FRP Retrofit of the Ring-Beam of a Nuclear Reactor Containment Structure

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Synopsis:

The Gentilly-1 nuclear power plant, in Quebec, Canada, was decommissioned in 1978. Since that time, the containment structure has been used for the storage of the moderately contaminated nuclear reactor. The enforcement of more rigorous environmental regulations, as well as economic considerations, have raised the decommissioning period from 40 to 100 years, thus severely increasing the durability requirements for the structure.

The containment structure, constructed of thick prestressed concrete, was in good condition except for the secondary concrete. The latter is a keystone for the durability of the structure because it fills the recesses and protects the terminations of the tendons against corrosion. The differential shrinkage caused cracking and debonding and, with freeze-thaw cycling over the years, the secondary concrete had to be removed and replaced. The ring-beam, at the top of the containment structure, was severely affected because the numerous tendons of the roof terminate at that level.

The retrofit of the ring-beam consisted of replacing the secondary concrete with high-quality shrinkage-compensated mortar and concrete, followed by FRP wrapping. The layout of the FRP wrap was designed to mitigate the adverse effects of the new secondary concrete shrinking-induced cracks. Most of the concrete cold joints were covered by the FRP wrap, which was anchored on the dome roof to provide an effective support.

Keywords: Concrete repair, surface-bonded FRPs, nuclear power plant, structural monitoring, fiber optic sensors.

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1. HISTORY

1.1 Decommissioning of the Reactor

The Gentilly-1 nuclear power plant, located in the province of Quebec in Canada, is a CANDU prototype system that was developed by Atomic Energy of Canada Limited (AECL) to be delivered to the Hydro-Quebec power utility. It was designed in the late 1960s and its construction was completed in 1974. The operating tests of the reactor were never deemed satisfactory for safe operation, and AECL decided to decommission it in 1978. The nuclear fuel was removed as well as all highly contaminated parts. Because the reactor vessel was too large to be removed, it was decided to keep the containment structure as a storage building for the decommissioning period of time. For technical and economical considerations, it was concluded in 1984 that returning the site to a condition of completely unrestricted access and usage was not immediately necessary. Also, as a result of more rigorous environmental regulations, the decommissioning period was increased from 40 to 100 years.

Despite the fact that Gentilly-1 is decommissioned, from the AECL and public points of view the facility must nevertheless appear to be in perfect condition. Thus, there is a stringent requirement on the overall appearance and long-term durability of the containment structure.

1.2 Deterioration of the Containment Structure

The containment structure, which houses the reactor chamber, is a thick-walled prestressed concrete structure. The 1.2-meter-thick wall and 0.6-meter-thick dome roof were designed to contain the low level of radiation of the operating reactor and to resist the Design Basis Accident (DBA) load. The construction procedure of such a prestressed structure implied that a large number of recesses for the tensioning heads of the prestressing tendons had to be filled with secondary concrete. The vertical tendons of the wall and the tendons of the dome roof are all anchored in the ring-beam, an annular concrete element 3.4 meters high and 2.1 meters thick.

The cause of the deterioration problem was the shrinking of the secondary concrete placed after the cure of the structural concrete, leading to cracking and debonding. The effects of this initial cracking were exacerbated by the poor quality of the installed materials, detailing of the reinforcing steel, and the freeze-thaw cycling. The deterioration of the secondary concrete of the ring-beam was rather rapid, and repairs were first done in 1985. In 1993 AECL was obliged to repair the ring-beam again; the secondary concrete was so deteriorated in some areas that it was necessary to remove it to prevent it from falling down. As shown in Figure 1, the appearance of the structure was very poor. In 1996, the planned solution to cover the ring-beam with steel cladding was abandoned. Due to the poor results of the past, the high costs of the repairs, and the lengthening of the decommissioning period, AECL decided to conduct an extensive study on the integrity of the containment structure to seek out the best solution.

1.3 Study, Analysis and Tests

The first step of the investigation of the containment structure consisted of a visual inspection. The AECL inspection staff found that the south side of the ring-beam, which has the greatest exposure to sunlight, suffered the most damage with extensive deterioration of the secondary concrete. Questions concerning the safety of the structure were raised since indications of corrosion and alkali-aggregate reactions were observed.

In 1998, AECL conducted a detailed assessment of the containment structure (1) and other problems were also observed, such as:

- local degradation of the structural concrete, which was thought to be accelerated through contact with the poor quality secondary concrete;
- the presence of contaminants in the structural concrete such as dust, and chips and pieces of wood;
- vertical cracks in the buttresses concentrated in the secondary concrete; and
- corrosion of the horizontal post tensioning ducts and tendons.

However, this investigation confirmed that the overall structure was safe and that the structural concrete, including the ring-beam, was in a satisfactory condition. The stresses in the concrete and the prestressing cables were measured by various tests and found to be in agreement with the expected levels. The mechanical, physical and chemical properties of the concrete were tested, leading to a confirmation of the presence of alkali-aggregate reactions that could exert stress levels comparable to those associated with the prestressing cable network if the moisture of the concrete exceeds 80%. Other properties of the structural concrete were found to be satisfactory. Finally, the corrosion observed on the post tensioning cables was attributed to the construction method employed; it was concluded that no further corrosion took place since the completion of the initial construction.

Being assured of the good structural condition of the containment structure, AECL decided to seek out a long-term solution to the problem of the secondary concrete of the ring-beam. AECL contacted various groups of specialists in concrete repair and retrofitting, including the Network of Centres of Excellence ISIS Canada (Intelligent Sensing for Innovative Structures). The remedial work, implemented in 2000-2001, consisted of replacing the secondary concrete with high-quality shrinkage-compensated mortar and concrete, followed by fiber reinforced polymer (FRP) wrapping (2).

2. INNOVATIVE LONG-TERM SOLUTION

To assure the long-term durability of the ring-beam repair, the engineering team focussed on three complementary aspects: first, the use of state-of-the-art concrete repair technologies for the replacement of the secondary concrete; secondly, the use of FRP materials to secure and protect the repairs; and finally, the implementation of a rigorous quality control program for every aspect of the repair work. The in-field application of the stringent contractual requirements has resulted in a high quality, durable repair of the ring beam.

The experience of AECL and ISIS Canada was called upon in 1999 to prepare the Technical Specifications document (3) for the repair work to be performed on the ring-beam. The experts from ISIS Canada prepared the first draft, where innovative FRP repair technologies were proposed. The engineers of AECL subsequently reviewed the document to modify it to their standards, particularly with regard to the aspects of rigorous quality control required by the nuclear industry. Finally, both groups discussed minor changes and reached a consensus on the Technical Specifications document.

2.1 Concrete Repair

The long-term durability of the repair was a critical objective since the required service life of the containment structure is being extended by 75 years. The major concerns about the durability of the repair of the ring-beam are related to the freeze-thaw cycling. Any crack filled with water will eventually damage the concrete. For that reason the shrinkage of the repair concrete mix must be minimized, and the surface water should be eliminated quickly.

The first step of the repair was the removal of the non-adherent or unsound concrete (Fig. 2). The defective areas were determined using the conventional sounding method. The perimeter of the repair area was saw-cut to obtain clean joints, and the depth was reduced when reinforcing steel was encountered. For demolition, the rating of the jackhammer was limited (15 pounds) to reduce micro-cracking of the sound concrete. In many locations, the depth of unsound concrete was more than 400 mm and AECL decided not to go deeper based on the fact that the repair concrete would be steel reinforced and provide adequate protection. The demolition was completed with a sand blast preparation to open up the concrete pores.

Two different procedures for the replacement of the damaged concrete of the ring-beam were used to take into account the depth of the repair. With the first procedure, for repairs of depths less than 50 mm, non-sag, shrinkage-compensated mortar was applied by hand trowelling. With the other procedure, for depths exceeding 50 mm, flowable, shrinkage-compensated concrete was poured in the formwork. In addition, for the second procedure, an internal skin reinforcement composed of 10-mm-diameter steel bars was used to control cracking, and adhesion to the structural concrete was improved by using 15-mm-diameter dowels (Fig. 3). In all cases, cementitious repair materials were applied only on saturated, surface-dry concrete, and water cured for seven days. Curing compounds were forbidden because they can reduce the adhesion of the FRP wrap and must therefore be removed. The specified mechanical properties of the mortar and the concrete in the two procedures were identical and as follows:

- compressive strength at 7 days: 30 MPa;
- direct tensile bond strength: 1.0 MPa;
- no bleeding or segregation; etc.

2.2 FRP wrapping

In addition to the high quality concrete repair, the secondary concrete was wrapped with surface-bonded glass FRP material to improve the durability of the ring-beam repair. The FRP wrap is not intended to prevent the formation of shrinkage cracks since dry concrete was mandatory for the FRP installation, and cracks appeared immediately at the end of the water cure of the fresh concrete (Fig. 4). Nevertheless, the wrap is able to mitigate the adverse effects of cracks because of the following characteristics and properties of the FRP:

- On installation, the primer component of the FRP system is liquid; it seals the cracks, and the cured FRP material is waterproof. Accordingly, rainwater cannot access the cracks behind the FRP wrap, reducing considerably the adverse effects of freeze-thaw cycling.
- The FRP material is non-corrosive, allowing it to be bonded on the surface as a structural skin reinforcement to reduce the widths of possible additional cracks.

- The FRP material exhibits a linear-elastic behaviour until failure. Thus, any relative movements of the cracks tips are forced to return to their initial positions.
- The high strength of the FRP material enables it to sustain the weight of any delaminated concrete, in the unlikely event that this should occur.

The application of the FRP wrap on the ring-beam required a standard concrete surface preparation standard for this type of material; that is, a profile correction, the rounding of all corners to a 30-mm radius, a sand-blasting of the concrete, and dry conditions. The installation of the FRP wrap followed the hand-applied wet lay-up procedure recommended by the manufacturer. Basically, the application steps are as follows:

- apply the primer, a low viscosity epoxy that penetrates the concrete pore structure to maximize the bond;
- level the surface with putty, an epoxy paste (Fig. 5);
- install the vertical strips of the FRP wrap according to the following procedure:
    - cover the surface of the FRP strip with the first layer of resin, a low sag epoxy that impregnates the fibers;
    - install the glass fiber fabric on the fresh resin, work the material to ensure the penetration of the resin between the fibers (Fig. 6);
    - complete the saturation of the fiber fabric with the second layer of resin;
- install the horizontal strips of the FRP wrap, using the same procedure as that for the vertical strips; and
- apply a finish coating, to protect the FRP from sunlight and to improve aesthetics by providing a grey concrete-like colour (Fig. 7).

The installation and cure of the FRP system required a minimum temperature of 5°C, as well as water protection.

The configuration for the FRP wrap is an open grid, as shown in Figure 8, to allow moisture exchange between the concrete and the atmosphere. Drainage of the massive concrete is assured by the grid openings at the bottom of the ring-beam. The arrangement of the grid is designed to cover most of the joints between the structural and secondary concrete. To ensure that the vertical strips are structurally capable of sustaining the weight of any possible delaminated concrete, they were anchored by providing the full development length on the dome roof. The required performance-based properties of the glass FRP were specified as follows:

- minimum tensile strength in the fiber direction 500 MPa
- minimum elastic modulus in the fiber direction 25 GPa
- minimum ultimate tensile strain 2 %
- minimum thickness 1 mm

2.3 Quality control

The achievement of long-term durability requires high quality materials along with a proper installation. A quality control program was therefore mandatory for this project. Accordingly, the Technical Specifications document included many inspection and test requirements. The quality control program began with the selection of the contractor and materials, and continues with the monitoring of the various sensors installed during the repairs.

The contractor that was selected was required to demonstrate his experience in the repair of similar large-scale concrete structures, as well as his knowledge in the rehabilitation of civil engineering structures using FRP materials. Prior to any work on the site, the demolition, the concreting, and the FRP installation, the contractor had to produce a detailed procedure for the quality assurance plan. Also, the FRP materials selected were limited to systems manufactured specifically for the strengthening of concrete structures.

Quality control for the concrete repair began with the preparation of qualification patches built and tested to demonstrate the properties of the materials and the efficiency of the procedures (Fig. 9). Various concrete mixes were applied, cured, and tested on the wall of the containment structure at ground level. On the actual repair on the ring-beam, the quality was controlled by sounding inspection to detect possible voids and delaminations, and random pull-out tests to measure the adhesion to the structural concrete (Fig. 10). Although the concrete strength and adhesion were higher than the specified values, it was impossible to completely eliminate shrinkage-induced cracking (Fig. 4).

To assure a high quality of the FRP wrap it was important to properly prepare the concrete surface. Thus, the rounding of the top and bottom corners of the ring-beam, the correction of all defects, and the sandblast cleaning were inspected before the installation of the FRP. The quality of the FRP installation was inspected by verifying that the working times of the various epoxies and the cure of the resin were respected, and by inspecting for delaminations or voids under the cured FRP wrap. In areas having unacceptable defects, the FRP was removed and replaced. Such defects were extremely rare.

The long-term quality control is assured by monitoring the various strain and temperature sensors both embedded in the repair concrete (Fig. 11), and bonded on the FRP surface (Fig. 12). This project was used to compare field measurements from proven technologies, such as vibrating wires and thermocouples, to those obtained with innovative fiber optic sensors (FOSs). On the Gentilly-1 ring-beam, two types of temperature FOSs (long gauge and Brillouin scattering) and six types of strain FOSs (Bragg grating, long gauge, Fabry-Pérot, Brillouin scattering, “smart patches”, and low coherence interferometry) were installed. The strains and temperatures are to be monitored on a regular basis.

3. CONCLUSION

The secondary concrete of the ring-beam of the Gentilly-1 nuclear containment structure was severely deteriorated. Although the strength of the thick-walled prestressed concrete structure was excellent, repairs were needed for durability and aesthetic considerations. The long-term durability of the repair was the primary objective since the containment structure will store the moderately contaminated reactor for the next 50 to 80 years.

The long-term durability objective of the ring-beam repair was attained by using state-of-the-art technologies for the replacement of the secondary concrete, together with the use of FRP materials for securing and protecting the concrete repairs. A rigorous quality control was implemented for every aspect of the repair work. The innovative application of FRP wraps to mitigate the adverse effects of shrinkage cracks was successfully carried out.

Close monitoring of the structure using various sensor technologies will provide information on the behaviour of the repairs, thus allowing for prompt maintenance interventions if required.

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