TECHNICAL MEMORANDUM

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GEOTECHNICAL DESIGN RECOMMENDATIONS
AUXILIARY SPILLWAY, LOWER COLLIERY DAM
NANAIMO, BC

This technical memorandum summarizes the subsurface conditions encountered at the site based on our recent geotechnical investigation, and provides the recommended geotechnical parameters as input to the structural design of the proposed auxiliary spillway structure.

This technical memorandum should be read in conjunction with the **"Important Information and Limitations of this Report"** which is attached following the text of this memorandum. The reader's attention is specifically drawn to this information as it is essential for proper use and interpretation of this report.

1.0 SEISMIC CONSIDERATIONS

1.1 Background and Proposed Auxiliary Spillway

The failure consequence classification of the Lower Dam is 'Very High' and Golder understands from discussion with the City of Nanaimo that the auxiliary spillway is not defined as a 'post-disaster structure'. Canadian Dam Association (CDA) Dam Safety Guidelines (2007) identify that the earthquake design ground motion (EDGM) should be selected based on the consequences of dam failure. Based on the failure consequence designations of the lower dam (very high) and middle dam (high), the earthquake levels for use in deterministic assessments for the dam structure are considered to be a 1-in-5,000 year event and a 1-in-2,500 year event, respectively (Table 6-1, CDA Dam Safety Guidelines, 2007).

The auxiliary spillway is designed to provide additional spillway capacity without impacting the existing spillway. It would only be activated in the event of a storm event and associated high water levels within the reservoir. This secondary spillway will consist of a labyrinth weir structure located to the south of the existing spillway and outside of the footprint of the existing dam and spillway. The labyrinth weir has plan dimensions of approximately 13.8 m by 13.0 m with a height of 3 m. Downstream from the labyrinth weir, the water discharge flows beneath a bridge structure, and then along a riprap protected, tapered open channel into a 6 m wide open channel before discharging into Harewood Creek.

The primary function of the auxiliary spillway is to provide secondary flood routing during a storm event. It is not considered a substitute spillway, and is not directly or indirectly connected to or integral with the existing spillway and dam structure as an appurtenant structure. Further, CDA Guidelines require that designs are carried out to meet the design seismic event and the design storm event, as two independent events (i.e. not concurrent).

1.2 Seismic Design Parameters

Typically, dams and appurtenant structures are located in remote areas and require site specific seismic hazard assessments to be carried out (CDA Dam Safety Guidelines, 2007). Since the auxiliary spillway is located within an urban region close to the Lower Mainland, is not an appurtenant structure of the Lower Dam (not connected to, or contiguous with, the Lower Dam), and taking into consideration the limited size of the auxiliary spillway (less than 3 m in height) as well as its use solely for secondary flood routing, Golder recommends that seismic hazard parameters and uniform hazard spectra seismic events comparable to that for other commercial and industrial structures designed in conformance with the current (2010) National Building Code of Canada (NBCC), be adopted for use in design of the auxiliary spillway. Following discussions with Herold Engineering Ltd., Golder recommends that a 475 year return period seismic event is adopted for serviceability limit state (SLS) design and a 2,475 year return period seismic event is adopted for ultimate limit state (ULS) design of the structure.

Site-specific seismic motion parameters for the subject site were obtained from the National Resources Canada website (*http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-eng.php*) and are summarized in Table 1 (see also Attachment 1). The ground motion parameters have been established for two return periods that correspond to a 10 % probability chance of exceedance in 50 years (equivalent to 1 in 475 year event) and 2 % probability of exceedance in 50 years (equivalent to 1 in 2,475 year event). They correspond to Class C ground motions, for soil profiles with an average N60 count of the upper 30 m greater than 50 blows per 300 mm.

Table 1: Site Specific Probabilistic Firm-Ground Motion Parameters (Site Class C)

Return Period	PHGA	Sa (0.2)	Sa (0.5)	Sa (1.0)	Sa (2.0)	Approximate Magnitude
475 Years (10% Probability of exceedance in 50 years)	0.268	0.532	0.357	0.181	0.089	M6.9
2,475 Years (2% Probability of exceedance in 50 years)	0.499	1.013	0.692	0.351	0.178	M7.0

Note: PHGA refers to peak horizontal ground acceleration; Sa refers to spectral acceleration for a given period.

These seismic hazard parameters are derived from the probabilistic hazard model developed by the Geological Survey of Canada (GSC). This model is based on the results of extensive work conducted by the GSC. This approach is consistent with industry standards and has been adopted in the National and BC Building codes for design of buildings and structures.

Golder did not carry out a detailed seismic hazard assessment for the auxiliary spillway which considers the proximity of known or potential faults to the site with recorded seismicity over many years. We consider the probabilistic approach described above to be suitable and adequate for the design of the proposed auxiliary spillway. However, Golder's technical memorandum on 'Dynamic Soil-Structure Interaction Analysis of the Colliery Dam, Nanaimo', dated July 16, 2014, summarized an assessment of the seismic behaviour of the Lower Colliery Dam when subjected to the shaking levels corresponding to the 10,000-year (equivalent to the Maximum Credible Earthquake (MCE). "Firm-ground" peak horizontal accelerations applicable for a return period of 10,000-years were estimated by combining all available data on PGA as a function of the annual probability of exceedance. Based on the available data, the PGA that corresponds to a return period of 10,000-years is established as approximately equal to 0.8g. The NBCC PGA values for the more frequent (475-year and 2,475-year) earthquake events are considered to be consistent with that for the much lower probability MCE event.

1.2.1 Ground Motions and Foundation Factors for Spillway

The ground motions provided in Table 1 are representative of a firm-ground site; that is, a site with very dense soil or soft rock in the upper 30 m of the soil profile.

Since 2006, the BC Building Code (BCBC) has adopted the use of foundation factors that are dependent on local site soil conditions, shaking level, and site period. The effects of local site conditions are characterized based on the average strength properties of the soil/rock in the upper 30 m, and six different site classes varying from Site Class A to F have been identified. For a given site class, the effects of shaking level and period are incorporated via the short-period and long-period foundation factors Fa and Fv defined in Tables 4.1.8.4B and 4.1.8.4C of the BCBC, respectively.

Based on the results of the investigation, the average standard penetration resistance, N60, in the upper 30 m of soil column is in excess of 50 blows per 300 mm at the site. Therefore the site is considered to be Site Class C and the corresponding site-specific short period and long-period foundation factors, Fa and Fv, are 1.0.

The subsurface conditions are considered to have a very low potential for liquefaction for both design seismic events (1 in 475 year and 1 in 2,475 year return periods).

2.0 GEOTECHNICAL ASSESSMENT

2.1 General

Based on the results of the August 2015 geotechnical investigation, the site of the proposed labyrinth weir and bridge spanning the auxiliary spillway is underlain by compact sand and gravel soils to depths ranging from 1.4 m to 2.6 m below ground surface. Compact sand deposits, with minor amounts of silt, underlie the coarser granular soils and extend to depths of about 3.3 m to 4.9 m at individual testholes, which in turn are underlain by soft to stiff silty clay, with varying proportions of sand. A dense to very dense sand material, with varying amounts of silt and gravel, was encountered underling the fine-grained deposits within BH15-01 and BH15-02 at depths of 7.0 m and 7.2 m, respectively.

Very dense glacial till-like soils were encountered underlying these deposits within BH15-01 and BH15-02, with both boreholes terminated within these glacial deposits at depths of 17.4 m and 12.8 m, respectively. The silty clay layer within BH15-03 was underlain by an approximate 0.3 m thick compact silty sand layer at a depth of 5.3 m. Weathered conglomerate bedrock was encountered within BH15-03 at a depth of about 5.6 m and was underlain by fresh conglomerate at 6.1 m depth that extended to the borehole termination depth of roughly 8.8 m.

Groundwater was encountered at depths of 3.6 m and 4.1 m below ground surface within BH15-01 and BH15-02, respectively.

2.2 Foundation Design Recommendations

Based on existing design information, it is understood that the proposed labyrinth weir and bridge will have a top of slab elevation of 69.1 m. The original ground surface elevation at the location of the labyrinth weir and bridge ranges from approximately El. +74 m to El. +75 m elevation.

Since the depth from existing ground surface to the top of slab elevation ranges from about 5 m to 6 m across the site, it is anticipated that sand, with varying amounts of gravel and silt, and/or silty clay, with varying amounts of sand, will be encountered at and extend up to 1.5 m below the anticipated elevation of the underside of the slab of the labyrinth weir.

Similar subsurface conditions are anticipated at the underside of the slab over at least a portion of the bridge structure. However, conglomerate bedrock will likely be encountered close to the transition zone of the bridge structure and the channel, and may underlie portions of the bridge structure at or close to foundation grade. The extent of conglomerate at the site is not known since it was only encountered within the footprint of the weir and bridge at borehole BH15-03, at an approximate elevation of El. +68.2 m. During the September 2015 test pit investigation within the downstream channel, inferred bedrock was encountered at elevations ranging from approximately El. +68.6 m to El. +67.9 m.

The soft fine-grained soils are compressible and, as such, are not a suitable subgrade layer. Similarly, the sand material encountered at the proposed foundation and slab elevation of the weir within BH15-01 is saturated, with variable silt content, and is expected to be highly susceptible to disturbance and difficult to prepare suitably.

Till-like soils were encountered underlying the sand and clayey silt at an approximate elevation of El. +67.7 m, within borehole BH15-01, and approximately El. +67.6 m, within BH15-02. Based on the limited depth to the till-like soils from the proposed top of slab elevation, we recommend and have assumed that the perimeter footings of the labyrinth weir and bridge will be founded on till-like soils and/or conglomerate bedrock, with a minimum embedment depth of 0.3 m.

For the structural slab located within the weir and bridge footprint, we recommend and have assumed that soft or loose and saturated soils and any disturbed materials will be subexcavated and this slab will be founded on well compacted structural fill that has been placed on top of the prepared till-like subgrade and/or bedrock subgrade to the underside of the slab.

The very dense, till-like soils and the fresh conglomerate encountered are considered suitable for support of the proposed structure. The recommended bearing resistance of the intact sedimentary bedrock exceeds that of the till-like soil. However, for design purposes, we recommend that a single bearing capacity is adopted for the labyrinth weir and bridge structures.

The recommended allowable bearing resistance under static loading for the spillway structure and the recommended friction coefficients for the interface contact between the structural slab and the foundation soils are presented in Table 2, below.

Table 2: Recommended Foundation Design Parameters for Footings

Parameter	Value
Allowable bearing pressure (Factor of Safety = 3)	617 kPa
Ultimate Limit State (ULS) Factored Bearing Resistance – Resistance factor Φ = 0.5 (Canadian Foundation Engineering Manual 2006)	925 kPa
Base Friction Coefficient (tan δ) – For concrete poured over crushed gravel base	0.55
Base Friction Coefficient (tan δ) – For concrete poured over non-plastic silt or stiff clay/silty base	0.35
Base Friction Coefficient (tan δ) – For concrete poured over sedimentary bedrock base	0.70

The modulus of subgrade reaction of the subgrade material may be used to estimate its elastic deformation characteristics. It is important to note, however, that the modulus of subgrade reaction is not a fundamental soil property. In addition to the deformation characteristics of the subgrade, it is dependent on the geometry and stiffness of the structural member in contact with the subgrade material. We therefore recommend the following relationship be used in the determination of the modulus of subgrade reaction for structural analysis of a slab.

$$k = \sqrt[3]{\frac{E_s}{E_c}}\frac{E_s}{(1-{\nu_s}^2)h}$$

Where,

k = Modulus of subgrade reaction (kPa/m);

E_s = Young's modulus of soil subgrade (kPa);

E_c = Young's modulus of structural element slab (kPa);

ν_s = Poisson's ratio of soil subgrade, and

h = thickness of slab (m).

The following range of soil parameters are recommended for substitution in the above equation:

E_s = 15,000 to 20,000 kPa, and

ν_s = 0.3 to 0.35.

2.3 Post – Construction Settlement

The undisturbed very dense till-like soils and conglomerate bedrock, expected to be encountered at foundation level of the labyrinth weir and bridge structure, are not considered likely to compress significantly under the foundation loads imposed by these structures, provided subgrade preparation is carried out as recommended in Section 4.1. Consequently, significant long-term post construction settlements are not expected. Nominal construction total and differential settlement, less than 25 mm, is anticipated for foundations supported on intact, undisturbed till-like material and weathered rock.

3.0 LATERAL EARTH PRESSURE

In order to prevent excessive additional seepage into the underslab structural fills and beneath foundations nominally embedded within the till-like soils, it is considered essential that the backfill materials for the below grade portions of the labyrinth weir and bridge have permeability characteristics similar to, or less than, the pre-existing native soils in this area. Based on available geotechnical data, suitable low permeability materials could include the well graded silty sand and gravel soils (till-like soils), or the silty clay soils at the site. However, while these soils may possess suitable permeability characteristics, they are moisture sensitive and will be more difficult to place and suitably compact and (in particular the silty clay) will be subject to long term settlement.

At the time of preparation of this memo, the available quantities of these materials are unknown and it is therefore uncertain which of these soils (if any) will be present in sufficient quantities to provide the necessary backfill quantity. Therefore, we have considered both the use of the suitably compacted, lower permeability native sandy silty clay and/or sandy silt till-like soils or, alternatively, free draining granular structural fill as backfill material. However, it should be noted and recognized that use of higher permeability materials, such as free-draining structural fill, will require additional seepage and drainage control measures to prevent excessive seepage into the underslab fills and/or along the exterior of the auxiliary spillway structures and into the downstream open channel.

The recommended geotechnical engineering parameters for the design of the proposed auxiliary spillway walls with a free draining structural fill are presented in Table 3 and 5, below, whilst the recommended parameters for a fine grained backfill material (assumed permeability value of 2×10^{-6} or less) are presented in Table 4 and 6, respectively.

The parameters tabulated below are based on the following assumptions:

- The perimeter footings are founded on very dense till-like soils or sedimentary bedrock;
- Suitable subgrade preparation is carried out;
- The slope of the backfill surface around all spillway walls is horizontal;
- The back face of the wall is vertical; and
- No surcharge loads, other than that due to a gentle to moderate (2H:1V) slope rising from top of wall (approximately elevation 73.4 m) to original ground surface (elevation 74 to 75 m), are applied on the backfill adjacent to the spillway that would induce additional lateral stress on the walls.

Specifically for the free-draining fill, it was assumed that this material has a fines content of less than 5 percent. For a low permeability backfill we have based our parameters on a low plasticity, fine grained material which will present an upper bound case for lateral loading.

3.1 Static Loading Conditions

For rigid walls restrained from lateral movement (non-yielding), the static earth pressure forces acting on the wall may be calculated, as illustrated on Figure 1 and the parameters presented in Table 3, for free draining structural fill, or in Table 4, for a low permeability fill, respectively.

Retaining wall structures which are free to rotate about their base enough to permit displacements at the top of the wall of at least 0.1 percent of the total height of the wall (stiff walls) may be designed using 75 percent of the rigid wall value.

For walls that are flexible and free to rotate sufficiently to develop active earth pressure conditions (at least 0.5 percent), the lateral earth pressure will correspond to Ka under static conditions and Kae under seismic conditions. The Kae coefficient includes both the static (Ka) and dynamic components of the earthquake induced loadings.

Table 3: Free Draining Structural Fill: Recommended Geotechnical Parameters for Design of Spillway - Static Condition

Parameter	Value
Unit weight of backfill, γ_f (kN/m^3)	20
Buoyant unit weight of backfill, γ_{fb} (kN/m^3)	10.2
Friction angle of backfill, Φ' (degrees)	34°
Normal Operating Water Level Elevation	+71.6 m
Coefficient of earth pressure at rest (K_0)	0.44
Coefficient of active earth pressure (K_a) – Coulomb theory	0.25
Coefficient of passive earth pressure (K_p) (Factor of Safety = 2 on peak) – Coulomb theory	4.3
Friction angle between wall and backfill – Coulomb theory (degrees)	22°

Table 4: Low Permeability Fill: Recommended Geotechnical Parameters for Design of Spillway - Static Condition

Parameter	Value
Unit weight of backfill, γ_f (kN/m^3)	17
Buoyant unit weight of backfill, γ_{fb} (kN/m^3)	7.2
Friction angle of backfill, Φ' (degrees)	26°
Normal Operating Water Level Elevation	+71.6 m
Coefficient of earth pressure at rest (K_0)	0.56
Coefficient of active earth pressure (K_a) – Coulomb theory	0.35
Coefficient of passive earth pressure (K_p) (Factor of Safety = 2 on peak) – Coulomb theory	2.17
Friction angle between wall and backfill – Coulomb theory (degrees)	17°

Retaining walls supporting surcharges, such as vehicle loads, building appurtenances, and/or sloping backfill should be designed to resist the additional lateral loads imposed by these surcharges.

If adequate drainage is not provided for the below grade walls, by way of a perimeter tile drainage system or similar drainage control measures, the walls should be designed to withstand full hydrostatic pressures in addition to the lateral earth pressures.

3.2 Seismic Loading Conditions

For seismic conditions, the dynamic pressure under earthquake loading must be accounted for. If the walls of the spillway are rigid and non-yielding, it is recommended that the lateral earth pressure coefficient under seismic loading conditions be calculated using the procedure indicated on Figure 1 and a peak horizontal ground surface acceleration, A, of 0.268g for SLS design, and 0.499g for ULS design, as outlined in Table 1, together with the engineering properties of the backfill materials presented in Tables 5 and 6, as appropriate. For deformable walls, where the wall is free to rotate between 0.1 to 0.2 percent of the height of the wall, H, the maximum seismic pressure may be calculated as 75 percent of the rigid wall value. The pressure should be redistributed in equivalent rectangular form over the embedded height of the wall.

For a sufficiently flexible wall, where movement at the top of the wall of at least 0.5 percent of H can be tolerated, the lateral earth pressure under seismic loading conditions can be determined using the Mononobe-Okabe method. For this scenario, it is recommended that the lateral earth pressure under seismic loading condition be calculated using the procedure indicated on Figure 2 and a dynamic earth pressure coefficient Kae of 0.34 for SLS design, or 0.45 for ULS design for a free draining structural fill. For a low permeability backfill, a Kae value of 0.45 for SLS design, and 0.58 for ULS design should be used. The total lateral earth pressure under seismic conditions, including both the dynamic component and the static earth pressure component, is illustrated in (v) of Figure 2. However, additional hydrodynamic loading due to the presence of the groundwater level above the base of the wall should be taken into consideration. For low permeability backfill, the hydrodynamic lateral pressure component can be calculated by use of total, not buoyant unit weight, of the backfill. Use of coarse, high permeability backfill may result in some increase in hydrodynamic loading.

Tables 5 and 6 below present the lateral earth pressure coefficients for both static and seismic conditions for the different wall and backfill types described above.

Table 5: Free Draining Structural Fill: Lateral Earth Pressure Coefficients

Type of Wall	Movement at Top of Wall	Static Conditions		Seismic Earth Pressure
		K_0	K_a	
Rigid Wall – Non Yielding	0% of H	0.44	N/A	Refer to Figure 1. A = 0.268 g (for SLS Design) A = 0.499 g (for ULS Design)
Stiff Wall	0.1% to 0.2 % of H	0.44	N/A	0.75 times non-yielding value. Redistribute as rectangular load.

Type of Wall	Movement at Top of Wall	Static Conditions		Seismic Earth Pressure
Flexible Wall	>0.5% of H	N/A	0.25	Use Mononobe-Okabe method. K_{ae} = 0.34 (for SLS Design) K_{ae} = 0.45 (for ULS Design)

Table 6: Low Permeability Fill: Lateral Earth Pressure Coefficients

Type of Wall	Movement at Top of Wall	Static Conditions		Seismic Earth Pressure
		K_0	K_a	
Rigid Wall – Non Yielding	0% of H	0.56	N/A	Refer to Figure 1. A = 0.268 g (for SLS Design) A = 0.499 g (for ULS Design)
Stiff Wall	0.1% to 0.2 % of H	0.56	N/A	0.75 times non-yielding value. Redistribute as rectangular load.
Flexible Wall	>0.5% of H	N/A	0.35	Use Mononobe-Okabe method. K_{ae} = 0.45 (for SLS Design) K_{ae} = 0.58 (for ULS Design)

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Subgrade Preparation

Within the footprint of the proposed labyrinth weir and bridge, it is anticipated that saturated or wet sand with varying amounts of gravel and silt, and/or soft, silty clay, with varying proportions of sand, will be encountered at the anticipated elevation of the underside of the structural slab. As discussed in Section 2.0 above, we recommend that these sand and clayey deposits be overexcavated and that the undisturbed, dense till-like soil and/or conglomerate bedrock is exposed across the footprint of the structures. We recommend that the perimeter footings of the labyrinth weir and bridge be founded on till-like soils or conglomerate bedrock, with a minimum embedment depth of 0.3 m. For the structural slab located within the perimeter footings, we recommend that this be founded on well compacted structural fill that has been placed on top of the prepared till-like subgrade or bedrock subgrade to the underside of the slab.

The exposed till-like subgrade should be cleaned and subexcavated, as required, to remove all loosened, saturated or otherwise unsuitable material and inspected by an experienced geotechnical engineer, prior to placement of compacted Structural Fill, described below. Exposed, protruding cobbles or boulders encountered in the subgrade may require removal as they could result in local hard support conditions for footings. Alternatively, structural design of the footings should be adjusted to accommodate the differential support conditions.

Although strong in their unweathered conditions, till-like soils are moderately to highly susceptible to loss of strength and erosion when exposed to weathering or seepage, particularly at localized granular and water-bearing zones. Care and attention should be exercised in not allowing the subgrade to be exposed to sustained wet weather or construction traffic. Water should not be allowed to pond on the prepared subgrade surface. The approved subgrade should be covered with crushed gravel base course material, as described below, immediately following completion of subgrade preparation and inspection to minimize disturbance. Alternatively, it is recommended that consideration be given to initial placement of a minimum 50 mm thickness of lean concrete immediately following completion of subgrade preparation and inspection to minimize potential disturbance or softening prior to or during placement of the structural fill or pouring of foundations.

The conglomerate, although very dense and strong in place, is also moderately susceptible to softening and disturbance when exposed following excavation, in particular in the presence of seepage or ponding of surface runoff. It is recommended that a 50 mm working mat of lean mix concrete should be placed over the entire footing and slab excavation area immediately after cleaning and inspection.

4.2 Backfilling and Compaction Requirements

The prepared subgrade for the slabs of the labyrinth weir and bridge should be brought up to underside of the slabs using clean 19 mm minus crushed gravel in conformance with the latest Master Municipal Construction Documents (MMCD). Structural fill should be placed in lifts not exceeding 300 mm loose thickness and compacted to 95 percent of the modified Proctor maximum dry density (MPMDD) for the material.

At the time of writing this memo, the type of material to be used as backfill around the outside of the labyrinth weir and bridge structure had not been decided upon, with both free-draining backfill and the re-use of native, less permeable soils being considered.

If granular backfill is used, this material should consist of well-graded, free-draining sand, or sand and gravel, containing less than 5 percent material passing the USS No. 200 sieve size.

If less permeable backfill material is used, consideration may be given to use of suitable portions of the excavated native silty clay or till-like soils around the outside of the labyrinth weir and bridge structure, provided that the moisture content is below the optimum level for compaction. To permit use of these onsite soils, they should be suitably excavated and stored to prevent wetting and mixing with other types of higher permeability soils or materials. As described above, fine grained soils having water contents above the optimum for compaction will be difficult or not possible to suitably place and compact during backfilling. Post construction total and differential settlements should be expected if such fine grained soils are used, and are likely to require future fill placement and regrading of the backfill and adjacent areas.

Clear crushed gravels are not recommended for use as structural fill or backfill due to the potential for migration of fines from surrounding soils into these materials, thereby resulting in loss of ground and support.

Backfill should be placed in lifts not exceeding 300 mm loose thickness and compacted to 95 percent of the modified Proctor maximum dry density (MPMDD) in areas where limited post construction settlement is desired or required. Placement in 300 mm lifts and nominal compaction may be considered for backfill in areas where larger post construction settlements are acceptable.

To avoid overstressing and damage to walls, use of heavy compaction equipment should be avoided adjacent to below grade walls. Only light hand operated compaction equipment should be utilized in these areas.

5.0 CLOSURE

We trust that this is sufficient for your immediate requirements. Should you have any queries or comments, please do not hesitate to contact the undersigned.

GOLDER ASSOCIATES LTD.

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Attachments: Figure 1: Lateral Earth Pressure Diagram for Rigid Wall (Non-Yielding)
Figure 2: Lateral Earth Pressure Diagram for Flexible Wall (Yielding)
Attachment 1: 2010 National Building Code Seismic Hazard Calculation

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Important Information and Limitations of this Report

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

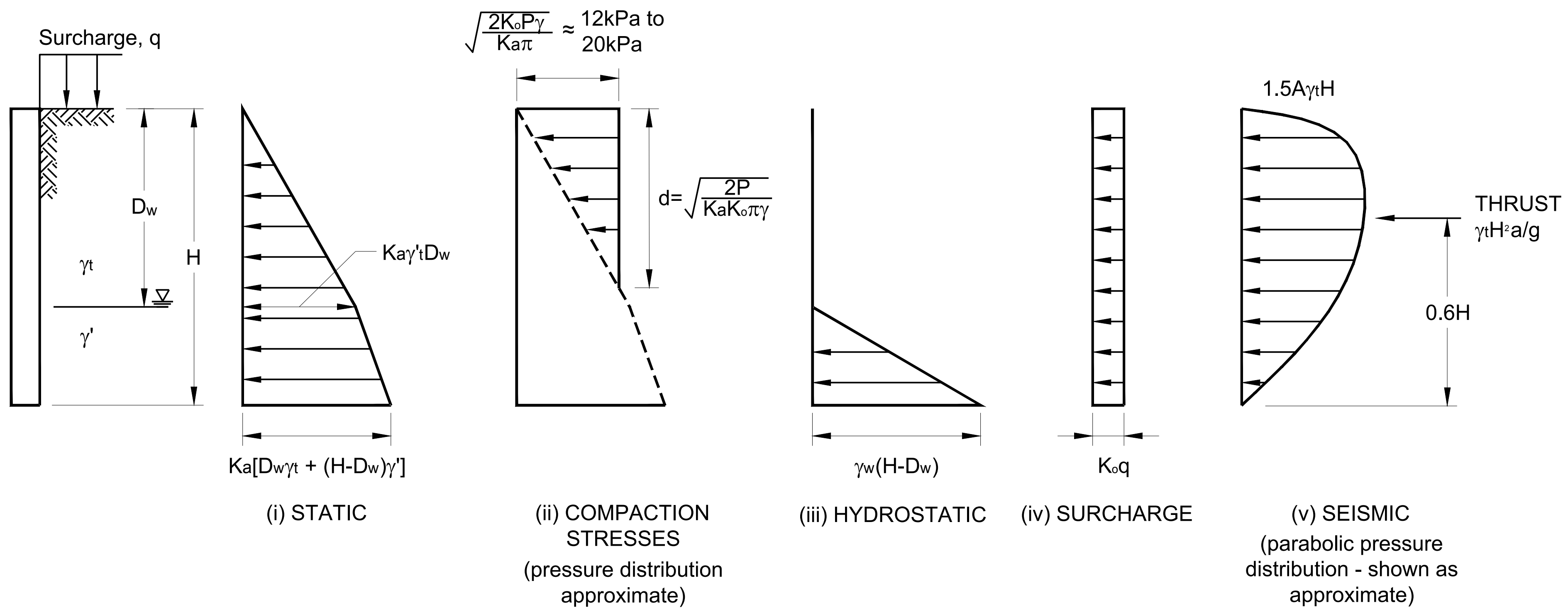
Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

Restrained Wall (Non-Yielding) - Level Ground Surface



γ_w = Water Unit Weight
γ_t = Total Unit Weight
γ' = Buoyant Unit Weight
K_o = At-rest Coefficient of Lateral Earth Pressure
K_a = Active Co-efficient of Lateral Earth Pressure

q = Surcharge
D_w = Depth to Water Table
H = Height of Wall
d = Depth Over Which Compaction Stresses are Applicable
a = Peak Horizontal Acceleration

$$P \text{ (Roller Load)} = \frac{\text{Dead Weight of Roller + Centrifugal Force}}{\text{Width of Roller}}$$

NOTES

1. SEE TECHNICAL MEMORANDUM 13-1447-0516, DATED OCTOBER 19, 2015 FOR DESIGN PARAMETERS.
2. FOR STATIC CONDITIONS: CONSIDER THE COMBINED ACTION OF (i)+(iii)+(iv) FOR OVERTURNING, SLIDING AND BEARING, AND CONSIDER THE COMBINED ACTION OF (i)+(ii)+(iii)+(iv) FOR STRUCTURAL DESIGN.
3. FOR SEISMIC CONDITIONS: CONSIDER THE COMBINED ACTION OF (i) + (iv) + (v). CONSIDER (iv) FOR DEAD LOADS ONLY.

CLIENT
CITY OF NANAIMO

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-19
PREPARED	SRR
DESIGN	TM
REVIEW	JG
APPROVED	BD

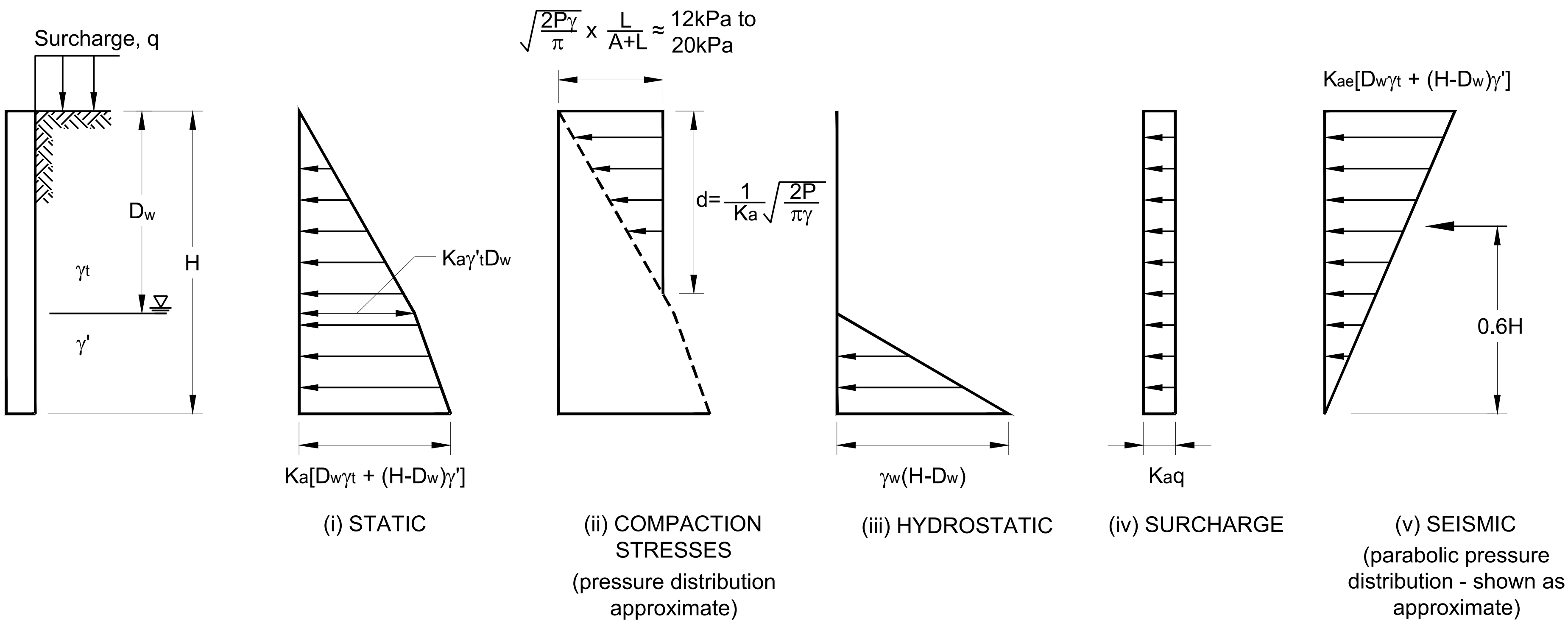
PROJECT
COLLIERY DAMS
NANAIMO, BC

TITLE
LATERAL EARTH PRESSURE DIAGRAM FOR RESTRAINED WALL (NON-YIELDING)

PROJECT No.	PHASE	Rev.	FIGURE
13-1447-0516	4000	1	1

Path: \\golder.gds\gal\burnaby\CAD-GIS\Client\City_of_Nanaimo\99_PROJECTS\13-1447-0516\02_PRODUCTION\4000\4500_Tech Memo\1314470516-031-TM-Rev0\ | File Name: 1314470516-031-TM.dwg

Flexible Wall (Yielding) - Level Ground Surface



γ_w = Water Unit Weight
γ_t = Total Unit Weight
γ' = Buoyant Unit Weight
K_a = Active Co-efficient of Lateral Earth Pressure
q = Surcharge

D_w = Depth to Water Table
H = Height of Wall
d = Depth Over Which Compaction Stresses are Applicable
K_{ae} = Dynamic Active Co-efficient of Lateral Earth Pressure

$$P \text{ (Roller Load)} = \frac{\text{Dead Weight of Roller + Centrifugal Force}}{\text{Width of Roller}}$$

NOTES

1. SEE TECHNICAL MEMORANDUM 13-1447-0516, DATED OCTOBER 19, 2015 FOR DESIGN PARAMETERS.
2. FOR STATIC CONDITIONS: CONSIDER THE COMBINED ACTION OF (i)+(iii)+(iv) FOR OVERTURNING, SLIDING AND BEARING, AND CONSIDER THE COMBINED ACTION OF (i)+(ii)+(iii)+(iv) FOR STRUCTURAL DESIGN.
3. FOR SEISMIC CONDITIONS: CONSIDER THE COMBINED ACTION OF (iv) + (v). CONSIDER (iv) FOR DEAD LOADS ONLY.

CLIENT
CITY OF NANAIMO

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-19
PREPARED	SRR
DESIGN	TM
REVIEW	JG
APPROVED	BD

PROJECT
COLLIERY DAMS
NANAIMO, BC

TITLE
LATERAL EARTH PRESSURE DIAGRAM FOR FLEXIBLE WALL (YIELDING)

PROJECT No.	PHASE	Rev.	FIGURE
13-1447-0516	4000	1	2

ATTACHMENT 1 -
2010 National Building Code Seismic Hazard Calculation

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: , Golder Associates Ltd. August 24, 2015

Site Coordinates: 49.1492 North 123.9616 West

User File Reference: Colliery Dam

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
1.013	**0.692**	**0.351**	**0.178**	**0.499**

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*** Warning: You are in a region which considers the hazard from a deterministic Cascadia subduction event for the National Building Code. Values determined for high probabilities (0.01 per annum) in this region do not consider the hazard from this type of earthquake.

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.245	0.532	0.726
Sa(0.5)	0.163	0.357	0.489
Sa(1.0)	0.082	0.181	0.248
Sa(2.0)	0.040	0.089	0.123
PGA	0.125	0.268	0.360

References

National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français