DEPARTMENT OF ECOLOGY
State of Washington

2020 Periodic Inspection Report

Lords Lake North Dam and East Dam
Jefferson County, Washington

August 2020

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Lords Lake North Dam and East Dam
Jefferson County, Washington

The dam safety inspection of Lords Lake North Dam and East Dam, engineering analyses, and technical material presented in this report were prepared under the supervision and direction of the undersigned professional engineers, in accordance with RCW 43.21A.064(2).

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Introduction

In accordance with RCW 43.21a.064(2), the Department of Ecology, Dam Safety Office (DSO) has the responsibility and authority to inspect the construction of all dams and other works related to the use of water, and to require necessary changes in construction or maintenance to reasonably assure safety to life and property. This report has been prepared in accordance with this statute.

The purpose of this report is to present the results of an inspection of the Lords Lake North Dam and East Dam. The report provides some background information and a description of the project; results of the June 2, 2020, inspection; and, remedial actions based on the findings.

Project Information

Background

Lords Lake is located about 4.5 miles northwest of Quilcene, Jefferson County, Washington on an unnamed tributary of Howe Creek, which in turn is a tributary of the Little Quilcene River (see Figures 1 and 2). The facility is owned by the City of Port Townsend (CPT) and operated by the Port Townsend Paper Corporation (PTP).

The reservoir is an enlargement of a natural lake created by two earthfill dams. Lords Lake is used as a secondary source of water supply for the Port Townsend Paper mill and the City of Port Townsend during low flow periods on its primary source, the Big Quilcene River. Additionally, Lords Lake acts as backup water supply when the primary source is turbid. The lake is filled by a diversion pipeline from the Little Quilcene River. Water from the lake enters a pipeline, which carries water to the mill and city.

Construction of the Lords Lake Project began in 1956. Placement of fill for the East Dam began in July and was completed in October 1956. Embankment construction of the North Dam started in September 1956, but was halted in November for the winter. Construction resumed in the spring of 1957 and the embankment and spillway were completed in early summer.

First filling of the reservoir was started in the fall of 1957. When the reservoir level reached Elevation 910 (National Geodetic Vertical Datum, NGVD), 16 feet below dam crest level, seepage was noted along the East Dam toe and abutment areas. Seepage was also noted from a spring on the west abutment of the North Dam. Reservoir filling was subsequently halted, and the Bechtel Corporation was retained to determine the source of seepage and make recommendations for remedial work.

Bechtel determined that the seepage from the North Dam resulted primarily from a spring that existed prior to construction. The East Dam seepage, however, was a more significant concern. A more detailed soils exploration program was undertaken in January 1958, and 21 test holes were drilled through the East Dam into the underlying foundation rock. The drilling revealed that seepage was occurring through lenses of a more pervious material which overlaid the rock surface across the entire footprint of the dam. These holes were grouted with 156 sacks of cement in an effort to reduce the rate of seepage. The grouting program apparently was not effective in materially reducing seepage.

In February 1958, reservoir filling resumed and by March 31, the reservoir had filled to Elevation 919.5 feet, the spillway crest level. Total seepage increased to about 50 gallons per minute (gpm) at the East Dam, then remained constant. No evidence of fines or piping of embankment materials was noted according to inspection reports. Seepage reportedly stabilized, then lessened over the summer and fall of 1958 as the reservoir level dropped.

The reservoir filled during floods to about Elevation 920 in February 1959. Total seepage through the dam had decreased to about 31 gallons per minute. Bechtel concluded that seepage through the dam had stabilized, and that the stability of the downstream slope was not compromised. Although Bechtel recommended continued monitoring of the weirs and piezometers, the operator eventually discontinued the monitoring program in the 1960's.

Mr. Lowell Tiller, former Engineering Supervisor at Crown Zellerbach Corporation, noted during the 1989 Dam Safety Section (DSS) periodic inspection of the dams, that the interior of the old 36-inch reinforced concrete pipe (RCP) outlet was examined in 1977, prior to the installation of a new 30-inch steel pipe. At that time, Mr. Tiller inspected the interior of the old RCP conduit and found it to be in good condition with no significant seepage at the pipe joints.

In May 1978, the dam was inspected by CH2M Hill as part of the National Dam Safety Program administered by the Corps of Engineers (COE). The subsequent COE Phase I inspection report (USACE, 1979) expressed concern about the stability of both dams, due to seepage and the use of inappropriate soil properties in the original design computations. The report recommended that further geotechnical evaluation be performed to ascertain the adequacy of the embankments.

In 1979 as a follow up to the Phase I report, Crown Zellerbach installed 4 piezometers in each dam, constructed weirs, and resumed a monitoring program. Crown Zellerbach retained Harding Lawson Associates (HLA) in 1982, to perform a Phase II geotechnical evaluation of both dams. HLA concurred with Bechtel that the seepage beneath the dam was not piping fines from the embankment or foundation, and concluded that the embankments had adequate factors of safety for static slope stability. Stability under seismic loadings was not considered. HLA recommended continuing the monitoring program.

In June 1989, the DSS performed a periodic inspection of the project. In the resulting 1990 inspection report, the dams were found to be fairly well maintained structures, and that the North Dam was in satisfactory condition. However, the East Dam was found to have inadequate seismic stability due to liquefaction of the lower portion of the embankment during the design earthquake. These concerns required that the owner conduct additional field and engineering studies to address the problem and to take corrective action where necessary.

In response to these findings, PTP retained Applied Geotechnology Inc. to perform geotechnical studies at the East Dam. In Phase One, the consultant excavated test pits to verify the presence of the rock toe drain, installed a network of drains to allow collection and measurement of seepage, and performed a preliminary seismic assessment of the embankment. In their Phase 2 study, the consultant drilled additional borings on the dam, examined past records in more detail and reevaluated the potential for liquefaction under the then-current design earthquake loading.

Based on their studies, Applied Geotechnology concluded that significantly higher soil densities existed within the embankment than assumed by the Dam Safety Section, and a liquefaction failure of the dam was not likely. Ecology accepted these findings based on the information available at that time, and no further remedial work was required.

In 1996, DSO performed the second comprehensive inspection of the dams. In the resulting 1999 inspection report, DSO determined that the dams were in good condition, with the exception of some minor maintenance and monitoring deficiencies. The engineering evaluation and analyses indicated that at the time, the dams met current engineering standards for dam design with regards to floods and earthquakes. However, it was noted that for the East Dam, the margin of safety against a failure under the design earthquake was small. Consequently, the owner was informed that as understanding of the seismic picture in the Pacific Northwest improves, it was possible that the design earthquake would increase to a point where another stability analysis would be necessary.

According to Mr. Andy Karlsnes with PTP as noted during the 2003, DSO inspection of the dams, the reservoir was completely drained in the fall of 2002, and the interior of the outlet tower was inspected at that time. No defects were found with the reinforced concrete or gates. New gate stems for the sluice gates were installed at that time.

On March 25, 2003, the DSO performed the third comprehensive inspection of the dams. The dams were found to be in good condition. However, the following deficiencies were noted: inadequate seepage monitoring at the North Dam; minor maintenance deficiencies such as vegetation on the lower section of the North Dam downstream slope, animal burrows on East Dam, vehicle ruts on the North Dam crest, logs and debris in front of the spillway entrance channel, and siphon valve on spillway should be removed; and, the Emergency Action and Operation & Maintenance Plans needed to be reviewed and updated.

The fourth periodic inspection of the dams by the DSO was conducted on February 2010. The results of the inspection indicated that the dams appeared to be well maintained and in good condition. A few minor maintenance and operational issues were found: animal burrows on the East Dam should be backfilled, removal of logs and debris from the spillway entrance channel, review and update the Operation and Maintenance Plan, and the need by the owner to conduct annual inspections of the dams.

The most recent inspection of the dams by the DSO was conducted in March 2015. The results of this inspection are summarized in the following section. During the inspection, Mr. Ian Jablonski with CPT inquired about the possibility of raising the reservoir level at Lords Lake by 1-2 feet (or more if possible) to store more water given the likelihood of drought conditions.

A formal request to raise the level was presented to DSO by CPT on May 9, 2016. In response to the CPT's request, the DSO reviewed the hydrologic modeling for the facility and concluded that the level of the lake could be raised up to a maximum of 3 feet above the normal overflow elevation for the spillway. Based on the results of engineering analyses conducted as part of DSO's 2015 inspection, CPT was authorized to temporarily fill the reservoir no higher than elevation 922.5 feet.

CPT was also informed that a permanent change in the pool elevation to elevation 922.5 would require that CPT conduct hydrology/hydraulic and seismic stability analyses to demonstrate that the dams meet the requirements outlined in the DSO guidelines for the proposed pool elevation.

In response to DSO's 2015 inspection findings, CPT retained the services of Golder Associates, Inc. in 2019, to evaluate the stability of the East Dam. As part of this evaluation, Golder conducted a subsurface exploration program on April 2019. The exploration consisted of drilling two boreholes on the crest and two on the downstream toe area of the East Dam.

Soil conditions encountered on the borehole drilled on the crest at the northern section of the embankment included a loose silty sand soil layer, approximately at a depth of 25 to 35 feet below the crest level. At the downstream toe area, also on the north side of the dam, the shallow surface layer encountered consisted of an approximately 12-foot thick layer of a very loose clayey sand. At the southern section of the dam, soils encountered consisted of a medium dense gravelly sand, identified as embankment fill, directly overlying bedrock. Groundwater was determined to be approximately 15 feet below the crest of the dam based on piezometric readings.

Subsequent to the soils exploration, Golder conducted geotechnical analyses to evaluate the stability of the East Dam. The main findings of Golder's analyses included:

- Significantly different soil conditions than those encountered during previous explorations.
- Potential liquefaction of lower section of embankment soils and shallow surficial soils at the downstream toe area on the northern section of the East Dam.
- Static and seismic slope stability factors of safety less than required minimums.
- Seismically induced displacement on the order of 0.2 to 1.4 feet.

DSO reviewed Golder's geotechnical report (2019); and, based on DSO's own geotechnical analyses using the information presented in the report, DSO concurred with Golder's findings.

During the June 2, 2020, DSO inspection of the facility, Mr. John Straub indicated that PTP has decided to postpone any additional studies and will focus on a possible retrofit of the facility to increase storage capacity in the near future.

Previous Inspection Summary

The previous periodic inspection by the DSO was performed on March 19, 2015. The following findings and observed deficiencies were noted in a letter-report dated March 28, 2016:

- Presence of a seepage area on the downstream foundation area at the East Dam.
- Animal burrows on the downstream slope of the East Dam.
- Small accumulation of floating debris against the log boom.
- Missing sections on the log boom.

- Update the O&M Manual.
- Update the EAP.
- Conduct periodic inspections of the dam on an annual basis and provide a copy of the inspection form to the DSO.
- Geotechnical analyses indicated potential liquefaction of lower embankment materials on the East Dam, slope stability factors of safety under seismic loading less than required minimums, and seismically induced displacement on the order of 0.65 feet.
- Additional geotechnical and hydrology/hydraulic analyses would be required to support a permanent raise of the pool level.

Field Inspection

The primary field inspection of the Lords Lake North Dam and East Dam was performed on June 2, 2020. The Dam Safety inspection team consisted of the following personnel:

Name	**Aspects Covered**
Gustavo Ordonez, P.E.	Coordinator, Geotechnical
Martin Walther, P.E.	Hydrology/Hydraulics

Ian Jablonski and Michael Spears with the City of Port Townsend; and, John Straub, Steve Much, and Matt Lewis with Port Townsend Paper Corporation were present during the inspection.

Reservoir

Lords Lake is an enlargement of a natural lake created by two earthfill dams. The lake is used as a secondary source of water supply for the Port Townsend Paper mill and the City of Port Townsend during low flow periods on its primary source, the Big Quilcene River. Additionally, Lords Lake acts as backup water supply when the primary source is turbid. The lake is filled by a diversion pipeline from the Little Quilcene River. Water from the lake enters a pipeline, which carries water to the mill and city.

Lords Lake has a surface area of approximately 56 acres at the spillway crest elevation of 919.5 feet and impounds about 1480 acre-feet. The lake can impound a total of 1850 acre-feet at the dam crest elevation of 926.0 feet.

At the time of the inspection, the lake level was approximately at elevation 919.6 feet, which was 6.4 feet below the dam crest.

A visual inspection of the hillsides immediately surrounding the lake indicated that the slopes above the lake are heavily forested; however, there were no obvious signs of slope movements that could adversely affect the dams. Similarly, an inspection of the reservoir rim based on DNR lidar coverage of the area (see Figures 3, 4 and 5) and WA DNR landslide information[1] (see Figure 6) revealed no obvious evidence of significant landslides on the slopes around the reservoir that could pose a threat to the safety of the dams.

During the inspection, Mr. Jablonski indicated that CPT will be requesting authorization to raise the lake level by 1 foot during the summer of 2020. We indicated that the authorization would likely be provided but noted that in the near future raising the lake level may not be authorized given an increase in the probability that the design earthquake will occur. Further, we recommended Mr. Jablonski to request authorization for an additional 1 foot to provide additional capacity if needed.

A formal request to raise the level two feet to elevation 921.5 was submitted by CPT to DSO on June 3, 2020. In a follow up letter dated June 8, 2020, the DSO authorized a temporary raise of the lake level to elevation 920.5 feet; and, pre-authorized an additional raise to elevation 921.5 feet based on updated seasonal climate outlooks.

Embankments, Abutments, and Foundations

North Dam - Lords Lake North Dam is an earthfill embankment rising a maximum of 36 feet above natural grade (Crest Elevation 925.8 feet), with a sloping central impervious core section and downstream rock toe. According to compaction test reports, the embankment shells were constructed of a sandy silt material with 30 to 60 percent gravel. The core consisted of a siltier material with a smaller percentage of gravel (about 20 to 35 percent). According to the plans, a cutoff trench was excavated into the foundation under the upstream face of the dam.

As per the original specifications, the rock toe was to be constructed of a clean rock ranging from one cubic foot down to two inches in size. A sand and gravel filter layer was to be placed between the rock toe and fine grained embankment materials. According to all available records, it appears the rock toe was installed. It is not known if the filter layer was placed. A "french drain" was constructed in the downstream shell to discharge flow from a preexisting natural spring in the left (west) abutment. The upstream face of the dam has a slope of 3H:lV, while the downstream face has a 2H:lV slope.

A visual inspection of the dam crest indicated that there were no visible signs of distress such as cracks, depressions, or sinkholes.

There were no visible signs of instability along the upstream side of the crest or the uppermost section of the slope that may indicate problems with the slope. The riprap layer appears to provide adequate erosion protection.

There were no visible signs of cracking, settlement, seepage, leakage, or new wet areas observed on the downstream slope. As observed during prior inspections, the downstream toe area along the western (left looking downstream) side of the embankment was saturated and spongy. There were no visible signs of water flowing or sediment being carried, however, the amount of seepage is such that water accumulates in this area.

[1] WA DNR Geologic Information Portal accessed on June 10, 2020.

It is required to periodically monitor this seepage area to determine if there are any changes in the seepage, e.g., increase in the amount of water, amount of sediments in the water, signs of distress on the slope, etc. In the event of any changes in the seepage, please contact us as soon as possible to determine if any corrective action is necessary. The City should consider the option of building a drain (e.g., French drain) along the western groin and downstream toe areas to collect the seepage and discharge it to an appropriate location downstream.

The toe drain outlet was visually inspected and the water appeared clear, with no sediment noted. Some minor amount of vegetation growth was observed around the toe drain and concrete weir box (see Photos 1 and compare to Photo 2). It is required to clear all vegetation from this area to ensure that the vegetation does not hamper inspection or operation of the drain and weir; remove sediment accumulated in the weir concrete box; and, similar to the toe drain, periodically monitor flow volume and characteristics on the weir.

East Dam - Lords Lake East Dam is a homogenous earthfill embankment with a maximum height of about 40 feet above natural ground. The embankment was to be constructed of materials from the same borrow areas as the North Dam. However, an impervious core was not provided. According to the construction plans, a 10 foot wide cutoff trench was excavated into the foundation under the upstream face of the dam. Neither the cutoff trench nor the remainder of the dam footprint was to be stripped to bedrock.

As with the North Dam, a downstream rock toe with a filter zone was provided. In 1992 this toe drain was exposed as part of Phase 1 investigations by Applied Geotechnology (1991), and finger drains and a blanket drain were installed to improve seepage collection. The upstream face has a slope of about 3H:lV and is protected by riprap. The downstream face has a slope of 2¼H:lV.

At the time of the inspection, the upper 6.5 feet of the upstream slope was above water. A layer of riprap appears to provide adequate erosion protection (see Photo 3). However, based on photos taken circa 1999 when the reservoir was lowered (see Photo 4), it appears that the riprap layer does not cover the entire upstream slope. No signs of slumping, cracks or uneven settlement were apparent along the section of the upstream slope above the water line.

The crest was in good condition overall with no signs of cracking, slumping or uneven settlement.

Our examination of the downstream slope revealed that there were no signs of slumping, settlement, cracking, or seepage. The only maintenance deficiency observed was some vegetation at the saddle embankment section (see Photo 5). However, this vegetation was promptly removed after the inspection (see Photo 6 provided by CPT).

Similar to prior inspections of the dam, a seepage area was observed on the downstream toe foundation area of the East Dam (see Photo 7). The source of the water is probably from seepage through the dam and/or foundation soils. This area should be periodically monitored to determine if there are any significant changes on the amount of water and/or if signs of increased seepage such as bubbles or dirty water flow are present. In the event of any changes, the DSO should be notified as soon as possible to evaluate the situation.

To facilitate monitoring of the above seepage, the City may consider constructing a simple drain system in the area of the seep. It appears that the seepage is localized to a small area, hence a shallow, short drain (e.g., French drain) constructed in the area would probably be adequate to collect the flow. The drain should be provided with an outlet pipe that could daylight at the location of seepage measurement point number 2 where the flow could be monitored. Please contact your engineering consultant for help on the design of the drain and its construction.

Overflow Spillway

The overflow spillway for the Lords Lake facility is located on the east (right) abutment of the North Dam. The spillway consists of a concrete side channel weir entrance section, followed by a concrete chute conveyance section that carries flows down the east abutment contact to the toe of the dam.

The weir entrance section consists of three bays with a total crest length of 16 feet and a crest elevation at 919.5 feet. The weir entrance section discharges into a concrete chute oriented at a right angle to the dam axis. The floor of the chute has a minimal slope in the vicinity of the weir, but increases to 2H:1V to match the downstream slope of the dam, through a convex vertical curve transition. The width of the chute contracts to five feet through this transition. The height of the chute side walls transitions from 7.6 feet at the entrance section to 4 feet in the 2:1 section on the downstream slopes.

No stilling basin exists for this spillway. Instead, near the downstream end of the spillway, a concave vertical curve transition directs spillway flows into a riprap channel section. Energy dissipation takes place in the riprap section of the channel.

A visual inspection of the entrance section indicated that it appeared to be in satisfactory condition. The layer of moss that covered the upper section of the spillway structure and somewhat obscured visual inspection of the concrete has been removed since the previous inspection. There was no evidence of cracking or spalling on the concrete piers, floor or sidewalls.

As noted during prior inspections of the dam, a small accumulation of floating debris against the log boom was observed (see Photo 9). It is required that any debris be periodically removed to ensure the spillway will perform as designed.

Although the log boom appears to be doing an adequate job of limiting wave erosion and containing floating debris, the log boom is not a continuous line. Hence, either the different sections have separated from each other or the log boom is not properly anchored at the reservoir side slopes. Accordingly, it is required that a new, manufactured log boom and debris barrier be provided for the reservoir and installed some distance upstream from the dam.

Visual inspection of the spillway chute section downstream from the entrance channel revealed that it appeared to be in satisfactory condition. There was no evidence of misalignment on the sidewalls, with only some minor cracking observed. Due to water flow on the channel, we were not able to inspect the channel floor and the area at the end of the concrete chute.

Removal of the moss layer allowed us to visually inspect the concrete sidewall along the eastern side (i.e., along the toe of the hillside) of the spillway channel. It was observed that the section of wall at the crest of the dam showed a slight curvature at its upper section (see Photo 8). The original plans show a straight alignment along the wall. However, there were no obvious signs of cracking on the wall that may indicate the wall has moved due to forces induced by the soils against it. Further, it is not known if the current shape of the wall is the as-built geometry.

As seen on Figure 4, DNR coverage of the hillside along the eastern side of the dam does not show obvious signs of landslide activity. However, this section of the wall and hillside area above the channel should be periodically inspected and monitored to ensure there are no ground movements that may be adversely affecting the structural integrity of the wall.

Outlet Works

The outlet works for Lords Lake are housed in a concrete wet well outlet tower at the East Dam. A 100 foot long wood and steel catwalk connects the tower to the dam crest. The tower has three sluice gates that are operated from the top of the tower. Two of the gates are located on the exterior of the tower and are used to withdraw water from the reservoir at different elevations. The upper exterior gate is 30 inches square, with an invert elevation at 902.3 feet. The lower external gate is 36 inches square with an invert elevation at 885.0 feet. The remaining gate is a 36 inch square sluice gate on the inside of the tower which controls flow into the outlet conduit. The invert elevation of this gate is 885.0 feet.

The outlet conduit consists of a 30 inch diameter steel pipeline which connects downstream from the East Dam to the water supply pipeline from the Big Quilcene River. This pipe was installed within the original 36 inch diameter reinforced Concrete Pipe (RCP) outlet conduit in 1977. The RCP was sleeved with a steel pipe, and the annulus between the two pipes was grouted.

Originally, the RCP outlet conduit was designed to discharge into an overflow box at the downstream toe of the dam. Flow from the overflow box would then enter a pipeline to the city. When the new conduit was installed, the pipe was carried through the box and directly connected to the pipeline, thus removing the overflow box from service.

A visual inspection of the visible portion of the outlet tower indicated that it appeared to be in good condition. There were no visible cracks or spalling on the concrete surfaces. The gates were not operated during the inspection.

Access to the tower is possible through a 24-inches diameter opening at the top of the tower. The opening is provided with a steel lid and secured with a lock to prevent access. The lid was opened during the inspection which allowed us to visually inspect the interior of the tower. However, the tower was full of water up to approximately the same level as the lake, thus we were only able to observe the upper section. There were no obvious signs of distress or problems noted. Further, it was observed that a ladder was not provided in the tower to facilitate access to its interior.

As discussed during the inspection, we recommend that the interior of the outlet pipe between the tower and the overflow box at the downstream toe of the dam be inspected with the aid of a remotely controlled camera. This will probably require draining water from the tower and section of pipe, and safe access to the bottom of the tower. The inspection may be conducted as part of the future modifications to the dam noted by Mr. Straub.

Instrumentation & Monitoring

Instrumentation at the Lords Lake project includes four piezometers at each dam, seepage monitoring vaults and manhole at the East Dam, and a reservoir staff gage at the East Dam Outlet Tower. PTP staff visually examines seepage from the East Dam daily, and measures and records seepage, reservoir levels, and piezometer levels on a monthly basis.

Seepage from the rock toe and abutment areas at the East Dam is collected in the drainage system installed in 1991. Seepage from the center and left abutment areas is collected in manhole No. 1, which discharges downstream from the county roadfill at seepage measurement point number 1.

A visual inspection of the v-notch weir at the East Dam indicated that the flow appear clear, i.e., it did not appear to be carrying fines. It was observed that the weir has suffered some damage, likely from vandalism (see highlighted area on Photo 10). This damage should be repaired as soon as possible to ensure the entire seepage flow is measured.

Similarly, seepage observed at measurement point number 2 appeared clear.

There was some vegetation around the area of the seepage collection points, and some sediment accumulation observed on the v-notch weir concrete boxes at the North and East Dams. It is required that any vegetation that may hamper monitoring of the seepage be periodically removed. Further, the damage to the v-notch weir at the East Dam should be repaired and sediment removed from the concrete weir box to ensure the weir performs as-designed.

As per the DSO Guidelines, settlement monuments are required for an intermediate (i.e., >= 15 and < 50 feet high) dam. Due to an oversight on our part, these monuments have not been required during prior inspections of the dams. However, this instrumentation will help proper monitoring of the embankment performance during the life of the project.

Accordingly, monuments should be provided as soon as practical. The monuments can be as simple as a short section of steel pipe driven into the crest of the embankments or as complex as a survey monument encased in concrete. The monuments should be located at intervals of 25 feet or less with one monument at each abutment that should not be affected by embankment settlement.

Evaluation and Analysis

Condition Assessment

Overall, based on observations made during the inspection, the Lords Lake North Dam and East Dam are well maintained and operated. The condition assessment of the Lords Lake North Dam is considered to be **Satisfactory**. This condition assessment is in line with the system used by the National Inventory of Dams (USACE, 2008) to classify dams with no existing or potential dam safety deficiencies recognized.

The Lords Lake East Dam is considered to be in **Poor** condition. This condition assessment is in line with the system used by the National Inventory of Dams to classify dams with a dam safety deficiency recognized for loading conditions which may realistically occur, and remedial action is necessary. This assessment is based on the East Dam not meeting the minimum stability requirements under seismic loading.

Downstream Hazard Assessment

Prior to the 2020 inspection, the setting downstream from the Lords Lake Dams was classified as having a *High* downstream hazard with a Hazard Class of 1A if a dam failure should occur.

The DSO guidelines define this classification as having a potential loss of life at more than 100 inhabited structures; with extreme economic losses; and, a downstream area described as highly developed, densely populated suburban or urban area with associated industry, property, transportation and community lifeline features. Environmental damage associated with this classification would include potential for severe water quality degradation from reservoir contents and long-term effects on aquatic and human life.

As part of the inspection, the downstream hazard potential was reassessed. This was accomplished by visual inspection of the downstream valley with the aid of topographic maps, aerial photographs and the inundation maps developed for the dams.

Our assessment revealed that the consequences of a failure of either Lords Lake North Dam or East Dam would include the potential for loss of life and property damage on at least 100 homes and businesses, washing out several roads and bridges, and destroying farmland downstream. In addition, Lords Lake would be lost as a water supply, and severe erosion and sedimentation would likely occur along Howe Creek and the Little Quilcene River. Based on these findings, the hazard classification for both the Lords Lake North Dam and East Dam should remain at **Hazard Class 1A.**

For the purpose of assigning a “warning potential rating” to the dams, i.e., a parameter used by DSO when creating a prioritization list of compliance/enforcement efforts, the first residence is located approximately 2 miles downstream from the East Dam. Estimated flood travel time to this residence is 18 minutes.

Hydrology and Spillway Adequacy

The spillway for the Lords Lake Dams can pass the runoff from a PMP (Probable Maximum Precipitation) dam safety storm. This event has an annual exceedance probability of 1 in 1,000,000, and an exceedance probability of 1 in 10,000 over a 100-year period. The Long duration storm was found to be the controlling event. The spillway can pass the dam safety inflow design flood of 147 cfs with 2.2 feet of freeboard remaining on the dam.

Several recent drought years have prompted requests for permission from DSO for Lords Lake to install weir boards in the spillway entrance to accumulate extra storage in the reservoir during the early summer months for use later in the summer and early fall when the river water sources become unavailable earlier than normal.

A detailed hydrologic analysis examined the impacts of various weir board configurations on the ability of the spillway to pass the dam safety inflow design flood with adequate freeboard on the dams. The details of the hydrologic analysis, described in a separate hydrologic analysis report compiled by the Dam Safety Office (Walther, 2020), are summarized below.

The analysis examined which weir board configurations would or would not meet dam safety freeboard requirements with weir boards in the spillway entrance to store additional water in the reservoir during drought periods. The main findings from the analysis include:

- With weir boards up to 1.5 feet high, with a spillway overflow elevation up to 921.0 feet, the Lords Lake spillway can pass the runoff from a PMP storm for any time of year, including winter months when there is snow in the watershed, with adequate freeboard on the dams. This meets DSO requirements for freeboard during inflow design flood (IDF) conditions for any time of year.

- With weir boards up to 2.0 feet high, with a spillway overflow elevation up to 921.5 feet, the spillway can pass the runoff from a PMP storm during periods when there is no snow in the watershed, with adequate freeboard on the dams. This meets DSO requirements for freeboard during IDF conditions whenever there is no snow in the watershed.

- With weir boards up to 2.5 feet high, with a spillway overflow elevation up to 922.0 feet, the spillway can pass the runoff from a Step 7 storm during periods when there is no snow in the watershed, with adequate freeboard on the dams. Design Step 7 has the same risk for a 4-month period that Design Step 8 has for a 12-month period, so the hydrologic risk would be considered equivalent to Step 8 during that 4-month period.

  With the proviso that weir boards up to 2.5 ft. high would be limited to a 4-month period, this would meet DSO requirements for freeboard during IDF conditions when there is no snow in the watershed.

As noted above, with no weir boards in the spillway, the Lords Lake spillway has adequate capacity to meet DSO freeboard requirements.

This analysis considers hydrologic risk only, does not consider geotechnical risk from higher water levels on the dams. Storage of water in Lords Lake above the spillway overflow elevation 919.5 feet requires consideration of geotechnical risk, in addition to hydrologic risk as evaluated in this report. This analysis does not grant permission to install weir boards in the spillway, even for situations that meet DSO freeboard requirements. Any placement of weir boards in the spillway requires prior specific permission from the Dam Safety Office.

Embankment Stability

In response to DSO's 2015 inspection findings, CPT retained the services of Golder Associates, Inc. in 2019 to evaluate the stability of the Lords Lake East Dam. As part of this evaluation, Golder conducted a subsurface exploration program on April 2019.

Subsequent to the soils exploration, Golder conducted geotechnical analyses to evaluate the stability of the East Dam. The main findings of Golder's analyses included:

- Significantly different soil conditions than those encountered during previous explorations.
- Potential liquefaction of lower section of embankment soils and shallow surficial soils at the downstream toe area on the northern section of the East Dam.
- Pseudo-static factors of safety less than 1.
- Seismically induced displacement on the order of 0.2 to 1.4 feet.

DSO reviewed Golder's geotechnical report (2019); and, based on results from DSO's own geotechnical analyses using the information presented in the report, DSO concurred with Golder's findings.

As part of the 2020 periodic inspection of the facility, we conducted pseudo-static slope stability and seismic deformation analyses to review the results of analyses performed as part of our 2015 periodic inspection of the dam using an updated seismic hazard at the site.

The scenario earthquake for the location of the Lords Lake East Dam was updated based on the deaggregation results obtained from the USGS Seismic Hazard website[2]. For an earthquake with a return period of 2475 years, the primary risk at the site would be a Cascadia Subduction Intraslab event with a magnitude (M) of 7.1, at a distance (R) of 53 kilometers, with epsilon (ε_o) of 1.22, and with a PGA of 0.65 g's.

For the slope stability analysis, a pseudo-static coefficient of 0.25 g's was obtained based on the procedure outlined in Section 6.2.2 of the FHWA LRFD seismic manual (Federal Highway Administration, 2011).

Based on a review of the data presented in the report prepared by Golder (2019), we created a cross sections similar to Golder's sections A-A' and B-B'; and, used similar material properties for the different soil types in our analyses.

For our analyses, the critical failure surface is one that could breach the embankment and precipitate an uncontrolled release of the reservoir contents. Accordingly, the search criteria for the failure surface were limited to the upper section of the upstream slope. Further, seepage

[2] USGS Unified Hazard Tool accessed on April 20, 2020.

conditions were based on a lake level of 920.0 feet and a review of piezometric data from February 2015 to October 2019.

The slope stability analyses were conducted using the Slope/W computer program (Geo-Slope International, 2019). The results of the slope stability analyses yield pseudo-static factors of safety of 0.89 and 0.96 for sections A-A' and B-B', respectively (see Figures 7 and 8). Yield coefficients of 0.18 and 0.21 g's were determined for similar failure surfaces at Sections A-A' and B-B', respectively.

From these analyses, the computed factor of safety under seismic loading is less than the minimum 1.1 required by DSO. Although failure to meet the 1.1 factor of safety does not automatically indicate catastrophic failure, it rather dictates that more sophisticated displacement-type analyses are required to assess embankment deformations; and, to compare the displacements to the freeboard available prior to the earthquake in order to assess the likelihood of a breach.

As a first approach, embankment deformations induced by seismic loading were estimated using the simplified displacement method by Bray et al. (Bray et al., 2018). As noted by Dr. Bray[3], this method is also applicable to intraslab events. The method was used to obtain an estimate of seismically induced deformations without consideration of strength loss as a result of liquefaction of embankment materials. Accordingly, as noted by Bray et al., caution is warranted when interpreting the results.

For the analyses, the input ground motions were based on the M-R values obtained from the USGS website as noted before. The deterministic response spectrum was obtained with the BC Hydro ground motion prediction equation (Abrahamson et al., 2016). A value of 1.08 was used for epsilon (ε_o) to obtain a spectral acceleration value of 0.65 g's at a period of 0.01 seconds, i.e., similar to the USGS uniform hazard spectrum value.

The results of the simplified seismic displacement analysis, summarized in Figures 9 and 10, indicate that displacements induced at Sections A-A' and B-B' would be on the order of 0.3 to 1.1 feet and 0.2 to 0.9 feet, respectively.

As a second approach, we conducted 2-D, equivalent linear time history analyses and deformation analyses for Section B-B' using the computer programs Quake/W and Sigma/W, respectively. The time history analyses were based on a suite of seven two-component, outcrop horizontal ground motions. Details of the ground motions selected are provided in Table 1. The time history files were obtained from preliminary data developed for PEER's Next Generation Attenuation Subduction project.

For three of these motions, the closest distance was greater than would have been recommended when selecting ground motion records. For the Kushiro-oki event, the $V_{S,30}$ value corresponds to Site Class D, and to Site Class B/C for the others. However, there is not a large amount of records available for subduction intraslab events; hence, although some of these records would not meet common guidelines for their selection, we used these motions to ensure we had a minimum of seven ground motion records for our analyses.

[3] Personal communication in May 2019.

Table 1: Ground Motion Information

Earthquake Name	Magnitude	Closest Distance (km)	$V_{S,30}$ (m/sec)
Nisqually - Washington, USA - 02/28/2001 - PCEP - ENE & ENN	6.8	55	375
Tarapaca - Chile - 06/13/2005 - CUYA-L & CUYA-T	7.8	156	415
Geiyo - Japan - 03/24/2001 - EW & NS	6.8	44	396
Kushiro-oki - Japan - 01/15/1993 - D5C-EW & D5C-NS	7.6	142	262
Miyagi Pre - Japan - 04/07/2011 - MYGH04W2 & MYGH04S2	7.1	73	850
Punitaqui - Chile - 10/15/1997 - Illapel-L & Illapel-T	7.1	80	486
Iniskin - Alaska, USA - 01/24/2016 - ARTY-HNE & ARTY-HNN	7.1	290	750

The ground motions were linearly scaled so that the average of their geometric-mean response spectra approximately matched the deterministic spectrum. The resulting geometric mean spectrum of the two-component scaled motions, the average spectrum of the suite of ground motions and the target deterministic spectrum are shown in Figure 11. The scaled outcrop motions were converted to bedrock-within motions for use at the base of the 2-D Quake/W model using SHAKE (Schnabel et al., 1972).

The results of the Sigma/W deformation analyses show downward vertical movements on the order of 0.4 to 1 foot from the top of the embankment and lateral translation on the order of 0.1 to 1-foot displacements at Section B-B'. An example of the results obtained for the Nisqually ENE component record are presented in Figures 12, 13, and 14. Since the deformations computed would only be a small fraction of the normal 6 feet of freeboard, failure of the dam by overtopping along section B-B' during the design earthquake appears to be acceptably unlikely.

A limited number of 2-D time history and deformation analyses were performed for Section A-A' to assess the level of deformations and the effect the width of the potentially liquefiable zone may have on embankment response. The results of these preliminary analyses yield vertical deformations up to 90% greater than those at Section B-B'; and, up to an additional 20% greater when considering the full width of the lower embankment section as potentially liquefiable.

However, it is expected that deformations at Section A-A' would likely be greater than these preliminary estimates if more sophisticated analyses, e.g., 2-D nonlinear-effective stress, were used. Hence, the likelihood of an overtopping failure along Section A-A' can't be confidently assessed at this time.

The deformations estimated at Section A-A' should also be interpreted considering the limitations of the Equivalent Linear Model in Quake/W and the applicability of the dynamic deformation option in Sigma/W to liquefiable soils. Further, given the valley geometry, a 3-D model may predict markedly different results. Hence, these estimates are only considered to provide an order of magnitude-index for expected deformations.

It should be noted that the level of deformations estimated at Sections A-A’ and B-B’ could cause appreciable damage to appurtenance works such as outlets and toe drains, and transverse cracking on the upper section of the embankment is very likely to occur. In an extreme case, concentrated leak erosion could subsequently develop and lead to a catastrophic release of the reservoir contents. Furthermore, loss of freeboard as a result of strong ground motions would be of a greater concern for higher reservoir levels.

In conclusion, based on the results of the pseudo-static slope stability and deformation analyses the Lords Lake East Dam does not meet the minimum stability requirements under seismic loading conditions as outlined in the Dam Safety guidelines.

Accordingly, similar to our conclusions outlined in our 2015 inspection report, it is necessary to have a clearer picture of the lateral extent of the potentially liquefiable embankment zone and of the magnitude of embankment deformations induced by seismic loading.

To this end, it is necessary to better delineate the lateral extent of the loosely compacted embankment materials; to conduct additional dynamic analyses applying a more sophisticated model, e.g., nonlinear-effective stress, to better represent the behavior of the potentially liquefiable embankment materials and embankment response; and perhaps, to use a 3-D model to better evaluate the embankment response under seismic loading given the valley geometry.

Accordingly, CPT is required to retain the services of an engineering consultant to develop alternatives to improve the stability of the East Dam under seismic loading conditions to ensure the dam meets the minimum stability requirements as per the dam safety guidelines; and, to reduce earthquake-induced embankment deformations to minimize the risks of an uncontrolled release of the reservoir contents.

Instrumentation & Monitoring

CPT staff measure and record seepage flow, reservoir elevations, and piezometer levels at the dams on a monthly basis. Piezometer and seepage data from February 2015 to October 2019 for the North and East Dams were provided by CPT and reviewed for this inspection.

A review of the piezometer data indicates that there have not been significant changes in the phreatic surface that would indicate problems related to seepage through the embankment. The readings seem to track with lake level elevations.

The seepage data indicates that the seepage rates through both dams have remained consistent over the review period. The seepage flows appear to be influenced by the reservoir elevation, and not adverse trends are noted. Since seepage flows have remained clear, and have not increased over time, the seepage flows are considered normal.

It is required to establish alarm levels for the piezometers that would represent anomalous but not necessarily hazardous conditions. Piezometric readings at or above these levels will trigger additional monitoring and will require follow up by the engineering consultant to determine if further action is required. The alarm levels should be noted in the O&M Manual for the facility.

Operation and Maintenance

Our inspection revealed that the operation and maintenance of the dams appeared to be acceptable and the dams were in good condition. A review of the current Operation and Maintenance (O&M) Manual dated September 2019, showed that it contains adequate descriptions and procedures for the routine operation, maintenance, monitoring and inspection of the facility. Overall, the O&M Manual meets the minimum DSO requirements.

To improve the O&M Manual, it is recommended to:

- Include names and contact information of personnel responsible for conducting the operation and maintenance activities outlined in the manual.
- Include a reference to, or add as an appendix, the DSO Guidelines Part III – An Owner's Guidance Manual. This manual provides detailed information on operation, monitoring, annual inspection, and long-term maintenance of dams.

More detailed information regarding the O&M Manual can be found in the following documents:

Guidelines for Developing Dam Operation and Maintenance Manuals

DSO Guidelines Part II – Project Planning and Approval of Dam Construction and Modification

For further assistance updating the O&M Manual, please contact Jodi Goodman by electronic mail at jodi.goodman@ecy.wa.gov or by telephone at (360) 407-6613.

Emergency Preparedness

The Emergency Action Plan (EAP) for the dams, dated March 2015, was reviewed as part of this inspection. Based on this review, the EAP meets the minimum requirements for an EAP as recommended in the Dam Safety Guidelines. However, the plan has not been updated since 2015. Consequently, CPT & PTP need to review and update the contacts and phone numbers in the EAP as necessary.

To help you with the update, a draft EAP for the dam was submitted to Mr. Jablonski via electronic mail prior to the inspection. The EAP has been completed to the point that it is a basic functional document. The dam owner or their agent responsible for inspecting, operating, and maintaining the dam needs to complete their review of the EAP to confirm its appropriateness and accuracy, and to add any missing information.

More detailed information to help you with completing the draft EAP is provided in the following publication:

Guidelines for Developing Dam Emergency Action Plans

If you need any assistance or have additional questions, please contact Charlotte Lattimore by e-mail at charlotte.lattimore@ecy.wa.gov or by telephone at (360) 407-6066.

Annual Inspections

As required by the DSO guidelines, the owner has recently started annual inspections of the dams. Copies of the inspection forms have been submitted to the DSO in a timely manner. The results of the owner's inspections indicate that the dams are well maintained.

Remedial Actions Required

Below is a list of deficiencies we found during our periodic inspection. These deficiencies must be addressed to meet dam safety guidelines. Within **60 days** after receiving this report, please respond to Jodi Goodman, Environmental Specialist-Dam Compliance by email to jodi.goodman@ecy.wa.gov or letter with a plan and a timeline for completing the tasks listed below.

1. Periodically monitor the seepage areas along the left groin section and toe of the North Dam to determine if there are any changes in the seepage, e.g., increase in the amount of water, amount of sediments in the water, signs of distress on the slope, etc.
2. Periodically clear all vegetation from the area around the outlet drain pipe and weir concrete box at the North Dam to ensure that the vegetation does not hamper inspection or operation of the drain and weir; remove sediment accumulated in the weir concrete box; and, similar to the toe drain, monitor flow volume and characteristics on the weir.
3. The seepage on the downstream foundation area at the East Dam needs to be periodically monitored to determine if there are any significant changes on the amount of water and/or if signs of increased seepage such as bubbles or dirty water flow are present.
4. Periodically remove debris accumulated against the log boom.
5. A new, manufactured log boom and debris barrier needs to be provided for the reservoir and installed some distance upstream from the North Dam.
6. Periodically inspect and monitor the eastern section of the spillway wall on the embankment crest and hillside area above the spillway channel to ensure there are no ground movements that may be adversely affecting the structural integrity of the wall.
7. Repair damaged section on the v-notch weir at seepage measurement point number 1 at the East Dam.
8. Remove vegetation around seepage measurement points and sediment accumulated on the concrete weir boxes at both dams.
9. Install settlement monuments along the crest of both dams.
10. CPT is required to retain the services of an engineering consultant to develop alternatives to improve the stability of the East Dam under seismic loading conditions to ensure the dam meets the minimum stability requirements as per the dam safety guidelines; and, to reduce earthquake-induced embankment deformations to minimize the risks of an uncontrolled release of the reservoir contents.
11. Alarm levels for the piezometers that would represent anomalous but not necessarily hazardous conditions need to be established. Piezometric readings at or above these levels will trigger additional monitoring and will require follow up by the engineering consultant to determine if further action is required. The alarm levels need to be noted in the O&M Manual for the facility.
12. Update the Operations and Maintenance Manual and the Emergency Action Plan.

Reminder: In performing these required remedial actions for the dam, the dam owner is still responsible for complying with other applicable local, state, and federal requirements, including compliance with environmental review and permitting requirements. Please refer to the Governor's Office for Regulatory Innovation and Assistance - ORIA website for more information.

Recommended Actions

Overall, based on observations made during the inspection, the Lords Lake North Dam and East Dam are well maintained and operated. From the results of the inspection, the following are recommended actions to improve the maintenance and operation of the facility.

1. The City should consider the option of building a drain (e.g., French drain) along the western groin and downstream toe areas of the North Dam to collect the seepage and discharge it to an appropriate location downstream.

2. To facilitate monitoring of the seepage on the downstream foundation area at the East Dam, the City may consider constructing a simple drain system in the area of the seep. It appears that the seepage is localized to a small area, hence a shallow, short drain (e.g., French drain) constructed in the area would probably be adequate to collect the flow. The drain should be provided with an outlet pipe that could daylight at the location of seepage measurement point number 2 where the flow could be monitored.

3. As discussed during the inspection, we recommend that the interior of the outlet pipe between the tower and the overflow box at the downstream toe of the East Dam be inspected with the aid of a remotely controlled camera.

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Additional Resource Material

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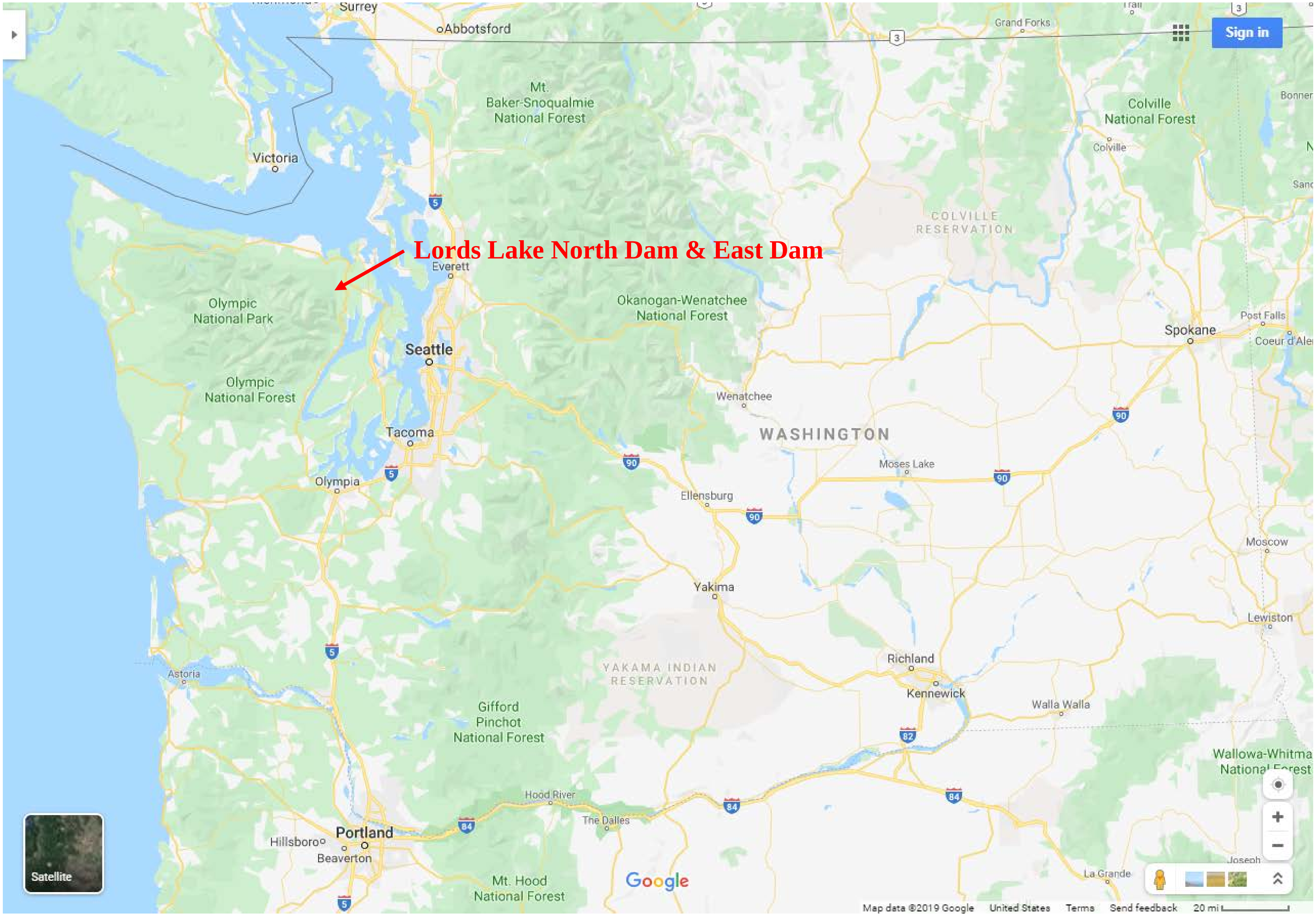

DEPARTMENT OF ECOLOGY State of Washington **WATER RESOURCES PROGRAM** **DAM SAFETY OFFICE**	2020 Periodic Inspection Report Lord Lake North Dam & East Dam Jefferson County, Washington DSO Files: JE17-0243 & JE17-0357	LOCATION	Figure: 1

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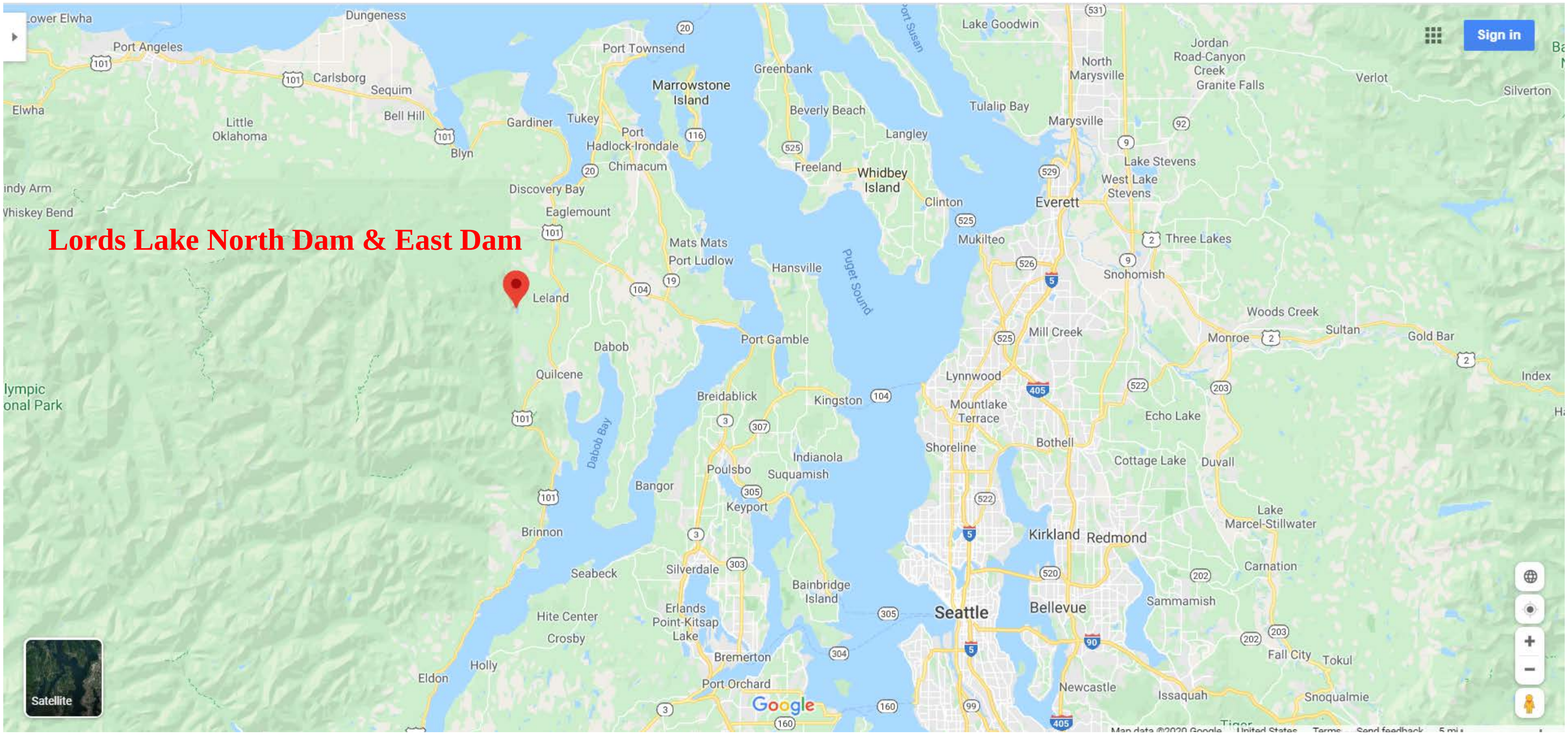

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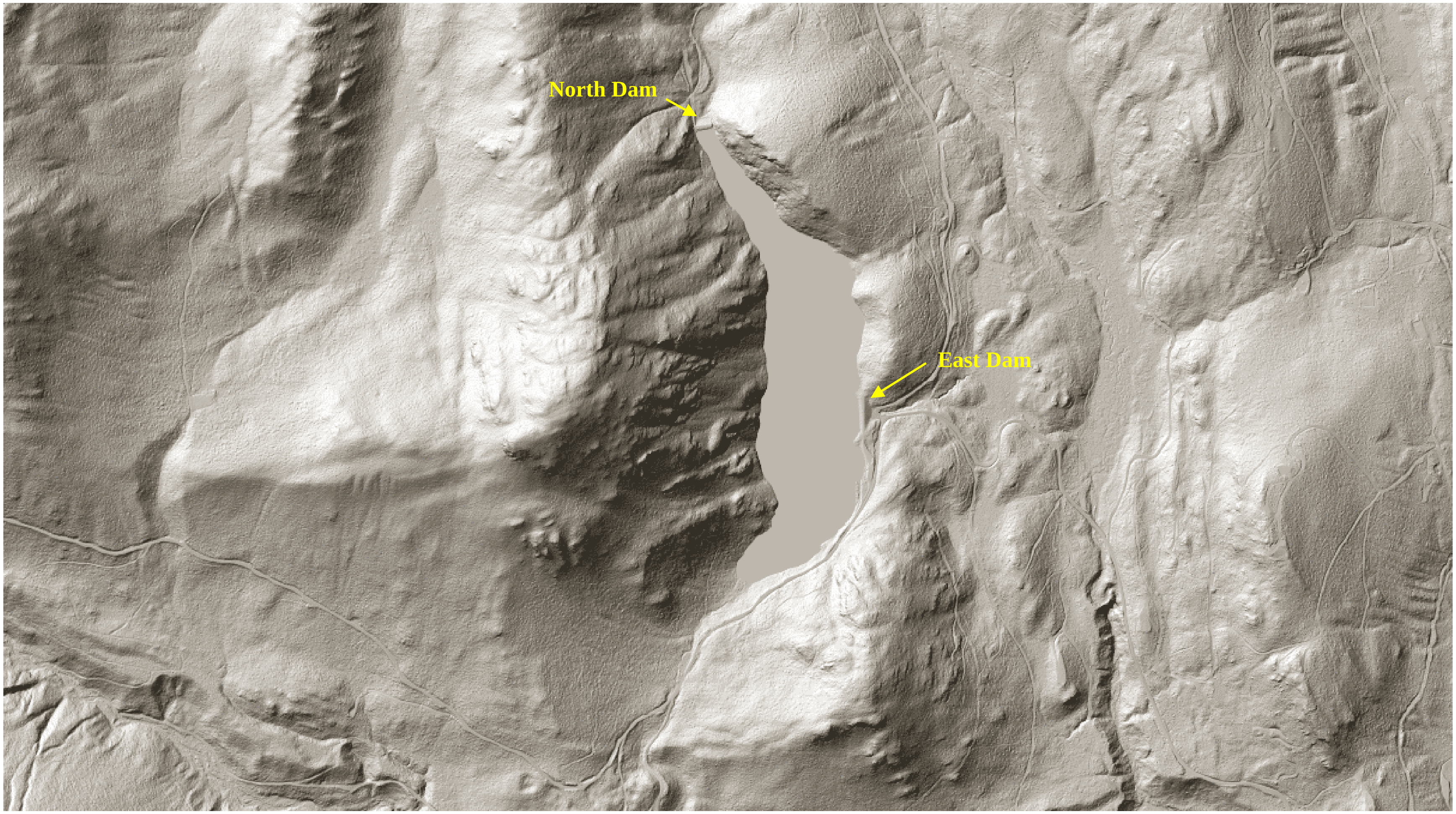

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DNR LIDAR COVERAGE - AREA

Figure:
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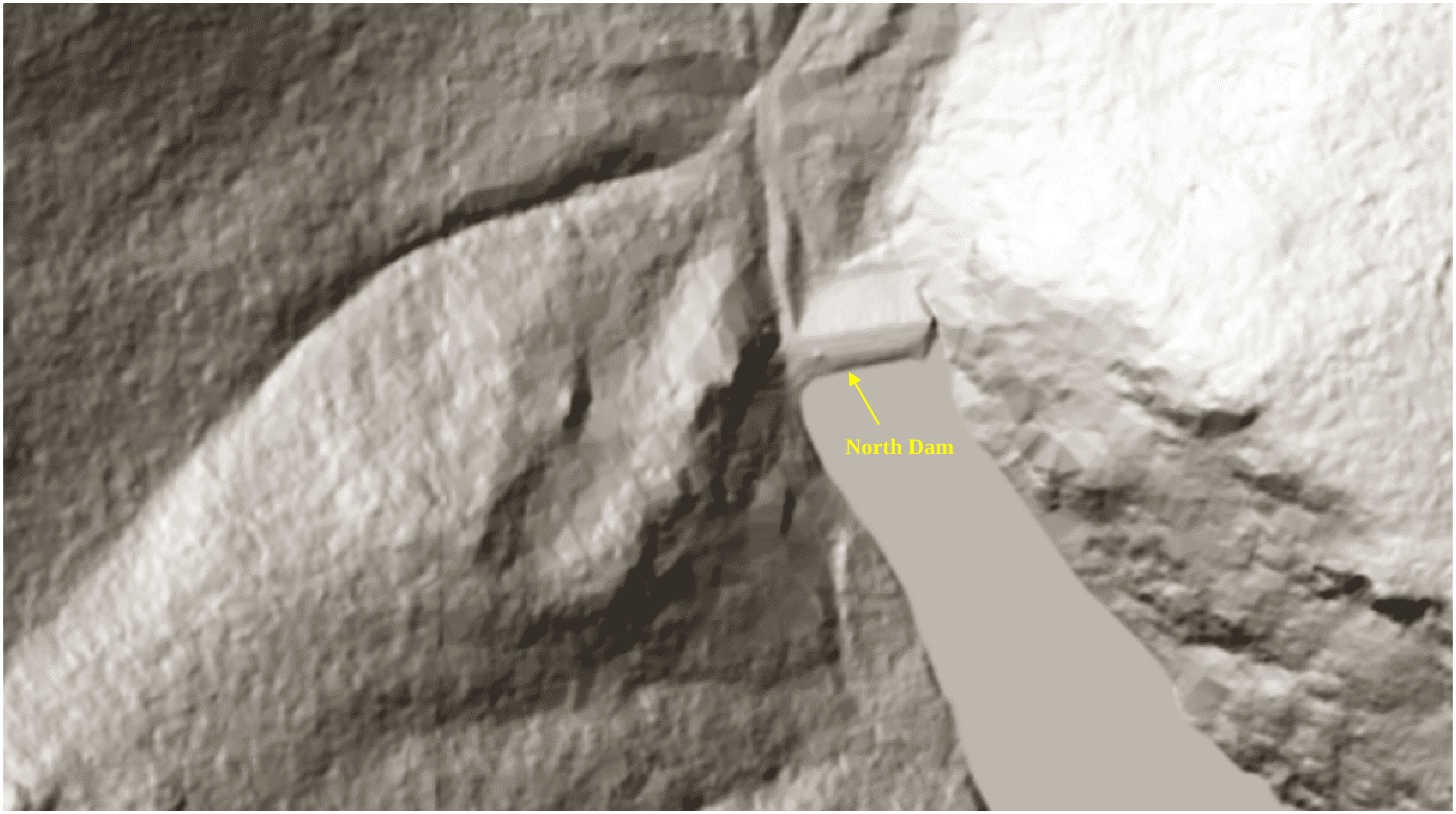

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DNR LIDAR COVERAGE
NORTH DAM

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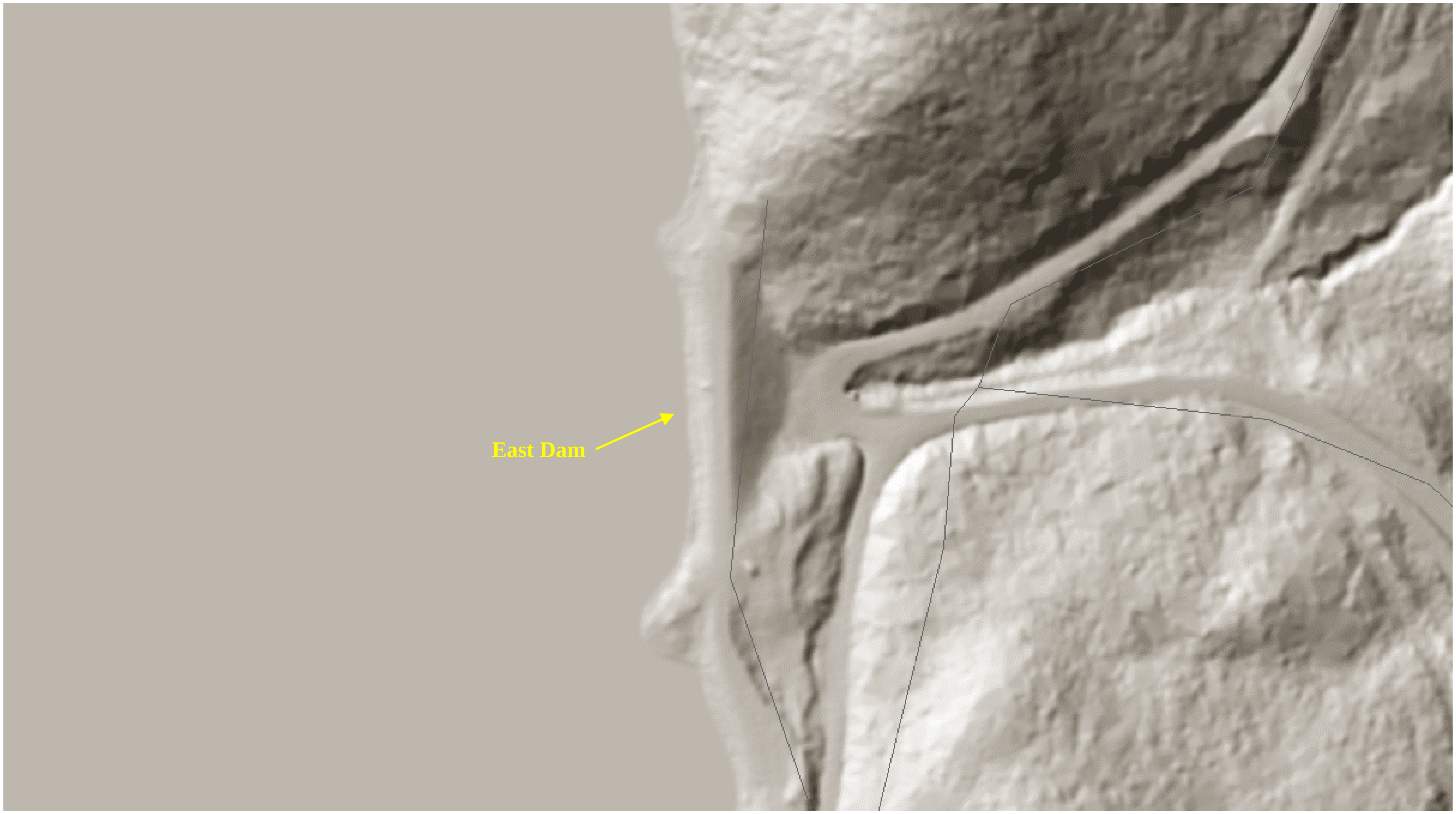

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DNR LIDAR COVERAGE
EAST DAM

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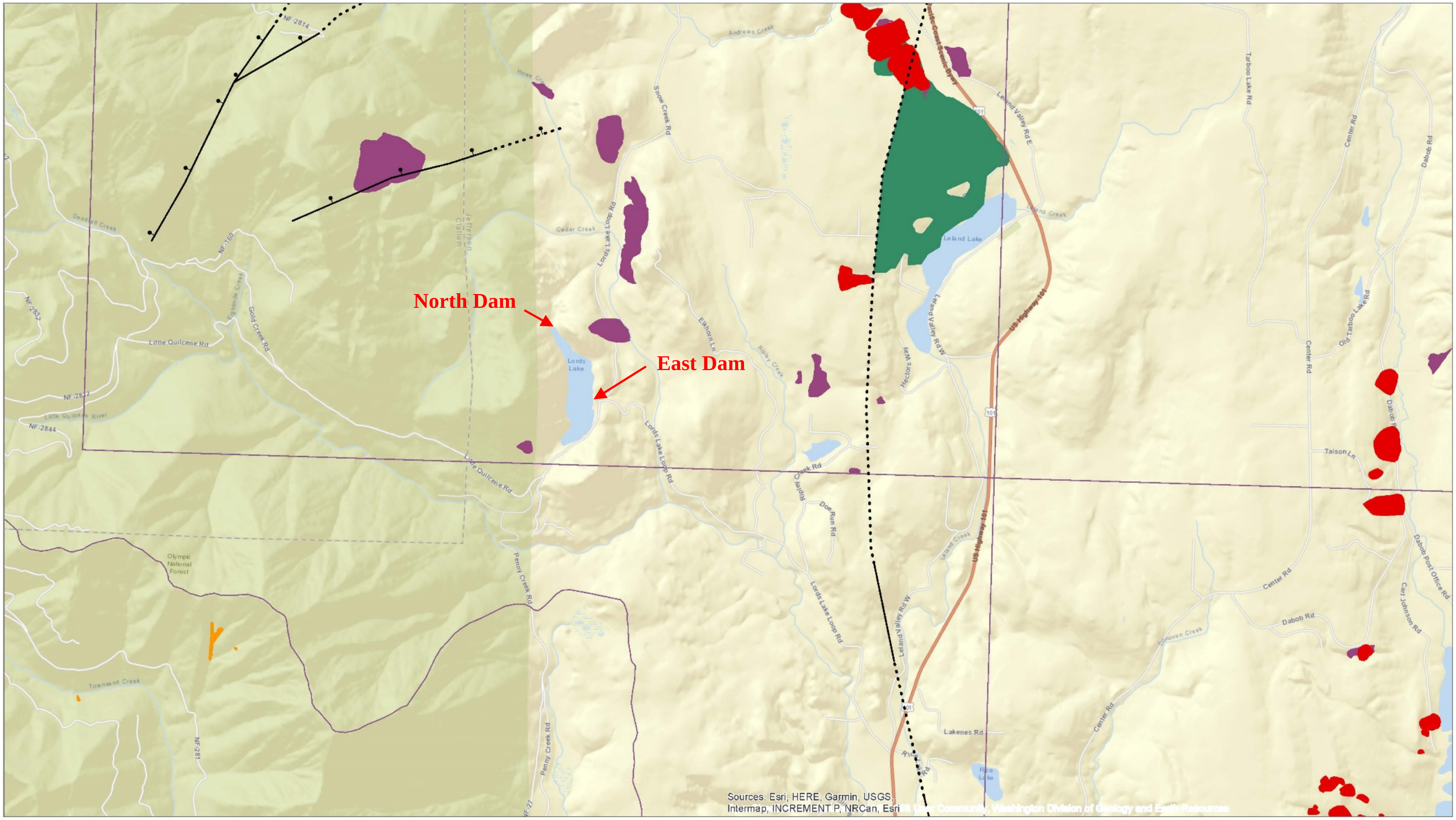
Lords Lake North & East Dams



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WA DNR-GIP LANDSLIDES

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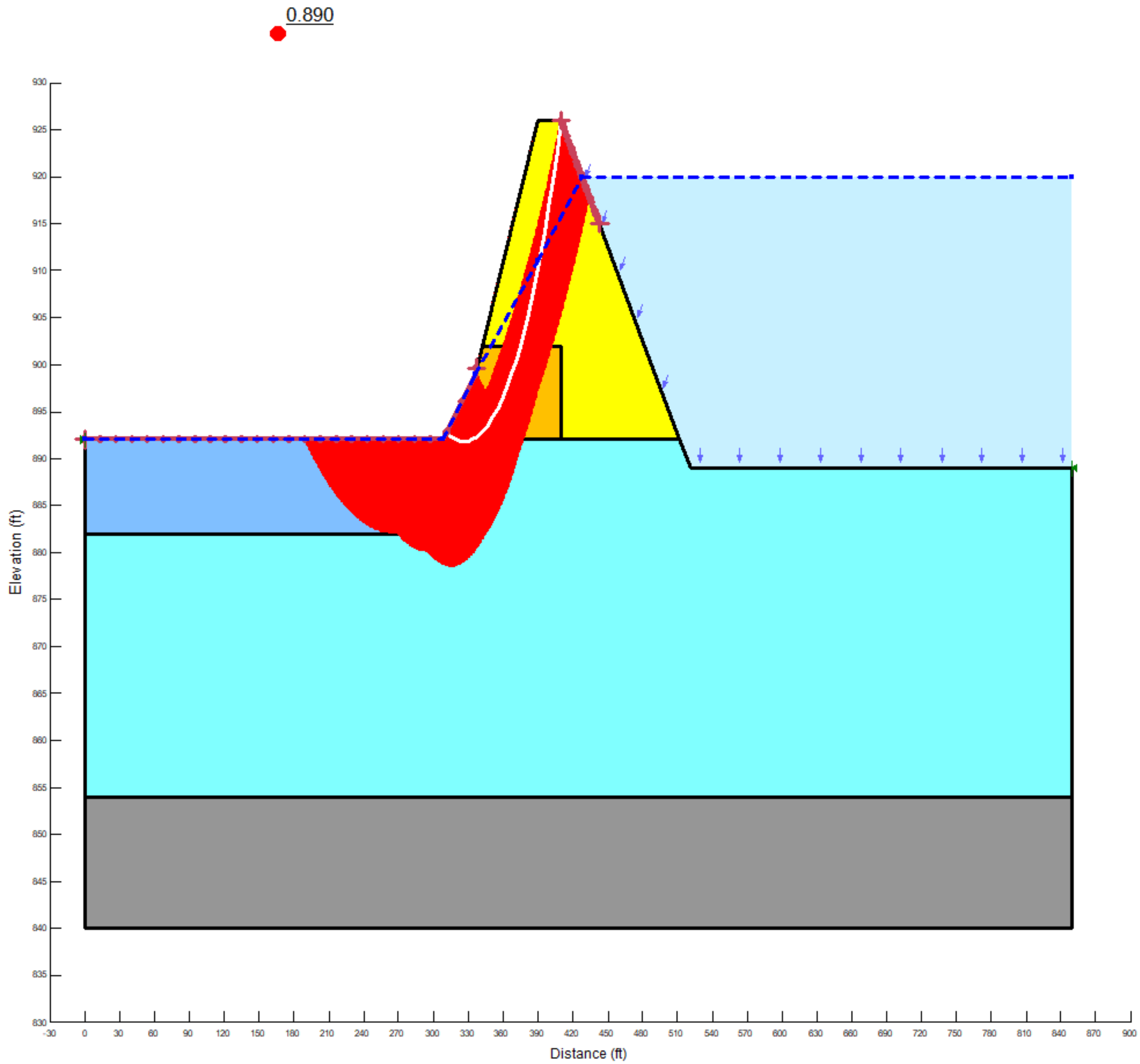

DEPARTMENT OF ECOLOGY State of Washington **WATER RESOURCES PROGRAM** **DAM SAFETY OFFICE**	2020 Periodic Inspection Report Lord Lake North Dam & East Dam Jefferson County, Washington DSO Files: JE17-0243 & JE17-0357	LORDS LAKE EAST DAM SECTION A-A’ PSEUDO-STATIC FACTOR OF SAFETY	Figure: 7

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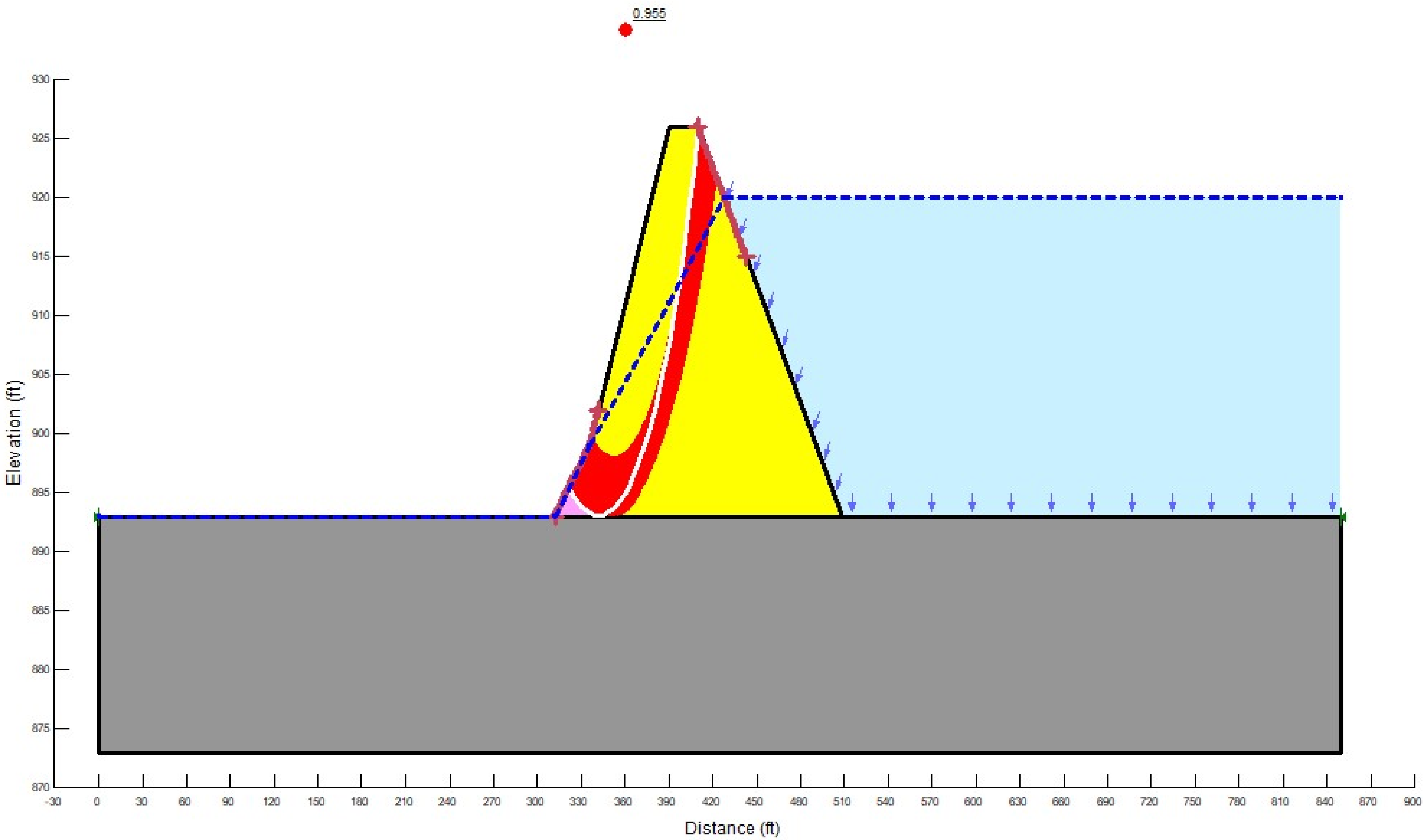

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LORDS LAKE EAST DAM
SECTION B-B'
PSEUDO-STATIC FACTOR OF SAFETY

Figure: 7

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Simplified Procedure for Estimating Seismic Slope Displacements in Subduction Zones

by Jonathan D. Bray, Jorge Macedo, and Thaleia Travasarou

Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zone Earthquakes, ASCE JGGE, 2018, 144(3): 04017124, DOI: 10.1061/(ASCE)GT.1943-5606.0001833

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.180		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.15	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.22	seconds	
Moment Magnitude (Mw)	7.1		
Spectral Acceleration (Sa(1.5Ts))	1.445585	g	Input the Spectral Acceleration at the base of the sliding mass assuming there is no material above it.

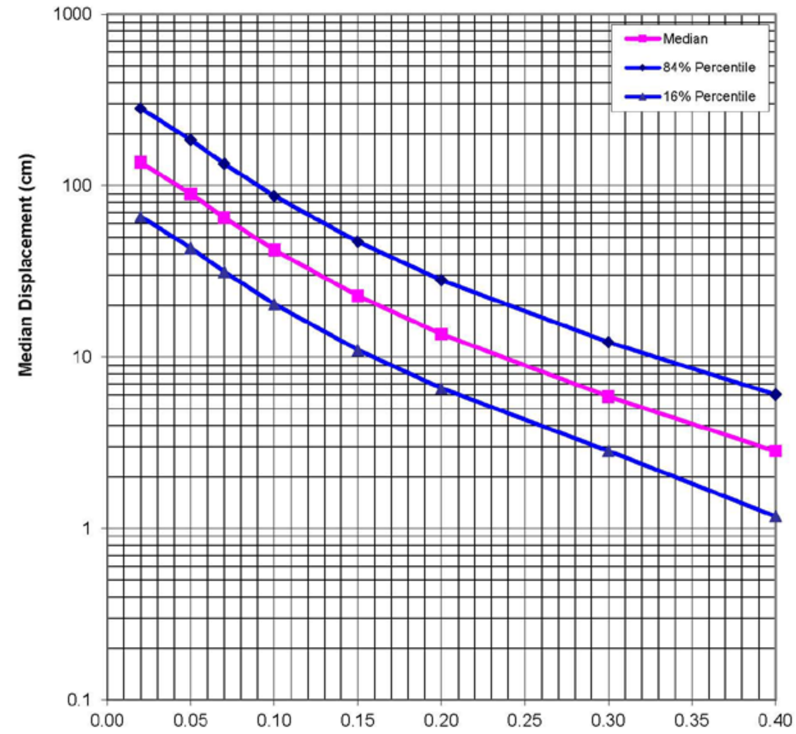
Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	183	cm

Intermediate Calculated Parameters				
Non-Zero Seismic Displacement Est (D)	16.52	cm	eq. (4) or (5)	2.80458656
Standard Deviation of Non-Zero Seismic D	0.73			

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. (2) or (3)
D1	**8.0**	cm	calc. using eq. (6)
D2	**16.5**	cm	calc. using eq. (6)
D3	**34.1**	cm	calc. using eq. (6)
P(D>d_threshold)	**0.00**		eq. (6)

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.00	136.3	136.3	281.6	65.9
0.05	0.00	89.3	89.3	184.6	43.2
0.07	0.00	64.9	64.9	134.1	31.4
0.1	0.00	42.0	42.0	86.8	20.3
0.15	0.00	22.7	22.7	46.9	11.0
0.2	0.00	13.6	13.6	28.1	6.6
0.3	0.00	5.9	5.9	12.2	2.8
0.4	0.07	3.0	2.82	6.03	1.17



Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.8, Ts between 0 and 2 s, Sa between 0.002 and 4.5 g, M between 5.5 and 9
7. When Ts is close to 0.00 s and the sliding block is assumed to be rigid, Ts can be set to 0.00 s and Sa(1.5Ts) can be set to PGA.
8. When a value for D is not calculated, D is < 0.5cm
9. ky should be estimated with a slope stability program; the simplified equations shown below provide approximate values.
10. Examples of how Ts is estimated are shown below.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

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Simplified Procedure for Estimating Seismic Slope Displacements in Subduction Zones

by Jonathan D. Bray, Jorge Macedo, and Thaleia Travasarou

Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zone Earthquakes, ASCE JGGE, 2018, 144(3): 04017124, DOI: 10.1061/(ASCE)GT.1943-5606.0001833

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters		
Yield Coefficient (ky)	0.210	
Initial Fundamental Period (Ts)	0.14	seconds
Degraded Period (1.5Ts)	0.21	seconds
Moment Magnitude (Mw)	7.1	
Spectral Acceleration (Sa(1.5Ts))	1.505175	g

Based on pseudostatic analysis
1D: Ts=4H/Vs 2D: Ts=2.6H/Vs

Input the Spectral Acceleration at the base of the sliding mass assuming there is no material above it.

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	15.5	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	13.12	cm	eq. (4) or (5) 2.57444097
Standard Deviation of Non-Zero Seismic D	0.73		

Results			
Probability of Negligible Displ. (P(D=0))	**0.00**		eq. (2) or (3)
D1	**6.3**	cm	calc. using eq. (6)
D2	**13.1**	cm	calc. using eq. (6)
D3	**27.1**	cm	calc. using eq. (6)
P(D>d_threshold)	**0.41**		eq. (6)

Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
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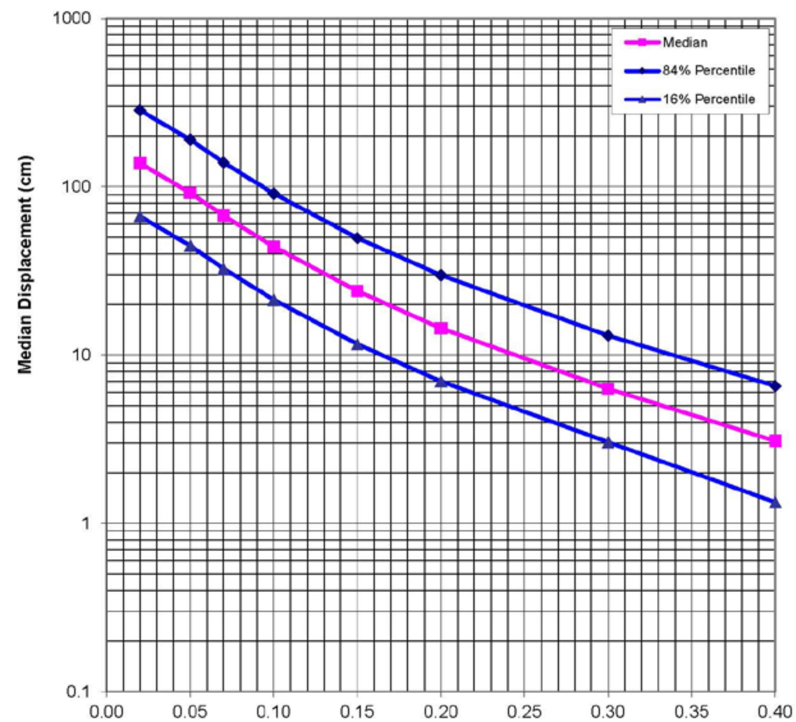
9. ky should be estimated with a slope stability program; the simplified equations shown below provide approximate values.
10. Examples of how Ts is estimated are shown below.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.00	137.4	137.4	283.9	66.5
0.05	0.00	91.9	91.9	189.9	44.5
0.07	0.00	67.2	67.2	139.0	32.5
0.1	0.00	43.9	43.9	90.7	21.2
0.15	0.00	23.9	23.9	49.4	11.6
0.2	0.00	14.4	14.4	29.8	7.0
0.3	0.00	6.3	6.3	13.0	3.0
0.4	0.06	3.2	3.08	6.53	1.33



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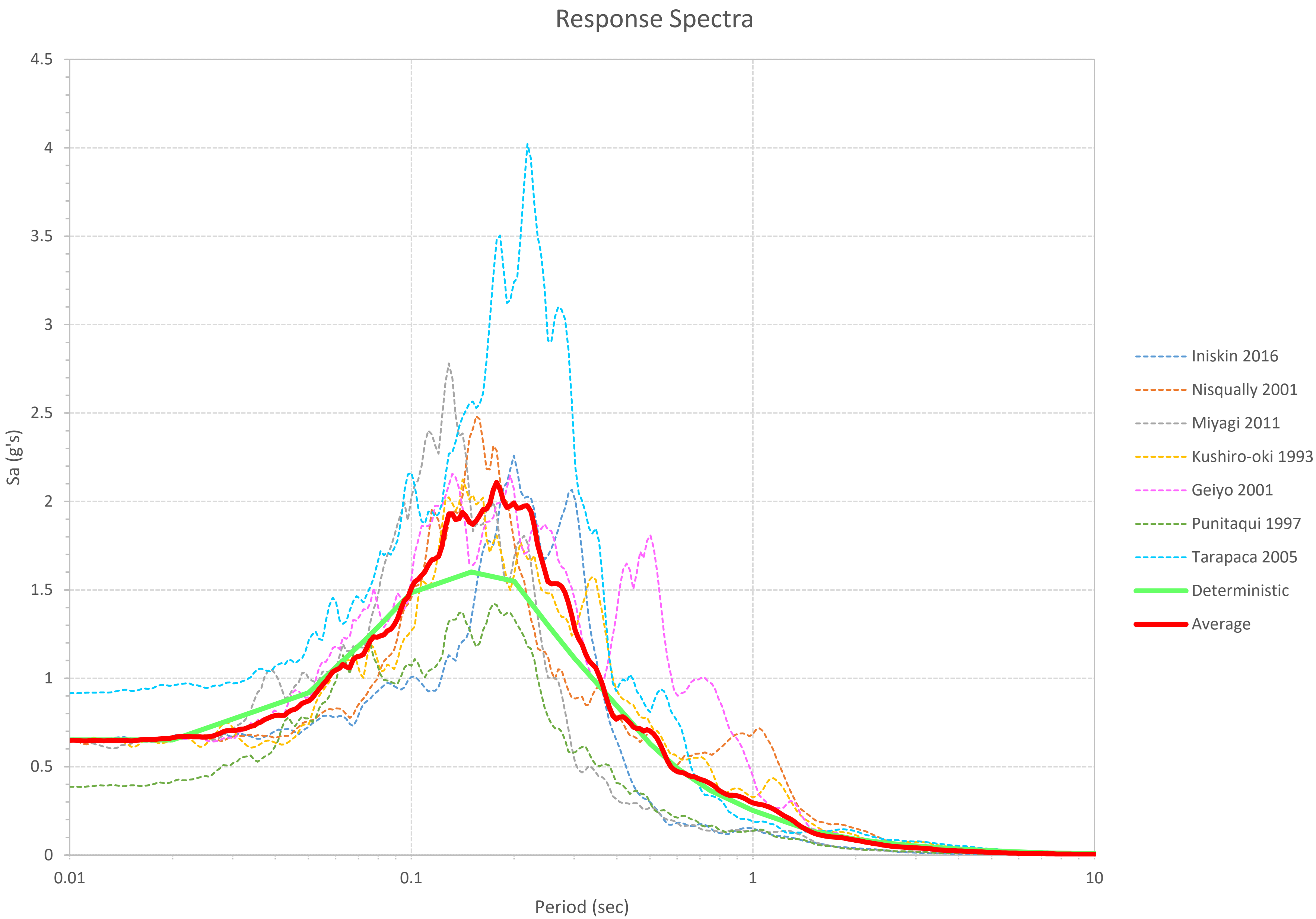

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RESPONSE SPECTRA

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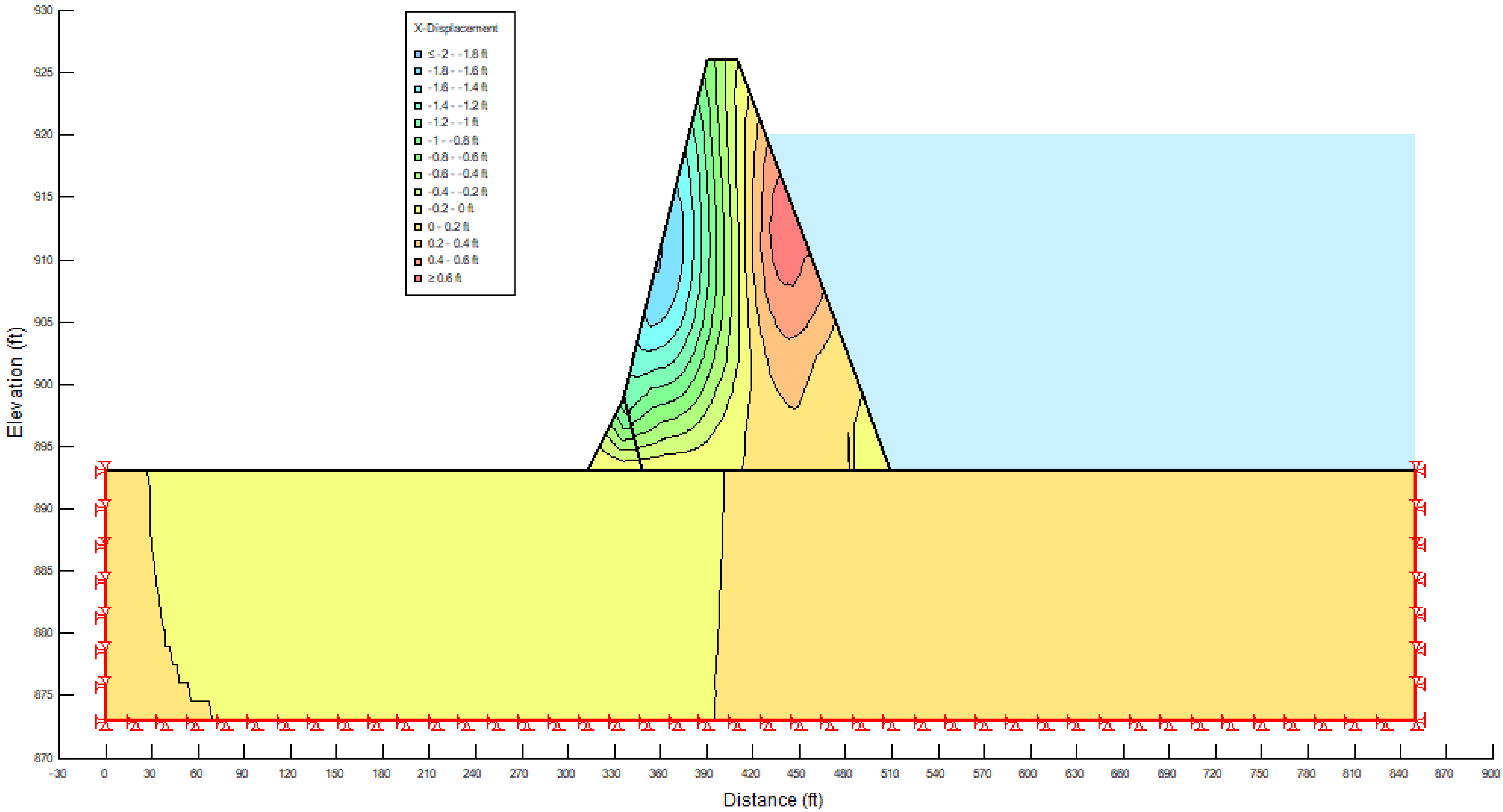

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SECTION B-B'
SIGMA/W X DISPLACEMENTS
NISQUALLY ENE COMPONENT

Figure:
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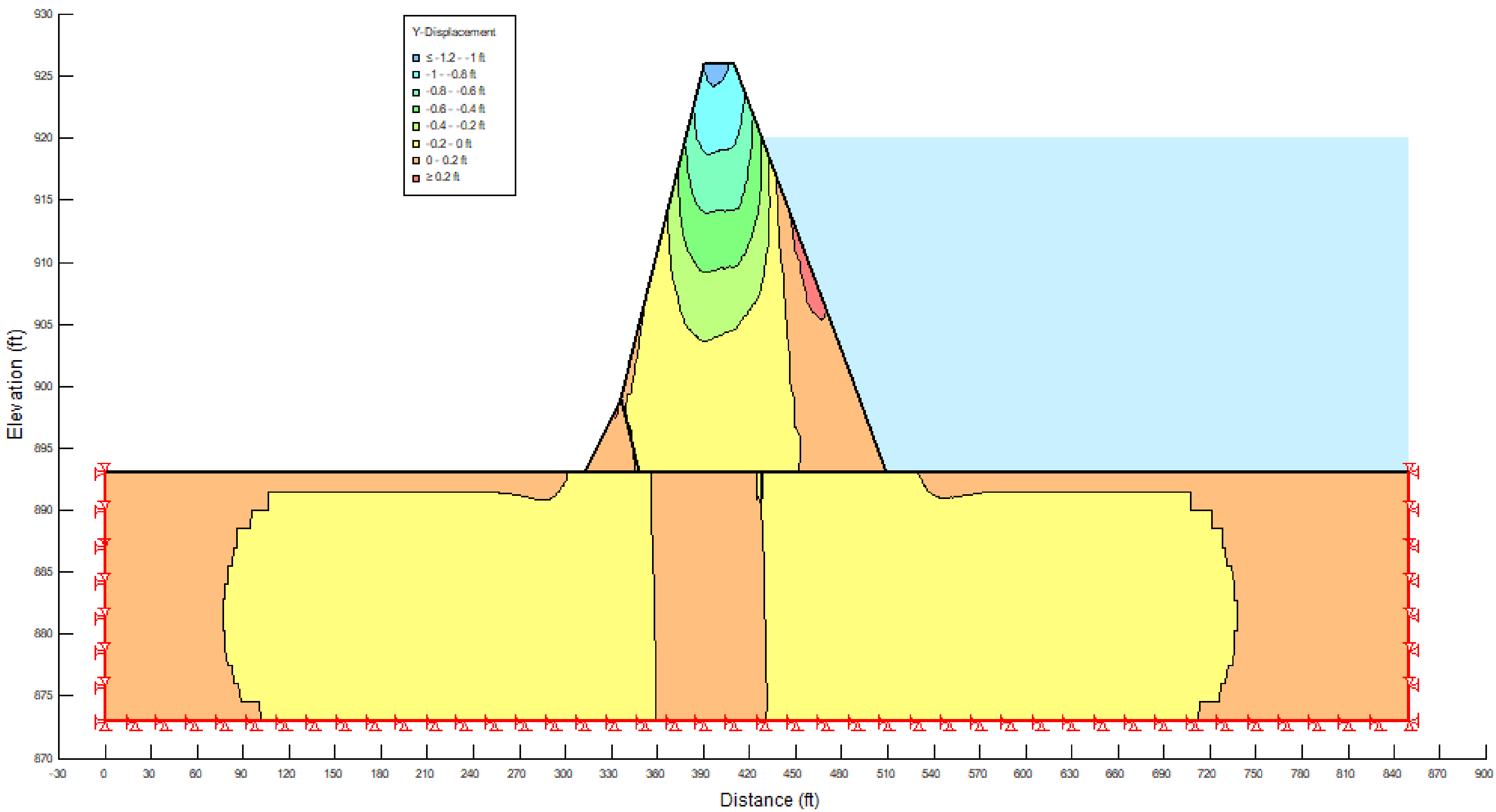

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SECTION B-B'
SIGMA/W Y DISPLACEMENTS
NISQUALLY ENE COMPONENT

Figure:
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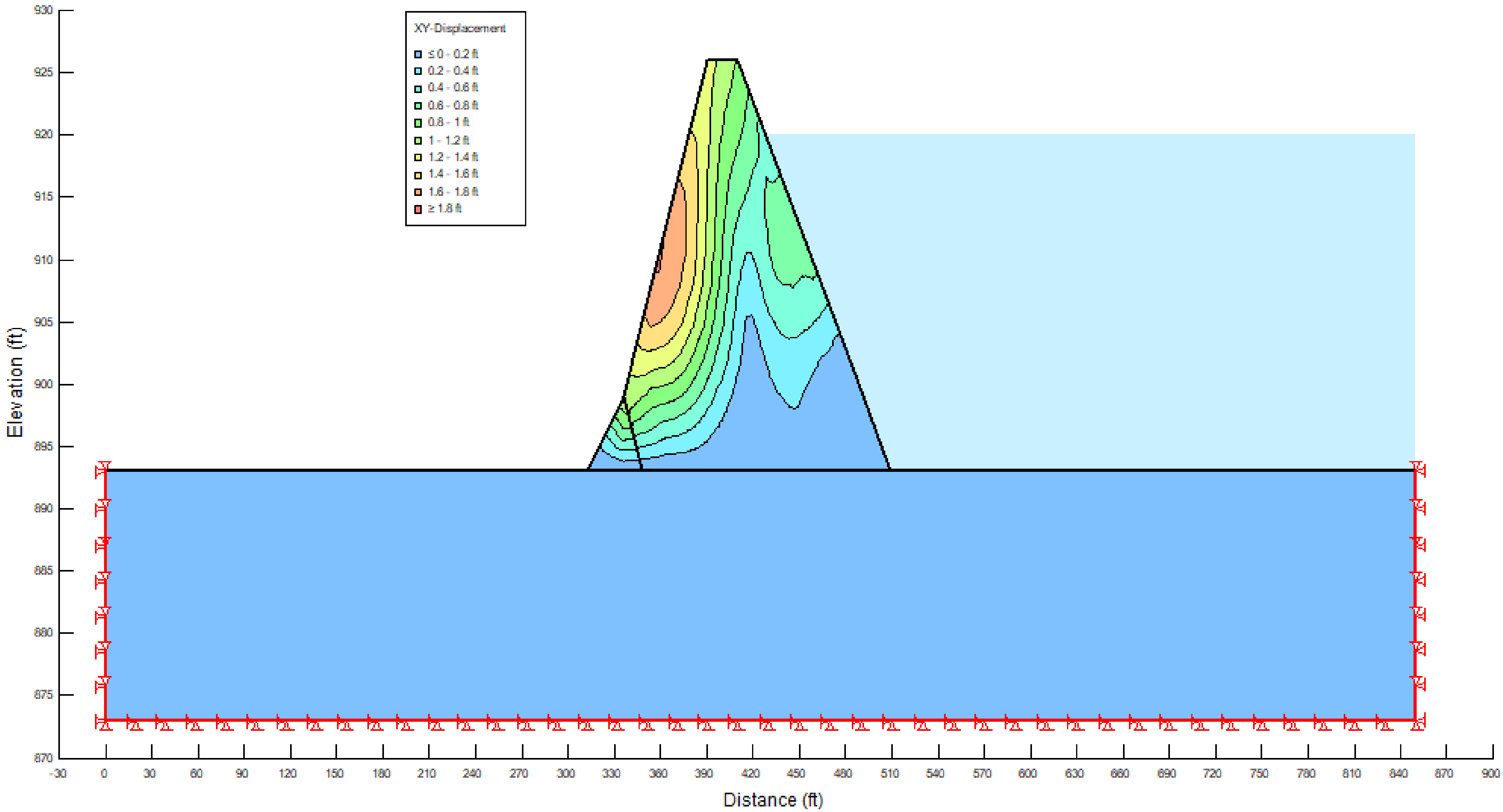

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SECTION B-B’
SIGMA/W X-Y DISPLACEMENTS
NISQUALLY ENE COMPONENT

Figure:
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Appendix A: Project Data Sheet

General

State I.D. No.	North Dam: JE17-0243 East Dam: JE17-0357
Owner and Operator	City of Port Townsend
Location	4.5 miles northwest of Quilcene North Dam: T 28 N, R 2 W, Section 23 North Dam: Lat 47.88773° Long -122.937612° East Dam: T 28 N, R 2 W, Section 33 East Dam: Lat 47.880833° Long -122.931498°
Construction Completed	1956-1957
Major Modifications	NA
Purpose	Water Supply
Public Water System ID No.	69000R
NPDES / State Discharge Permit No.	NA
NID Condition Assessment	North Dam: Satisfactory (inspected 2020) East Dam: Poor (inspected 2020)
Downstream Hazard Potential	1A - High
Downstream Flood Path	Unnamed stream channels to Howe Creek to Little Quilcene River to Quilcene Bay (Hood Canal)

Reservoir

Watershed	Unnamed tributary of Howe Creek, within Little Quilcene River watershed
Drainage Area	0.5 square miles
Spillway Overflow Elevation	919.5 feet (reservoir depth 34.5 feet)
Surface Area at Spillway Overflow	56 acres
Active Storage at Spillway Overflow	1480 acre-feet
Dam Crest Elevation	926.0 feet (reservoir depth 41.0 feet)
Surface Area at Crest Elevation	59 acres

Active Storage at Crest Elevation	1850 acre-feet

Dam Embankment

North Dam	
Type	Zone earthfill with central sloping core and rock toe
Structural Height	40 feet
Hydraulic Height	36 feet
Crest Elevation (typical)	926.0 feet
Crest Elevation (minimum)	925.8 feet
Elevation Datum	NGVD (Spillway = 919.5 feet)
Crest Length	165 feet
Crest Width	15 feet
Upstream Slope	3H:1V
Downstream Slope	2H:1V
East Dam	
Type	Homogeneous earthfill with rock toe
Structural Height	42 feet
Hydraulic Height	40 feet
Crest Elevation (typical)	926.0 feet
Crest Elevation (minimum)	925.8 feet
Elevation Datum	NGVD (Spillway = 919.5 feet)
Crest Length	600 feet
Crest Width	15 feet
Upstream Slope	3H:1V
Downstream Slope	2.25H:1V

Overflow Spillway

Type	Concrete chute with side channel weir entrance
Location	North Dam, right (east) abutment
Discharge Capacity	50 cfs at water level 921.0 feet
(full open, with no weir boards)	150 cfs at water level 923.8 feet

	250 cfs at water level 926.0 feet
Overflow Elevation	919.5 feet (concrete base, with no weir boards)
Overflow Control Section	16 feet wide, vertical sides, 6.5 feet deep (3 bays, each 5.3 feet wide, 16 feet total width)
Discharge channel – section	5 feet wide, vertical sides, 4 feet deep
Discharge channel – profile	115 feet long: 25 feet at slope 0.025 ft/ft, then 90 feet at slope 0.40 ft/ft (2.5 H:1V)
Inflow Design Flood – Discharge	147 cfs
Inflow Design Flood – Storm	Step 8, 100% PMP; Long duration storm
Inflow Design Flood – Precipitation	24 hr = 15.50 inches, 72 hr = 25.13 inches, snowmelt = 1.74 inches, total = 26.87 inches

Emergency Spillway

NA	

Outlet Works

Type	Concrete tower with low outlet pipe
Location	East Dam, center of dam
Discharge Capacity (both gates open)	32.6 cfs (21.1 mgd) at water level 919.5 feet (WL 919.5 feet = reservoir depth 34.5 feet) 32.0 cfs (20.7 mgd) at water level 908.0 feet (WL 908.0 feet = reservoir depth 23.0 feet) 31.0 cfs (20.0 mgd) at water level 890.0 feet (WL 890.0 feet = reservoir depth 5.0 feet)
Tower Dimensions	7 feet x 7 feet (external) x 42 feet high 5 feet x 5 feet (internal dimensions)
Gated Orifices	2.5 ft. x 2.5 ft. at elevation 902.5 feet (elev. 902.5 feet = reservoir depth 17.5 feet) 3.0 ft. x 3.0 ft. at elevation 885.0 feet (elev. 885.0 feet = reservoir depth 0.0 feet)
Flow Controls	Upstream – sluice gates at each orifice plus outlet pipe entrance Downstream – valves on water supply pipeline

Outlet Conduit – section	30 inch diameter steel pipe
Outlet Conduit – profile	230 feet long at slope 0.02626 ft/ft (2.626%), then 9.3 miles at avg. hydraulic slope 0.57% to City Lake
Drawdown Capacity (both gates open) (max drawdown at minimal inflow)	1.1 feet/day at water level 919.5 feet (WL 919.5 feet = reservoir depth 34.5 feet) 1.3 feet/day at water level 908.0 feet (WL 908.0 feet = reservoir depth 23.0 feet) 1.9 feet/day at water level 890.0 feet (WL 890.0 feet = reservoir depth 5.0 feet)

Seepage Instrumentation and Monitoring

Seepage control features	Drain rock toe on both dams
Seepage measurement type	Bucket- timer and weir plate
Seepage measurement location	3 drainage collection points along downstream toe area of East Dam; 1 drainage collection point on downstream toe of North Dam
Seepage monitoring type	4 piezometers on each dam
Seepage monitoring location	Downstream slope near crest and toe

Appendix B: Photographs

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Photo 1: Weir and drain pipe at downstream toe area of North Dam.

Photo 2: Weir and drain pipe at downstream toe area of North Dam during 2015 DSO inspection.

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Photo 3: Upstream slope at East Dam.

Photo 4: Upstream slope at East Dam (October 2002).

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Photo 5: Southern saddle section of East Dam.

Photo 6: Southern saddle section of East Dam cleared of vegetation after inspection.

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Photo 7: Seepage area on downstream foundation area at East Dam.

Photo 8: View of spillway at entrance channel. Concave area highlighted.

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Photo 9: Spillway entrance channel at North Dam.

Photo 10: V-notch weir at seepage measurement point number 1. Damage section highlighted.

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