Case history – seismic upgrade of the Seymour Falls Dam

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ABSTRACT
The 50-year old Seymour Falls Dam is a key element in Metro Vancouver's water supply network, but in the early 1990s it was realised that the dam did not meet modern and evolving seismic design standards. Upgrades, constructed in 2004 to 2007, included innovative foundation improvements using explosive and dynamic compaction, significant new drainage and seepage improvements, a new 20 m high earthfill dam and a new larger concrete gravity wall. This award winning project was completed without interruption of reservoir operation and brings Seymour Falls Dam into full compliance with current established earthquake safety standards.

RÉSUMÉ
Le barrage cinquantenaire Seymour Falls est un élément important du réseau d'approvisionnement en eau de Métro Vancouver, mais au début des années 90 on a réalisé qu'il ne rencontrait pas les standards sismiques de conception modernes et évolutifs. Les mesures de réhabilitation construites de 2004 à 2007 inclurent des améliorations innovatrices à la fondation utilisant des explosifs et de la compaction dynamique, des améliorations significatives de drainage et percolation, un nouveau barrage de terre de 20 mètres de haut ainsi qu'un nouveau mur gravitaire en béton. Ce projet gagnant de Prix a été complété sans aucune interruption des opérations du réservoir, et permet au barrage Seymour Falls de rencontrer tous les standards sismiques établis.

1 INTRODUCTION

Metro Vancouver (MV) provides a reliable source of safe, high-quality drinking water to over 2,000,000 people in its 18 member municipalities. This includes acquiring and maintaining the water supply, treatment to ensure quality, and delivery to the municipalities. Water is collected from three mountainous watersheds: Capilano, Seymour and Coquitlam, and delivered by an extensive system of 22 reservoirs, 15 pumping stations and over 500 km of supply mains.

The Seymour Falls Dam provides approximately one-third of the total regional water supply and is a key element in the system. As shown on Figure 1, the dam is located on the Seymour River, approximately 18 km north of the Burrard Inlet within the Lower Seymour Conservation Reserve (LSCR). The LSCR is open to the public for recreational activities and a salmon hatchery is located approximately 300 m downstream of the dam. This environmentally and socially sensitive setting posed significant challenges during the seismic upgrade construction.

At the site of a 6 m high concrete dam constructed in the 1920s, a new dam was built in the 1960s to 30 m height with provision for a second raise (another 17 m) which has not been built. Prior to the current seismic upgrade, the dam comprised a composite structure consisting of a 235 m long slab and buttress concrete section, a concrete gravity retaining wall, transitioning to a 30 m high 220 m long earthfill embankment with an extensive upstream impervious earthfill blanket (Figure 2).

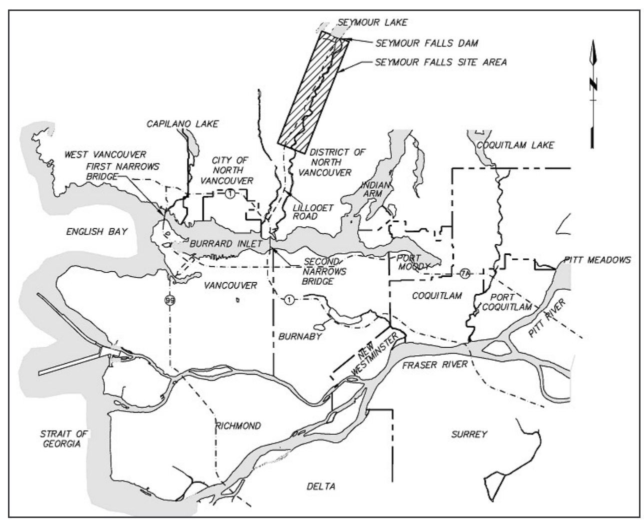

Figure 1. Location Plan of Seymour Falls Dam (from Murray et al. 2005)

This paper provides a review of the significant events and issues encountered during construction of the earthfill embankment. We have included a brief description of the setting and the design stage, but previous papers (Siu et al, 2004 and Murray et al 2005) provide additional details of the geologic setting, and the design criteria and analyses.

2 SETTING

The concrete section of the dam is built on a bedrock spur extension off the east valley wall whereas the earthfill dam is built on the Cougar Creek fan, a debris fan deposited after the latest glacial retreat 10,000 years ago. Cougar Creek sediments were rapidly deposited in the Seymour Valley by erosion of material from the valley sides inter-fingered with alluvial material transported down the valley. The Cougar Creek Fan covers a semicircular area of radius about 800 m and fills a 150 m deep buried valley.

The upper 20 m to 40 m of the fan includes some very loose granular material as well as extremely coarse material including boulders, cobbles, sand and gravel. Where bedrock is greater than about 30 m deep, the fan materials are progressively finer below the bouldery layer, transitioning to coarse sand at about 30 m depth with denser preglacial deposits at lower depths. Groundwater levels are influenced by both the reservoir and local fan recharge with significant under-seepage flows below the earthfill embankment, flowing south and east around the buried bedrock spur and then to the Seymour River.

The site is in an extremely high rainfall environment with about 4m of rain per year with a pronounced rainy season from November to April making most site work and especially earthfill work impossible during the winter months. Heavy snow fall and freezing temperatures would occasionally cause site shut down due to avalanche hazards on the access road.

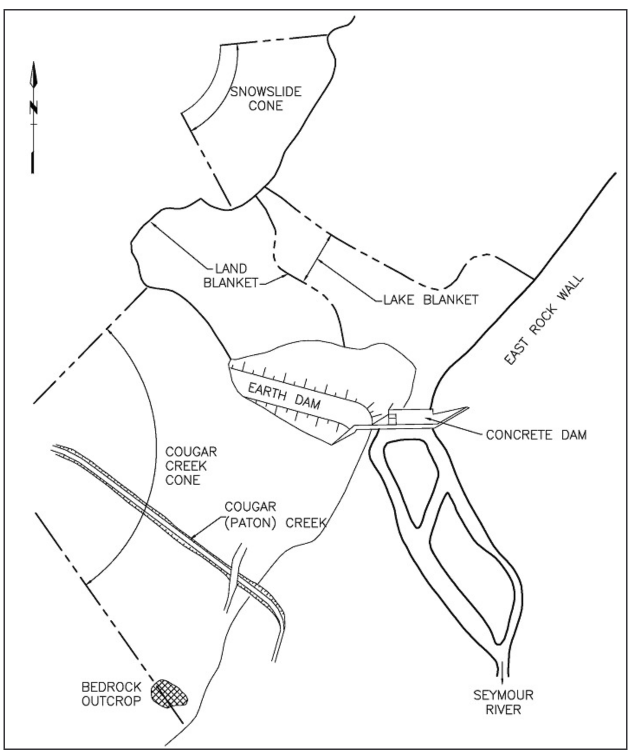

Figure 2. Schematic Plan View of 1960s Dam

2.1 1960 Composite Dam Components

The 1960 earthfill dam consists of three major elements, the main embankment, the land blanket and the lake blanket shown schematically in Figure 2. The zoned embankment fill comprises granular filters and shells surrounding an inclined clay core. The impervious inclined central core extends below the upstream shell and is connected to a 1.5 m thick impervious lake and land blanket of clayey silt which extends about 200 m into the reservoir to tie into rock and natural lacustrine silts. The blanket is not a perfect cut off and overlies pervious fan deposits which transmit significant seepage beneath the dam.

The concrete section consists of a slab and buttress dam, a series of upstream sloping concrete slabs supported on buttress walls spaced at 6.7 m centers. Discharge facilities include twelve overflow spillway bays with a total width of 73.2 m, two 1,524-mm diameter outlets fitted with Howell Bunger valves and one 610-mm diameter low level outlet. At the west end of the concrete section, the concrete transition block connects the concrete section to the earthfill embankment. Downstream of the transition block, the embankment fill is retained by a mass concrete gravity wall (GRW). The entire concrete section, including the transition block and the concrete gravity wall, is founded on bedrock.

Drinking water supply is provided through two screened low level intakes at the concrete section which combine into one 2,390 mm diameter steel pipeline (Main #2). The water main connects the reservoir to the GVWD water distribution network after treatment at a downstream chlorination plant.

3 DESIGN HISTORY

During the 1980s, studies indicated that the potential for a moderate or large earthquake in the region was much greater than previously anticipated and that initiated a review of the dam to ensure seismic standards were met. A partial upgrade was conducted in 1994 on the Concrete Section to resist the Design Basis Earthquake (DBE), corresponding to a National Building Code of Canada (NBCC) level of earthquake with a return period of 475 years. At the time, no upgrade was carried out on the earthfill section. Although the 1994 upgrade met DBE requirements it did not meet Maximum Credible Earthquake (MCE) requirements as defined by Canadian Dam Association dam safety guidelines (CDA 1999).

From 1998 to 2003, Klohn Crippen Berger prepared a detailed design for the MCE upgrade of the Earthfill Section, while Hatch Energy conducted the MCE design for the Concrete Section. The detailed design of the MCE upgrade was completed in 2003.

3.1 Key Design Criteria

Seismic response spectra were defined and design seismic parameters representing the MCE were established, as part of a site specific seismic hazard assessment (BCHI, 1998) including PGA of 0.35g (M7.5 intraplate) to PGA of 0.65g (M6.5 local fault).

Basic design criteria were selected for the upgrade consistent with the very high consequences of failure. These included FOS >1.3 downstream post earthquake and FOS >1.1 for upstream crest post earthquake using residual strength in liquefied zones, and less than 1 m post MCE dam crest horizontal deformation.

Additional project criteria included the requirement for uninterrupted operation of the dam and maintenance of current levels of earthquake and flood protection during construction. Consequently, for failure modes which could impact reservoir containment, long term safety factors, appropriate to very high consequence dams, were maintained at all times, even for temporary construction conditions. Protection of reservoir water quality was also paramount.

It was also necessary to monitor and prevent environmental impacts, including impacts to the highly sensitive salmon hatchery and incubation facilities, located 300 m downstream of the dam, and which partly relies on water supply from groundwater within the Cougar Creek Fan as well as water supplied from the reservoir.

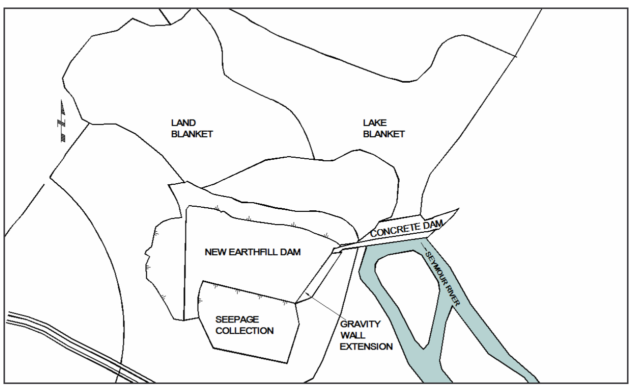

Figure 3. Schematic Plan of Seismic Upgrade Components.

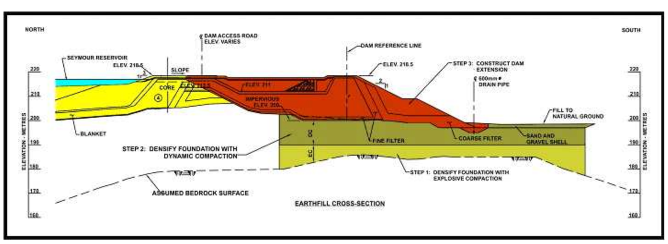

Figure 4. Schematic Section of Seismic Upgrade Components (Murray et al, 2005)

3.2 Key Design Issues

For the earthfill dam, the key design issue was the liquefaction susceptibility of the coarse granular deposits in the Cougar Creek Fan lying under the 1960s dam and the new dam. Under the design earthquakes (Murray et al, 2005) these deposits were predicted to liquefy extensively.

Liquefaction susceptibility field testing was assessed by several techniques. Becker Penetration test (BPT) correlations to SPT were found to be ineffective at the site. This was attributed to the very loose nature of the ground which led to the BPT system often pushing coarse particles ahead rather than “penetrating” the ground. Innovative methods of obtaining meaningful standard penetration test (SPT) values were established. SPT N-values, obtained using conventional SPT field procedures, were calculated based on a blow count of 4 x the lowest 3 consecutive blows recorded over 25 mm increments, in the penetration interval of 0.15 to 0.45 m. This method provided reasonable data in the cobbly ground and was used for pre- and post-construction improvement measurements.

Soil liquefaction was assessed using both Seed’s simplified analysis (Youd et al, 2000) using SHAKE (Schnabel et al 1972) and a self-triggering total stress approach (Beaty and Byrne, 1998) utilizing the 2D finite difference program FLAC (Itasca 2000). The combined analyses provided a screening estimate of the liquefaction potential of the foundation soils, estimates of the earthquake induced deformation of the dam, and were used to optimize the layout of required ground improvement. Foundation ground improvement was focused in areas with predicted liquefaction.

As a general observation, analyses showed that the $(N_1)_{60\text{-}CS}$ required to achieve a safety factor > 1.1 against liquefaction is about 20-blows/0.3 m, which was about twice the pre-improvement site average blow count.

3.3 Seismic Upgrade Components

To meet the design criteria, including operational and environmental requirements, it was critical that the dam remain operational during the upgrade work. The selected upgrade option included building a new earthfill dam on an improved foundation, downstream of the 1960s dam, which allowed the 1960s dam to remain fully operational and provide PMF flood protection during the construction. The upgrade components for the earthfill section are shown schematically in plan and section on Figures 3 and 4 and include:

- A densified foundation to nominally 30 m below original ground surface.
- Improved drainage measures including pressure relief wells, drainage blanket, and surface drain collection systems.
- A new zoned earthfill embankment providing a conventional impervious core with wide granular shells and wide filters connected to the under-drainage system.

An instrumentation network for construction and post-construction monitoring was installed including electric and pneumatic piezometers, survey monuments, seismographs, monitoring wells and flow monitoring stations.

The new dam also acts as a downstream stabilizing buttress to the old dam, mitigating future downstream deformation of the 1960s embankment, although the new dam is designed to stand alone even with full breach of the 1960s core. Included in the design is a connection of the new dam core to the old core and reservoir blankets (Figure 4).

Improvement of the new dam foundation required compaction of the ground downstream of the 1960s dam to prevent earthquake induced liquefaction in zones critical to the stability of the new dam. This was achieved using a combination of explosive compaction (EC) and dynamic compaction (DC) techniques.

The seismic upgrade also included construction of an approximately 30 m high by 100 m long extension to the concrete gravity retaining wall (GRWE). The GRWE forms the left abutment of the expanded earthfill dam, retaining the earthfill and forming a transition from the earthfill embankment founded on sediments to the concrete slab and buttress dam founded on bedrock. The GRWE was built immediately downstream of the existing Gravity Retaining Wall (GRW). Siu, et al, 2004, provides additional design details of the GRWE and other concrete improvements.

The new embankment centreline is approximately 75 m downstream of the 1960s dam centreline, and the new crest (including remnant of existing embankment) is now about 75 m wide (measured down-valley). The new dam is approximately 180 m long (parallel to the existing dam), and about 21 m maximum height above the improved foundation.

4 CONSTRUCTION

The ground improvement began with site preparation, demolition and excavation in February 2004, with the first explosive compaction blast in May 2004, and the last blast in January 2005. The DC followed beginning in December 2004, with excavation and panel preparation. DC and EC work continued concurrently in January 2005 until the EC work was completed. The DC was completed in April 2005 followed by construction of the concrete gravity wall extension (GRWE). The other miscellaneous concrete works began in May 2004 and continued to 2006. The drainage works and earthfill embankment were built between April 2005 and November 2007.

4.1 Site Preparation and Instrumentation

The first stage of the foundation improvement required preparation of the site by excavating a series of benches down an average of 10 m into the top of the Cougar Creek fan, downstream of the old dam.

Excavation of about 190,000 m^3 of primarily bouldery, sand and gravel was conducted in advance of the EC and DC work, with excavation and panel preparation generally proceeding from the highest elevation on the west abutment to the lowest elevation near the existing gravity wall. Excavated materials were stockpiled at one of several disposal or stockpile sites, separated for reprocessing and re-use of granular materials.

The excavation encountered several hundred boulders greater than 5 cubic-metres in volume, and dozens of boulders of greater than 20 cubic-metres and up to 80 cubic metres. Large boulders were split by blasting, to allow handling and moving by the site fleet of Cat 345 excavators and Volvo A35 off-road trucks. A special pay item was included in the Contract documents for boulders over 5m^3 (1000 were envisaged in the Engineers quantity estimate), and a new special pay item was negotiated for the dozens of much larger boulders.

Prior to construction a new water supply pipeline, monitoring wells and miscellaneous upgrades were provided to protect the nearby downstream Seymour Salmon Hatchery. The monitoring wells were installed to monitor ammonia, nitrate and turbidity levels in the groundwater during the EC blasting, due to the sensitivity of the hatchery rearing ponds to even minor increases in these chemicals. Provisions were included at the hatchery to divert groundwater in the event that chemical levels exceeded allowable limits.

Old piezometers were decommissioned or protected. Six new piezometer bundles were installed in the foundation at the toe of the 1960s earthfill dam including pneumatic, electric strain-gauge, and electric vibrating wire piezometers. The piezometers were used to monitor porewater pressure changes during and porewater dissipation after EC blasting. In addition the piezometers provided data on groundwater levels during dewatering for the GRWE excavation and the DC trench excavation.

A set of temporary survey monitoring points (TMPs) were established on critical structures, including the toe and crest of the existing embankment, the existing gravity wall, the concrete transition block, the concrete spillway piers and deck, the chlorination building, and on Main #2. These survey points were monitored before and after each blast to check that movements were within acceptable tolerances. Eleven deep settlement posts (DSPs) were installed in the foundation footprint, with the base of the posts just within the top of the EC improvement zone, to allow comparison of settlement at depth against settlement at surface. Greater settlement at depth than surface might be indicative of arching of soils above the EC zone.

4.2 Ground Improvement

4.2.1 EC and DC Program Design

Compaction by the EC method involves detonating explosive charges at selected depths and on a grid pattern with up to 3 passes. Detonation in each pass occurs in controlled sequences to precipitate liquefaction in limited sections of the dam foundation, essentially simulating the effects of an earthquake in liquefaction susceptible ground, combined with effects of compression, shearing and volumetric strains. Following blast induced liquefaction, the soil mass re-consolidates as pore water pressures dissipate.

Dynamic compaction involves the repetitive dropping of large weights in a grid pattern, with the direct application of high impact energy causing the compaction and strengthening of the ground. This is achieved by dropping a large steel tamper several times from heights of up to 30 m at a given grid point, creating craters of up to 3 m depth, with one to three offset grid patterns in the "high-energy" phase, followed by infilling of the craters, and an ironing phase, for shallow surface compaction, with a lighter tamper dropped from a lower height.

A trial compaction program was conducted in 1998, using Explosive Compaction (EC) from depths of about 10 to 30 m, and Dynamic Compaction (DC) for the upper 10 m. The trial DC program achieved $(N_1)_{60\text{-}CS}$ > 25 blows/0.3 m down to at least 10 m. The effectiveness of the DC dropped off significantly below 10 m depth. The EC trial achieved $(N_1)_{60\text{-}CS}$ > 20 blows per 0.3 m in zones in the range 10 m to 20 m depth below ground level but gave little or no improvement below about 25 m depth with charges set between 10 and 20 m depth. Test hole drilling between the application of the EC and DC trials indicated that some arching had occurred in the bouldery ground above the EC zone. Post trial FLAC analyses showed that the combined densification achieved in the trial EC/DC program was sufficient to meet the criteria for a stable dam, which could operate, post MCE.

The trial program allowed the preparation of site-specific equations for design of EC based on empirical modifications to published forms of equations by Narin Van Court (1998). Site specific relationships were developed for variations with distance for peak particle velocity (PPV), regional pore pressure ratio (PPR) and EC induced settlement. These values were used to determine expected velocity at existing structures and to predict the extent of blast liquefaction zones.

Gained experience from the 1998 trial program, aided the ground improvement design for the new dam. The design was, (a) Pre- excavate the upper 10 m of the site; (b) Conduct EC from 10 m to 20 m depth below the excavation base; and (c) After EC, conduct DC on the base of excavation from 0 to 10 m depth.

4.2.2 Production Explosive Compaction

Explosive compaction was conducted over approximately two-thirds of the footprint of the new dam in fifteen panels, with three passes per panel. EC proceeded from the highest bench on the west abutment down to the lowest bench, closest to the gravity wall. The staging was partially selected to allow the first few blasts to occur in the least critical area, allowing the project team the opportunity to fine-tune the blasting program.

Panels averaged about 900 m^2 with each delineating areas with similar proximity to structures or depth to bedrock to allow panel-specific blast patterns and charge densities to be specified. The specifications therefore prescribed the EC blast patterns for each panel, based on the predicted PPV and PPR results using the formulae derived during the Trial Program. The Contract specified hole spacing, depth, charge weight and sequence of charge detonations were provided. The average charge density (powder factor) ranged from 0.10 to 0.15 kg/m^3 of treated ground. By specifying the blast patterns and charge weight we were able to predict and control vibrations and pore pressure response within and adjacent to panels.

For each panel, boreholes were drilled and cased in three passes, with an average of 18 holes per pass. Hole depths averaged about 20 m below the excavated ground surface and charges were placed at 10 to 20 m below ground. Approximately 800 holes, totalling over 16,000 m were drilled using a Rotex Oy Symmetrix® downhole hammer system that proved very effective in the coarse bouldery ground. In general, drilling progressed at an average rate of about 18 lineal metres per rig per 12 hour shift, with two rigs working 6 days/week, 24 hours/day. Drill holes were advanced using 168 mm OD steel casing, then 100 mm ID PVC casing was installed, and the steel casing removed. Casings were filled with water to facilitate installation, but were pumped dry prior to loading with explosives to minimize potential hydrodynamic shock between charges.

The nearest EC blast panel was about 300 m from the fish hatchery, 18 m from the Main #2 and the chlorination building, and about 8 m from the 1960s earthfill dam toe and the gravity retaining wall. The nearest DC drop points were at similar distances, except DC extended to within 8 m of the water main (Main #2) and the chlorination building.

Holes were loaded with Iremite® TX, a cap-sensitive emulsion manufactured using high strength microballoons for use in applications with high transient over pressures. Up to 30 kg were placed in each deck with a minimum 2 m vertical separation between the top and bottom of subsequent decks. Electronic detonators capable of 1 ms delay precision were used. In total, over 33,000 kg of explosive was detonated in 56 blasts. Timing delays between explosives in adjacent holes were set at about 25 to 50 ms, so that propagating blast pressures liquefied the ground across a given elevation at essentially one time. The timing was staggered so the blast induced vibrations propagated away from the dam and the next detonating charge, instead of potentially stacking towards the dam and nearby structures.

Significant settlement, locally over 2m vertical strain, was achieved during the EC work, meeting the project requirements, while staying within the restrictive limits of PPV and PPR. Detailed surveillance monitoring was conducted prior to and following each blast, including review of piezometric data, survey of monitoring points, mapping of tension/settlement cracks around blast panels, vibration monitoring at critical installations, water overpressure measurements in the salmon rearing ponds, and assessment of pore pressure response at the toe of the existing embankment.

All blasts were carefully instrumented and monitored to ensure detonation of all charges, including the use of Nonel tubing or coaxial cable monitoring to allow a check on the number of detonations per sequence. A few misfires were suspected, but each was dealt with by an established protocol to investigate and, if necessary, detonate the charge. One sympathetic detonation occurred in adjacent charges and was attributed to a faulty batch of detonators; the suspect batch was removed from use.

Extensive monitoring of groundwater in the four monitoring wells installed downstream of the EC blast area mitigated water quality concerns. Trace quantities of nitrite and ammonia were occasionally noted in the well water, but were within specified limits.

Surface settlement at the EC panels was compared with that measured at the deep settlement posts. By the third pass in each panel, little difference was noted, leading to the conclusion that the effects of multiple passes in blast panels were beneficial in minimizing the effect of arching in the bouldery ground above the blast zone. Average settlements met or exceeded the upper target of 5% strain in the EC zone.

4.2.3 Production Dynamic Compaction

The dynamic compaction work was conducted over an area of about 22,000 m^2, covering most of the new dam footprint, as well as an area about 25 to 75 m south of the new dam toe (for a possible future dam raise). A small margin of 5 to 8 m width was maintained between the edge of the DC zone and the existing embankment and facilities.

The DC crane was a 350-ton DC-purpose built Lampson crane (LDC-350) capable of dropping the weight from a maximum of 30 m. Three steel tampers were used; a 25 tonne tamper, an 18 tonne tamper, and a 16 tonne ironing tamper. The majority of the heavy energy application was done with the 25-tonne tamper, but the 18-tonne tamper was used in two locations where proximity to Main #2 or the chlorination building caused some concern over PPV levels, requiring a reduction in compaction energy. The majority of the production drops were from 26 m height, providing drop energy of 625 tonne-metres/drop when using the primary tamper. The average energy application was 550 tonne-metres/m^2 plus 90 tonne-metres/m^2 for ironing, as per design.

The DC was conducted in 7 main panels. The DC energy was applied in three high or heavy-energy passes. The first and second passes were conducted on 10 m by 10 m grids offset by 5 m from each other, with the third pass conducted on the intervening 5 m grid points. Typically, 40 drops were conducted at each Phase 1 drop point, 22 drops at each Phase 2, and 10 drops at each Phase 3 location, roughly a reduction by half phase-to-phase. Craters were backfilled between phases and an overall pre- and post-DC survey and material balance was conducted to calculate settlement from DC, with correction made for the crater backfill.

Dewatering was required, through ditches, sumps or pumping wells, where groundwater was closer than 1 m to surface. DC used similar settlement and PPV relationships to those developed for EC, but without the need for overpressure or chemical residual monitoring.

4.2.4 Ground Improvement Quality Testing

During the EC and DC phases, standard penetration testing (SPT) was conducted using conventional ASTM SPT testing techniques with a mud rotary rig. All SPT were instrumented with an energy analyzer to measure hammer efficiency. The drill bit used specially designed jets to direct mudflow upward and not at the base of the hole. SPT blows were calculated as previously described. Drilling was carefully advanced through cobbles and boulders and SPT started immediately when finer material is encountered.

SPT data were reduced to derive $(N_1)_{60\text{-}CS}$ values for use in liquefaction assessment. An important component of the $(N_1)_{60\text{-}CS}$ estimate is obtaining fines content. Good sample recovery was achieved by wrapping the split spoon core catcher in cling film. The SPT testing indicated that the EC and DC work met performance criteria.

Perhaps more telling than the improvement in SPT values was the settlement achieved during the ground improvement. The average settlement following the EC work exceeded the design target settlement range of 2% to 5% settlement. In most areas this equalled 200 mm to 500 mm settlement.

The average settlement in the DC area was approximately 450 mm ± 100 mm, and met the design target settlement range of 2% to 5% settlement, assuming a 10 m treatment depth.

The combined EC and DC settlement ranged from about 200 mm to about 2.5 m settlement as shown on Figure 5.

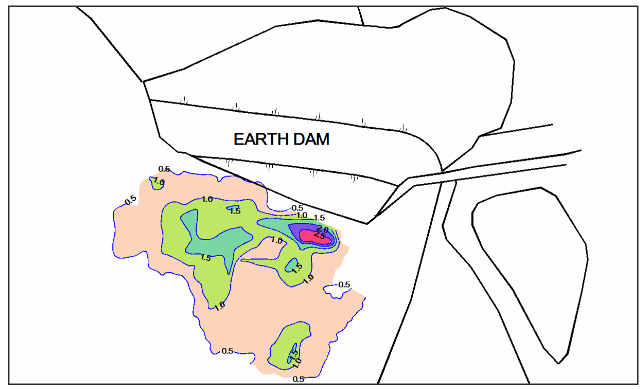

Figure 5. Combined EC and DC settlement.

4.3 Drainage Improvements

Following heavy rain in late 2004, several seeps were observed across the site in the newly exposed foundation through the average 10 m deep bench cuts. By careful monitoring of these seepages (some of which were also observed prior to 2004 during previous heavy rainfall events) and conducting a planned pump test (to aid in construction dewatering), we had the unique opportunity to improve and calibrate the site groundwater model through the use of previously unavailable data. The revised model, which also reflected the changed conditions from the ground densification, allowed for improvement of the design of the drainage elements for the final dam including both seepage and surface water drains. The dam drainage system was initially designed to provide sufficient capacity to meet a post earthquake condition with the upstream lake blanket removed, and the 1960s earthfill embankment slumped, under probable maximum precipitation (PMP) conditions, i.e., the PMP inflow to the reservoir following an MCE event, combined with PMP infiltration into the regional groundwater model.

This extreme condition is a valid design case for this high consequence dam since one of the design criteria is for the dam to operate without significant repairs after the design earthquake.

4.3.1 Seepage Collection System

The dam drainage system includes a line of 10 passive pressure relief wells completed to bedrock, a 3 m thick toe drainage blanket, and an extension to an existing 1960 design element consisting of a large rock fill/gravel drain extending north-south along bedrock under the width of the embankment, termed Drain # 1. Drains flows are collected separately in pipe systems to allow for assessment of flows in different components.

The new Drain #1 has an effective flow area of 6.9 m^2, and extends about 65 m from where it ties into the old Drain 1 at the toe of the 1960s earthfill dam. The old Drain #1 continues upstream to about the centreline of the old earthfill embankment. The new Drain #1 comprises 200 mm minus rockfill wrapped by a 75 mm minus clean gravel placed on bedrock and adjacent to the west toe of the new GRWE.

The pre-upgrade average wet season flows in Drain #1 were about 2600 L/min with peaks up to about 5,000 L/min. Post upgrade these flows were generally in the range of 3,000 to 20,000 L/min, with peaks noted to 32,000 L/min indicating the reconstructed and extended drain was significantly more efficient than the old Drain 1.

4.3.2 Surface Collection System

Surface water collection from the large flat area between the new and old embankment crests was designed as a separate system from the dam seepage collection system both to limit the impact of surface water on the capacity of the seepage system as well as to allow separate monitoring and assessment of the seepage system without the influence of the surface water.

4.3.3 Flow Monitoring

The seepage flows are monitored from each area (Drain 1, drainage blanket, GRWE drain, relief wells, and final outlet) by multi-level weirs and recorded at 5 minute intervals at centrally located dataloggers. This data will be used to recalibrate the site groundwater model following several seasons of post-construction flow monitoring. To date, flows have been within design expectations.

4.4 New Earthfill Embankment

4.4.1 Zoned Embankment

In early 2006 following completion of the drainage elements and the GRWE, construction of the zoned earthfill embankment began. The new embankment fill includes general shell fill, coarse and fine filters, impervious core, and three riprap/rockfill zones.

The fill placement began with the construction of the core of the new dam with associated filters and shell zones. Once the new dam core was built, the crest of the old dam was partly excavated to allow tie-in of the new core to the old core.

The tie-in zone between the two cores comprises a horizontal layer of impervious material, overlain by granular fill, which partly acts as a “bathtub” between the two cores keeping a wetting zone above the impervious new core.

The following three issues were encountered during the construction of the earthfill embankment.

4.4.2 GRWE Crack Filler Zones

During construction of the mass concrete GRWE, the potential for cracks was reconsidered. It was considered prudent to design the embankment fills to guard against the potential of piping embankment fills through potential cooling cracks. The embankment zone was modified to include two new zones of material to serve as crack healer fills in the unlikely event of significant cracking of the concrete GRWE in areas where it lay in contact with the impervious core. These two new earthfill zones preclude the possibility of piping of impervious core materials.

Figure 6. Earthfill Embankment under Construction 2006

4.4.3 Alternate Impervious Fill Zones

Initially good progress was made placing the impervious fill, including a substation of till material for the impervious clay for the first 3m of the core. Above about elevation 206 m, the material was switched back to the original specified clay impervious material. The natural impervious clay included wetter clay layers which were 30% over optimum moisture content, requiring significant drying time to condition the clay. This drying time began to impact the progress.

The critical path activity for completing the embankment fills was placing the moisture sensitive impervious core material. A potential solution was to find a suitable substitute for the clay core material. There was insufficient till to switch back to that material. The Owner, the Engineer, and the Contractor worked together to come up with a solution to the slow mining and conditioning rate. An adjacent sand unit was borrowed and following several trials and extensive lab and field testing, a blended sand/clay material was found to be an acceptable substitution for the pure clay impervious. Revised material specifications were prepared for the core, and the blended sand/clay was introduced as a substitute for the impervious. This allowed the core of the new dam to be completed in 2006. The core tie-in was nearing completion in October 2006, however, when the rains began.

4.4.4 Winter Shutdown

In November 2006, 1500 mm of rain in 15 days caused a shutdown of the work. Little to no permanent fill placement occurred in November 2006 due to rain. Although the new dam was complete and provided PMF protection, the tie-in to the 1960s dam core was not, and the Contractor elected to shut down operations in December 2006.

During the 2006-2007 winter shutdown the crest of the old dam was kept partly in place to provide a protective berm. This berm protected the reservoir water quality by preventing runoff from the works to the reservoir.

The winter shutdown included placement of a 750 mm to 1 m thick cover of sacrificial protective fill materials which were removed in Spring 2007. The cover fills successfully protected the permanent fills allowing the work to restart with a minimum of rework.

The final fills, surface drainage, and roadways were completed in 2007. A total of approximately 260,000 m^3 of fill was placed for the new embankment. The project achieved substantial completion in October 2007.

5 CONCLUSION

The seismic upgrade of the Seymour Falls Dam was completed per design with three significant changes. The under-drainage system was upgraded to meet revised predictions of groundwater flow volumes, as encountered during construction. Secondly, a zone of till and associated sand and gravel filter was added next to the concrete gravity wall extension to provide protection from possible piping of the impervious core where in contact with the new mass concrete GRWE. Thirdly, the impervious core of the new dam was revised to allow zones of till and a silt/fine sand/clay blend to mitigate mining and processing difficulties with the planned original clay core material.

The ground improvement phase was successfully completed using the complementary techniques of explosive and dynamic compaction in very difficult ground conditions, including very bouldery ground. The combined settlement induced in the foundation from these methods averaged about 600 mm and ranged up to 2.5 m indicating approximately 5 to 10% increase in relative density considering the treatment depth of about 20 m.

Severe operational constraints were implemented and met in order to maintain the integrity of nearby facilities and the ground improvement was completed with no service outages of the Seymour reservoir, and no measured impacts to the nearby downstream salmon hatchery. The use of trial programs prior to and during the construction assisted the design and provided tools for monitoring construction effectiveness and progress. Innovative monitoring and verification methods including detailed analysis of SPT test results were developed.

The combination of EC and DC had the added advantage of overcoming potential limitations of both methods. The EC provided improvements at depths beyond the limits of conventional DC, and the DC effectively removed any arching of materials in the upper bouldery ground, a potential result of the EC program.

The constructed new dam brings the Seymour Falls Dam facility into full compliance with the current Canadian Dam Association earthquake safety guidelines and Provincial and industry standards.

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