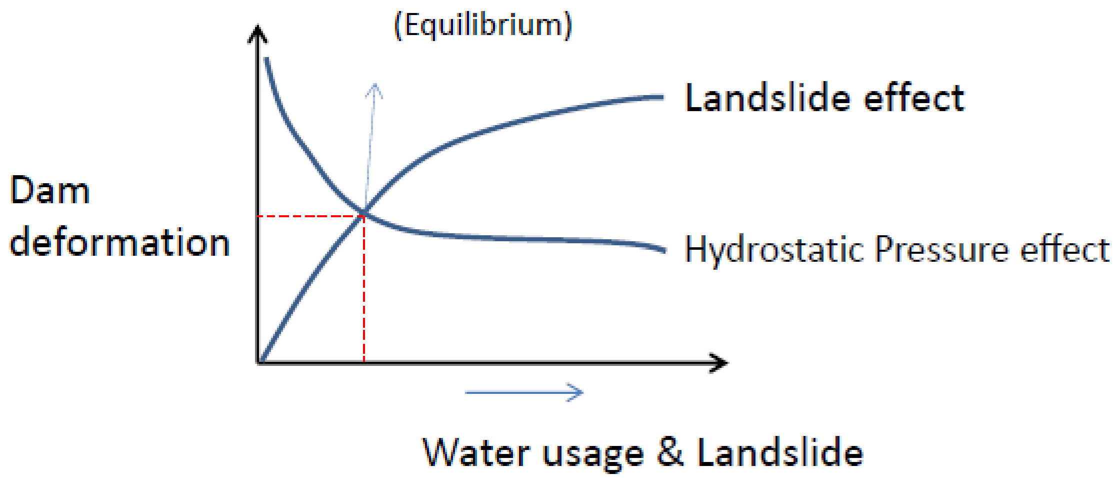

Figure 5.5*: Relationship between dam deformation, landslide and water level*

The balance between the landslide and the hydrostatic pressure can only be evaluated with more detailed continuous measurements of the effects of hydrostatic pressure and the landslide on the dam and its surrounding areas.

The relationship between water level and south bank landslide means water levels in the reservoir should be kept within safe thresholds. This creates a *natural remedy* for resisting the landslide which, ultimately, may minimize the dam deformations. This relationship can also be linked to the recent hydropower-capacity extensions to assess whether or not the hydropower plants can generate to the intended capacities safely, given the landslide-water level equilibrium which must not be exceeded. Exceeding the equilibrium water level may result in more landslides due to less hydrostatic resistance.

This kind of theoretical-based analysis requires all-time and error-free measurements on both the dam and surrounding areas. This can only be achieved with the automated GNSS-based measurements due to its improved techniques of error-handling during measurement cycles.

The ultimate goal is to ensure minimized landslide effect on the concrete structure to avoid further buckling which may lead to dam failure. The landslide effect can only be minimized by a certain water level in the reservoir which exerts equal and opposite force to the landslide through the dam buttresses. Thus Newton’s third law of motion may provide an answer to the landslide problem of the Kariba Dam.

6.0 DISCUSSION

The general understanding of the causes of the Kariba Dam unusual deformation has, for a long time, been attributed to the Alkali Aggregate Reaction (AAR) by Zambezi River Authority (ZRA) dam Engineers. This seemed true until around 2010. According to the geodetic deformation measurements carried out since 1970s, the dam crest heaving has showed significant reduction to a rate of 0.4mm/year between the years 2000 and 2010, an indication that AAR was reducing and the dam was settling.

The AAR was measured to be swelling at the rate of $5\text{-}10.10^{-6}$mm/year using geotechnical methods. However, the geodetic deformation measurements taken on the dam crest and galleries between 2010 and 2015 did not reflect this trend. The levelling measurements taken between 2012 and 2015 showed an increase in the upward heaving rate of the crest to about 1.7mm/year, about 4 times the expected reduction rate of 0.4mm/year.

The AAR swelling rate of $5\text{-}10.10^{-6}$mm/year as measured over the years since 1970 is too insignificant to cause an upward crest heaving of 1.7mm/year which was measured during the leveling exercise of the dam crest between 2012 and 2015. This phenomenon distances the AAR effects from the unusual dam deformations experienced between 2010 and 2015.

The sudden jump of deformation from 0.4mm/year to 1.7mm/year strengthens the assertion that AAR is not responsible for the strange dam deformations such as the upstream irreversible swelling rate of 13.7mm/year and 8mm/year on the South and North Bank, respectively. This is because the Alkali aggregate in concrete is diminishing and therefore cannot lead to an increased swelling rate of the Kariba Dam. For this reason, in this study, the AAR effect on the dam was estimated to stop by the year 2018. Therefore, any unusual Kariba Dam behaviour can only be attributed to other factors, such as human activities on and around the dam.

The ruling out of AAR as a candidate responsible for the unusual Kariba Dam behaviour leaves only one factor in question, the human activities (drilling and blasting), which may trigger earth tremors, in addition to the natural seismic activities; thus causing more landslide on the South Bank. Given the poor geology of the South Bank, a devastating landslide during and after the hydropower-extension earth works is possible and could not be understated in this research. It is for this reason that other deformation results on and around the dam were analyzed as well.

The dam deformation results reviewed that the arch ends have tangentially moved northwards by 7.2mm and 3.2mm over the period of 20 years on the South and North Bank, respectively. This northward tangential movement of the dam arch, however small, indicates that there is an axial load on the South Bank arch end where the drift is bigger and has caused a net force, an occurrence which destabilizes the dam arch. The unusual and undesired dam behaviour on the South Bank arch end is the northward movement; the normal tendency is the southward movement as induced by the hydrostatic and hydrodynamic forces through the abutment. This unexpected tendency is an indication that the hydrostatic forces have been overcome by some axial load which can only be attributed to the South Bank landslide.

Another indicator of the landslide occurrence on the South Bank is the fact that the South Bank half section of the dam crest has a bigger upward deformation than the Northern section. Additionally, control-point drifts on the south bank slope are bigger than the north bank ones, an indication of the slope instability. These deformations are also coupled with a downstream folding tendency of the Kariba Dam arch end as depicted by points SB20, RD1 and B750, shown in drawing 1 in Appendix C. This folding movement of the South Bank arch end could only result from a landslide push.

The south bank landslide may be amplified by blast and drill effects or earth tremors from the Reservoir Induced Seismicity (RIS), a major potential trigger of the dam failure. Any earth tremor now is likely to have a devastating effect on the dam because of potentially weaker hills which support the dam.

Therefore the blast and drill vibrations from the south bank hydropower extensions must be handled with due care and precautions so that the amount of force being exerted on the north bank is minimized, otherwise this may result into accelerated dam deformations which may in turn shorten the lifespan of the economic-pillar infrastructure.

It is a well-known fact that the Kariba Dam is a quasi-rigid structure with slightly varying concrete strengths and corrosion rates owing to varying wetness and exposures to the sun which induces temperature variations on the dam surface. Some sections of the dam receive more sunshine and hence experience more surface temperatures than the other sections. Additionally, some dam sections such as those around the spillways experience more wet conditions due to water spillages when the gates are opened, hence the spillway section is expected to corrode more than the other sections. Therefore, different portions of the Kariba Dam deform differently, both in magnitude and shape. This phenomenon manifests itself in the fact that the spillway section of the dam experiences more cracks than any other section on the dam. How the Kariba Dam deforms is also dependent upon the geological conditions of its footings and the point of load application. In this study, it was observed that the South Bank deformed more than the North Bank because the South Bank experienced landslide due to poor geology which possibly got worsened by the hydropower-extension earth works.

The rotational tendencies of the Kariba Dam reviewed in this scientific study are a result of how the dam is anchored or constrained relative to the point and direction of load applications. The Kariba Dam shape also has influenced the direction of movement of the dam arch under the landslide ‘push’. For example, the downstream-southward rotational tendency observed through object points RG1 and RG3 on the North Bank is as a result of the dam shape influence (See drawing 1 in Appendix C). The dam shape has also played a role in the direction of the upstream buckling, while variations in upstream deformation magnitudes are partly influenced by mechanical components such as the spillway gates which constrain deformation. Thus, the central or spillway section has experienced less upstream deformations than the arch ends (compare CD2 and K202 deformations in drawing 1 in Appendix C).

The south bank slope was also analyzed to have a vertical southward rotational tendency, possibly owing to the additional improvised anchorage of the slope to the stable rock through toggle bars which have ended up becoming pivots about which the slope tend to rotate under the landslide influence. This assertion, however, requires a detailed geological study to supplement the survey results which have consistently showed a vertical rotational tendency on the South Bank slope.

It must be borne in mind that the increase in the dam deformation rates coincided with the extensions of both the North and South Bank Hydropower Stations which resulted in an increased number of caverns or tunnels being drilled into the hills which provide anchorage to the dam through its buttresses. Although the extension works on the North Bank started around the year 2008, the time lag of about 2 years between the unusual dam deformation response and the earth works is normal, although the time lag now seems to have reduced from 2 years to few days or near-real time with increased earth works on the South Bank.

The exact point of application of this unexpected axial landslide load, its magnitude and its behaviour on the South Bank is not fully known yet. In order to stop or minimize this force on the South Bank, there is need to carry out a detailed geological study on the South Bank, while monitoring using an improved geodetic monitoring method proposed in this study. This approach to problem solving is true to the adage, "you can't fight what you don't know or have never seen".

This situation (rapid dam deformation response to earth works) requires real-time deformation monitoring to ensure safety through an early alarm system.

The fact that the deformation monitoring system has not been improved on the Kariba Dam implies there is a possibility that deformation magnitudes have been ill-detected, hence under-reported by ZRA Engineers. This is compounded by the fact that the Geodetic Networks used as reference during deformation measurements are not stable, owing to the South Bank poor geology. Thus, the observed deformations are possibly just relative motions between control points in the Geodetic Networks and not the real motions of the dam as would be observed from the stationery or stable control points.

The possible deformation ill-detection of the Kariba Dam can only be solved by first upgrading the deformation monitoring system for which monitoring network is dynamic and self-adjusting, as and when control points drift. The self-adjusting mechanism in an automated monitoring system removes errors which may otherwise be passed on in the results and cause wrong interpretation.

The optimism expressed by the ZRA dam Engineers about the safety of the Kariba Dam might be due to ill-detection of the dam deformations using traditional methods. Hence, lack of monitoring system upgrade is one of the dangers likely to cause failure of the Kariba Dam because major problems may remain undetected.

One may wonder as to why this study seemed to have gone into geological aspects when it was about whether or not the GNSS-based geodetic deformation monitoring system could work effectively for the Kariba Dam.

The deformation monitoring systems are designed to monitor the stability of dams and their surroundings. The monitoring of the surroundings is important because the control networks, which are used as reference, are anchored on the surrounding geology of a monitored object, and must remain stable in order to obtain reliable results from a monitoring system.

Therefore, designing a monitoring system for a particular dam requires knowledge of the geodynamics around the dam. For this reason, a bit of geological knowledge and the associated slope movements were a vital input into the design of the new monitoring system for the Kariba Dam. It was from the geodynamics analysis of the Kariba Dam surrounding areas that the traditional methods currently employed in the Kariba Dam geodetic monitoring were found unsuitable; the use of the Total Station alone on such an unstable control networks were prone to positional errors. This necessitated a GNSS-based system which integrates Total Stations to maintain the required accuracy, but without positional errors inherent in the results, due to its self-adjusting techniques when control points drift.

Therefore, the Prism Array Network (PAN) and the automated GNSS-based deformation monitoring system designed in this study took into account the south bank geology, the dam deformation pattern and the size of the area to be monitored. Once installed, it is designed to report accurately on South Bank slope instabilities and dam deformations in response to these slope instabilities, especially after the ambitious hydropower extensions on both the North and South Bank.

The proposed monitoring system in this study is effective and has been installed on a lot of dams around the world. However, only two case studies (Steenbras in South Africa and Sermo dam in Indonesia) were referred to in this particular study.

The Kariba Dam system upgrade will see dam managers, as classified users, having accurate dam deformation results which are a vital input into the timely planning and mitigation measures in case of a disaster. In addition to receiving accurate information, the classified users will also be able to monitor the dam deformations via a website and text messages at the comfort of their offices or their homes, any time of the day or night.

Given the new dam deformation patterns, the nature of the geology of the South Bank and how the traditional monitoring methods fail under the unstable geology around the dam, the deformation monitoring system upgrade for the Kariba Dam is worth investing in, because the benefits outweigh the possible losses which may otherwise result from a dam failure.

7.0 CONCLUSION AND RECOMMENDATIONS

7.1 Conclusion

The objectives of this study were all attained; with the new factors proved to have effects on the dam deformation. The results show that the south bank is deforming more than the north bank due to the poor geology of the south bank which triggered the landslide. The landslide is the major factor responsible for all the unusual deformations including the vertical and horizontal rotations at the Kariba Dam.

The following are the new discoveries from this research:

- Vertical rotation or torque of both south and north bank triggered by the anchor cables in the south bank, as main cause to the unusual deformations detected both on control pillars and the dam arch
- Deformation pattern of the trigonometric controls linked to the rotational and translational deformations of both the dam and the surrounding areas.

The south bank landslide, mistaken for a pure northward translation by ZRA and other consultants, is mainly associated with vertical and horizontal clockwise (southward) rotations possibly caused by south bank anchor bars and landslide (see drawing 6 in the Appendix C), although this finding needs a further geological study to explain extensively what might have triggered this rotation tendency, and how deep it may be.

This new revelation has been made possible through the visualization techniques employed in the study. Before this finding, most of the new deformations were viewed as puzzles, especially the southward movement of SB2, but not anymore because this research has given a frame-work within which such deformations may be understood or explained.

The findings in this research justify the Integration of GNSS in the deformation of the Kariba Dam through co-location techniques because of the poor geology which renders the geodetic network inaccurate owing to errors resulting from the drifting control points. Using a Total Station alone in this environment does not remove errors in the deformation results, despite its accuracy. Thus, a GNSS was justified as it is the only survey instrument which works out the positional errors in real-time and automatically between measurement rounds. This way, the dam deformations can be fully detected and reported.

The upgrade of the Kariba Dam deformation monitoring system will increase knowledge about the mechanical behaviour of the dam under different conditions through automated, real-time and detailed measurements which reflect the actual deformations more.

The upgrade is recommended because the traditional method currently employed yield erroneous, and in some cases contradicting results owing to the poor geology of the ground which anchors the control points. The upgrade comes with an alert issuance mechanism which will be used as a tool in timely mitigation measures in case of a disaster.

It must be noted that the analysis in this study was done from the survey point of view; a detailed geological study must be conducted in order to ascertain deformations of the South Bank as the nature of the problem proved to have a geological dimension as well. Only then will the full behaviour of the landslide be known and stopped or minimized.

Therefore, this research is vital as it has formed a basis for future studies and monitoring system improvements at the Kariba Dam. Not only has this research provided a basis for future studies on the Kariba Dam, but has also provided a framework within which deformations will be explained both now and in future.

7.2 Recommendations:

Following the revelation of new deformation patterns and substantiation of the landslide as the main causative factor, the following are recommended:

- More reliable instrumentations must be installed in order to monitor and confirm the new deformations of the Kariba Dam by ZRA. Preferably a prism array and GOCA System must be installed as soon as possible because new deformations are not seasonal but random

- The vertical and horizontal clockwise rotations of the south and north bank must be further researched on in order to know more about the geodynamics responsible for the strange deformations at the Kariba Dam. The illustration in drawing 6 in Appendix C is a postulation of the causes of the deformations based on the analysis of few points around the Kariba Dam. Thus geological studies must be conducted as well.

- Control points K202 on the North Bank and K204 of the South Bank arch ends showed more buckling tendencies than those of central arch P434 and CD2 (see drawings 1 in the Appendix C);this may be a sign of the dam arch ends breaking outwards and hence must be observed closely

- A study on IBIS-FL GeoRadar Scanner System must be conducted by the University of Zambia (UNZA), or any other researchers for purposes of addressing the landslide problems in full

- An assessment of the landmass-void safe ratios of the south and north banks must be carried out so that the hills do not end up housing more tunnels than they can bear by ZRA geologists or any other consulting experts

- The possibility of adding more anchor cables must be considered by ZRA to stop the vertical rotation tendency of the south abutment. Until new anchor cables are added, caution must be exercised not to over re-tension the toggles bars as this may cause more rotational movements

- Even before the monitoring system is upgraded, the dam geodetic monitoring frequency must be increased per year by ZRA to more than two times
- The actual network for the proposed GOCA system was not designed in this research. This must be done upon implementation of the system based on the manufacturer's specifications on distances and accuracies of the preferred instruments
- It is not yet clear how the translational and rotational deformations will affect the hydropower tunnels under the hills where sinking tendencies have been detected; the hydropower tunnels of both south and north bank could be experiencing stresses and strains from the rotational tendencies of the hills and the dam. Therefore, geotechnical monitoring of the tunnels should be intensified so as to know exactly the depth of these deformations, and ensure safety of the hydropower channels.

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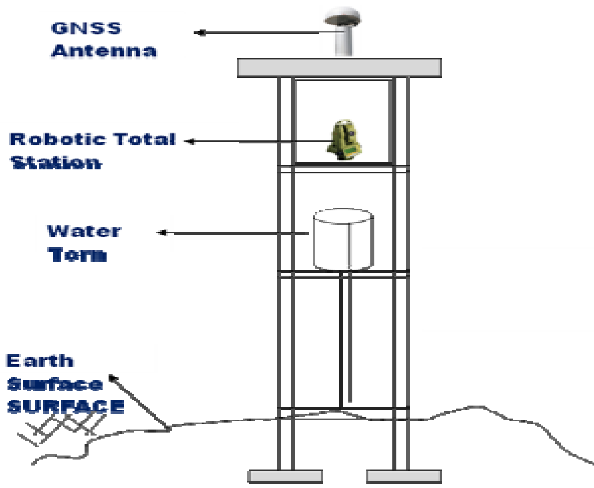
9 APPENDICES

Appendix A: Sermo dam GOCA system photos

Base station GNSS CORS at Sermo Dam, Indonesia



(a) (b)

(c)

Figures above; (a) Design, (b) first tower for RTS and GNSS CORS at SRM1 station and (c) Box for GNSS CORS at SRM1 station

GNSS CORS receiver type: Leica GRX1200+ and antenna CHOCK RING (GNSS Antenna with Dome, geodetic type)

Prism installation at Sermo Dam

(a) (b)

GNSS CORS at Back sight station

(a) (b) (c)

Figures above: GNSS CORS with single Prism as back sight (a) and (b); (c) box for GNSS CORS receiver at SRM2 station

GNSS receiver types: Javad antenna GrAntG3T with Cone

Sensors of Tilt, Zoom digital IP Camera (CCTV)

(a) CCTV presentation of situation

(b) (c)

Figures (b) and (c); Automatic Water Level Recording (AWLR)

Appendix B:

Control-Point Coordinates & Displacements

Appendix C:

Drawings derived from Appendix B data