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SEISMIC UPGRADE OF THE SEYMOUR FALLS DAM: DESIGN PHASE

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SUMMARY

This paper describes the continuing efforts of the Greater Vancouver Water District (GVWD) in improving the seismic resistance of the Seymour Falls Dam, Vancouver, British Columbia. The GVWD provides a reliable source of quality drinking water to over two-million people in its member municipalities. The Seymour Falls Dam is a key element in the GVWD's network of three watersheds. The existing dam is a 30 m high composite structure consisting of a slab and buttress concrete section, a concrete gravity retaining wall, an earthfill embankment and an extensive upstream impervious earthfill blanket. Due to significant urban development downstream, the provincial dam safety branch has rated the dam a very high consequence structure in the event of a failure. The GVWD recognized that the 40 year old structure does not meet current seismic standards. An interim upgrade was carried out in 1994 to enable the concrete dam to withstand roughly a 1/475 year earthquake event. A construction program, scheduled to commence in 2004, will bring the dam to full compliance with the current Canadian Dam Association earthquake safety guidelines and meet provincial standards, enabling the dam to withstand a Maximum Credible Earthquake. The remedial work includes construction of a new downstream earthfill dam following extensive ground improvement of the new dam foundation, a 100 m extension of a 30 m high concrete gravity retaining wall and further structural upgrades of the concrete dam. One of the key challenges is that all construction activities must be carried out while the Seymour reservoir remains in service to supply drinking water during the four year construction period. Extensive public consultation and negotiations with regulatory agencies were carried out by the GVWD. Several mitigation measures, including a major upgrade to a downstream salmon hatchery, will be implemented by the GVWD to minimize construction impacts.

INTRODUCTION

The Greater Vancouver Water District (GVWD) provides a reliable source of safe, high-quality drinking water to 17 member municipalities. This includes acquiring and maintaining the supply, treating it to ensure its quality and delivering it to the municipalities. Water is collected from three mountainous watersheds: Capilano, Seymour and Coquitlam. It is delivered by an extensive system of 22 reservoirs, 15 pumping stations and over 500 km of supply mains.

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The Seymour Falls Dam provides approximately 40% 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 and at the northern limit of the Lower Seymour Conservation Reserve (LSCR). Shown as the hatched area on Figure 1, the LSCR is open to the public for recreational activities, including cycling, hiking and in-line skating, and education programs. A salmon hatchery is also located approximately 300 m downstream of the dam. This environmentally and socially sensitive setting poses significant challenges to the proposed seismic upgrade construction for the dam.

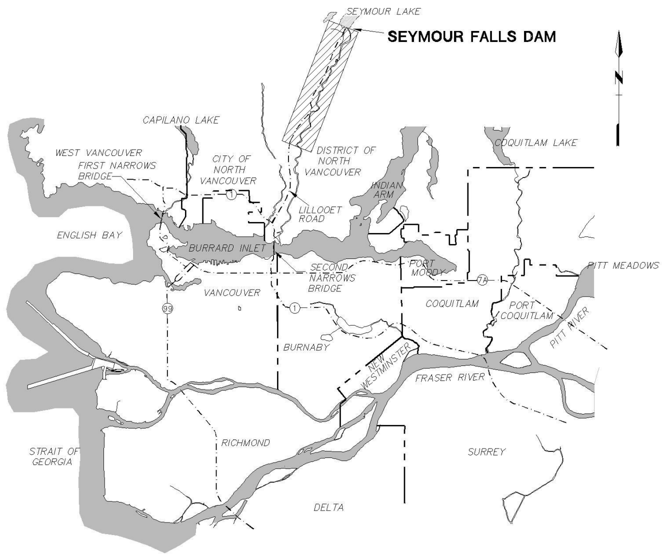

Figure 1: Site Location Plan

In the late 1920s, a 6 m high concrete dam was constructed at the existing Seymour Falls Dam site to supply water to the Vancouver Area. With the rapid growth of population in the Lower Mainland, the GVWD decided in the 1950s to build a higher dam to provide additional water storage. The existing dam was originally designed to have an ultimate height of about 47 m above the riverbed. Stage 1 of the dam was constructed in the early 1960s to a height of about 30 m, with provision for future raising. The Stage 2 dam raise was not built. A photograph and the layout of the existing dam are shown respectively on Figures 2 and 3. The dam is a composite structure consisting of a 235 m long slab and buttress concrete section, a concrete gravity retaining wall, and a 220 m long earthfill embankment and an extensive upstream impervious earthfill blanket.

Figure 2: Photograph of Existing Seymour Falls Dam

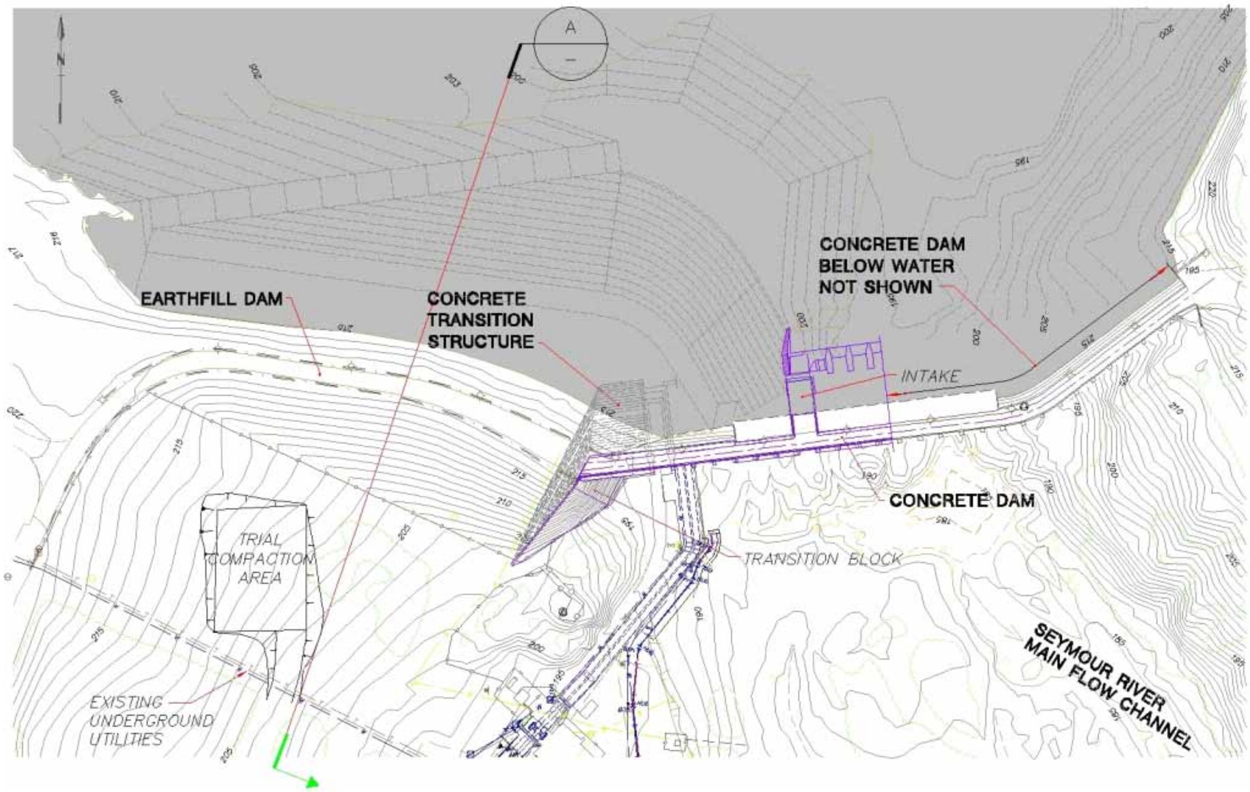

Figure 3: Layout of Existing Seymour Falls Dam

GEOLOGIC SETTING

The Seymour River runs in a valley that has experienced many periods of glaciation. Following the retreat of the glaciers, land levels rose relative to the sea level. Sediments were rapidly deposited in the Seymour Valley by erosion of material from the valley sides and alluvial material transported down the valley. As the land rose and as the thickness of valley bottom deposits increased, rapid deposition of material from the valley sides dammed the river valley in several locations creating lakes. One such lake was formed at the existing dam site by a debris fan known as the Cougar Creek Fan. In both the marine and fresh water depositional phases, the Cougar Creek Fan was deposited contemporaneously with fine-grained, deep-water marine and shallower water lacustrine deposits.

The Cougar Creek Fan, which enters the dam site from the west, covers a semicircular area of radius about 800 m. The earthfill dam is underlain by the distal east end of the Cougar Creek Fan. Projecting out from the east abutment to the edge of the Cougar Creek Fan is a narrow bedrock spur, the upper surface of which is approximately level with the riverbed and on which the concrete dam is founded. The majority of the bedrock foundation for the concrete structures is composed of a hard, fresh to slightly weathered granodiorite. Locally, andesite dikes bisect the rock mass, possibly as a result of an intrusion following an ancient shearing event. Their presence has resulted in a reduction of quality of the surrounding andesite and granitic rock, characterized by closely spaced, slightly open, fractures. The primary discontinuities within the bedrock foundation consist of two orthogonal joints with a lesser-developed subhorizontal joint, which may have developed in response to post-glacial rebound. Also, there are a number of secondary joint sets, which is typical for most granitic formations. The bedrock levels fall steeply to the west and the old rock valley thalweg is buried by over 150 m of Cougar Creek Fan deposits and runs beneath the right abutment of the dam.

The upper 20 m to 40 m of the Cougar Creek Fan is designated as the upper Cougar Creek Fan deposit and contains some very loose granular material. The surficial 18 m of the upper Cougar Creek Fan deposit is extremely coarse bouldery material. Below the bouldery layer the material becomes progressively finer, transitioning to coarse sand at about 30 m depth. More dense preglacial lower Cougar Creek Fan deposits are found below.

Ground water levels at the site are heavily influenced by both the reservoir, which is contained by a partially effective impervious blanket, and by seepage from the local catchment. Generally, water flows south and east below the existing dam around the buried bedrock spur and then to the Seymour River.

DESCRIPTION OF THE EXISTING DAM

The Earthfill Section consists of three major elements, the main embankment, the land blanket and the lake blanket. The main embankment consists of six major zones. A typical cross section is shown on Figure 4. The impervious section of the embankment is an inclined central core composed of compacted clayey silt. Upstream of the core, a transition section of sand and gravel was constructed between the core and the granular shell. The upstream shell is a well graded mixture of sand, gravel, cobbles and boulders. The core extends below the upstream shell and is connected to an impervious blanket of clayey silt at the toe of the dam. The impervious blanket extends about 200 m into the reservoir to tie into rock and into the natural lacustrine silt. The impervious blanket is 1.5 m thick and was constructed in two phases consisting of a "lake blanket" in the pre-1960 reservoir and a "land blanket" over the Cougar Creek Fan sediments. The blanket is not a perfect cut off and there are windows at the upstream end which allows significant seepage to pass beneath the dam. Downstream of the core are fine and coarse filter zones of clean, well graded sand, and clean gravel, respectively. The downstream shell consists of pit run gravel and sand with some cobbles.

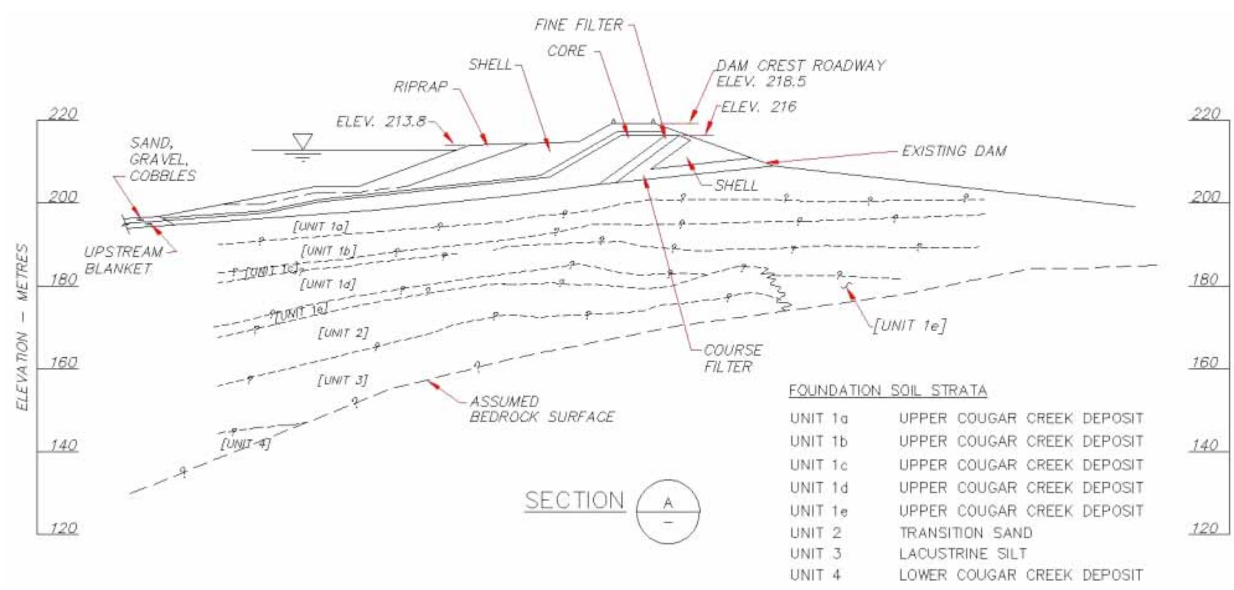

Figure 4: North-South Cross Section

The Concrete Section consists of a slab and buttress dam, comprised of a series of upstream sloping concrete slabs supported on buttress walls spaced at 6.7 m centres. The discharge capacity of the dam is provided by twelve 6.1 m wide 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 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. The dam also includes an intake tower and a small hydroelectric plant. The entire Concrete Section, including the transition block and the concrete gravity wall, is founded on bedrock.

DAM SAFETY ASSESSMENT

Historically, it is known that moderate earthquakes periodically occur in the coastal regions of southwestern British Columbia including the Lower Mainland. However, during the 1980s, studies indicated that the potential for a moderate or large earthquake in the region is much greater than previously anticipated. Rogers [1] postulated that an earthquake along the Cascadia subduction zone would result in an earthquake of roughly M8 to M9. Paleoseismic evidence along the west coast of Washington and Oregon suggest that such major seismic events have a recurrence interval of several hundred years. Although these events are predicted to occur 200 to 300 km from Greater Vancouver, the long duration at even a relatively low acceleration would still result in substantial damage to the region's lifelines. These findings were accentuated in 1989 with the occurrence of the Loma Prieta earthquake in San Francisco. Even though seismic events are over relatively quickly, the loss of lifeline infrastructure such as water supply dams, can cause not only major public health and safety problems, and disrupt normal activity in the region for weeks or months, but also result in many deaths and subsequent property and environmental losses.

Although the existing Seymour Falls Dam is in excellent condition, it does not meet current seismic standards. The provincial dam safety regulation requires all dam owners to be responsible for the safety of their structures and carry out necessary updates to meet current design standards. Due to significant urban development downstream both in terms of resident population and property development, the provincial dam safety branch has rated the dam a very high consequence structure in the event of a failure. Based on the Canadian Dam Association (CDA) – Dam Safety Guidelines [2], this will require the dam to resist a Maximum Credible Earthquake (MCE) established by the deterministic method or an earthquake with an annual exceedance probability of 0.0001 derived by the probabilistic method. This is also consistent with GVWD's internal seismic design criteria, which require all dams and other important facilities to be designed for the MCE event. For the slab and buttress section of the dam, the Federal Energy Regulatory Commission (FERC) - Engineering Guidelines for the Evaluation of Hydropower Projects [3], which specifically address the review of buttress dams, was used to supplement the requirements of the CDA Guidelines.

The GVWD took the initial steps towards upgrading the Seymour Falls Dam in the late 1980s by retaining Klohn Crippen of Vancouver, British Columbia to carry out engineering assessment and conceptual design. Their assessment indicated that the Concrete Section was more deficient than the Earthfill Section. 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 level of earthquake with a return period of 475 years. No upgrade was carried out on the Earthfill Section at that time since the Earthfill Section was deemed to be stable under the DBE.

In 1998, the GVWD tendered the detailed design of the MCE seismic upgrade of the Seymour Falls Dam. The contracts for the Earthfill Section and the Concrete Section were won respectively by Klohn Crippen and Acres International Limited, both of Vancouver, British Columbia. Concurrently with the detailed design work, the GVWD started negotiating with the federal, provincial and municipal agencies as well as the Seymour salmon hatchery to obtain approval for the seismic upgrade. After a lengthy negotiation, the GVWD agreed to and conducted a major upgrade to the hatchery infrastructure in 2003 to eliminate potential construction impacts to the hatchery. Construction will pose significant environmental and social challenges at the Seymour Falls Dam. The GVWD started conducting extensive public consultation with all stakeholder groups in 1998. Several public mitigation measures to be implemented during construction include maximizing the re-use of excavated foundation materials for the embankment construction, utilizing existing gravel pit within the LSCR and batching concrete on site to minimize import of material and hence truck traffic and completing a new $3 million recreational pathway to minimize impacts on the recreational users within the LSCR.

The detailed design of the MCE upgrade was completed in 2003. The seismic upgrade construction was tendered in late 2003 and awarded to Peter Kiewit Sons Co. in January 2004. Construction is expected to commence in March 2004 and will be completed in December 2007.

Methodology used in the assessment and identified deficiencies are discussed below.

SEISMIC HAZARD ASSESSMENT

The seismicity of the site was evaluated in 1998 by BC Hydro International assisted by the Geological Survey of Canada [4]. The controlling events for the site are local random earthquakes, local deterministic sources, the closest being the Britannia Fault, and deep seated intraplate events beneath Georgia Strait.

Recommended design response spectra for these events for use in dam remediation were

- M7.5 intraplate earthquake: Deterministic 84th percentile.
- M6.5 Britannia Fault local earthquake: Deterministic 50th percentile.
- Random local earthquake: Uniform Hazard Response Spectrum (UHRS) with a 10^{-4} annual exceedance frequency.

Target parameters used for earthquake time history scaling are shown in Table 1.

Table 1 Target Parameters for Design Earthquake

Earthquake	Parameter	Target Value
M7.5 Georgia Strait	• Peak Ground Acceleration (PGA) • Peak Ground Velocity (PGV) • Peak Ground Displacement	0.35 g 24 cm/s 9 cm
M6.5 Local Event	• Peak Ground Acceleration (PGA) • Peak Ground Velocity (PGV) • Peak Ground Displacement	0.5 g 30 cm/s 5 cm to 10 cm

The time histories were selected based on earthquake magnitude, distance and peak ground acceleration, and then scaled to fit the target spectrum over the full period range as well as to the target values in Table 1. At the onset of the analysis work, it was determined that three earthquake time histories would be selected for each design earthquake source. For those sources where a sufficient number of time histories were not available, the time histories with the best possible fit were selected and deviations from Bolt's criteria, interpreted by Idriss [5], were noted to assist the final selection of records for use in analysis.

For the Concrete Section, the cross-valley predominant period is 0.07 seconds; the upstream-downstream predominant period is 0.03 seconds.

EARTHFILL SECTION

Earthfill Design Criteria

Basic criteria for the upgrade appropriate for the very high consequence structure were for safety under the Maximum Credible Earthquake (MCE) and the Probable Maximum Flood (PMF). Additional project criteria included the requirement for uninterrupted operation of the dam and maintenance of current levels of earthquake and flood protection during construction. The following key criteria were identified:

Table 2 Design Criteria

Item	Description	Value
Maximum Reservoir Level	Summer PMF with stop logs	El. 217.36 m
Normal Reservoir Level	Summer with stop logs	El. 214.74 m
Slope Stability Analysis: Safety Factor against Slope Failure	Static Rapid draw down Post earthquake liquefied ground	1.5 1.3 1.1
Cyclic Pore Pressure: Safety Factor against Liquefaction	i) $\geq$ 1.4 ii) 1.4 to 1.1 iii) $\leq$ 1.1	i) Use full effective stress strength and static water table ii) Interpolate between i) and iii) iii) Residual strength (Seed [6])
Deformation	• Minimum post MCE dam crest • Horizontal deformation	El. 217.36 m $\leq$ 1 m or 10% of layer thickness

Foundation and Liquefaction Assessment

In Situ SPT

For the earthfill dam, the main issue is the liquefaction susceptibility of the coarse granular deposits in the Cougar Creek Fan. Extensive site investigations were completed between 1990 and 1998 to assess if the coarse granular Cougar Creek Fan deposits were susceptible to liquefaction under design earthquakes. A considerable problem in assessing the liquefaction potential was the difficulty of obtaining meaningful SPT values because of the influence of very coarse particles up to boulder size.

Investigations used conventional SPT, Becker Penetration Test (BPT) and shear wave velocity testing among other methods and considerable effort was made to correlate BPT with conventional SPT done in accordance with ASTM 1586-84 [7]. Methods proposed by Harder [8] and Sy [9] were used for the correlations. Eventually it was concluded that, although the Harder BPT to SPT conversion gave results closer to ASTM SPT results than Sy's method, neither BPT conversion gave a reliable correlation to SPT. Although, no conclusive reason for the poor BPT to SPT conversion was identified, Klohn Crippen believes it was due to formation of plugs of coarse material being pushed in front of the Becker closed end casing. These plugs are not accounted for in the analysis but would form in a highly variable way in the loose heterogeneous Cougar Creek fan deposits.

Some pre-construction penetration testing was done and all penetration testing during construction will be done using conventional ASTM SPT testing techniques and a mud rotary rig as follows:

- All tests are instrumented to measure hammer efficiency;
- The drill bit has specially designed jets to direct mudflow upward and not at the base of the hole;

- SPT blows per 25 mm are counted and summed where possible over the penetration range 0.15 m to 0.45 m in the normal manner;
- If full penetration is not achieved an SPT value is calculated as 4 times the lowest 3 consecutive blows recorded over 25 mm increments; and
- Drilling is carefully advanced through cobbles and boulders and SPT started immediately when finer material is encountered.

In this way a typical 20 m deep hole with a target SPT spacing of 1 m can been completed in about 5 days and normally about 10 to 15 useable SPT values can be obtained.

SPT's are reduced in accordance with the procedure by Youd [10] to derive $(N_1)_{60ECS}$ (normalized equivalent clean sand) values for use in liquefaction assessment. An important component of the $(N_1)_{60ECS}$ estimate is obtaining fines content. Despite the loose, coarse nature of the deposit, good sample recovery has been achieved by wrapping the split spoon core catcher in plastic cling film as suggested by Idriss [11]. Using the noted method and discarding blows impacted by coarse particles, a site wide average $(N_1)_{60ECS}$ of about 11 blows per 0.3 m was estimated for the existing Cougar Creek fan granular deposits.

Liquefaction Assessment

Soil liquefaction was assessed in two ways firstly using a conventional Seed simplified analysis, as modified by NCEER and reported recently by Youd [10]. For this approach SHAKE was used to calculate the cyclic stress ratio. Secondly a self triggering approach by Beaty [12] was applied using the 2D finite difference program FLAC. Both total and effective stress methods were analyzed in FLAC.

The SHAKE based simplified approach indicated extensive zones of liquefaction with almost all the Cougar Creek fan granular deposits failing to meet the minimum safety factor of 1.1 against liquefaction. The FLAC analysis included an incremental approach to softening of soil by keeping count of the number of cycles to cause liquefaction. Thus soil behaviour in the FLAC model was influenced by material which liquefied or softened earlier in the earthquake. Although FLAC predicted less extensive liquefaction than the simplified method, the consequences of liquefaction still included dam crest settlements of nearly 4 m, likely failure of blanket and core possibly leading to piping.

As a general observation the $(N_1)_{60ECS}$ required to achieve a safety factor of > 1.1 against liquefaction is about 20 blows/0.3 m, which is about twice the existing site average blow count.

Once it was determined that ground improvement was necessary as part of the seismic upgrade, a number of ground improvement techniques were considered including the following:

- Vibro replacement
- Compaction piles
- Dynamic compaction
- Soil replacement
- Explosive compaction

The selection of the best option was driven by a number of criteria including ground conditions, operational impacts, degree of improvement required and environmental considerations.

Options for upgrading the existing earthfill dam to survive the MCE included improving the existing structure, placing fill or undertaking other stabilizing measures upstream, or modifying the downstream geometry. The preferred upgrade option was to build a new dam downstream on a compacted foundation.

The new dam provides a stabilizing buttress to the existing structure and also connects the existing and new dam cores to provide continuity with the reservoir blankets.

Ground Compaction

Remediation of the dam requires compaction of the ground to prevent earthquake induced liquefaction in zones critical to the dam stability. To assist in the design of the work, a trial compaction program using Dynamic Compaction (DC) with an input energy of 420 tonne-meters and Explosive Compaction (EC) was completed in 1998 within the footprint of the new dam. The trial DC program achieved $(N_1)_{60} >$ 25 blows/0.3 m down to at least 10 m depth. EC trial achieved $(N_1)_{60} > 20$ 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. Post-compaction FLAC analyses showed that the overall effect of the DC/EC program was sufficient to achieve a stable dam post MCE.

Consequently a full scale program of DC and EC for the current construction phase was designed as follows:

- Pre- excavate the upper 10 m of the site;
- Conduct EC from 10 m to 20 m depth below the excavation base; and
- After EC, conduct DC on the base of excavation.

The EC/DC is designed to achieve the following;

- Meet site Peak Particle Velocity limits which vary from 13 mm/s at the sensitive fish hatchery to 150 mm/s for the toe of the existing earth dam;
- Meet settlement limits at structures; generally settlement of less than 25 mm was specified;
- Meet pore pressure rise and water body overpressure restrictions of key points; pore pressure rise was controlled by slope stability analysis; overpressure was to be 50 kPa or less at spawning pools in the Seymour River;
- Achieve a pore pressure rise ratio of 0.9 or higher in the active EC panel; and
- Meet restrictions for chemical residuals at surface discharge, <25.7 mg/L ammonia, < 200 mg/L nitrate.

The following site specific equations were developed during the trial program. The equations, in general are empirical modifications to published forms of equations by Narin Van Court [13].

Peak Particle Velocity PPV (mm) $= (R_H/W^{1/2})^{-1.5} \times 2500$
Regional Pore Pressure Ratio (PPR) $= 1.25 - 0.2\,(R_p/W^{1/3})$
PPR in Blast Pattern $= 1.35 - 0.15\,(R_p/W^{1/3})$

where:
R_H is hypocentral distance (m)
R_p is plan distance (m)
W is maximum charge weight per delay, Kg
PPR is the ratio of increase in pore water pressure to pre-blast in-situ vertical effective stress.
PPR = 1.0 represents complete liquefaction.

Peak overpressure limits are assumed to be met if PPV limits are not exceeded. Chemical residuals concentrations were based on field measurements.

To meet safe settlement limits, an empirical approach based on measured performance during the trial EC Program was used. The horizontal distance to 50 mm settlement is calculated by projecting a line to the surface at 2V:1H from the base of the closest blast hole and adding 4 m. The distance to 25 mm settlement is calculated the same way but an additional 6 m is added. EC is to be completed in three passes over a series of nominally 30 m by 30 m panels over the depth interval 10 m to 20 m below excavated ground level and using a charge weight of 0.10 to 0.15 kg/m^3 of treated ground.

DC was designed using similar settlement and PPV relationships. For full scale field compaction an energy of 575 tonne/m per drop and total energy of 550 tonne m/m^2 were selected based primarily on Mayne [14].

Target SPT $(N_1)_{60ECS}$ values for the work, based on depths below the compaction surface, are:

0 to 10 m	DC zone	25 blows/0.3 m
10 to 14 m	EC Zone	18 blows/0.3 m
14 to 20 m	EC Zone	13 blows/0.3 m

Because of the difficulty and expense of obtaining SPT results, the EC effectiveness will be primarily evaluated by achieved settlements, which are expected to be about 5 % (i.e. 0.5 m) of the depth of EC treated ground.

Embankment Dam Design

Following the completion of ground improvement, the concrete gravity wall will be extended to form the left abutment of the earthfill dam. The earthfill dam will then be constructed over the densified ground. The dam section is a conventional impervious cored embankment using local clayey silt with wide granular shells and wide filters and drains. The impervious core will be extended to tie into the existing dam core thus providing a continuous low permeability barrier, connecting to the existing lake and land blankets. The proposed seismic upgrade for the Earthfill Section is shown schematically on Figure 5.

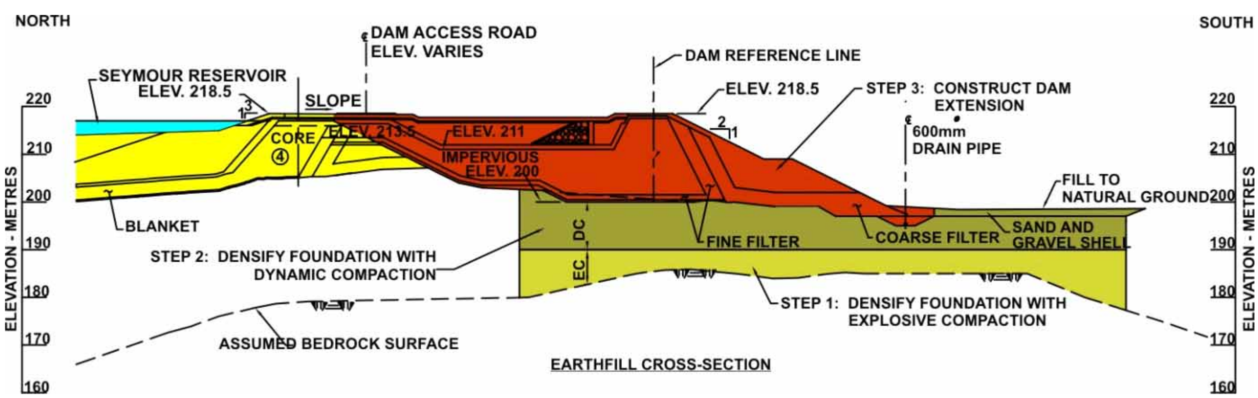

Figure 5: Proposed Ground Compaction and Embankment Extension

Dam performance will be monitored by a series of electric and pneumatic piezometers terminated in an instrument house with capability for data logging and telemetry. A series of pressure relief wells are included along with a network of foundation drains to quickly relieve earthquake induced pore pressures and to deal with any long-term increase in seepage due to earthquake damage to the impervious lake blanket.

CONCRETE SECTION

Concrete Design Criteria

Primary criteria used for the Concrete Section is as noted in the Earthfill Section Part. Additional criteria relating specifically to the Concrete Section are as follows:

- Sliding Factor of Safety of the buttresses for the MCE loading condition equals 1.0, with an allowance for 5.0 mm of slip displacement.
- All computed buttress stresses to be compared against allowable tensile and compressive values determined from detailed geotechnical investigations.
- Remedial measures will be designed using current versions of the National Building Code, CSA/CAN A23.3 [15] and CSA/CAN S16.1 [16].
- Stability design of the new structures will be in accordance with the CDA Guidelines. Performance indicators will be checked to ensure that under all loading conditions, minimum Target values are met.

Analytical Approach

Dynamic analysis of the concrete dam structures was carried out using the SAP2000 finite element computer program, appropriate MCE earthquake time histories, and the SADSAP post processor. Several local models and an overall global three-dimensional finite element model were developed to assess the Concrete Dam structure. Finite element models analysis (FEA) were generated using state-of-the-art graphical mesh generation software, with the finite element program SAP2000 used for the stress analyses.

In addition to the global model, which was primarily used to assess the dam in the cross-valley direction, several local models of the concrete portion of the dam were constructed to assess the dam in the upstream-downstream and cross-valley directions. Each of the local FEA models was of significantly greater detail than those for the global model. Because of the time and analysis effort that would be required to construct, analyze, and perform the post-processing of a local model of each buttress in the upstream-downstream direction, a time and cost saving measure was used. An analysis method was proposed that would allow extension of the FEA results of those buttresses analyzed to the remaining buttresses that were not. Slip displacement calculations were performed for each local model. The results were used in conjunction with a detailed pseudo-static stability, upstream-downstream, analysis of each buttress. Ratios of sliding stability safety factors of non-FEA modeled buttresses to those of modeled buttresses were used to estimate the amount of “slip” that would occur at each buttress during the MCE event.

The slip displacement analysis was based on the methods of Mir [17] and was proposed for use at Seymour Falls Dam as an analysis tool that would potentially limit the rehabilitation measures needed to satisfy the MCE condition. The time history results were processed in a manner that allowed each time step of the analysis to be reviewed with regard to the driving and resisting forces and calculating the time step slips that would occur during periods when the sliding safety factor was less than one. The accumulated time step slips that occurred over the entire time history resulted in an overall slip (calculated at each buttress) which was compared to an accepted maximum value of 5.0 mm.

For all of the local buttress models, snap shots of the time history analysis of the vertical stress at the base of the dam and of the principal stresses throughout the buttresses were reviewed. In all cases, it was determined that the maximum principal stress that resulted from the MCE event was significantly less than the allowable dynamic tensile strength of the concrete. As well, for all buttresses, the maximum compressive stresses found at the base of the buttresses were significantly less than the allowable stress.

Detailed Design of MCE Upgrades

Detailed design of the required works will allow Seymour Falls Dam to safely withstand the MCE. A brief description of the various design elements follows.

Concrete Dam Buttresses Several, detailed local finite element analysis (FEA) models, comprised of one or two buttresses complete with faces slabs were developed for key, selected buttresses to analyze the concrete section for the MCE event. The FEA results were input into a post processing program which calculated the accumulated slip displacement that would occur over the range of the time history of the design seismic event. The results of the selected FEA models were extended to all of the dam buttresses with the use of pseudo-static analyses.

To accommodate this potential slip, it was determined that some of the uphill, left abutment, buttresses could potentially slide laterally downhill if allowed to slip in the downstream direction. To prevent the catastrophic failure, stabilization blocks will be installed on the downhill side at the toe of three critical buttresses. These reinforced concrete blocks will be doweled into bedrock.

Based on the results of the detailed FEA work, the bending and shear stresses were assessed in the upstream face slabs and critical areas of the concrete dam buttresses. Adequacy of the present dam configuration, including shear walls, under MCE loading was reviewed using detailed design checks comparing the actual stresses against allowable values. Maximum stress levels, under MCE loading, in the buttresses and face slabs were found to be within acceptable levels.

Through the use of the slip displacement calculations, it was determined that no major rehabilitation work was required to the buttresses. Due to the short duration and small number of critical acceleration exceedances, the computed slip displacements for all of the buttresses were relatively small, all less than 4.0 mm. This was considered acceptable, thus determining that no further remedial action was required for the buttresses.

A lesser detailed global model, which was comprised of all concrete elements between and including the transition block, buttresses, face slabs, bridge deck and east abutment, was constructed to analyze the dam in the cross-valley direction. Deficiencies in the bridge deck connection to the buttresses and spillway piers were found. It was also determined that the current configuration of the concrete dam provided sufficient stiffness to allow the dam to safely resist cross-valley MCE loading without the requirement of the installation of additional shear walls.

Transition Block The transition block was modeled within the global FEA model and included the surrounding fill as originally detailed and currently in place at the site. Slip displacement calculations proved the structure stable with no further remedial or stabilizing measures required.

Gravity Wall Extension The need for a Gravity Wall Extension (GWE) arose during the course of the design work as a result of the required construction of a new earthfill dam located immediately downstream of the existing Gravity Retaining Wall, which is located downstream of and connecting to the transition block. The location, length and shape of the GWE were optimized with regard to cost while meeting the requirements of the earthfill dam and the CDA [2]. Stabilization measures in the form of post tensioning a portion of the existing gravity retaining wall was required to allow it to meet the design requirements.

Upstream Shear Walls With the use of current powerful finite element programs, which allowed accurate modeling techniques to be used, Acres were able to demonstrate that the existing structure provided sufficient lateral stiffness to withstand the MCE and that the installation of additional upstream shear walls was not required.

Spillway and Intake Bridges Design checks of the spillway and intake bridges using stresses from the global FEA analyses, along with those from a local, detailed bridge deck model were performed. The existing pinned connection between the bridge deck and the top of buttress or spillway portal frame was reviewed and found to be inadequate in preventing the deck elements from sliding relative to the supporting element below. Remedial measures in the form of additional above or below deck “pinning” of the existing precast concrete deck panels were designed. The upgrade is primarily comprised of the installation of shear dowels (above deck into buttresses) or connection brackets and adhesive anchors (below deck into spillway bridge portal frames).

Spillway Piers The connecting elements of the spillway piers at the spillway rollway interface were found to be overstressed under both cross-valley and upstream-downstream MCE loading. To minimize the amount of inertial load that would be transferred from the concrete bridge deck to the piers, a seismic isolation device was modeled in the FEA model. The two-dimensional “spring” was located at the base of the spillway bridge portal frames and eliminated all horizontal forces from the deck from being transmitted into the spillway piers below. This remedial measure, although limiting the horizontal inertial loads, did not totally eliminate the overstress that occurred at the spillway pier/rollway interface.

The remedial works consist of the installation of spillway bridge portal frame seismic isolators and the post tensioning of the spillway piers. The isolator system consists of the installation of an 80 mm thick seismic isolation pad beneath all of the portal frame legs. The pad, installed in a cut out segment of the existing portal frame leg, is contained within a steel frame that is connected with adhesive anchors to the portal frame leg above and the spillway pier below. The isolator allows movement of the portal frame leg in both horizontal directions, thus restricting any horizontal load from being transferred from any structure element located above the isolator. With the isolators in place, and to provide stabilization to the portal frames, upstream and downstream steel (HSS section) cross bracing connecting the upper and lower portions of alternating portal frames were required.

The pier upgrade consists of the installation of 30, double corrosion protected, 26–15mm diameter 7 wire, low extraction Grade 1860 MPa strand post-tensioned anchors installed from the top of the spillway piers and embedded, or bonded, within the lower buttresses. The anchors do not extend into bedrock. The total amount of pre-compressive stress was calculated to offset the maximum tensile stress that develops at the pier/buttress interface during either the cross-valley or upstream-downstream MCE event. The spillway pier strengthening is shown on Figure 6.

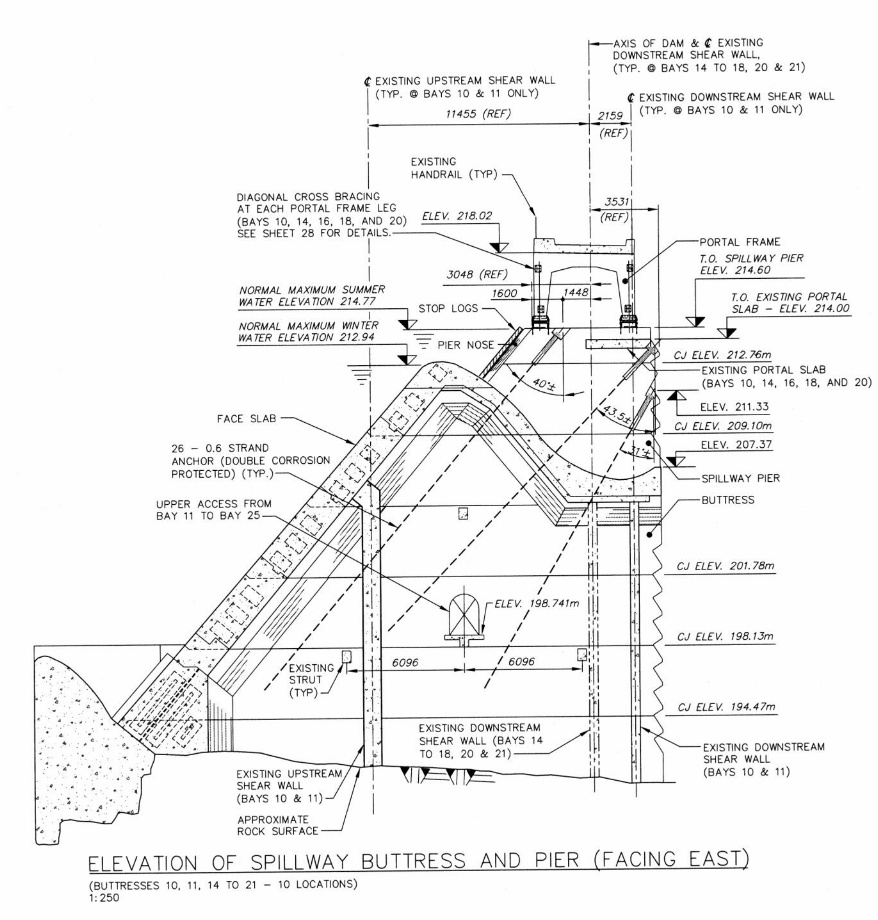

Figure 6: Spillway Pier Strengthening

Intake The GVWD required that the Intake Structure should remain operational following the MCE event to allow the water supply system to provide uninterrupted water service. A detailed FEA model was constructed and the analysis was performed in both the cross-valley and upstream-downstream directions. Final results confirmed through innovative FE analysis, Building Code interpretation and design assumptions led to reduced rehabilitation requirements (as compared to previous recommendations) consisting of the installation of relatively minor, localized buttress strengthening measures. Two areas of the intake structure will be strengthened:

- The upper 1200 mm of the buttress walls (Buttress 12 and 13), immediately below the screen house building was found to be overstressed in shear. Steel plates will be anchored to both faces of each intake buttress with 20 mm diameter adhesive anchors;

- The lower triangular shaped portion of each buttress, immediately below and downstream of the face slabs, was also determined to become overstressed in shear during an MCE event, which would likely result in extensive cracking. This section of buttress acts as a bulkhead and is subjected to full reservoir pressure. It was determined that the cracking would lead to extensive leakage and possible failure of these areas which could lead to significant damage to the facilities. To prevent a catastrophic type failure, strengthening measures in the form of the installation of steel beams, anchored to the adjacent substructure or walls will be installed to reinforce the buttresses to ensure local failure does not occur.

CONCLUSIONS

Seymour Falls Dam is an important and integral component of the water supply system for Greater Vancouver. Careful planning, innovative engineering, effective pubic consultation and early negotiation with regulatory agencies all lead to the cost-effective and sustainable design. The upcoming seismic upgrade construction will bring this dam to full compliance with current seismic design and provincial standards.

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