Seismic Upgrade of an Intake Tower using Underwater Base Isolation, Preliminary Design; Vancouver Island, Canada

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ABSTRACT :

The Strathcona Dam located on Vancouver Island, in British Columbia, Canada, includes a 50 m high intake tower, critical to be fully functional following a severe earthquake to enable closing of the intake gates to ensure post-earthquake stability of the earth dam. The intake tower, constructed in 1956, consists of a 34 m high concrete portion (mostly underwater) and a 16m high steel superstructure. The seismic withstand of the existing tower is inadequate relative to the Maximum Design Earthquake (MDE) with PGA of 0.8g. The selected seismic upgrade scheme involves installing lead-rubber isolation bearings about 13 m below the water surface, along with local strengthening of walls, without lowering the water level of the reservoir behind the dam. This paper discusses the 3D nonlinear time-history analyses carried out, the selection of appropriate ground motion records for the analyses, the extensive full-scale testing of two isolators at the University of California, San Diego with six degrees-of-freedom characterization using motions computed from the project analyses, the preliminary design of the intake tower to accept the isolators, the design issues related to the uplift at the isolators for the upstream/downstream rocking response, and the preliminary design of other modifications to the tower to accommodate the more than 450 mm of expected lateral deformation of the portion of the structure above the isolators relative to the lower portion. It is understood that this project, if carried through to construction, would represent the first use in the world of continually submerged isolators for a seismic upgrade.

KEYWORDS: base isolation, intake tower, underwater, seismic upgrade, lead rubber bearing

1. INTRODUCTION

The Strathcona Dam, constructed between 1955 and 1958, is located on the Campbell River on Vancouver Island, some 200 km northwest of Vancouver, Canada. The facility, owned and operated by BC Hydro, includes an earthfill dam, intake tower, power conduit, 68 MW powerhouse, spillway, and switchyard.

The dam retains a 56 km long reservoir and is 30 km upstream of the city of Campbell River, with a population of about 35,000. This dam is located in an area where two earthquakes of Magnitude 7.0 (M7) or greater have been recorded in the last century. BC Hydro's Dam Safety Management System classifies Strathcona Dam as a "VERY HIGH" consequence facility.

2. INTAKE TOWER DESCRIPTION

The intake tower consists of a concrete intake and transition section at the base, the main concrete tower shaft, a steel superstructure supporting gate hoisting equipment and an access bridge spanning to the earthfill dam. It provides water to the powerhouse via the power conduit located in a rock trench passing through the entire base of the earthfill dam (see Fig. 1). The concrete section of the tower is 36 m from the top of the base slab (founded on bedrock) to the deck level supporting the superstructure and is partly embedded in the earthfill dam. The

footprint of the intake tower above the transition section is 16.2 m by 7.8 m in the cross-valley (X/V) and upstream-downstream (U/D) directions, respectively. The tower wall thicknesses vary from 0.9 m to 2.75 m.
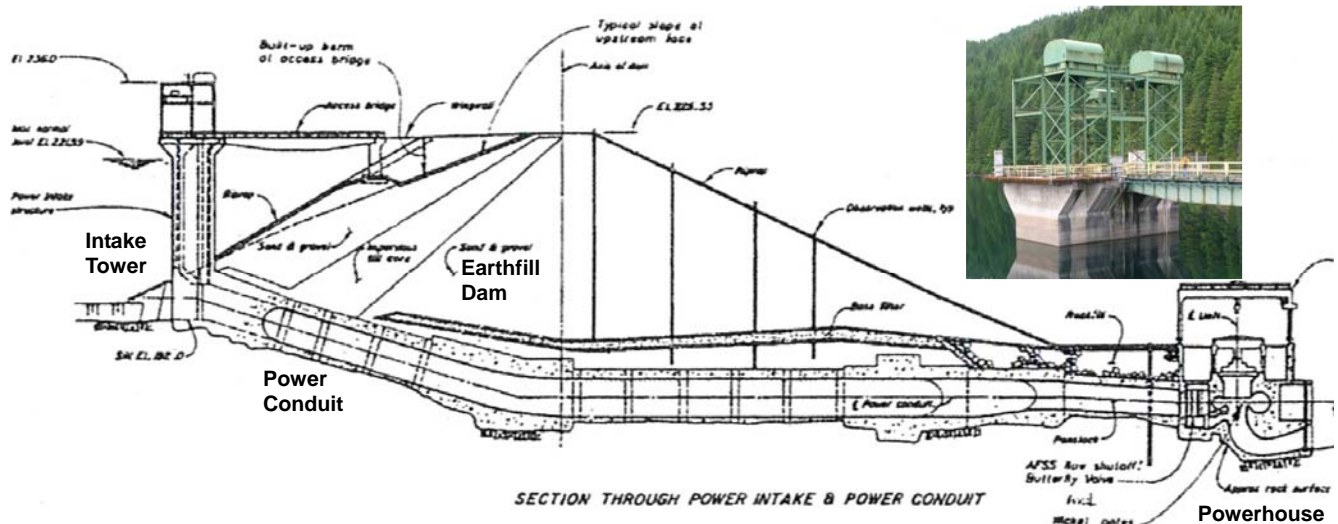

Figure 1 Strathcona Dam: intake tower, earthfill dam, power conduit and powerhouse

There are two open 'cells' within which the gates travel. At the base of the tower and mouth of the intake the tower is supported on three piers. The transition section tapers in plan from 16.2 m on the upstream side to 11 m at the power conduit. The top deck slab is 0.6 m thick, and the roof slab of the transition section is 1.8m thick.

The intake tower houses two intake gates that are required to lower and close off water flow into the power conduit a few minutes after any earthquake with a PGA on rock greater than 0.05g. This upstream shutoff is critical to ensure that the earthfill dam remains stable should the power conduit through the dam become damaged, which could allow undesired flow into the dam material potentially affecting the core and/or destabilizing the dam. The intake tower, as-is, was found to be significantly deficient for the MDE demand and a seismic upgrade is planned. The remainder of this paper describes the preliminary design developed to date for the seismic upgrade of the intake tower.

3. SEISMIC UPGRADE CONCEPT

The proposed upgrade involves the installation of lead-rubber isolation bearings at a depth of about 13 m below the normal water level. This will require cutting through the concrete tower cross-section at that submerged level, below which the intake gates normally park at open position, and removal of portions of the concrete walls to create space for the installation of the isolators.

With isolators in place, the response would be modified such that the portion of the concrete tower above the isolators would remain elastic without the need for additional strengthening. The concrete tower below the isolators would be strengthened by additional passive anchors to achieve elastic behavior in the U/D direction and near-elastic behavior in the X/V direction, which is necessary to ensure that the gates can be lowered from their normal parked position to the fully closed position after an earthquake. Seven passive anchors comprising 3-bar bundles of 65 mm diameter grade (830/1035 MPa bars) would be used for each pier.

Preliminary analyses indicated that 16 lead-rubber bearings will provide the level of isolation required for the MDE demand. The proposed isolators are 1.05m in diameter, 590mm high, with 40 layers of rubber and shim plates and a 200 mm diameter lead core, each weighing about 2400 kg. The isolators would be placed beneath all four perimeter walls and the central wall. As the upstream walls are only 0.9 m thick, there is not enough

space to install the isolators. Therefore, corbels must be built on the inside of the upstream wall to provide the needed support. This arrangement evenly distributes the gravity load amongst all the isolators. Figure 2 shows vertical and horizontal cross-sections through the tower, the plan layout of the isolators, a detail of the isolator anchorage and external steel plate with rubber seal, and a photo of the proposed lead-rubber bearing.
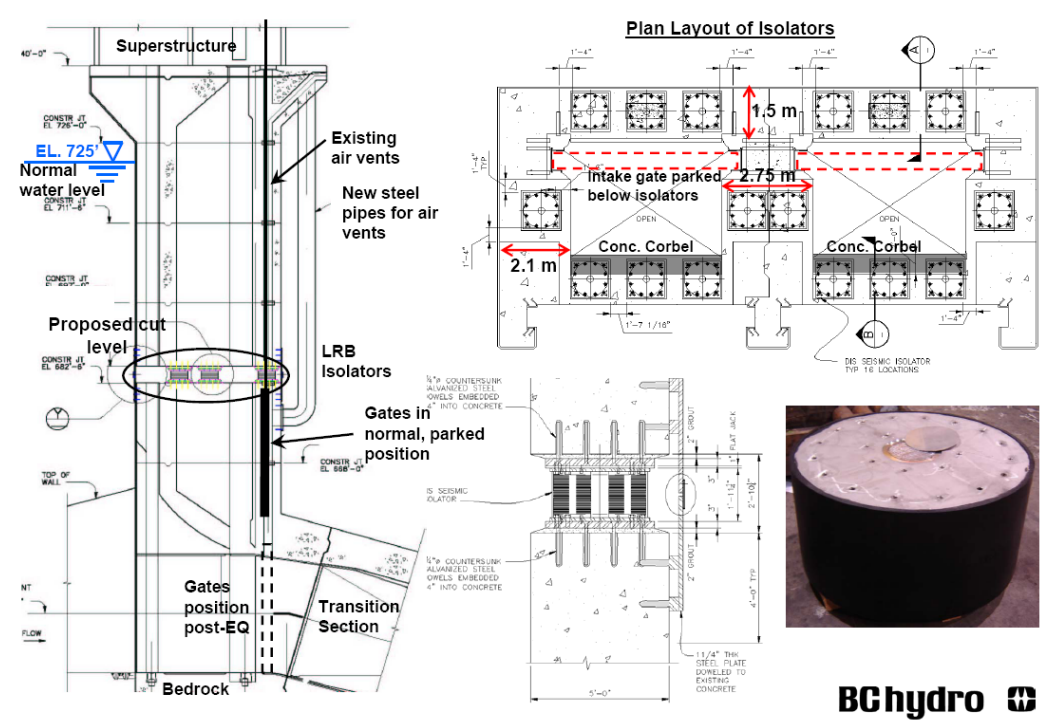

Figure 2 Intake tower cross-sections, layout of isolators, isolator anchorage, photo of typical isolator

4. ISOLATOR TESTING

Two prototype lead-rubber bearings were fabricated by Dynamic Isolation System (DIS), the largest manufacturer of elastomeric seismic isolation bearings in North America. The isolators were tested at both DIS and the University of California at San Diego (UCSD) facilities.

Tests at DIS were carried out in general conformance with IBC (2006) and ASCE 7-05 requirements for prototype testing of seismic isolators to determine the mechanical properties of the isolators under slow horizontal cyclic and constant vertical force demands. Tests at UCSD allowed six degree-of-freedom (DOF) characterization under real-time earthquake records to reconfirm the mechanical properties, to determine the performance of the isolators under combined earthquake time-history bi-directional displacement, axial compression or tension, rotation about two axes, and torsion (six DOF input as computed from preliminary analyses), and to investigate the limit state capacities of isolators in shear and tension. Each isolator was subjected to three real-time MDE earthquake records, plus nine other axial/shear tests with slow-speed loading rates between 50 and 65 mm/s.

The isolators showed excellent behavior throughout the entire DIS and UCSD test programs. Comparison of the UCSD test results with the corresponding DIS results showed excellent repeatability, even after multiple severe earthquake loadings. The isolators demonstrated the ability to sustain shear deformations exceeding 600 mm, and compressive axial loads of 14,000 kN, substantially exceeding the values predicted by the design analyses. One isolator was subjected to increasing uplift (tension) displacement cycles while held at a shear deformation of 600 mm, and sustained uplift displacements of 20 and 30 mm in the first two cycles, rupturing in the third

cycle at a displacement of about 22 mm. The isolator experienced about 50 minutes of sustained tension and shear in the three tension test cycles which is far more than the fractions of a second of actual tension demand that might occur during a very severe earthquake. Selected photographs of the tests are shown in Figure 3.
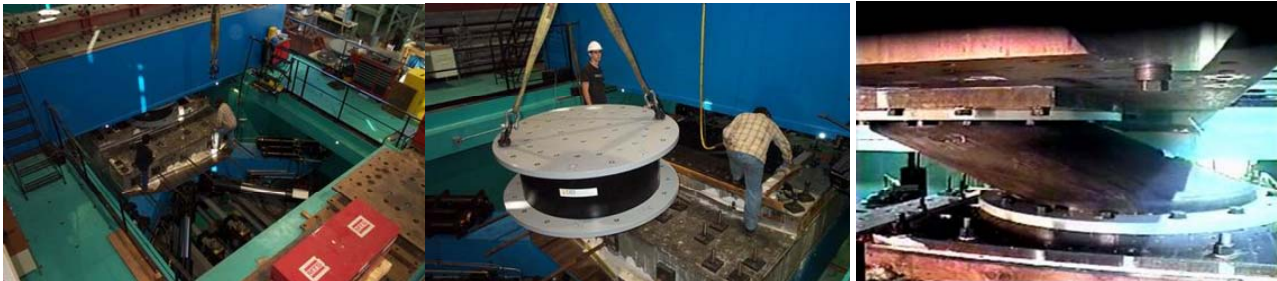

Figure 3 UCSD test machine, prototype isolator test specimen, testing at large shear displacement

5. TIME HISTORY SELECTION

BC Hydro developed a Uniform Hazard Response Spectrum (UHRS) for the site for a Maximum Design Earthquake (MDE) with a return period of 10,000 years that has a Peak Ground Acceleration (PGA) of 0.8g. The Operational Basis Earthquake (OBE) has a 1/150 year return period with seismic demand at 15% of MDE.

Ground motion time-history records consistent with this UHRS were selected using the Conditional Mean Spectrum considering ε (i.e. CMS-ε) method (Baker and Cornell 2006). The project geotechnical consultant developed an ensemble of 23 earthquake records (two horizontal components and one vertical component for each record) selected to match scenario earthquake target spectra consistent with the UHRS in the short (0.1 to 0.3 second), medium (0.4 to 0.8 second) and long (2.5 to 3.5 seconds) period ranges. These scenario records were selected at the period ranges of significance for the different components of the project. Figure 4 shows UHRS for the site and average horizontal target spectra in the three period ranges of interest.
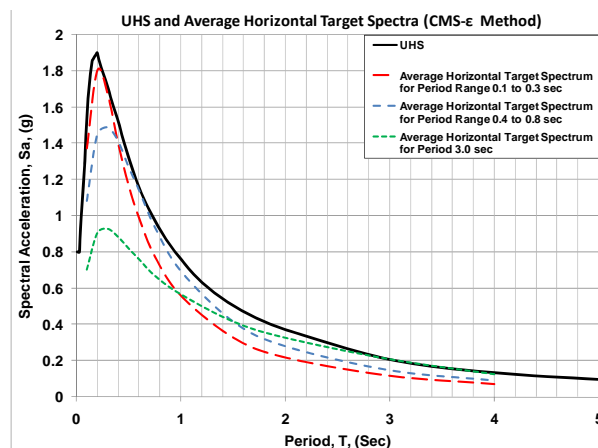

Figure 4 Uniform Hazard Response Spectrum (UHRS) and average horizontal target spectra

The ensemble includes three spectrally matched and three linearly scaled records in both the short and medium period ranges (12 records) plus seven linearly scaled and four spectrally matched records in the long period range (11 records). The maximum and minimum horizontal components were also reversed between upstream-downstream and cross-valley directions, thus creating a total of 46 unique inputs for the analysis.

6. COMPUTER MODELING AND ANALYSES

Nonlinear time-history analyses were carried out using the commercially available structural analysis program SAP2000 [Computers and Structures, Inc., 2008] and the computer program DAC3N, a nonlinear structural analysis program that includes state-of-the-art multi-spring isolator elements with axial-shear interaction capabilities [Kikuchi and Watanabe, 2004; Yamamoto et al., 2008].

The SAP2000 model included primarily solid elements for the concrete portion, gap elements for the foundation/bedrock interface, "Rubber Bearing" elements to simulate the shear hysteresis behaviour of the isolators and a multi-linear plastic pivot hysteretic element used to simulate the hysteretic behaviour of the isolators in tension, as observed in the UCSD testing (see Figure 5).
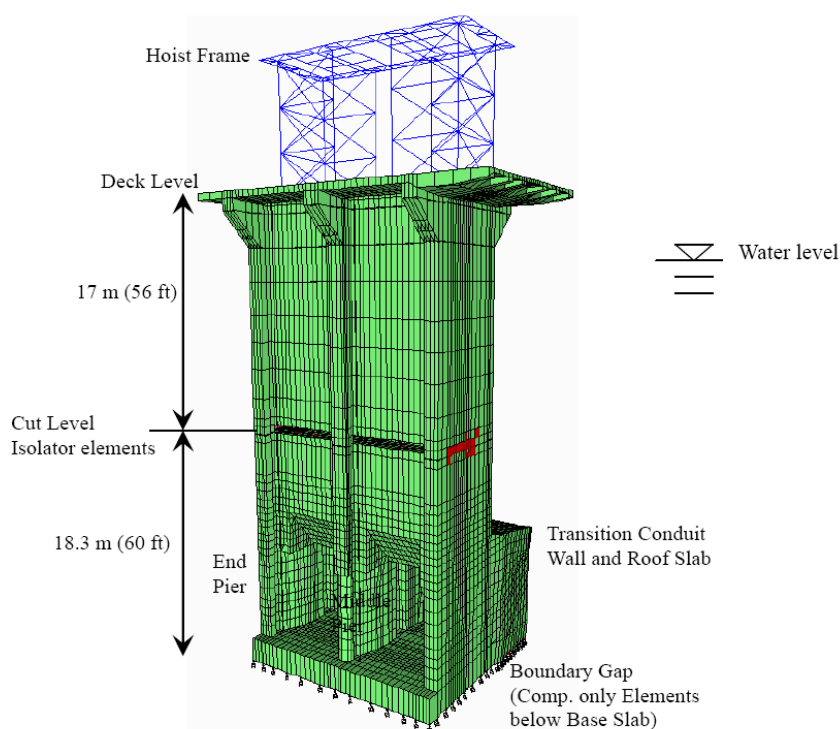

Figure 5 A 3D view of SAP2000 computer model of Strathcona intake tower

The rubber bearing element in SAP2000 has the limitation of not being able to model the interaction between shear and axial load in elastomeric isolators. To address this limitation, analyses were carried out for both upper and lower bound isolator shear properties corresponding to low and high axial loads on the isolators, based on the hysteretic curves obtained from the UCSD prototype testing.

DAC3N is a nonlinear structural analysis program which includes a recently-developed refined isolator element capable of simulating the effects of axial-shear interaction on isolator behavior [Yamamoto et al., 2008]. As shown in Figure 6, the multi-spring isolator model combines multiple axial springs acting in parallel (analogous to a beam on a flexible foundation) with axial and shear elements in series. The multiple axial springs have nonlinear tension-compression behavior that effectively models the axial hysteresis.

The multi-spring model in DAC3N was calibrated with the bearing response data obtained from the UCSD testing. The multi-spring model is the most sophisticated model currently available to predict the axial-load-shear behaviour of elastomeric isolators, however, at this time is limited to 2-dimensional analyses.
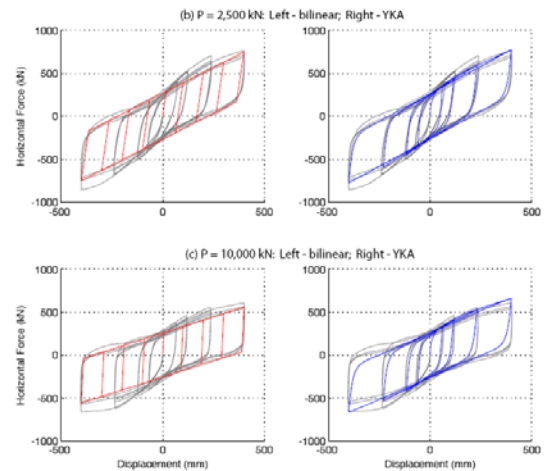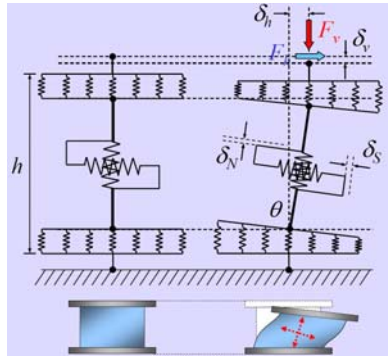
Figure 6 Schematic of multi-spring DAC3N model, comparison of SAP2000 upper and lower bound shear hysteresis loops (middle) versus DAC3N (right)

The tower model above and below the isolators was modeled as a two dimensional U/D lumped mass system with the same properties as the SAP2000 model. Figure 6 compares hysteresis shear model plots for upper and lower bounds in SAP2000 versus shear hysteresis loops in DAC3N with axial-shear interaction.

Linear elastic material properties were assumed for concrete. The properties of the isolators used in the analyses were: linear axial compressive stiffness of 2930 kN/mm; axial tension stiffness using a pivot hysteresis model with 3% initial axial tension stiffness; bi-linear shear hysteresis behavior with initial shear stiffness of 16 kN/mm for low axial load, and 50 kN/mm for high axial load; post-yield stiffness in shear based on 7.5% of elastic stiffness at low axial load and 1.5% of elastic stiffness at low axial load; yield strength for shear of 290 kN for low axial load and 260 kN for high axial load.

The SAP2000 model included the effective "added mass" of water, equivalent to approximately 40% of the structural mass. To calculate the water mass associated with the portion of the tower above the isolators a simple numerical analysis was carried out of a submersed rectangular object, similar in size to the intake tower, subjected to various wave frequencies using the AQWA suite of software.

In the SAP2000 analysis, the first fundamental period of vibration of the isolated tower in the U/D direction is about 2.94 seconds, in the X/V direction is about 2.82 seconds, and torsionally is about 3.54 seconds. The SAP2000 analysis included the effects of dynamic soil pressure loading on the portion of the tower in contact with the earthfill dam. Time history soil pressures, different for each wall in contact with the soil, were applied based on FLAC analyses performed by the project geotechnical consultant.

The SAP2000 analyses indicated that the average static axial load on each of the isolators (including buoyancy effects) is about 1676 kN. The maximum shear displacement computed for the isolators is 460 mm with a corresponding peak axial compressive load of 9950 kN. The analysis results indicate that the shear and axial demands on the isolators are not severe, in terms of the capacity of the isolators. The prototype isolator tests indicated a stability limit of 14,000 kN axial load in compression at a shear displacement of 550 mm. The maximum uplift displacement demand is computed as 16 mm for isolator shear properties corresponding to a low axial load and 8 mm for isolator shear properties corresponding to a high axial load. The SAP2000 results predict that the isolator undergoes only one half cycle in extension of 16 mm, thus well within the tested tensile limit (albeit slightly above the 10 mm limit recommended by the isolator supplier).

The peak horizontal acceleration just below and above the isolators is about 1.5g and 0.5g, respectively. This indicates a considerable de-amplification of ground motion by the isolation system. The peak vertical

acceleration at the deck level is about 4g indicating an amplification factor of more than 4 (PGA of 0.8g). Vertical isolation of the control building located on the deck is necessary to reduce accelerations of key electrical equipment, critical to ensure lowering of the gates, to below 1g per IEEE 693 requirements.

It is noted that the SAP2000 analysis conducted thus far has utilized the Fast Nonlinear Analysis (FNA) technique. The FNA method is fast, accurate and efficient for structural systems which are primarily linear elastic with a limited number of predefined nonlinear elements, such as the "Gap" and "Rubber Bearing" elements. The FNA method does not include P-delta effects nor large displacement effects. The analyses did not converge when the step-by-step procedure incorporating large displacements in SAP2000 was used. As a result, the software DAC3N was used to see if it predicted different behavior than the SAP2000 FNA analyses.

In general, the DAC3N analyses show good agreement with the SAP2000 model analyses in terms of maximum shear displacement. The differences in shear displacement are less than 20 percent, with the DAC3N results generally higher. Greater differences between the two analyses were found for the isolator uplift displacements.

The DAC3N analyses predict larger uplift demands on the isolators than the SAP2000 analyses, with more tension excursions beyond 10 mm. At this time further sensitivity analyses are being carried out and results for the cross-valley direction are also being evaluated. To date, in addition to explicit modeling of the isolator hysteretic behavior, the analyses have used 5% modal damping which is considered to be conservatively low in the horizontal direction due to the potential of significant damping for the portion of the tower above the isolators moving back and forth through the water. Computational fluid dynamic analyses are being undertaken to evaluate hydrodynamic damping with regards to possibly reducing translation, rocking, and uplift demand.

7. OTHER DESIGN ISSUES

To enable the interior of the intake tower to be dewatered in the future, a flexible rubber seal is required around the entire perimeter of the tower at the cut level for the isolators. The intent is for the seal to remain functional for the OBE, where horizontal movements at the cut level are in the range of 40mm.

The results of MDE analyses indicate an offset of less than 25 mm (most records indicate an offset of less than 10 mm) between the portion of the tower above the isolators and the lower section at the end of earthquake shaking. In general, rubber bearing isolators have re-centering (recovery of offset displacement) ability following a seismic event due to the intrinsic elastic stiffness of the rubber and the slow-rate, creep characteristic of the lead core. Tests carried out by DIS on one prototype isolator, at zero axial load showed that the isolator re-centered from 25 mm offset to within 8 mm of its original position after approximately 12 hours. The new intake gates, stoplogs and their guides would be designed to accommodate this 8 mm offset condition.

The isolators will be permanently submerged in a potable water reservoir. Testing is in progress to ensure that the rubber compounds for the isolators meet NSF standards. Furthermore, testing is also in progress to determine if long-term immersion in water affects the shear modulus or rupture properties of the compounds. The test program will subject the compounds to water pressure similar to that at the proposed installation, using actual water from the reservoir.

The change of rubber shear modulus over time, and related loss of isolator damping, are variables that need to be considered in estimating long-term bearing properties. Based upon available information, preliminary estimates of long-term rubber stiffening are 15-20% after 20 years and 30-40% after 50 years. These upper bound estimates in rubber shear modulus change over time should provide a conservative indication of the change in response of the isolated structure over the design life. It is proposed to test these parameters using four spare isolators at 5 years, then every 10 years thereafter to determine if the parameters remain within pre-set specifications. It is planned that the spare isolators be pre-compressed to the same axial load as the installed isolators and stored underwater within the tower at the same submerged elevation. The isolator

connections will be designed to allow for future replacement, should any degradation occur or the tests indicate the performance will be compromised by changes in the key isolator properties.

In order to ensure that the intake tower has some capacity during the construction to resist a low intensity, frequent return period earthquake, solid steel plates will be installed on the side walls at the cut elevation to tie the upper and lower portion of the concrete tower together until the construction work is complete.

To accommodate uplift displacements exceeding 10 mm, a "loose bolt" isolator connection detail was considered, which provides a free clearance between the connecting bolts of one isolator flange plate to permit some uplift displacement without inducing excessive tensile stress in the isolator. This detail has been used in at least two dozen isolated buildings in the U.S. and Japan. Typical loose bolt clearances are in the range of 12–19 mm. At this time all efforts are being made to evaluate fully fixed connections for the isolators relative to performance objectives, and avoid the use of loose bolt connections.

8. SUMMARY

The preliminary design of the seismic upgrade of the Strathcona Dam intake tower indicates that the use of base isolation is a cost effective solution that can achieve the stringent overall performance requirements. Various contractors were consulted to develop a construction sequence that is feasible, safe and minimizes the use of divers for the underwater work. The reservoir level can be maintained during construction.

For this proposed upgrade, special attention is being paid to the following items: appropriate analytical modeling of the isolators, subject to combined horizontal and vertical time histories, especially with regards to uplift; high vertical accelerations at the deck and superstructure levels affecting key electrical and mechanical components that must function post-earthquake to enable gate closure; sealing around the submerged cut to enable future dewatering; new external air vents replacing those within the concrete walls cut to accommodate the isolators, and critical to ensure air can enter the power conduit during dewatering to avoid collapse due to vacuum; and constructability in a timely and safe manner.

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