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ABSTRACT

The Little Quinnesec Falls Hydroelectric Dam is located on Menominee River in Niagara, Wisconsin. It is owned and operated by Consolidated Papers, Inc. (Niagara Division). The dam generates about 10,200 kW of hydroelectric power. Under FERC re-licensing program extensive field investigations were performed. The field investigations involved coring through the gravity section of the dam and foundation rock, taking video of the boreholes using borehole camera and installing piezometers to monitor uplift pressure under the dam. The boreholes were between 8-m and 16-m in length. The interface between concrete dam and bedrock was found to be at a depth of 5-m to 8-m. The coring was performed up to 1.5 m into the bedrock. The bedrock was found to be Gneiss, light gray, coarse grained and massive. Complete water loss was noted in most of the boreholes through the dam. Dye tests indicated possible link between the boreholes and the upstream pool. The borehole camera also detected fine cracks and presence of timber at the interface between the concrete dam and the rock foundation. Based on these field investigations and subsequent analyses, stabilization measures were recommended. It included installation of vertical post-tensioned anchor system through the dam and the power plant. The work consisted of anchor installation, testing and stressing following successful performance testing of the anchors. Following stressing, secondary and final stage grouting was performed and the block-outs restored. This paper presents step-by-step procedure followed from field investigation to the installation of the anchors to fix the dam.

INTRODUCTION

The Little Quinnesec Falls Hydroelectric Plant and Dam are located on Menominee River in Niagara, Marinette County, Wisconsin. They are owned and operated by Wisconsin Paper Corp. (Consolidated Papers, Inc., Niagara Division). The dam generates about 10.2 MW of hydroelectric power. The location of the dam is shown in Fig. 1.

Fig. 1. Location of the dam.

The Little Quinnesec Dam, built in early '50s, consists of four gated bays, an emergency spillway and a gravity portion where the power plant is located. The gravity portion of the dam is 14 m high and 40 m in length. Figure 2 shows a view of the dam and the power plant.

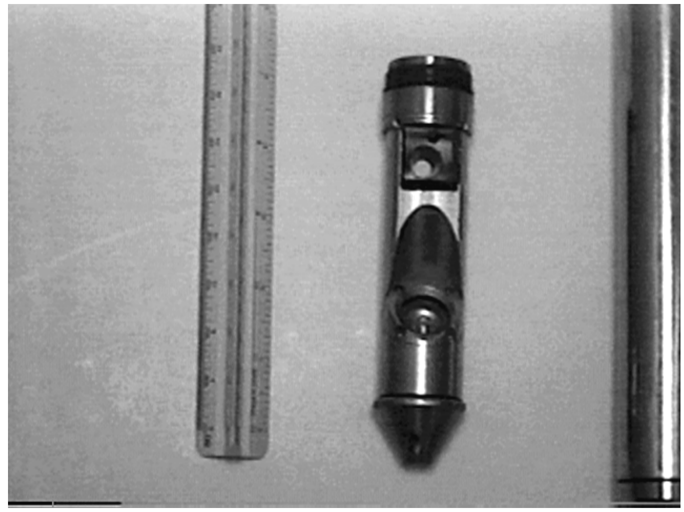
Fig. 2. A view of the dam and the power plant.

Under FERC re-licensing program extensive field investigations were performed. The field investigations and the subsequent numerical stability analyses indicated that the gravity portion of the dam did not have adequate factor of safety. Based on the results of field investigations and numerical analyses, a decision was made to anchor the gravity portion of the dam with post-tensioned vertical anchors. Subsequently vertical anchors were designed and installed in the dam without disrupting the normal operation of the power plant. This paper presents step-by-step procedure followed from field investigations to the installation of the anchors to fix the dam.

FIELD INVESTIGATION PROGRAM

Under FERC re-licensing program extensive field investigations were performed. The field investigation program was contracted to STS Company of Greenbay, WI. The program included coring through the gravity section of the dam and foundation rock, recovery of samples from the concrete portion of the dam and the foundation rock, laboratory strength tests of the core samples to determine their strengths and installation of piezometer at some of the selected boreholes to estimate uplift pressure under the dam. The boreholes, 127 mm in diameter, were between 8-m and 16-m in length. The interface between concrete dam and bedrock was found to be at a depth of 5-m to 12-m. The coring was performed up to 1.5 m into the bedrock. The bedrock was found to be Gneiss, light gray, coarse grained and massive. Loss of core at the concrete-rock interface and complete water loss were noted in most of the boreholes through the dam. Dye tests indicated possible link between the boreholes and the upstream pool. At this point a black and white borehole camera was brought in to visually inspect the boreholes. Figure 2 (b) shows a close-up view of the borehole camera and Fig. 2 (a) shows the video acquisition system of the borehole camera utilized in the present study. The borehole camera detected fine cracks and presence of timber at the interface between the concrete and the rock foundation in almost all the boreholes. Figure 3 shows a magnified view of a crack detected at the interface between the concrete dam and the foundation rock.

(a) Video acquisition system of the borehole camera.



(b) Borehole camera.

Fig. 2. Borehole camera & its assembly.

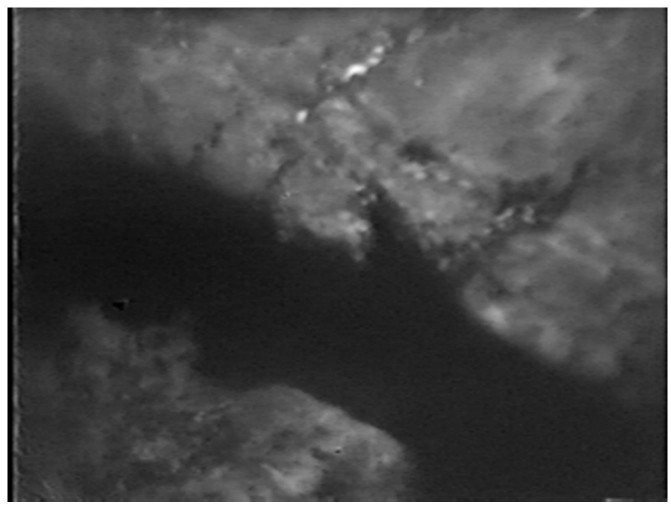


Fig.3. Magnified View of a Crack at the Interface.

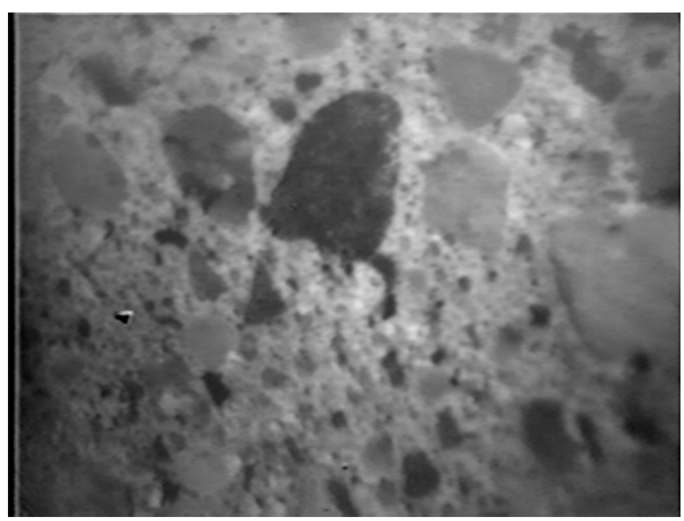


Fig. 4. A magnified view of the concrete portion of the dam.

The condition of the concrete portion was also inspected by the borehole camera. Figure 4 shows a magnified view of the concrete portion of the dam as observed from within one of the boreholes. The concrete aggregates were found to be of 25.4 mm to 152.4 mm in size. No prominent sign of deterioration and aging of the concrete could be noticed from the video. Based on these field investigations and subsequent stability analyses, stabilization measures were recommended. It included installation of vertical post-tensioned anchor system through the gravity portion of the dam and the power plant.

DETERMINATION OF FOUNDATION STRENGTH

The determination of the strength of the bedrock was of paramount importance and was critical in the assessment of overturning and sliding stability of the dam and the power plant during and after the installation of the post-tensioned rock anchors. The residual strength of the bedrock was determined from the laboratory tests and was used to check the stability of the dam. The residual strength of the bedrock was represented by a friction angle of 30 degrees and no cohesion. This value was the lower bound of a large number of laboratory tests performed on the cored samples. The bond zone for the rock anchors was determined to be located within a competent gneiss below the existing foundation. Based on pull out tests within this rock, an average bond strength of 482.6 kN/m^2 was used for design purposes. This resulted in the use of 6 to 7.5-m bond zone for the vertical anchors.

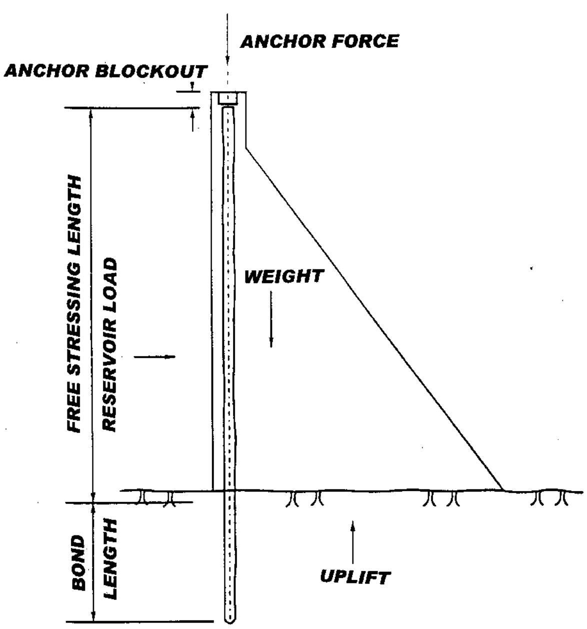


Fig. 5. Anchorage condition.

BASIC PARAMETERS OF ROCK ANCHORS

A total number of 12 vertical anchors were installed in the gravity dam and power plant portion. Out of these 12 anchors, 6 anchors were installed in the left side monolith and another 6 of them were installed in the right side monolith of the dam. The anchors were designed using the guidelines provided by PTI [1990], Williams [1992] and DSI [1990]. Figure 5 depicts the condition for which the anchors were designed. The anchors in

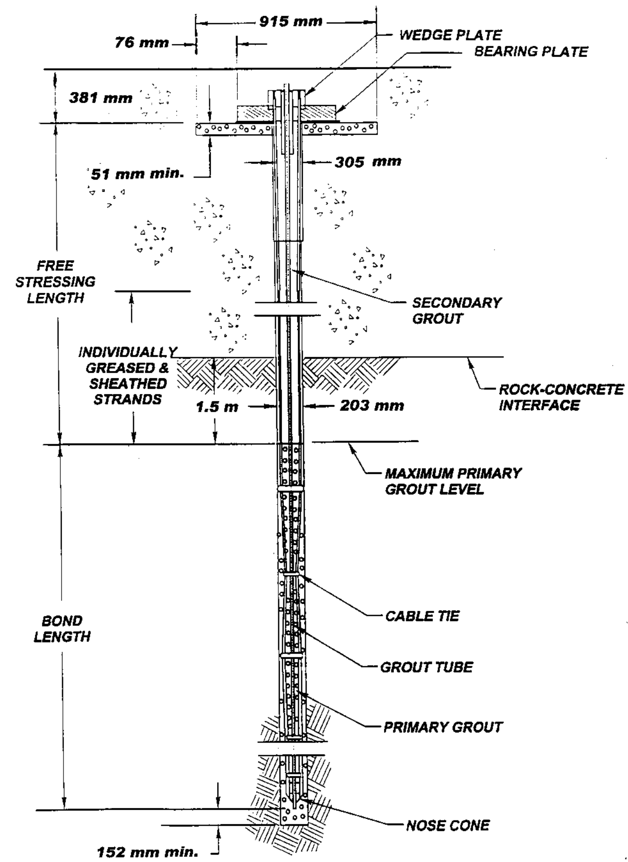


Fig. 6. Anchor details.

the left side monolith was designed for 2189 kN. The transfer (lock-off) load for these anchors was 2553 kN and the proof test load was 2918 kN. The anchors in the right side monolith were designed for 2971 kN. The transfer (lock-off) load and the proof test load for these anchors were 3465 kN and 3963 kN, respectively. All the anchors were multi-strand anchors. The left side anchors had 14 strands per anchor. The right side anchors had 19 strands per anchor. All the strands or tendons were provided with double corrosion protection. The 15.24 mm diameter strands were sheathed in a greased plastic tubing along its free length to debond the stressing length during grouting. Figure 6 shows anchor detail schematically. Several tensile tests were performed on the individual strands. Based on these tests, the average breaking load and the load at 1% strain level were 266 kN and 248.6 kN, respectively.

INSTALLATION OF ROCK ANCHORS

The installation work consisted of anchor installation, testing and stressing following successful performance testing of the anchors. Following stressing, secondary and final stage grouting was performed and the block-outs restored. The fore bay water level was constant at about EL. 287.7 m during this whole period. Layne Christensen Company was the contractor for the rock anchor installation work. Layne was performing the anchor hole preparation, tendon installation and testing. Contech Systems Limited was the tendon supplier and was present during performance anchor installation and testing. The installation of the anchors in the power plant was especially critical. The high voltage electrical lines and other utilities had to be avoided. Additional problem was lack of headspace in the plant. Some of the anchors were installed through the roof of the powerhouse. Figure 7 shows the installation of anchors in the power plant.

Fig. 7. Installation of anchor through roof of the powerhouse.

Initially a 305-mm hole was cored within each monolith to assess bond zone before drilling of the remaining anchor holes using down-the-hole hammer equipment. This allowed confirmation of anchor lengths that were pre-assembled at the factory. After being drilled, all boreholes were pregrouted and then redrilled to prevent water seepage from washing out or diluting the grout after the anchors were installed and before the final grout could set. A Type I/II cement manufactured by LaFarge Corporation in Alpena, Michigan was used for the consolidation and primary grouting of the anchor holes. A plasticizer, Sikament 300, was used with the consolidation and primary grouting. Non-shrink Sealtight 588 Precision Grout was used for the topping grout.

<u>Tendon Installation and Primary Grouting</u>

Tendons for all the anchors were installed by crane. The tendons were inspected for any damage as they were being installed. All damaged spots were repaired with a epoxy repair kit recommended by the tendon manufacturer (see Fig. 8). Following installation of the tendons in all the drill holes, all the anchors were primary grouted.

Fig. 8. Epoxy coating damaged by apparent abrasion.

<u>Tendon Testing</u>

Performance tests were conducted on two of the anchors – one located on the left monolith and another one on the right monolith. After application of an initial alignment load (see Fig. 9), the tendons were stressed up to 133% of the design load in cycles. Figures 10 and 11 show the stressing chair, restressable head and the jack used for stressing the anchors.

Fig. 9. Stressing to alignment load.

Figure 12 shows the stressing arrangement for one of the anchors. Figure 13 shows the arrangements for the proof test. A creep test was then performed. Both performance anchors passed the creep test. The creep tests did not show any foundation problems in maintaining load over the anchor design life. The total elastic movements on both anchors were within acceptable

Fig. 10. Stressing setup with pressure gage on the right.

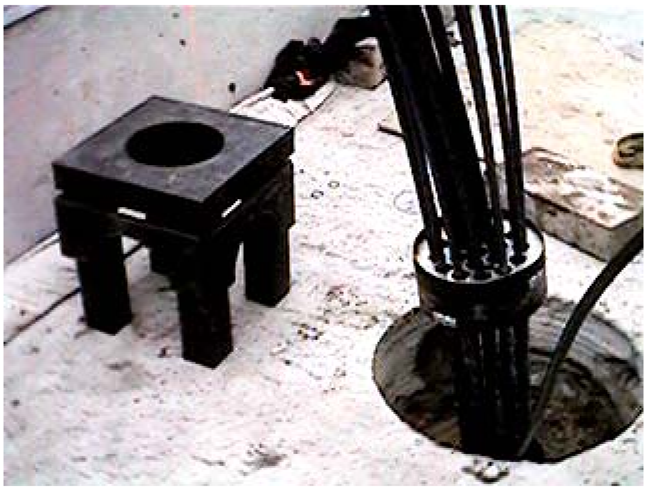


Fig. 11. Performance anchor with restressable head before stressing. Note stressing chair in the background.

Fig. 12. Stressing jack on anchor.

Fig. 13. Dial gage set up for a proof test.

Fig. 14. Close-up view of semi-circular shims under an anchor head.

tolerances. Immediate lift off tests showed a need for shimming on both anchors. A shim is a donut shaped steel disk that is split diametrically to allow insertion under the anchor head to avoid transfer load loss. Transfer load loss happens when the jack grabs the strands to apply the desired stressing load and then let the load off. At that time some of the applied loads are lost as the strands get locked into the anchor head by the wedges in the anchor head holes, and this loss of load is compensated with a shim. The anchor on the right monolith was shimmed with two 9.5 mm plates and one 6.35 mm plate was used for the anchor on the left monolith. After shimming, the lock-off load was within acceptable tolerances. Figure 14 shows a close up view of the shim under the anchor head. An additional lift off test was performed about 1 day after lock-off, and showed no loss of load. A final lift-off test was performed about nine days after lock-off and again, no discernible loss of load were seen. The performance anchors meet all specifications and were accepted. Figures 15 through 21 show some of the details of the anchor installations.

Fig. 15. Tendon being set down on blocking using rod.

Fig. 16. Close-up of wedges in anchor head after lock-off.

Fig. 17. Restressable head after lock-off.

Fig. 18. A view of wedge seating immediately after mono strand alignment loading and before removal of restrainer plate on top of stressing jack head.

Fig. 19. Close-up view beneath restrainer plate.

Fig. 20. Tendon tips after epoxy repair.

Fig. 21. Side view of epoxy repaired tendons after cutoff.

Secondary Grouting

All the anchors were secondary grouted after the completion of all stressing. Final stage grouting and block-out restoration were performed after secondary grouting to fill the remaining voids.

INSTRUMENTATION OF THE DAM

The Little Quinnesec Falls Hydroelectric dam and power plant have survey alignment pins and tilt plates installed previously to measure the horizontal and vertical movements, and rotation of the dam. During the present field investigations a number of piezometers were installed to measure the uplift pressures under the powerhouse. The dam had no inclinometers. The movement of the power plant portion of the dam was an important consideration during installation and stressing of the anchors. All the movements were found to be within the tolerable limits. The piezometers indicated that there was no excessive hydrostatic pressure within the foundation of the dam, which might have been sealed off by the pregrouting requirements for the vertical anchors.

CONCLUSIONS

No major construction difficulties were experienced during installation and stressing of the vertical anchors. Minor problems consisted of replacement of a wedge rubber ring and the need to use drums to guide the tendons to avoid high voltage lines directly above the holes in the power plant. Secondary grouting was delayed because some of the secondary grout pipes did not pass water in a test just before grouting. The cause for this was found to be kinks in the pipe and primary grout plugging the secondary grout pipes. The decision to grout the anchors in two stages was found to be an important one. This allowed the transfer of load to the dam along the strand length instead of transferring it all to the anchor plate at the top. This procedure prevented the aged concrete dam from overstressing. The whole experience also indicated that a dedicated and experienced team is needed for successful completion of this sort of job. The previously gained knowledge in similar jobs helped the contractors make critical decisions in the field.

ACKNOWLEDGEMENT

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