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ASSESSMENT AND MONITORING OF POINTE DU BOIS G.S. POWERHOUSE

David MacMillan and Gordon McPhail
Principal and Senior Hydrotechnical Engineer
KGS Group, Winnipeg, Manitoba

ABSTRACT

The ongoing assessment, analysis, monitoring and management of Winnipeg Hydro's 72 MW Pointe du Bois powerhouse are described. Commissioned in 1911, concerns had been expressed since the 1940's over cracking and apparent movements of all 16 units in the 145 m long powerhouse. On the basis of Finite Element (FEM) modelling, strain tests, and analysis of concrete core samples, in 1992 KGS Group confirmed that the powerhouse had undergone considerable alkali-aggregate reaction and was continuing to do so. The varying direction and rates of movements due to concrete growth makes operation and maintenance of the existing horizontal double Francis units difficult. Detailed analysis and monitoring work commenced in 1994 and is continuing and includes: overcoring and reinforcing strain testing, 3D Finite Element(FEM)analyses, real time continuous monitoring of extensometers and inverted pendulums, along with periodic monitoring of pendulums, tape extensometer pins, and crack monitors. The assessment results are being used to identify and implement immediate repair measures such as shaft re-alignment as well as life extension measures. The results were used during the rehabilitation of Unit No.1, where the existing 3.5 MW Francis unit was replaced with a new 8.5 MW Sulzer Straflo turbine constructed on bedrock within the former drafttube. Winnipeg Hydro is presently evaluating a study that examined redevelopment of the site versus rehabilitation. The study recommended rehabilitating the plant from 72 MW up to 130 MW by replacing up to all 15 existing units with additional Straflo turbines.

Keywords: hydroelectric, powerhouse, alkali-aggregate reaction, rehabilitation, monitoring

INTRODUCTION

Winnipeg Hydro owns and operates two hydroelectric plants on the Winnipeg River in southeastern Manitoba. The most upstream of these, Pointe du Bois, is located about 30 km downstream of the Ontario border and about 160 km northeast of Winnipeg. Units 1 to 7 of Pointe du Bois were commissioned in 1911, with the remaining Units 8 to 16 developed by 1926. Upon completion, the plant included 16 double horizontal shaft Francis turbines ranging in capacity from 3.5 to 5.8 MW, with a plant capacity of 72 MW. In 1999, one of the existing 3.5 MW turbine generators was replaced with a new 8.5 MW Sulzer Straflo unit.

The Pointe du Bois generating station consists of: 1) a 150 meter long powerhouse, 2) 500 meters of gravity dam that form the immediate forebay, 3) 380 meters of gated spillway and 4) a 180 meter long rockfill dam. Figure 1 presents the typical cross section of the powerhouse through one of the original double Francis turbine units while Figure 2 presents the cross-section of Unit No.1 after the existing unit was replaced with a Straflo turbine.

Concern with cracking and movements within the powerhouse date back to before the 1940's. Reports completed in the 1950's and 1980's document cracking of the gate room floor, cracking of generator room crane rail supports, movement of the superstructure, cracking and leakage from the downstream turbine pit walls into the apron gallery and generator room, and misalignment and breakage of the turbine and generator shafts. Thermal forces, inadequate reinforcing, foundation problems and poor construction joint treatment were postulated as the cause of the structural distress observed.

During the 1980's, cracking along the generator room cranerail was repaired by post-tensioning while leakage into the generator room was reduced by injection of urethane grout. These repairs were performed without identifying the probable cause. Many of the problem areas identified in the 1950's and 1980's reports continue to be of concern today and work is now underway to address them.

As one aspect of their dam safety monitoring, in 1990 Winnipeg Hydro implemented a survey of the rails of the overhead crane above the generators as well as a crack mapping program. The crane rail surveys indicated that the upstream rail (which is located on the downstream face of the turbine pit) was arched up to 65 mm above its original elevation and translated 30 mm downstream from vertical. These results in conjunction with the observed distress throughout the powerhouse resulted in Winnipeg Hydro initiating a study to determine the probable cause of the movements and distress.

Presented briefly below is an overview of the original diagnostic work performed in 1991/92 that led to the confirmation of the cause of the distress as concrete growth due to AAR. This early work has been presented in detail previously in Langdon and McPhail [1994]. The overview is followed by a more detailed discussion on the work performed since 1994 such as the modeling, monitoring and remediation work that continues to date on the project, which is the main focus of this paper.

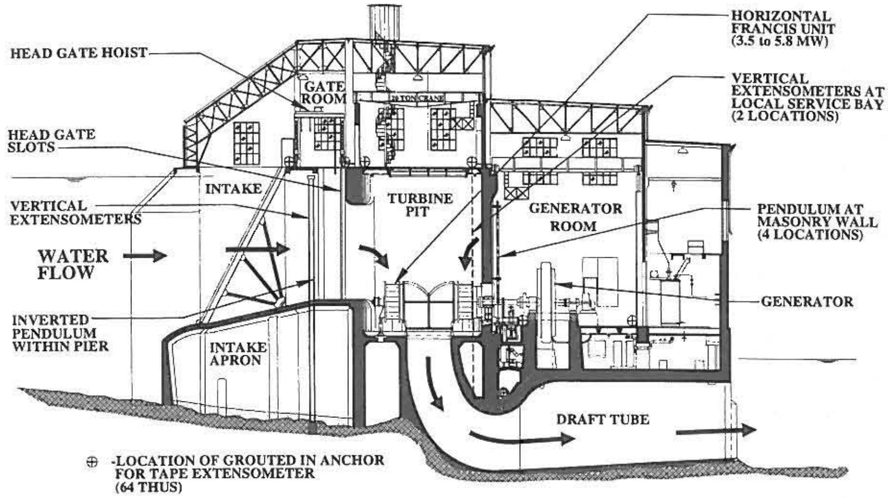

Fig. 1: Cross-section through existing unit with AAR monitoring system

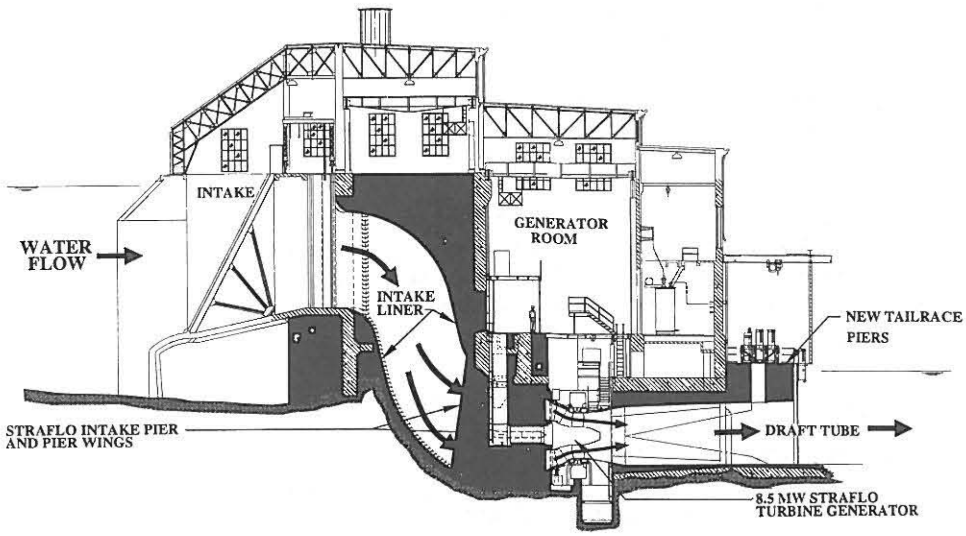

Fig. 2: Cross-secton through "Straflo" unit no. 1

1991/92 WORK AND RESULTS

In 1991, KGS Group were retained to perform a condition assessment, preliminary finite element analysis as well as testing of concrete cores of the powerhouse. KGS Group conducted a visual inspection and crack mapped the areas inspected. During the inspection, the following observations were noted:

- Throughout the powerhouse, cracks of up to 12 mm$\pm$ width run transversely and in the upstream-downstream direction.
- The upstream section of the generator room wall (the downstream wall of the turbine pits) has moved transversely (downstream) and upwards up to 30 to 40 mm.
- Near the top of the stairs to the intake, steel beams anchored in the turbine pit walls have moved upward 25 mm away from the brick facing founded on the generator floor below.
- At the west abutment, the powerhouse end wall has moved outward relative to the gravity dam wingwall.
- Portions of the base of some intake apron piers have lifted up to 6 mm off the supporting bedrock.
- The draft tube roof has separated at construction joints and allows spray in during operation.

The overall stability of the powerhouse was reviewed by means of traditional stability rigid body analysis. The analysis found the powerhouse to be stable under normal operating and dewatered conditions.

The powerhouse was analyzed using a finite element model (ANSYS). Given the investigative nature of the 1991/92 work, a typical powerhouse unit was modeled as a 2-dimensional linear elastic structure with elements of varying thickness to simulate the 3-dimensional concrete shape. This relatively simple FEM model allowed for a good approximation of the upstream-downstream effects of various load cases while reducing the time required for model formulation and solution. The model was used to examine stress levels in critical portions of the powerhouse cross-section and at the powerhouse/foundation interface.

Loading cases reviewed with the simple FEM model used in 1991 included: 1) self-weight; 2) hydrostatic loads under normal operating conditions; 3) thermal stresses; 4) various hypothetical foundation anomalies; and 5) various degrees of concrete expansion in the wetted portions due to alkali-aggregate reaction, ranging from 500 to 1000 μstrain (0.05 % to 0.1 %).

The finite element analysis for normal operating conditions found that stress levels were considerably less than required for cracking of the concrete. Similar results were obtained for both thermal effects as well as foundation anomalies. In addition, the predicted tension zones did not correlate well to the observed cracking locations nor the observed dimensional changes. The tensile stress patterns and overall pattern of deformations from AAR expansion, however, were found to closely correlate with the locations and relative magnitude of the observed cracking. In comparison to the actual magnitudes of cracking and movements observed, however, the results appear to indicate that the total growth to date exceeded the 0.1% postulated.

In the 1991/92 work, concrete expansion due to alkali-aggregate reaction was analyzed using two simple load cases, one modeling 0.05% cumulative expansion, the other modeling 0.1% cumulative expansion. During this preliminary study, the amount of expansion was assumed to be independent of stress. The more detailed modeling performed since 1991 uses stress and time dependent strain and growth rates to better represent the nature of AAR induced concrete growth along with detailed 3D models of the structure.

Along with visual assessment of the structure and the finite element analysis, a total of ten 150 mm ∅ concrete cores were taken from both Phase 1 (Units 1-7) and Phase 2 (Units 8-16) of the powerhouse. The examination and petrographic evaluation of the core samples was performed by Mr. Paddy Grattan-Bellew of NRC and proceeded concurrently with the FEM analysis. On the basis of visual, chemical, scanning electron microscope, and petrographic analysis, it was concluded that a slow alkali-silica reaction (ASR) was taking place in the concrete [NRC 1992]. The reactive material in the aggregate appeared to be particles of microcrystalline quartz and chert materials. Reaction rims were observed around some aggregate particles with ASR gel present in air voids. The alkali content observed was much higher than expected (3.7 to 5.3 kg/m^3 Na_20 equivalent) and was sufficient for AAR to continue. The aggregate was processed from rock blasted from the surrounding Canadian Shield. Pointe du Bois is one of three hydro plants in the Winnipeg River basin to have been tested and confirmed as being affected by AAR.

Using a relative damage rating index, the cores from the turbine pits were rated by NRC to be damaged to a degree similar to, or in excess of Beauharnois G.S. in Quebec, a plant which is known to be undergoing significant AAR effects [NRC, 1992]. On the basis of the cores, NRC concluded that a cumulative concrete expansion of at least 0.1% had taken place. As anticipated, the damage rating was greatest in the cores from the fully saturated portions such as the turbine pit walls.

Although not examined directly in the FEM study, it was postulated that high compression forces occur in the transverse direction of the powerhouse due to confinement of the expansion by adjacent units. There are no contraction joints between units. These high transverse stresses have been confirmed in the testing and assessment work that followed.

Upon completion of the concrete testing and FEM analysis, in 1992 KGS recommended a program of additional monitoring and investigations to be completed at Pointe du Bois powerhouse. These included:

- measure insitu strains and stresses in rebar, concrete and superstructure using overcoring and/or reinforcing tests;
- install crack monitoring gauges;
- detailed monitoring of the powerhouse by extensometer, pendulums and inclinometer to determine overall movements;
- further concrete testing and investigations within the powerhouse and on other structures;
- additional, more detailed FEM analysis in the transverse direction;

- quantification of the actual growth strain in several locations in the powerhouse followed by detailed three dimensional FEM analysis;
- implementation of any necessary life extension measures.

WORK SINCE 1992 TO DATE

The monitoring and test program recommended in 1992 has been implemented in a phased approach with the aspects examined and their extent being shaped by the results of the work performed. The work commenced in 1992 with direct strain measurements and is continuing. To date, monitoring and assessment measures that Winnipeg Hydro and KGS have performed or are presently implementing include:

- periodic generator crane rail surveys;
- installation of visual crack monitoring gauges;
- installation and monitoring of tape extensometer pins on a grid network throughout and at all levels of the powerhouse;
- direct strain measurements of reinforcing embedded in concrete;
- direct strain measurement of concrete using overcoring;
- installation of inverted pendulums in the intake pier noses;
- installation of extensometers in the intake pier noses;
- installation of real time data acquisition for the extensometers and inverted pendulums;
- installation of pendulums on the upstream face of the generator room;
- FEM analysis (using Ansys) of the powerhouse in the tranverse direction as well as three dimensional analysis of an existing unit along with the entire powerhouse;
- underwater drilling, recovery and assessment of cores taken from several of the draft tubes (which can not be dewatered).

The work and progress to date on each of the aspects is summarized below.

The generator crane rail survey results from 1990 on indicate that the turbine pits have expanded vertically in the past and continue to do so. The upstream wall is generally saturated, and thus appears to be undergoing significant AAR expansion, whereas the downstream wall is largely dry. A maximum difference of approximately 75 mm was measured near the middle of the PHASE 1 (east section) powerhouse near Unit 5.

Crack gauges with $\pm$ 0.5 mm reading tolerance have been located throughout the powerhouse and are being recorded periodically by plant staff. The utility of the data is limited in the short term, but will increase over time as the period of record increases and allows improved trending. The maximum movement observed to date at one crack has been about 1.5 mm, with considerable thermal (i.e. seasonal) variation.

The tape extensometer readings can detect dimensional variations to + 0.064 mm. The readings have been performed monthly or quarterly for the last few years, and indicate that significant movements (+.80 mm) and trends are continuing.

Insitu strain measurements of reinforcing bars in a number of selected units have been conducted by personnel from the nearby Atomic Energy of Canada Ltd (AECL) center at Pinawa. In the units tested to date, deformed square reinforcing bars were exposed, two strain gauges were installed on the bars, then the bar was cut and the resulting change in strain recorded. The locations and bars tested were: 1) main 40 mm vertical pier reinforcing, 2) horizontal reinforcing just above the turbine shaft wall casing; and 3) horizontal reinforcing in the face of the pier. The locations were selected to be representative of overall strains while minimizing the "background" stress produced within the dewatered unit by the adjacent operating units.

In addition to reinforcing strain tests, the FEM models were used to locate target locations for strain verification using the overcoring technique, whereby the 2 or 3D strains on a concrete core are determined by removing the core form the surrounding concrete and observing the resulting strain changes. This work has been performed by AECL as well as Ontario Hydro Technologies. The strains were measured using both USBM as well as door stopper gauges, which are easier to install but which at Pointe du Bois proved less effective and reliable.

The overall pattern and magnitude of strains observed have been consistent with that anticipated from the FEM analysis. The data from the testing in turn is used for model calibration and verification of the FEM models developed. In general terms, for the units away from the abutment ends (ie. Units 2 to 15), the observed strains in the main vertical reinforcing in the piers have been in the order of 0.1% to 0.15% (1000 to 1500 microstrain) tension in the steel reinforcing with calculated steel stresses of 200 to 280 MPa. The tested yield strength of the steel reinforcing is approximately 280 MPa. The horizontal pier reinforcing is in quite low tension of 0.02%, while the horizontal reinforcing in the downstream wall of the turbine pit is in quite high compression of 0.05 to 0.06%. Based on the overcoring results and field verified modulus values, the horizontal stress in the concrete in the downstream turbine wall is up to 14 MPa, while the compressive strength (fc') of the cores is typically 25 to 45 MPa.

To visualize the level of strain observed, at the time of testing Unit No. 5, the cast iron wall casing embedded in the downstream wall of the turbine pit (see Figure 2) for the head cover was cracked and had been leaking for several years. As a steel reinforcing bar that had been exposed above the embedded casing was being cut, the bar compressed so much that the blade was pinched off and the bar had to be recut before a stress measurement could be taken. Under normal conditions with no concrete growth, this bar should have been in zero to low tension rather than the high compression observed. In 1994, this wall casing was replaced.

At the outer units, Units No. 1 and 16, the lateral strain and compression in the downstream wall appears to drop off to near zero, however, the strain in the main vertical reinforcing remains high, 0.08 to 0.10 % tension. It would appear that displacement of the end walls is reducing the apparent lateral confinement of the AAR expansion, whereas towards the centre of the powerhouse the adjacent units effectively confine the lateral expansion. This confinement induces high lateral compression forces in the concrete (+14 MPa) and reinforcing of the downstream turbine pit wall, as well as in any embedded components such as the wall casing support for the headcover. Expansion of the end walls of the powerhouse relative to the adjacent gravity dam bulkheads is consistent with observations at the west abutment but has not been observed at the east end of the powerhouse near Unit No. 1.

The strain measurements performed to date indicate that the stresses are generally higher in the older portion of the powerhouse (ie. Units 1–7). These findings are consistent with movements recorded by crane rail surveys and tape extensometer readings taken to date.

In 1998, Winnipeg Hydro completed the installation of six extensometers and four inverted pendulums, the majority of which are located in the hollow noses of the intake piers. The extensometers are equipped with continuous LDVT readout heads that are continuously sampled and filtered. A single re-locatable readout device is used for the inverted pendulums, with the readout sampled continuously but at only one location at a time. Periodically the readout device is cycled through each inverted pendulum and the results stored for long term trending. The data is collected and downloaded daily, but has the capability to signal an alarm if predefined values are exceeded. Conventional pendulums are also being installed on the downstream face of the turbine pit walls that will be periodically monitored. Results to date confirm significant movement due to thermal cycling but a longer sampling period is required to confirm long term growth trends.

Data from the monitoring and tests are being used to verify the 3D FEM models developed. These include detailed transverse and sectional models, as well as 3D models of a unit as well as the entire 16 unit powerhouse. The models are developed and analyzed using the non-linear Ansys program running on a PC platform.

Further monitoring and testing are required to confirm the actual rate of AAR expansion. The cumulative vertical expansion exceeds 0.10% and may be as high as 0.25%. Due to the limited period of monitoring, the rate of growth is presently not defined accurately but will be with increased time and data.

REMEDIAL MEASURES

In response to the strain measurements and analysis, as well as operational problems encountered with aged components, a number of life extension or remedial measures have been identified and implemented since 1992, with more anticipated in the future. To date these include:

- Diamond wire sawing and then re-embedding head cover wall casings to allow the turbine/generator shafts and wicket gates to be re-aligned.
- Replacing cracked head cover wall casings.
- Re-boring and machining the head covers to re-align the wicket gates.
- Re-positioning and anchoring turbine bases down to the draft tubes.
- Re-alignment of turbine and generator shafts as well as wicket gates.
- Modification and redesign of the intake gates to create additional clearance within the guides to ensure gate closure under emergency conditions.
- Crack repair and stabilization where structural concerns exist.

- Stabilization of the intake piers and turbine pit walls of Unit 1 using post-tensioned DCPstrand anchors. The remaining piers are to be anchored in the 2000/2001.

As additional studies are performed and as the amount of monitoring data increases, these remedial measures will be extended and augmented as necessary to ensure continued safe and cost effective operation of the plant.

For horizontal units such as at Pointe du Bois, maintaining alignment over the five bearing points (3 turbine, 2 generator) while the supports undergo AAR movements can be difficult. A real time monitoring system on shafts and bearings is presently being examined that could reduce the potential for unscheduled outages due to bearing failures caused by excessive shaft misalignment. The procedure used now is to check the alignment during regular scheduled maintenance outages.

The modeling and monitoring information gathered to date was used extensively during the rehabilitation of the Pointe du Bois Turbine Unit No. 1. Due to aged mechanical components and continued operational difficulties associated with AAR induced movements, work commenced in 1997 on replacing the 87 year old double Francis 3.5 MW turbine/generator. The replacement unit was selected to be a new 8.5 MW Sulzer Straflo axial flow turbine/generator. This type of unit maximizes the turbine flow capacity for a given size of water passage by placing the generator poles directly around the rim of a propeller turbine (see Figure 2).

To limit the potential impacts on the new turbine/generator from the effects of continued AAR in the existing concrete, the new unit is founded directly on bedrock and the unit is supported from a post tensioned steel and concrete pier separated from the surrounding concrete. The upstream water passage between the pier and the intake is a steel liner while the new draft tube is reinforced concrete anchored to the underlying bedrock. The new concrete used low alkali cement and aggregate that had been tested for reactivity in accordance with CSA.

The work commenced in 1997/98 with underwater concrete placement and construction of the new tailrace piers necessary for dewatering the existing drafttube (see Figure 1 and 2). Demolition work in the draft tube commenced in 1998 with diamond wire saw cutting of the highly stressed draft tube elbow while the surrounding concrete and units were monitored. The saw cut took three attempts after the wire was pinched twice following cutting shutdowns due to equipment problems. Once the wire slot cut was completed, slot closure was monitored and closed up to 8 mm locally over three days. Demolition then proceeded in a conventional manner using a backhoe mounted concrete breaker as well as controlled blasting. First power of the Straflo unit was in November of 1999.

Based upon the technical and economic success of the Unit No. 1 replacement, Winnipeg Hydro is currently evaluating options for life extension of this asset. A recent study examined both redevelopment of the site with a new plant as well as rehabilitation and concluded that rehabilitation of the plant with up to 15 additional Straflo turbine units offered significantly greater benefits and reduced costs [KGS Group, 1995, 2000].

CONCLUSIONS

- After over 85 years of successful service, the mechanical and civil components of the Pointe du Bois G.S. powerhouse have exceeded the design life and are showing their age. In 1992, the ASR type of AAR induced concrete growth was identified as the principal cause of the cracking, movements and distress that had been noted in the powerhouse since the 1950's.

- Displacements throughout the Pointe du Bois powerhouse are presently continuing. Data from the recently installed monitoring system is being used to assess the movements, however, a longer period of record is necessary to define the rate of growth and its full effects.

- Operating and maintaining existing generating units in an AAR affected plant can be accomplished effectively but requires increased maintenance effort, costs, and downtime.

- The relatively slow growth rate of the ASR type of AAR at this plant allows the use of a "observe and remedy" approach, but remedial measures are being implemented and more will commence shortly as data increases and critical aspects are identified.

- Remedial measures are currently being implemented for extending the life of Pointe du Bois, and measures to extend the life for an additional 75 years are being evaluated. These include rehabilitation of Unit No. 1, where a new 8.5 MW Straflo turbine unit was installed in 1999.

- The FEM analysis in combination with core testing, monitoring, in-situ strain assessment, and FEM verification performed to date has proven to be a cost effective method of assessing the condition of the structure and identifying aspects requiring monitoring, remedy, or further study.

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