Report to:

City of Astoria

1095 Duane Street

Astoria, OR 97103

GEOTECHNICAL REPORT

PHASE 2 SEISMIC STABILITY EVALUATION
BEAR CREEK DAM
ASTORIA, OREGON

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EXECUTIVE SUMMARY

Bear Creek Dam is a concrete gravity structure that impounds the domestic water supply for the City of Astoria, Oregon. Oregon Water Resources Department has expressed concern about the seismic stability of the structure considering the current understanding of the regional seismic environment. Previous stability evaluations performed using simplified models concluded that the structure is marginally stable under static conditions and unstable under earthquake loading; however, these analyses neglected stability provided by the basalt abutments. The current study was completed to evaluate the stability of the structure taking into account the geology and geometry of the site.

Bear Creek Dam is situated in a unique geologic environment that complicates the evaluation of global stability. The structure is partially founded on a large block of competent basalt which is surrounded and underlain by weak sedimentary rocks. The basalt block has been fractured and eroded over geologic time resulting in a narrow valley with basalt side slopes and a sedimentary rock valley floor. The dam was sited and constructed to take advantage of the stronger basalt side slopes, even though the central section of the dam is founded on the sedimentary unit. Due to the geometry of the narrow valley, the basalt abutments have a significant stabilizing effect on the structure. The foundation conditions of the base also have a large influence on the stability. Previous evaluations analyzed only the maximum dam section founded on the weaker sedimentary rock, which resulted in the marginally-stable conclusion by others.

As-built cross-sections surveyed during original dam construction were used to develop a three-dimensional, geologic model of the dam and abutments. The three-dimensional model was used to develop stability cross-sections at several locations along the length of the dam. Structural evaluation of the dam concluded that the dam will act as a monolith, and as such can transmit load from the central portion of the dam to the abutments. The results of the stability analyses at each location were combined considering this load transfer to calculate a composite factor of safety for the entire structure under static and seismic loading conditions. The analyses confirm that the dam derives a large portion of its stability from the abutments. The central portion of the dam has a static factor of safety below 1.0 when analyzed individually, but the composite factor of safety is approximately 1.2 when the stabilizing effects of the abutments are considered. The composite factor of safety drops below 1.0 under seismic loading, and displacements during the design earthquake are calculated to be between 10 and 30 inches.

There is currently some uncertainty of the foundation conditions in the right abutment. Limited subsurface data and the as-built sections suggest the right abutment may be founded on sandstone or a mixture of sandstone and basalt. For conservatism, the analyses summarized in this report assume the right abutment bears on sandstone, which is significantly weaker than basalt. Additional explorations are recommended to better define the right abutment foundation conditions. If the right abutment is founded on basalt instead of sandstone, then seismic displacements could be significantly less than current calculations indicate.

1. INTRODUCTION

Bear Creek Dam is a concrete gravity structure that impounds domestic water for the City of Astoria, Oregon. The improved understanding of the seismic hazard of the region has prompted the City of Astoria to reevaluate the stability of the structure for ground motions that would result from a Cascadia Subduction Zone earthquake. This report summarizes static and seismic stability analyses performed for the structure.

1.1 Previous Stability Evaluations

The static and seismic stability of Bear Creek Dam has been evaluated previously using analysis techniques and design ground motions that reflected the understanding of the seismic environment at the time the evaluations were performed. The following paragraphs summarize the analyses and conclusions of the previous evaluations.

1993 Harza Evaluation. Harza Northwest, Inc., analyzed the stability of the Bear Creek Dam and summarized their work in a report titled “Bear Creek Dam – Final Report” dated November 1993. They attempted to perform a three-dimensional stability analysis using software developed by the Corps of Engineers, but the program was unable to generate a suitable equivalent horizontal plane to represent the steeply-sloping abutments of the dam. Two-dimensional stability analyses were completed on the maximum height section of the dam using limit-equilibrium analysis techniques. Linear and bi-linear uplift pressure profiles were used to calculate the sliding stability of the structure. Seismic stability was evaluated for a peak ground acceleration of 0.15g and a representative period of 0.3 seconds. The analyses modeled the foundation with soil strength parameters (i.e. softened sedimentary rock), and calculated factors of safety against sliding between 0.7 and 0.81 for static loading. Harza acknowledged that the abutments of the dam have a large influence on the stability, but concluded that the factor of safety is near 1.0 under static loading. Concepts to improve the stability were evaluated, including post-tensioned anchors and downstream buttresses. The report concluded that a roller-compacted concrete buttress downstream of the dam was the most cost-effective way to mitigate the stability.

1996 CH2M Hill Evaluation. CH2M Hill analyzed the stability of the structure and summarized their work in a technical memorandum titled “Technical Memorandum No. 4 – Bear Creek Dam Stability Analysis” dated November 1, 1996. Two-dimensional limit-equilibrium stability analyses were performed on the maximum height section to calculate the factor of safety against sliding. A linear distribution of uplift pressure was assumed to act on the base of the dam so long as vertical stress was calculated to be greater than uplift pressure. Where vertical stress was calculated to be less than pore pressures, full hydrostatic head from pool elevation was assumed to act. This “cracked base” analysis is more conservative than the analysis performed by Harza in 1993. Seismic stability was evaluated using a pseudo-static coefficient equal to 0.3. The analyses were based on several simplifying assumptions, and calculated that an effective cohesion equal to 450 psi is necessary along the entire base of the dam to achieve a static factor of safety equal to 1.0. CH2M Hill concluded that this amount of cohesion is not realistic, and like Harza, indicated that the abutments are providing stability that two-dimensional analyses at the maximum dam section cannot capture.

Three-dimensional stability analyses were performed using the computer program ANSYS in an attempt to quantify the stabilizing effect of the abutments. It was concluded that an effective cohesion equal to 27 psi was necessary to calculate a factor of safety equal to 1.0 under static conditions. An effective cohesion of 27 psi was judged to be reasonable for the conditions at the site, and CH2M Hill calculated that building a rock buttress or lowering the pool elevation 10 to 20 feet would meet target factors of safety equal to 3.0 under static conditions and 1.0 under seismic conditions.

1.2 Scope of Work

This report summarizes work completed for Phase 2 of the seismic stability evaluation of Bear Creek Dam. The scope of work for the current study included reviewing previous stability evaluations and as-built documents from City archives, developing a geologic model of the dam, performing laboratory testing on samples recovered during Phase 1 explorations, analyzing the stability of Bear Creek Dam under static and seismic loading, and estimating seismic displacements using Newmark-type analyses. The overall objective of the Phase 2 study is to perform stability analyses that account for the stabilizing effects of the basalt abutments.

2. SITE AND PROJECT DESCRIPTION

Bear Creek Dam is a concrete gravity dam located approximately 17 miles southeast of Astoria (see Figure 1). The dam is a 90-foot high concrete gravity dam with a centrally located, gated spillway (see Figure 2). The dam was originally constructed to a height of 75 feet between 1911 and 1913. In 1953, the dam was raised 15 feet to its current height (crest elevation of 665 feet). The dam is located in a narrow canyon with slopes varying between 1H:1V and ½H:1V. The land above the dam is gently sloped with ridges and hill tops rising gradually to the south and east.

Since construction of the dam, leakage has been noted primarily through the left abutment. Foundation grouting was performed in the left abutment after original construction and again when the dam was raised in an attempt to reduce the amount of seepage. The dam has two zones of prominent vertical cracks; one in the left abutment and one in the right abutment. The crack zones are shown on the site plan in Figure 2 and in elevation on Figure 3. The crack zones are near the lateral extents of the original dam and are likely related to settlement caused by the 1953 dam raise.

3. GEOLOGY

The geology at Bear Creek Dam was summarized in the Phase 1 of the seismic stability evaluation. The following excerpts are reproduced from the report titled "Geotechnical Data Report – Phase 1 Geotechnical Investigation, Bear Creek Dam Seismic Stability" dated March 2014 (Cornforth, 2014).

3.1 Regional Geology

Bear Creek Dam is located in the Northern Oregon Coast Range physiographic province. The geology of the region is defined by Oligocene and Miocene sedimentary deposition and Miocene volcanic and intrusive activity. The bedrock in the area consists of the Wickiup Mountain Member of the Astoria Formation that contains weak siltstones, mudstones, and fine-grained sandstone interbeds. The Astoria Formation was subsequently intruded and displaced by volcanic rocks of the Yakima Basalts (Columbia River Basalt Group), which inter-finger with the sedimentary units of the Astoria Formation. In addition, regional uplift and accretion have elevated the older marine sediments.

The geologic map (Figure 4) by Schlicker, et al. (1972), indicates that the bedrock in the vicinity of Bear Creek Dam is Miocene intrusive basalt. However, based on the petrology of hand and core specimens collected at the site, it appears that the rock is from the Columbia River Basalt (CRB) Group. This suggests that the bedrock at the dam location is likely invasive basalt, which formed by displacing soft, unconsolidated sediments in deltaic and estuarine coastal environments during deposition. It appears that Bear Creek follows a fault line or possibly a fault splay that likely bisects the dam abutments.

According to the geologic map, Bear Creek Dam is located on a large, ancient landslide. The ancient landslide headscarp is located approximately two miles east of the dam. Evidence of ancient landslide terrain and landslide debris was observed at the site; however, there is no indication that the dam or the abutments are actively moving or have moved in the recent past.

3.2 Site Geology

Geologic units encountered at the dam site, listed from youngest to oldest, include the following: overburden soil, Columbia River Basalt, and Astoria Formation. A brief discussion of each geologic unit is given below.

3.2.1 Overburden Soil

Overburden soils at the site consist of fill, colluvium, landslide debris, and alluvium. Colluvium and landslide debris consisted of soft to medium stiff, brown, slightly sandy, slightly clayey silt with numerous gravel- to cobble-sized basalt and sandstone fragments.

3.2.2 Columbia River Basalt

The basalt encountered in outcrops is typically medium hard to hard (R3 to R4), gray, slightly to moderately weathered, fine- to medium-grained. It is highly jointed on average with some zones very highly jointed and others only slightly jointed. A ½- to 3-inch wide gouge zone was observed at the base of left abutment on the downstream side of the dam and by the spillway apron on the right abutment. It is unclear if the gouge zones are related. The basalt exhibits columnar jointing and very few vesicles or highly altered zones.

3.2.3 Astoria Formation

The sedimentary outcrops encountered consisted of extremely soft to very soft (R0 to R1), thinly bedded, orange-brown siltstone and sandstone.

3.3 Subsurface Conditions

Subsurface conditions at the dam are complex and vary over very short distance. The following sections describe the distinct formations encountered during recent explorations at the site.

3.3.1 Concrete

The dam is comprised of two different types of concrete. Up to 16 feet of very highly to moderately fractured concrete with sand- to gravel-sized rounded aggregate was encountered in borings advanced from the dam crest. The concrete has scattered woody debris/organics and air voids throughout. This concrete corresponds with the raised portion of the dam that was constructed in 1953. No cold joints/lift lines were distinguishable in the concrete cores in this section of the dam.

Very highly to moderately fractured concrete with sand- to cobble-sized angular basalt aggregate was observed below a depth of 16 feet in borings advanced from the dam crest. This concrete is associated with original dam construction. The original dam concrete has larger, angular aggregate and the matrix has few organics or air voids. Concrete cold joints/lift lines are evident and typically tight (no visible separation along the lift).

3.3.2 Fill

Fill encountered in borings consists of soil and road base aggregate, and is generally less than 5 feet thick. Fill material ranges from slightly sandy, slightly clayey silt to dense, angular, crushed rock fragments (road base).

3.3.3 Columbia River Basalt

The basalt encountered in the borings is typically hard to very hard (R4 to R5), gray, slightly weathered, and fine- to medium-grained. It is highly jointed on average with some zones being very highly jointed and others only slightly jointed. In boring CC-4, cement-grout infilling up to 6-inches thick was observed in the joints of the basalt from elevations 610 to 600 feet.

3.3.4 Marine Sediments

Marine sediments were encountered in boring CC-3 beneath a 5-foot thick layer of basalt immediately underlying the dam. The marine sediment consists of stiff, slightly sandy, slightly clayey silt with numerous gravel- to cobble-sized rock fragments of decomposed to weathered siltstone and sandstone.

3.3.5 Astoria Formation

The Astoria Formation was encountered in boring CC-3 at a depth of 118 feet. The material consists of extremely soft to very soft (R0 to R1), gray, moderately weathered siltstone that is moderately fractured.

4. FIELD EXPLORATIONS

Subsurface explorations were completed at Bear Creek Dam by Harza in support of their 1993 stability evaluation and by Cornforth Consultants during Phase 1 of the seismic stability evaluation. Summary logs of the borings are included in Appendix A.

5. LABORATORY TESTING

Ten unconfined compression tests were completed on core samples of concrete and rock recovered during recent subsurface explorations. The tests were completed by the Earth Mechanics Institute at Colorado School of Mines in general accordance with ASTM D7012. The results from the tests are included in Appendix B, and were used to estimate the strength of concrete and the concrete-rock interface for stability analyses.

6. GEOLOGIC MODELING

The geology of the site is complex and changes over short distances. The margins of an invasive basalt flow are chaotic, and can exhibit extensive interfingering of basalt and host material. To develop a better understanding of the geometry of the dam and underlying geologic contacts, a three-dimensional model of the dam was developed.

6.1 Dam Foundation and Geometry

Bear Creek Dam is situated in a narrow valley that was erosionally formed as Bear Creek downcut through a body of invasive basalt. During construction of the dam, overburden soil and loose, fractured rock were removed to prepare the foundation. The geometry of the excavation was measured by the City's resident engineer, and recorded in an as-built survey notebook. As-built cross-sections were prepared on 5- to 10-foot spacing along the length of the dam. The sections indicate original ground surface, base of excavation, proposed dam section, and approximate limits of different types of materials excavated. Scans of the cross sections are included in Appendix C.

The cross-sections record different material types encountered during foundation excavation. These material types are denoted as "E", "R", "H&H", and "Clay". We have interpreted the first three designations as Earth, Rock, and Half & Half, respectively. At the base of the valley, the original design with one main seepage cutoff trench was modified to include three cutoff trenches. The as-built book also provides a detailed plan and isometric view of the additional cutoff trenches, labeling the drawing "Area of the Sandstone Formation of the Dam Foundation." We speculate that the dam designers originally anticipated basalt to be encountered in the base of the dam, and added the additional seepage cutoff trenches once they encountered sandstone. It is important to note that the resident engineer labeled the sandstone excavation in the base of the valley as "R". Our interpretation of this designation is that the resident engineer did not differentiate between sandstone and basalt. This becomes important in our analyses in how we model the right abutment foundation conditions.

6.2 Three-Dimensional Model

The as-built plans and cross sections from the resident engineer's notebook were used as the basis for a three-dimensional model of the dam and its foundation. The model was generated using SketchUp Pro software, a visualization software package that allows data to be spatially related in a model that can be rotated and viewed in any angle. Borings completed at the site were also added to the model to provide data on the lithology under the dam and abutments. Selected isometric views of the model are shown in Figures 5 through 7.

7. SEISMIC DESIGN PARAMETERS

Seismic design parameters were developed using the United States Geological Survey (USGS) 2,475-year return period, probabilistic uniform hazard response spectra. Response spectra were used for a modal analysis of the dam and to select representative acceleration time histories for use in Newmark analyses to estimate seismic displacements.

7.1 Seismic Hazard

Ground motions for analysis of seismic stability were based on the 2008/2010 and 2014 USGS National Seismic Hazard Maps (NSHM) (Petersen, et al. 2008, updated 2010, and Peterson et al. 2014) and the ground motion prediction equations used by USGS for development of the 2014 NSHM. The 2014 NSHM release includes the spectral acceleration values for peak ground acceleration (PGA), 0.2-second and 1.0-second spectral periods at the 475-year and 2,475-year return period. A comparison of the 2008/2010 NSHM with the limited 2014 data indicates a slight reduction in the overall hazard. For purposes of developing the seismic hazard for the project, we have used the 2008/2010 seismic hazard levels with the updated 2013/2014 ground motion prediction equations.

At this time, there is no consensus in the earthquake engineering community regarding a specific return period for the seismic design and evaluation of dams. However, it is generally agreed that critical structures, such as dams, should be designed for ground motions with return periods of thousands of years. Based on discussions with the City of Astoria and the Oregon Department of Water Resources, the 2,475-year probabilistic uniform hazard response spectra level motions were used for development and analysis of the seismic hazard.

The USGS Interactive Deaggregation shows that three seismic sources contribute to the seismic hazard at the dam site: i) a shallow, random crustal earthquake; ii) a Cascadia Subduction Zone (CSZ) interface earthquake; and iii) a CSZ intraslab earthquake. Figures 8 through 10 show histogram representations of the deaggregation results for 0 second, 0.2 second and 1.0 second spectral acceleration, respectively, for the 2,475-year return period. Table 7-1 shows that the Peak Ground Acceleration (PGA) is 0.61g for the 2,475-yr event, and also shows that the predominant contributor (94 percent) to the overall seismic hazard is from a CSZ interface earthquake

Table 7-1. Results of Probabilistic Seismic Hazard Deaggregation for Bear Creek Dam

Return Period	PGA (g)	Contribution from Principal Sources at PGA (%)		
		Random Crustal	Cascadia Subduction Zone Interface	Cascadia Subduction Zone Intraslab
2,475-year	0.61	< 3	94.0	4.7

7.2 Seismic Sources

For each seismic source, earthquake magnitude and distance pairs were developed based on the deaggregation results. Table 7-2 shows the source distance and magnitude pairs for each source.

Ground motion models and target response spectra reflected in Table 7-2 are discussed in detail in the following paragraphs.

Table 7-2. Recommend Ground Motions

Earthquake Source	Motion Percentile	Geology	Earthquake Magnitude	Distance (km)	PGA[1] (g)	$SA_{0.2s}$[1] (g)	SA_{1sec}[1] (g)
Random Crustal	84th	Rock Site	6.5	10	0.41	0.97	0.27
CSZ Interface	60th	Rock Site	9.0	23	0.56	1.18	0.58
CSZ Intraslab	84th	Rock Site	7.2	53	0.54	1.24	0.30
Uniform Hazard Spectral Acceleration on rock for 2,475-year Return Period					0.61	1.34	0.58

[1]Spectral accelerations based on ground motion prediction equations (see below).

7.2.1 Crustal Earthquake Sources

The 2014 NSHM utilizes Next Generation Attenuation (NGA) models developed under the leadership of the Pacific Earthquake Engineering Research Center (PEER) to estimate seismic ground motions.

There are five NGA crustal models: Abrahamson & Silva 2013, Boore & Atkinson 2013, Campbell & Bozorgnia 2013, Chiou & Youngs 2013, and Idriss 2013. USGS uses Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs. We included all five models to develop target response spectra and assigned equal weight (0.22) to each model, except for Idriss 2013, which was given approximately one-half the weight (0.12) of the others (see Figure 11). For reference, we have also shown on the figure the 2,475-yr Uniform Hazard spectra.

7.2.2 CSZ Interface Earthquake Sources

USGS adopted the following four ground motion models in the 2014 NSHM (Petersen et al. 2014), with weights as indicated:

- Atkinson and Macias (2009) – 0.30
- Atkinson and Boore (2003, global model) – 0.10
- Zhao et al. (2006) – 0.3
- BC Hydro (2012) – 0.3

These models and weighting factors were used to derive the target response spectrum (5% damping ratio) for the M=9.0 and R=23 km interface earthquake. The mean plus 0.25 standard deviation (60th percentile) ground motions corresponded well to the spectral accelerations from the 2008/2010 USGS Uniform Hazard Spectra for a 2,475-year return period. Figure 12 shows the response spectra for these four attenuation relationships. The weighted average for these four response spectra was used as the target spectrum.

7.2.3 CSZ Intraslab Earthquake Sources

USGS adopted the following four ground motion models in the 2014 NSHM (Petersen et al. 2014), with weights as indicated:

- Atkinson and Boore (2003, Cascadia model) – 0.167
- Atkinson and Boore (2003, global model) – 0.167
- Zhao et al. (2006) – 0.33
- BC Hydro (2012) – 0.33

These models and weighting factors were used to derive the target response spectrum (5% damping ratio) for the M=7.2 and R=53 km intraslab earthquake. The mean plus standard deviation (84th percentile) ground motions corresponded well to the spectral accelerations from the 2008/2010 USGS Uniform Hazard Spectra for a 2,475-year return period. Figure 13 shows the response spectra for these four attenuation relationships. The weighted average for these four response spectra was used as the target spectrum.

7.3 Ground Motion Time History Selection

Acceleration time histories for each seismic source were selected to match the target response spectra. Since the NGA relationships are based on the geometric mean of the two horizontal components of the acceleration time history, we compared the geometric mean spectra for each ground motion record with the target response spectra, although both horizontal components of the selected motions were used in the deformation analyses. Selected ground motion records were chosen considering earthquake magnitude, site geology, site-to-source distance, and response spectra acceleration characteristics.

7.3.1 Ground Motion Search Criteria

A database of recorded earthquake time histories are maintained by several organizations including: the Pacific Earthquake Engineering Research Center (PEER) Strong Motion Database; the PEER/Next Generation Attenuation (NGA) Database; and the Consortium of Organizations for Strong Motion Observation System (COSMOS) Virtual Data Center. We searched these databases using target and search criteria for the random crustal fault source shown in Table 7-3.

Table 7-3. Random Crustal Fault Earthquake Ground Motion Search Criteria

Criteria	Geology	Earthquake Magnitude	Closest Distance to Rupture Area (km)	PGA (g)
Target	Rock Site	6.5	10	0.41
Time History Search	Rock Site, $V_{S,30}$ of 350-760 m/s	6.0-7.0	5 to 30	-

There are limited recorded acceleration records for megathrust, subduction zone earthquakes. To model the CSZ interface earthquake, ground motion time histories were evaluated from the 1985 Michoacan, Mexico interface event (M=8.1), and the 2011 Tohoku, Japan event (M=9.0). For these two earthquakes, we searched the database maintained by the COSMOS Virtual Data Center (http://db.cosmos-eq.org/scripts/default.plx) for ground motion records that were recorded on rock or firm soil sites with a source-to-site distance between 15 and 130 km. We also evaluated synthetic time histories developed for projects in the region for CSZ interface earthquakes.

For the intraslab earthquakes, ground motions were evaluated from two events, the 2001 El Salvador earthquake (M=7.6) and the 1997 Michoacan, Mexico earthquake (M=7.1). Earthquake motions were obtained from the COSMOS database for rock sites with source-to-site distances between 35 and 95 km. Synthetic time histories that have been developed for a project in the Portland region for CSZ intraslab earthquakes were also evaluated.

7.3.2 Ground Motion Scaling Procedure

Ground motions were scaled and stretched to match the response spectra of the recorded motions to the target spectrum. A scaling factor was applied to the acceleration component of the acceleration time history to shift the response spectrum along the vertical axis. The scaling factor was determined as the ratio of the average spectral acceleration of the individual record to that of the target spectrum over this period. Subsequently, the ground motion was stretched, or scaled along the horizontal axis, so that the average response spectrum of the three ground motions closely matched the target response spectrum in the period range of 0 to 1.0 seconds. The stretch factor is applied to the time component of the acceleration time history.

7.3.3 Crustal Ground Motions

Crustal ground motions that met the search criteria were then compared with the target spectra shown in Figure 11. Five acceleration time histories were selected that closely matched the target response spectra, with particular emphasis in the period range from 0 to 0.5 seconds. Selected ground motions and scaling factors are shown in Table 7-4. No stretch factors were applied to the crustal motions. The individual response spectra (geometric mean of the horizontal pair) for the selected time histories are shown on Figure 14, and an average response spectrum for the five selected time histories is plotted on Figure 15. Unscaled acceleration time history plots for the selected ground motion records are included in Appendix D.

Table 7-4. Selected Ground Motions for Random Crustal Fault Earthquake Event

Earthquake	Station	$V_{s,30}$ (m/s)	Magnitude	Closest Distance to Rupture (km)	Recorded PGA (g)	Scaling Factor
San Fernando (02/09/1971)	Castaic – Old Ridge Route	450	6.6	22.6	0.30	1.45
Mammoth Lakes (05/25/1980)	Long Valley Dam (Upper Left Abutment)	537	6.1	15.5	0.34	1.51
Morgan Hill (04/24/1984)	Gilroy Array #6	663	6.2	9.9	0.26	1.44
Loma Prieta (10/17/1989)	San Jose – Santa Teresa Hills	672	6.9	14.7	0.27	1.42
Northridge (01/17/1994)	Toganga – Fire Station	506	6.7	22.3	0.25	1.52

7.3.4 CSZ Interface Ground Motions

Response spectra from the 1985 Michoacan, the 2011 Tohoku, and a synthetic motion from a previous Cornforth project were compared to the target response spectrum (as shown in Figure 12). The response spectra were scaled and stretched to match the target spectral accelerations for the period range of interest for the dam (0 to 0.5 seconds). Table 7-5 shows parameters of the selected earthquake ground motions.

Table 7-5. Selected Ground Motions for Interface Subduction Zone Earthquake Event

Earthquake	Station	Site Geology	Magnitude	Closest Distance to Rupture (km)	Recorded PGA (g)	Scaling Factor	Stretch Factor
Tohoku 3/11/2011	Toyosato	Soil and Rock	9.0	151.0	0.62	1.0	1.1
Michoacan 9/19/1985	Caleta de Campos	Rock	8.1	38.3	0.38	1.4	1.1
CSZ Interface Synthetic	-	Rock	8.5	174.0	0.12	4.3	1.1

Figure 16 shows the response spectra for the three scaled and stretched ground motions. Also shown in Figure 16 is the target response spectrum developed from the three ground motion attenuation models. Figure 17 compares the average of the response spectra for the three selected ground motions and the target response spectrum. Unscaled acceleration time history plots for the selected ground motion records are included in Appendix D.

7.3.5 CSZ Intraslab Ground Motions

Response spectra from the 2001 El Salvador, the 1997 Michoacan, and a synthetic motion from our files were compared to the target response spectrum (shown in Figure 13). The response spectra were scaled and stretched to match spectral accelerations for the period range of interest for the embankment (0 to 0.5 seconds). Table 7-6 shows parameters of the selected earthquake ground motions.

Table 7-6. Selected Ground Motions for Intraslab Subduction Zone Earthquake Event

Earthquake	Station	Site Geology	Magnitude	Closest Distance to Rupture (km)	Recorded PGA (g)	Scaling Factor	Stretch Factor
El Salvador 1/13/2001	Observatorio	Volcanics	7.6	91	0.40	1.3	1.1
Michoacan 1/11/1997	Caleta de Campos	Rock	7.1	37	0.38	1.4	1.1
CSZ Intraslab Synthetic	-	Rock	7.5	65	0.27	2.0	1.1

Figure 18 shows the response spectra for the three scaled and stretched ground motions. Also shown in Figure 18 is the target response spectrum developed from the three ground motion attenuation models. Figure 19 compares the average of the response spectra for the three selected ground motions and the target response spectrum. Unscaled acceleration time history plots for the selected ground motion records are included in Appendix D.

8. STABILITY ANALYSES

Bear Creek Dam is a tall, relatively narrow gravity structure. Subsurface explorations indicate the left abutment is keyed into basalt, whereas the right abutment, based on boring B-3, may be founded on a mixture of sandstone and/or basalt. Previous stability evaluations by others indicated low factors of safety, but concluded that the influence of the abutments on the stability of the structure is significant. The previous stability evaluations only considered the maximum dam section and neglected the stabilizing influence of the abutments. Analyses from the previous evaluations indicate the structure has a factor of safety less than 1.0 under static loading conditions. This section discusses our approach to include the stabilizing influence of the abutments in the stability analyses.

8.1 Modeling Approach

The stability modeling approach used for the current evaluation is based on the assumption that the concrete, gravity structure behaves as a monolith. To confirm this assumption, a dynamic stress evaluation was performed to confirm that tensile stress demands do not exceed the capacity of the monolith. Once this was confirmed, the sliding stability was evaluated at several cross sections to determine available resistance to static and seismic loading. The resistance at each section was applied to a tributary length of dam, and load was shared among adjacent analysis sections to arrive at a composite factor of safety.

8.2 Structural Analysis

The tallest section of Bear Creek Dam was evaluated for tensile stress demands associated with a 2,475-year earthquake event. The response spectrum used for the evaluation was obtained from the 2008 NSHM Maps (with the 2009/2010 update) and is shown in Figure 20. Tensile stress demands are highest on the upstream face of the dam due to motions acting in the upstream direction. The dynamic stress analysis was performed in accordance with Fenves and Chopra (1985), “Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams”. Tensile stress demands from a cantilever analysis are considered an upper bound during an earthquake since the analysis ignores the confinement provided by horizontal (cross-canyon) spanning of the dam between the rock abutments.

Tensile strength of concrete is limited by the bond of the cement paste to the aggregate. Relationships between concrete compressive strength and direct tensile strength have been developed by Cannon (1995) for both conventional and roller-compacted concrete. These direct tensile strength estimates consider the weakness that occurs at lift joint surfaces and the higher strength due to rapid strain rates such as those associated with earthquakes.

Concrete cores from Bear Creek Dam indicate the concrete is well compacted and lift joint surfaces are tight. Aggregate in the dam raise section appears to be less than 1½-inch maximum dimension and consists of rounded material with some fractured surfaces that improve bond between the aggregate and cement paste. The uniaxial compressive strength for three core samples in the dam raise section were 8,790 psi, 9,070 psi, and 7,622 psi providing an average uniaxial compressive strength of 8,500 psi. Aggregate in the original dam section appears to be less than 3-inches maximum dimension and is predominantly angular (crushed) rock. The uniaxial compressive strength of the four core samples from the original dam were 6,020 psi, 4,910 psi, 7,852 and 4,461 psi providing an average uniaxial compressive strength of 5,800 psi.

The minimum design tensile strength for conventionally placed mass concrete according to Cannon is equal to $3.0(f'_c)^{½}$ for maximum size aggregate greater than 1½ inches, and equal to $3.4(f'_c)^{½}$ for maximum size aggregate equal to or less than 1½ inches. Where a linear elastic dynamic analysis is performed to obtain tensile stress demands, the tensile capacity values can be increased by 2.0 to account for the non-linear nature of the stress-strain relationship and high strain rate conditions present during a major earthquake event. Therefore tensile capacity for this evaluation is assumed to be equal to $6.0(f'_c)^{½}$, providing a direct tensile stress capacity of 550 psi for the concrete in the dam raise section and 450 psi for the concrete in the original dam section.

The maximum tensile stress demands are summarized in Figure 21. In all cases, the tensile stress capacity exceeds the tensile stress demand for the maximum dam section. The capacity-to-demand ratio is expected to be even higher at sections of the dam away from the maximum section. Based on the results of the analysis, the dam can be assumed to act as a monolith between the two prominent, vertical cracks. The stability of the portion of the dam outside the prominent, vertical cracks was analyzed separately assuming no shear transfer across the cracks.

8.3 Stability Analysis Approach

Limit-equilibrium stability analyses were performed to assess the factor of safety against a sliding. Previous attempts to evaluate the stability of the monolith including the stabilizing influence of the abutments were not successful. For this reason, the current evaluation included two-dimensional analyses performed at several representative sections between the center of the valley and the abutments. The results of the two-dimensional analyses were combined to calculate the stability of the monolith.

8.3.1 Stability Model

The objectives of this study included calculating the sliding stability of the dam including the effects of an irregular base geometry and resistance provided by the abutments and material at the toe of the structure. A multi-wedge stability analysis was deemed most appropriate to account for the abrupt geometry of the structure and anticipated shear surfaces.

Seven cross-sections were selected to evaluate the stability of Bear Creek Dam. The dam geometry, foundation materials, and loads acting on the dam at each section were considered representative for a given tributary length of the dam. Figure 22 shows the locations of these sections and their tributary lengths. Five of these sections are located between the prominent vertical cracks, and one additional section is located at each abutment outside of the prominent vertical cracks. Each analysis section is shown in cross-section in Figures 23 through 29 and is described further below. Station references are given with respect to the stationing used during original construction of the dam. Geometry and foundation conditions at the cross sections were obtained from the as-built notebook prepared by the City's resident engineer during construction in 1911-1912.

Section L3 at Station 0+30 represents the far left abutment, outside of the prominent vertical cracks. Sections L2 at Station 0+50 and L1 at Station 0+65 represent the left abutment within the prominent vertical cracks. The as-built sections and more recent explorations by Cornforth Consultants and Harza indicate that, in general, the left abutment is keyed into competent basalt. However, one boring (CC-3) encountered stiff, fine-grained marine sediments a few feet below the cutoff trench at Section L1, which controls the stability at this section.

Section C1 at Station 0+93 represents the center region of the dam. The tributary length of the center region approximately corresponds to the full-height section of the dam. As-built drawings indicate that the center portion of the dam is founded on sandstone.

Sections R1 at Station 1+40 and R2 at 1+70 represent the right abutment within the prominent vertical cracks. As-built drawings indicate that the region of the right abutment represented by Section R1 is poorly keyed into rock. In addition, although recent geologic reconnaissance identified competent basalt immediately downstream of the dam, limited subsurface information provided by one exploratory boring (Harza B-3) indicates that the foundation materials in the right abutment consist of a mixture of very highly fractured basalt and sandstone.

A review of the as-built cross-section book indicates that sandstone and basalt were both identified during foundation excavation as “R” (designation for rock). Unfortunately, the records do not differentiate between sandstone and basalt. Therefore, the only information regarding foundation conditions underlying the right abutment is from Harza Boring B-3. Geologically, the basalt abutments located downstream of the left and right abutments was emplaced either intrusively or invasively and is mapped as having limited lateral extent. Accordingly, it is possible that the dam was constructed at the margin of the basalt body where it contacts marine sandstone. For the current analyses, the stability of sections R1 and R2 were conservatively modeled with a sandstone foundation (weaker than basalt).

8.3.2 Material Properties

Section R3 at Station 2+03 represents the far right abutment, outside the prominent vertical cracks. Records from a grouting program carried out after the dam was raised, as well as settlement of the raised portion of the dam in this region, suggest that the far right abutment is founded on overburden soils.

Material properties for the stability analyses were selected based on the results of laboratory tests, descriptions of materials encountered in explorations, and design manuals for the analysis of concrete gravity dams. The properties used for the analyses are summarized in the Table 8-1. Discussions related to the material properties are included in the subsequent paragraphs.

Table 8-1 – Material Properties for Stability Analyses

Material	Unit Weight (lb/ft^3)	Friction Angle (degree)	Cohesion	
			(psi)	(psf)
Overburden/Fill	125	30	0.0	0.0
Marine Sediment	125	26	1.0	150
Sandstone	140	35	1.0	150
Basalt – Left Abutment	160	52	8.4	1,210
Basalt – Right Abutment	160	54	9.5	1,370
Concrete	154	57	140	20,160

Overburden/Fill. Following construction of the dam, the areas between the structure and the temporary excavation slopes were backfilled. There is little information on the material used as backfill or on the methods used to place the backfill. Materials excavated during dam construction included basalt, sandstone, marine sediments, and overburden soil. Material properties for Overburden/Fill were selected as a lower bound of what could be expected for a mixture of these materials.

Marine Sediment. Unconsolidated marine sediment was encountered in boring CC-3. The sediment consists of stiff, slightly clayey to clayey silt. This material is interpreted to be weathered Astoria Formation that the invasive basalt displaced during emplacement. The strengths were selected to represent a mean value that could be expected for the unit.

Sandstone. Siltstone and sandstone deposits were encountered in borings CC-3, B-3, B-101 and B-102. The as-built cross sections prepared by the City's resident engineer also describe an area in the center of the valley that encountered sandstone. An outcrop of the sandstone was observed in the valley wall downstream of the dam. The material parameters used for this layer were selected based on the exposure located during field reconnaissance.

Basalt. During geologic mapping efforts, the condition of the rock masses on the left and right abutments were measured to establish engineering properties. The condition and spacing of the joints were used to establish the Geologic Strength Index (GSI), which was used to estimate the Mohr-Coulomb strength envelope for the rock mass. The generalized Hoek-Diederichs (2006) equation was used to estimate the shear strength of the rock masses.

Concrete. Explorations completed from the dam crest encountered three concrete mixes that generally correspond to the locations of the main cutoff trench, the original dam section, and the raised dam section. In general, the concrete recovered showed good quality concrete with good bonding between lift joints. Unconfined compression tests were completed on samples from each zone. The results of the tests were used in combination with a research report prepared by EPRI (1992) to characterize the strength of the concrete for stability analyses.

8.3.3 Uplift Pressure

Piezometers at Bear Creek Dam show that the cutoff trenches excavated into the foundation of the dam are effective at reducing uplift pressures. For analysis purposes, the distribution of hydrostatic uplift pressures was modeled with a bilinear envelope as illustrated in Figure 30. The analysis pool level was taken to be at elevation 662.5 feet. We understand that this level is a reasonable upper limit to the pool elevations reached during the summer months. The envelope used to calculate uplift pressures extends from the pool level to the water level measured by crest piezometers. The piezometer levels were recorded with a pool level of 660.7 feet and were subsequently adjusted for the design pool level as shown in Figure 30. The envelope was extended from this point to the tailwater elevation. For the center region and right abutment, the tailwater elevation was determined from piezometers. For the left abutment, the tailwater level was assumed to be 8 and 10 feet below the ground surface at Sections R1 and R2, respectively. At section R3, the water level recorded at the centerline of the dam crest is low enough that the assumed tailwater level does not influence the uplift pressure on the dam.

8.3.4 Seismic Loads

The effect of seismic loading on the sliding stability was evaluated by applying a seismic coefficient to the limit-equilibrium analyses. The inertial force is calculated by multiplying the seismic coefficient by the mass of each wedge in the limit-equilibrium analysis. A pseudostatic analysis of a concrete, gravity structure commonly applies a coefficient equal to the peak ground acceleration (PGA) divided by the acceleration due to gravity. For Bear Creek Dam, the PGA for an event with a 2,475-year return period is 0.61g.

The dam must also resist inertial forces due to the sloshing of impounded water in the reservoir, termed “hydrodynamic loads.” Westergaard (1931) showed that a reasonable approximation of this load may be calculated using the following equation:

$$P_w = \frac{7}{12} k_h \gamma_w H^2$$

where P_w is the hydrodynamic force, k_h is the horizontal seismic coefficient, γ_w is the unit weight of water, and H is the depth of the reservoir. The hydrodynamic force was added to the hydrostatic force to determine the total water force acting on the dam.

8.3.5 Potential Shear Surfaces

At each analysis section, several potential shear surfaces were evaluated for sliding failure. In general, three types of potential failure surfaces were evaluated: i) shallow surfaces passing along the base of the main dam through the cutoff trench; ii) mid-depth surfaces passing beneath the base of the cutoff trench; and iii) at some locations along the left abutment, deep shear surfaces passing through basalt into marine sediments. Multiple entry and exit angles were evaluated for each potential shear surface. Figures 31 through 37 show the potential shear surfaces considered for each analysis section. Critical failure surfaces for static and seismic cases are also identified using heavier line weights.

Where the as-built cross sections show that the toe of the dam does not abut rock, a failure surface was considered which exits along the contact between rock and the overburden/fill. This steep exit angle, in combination with the assumption of horizontal interslice forces, can cause numerical instability during the analyses. To resolve this issue, the lowest wedge of soil was modeled as a zero-shear-strength material that resists sliding through a combination of its own weight and the force normal to the shear surface.

8.3.6 Composite Factor of Safety Approach

The purpose of performing stability analyses at multiple sections is to arrive at a composite factor of safety for the entire dam accounting for the stabilizing effect of the abutments and material at the toe of the dam. This approach requires load to be transferred between adjacent analysis sections to distribute excess load to regions with excess capacity. Load transfer is assumed to occur only in the portion of the dam between the prominent vertical cracks.

The composite factor of safety approach is summarized as follows.

1. Estimate initial trial factor of safety of the central section (Section C1).
2. Calculate the horizontal force (load) that must be transferred from the center region (where capacity is deficient) to the adjacent regions of the dam in order to reach the trial value of factor of safety.
3. Estimate a trial load distribution to the left and right of Section C1 (Sections L1 and R1, respectively). For example, an initial estimate may be that the additional load is divided equally between Sections L1 and R1.
4. Calculate the horizontal force that must be transferred from Section L1 to L2, and from Section R1 to R2, to reach the trial factor of safety.
5. Calculate the factors of safety for Sections L2 and R2.
6. Revise estimate of load distribution between the left and right abutments and repeat steps 4 and 5 until the factor of safety of Section L2 matches that of Section R2.
7. If the resulting factor of safety of Sections L2 and R2 does not match the initial trial value, revise the trial factor of safety and repeat steps 2 through 6 until the values converge.

8.4 Results of Stability Analyses

Limit-equilibrium stability analyses were performed for the selected analysis sections under static and seismic loading. The following sections describe the selection of the critical shear surfaces, static and seismic factors of safety for individual analysis sections, and development of the composite factor of safety and the composite yield coefficient.

8.4.1 Definition of Critical Shear Surface

For the static case, the critical shear surfaces were identified as the surfaces resulting in the lowest composite factor of safety. For Sections R1, C1, and L1, the critical shear surface was defined as the surface that requires the most load to be transferred to adjacent sections. For Sections R2 and L2, which have no adjacent sections to transfer load to, the critical shear surface was defined as the surface with the lowest factor of safety when providing support to the adjacent interior section. For Sections R3 and L3, which cannot transfer or receive load, the critical shear surface is the surface with the lowest factor of safety.

For the seismic case, the critical shear surface was identified as the surface resulting in the lowest composite yield coefficient. For Sections R3 and L3, the critical shear surface is the surface with the lowest yield coefficient.

In many cases, the critical shear surface is the same as the critical surface that would be developed if load were not shared between adjacent sections. However, the surfaces differ in a few cases. For example, if Section R2 is evaluated independently from the rest of the dam, the critical shear surface exits the ground at an angle of approximately 29 degrees above horizontal. Once the section is assumed to support Section R1, the normal force on an inclined exit surface increases and a horizontal shear surface controls the stability of the section.

Two critical shear surfaces for each applicable surface are shown in Figures 31 through 37. A solid line corresponds to the most critical shear surface for the static analysis case, and a dashed line corresponds to the most critical shear surface when calculating the yield coefficient for the seismic analysis case.

8.4.2 Static Stability

As stated earlier, the objective of performing stability analyses on multiple cross sections is to determine the composite factor of safety of the dam. However, the factor of safety of individual sections is useful in identifying the regions of the dam that will require additional support, as well as the relative ability of other regions of the dam to provide additional support. The static factor of safety for the critical shear surface at each analysis section is shown in Table 8-2. These values are also reported in Figures 31 through 37. The additional capacity for horizontal load, which is the additional horizontal force that would bring the surface to a factor of safety of 1.0, is also reported when the factor of safety is above 1.0.

Table 8-2 – Summary of Static Stability Analyses

Analysis Section	Static FS	Shear Force (kip/ft)	Shear Strength (kip/ft)	Additional Capacity for Horizontal Load (kip/ft)
L3	15.52	35.4	549.5	1,574.9
L2	3.68	78.2	287.9	430.4
L1	2.53	239.4	605.1	842.5
C1	0.84	292.4	246.8	-
R1	0.70	163.4	114.2	-
R2	3.56	53.6	190.8	173.5
R3	2.51	12.4	31.1	24.2

8.4.3 Seismic Stability

The pseudostatic factor of safety for a PGA of 0.61g, which corresponds to an event with a 2,475-year return period, is shown in Table 8-3. For sections with a static factor of safety above 1.0, the yield coefficient of the section is also reported. These values are also reported in Figures 31 through 37.

Table 8-3 – Summary of Seismic Stability Analyses

Analysis Section	Pseudostatic FS $k_h = 0.61$	Shear Force (kip/ft)	Shear Strength (kip/ft)	Yield Coefficient
L3	3.43	167.4	574.9	3.40
L2	1.53	321.3	491.2	1.31
L1	1.26	602.2	756.4	1.06
C1	0.50	630.4	318.1	-
R1	0.30	358.8	107.1	-
R2	1.23	192.3	236.9	0.83
R3	0.80	43.0	34.4	0.41

8.4.4 Composite Factor of Safety

The analysis results shown in Tables 8-2 and 8-3 show that the left abutment is significantly stronger than the right abutment. This result is not surprising considering the right abutment was modeled as sandstone not basalt. Therefore, the right abutment possesses less capacity for additional horizontal load, which is reflected in the lower yield coefficient of the right abutment sections. Analysis Section R1 requires support from adjacent sections to remain stable under static conditions.

Characterizing the entire dam with one composite factor of safety would require that excess capacity in the left abutment can be provided to the right abutment. In our opinion, it is not reasonable to rely on this type of cantilever behavior for an unreinforced concrete structure. Separate composite factors of safety were therefore developed for the left and right abutments. The center region was split at its midpoint, so that one half of the region was included in the composite factor of safety of the left abutment and one half in that of the right abutment. The approach previously described for determining the composite factor of safety was modified slightly, since Step 3 (estimating the distribution of load to the left and right abutments) was no longer necessary. Instead, the full amount of support required by the portion of the center region attached to the left abutment was transferred to the left abutment, and likewise for the right abutment. In other respects, the composite factor of safety was consistent with the approach previously described.

The composite factors of safety for the left and right abutments are shown for the static and seismic cases in Table 8-4. Factors of safety are also shown for the far abutment regions outside of the prominent vertical cracks. These factors of safety are identical to the factors of safety for the respective analysis sections in Tables 8-2 and 8-3.

Table 8-4 – Composite Factors of Safety

Region	Composite Factor of Safety	
	Static	Pseudostatic with k_h = 0.61
Far Left Abutment	15.52	3.43
Left Abutment + 50% Center Section	1.92	1.01
Right Abutment + 50% Center Section	1.18	0.52
Far Right Abutment	2.51	0.80

8.4.5 Composite Yield Coefficient

For regions with a composite pseudostatic FS less than 1.1, a composite yield coefficient was determined for use in a Newmark displacement analysis. As with the composite factor of safety, the portion of the dam between the prominent vertical cracks was split between the left and right abutments. Composite yield coefficients are summarized in Table 8-5.

Table 8-5 – Composite Yield Coefficients

Region	Composite Yield Coefficient
Far Left Abutment	3.40
Left Abutment + 50% Center Section	0.64
Right Abutment + 50% Center Section	0.09
Far Right Abutment	0.41

8.5 Newmark Analyses

Newmark displacement analyses were performed using the composite yield coefficients and the selected earthquake acceleration time histories. Newmark analyses were not performed for the far left abutment section since the peak ground accelerations never exceed the yield coefficient. The results of the Newmark displacement analyses are summarized in Table 8-6. For each event, the displacement reported is the maximum value of the displacements calculated in the two orthogonal directions when considering both positive and negative polarity of the recorded motions.

Table 8-6 – Newmark Displacement Summary for Bear Creek Dam

Earthquake (Date)	**Station**	**Scaled PGA[1] (g)**	**Displacement (inch)**			
			Far Left Abutment	**Left Abutment**	**Right Abutment**	**Far Right Abutment**
		Crustal Events				
San Fernando (02/09/1971)	Castaic – Old Ridge Route	0.43	0.0	0.0	7.1	0.0
Mammoth Lakes (05/25/1980)	Long Valley Dam (Upper Left Abutment)	0.52	0.0	0.0	8.6	0.1
Morgan Hill (04/24/1984)	Gilroy Array #6	0.37	0.0	0.0	10.8	0.0
Loma Prieta (10/17/1989)	San Jose – Santa Teresa Hills	0.38	0.0	0.0	6.4	0.0
Northridge (01/17/1994)	Toganga – Fire Station	0.38	0.0	0.0	6.1	0.0
	Average of Crustal Events =		0.0	0.0	7.8	0.0
		Intraslab Events				
El Salvador (01/13/2001)	Observatorio	0.52	0.0	0.0	28.5	0.2
Michoacan (01/11/1997)	Caleta de Campos	0.53	0.0	0.0	9.0	0.0
Synthetic	-	0.54	0.0	0.0	22.8	0.0
	Average of Intraslab Events =		0.0	0.0	20.1	0.1
		Interface Events				
Tohoku (03/11/2011)	Toyosato (MYG007)	0.62	0.0	0.0	31.4	0.2
Michoacan (09/19/1985)	Caleta de Campos	0.53	0.0	0.0	9.0	0.0
Synthetic	-	0.51	0.0	0.0	20.9	0.0
	Average of Interface Events =		0.0	0.0	20.4	0.1

[1] Geometric mean of the two horizontal components

9. CONCLUSIONS AND RECOMMENDATIONS

Static and seismic analyses have been completed at multiple sections to assess the stabilizing benefit provided by the dam abutments. Structural analyses completed at the maximum section indicate that tensile demand is well below tensile capacity, indicating that the dam will behave as monolith between the prominent vertical cracks. Load and resistance can be "shared" between analysis sections within the monolith.

A three-dimensional model of the dam and foundation was developed based on as-built cross sections surveyed during construction, data from existing borings, and data from surface geologic mapping. The geologic model of the dam and foundation indicates left abutment bears on basalt, and has a relatively deep cutoff/key trench. Limited existing data suggests that the right abutment is at least partially founded on weaker, sedimentary rocks. Competent basalt outcrops immediately downstream of the right abutment, which indicates the right abutment is very close to the contact between basalt and sedimentary rocks.

Stability analyses indicate the left abutment is much more stable than right abutment due to geometry of cutoff trench and presence of basalt. Modeling the right abutment with a sandstone foundation indicates that there is not enough resistance to ensure that the dam will behave as a monolith. Therefore, composite stability analyses were performed on the right and left halves of the dam to estimate displacements that could occur during the design earthquake.

When analyzed individually, the FS of the center section of the dam under static conditions is calculated to be 0.8, which is consistent with the results of previous evaluations by others. This suggests that the center section is supported by the abutments since the dam is currently stable. The composite FS of the left abutment plus 50% of the center section is well above 1.5. However, the composite FS of the right abutment plus 50% of the center section is approximately 1.2. Under static conditions, the portions of the dam outside the prominent vertical cracks have calculated FS well above 1.5.

Seismic loads were modeled using a pseudo-static coefficient equal to 0.61. The composite FS of the left abutment plus 50% of the center section under seismic loading is slightly above 1.0. The FS of the far left abutment of the dam (outside the prominent vertical crack) is well above 1.5. The composite FS of the right abutment plus 50% of the center section is 0.5 under seismic loading, and the FS of the far right abutment is 0.8. These results indicate that the portion of the dam right of centerline will displace during the design earthquake.

For portions of the structure with a calculated seismic FS less than 1.0, displacements were estimated using Newmark analysis techniques. The analyses indicate that the displacements would be small at the portions of the dam outside the prominent vertical cracks and at the left abutment plus 50% of the center section. However, displacements of the right abutment plus 50% of the center section are estimated to range between 20 and 30 inches. This level of displacement could cause significant structural distress, extensive cracking, and leakage of the dam. In addition, the magnitude of the displacement could cause strain-softening behavior of the dam/rock interface, which would tend to increase displacements.

Since the calculated FS is highly dependent on subsurface conditions, and there is uncertainty in the subsurface conditions beneath the right abutment, we recommend additional explorations be completed to better define the conditions in the right abutment. If material stronger than sandstone is encountered beneath the right abutment, the FS would be higher and estimated seismic displacements would be lower.

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Limitations in the Use and Interpretation of this Geotechnical Report

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The geotechnical report was prepared for the use of the Owner in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive boring and test pit logs, cross-sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory borings, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory borings and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

The Summary Boring Logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the borings progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil conditions at these boring locations.

Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report; nor can our firm be responsible for any construction activity on sites other than the specific site referred to in this report.

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VICINITY MAP

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

JUN 2015

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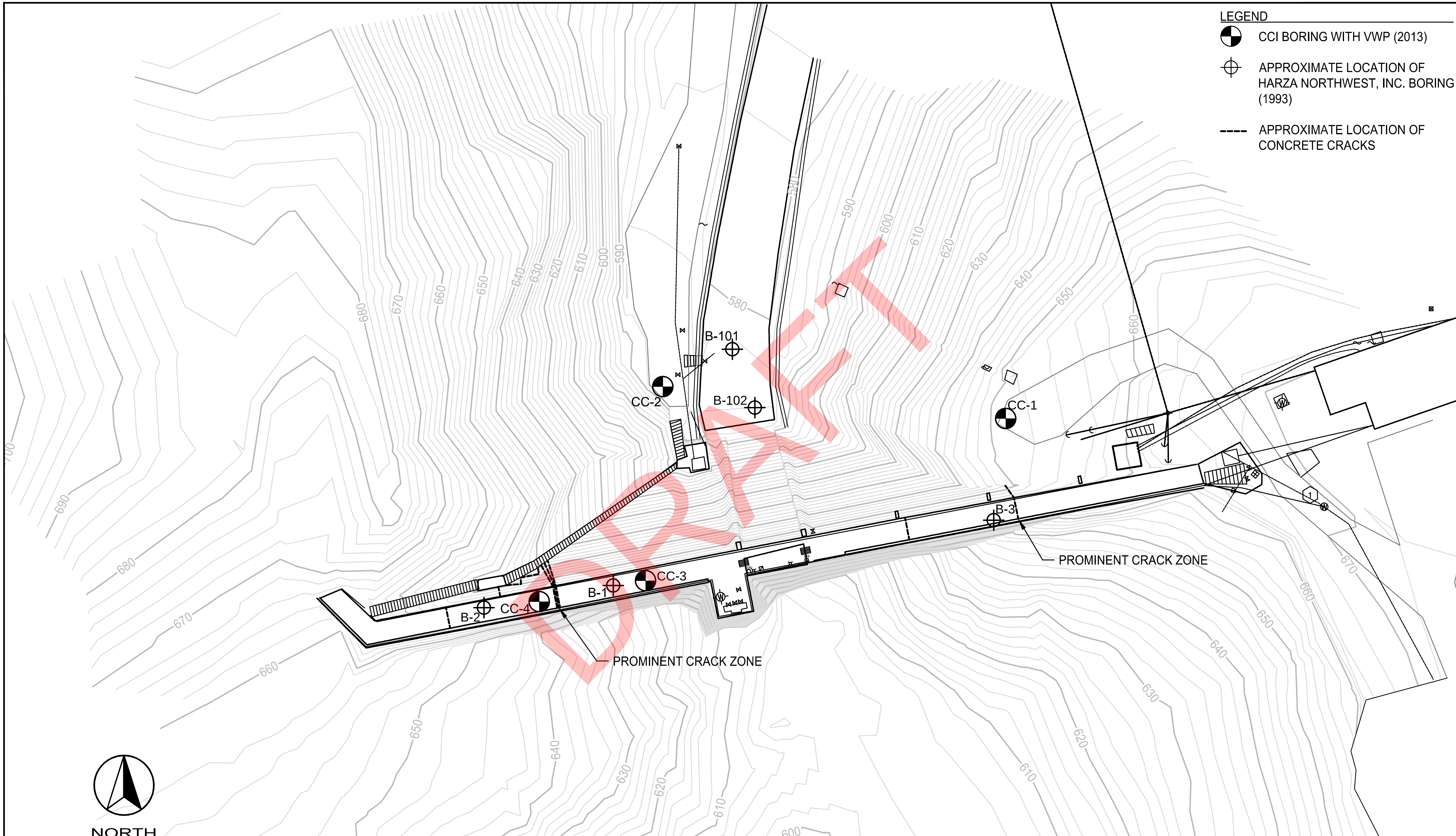
FIG. 1



0 30 60
SCALE IN FEET

NOTE: TOPO MAP PROVIDED BY OTAK, INC., DECEMBER 2013.

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SITE PLAN

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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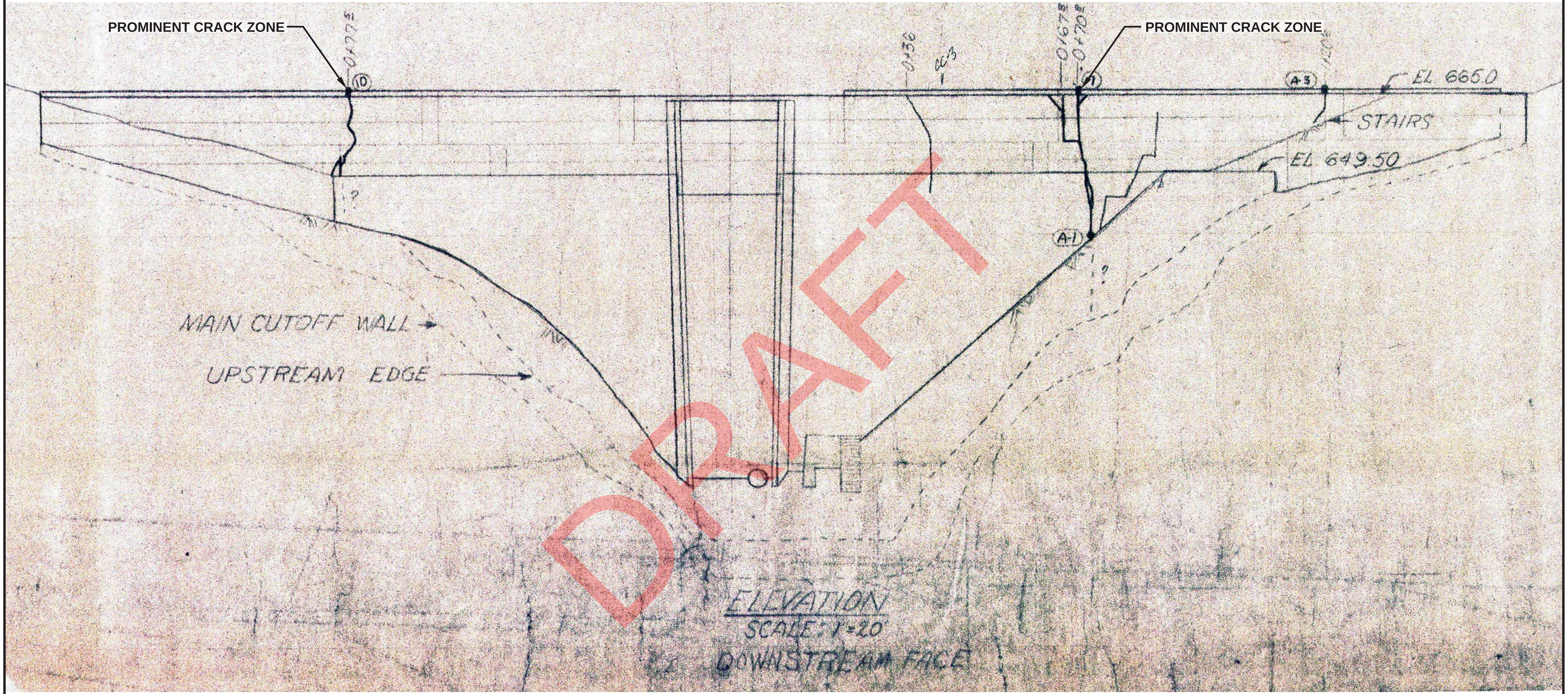
FIG. 2



NOTE: SECTION SCANNED FROM CITY OF ASTORIA AS-BUILT DRAWING DATED 12-7-67.

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DOWNSTREAM ELEVATION

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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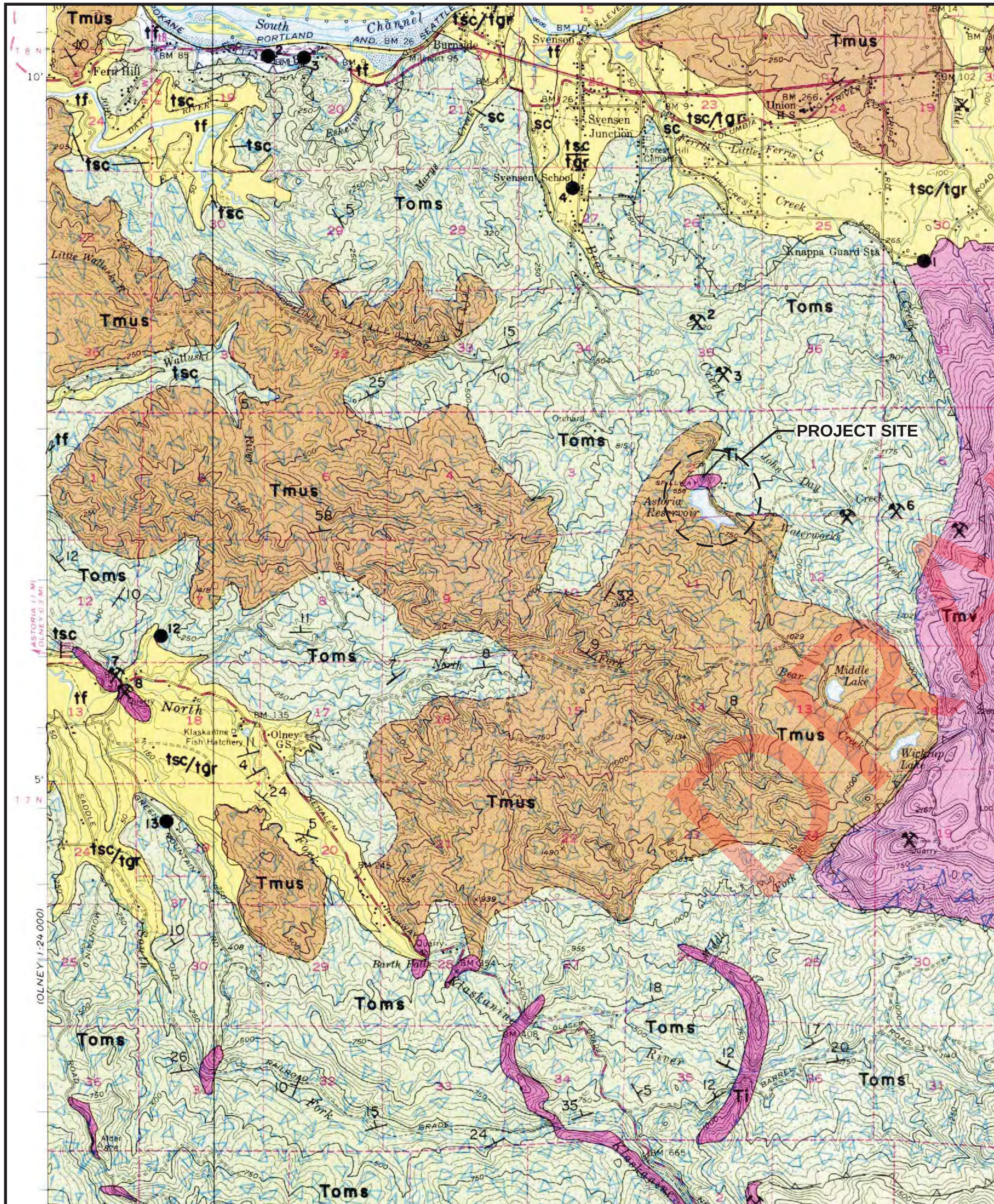
FIG. 3



EXPLANATION

UNCONSOLIDATED SURFICIAL UNITS

Terrace Alluvium

- tsc — terrace silty clay
- tgr — terrace gravel

STRATIGRAPHIC UNITS

- Tmus — Upper Miocene Sandstone
- Tmv — Miocene Volcanic Rocks
- Ti / Tic — Intrusive Rocks

UNCONFORMITY

- Toms — Oligocene to Miocene Sedimentary Rocks

GEOLOGIC SYMBOLS

Active Landslide

Inactive Landslide

Landslide Topography

Fault

Dashed where approximately located or indefinite, dotted where concealed. U, upthrown side; D, downthrown side.

Contact

Definite, Approximate

23

Strike and Dip of beds or flows.

Apparent dip of beds by aerial interpretation.

Rock Quarry

Gravel Pit

86

Water Well

NORTH

0 1 2

SCALE IN MILES

NOTE: GEOLOGIC MAP FROM "BULLETIN 74, ENVIRONMENTAL GEOLOGY OF THE COASTAL REGION OF THE TILLAMOOK AND CLATSOP COUNTIES, OREGON" 1972.

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GEOLOGIC MAP

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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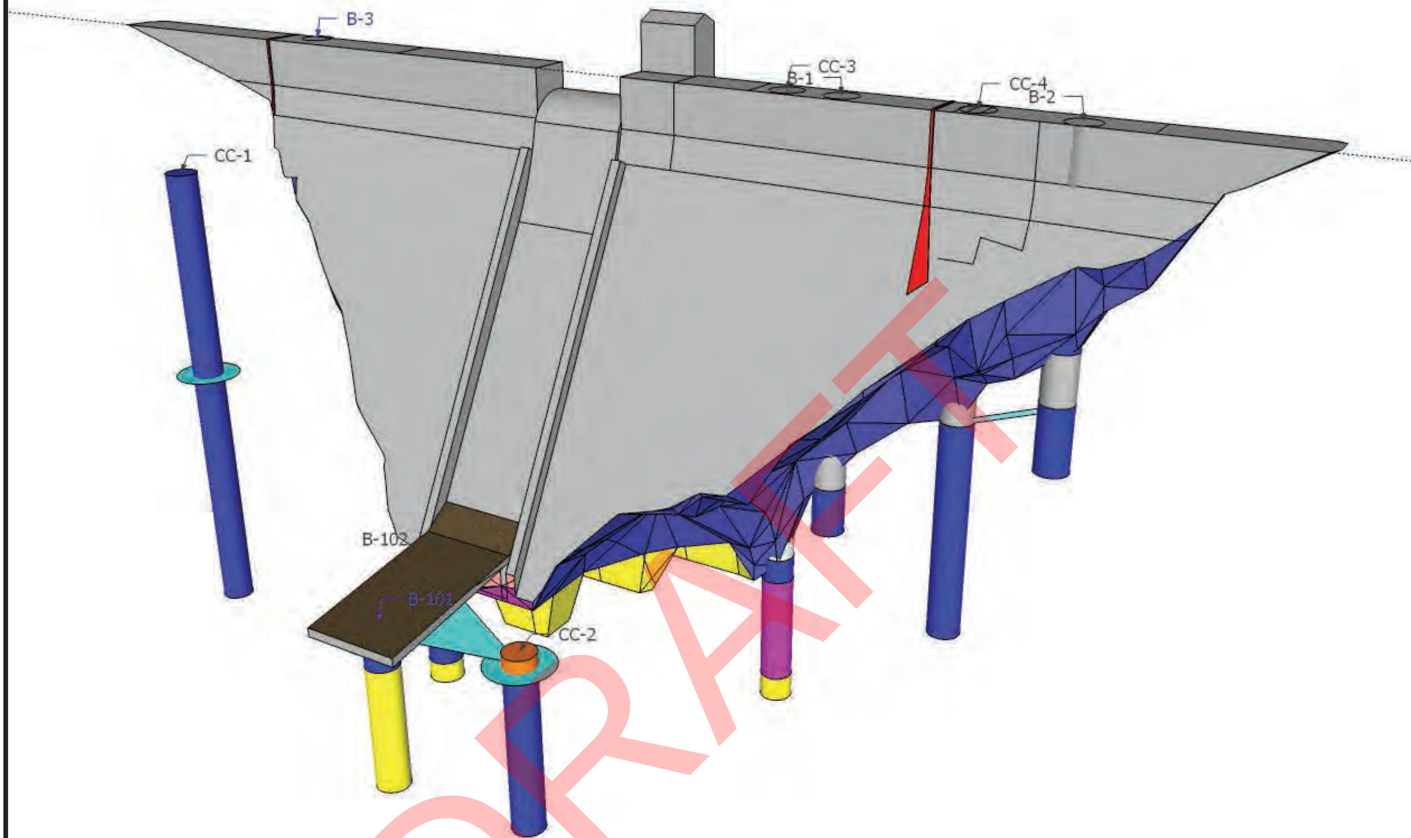
FIG. 4



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3D MODEL
VIEW LOOKING SE

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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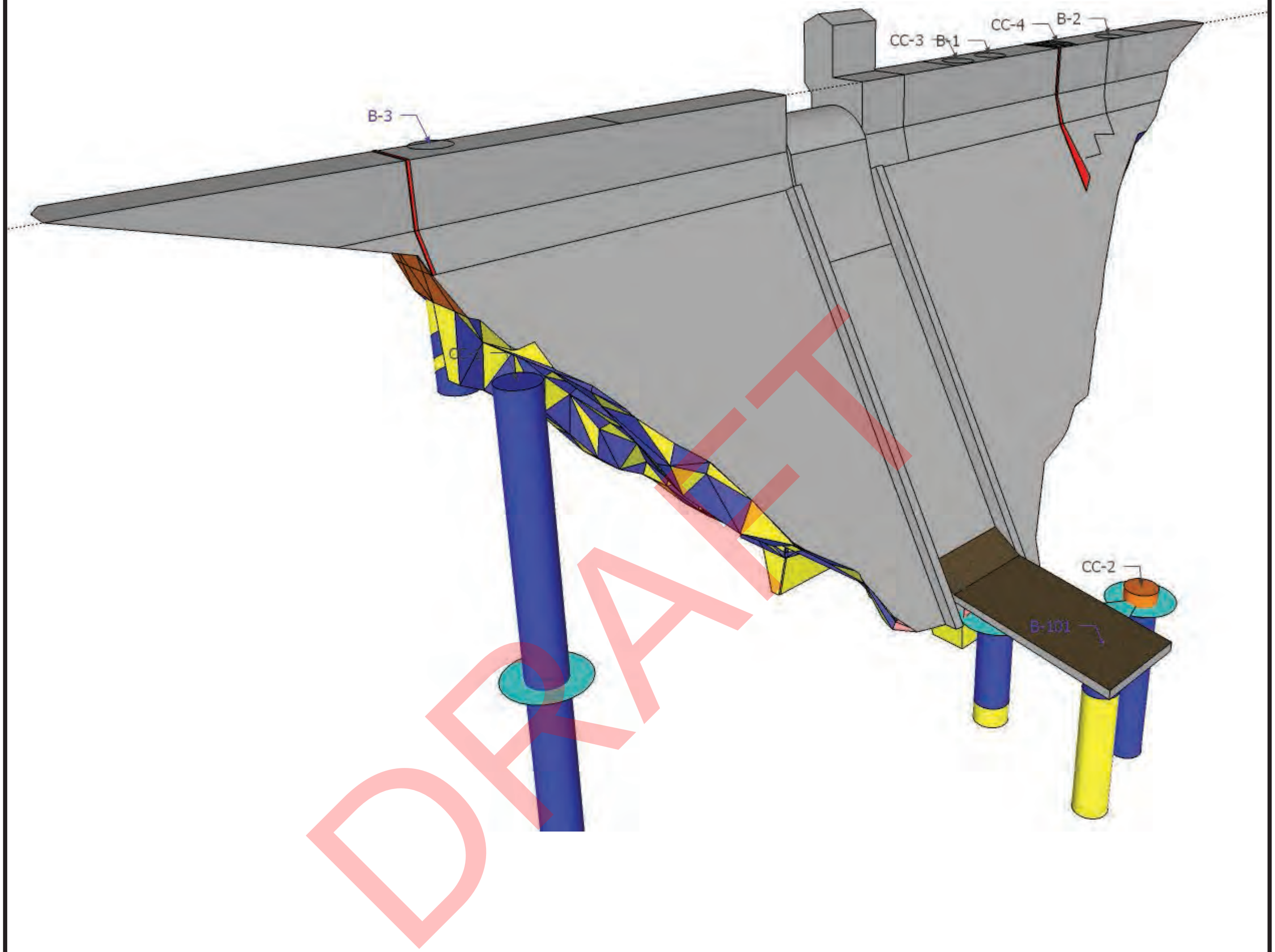
FIG. 5



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3D MODEL
VIEW LOOKING SW

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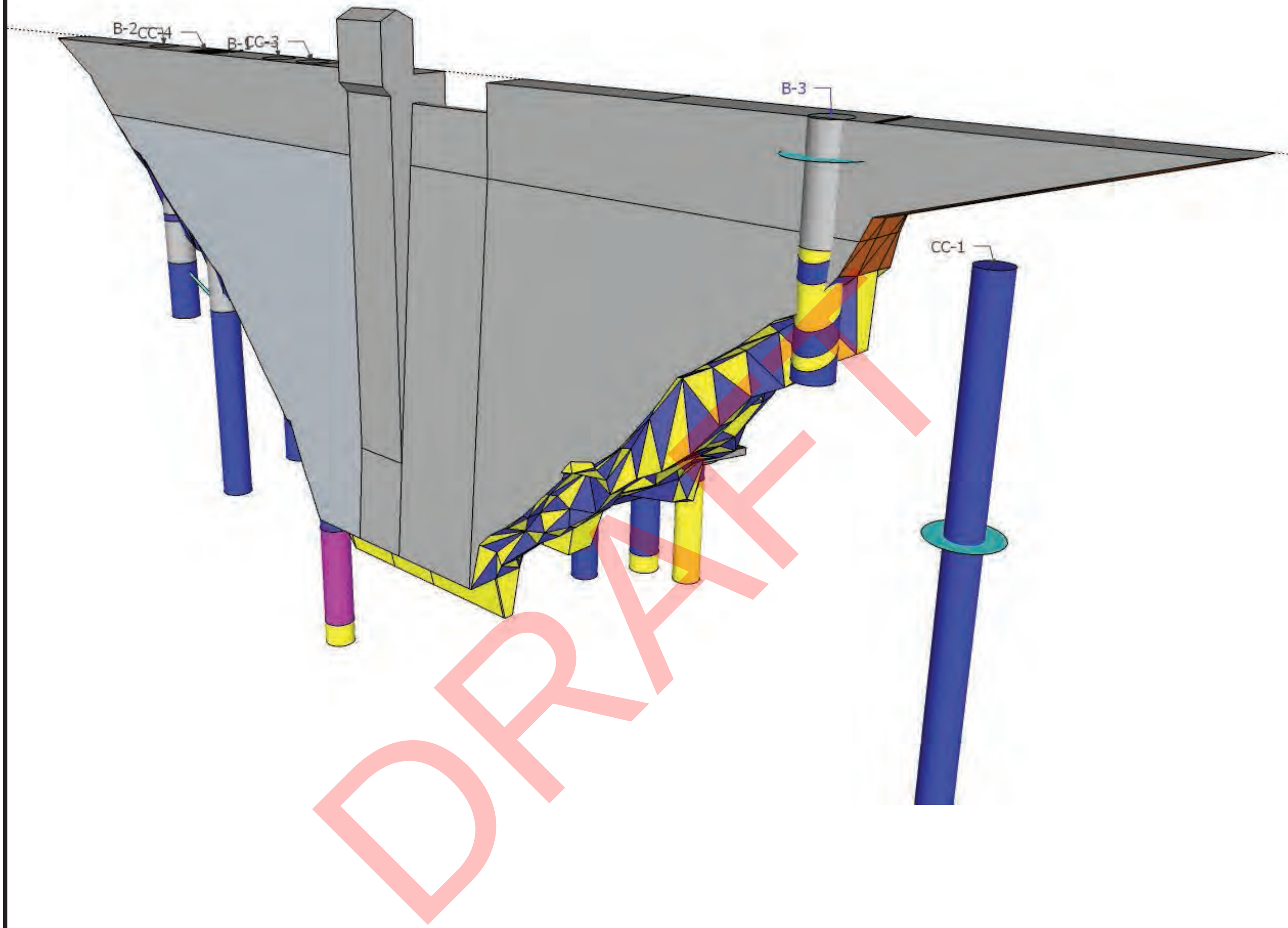
FIG. 6



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3D MODEL
VIEW LOOKING NW

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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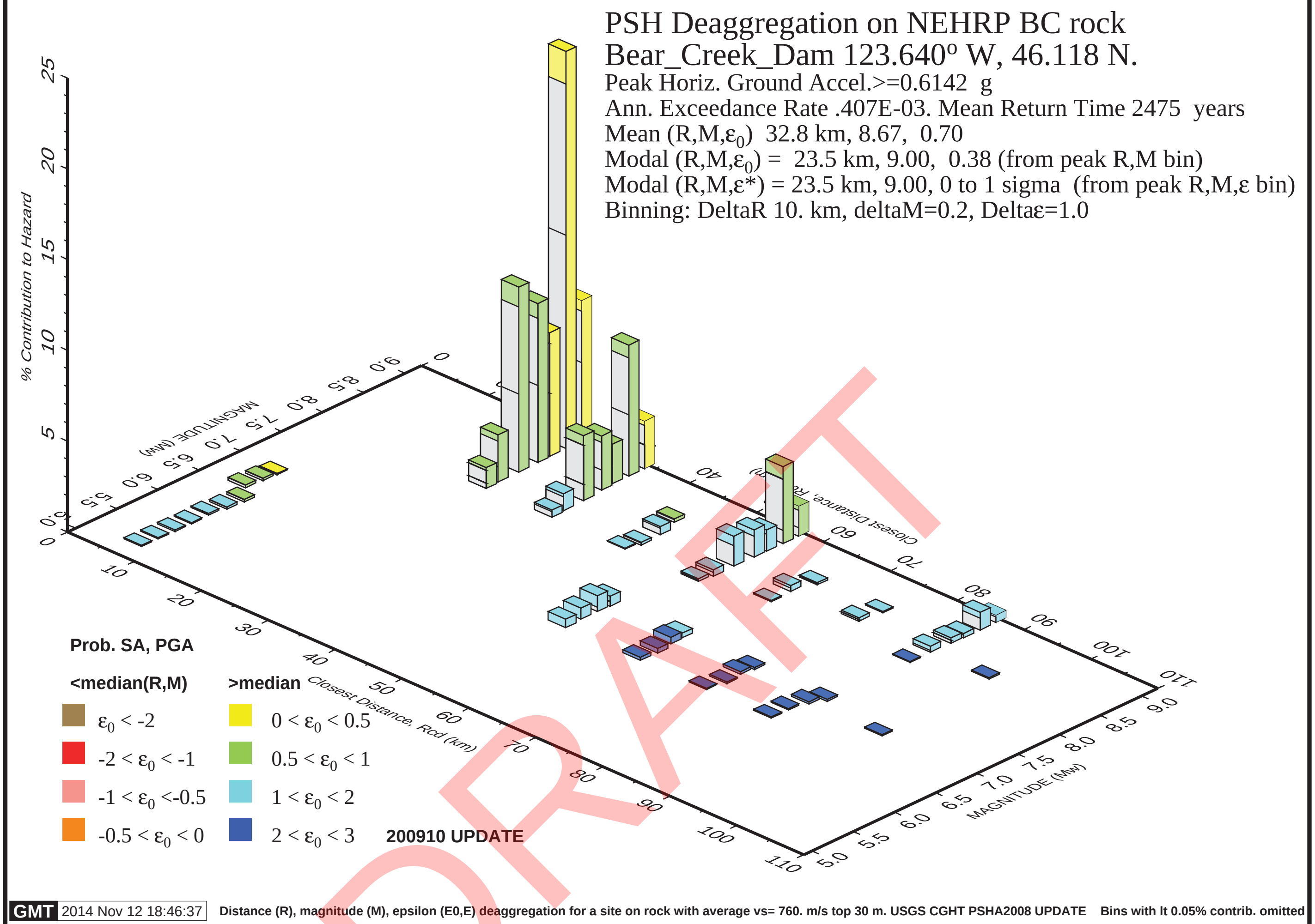
FIG. 7



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SEISMIC HAZARD DEAGGREGATION FOR PGA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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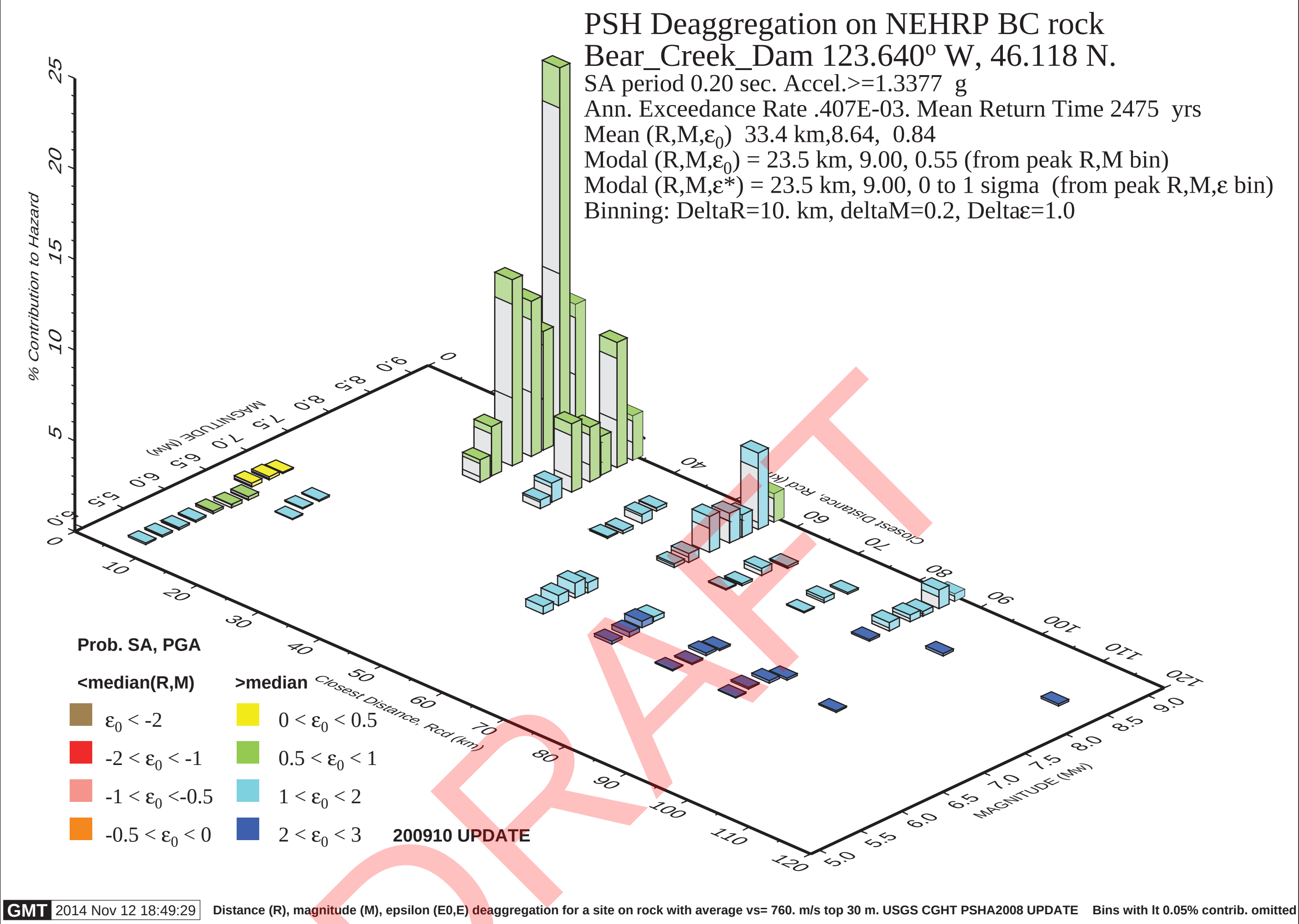
FIG. 8



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SEISMIC HAZARD DEAGGREGATION FOR 0.2 S

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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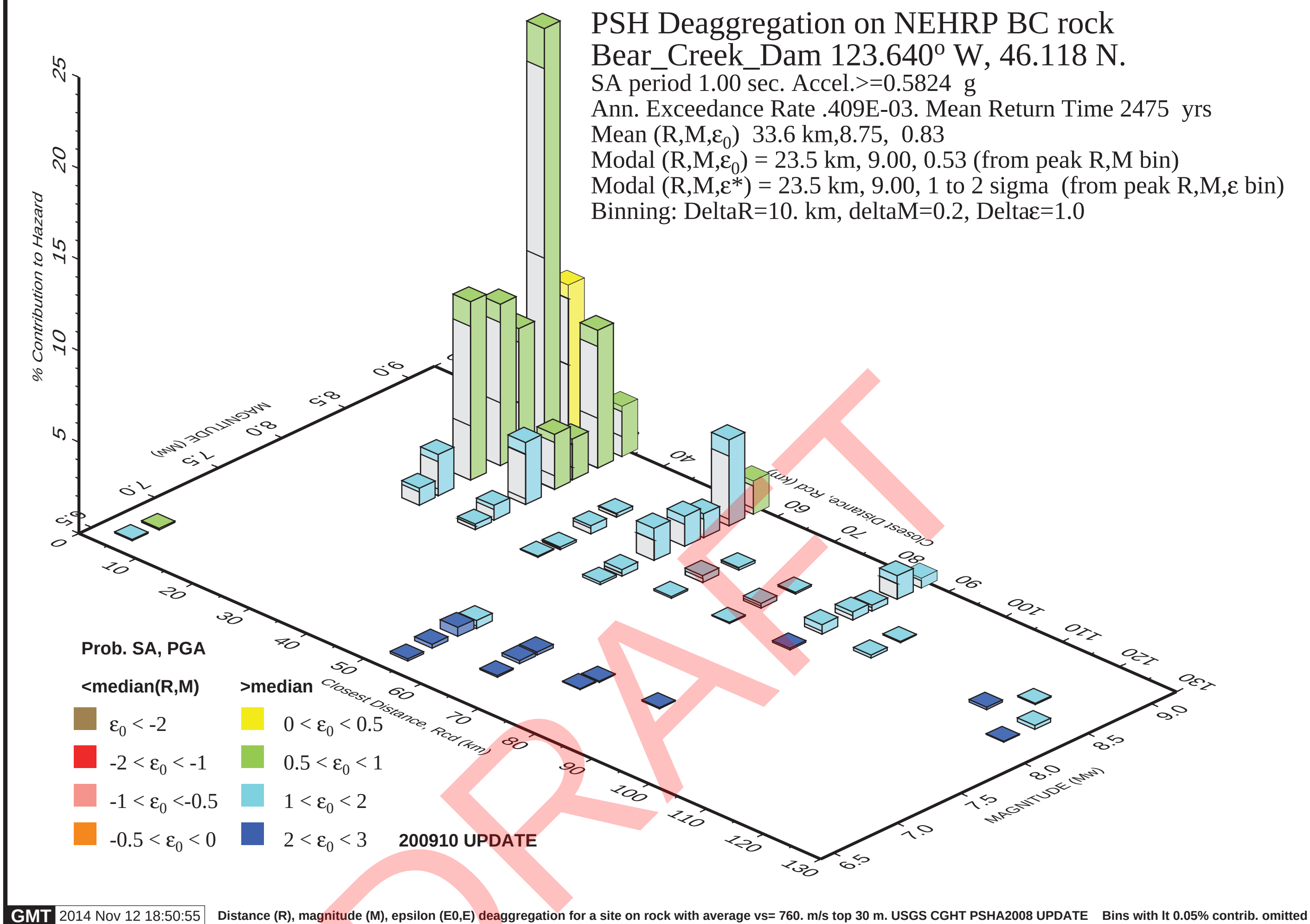
FIG. 9



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SEISMIC HAZARD DEAGGREGATION FOR 1.0 S

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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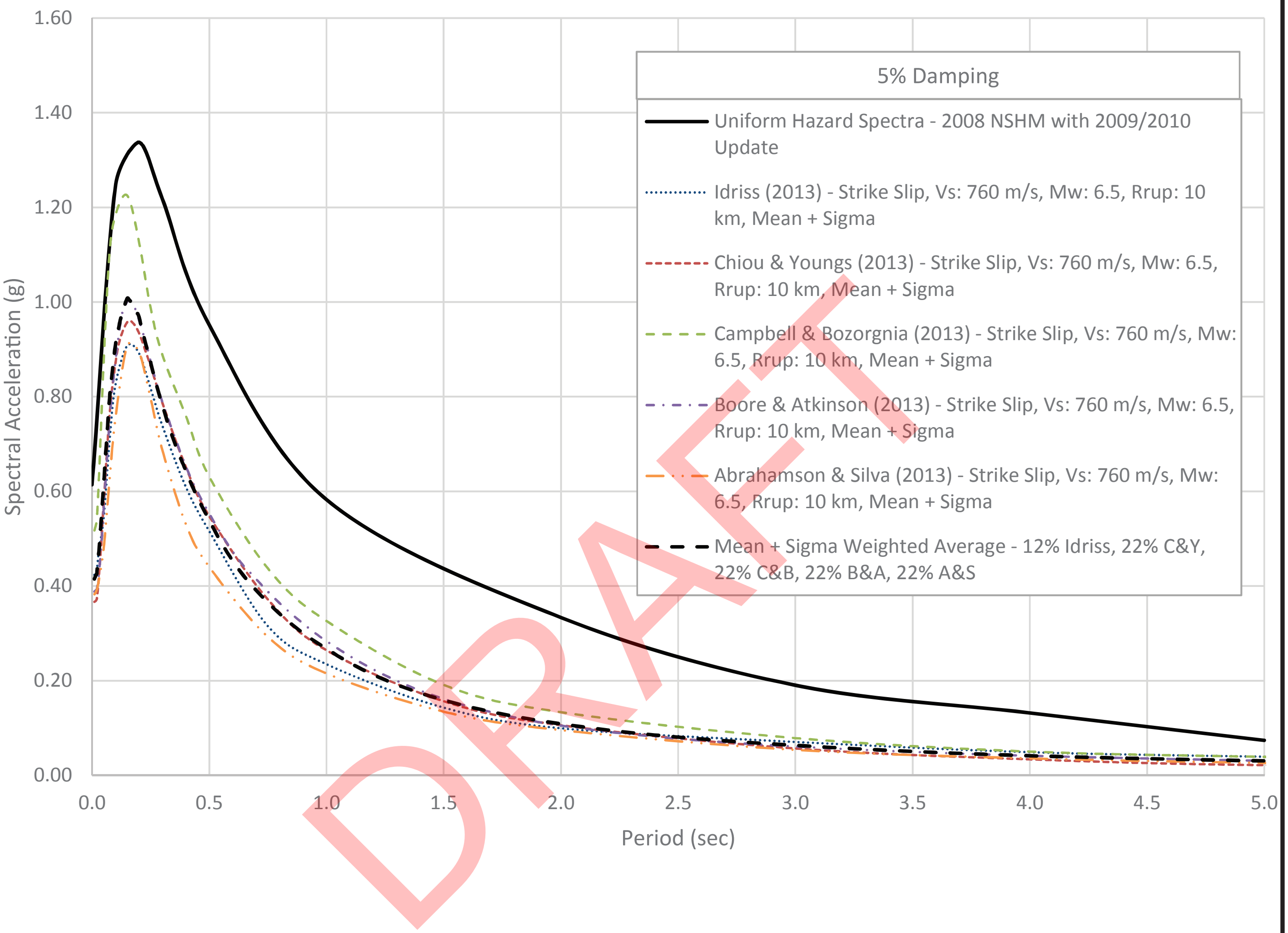
FIG. 10



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2392/11.ai NAU

CRUSTAL SOURCE TARGET RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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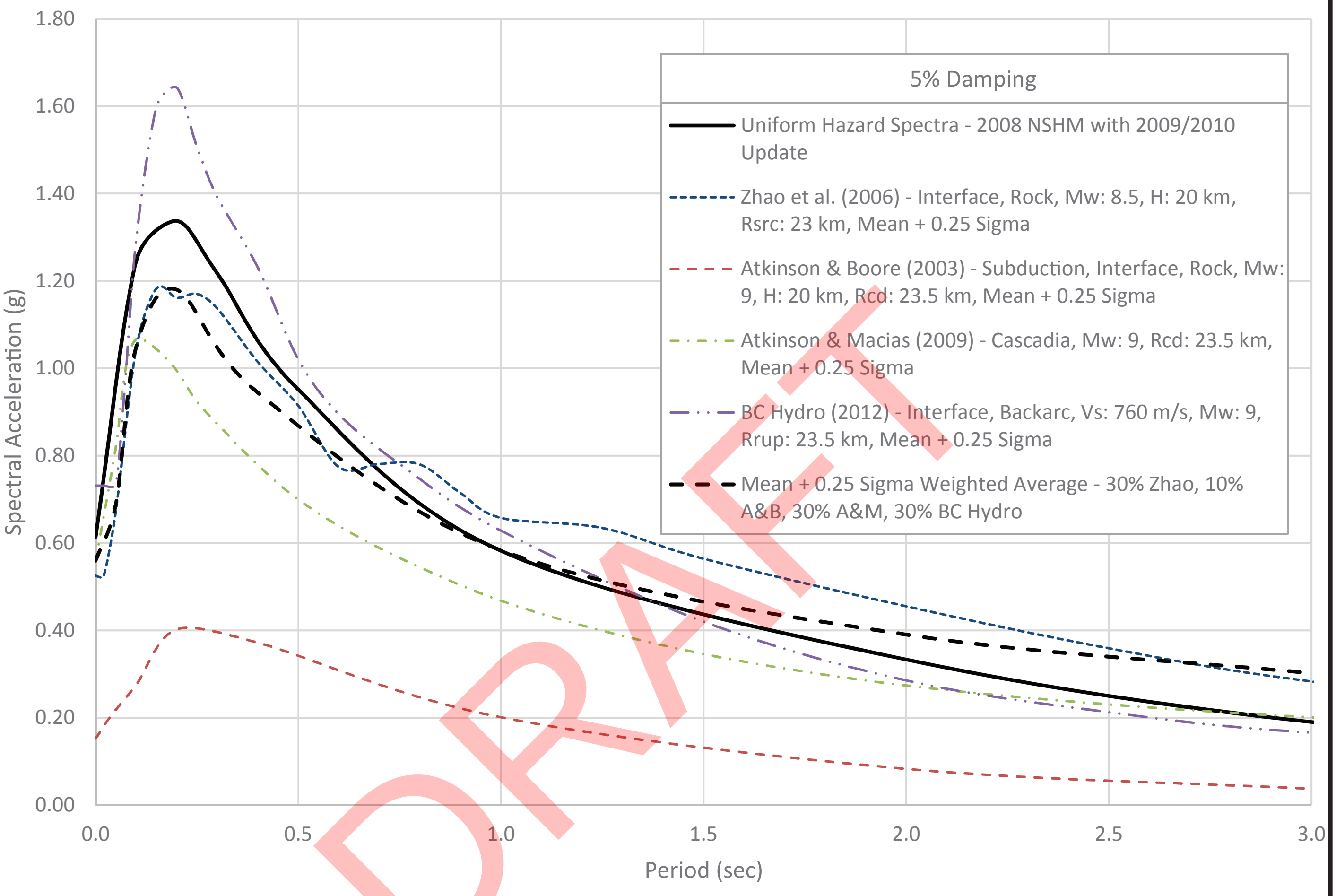
PROJ. 2392

FIG. 11



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INTERFACE SOURCE TARGET RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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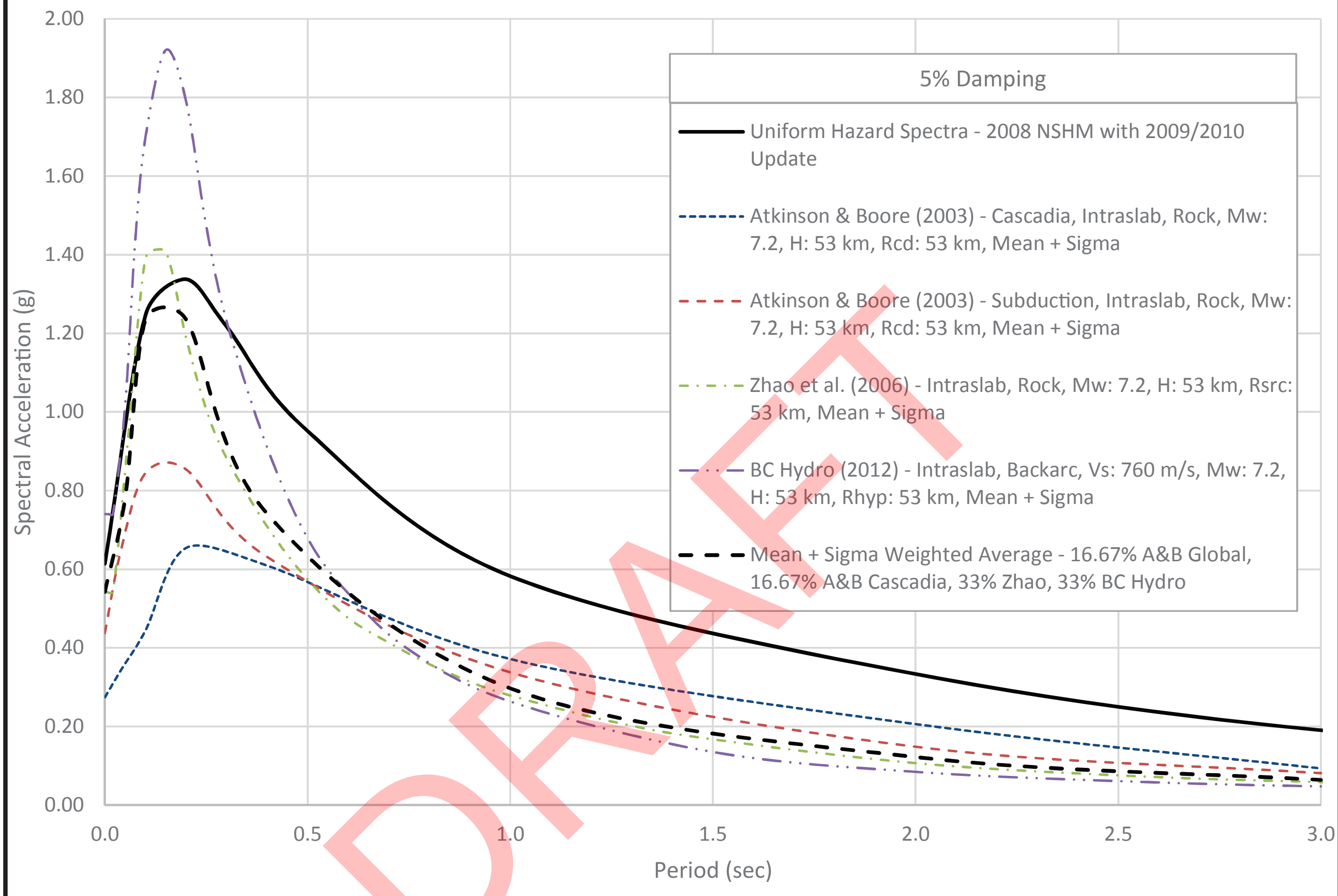
FIG. 12



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INTRASLAB SOURCE TARGET RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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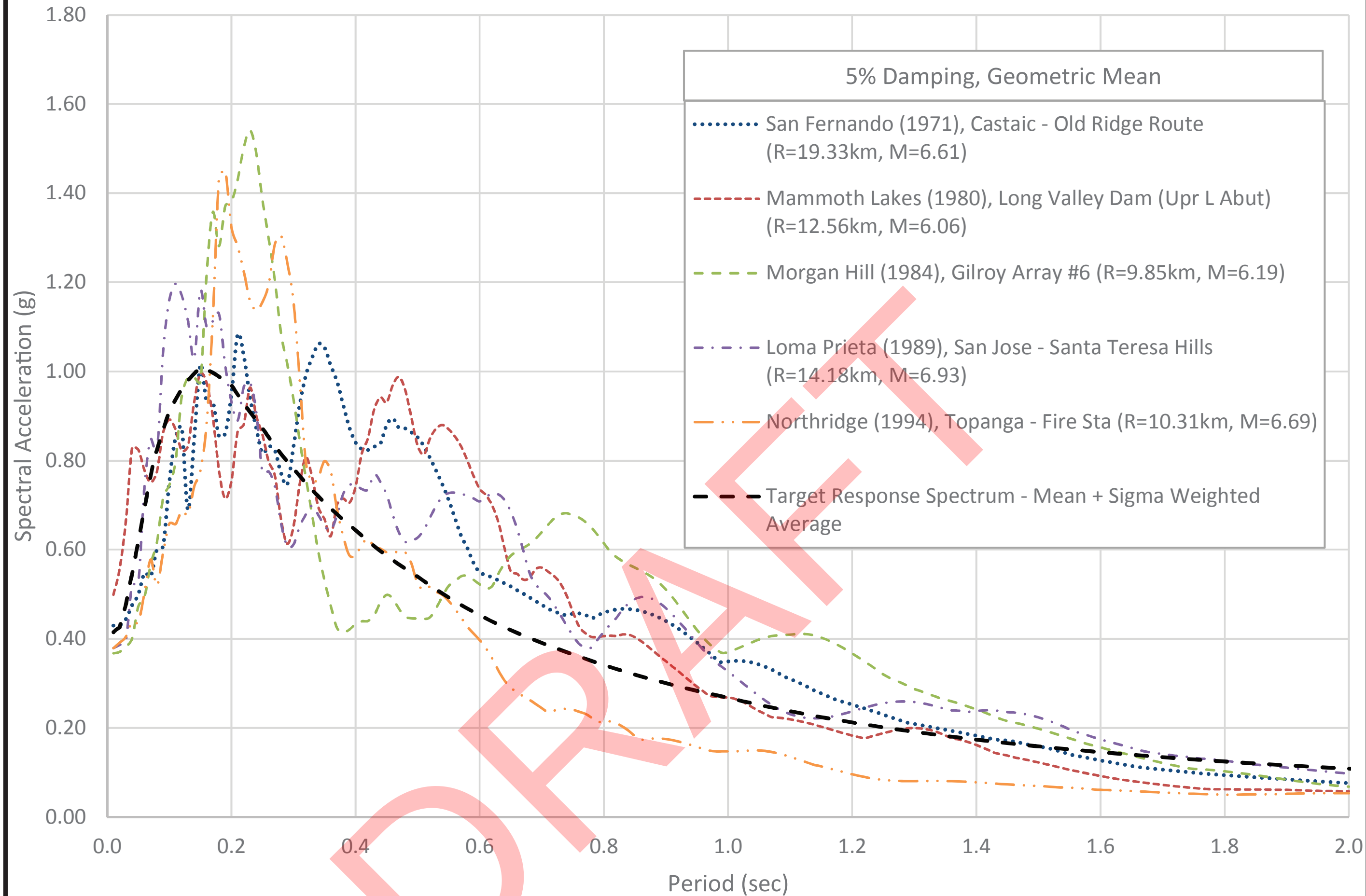
FIG. 13



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SCALED CRUSTAL MOTION RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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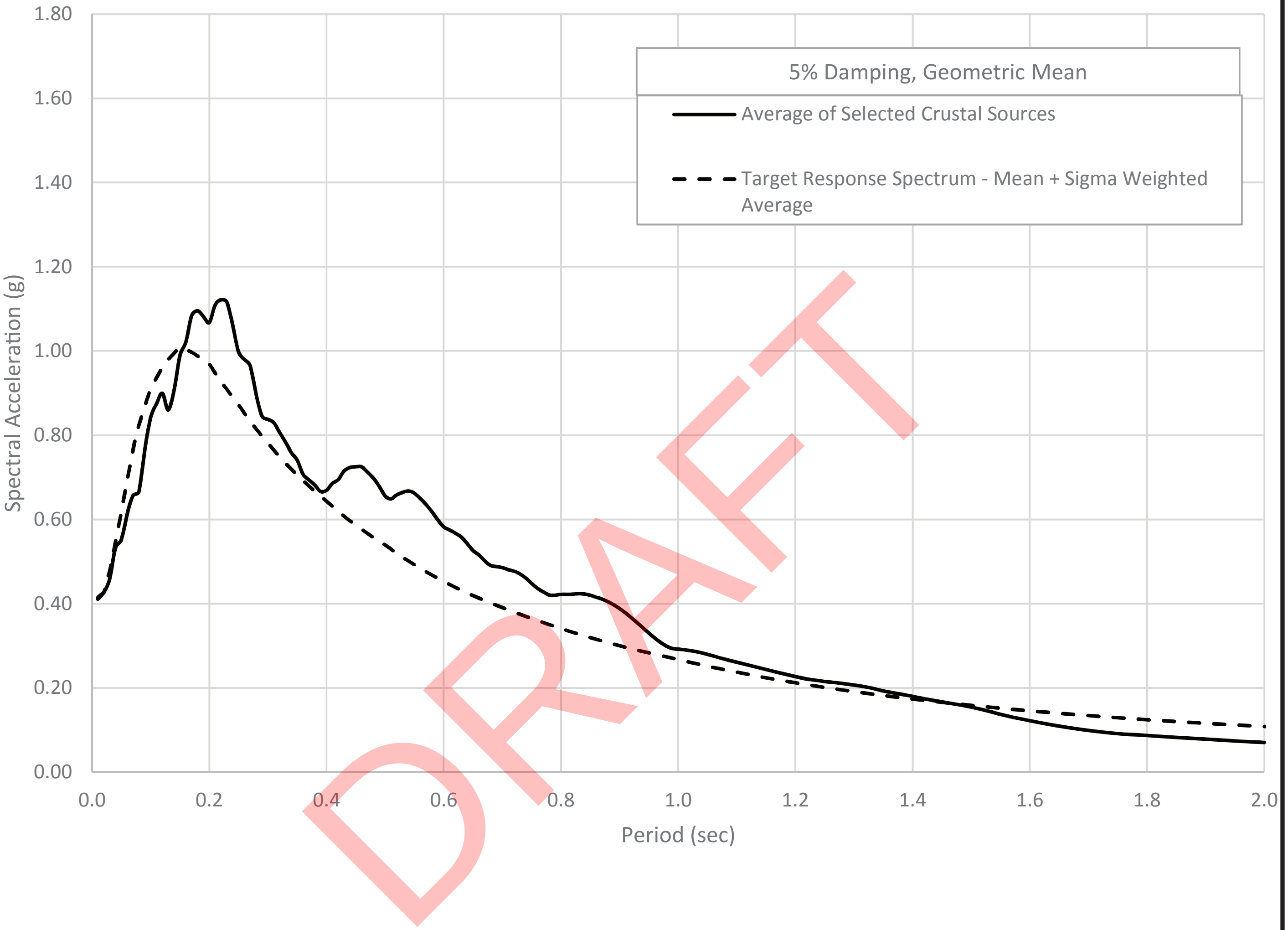
PROJ. 2392

FIG. 14



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AVERAGE CRUSTAL MOTION RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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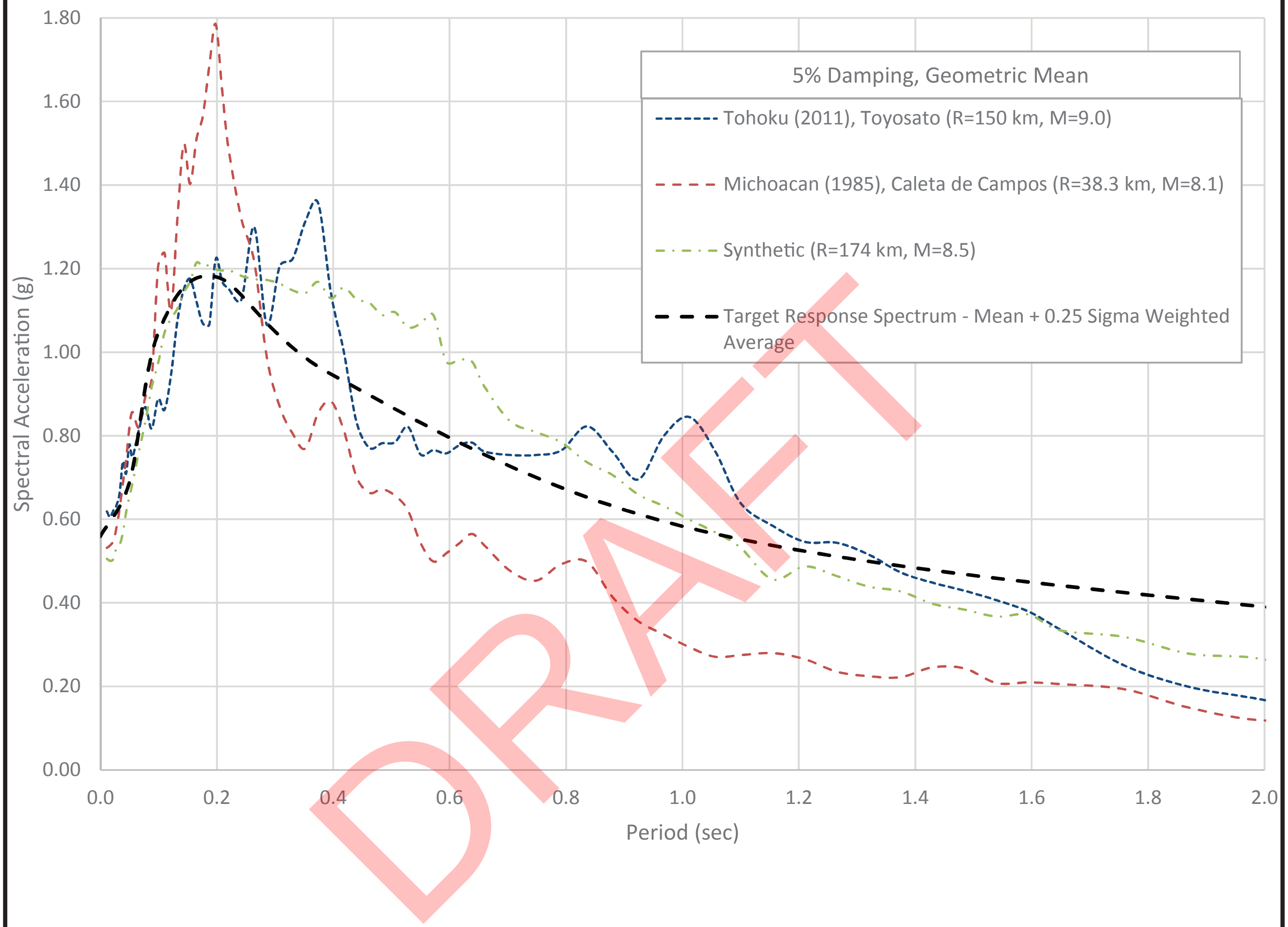
FIG. 15



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SCALED INTERFACE MOTION RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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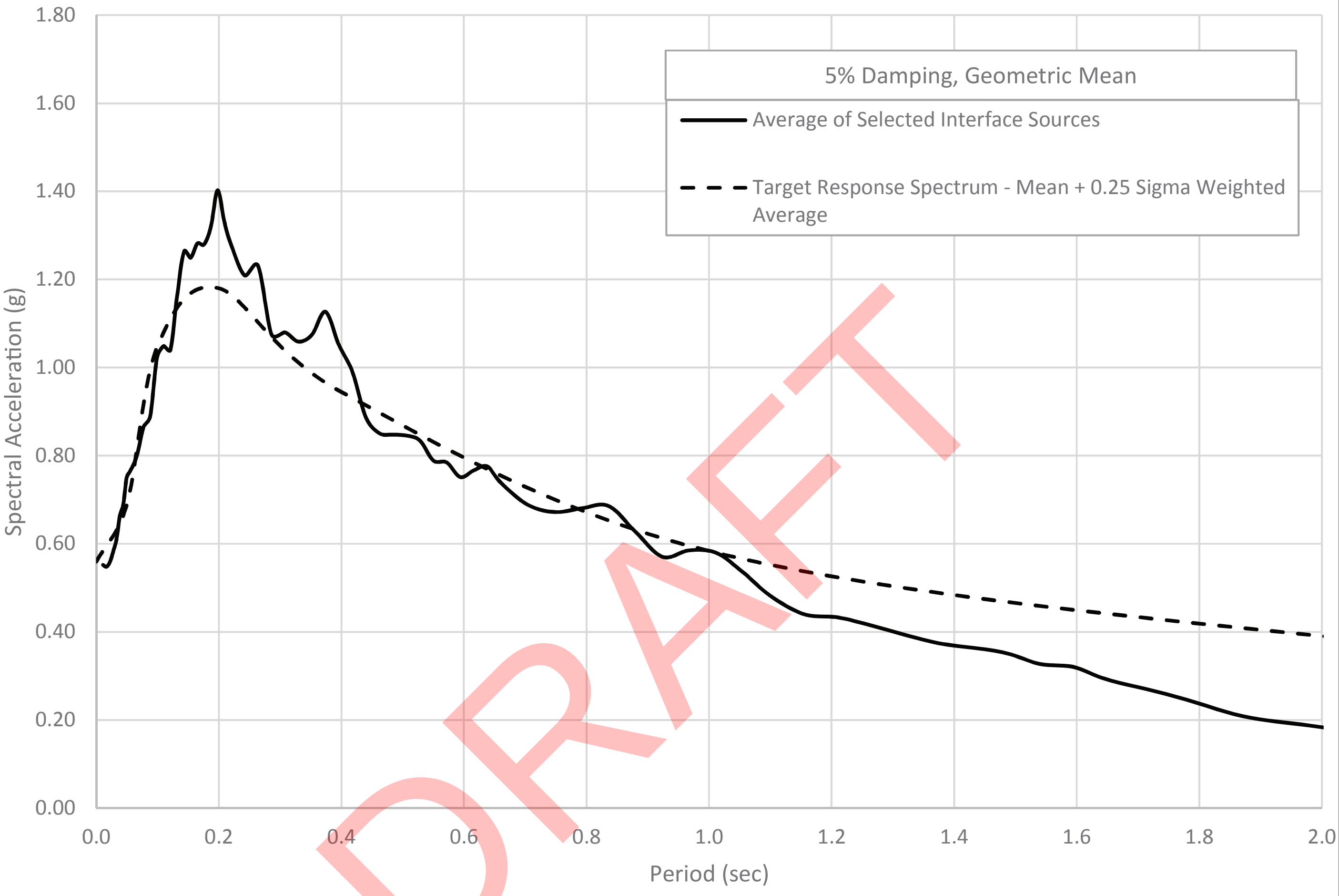
FIG. 16



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AVERAGE INTERFACE MOTION RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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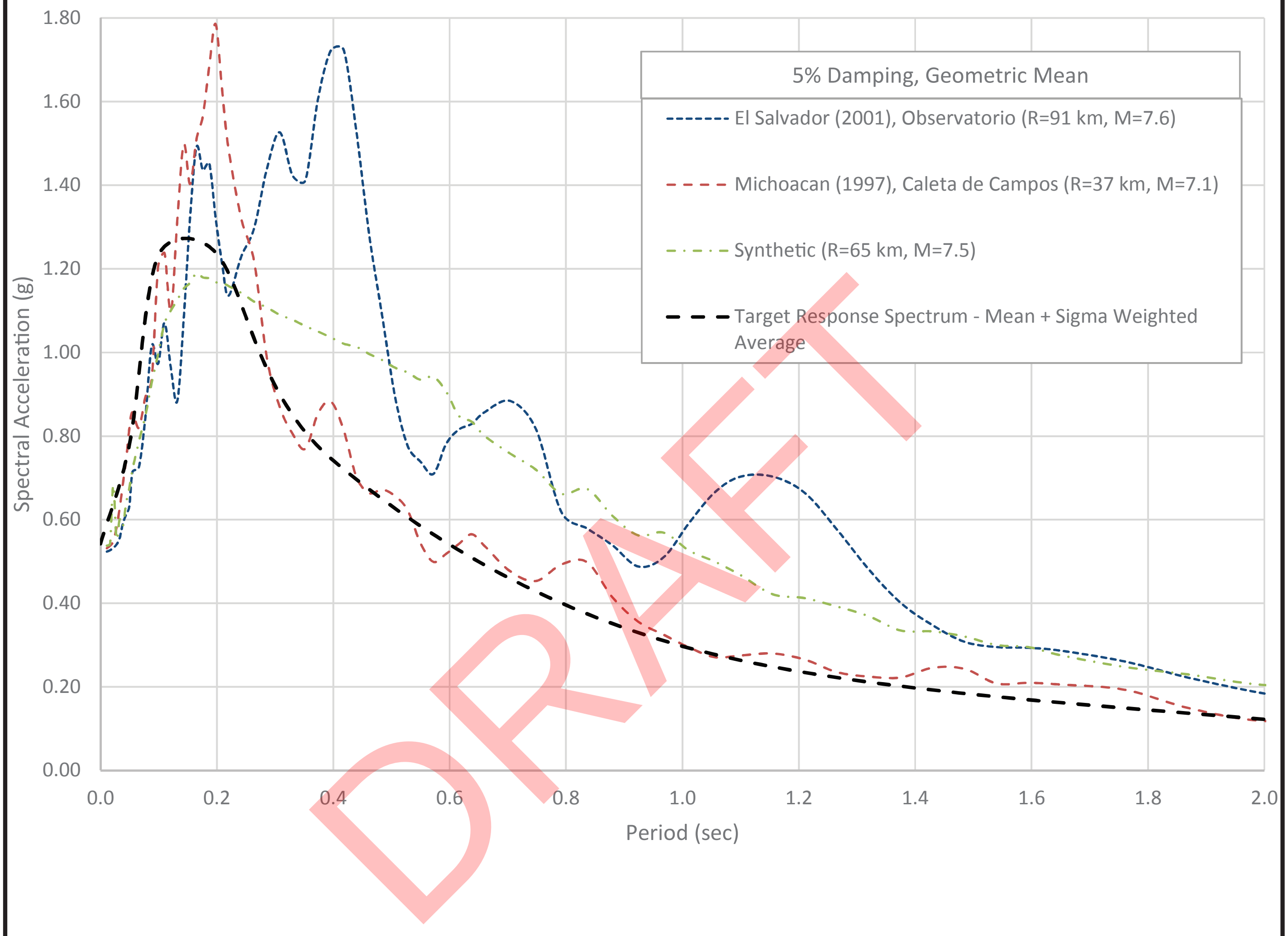
FIG. 17



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SCALED INTRASLAB MOTION RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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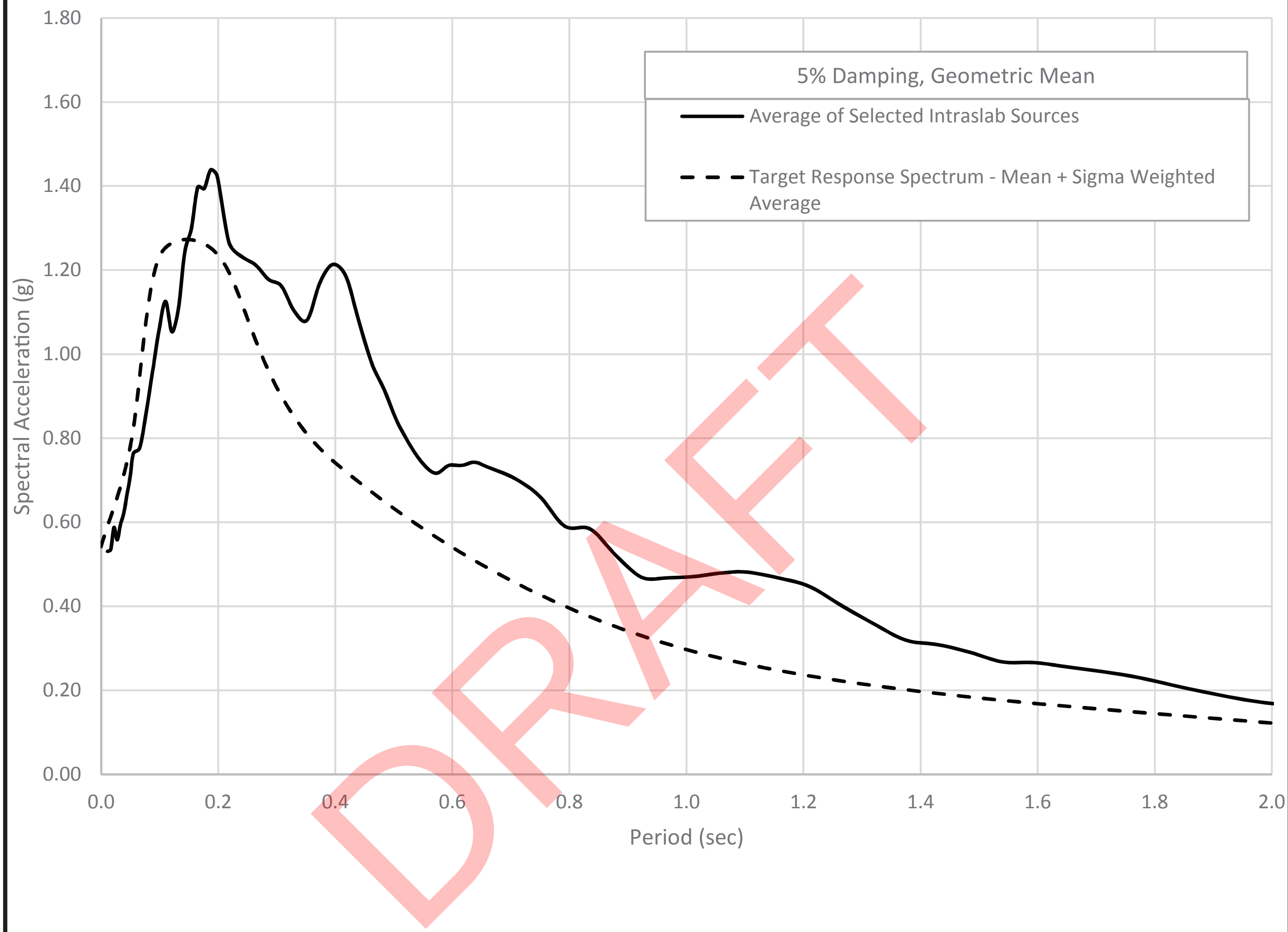
FIG. 18



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AVERAGE INTRASLAB MOTION RESPONSE SPECTRA

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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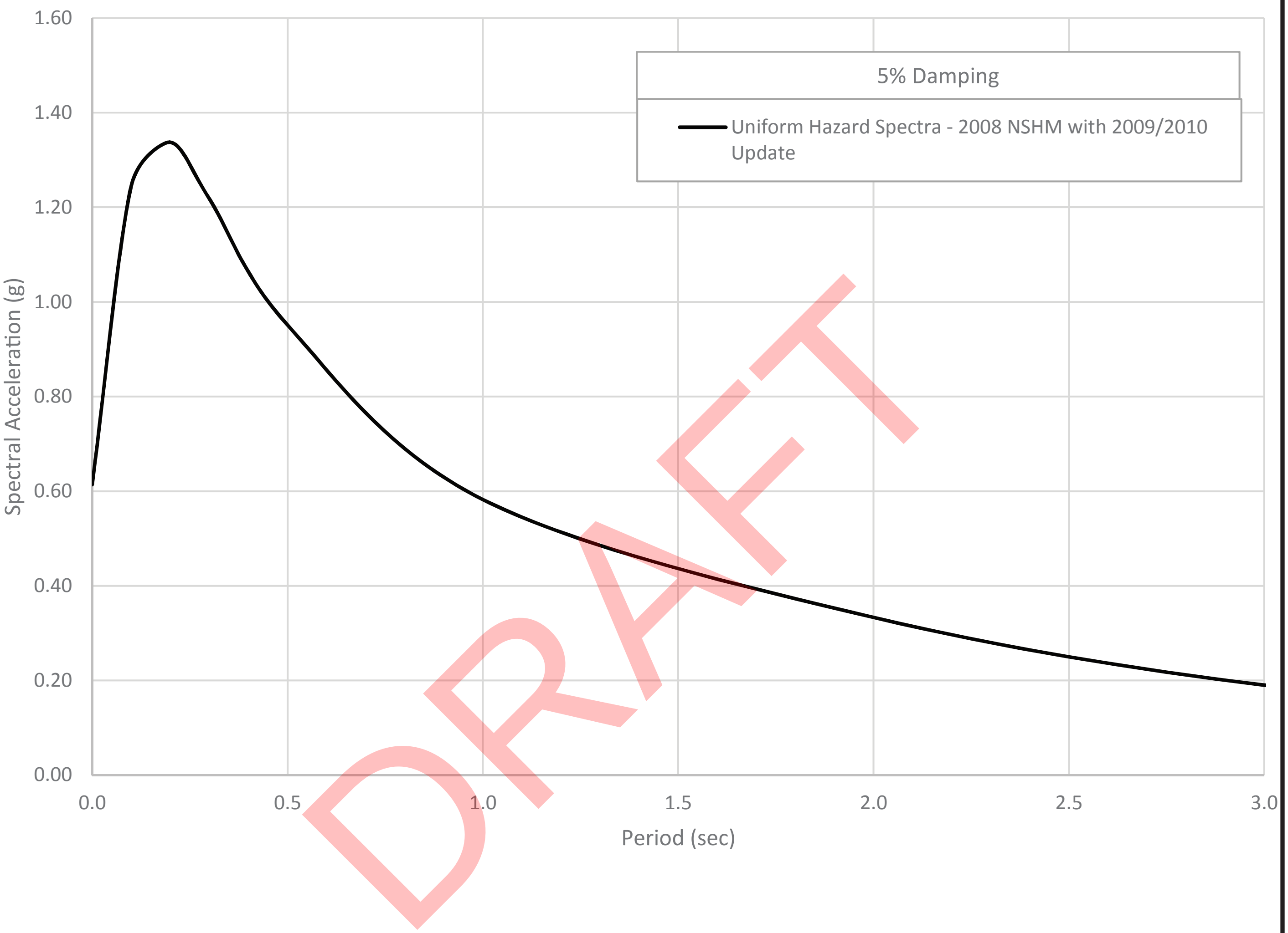
FIG. 19



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2392/20.ai NAU

RESPONSE SPECTRA FOR STRUCTURAL ANALYSIS

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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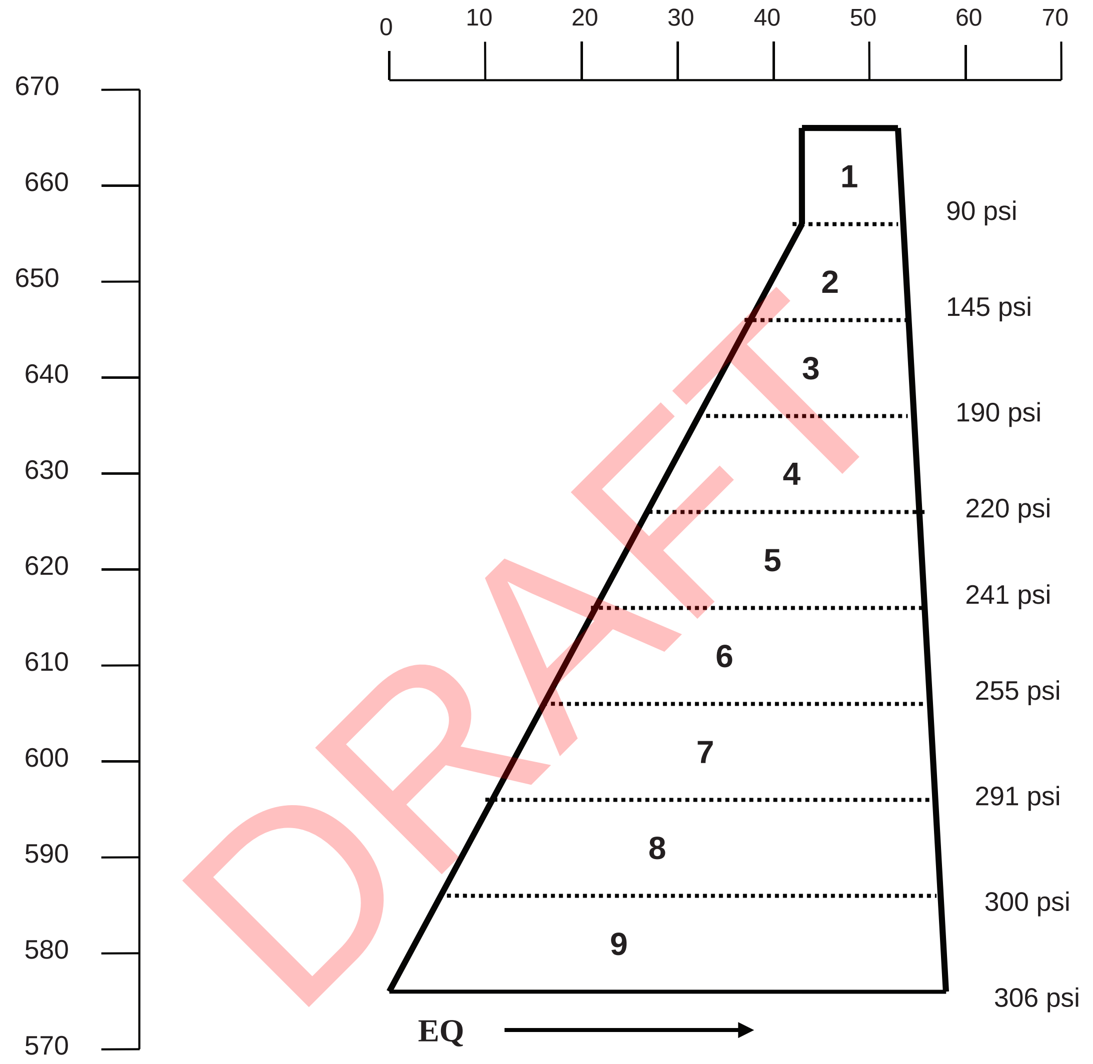
FIG. 20



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2392/21.ai NAU

MAXIMUM TENSILE STRESS DEMAND

BEAR CREEK DAM SEISMIC STABILITY
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FIG. 21

NOTE: TOPO MAP PROVIDED BY OTAK, INC., DECEMBER 2013.

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LOCATION OF ANALYSIS SECTIONS

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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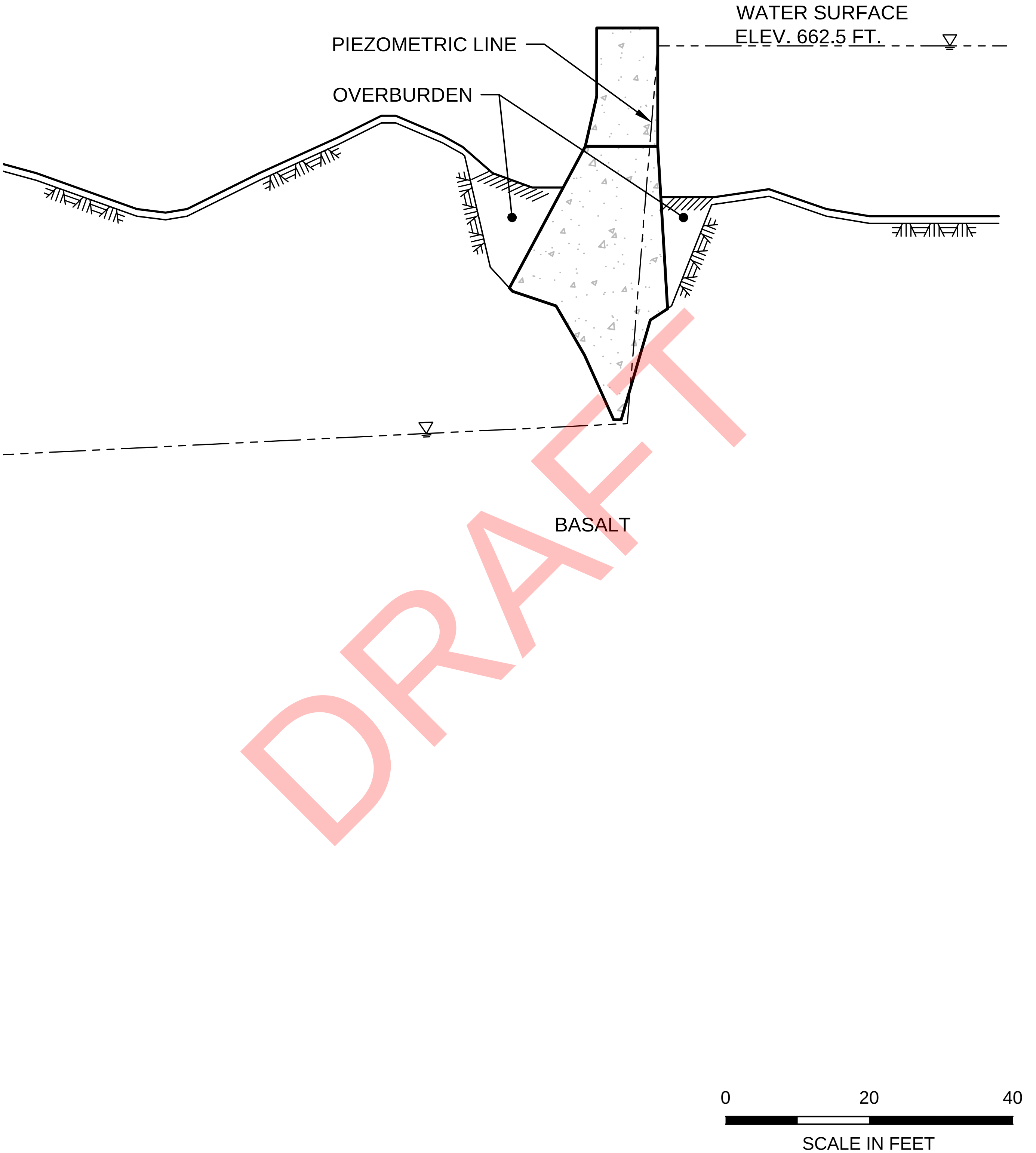
FIG. 22



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ANALYSIS SECTION L3
STA. 0+30

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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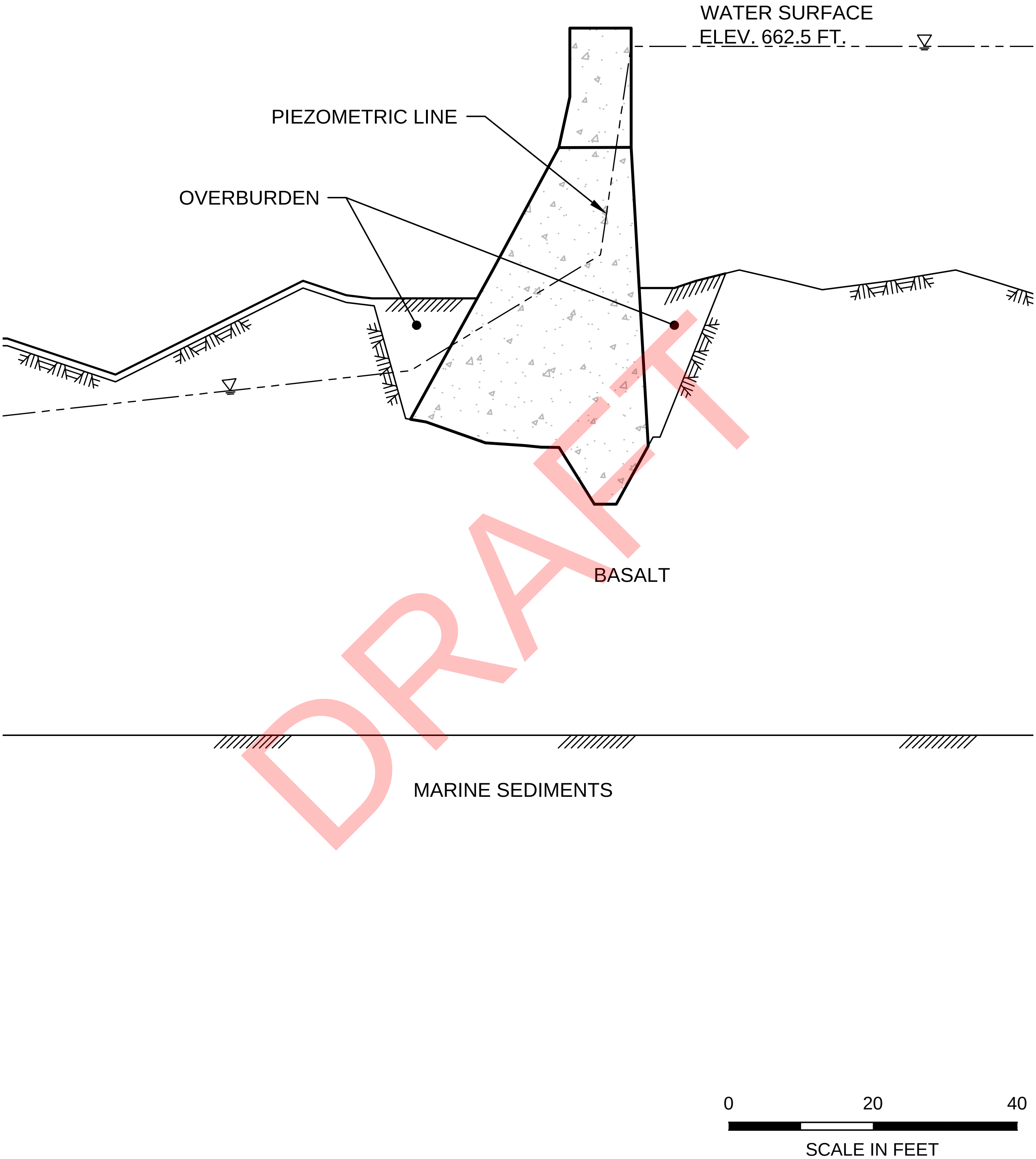
PROJ. 2392

FIG. 23

2392/STA 0+30.DWG NAU



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ANALYSIS SECTION L2
STA. 0+50

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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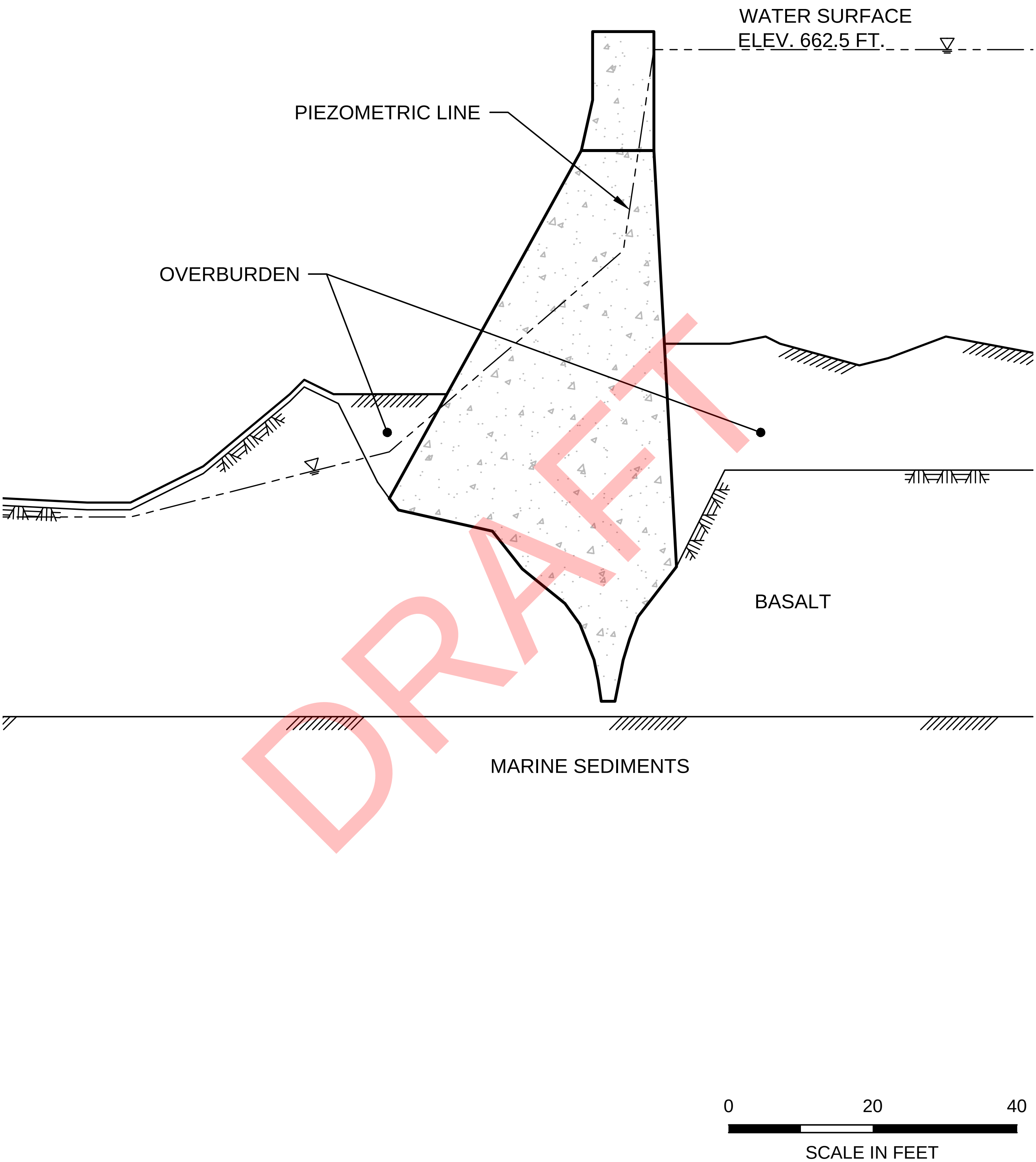
FIG. 24

2392/STA 0+50.DWG NAU



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2392/STA 0+65.DWG NAU

ANALYSIS SECTION L1
STA. 0+65

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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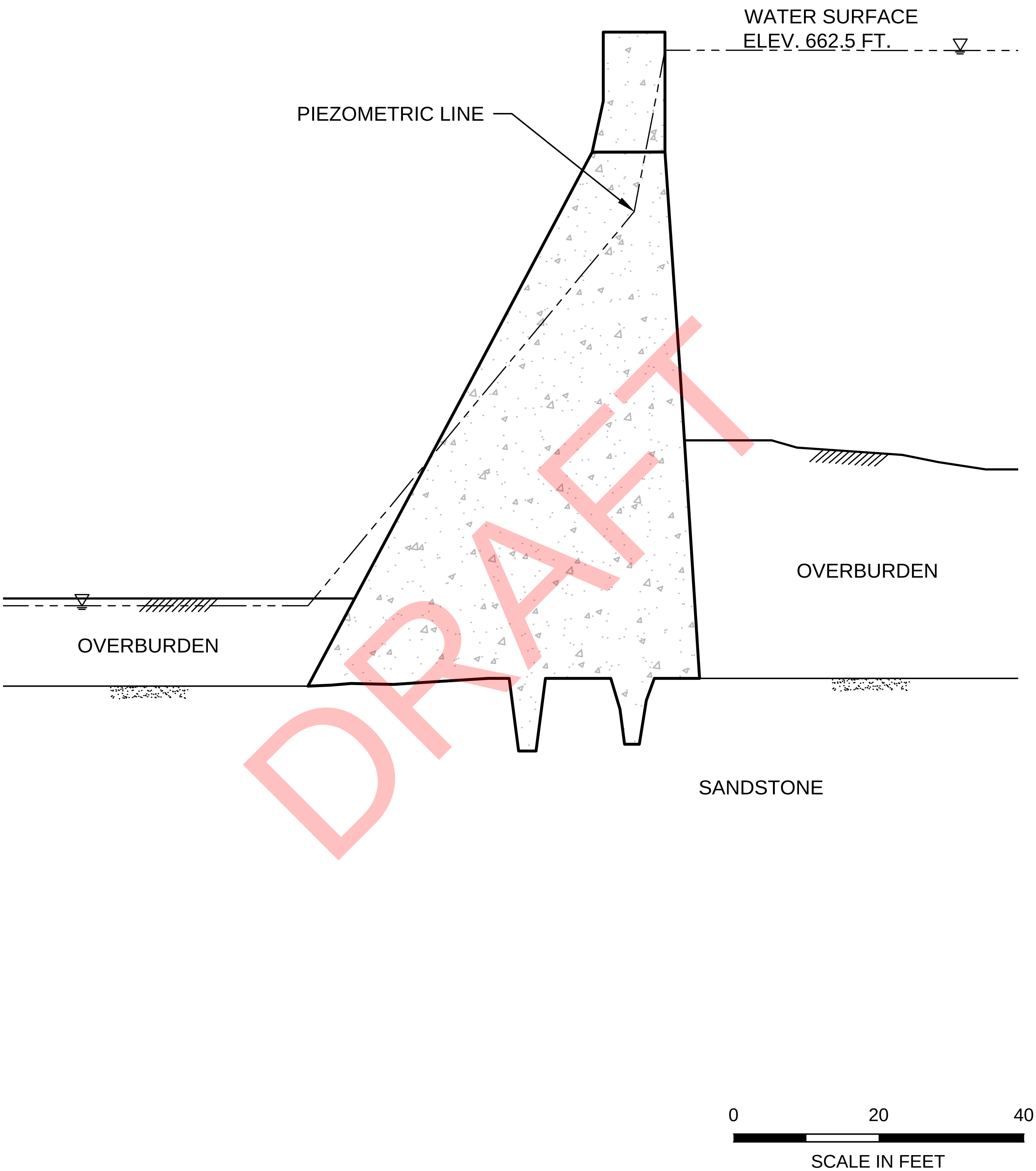
FIG. 25



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ANALYSIS SECTION C1
STA. 0+93

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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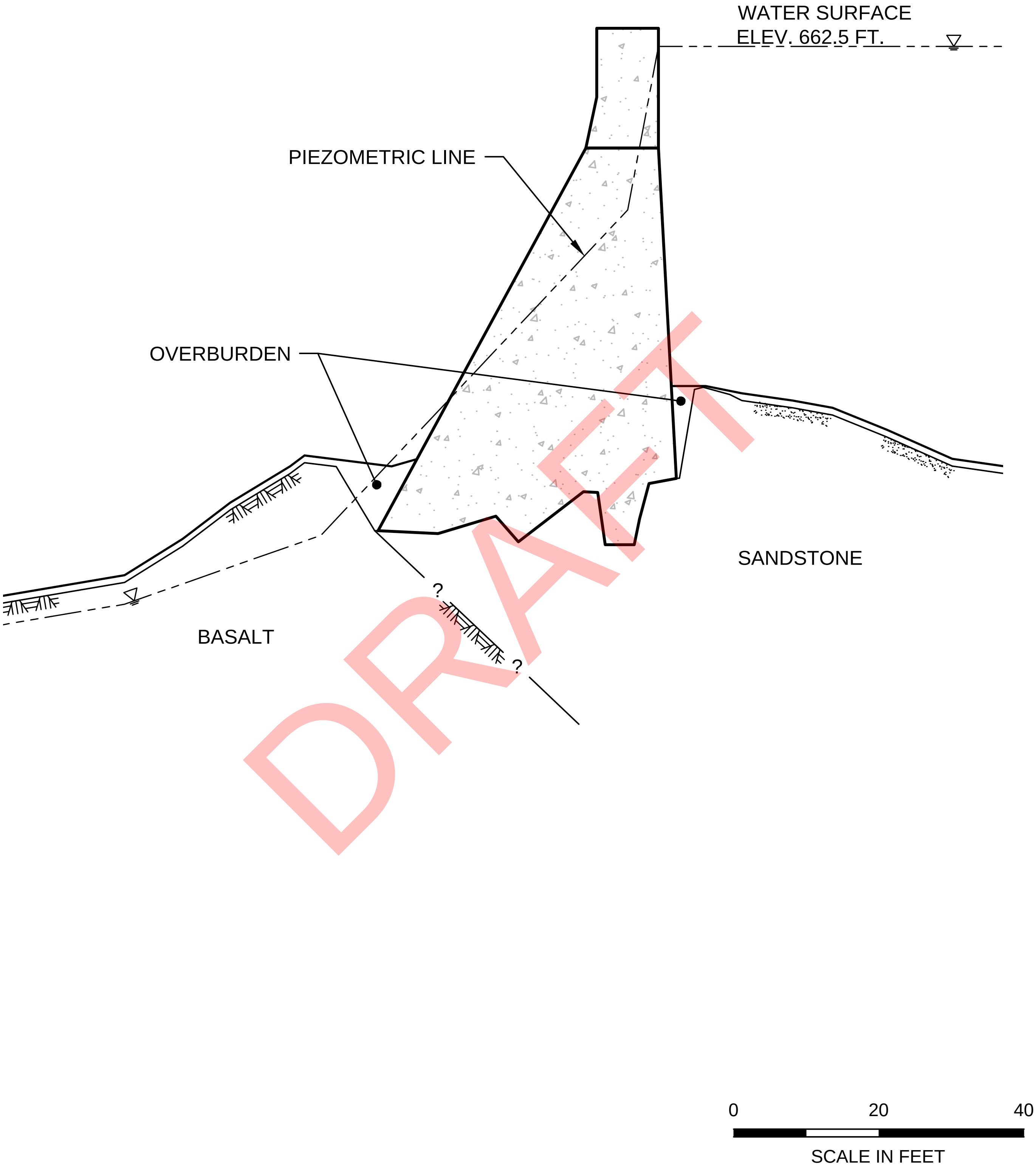
FIG. 26

2392/STA 0+93.DWG NAU



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ANALYSIS SECTION R1
STA. 1+40

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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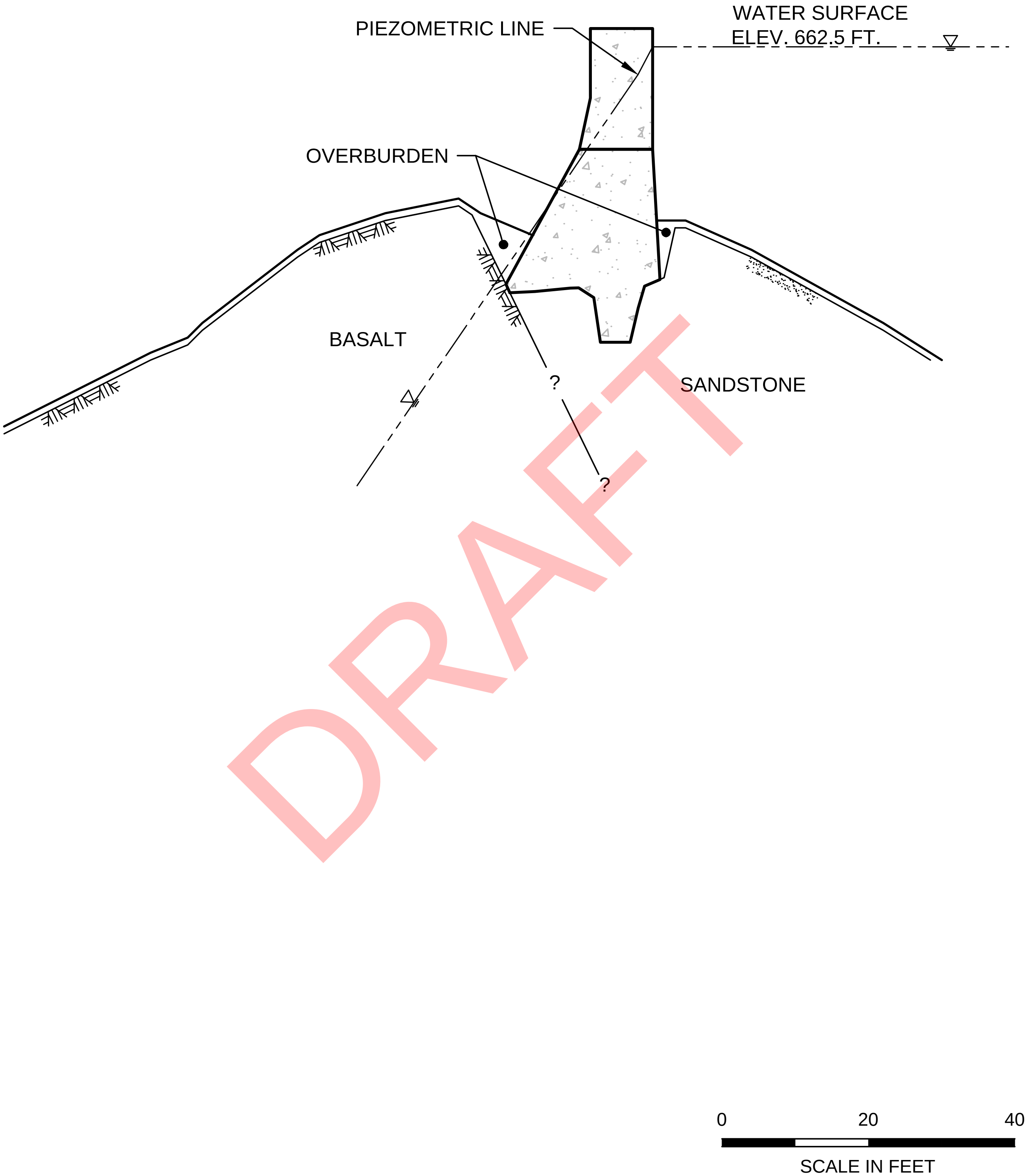
FIG. 27

2392/STA 1+40.DWG NAU



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ANALYSIS SECTION R2
STA. 1+70

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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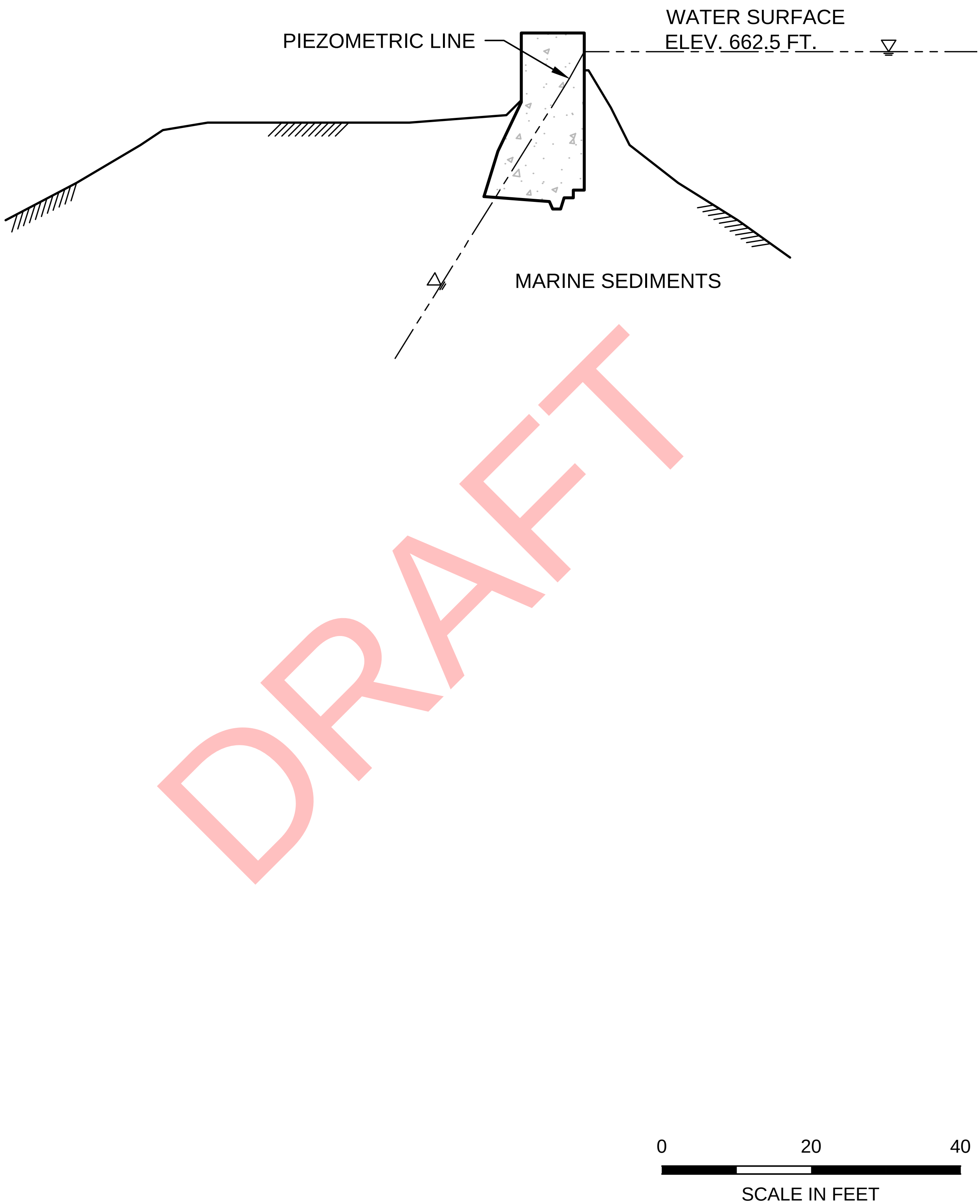
FIG. 28

2392/STA 1+70.DWG NAU



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ANALYSIS SECTION R3
STA. 2+03

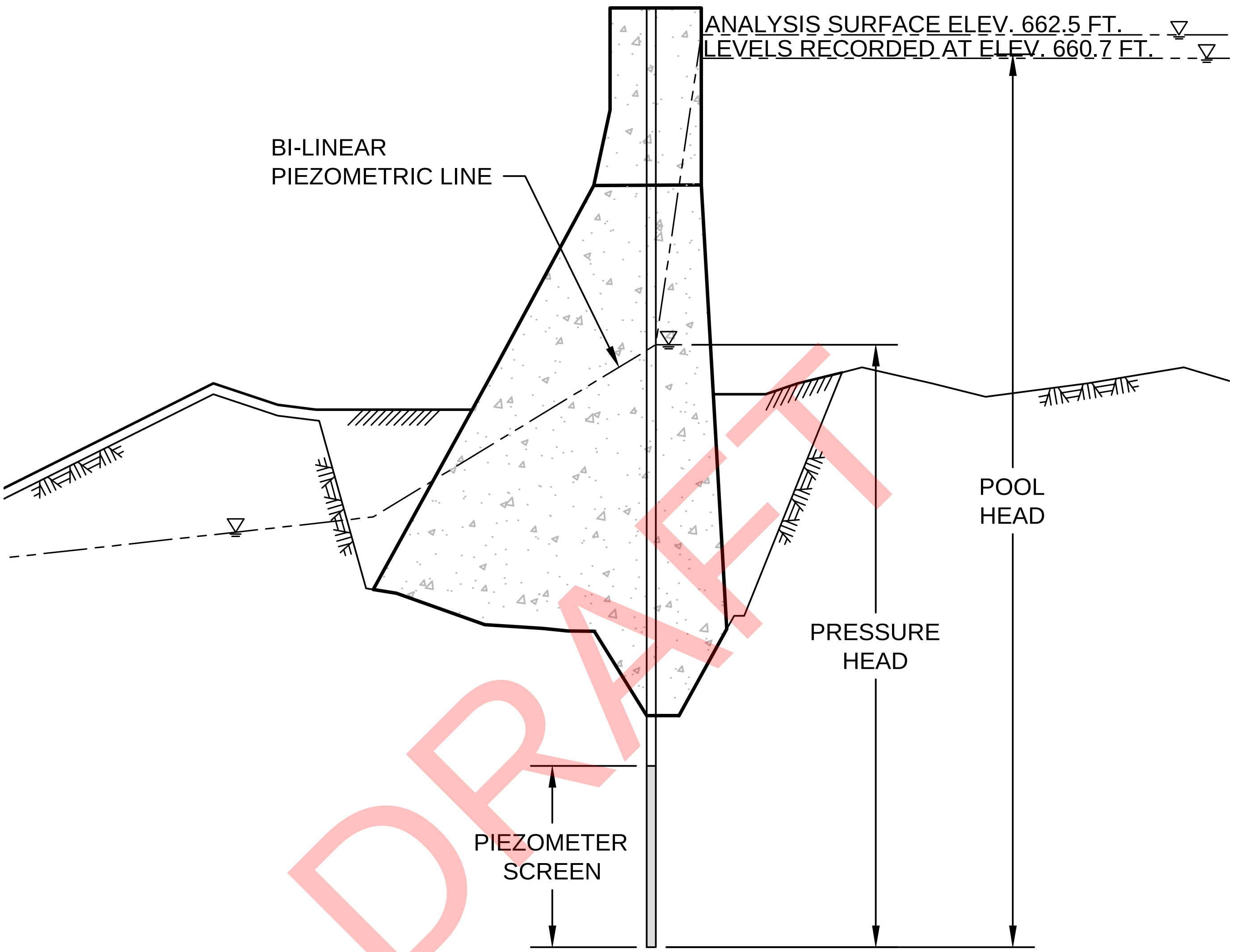
BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

JUN 2015

PROJ. 2392

FIG. 29

2392/STA 2+03.DWG NAU



$$\text{PRESSURE HEAD FOR [662.5']} = \frac{\text{PRESSURE HEAD FOR [660.7']}}{\text{POOL HEAD FOR [660.7']}} \times \text{POOL HEAD FOR [662.5']}$$

NOT TO SCALE

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PIEZOMETRIC ENVELOPE

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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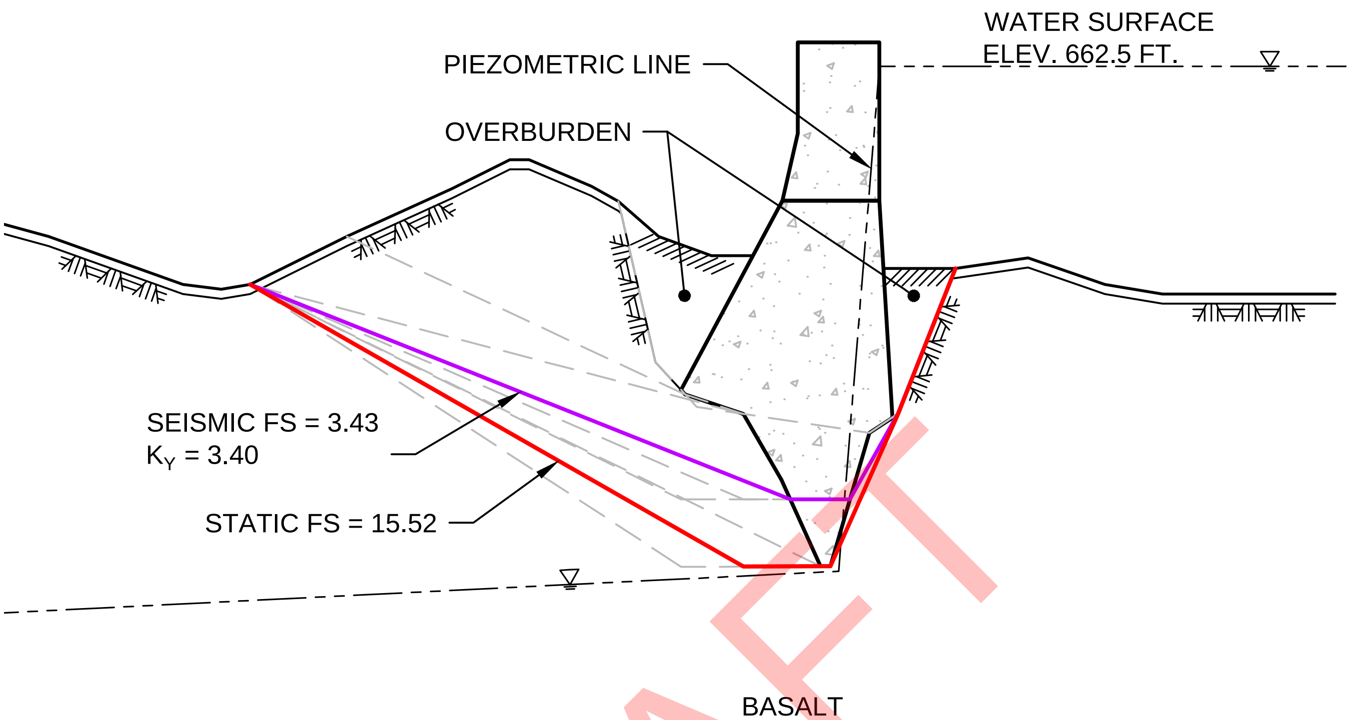
FIG. 30

2392/Piezometer Envelope.dwg NAU



LEGEND

- CRITICAL STATIC SURFACE (solid red line)
- CRITICAL SEISMIC SURFACE (solid purple line)
- OTHER SHEAR SURFACES EVALUATED (dashed grey line)

0 20 40

SCALE IN FEET

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POTENTIAL SHEAR SURFACES - SECTION L3

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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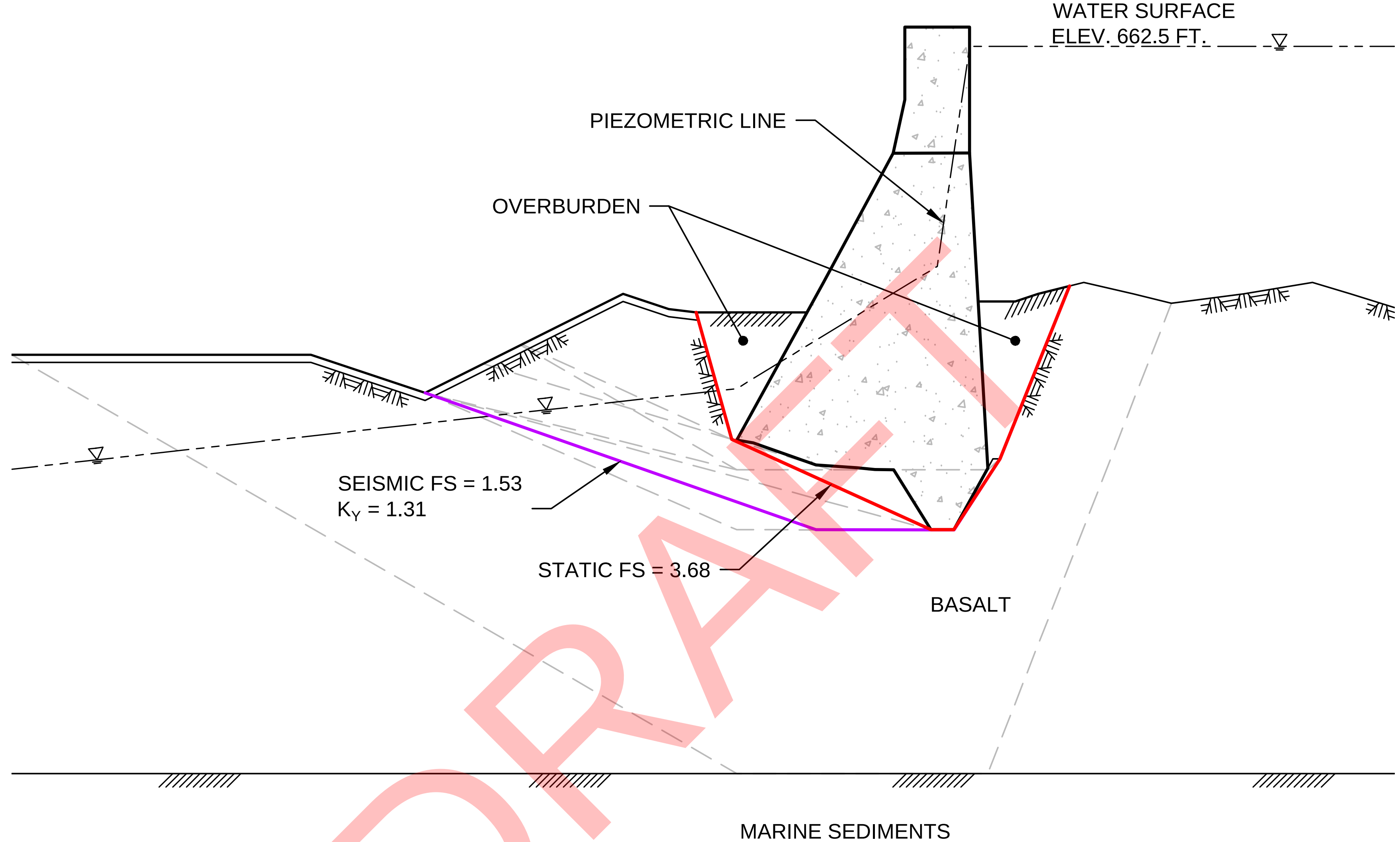
FIG. 31

2392/STA 0+30.DWG NAU



LEGEND

CRITICAL STATIC SURFACE

CRITICAL SEISMIC SURFACE

OTHER SHEAR SURFACES EVALUATED

0 20 40

SCALE IN FEET

2392/STA 0+50.DWG NAU

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POTENTIAL SHEAR SURFACES - SECTION L2

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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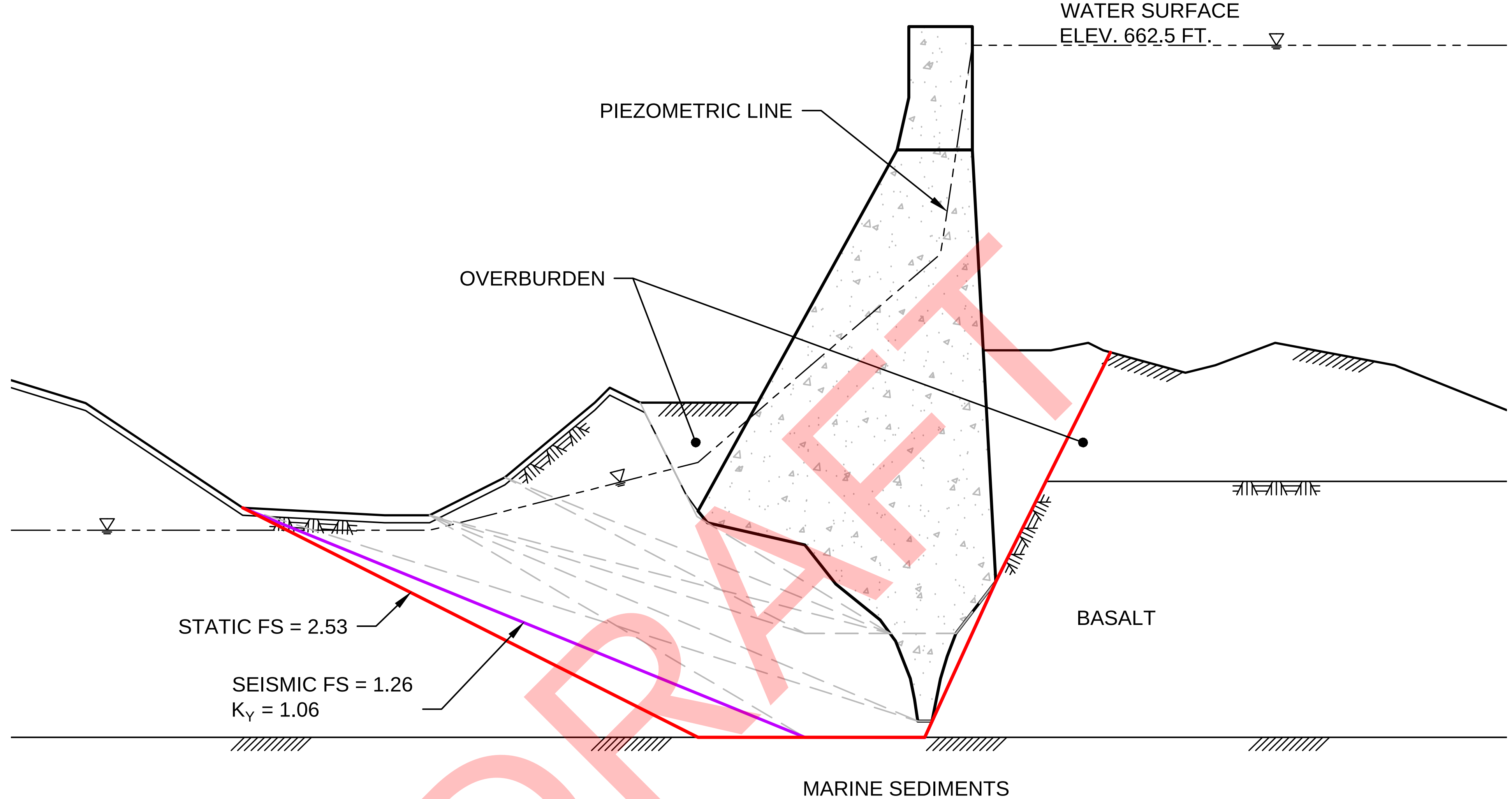
FIG. 32



LEGEND

- CRITICAL STATIC SURFACE (red line)
- CRITICAL SEISMIC SURFACE (purple line)
- OTHER SHEAR SURFACES EVALUATED (dashed grey line)

0 20 40

SCALE IN FEET

2392/STA 0+65.DWG NAU

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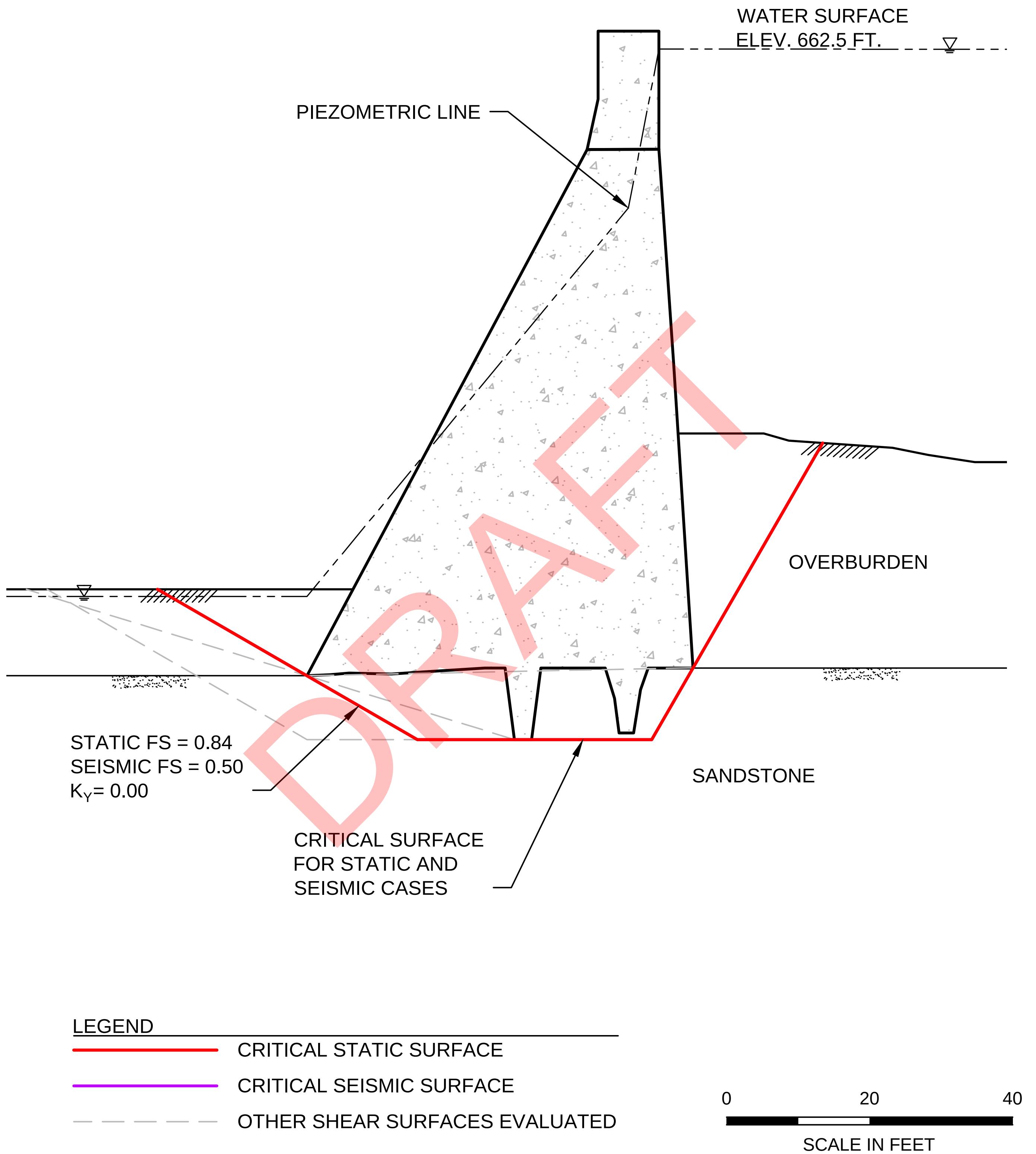
POTENTIAL SHEAR SURFACES - SECTION L1

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

JUN 2015

PROJ. 2392

FIG. 33



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2392/STA 0+93.DWG NAU

POTENTIAL SHEAR SURFACES - SECTION C1

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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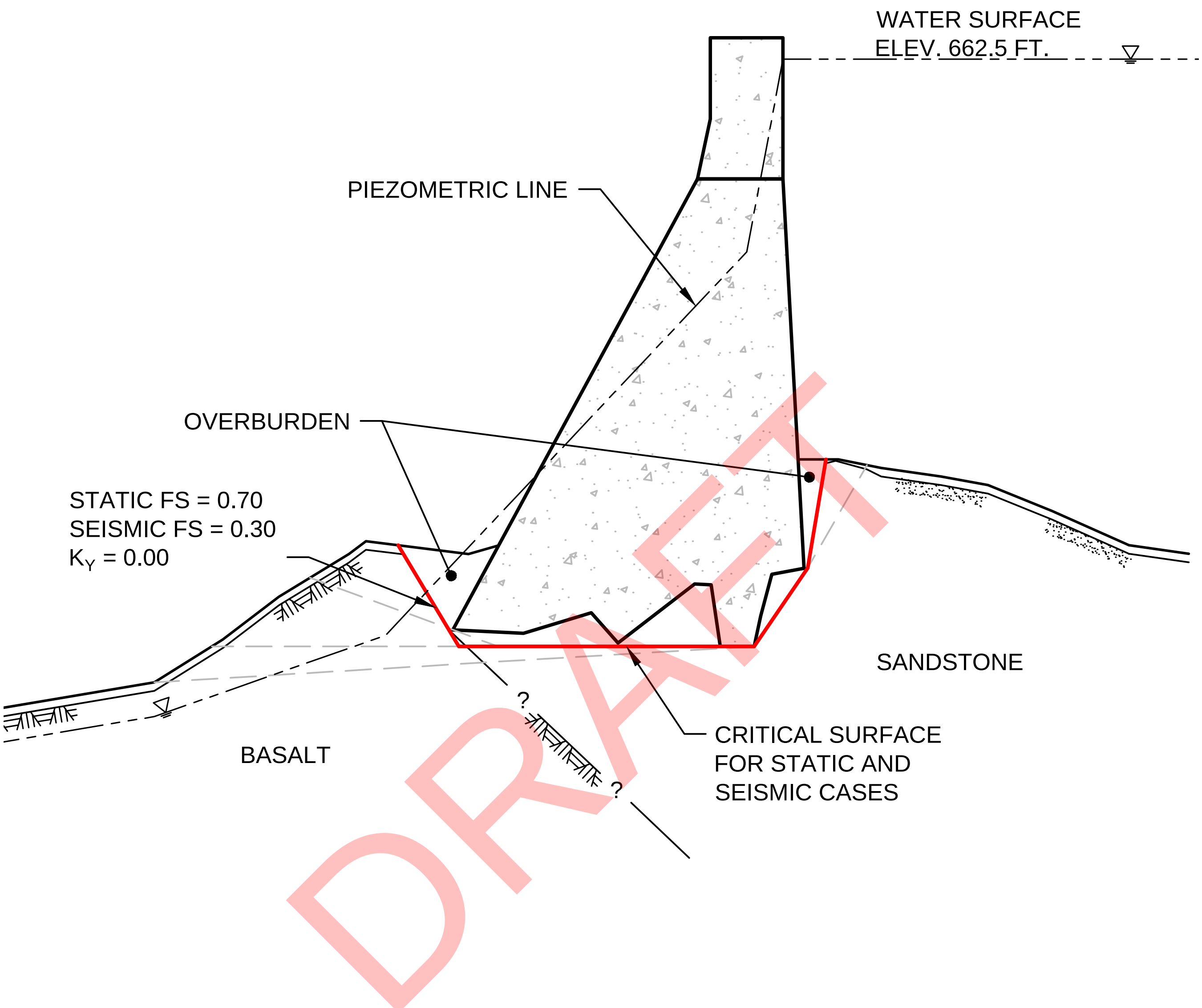
PROJ. 2392

FIG. 34



LEGEND

CRITICAL STATIC SURFACE

CRITICAL SEISMIC SURFACE

OTHER SHEAR SURFACES EVALUATED

0 20 40

SCALE IN FEET

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POTENTIAL SHEAR SURFACES - SECTION R1

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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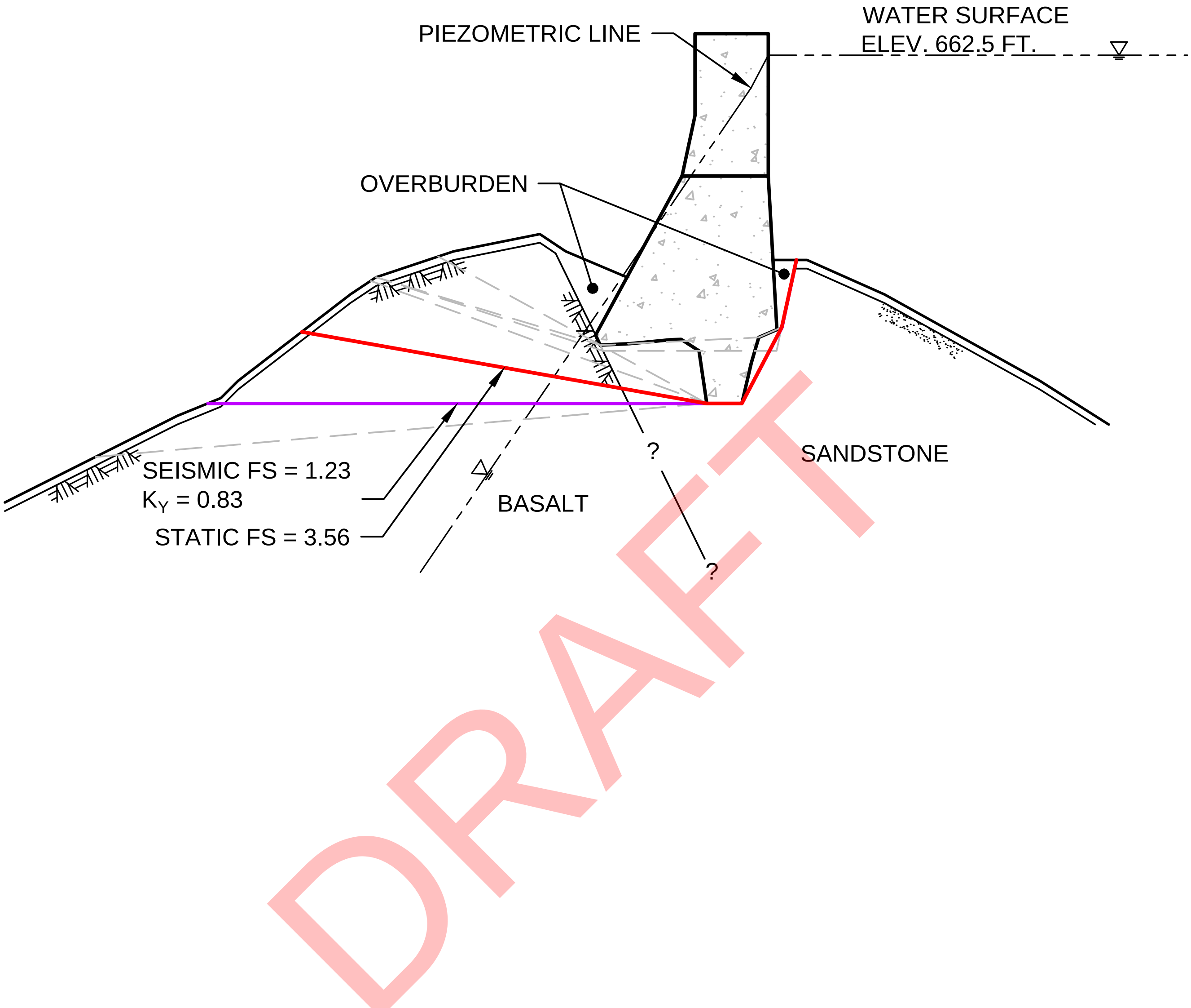
FIG. 35

2392/STA 1+40.DWG NAU



LEGEND

CRITICAL STATIC SURFACE

CRITICAL SEISMIC SURFACE

OTHER SHEAR SURFACES EVALUATED

0 20 40

SCALE IN FEET

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POTENTIAL SHEAR SURFACES - SECTION R2

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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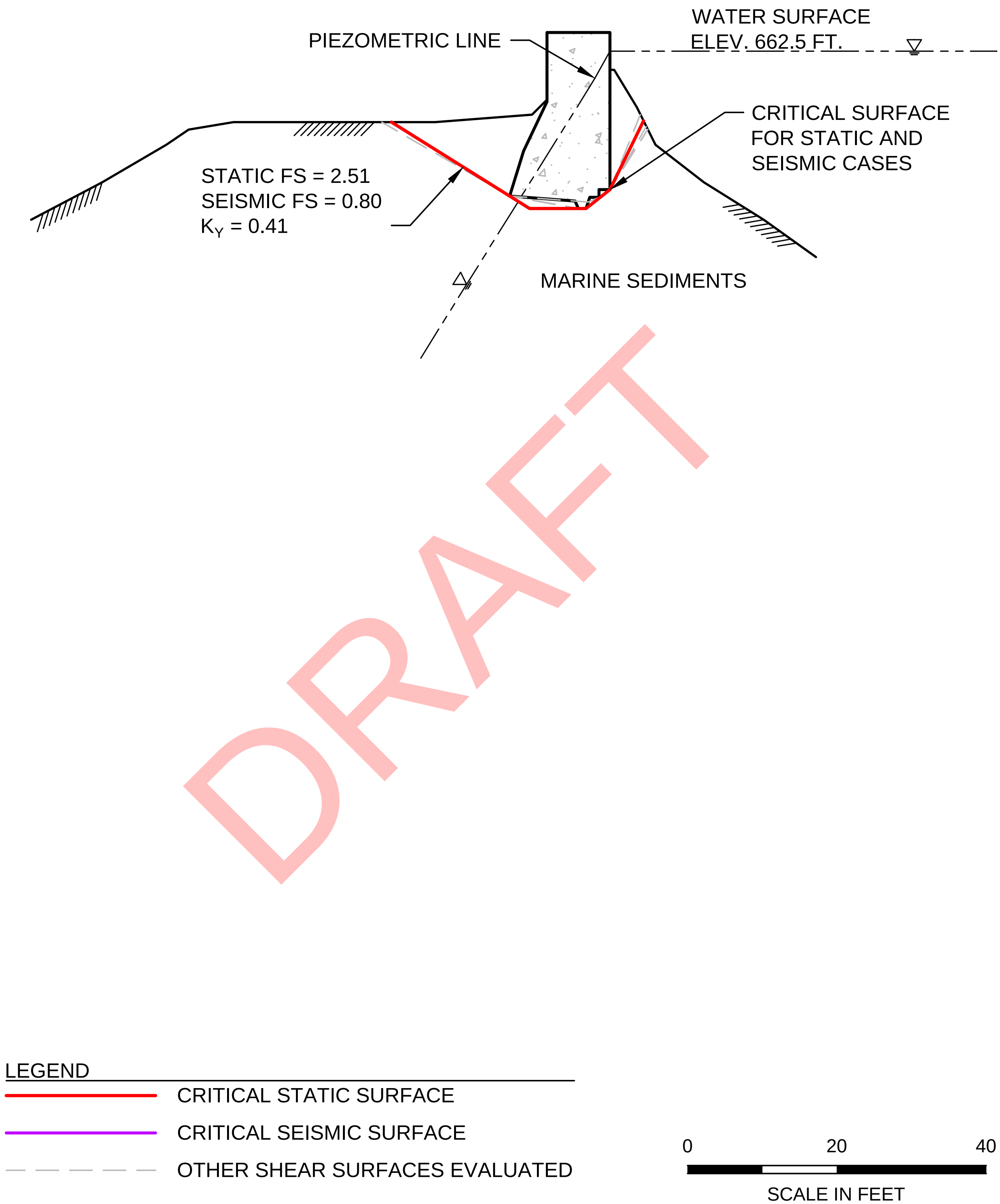
FIG. 36

2392/STA 1+70.DWG NAU



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POTENTIAL SHEAR SURFACES - SECTION R3

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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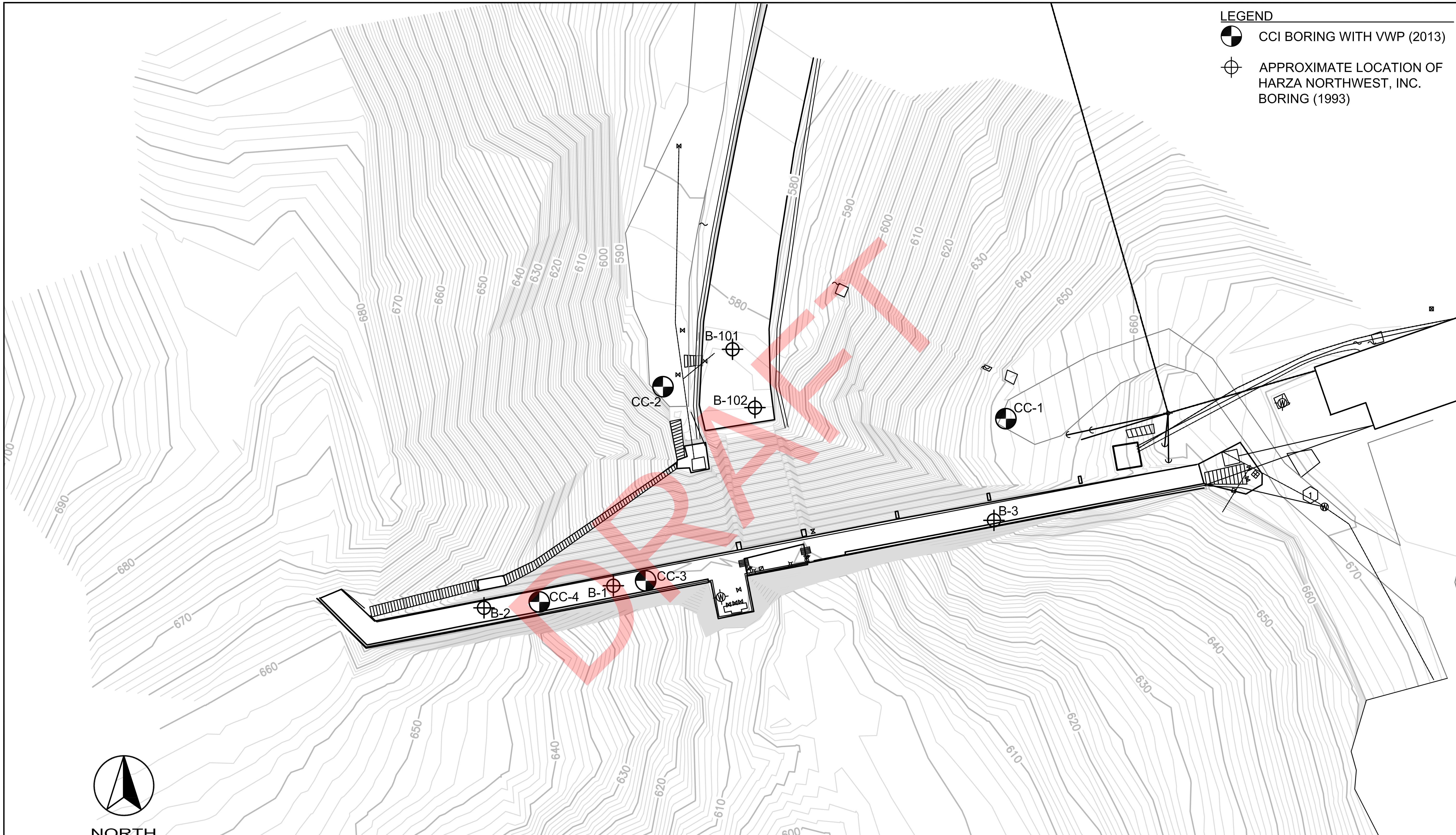
FIG. 37

2392/STA 2+03.DWG NAU

Appendix A

Summary Boring Logs



0 30 60

SCALE IN FEET

NOTE: TOPO MAP PROVIDED BY OTAK, INC., DECEMBER 2013.

2332/S67479B190.dwg NAU

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SITE PLAN

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

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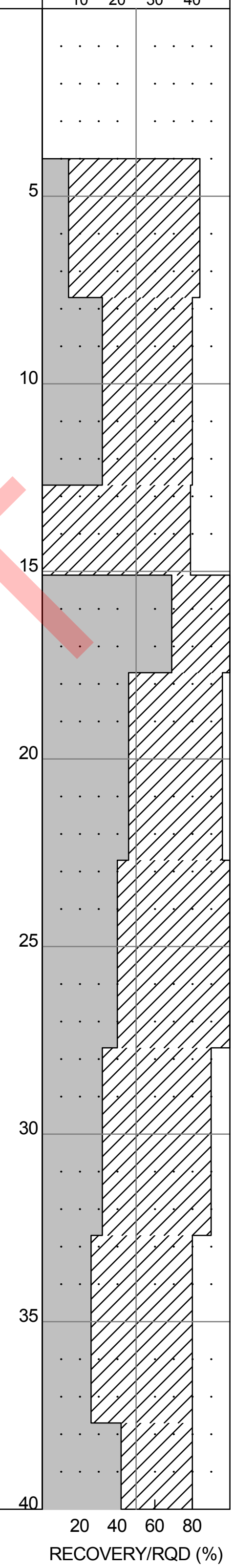
FIG. 2

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 650.8 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION	PENETRATION TEST (BLOWS PER FOOT) WATER CONTENT (%) 10 20 30 40
		Quick Drill to 4.0 feet to accommodate HQ3 core barrel - No Samples Taken … driller notes "harder" drilling at 3.5 feet					
646.8	4	HARD to VERY HARD (R4-R5), gray, slightly weathered BASALT; very highly to highly jointed, dominant 0 to 20° (smooth, planar) numerous 40 to 60° and 70 to 90° joints, occasional clayey SILT infilling along joints up to ¼-inch thick, abundant iron staining and black mineralization along joints up to 1-inch thick, medium hard (R3) weathered surfaces on joint up to 1/2" thick, occasional zones of sheared/diced clayey SILT in joint with slickensided surfaces (COLUMBIA RIVER BASALT?) … slickensides on 80° joint (85° rake) at 10.2 feet	R-1 R-2 R-3 R-4 R-5 R-6 R-7 R-8 R-9				
610.8	40						



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 45.7 FT (S/N: 1326217)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/14/2013 FINISH: 10/15/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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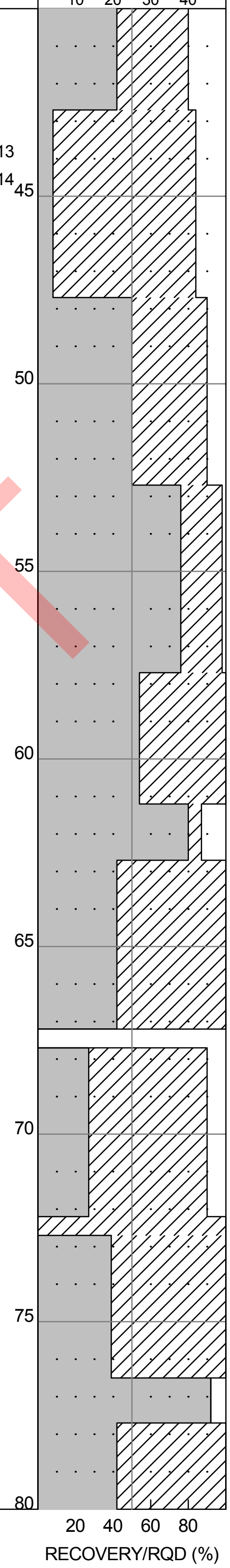
SUMMARY BORING LOG
CC-1 (1 of 3)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 4

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 650.8 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION
		(continued from previous page)	R-9			
			R-10			10/17/13; 01/15/14
		… diced/sheared zone from 44.7 to 46.2 feet with slickensides on 80° joint (87° rake) at 45.1 feet				
			R-11			
			R-12			
			R-13			
		… slickensides on 70° joint (70° rake) with ¼-inch diced textured infilling at 60.8 feet	R-14			
			R-15			
			R-16			
		… diced texture from 69.3 to 69.7 feet	R-17			
		… diced texture from 70.8 to 70.9 feet				
			R-18			
			R-19			
			R-20			
570.8	80		R-21			

PENETRATION TEST (BLOWS PER FOOT)
WATER CONTENT (%)
10 20 30 40



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- STANDARD PENETRATION TEST (BLOWS/FT.)
- WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 45.7 FT (S/N: 1326217)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

DRAFT

HAMMER ASSEMBLY: NOT APPLICABLE   SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE   BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/14/2013 FINISH: 10/15/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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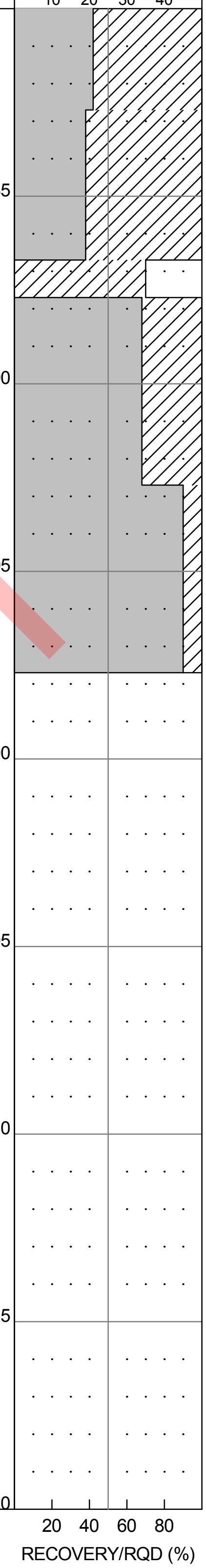
SUMMARY BORING LOG CC-1 (2 of 3)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 4

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 650.8 FT.	RUN NO.	SAMPLE NO.	SAMPLE PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION	PENETRATION TEST (BLOWS PER FOOT) WATER CONTENT (%) 10 20 30 40
		(continued from previous page)	R-21				
			R-22				85
			R-23				
			R-24				90
			R-25				95
553.1	97.7	Bottom of Boring: 97.7 FT					100, 105, 110, 115, 120



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 45.7 FT (S/N: 1326217)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE   SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE   BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/14/2013 FINISH: 10/15/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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SUMMARY BORING LOG
CC-1 (3 of 3)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

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FIG. 4

BOX 1 - 4.0 TO 14.7 FT

BOX 2 - 14.7 TO 22.7 FT

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2332/CC-1.ai NAU

CORE BOX PHOTOS
CC-1 (1 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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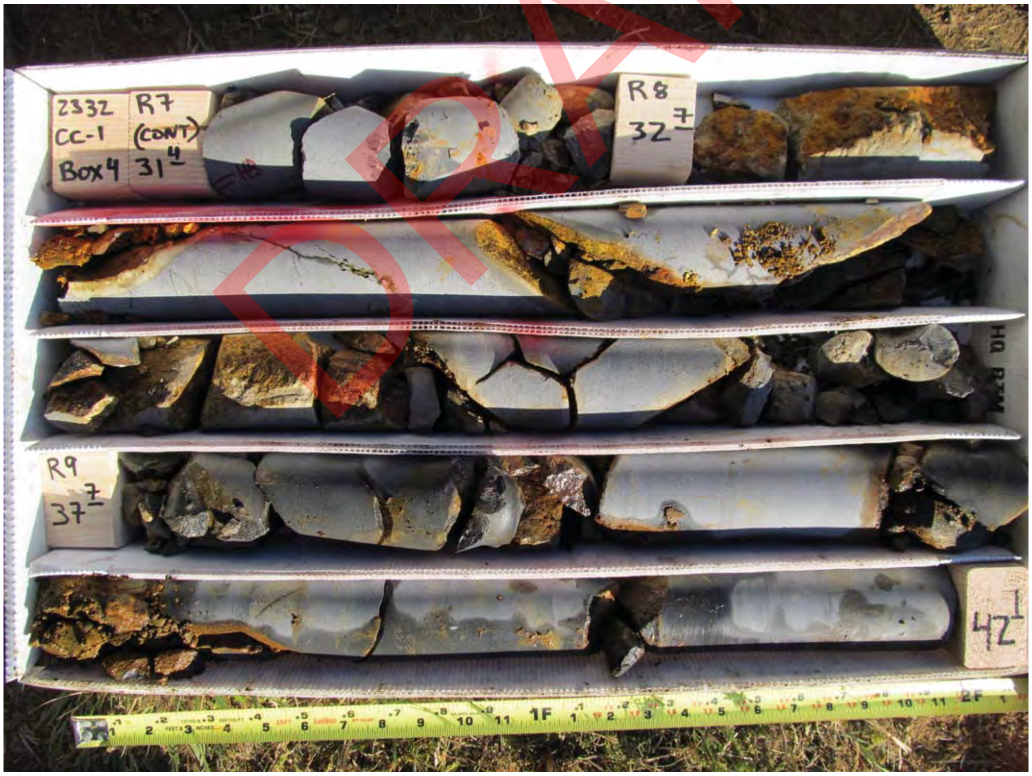
FIG. 5

BOX 3 - 22.7 TO 31.4 FT



BOX 4 - 31.4 TO 42.1 FT

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CORE BOX PHOTOS
CC-1 (2 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
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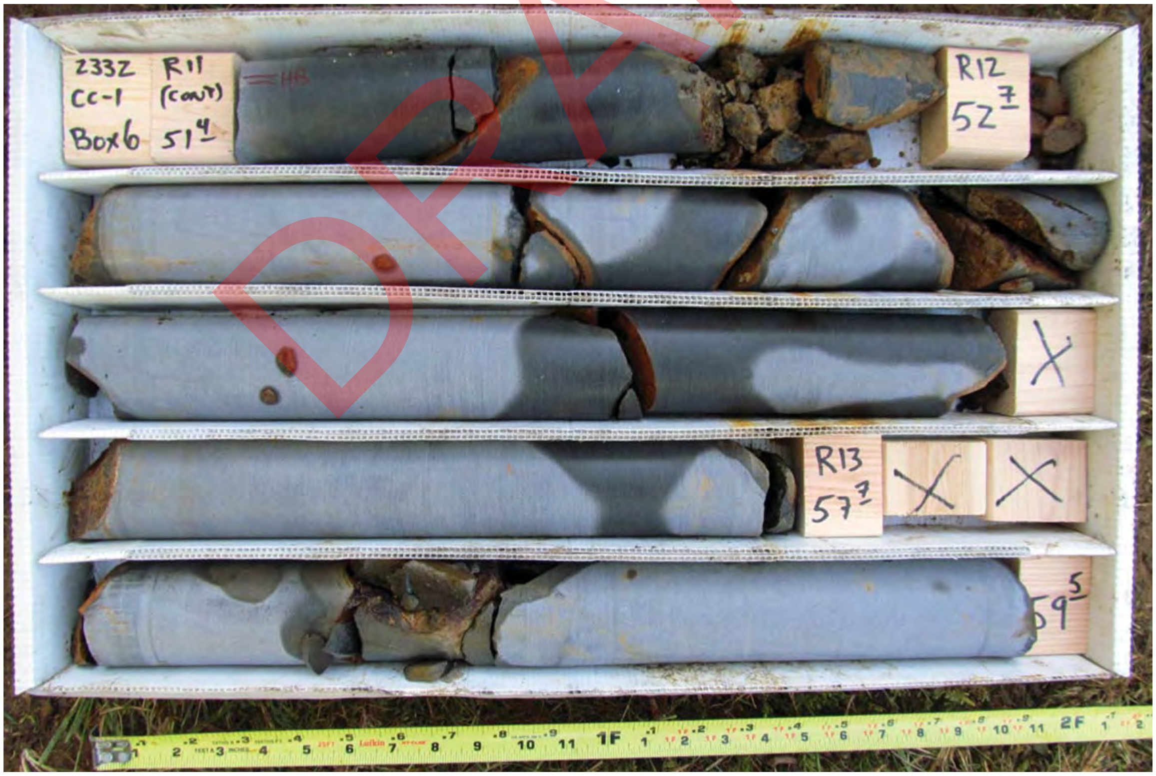
FIG. 5

BOX 5 - 42.1 TO 51.4 FT



BOX 6 - 51.4 TO 59.5 FT

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CORE BOX PHOTOS
CC-1 (3 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
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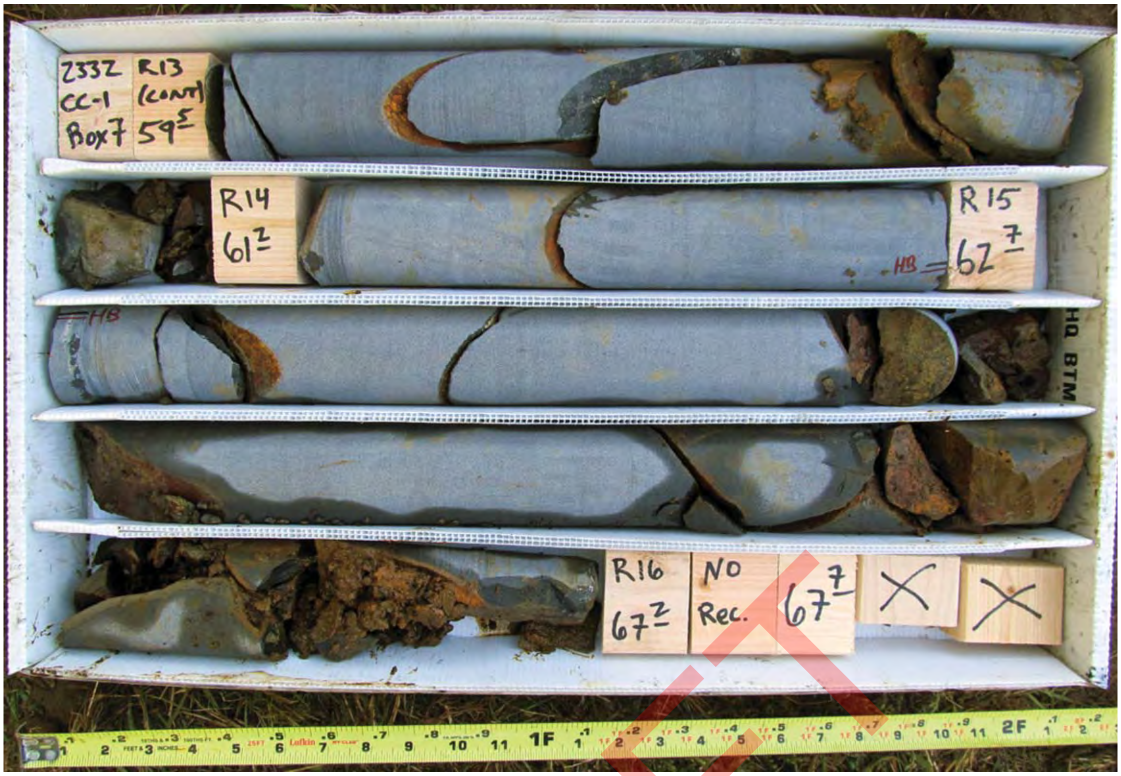
FIG. 5

2332/CC-1.ai NAU



BOX 7 - 59.5 TO 67.7 FT.

BOX 8 - 67.7 TO 76.0 FT

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CORE BOX PHOTOS
CC-1 (4 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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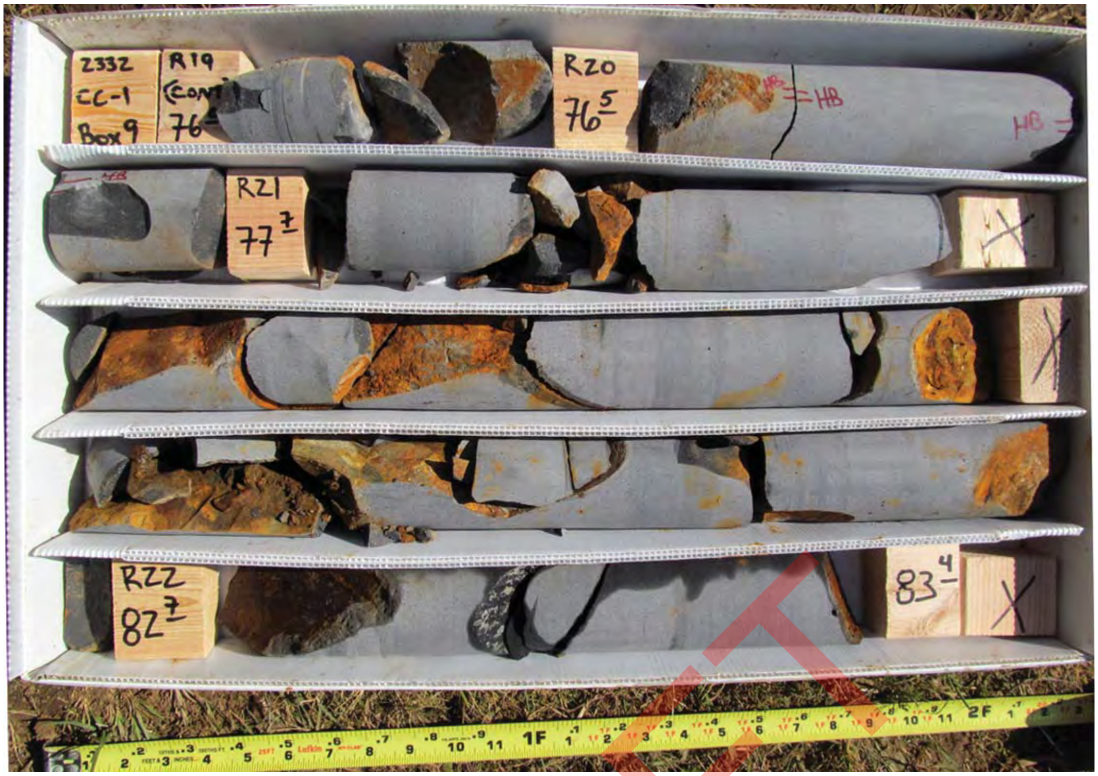
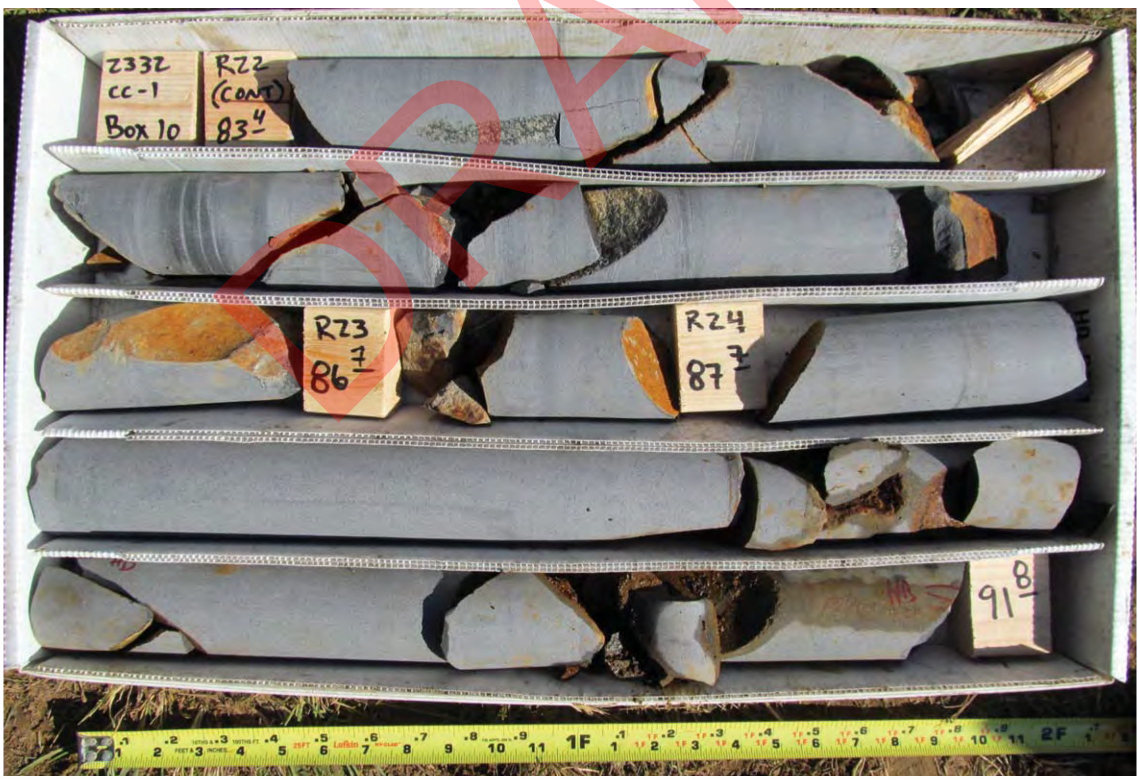
FIG. 5

2332/CC-1.ai NAU



BOX 9 - 76.0 TO 83.4 FT



BOX 10 - 83.4 TO 91.8 FT

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2332/CC-1.ai NAU

CORE BOX PHOTOS
CC-1 (5 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
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FIG. 5

BOX 11 - 91.8 TO 97.7 FT

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CORE BOX PHOTOS
CC-1 (6 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
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FIG. 5

2332/CC-1.ai NAU

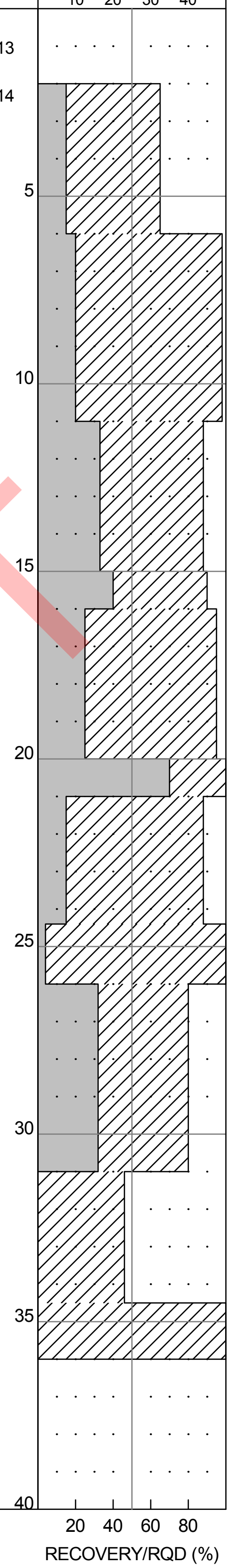
ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 587.8 FT.	RUN NO.
		Quick Drill to 2.0 feet to accommodate HQ3 core barrel - No Samples Taken	
585.8	2	DENSE, gray, silty, sand- to gravel-sized ROCK FRAGMENTS (ROAD BASE AGGREGATE)	R-1
583.2	4.6	HARD to VERY HARD (R4-R5), gray, slightly weathered BASALT; very highly to highly jointed, dominant 30 to 50° (smooth, planar) numerous 0 to 20° and 70 to 90° joints, occasional clayey SILT infilling along joints up to 1/2-inch thick, abundant iron staining and black mineralization along joints up to 1-inch thick, occasional zones of diced texture up to 1/2-inch thick, MEDIUM (R3), weathered surfaces on joints up to 1/2" thick, trace cement-grout infilling in joints up to 1/4-inch thick (COLUMBIA RIVER BASALT?) … ¼-inch of cement-grout infilling in joint at 7.3 feet … ½-inch of diced texture at 8.7 feet	R-2 R-3
		… trace cement-grout infilling in joints from 17.0 to 18.5 feet	R-4 R-5 R-6 R-7 R-8 R-9
		… driller notes color change in return from gray to brown, drill rods binding and drilling feels "gravelly" from 33.0 to 34.5 feet	R-10 R-11
551.8	36	Bottom of Boring: 36 FT	

SAMPLE: NO. | PEN. DATA

GROUND WATER/ INSTRUMENT INSTALLATION

10/17/13

01/15/14



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- STANDARD PENETRATION TEST (BLOWS/FT.)
- WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 33.5 FT (S/N: 1326218)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/16/2013 FINISH: 10/17/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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SUMMARY BORING LOG CC-2

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

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FIG. 6

BOX 1 - 2.0 TO 12.0 FT

BOX 2 - 12.0 TO 21.0 FT

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CORE BOX PHOTOS
CC-2 (1 OF 2)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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FIG. 7

2332/CC-2.ai NAU

BOX 3 - 21.0 TO 31.0 FT

BOX 4 - 31.0 TO 36.0 FT

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2332/CC-2.ai NAU

CORE BOX PHOTOS CC-2 (2 OF 2)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

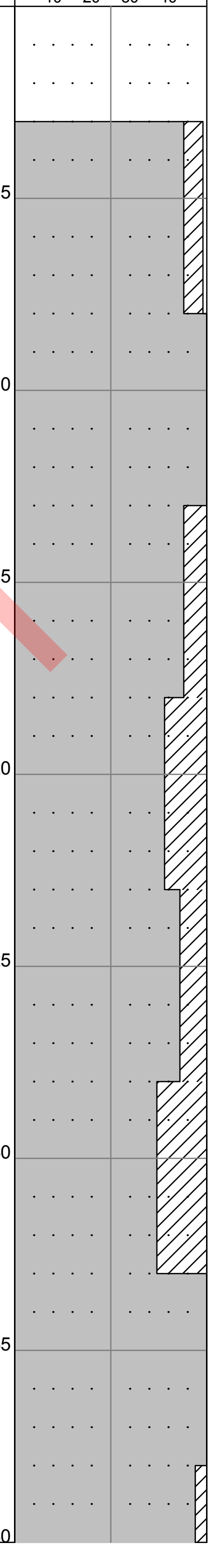
MAR 2014

PROJ. 2332

FIG. 7

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.1 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION
		Quick Drill to 3.0 feet to accommodate HQ3 core barrel - No Samples Taken				
662.1	3	Light gray CONCRETE; highly to moderately fractured, with scattered very highly fractured zones, dominant 0 to 20° (rough, planar), sand- to gravel-sized rounded river rock aggregate, scattered air voids up to 3/8-inch diameter, trace grass and woody debris (RAISED DAM SECTION)	R-1, R-2, R-3			
649.2	15.9	Light gray CONCRETE; highly to moderately fractured, with scattered very highly fractured zones, dominant 0 to 20° (rough, planar), sand- to cobble-sized angular basalt aggregate up to 5-inches maximum dimension, trace air voids up to ½-inch diameter, trace woody debris, typically concrete cold joints/lift lines are subtle and tight (ORIGINAL DAM SECTION) ... concrete cold joint/lift line at 16.6 feet ... concrete cold joint/lift line at 17.5 feet ... concrete cold joint/lift line at 20.3 feet ... concrete cold joint/lift line at 23.2 feet ... concrete cold joint/lift line at 27.2 feet ... concrete cold joint/lift line at 28.9 feet ... concrete cold joint/lift line at 30.8 feet ... concrete cold joint/lift line at 34.9 feet ... concrete cold joint/lift line at 36.7 feet ... concrete cold joint/lift line at 37.5 feet ... concrete cold joint/lift line at 38.5 feet ... concrete cold joint/lift line at 39.4 feet	R-3, R-4, R-5, R-6, R-7, R-8			10/16/13
625.1	40					

PENETRATION TEST (BLOWS PER FOOT)
WATER CONTENT (%)
10 20 30 40



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- STANDARD PENETRATION TEST (BLOWS/FT.)
- WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 102.0 FT (S/N: 1326214)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

DRAFT

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/9/2013 FINISH: 10/11/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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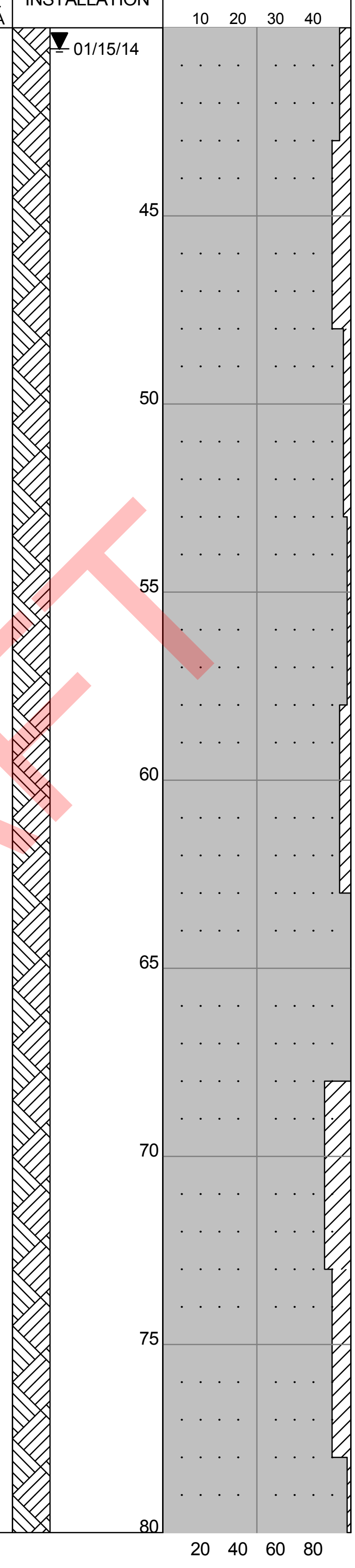
SUMMARY BORING LOG CC-3 (1 of 4)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 8

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.1 FT.	RUN NO.	SAMPLE NO.	SAMPLE PEN. DATA
		(continued from previous page) ... concrete cold joint/lift line at 40.5 feet ... concrete cold joint/lift line at 41.3 feet ... concrete cold joint/lift line at 42.2 feet	R-8		
		... concrete cold joint/lift line and woody debris/organics at 43.2 feet ... concrete cold joint/lift line at 45.2 feet ... concrete cold joint/lift line at 46.3 feet … circulation color changes from gray to brown at 47.0 to 47.8 feet, with trace sandy SILT infilling on fracture at 47.4 feet	R-9		
		... concrete cold joint/lift line at 49.1 feet ... concrete cold joint/lift line at 49.9 feet ... concrete cold joint/lift line at 51.0 feet	R-10		
		... concrete cold joint/lift line at 53.6 feet ... concrete cold joint/lift line at 54.9 feet ... concrete cold joint/lift line at 56.6 feet	R-11		
		... concrete cold joint/lift line at 58.6 feet ... concrete cold joint/lift line at 60.1 feet ... concrete cold joint/lift line at 61.4 feet ... concrete cold joint/lift line at 62.4 feet	R-12		
		... concrete cold joint/lift line at 63.4 feet ... concrete cold joint/lift line at 64.3 feet ... concrete cold joint/lift line at 66.0 feet ... concrete cold joint/lift line, scattered air voids at 67.3 feet	R-13		
596.8	68.3	Light gray CONCRETE; highly to moderately fractured, with scattered very highly fractured zones, dominant 0 to 20° (rough, planar), sand- to cobble-sized angular basalt aggregate up to 4-inches maximum dimension, scattered to numerous air voids up to 2-inches diameter, (DAM CUTOFF SECTION?) ... concrete cold joint/lift line at 68.8 feet ... concrete cold joint/lift line at 71.2 feet	R-14		
		... concrete cold joint/lift line at 72.9 feet ... concrete cold joint/lift line at 74.3 feet ... concrete cold joint/lift line at 74.9 feet ... concrete cold joint/lift line at 76.0 feet ... concrete cold joint/lift line at 77.6 feet	R-15		
585.1	80		R-16		

GROUND WATER/ INSTRUMENT INSTALLATION — PENETRATION TEST (BLOWS PER FOOT) — WATER CONTENT (%)



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 102.0 FT (S/N: 1326214)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/9/2013 FINISH: 10/11/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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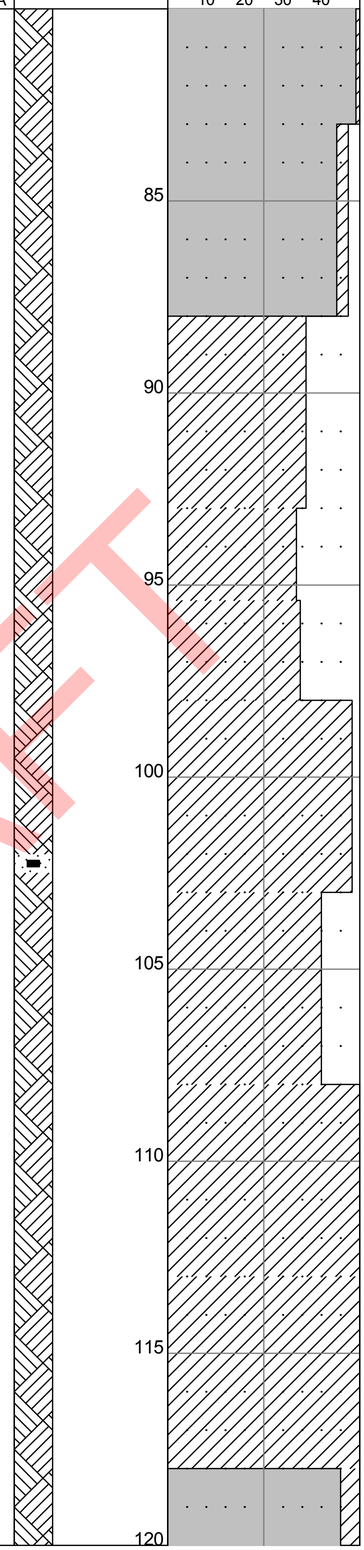
SUMMARY BORING LOG
CC-3 (2 of 4)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 8

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.1 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION
		(continued from previous page) ... concrete cold joint/lift line at 80.2 feet ... concrete cold joint/lift line at 81.5 feet	R-16			
		... concrete cold joint/lift line at 83.8 feet ... concrete cold joint/lift line at 86.8 feet	R-17			
574.1	91		R-18			
		MEDIUM HARD (R3), gray, moderately weathered BASALT; very highly jointed, (subrounded to angular gravel- to cobble-sized fragments), numerous random orientations (COLUMBIA RIVER BASALT?)	R-19			
569.1	96					
		STIFF, gray to blue-gray, slightly clayey to clayey SILT; numerous sand lenses and sand- to cobble-sized rock fragments up to 8-inches, trace diced texture, trace polished surfaces, trace mica, trace shells (MARINE SEDIMENTS)	R-20			
			R-21			
			R-22			
			R-23			
			R-24			
547.1	118					
			R-25			

PENETRATION TEST (BLOWS PER FOOT)
WATER CONTENT (%)
10 20 30 40



20 40 60 80
RECOVERY/RQD (%)

LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 102.0 FT (S/N: 1326214)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUNTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/9/2013 FINISH: 10/11/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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SUMMARY BORING LOG CC-3 (3 of 4)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 8

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.1 FT.	RUN NO.	SAMPLE NO.	SAMPLE PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION	PENETRATION TEST (BLOWS PER FOOT) WATER CONTENT (%) 10 20 30 40
542.1	123	EXTREMELY SOFT to VERY SOFT (R0-R1), gray, moderately weathered SILTSTONE; moderately fractured, 2 fractures 10 to 30° (rough, irregular), micaceous, trace shells, trace calcified angular gravel-sized fragments (ASTORIA FORMATION)	R-25				
		Bottom of Boring: 123 FT					

Depth scale: 125, 130, 135, 140, 145, 150, 155, 160

20 40 60 80
RECOVERY/RQD (%)

LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT
- WATER CONTENT
- PLASTIC LIMIT
- STANDARD PENETRATION TEST (BLOWS/FT.)
- WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 102.0 FT (S/N: 1326214)
3. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE
SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE
BOREHOLE DIAM.: 3 5/8"

DRILLER: WESTERN STATES
DATE START: 10/9/2013 FINISH: 10/11/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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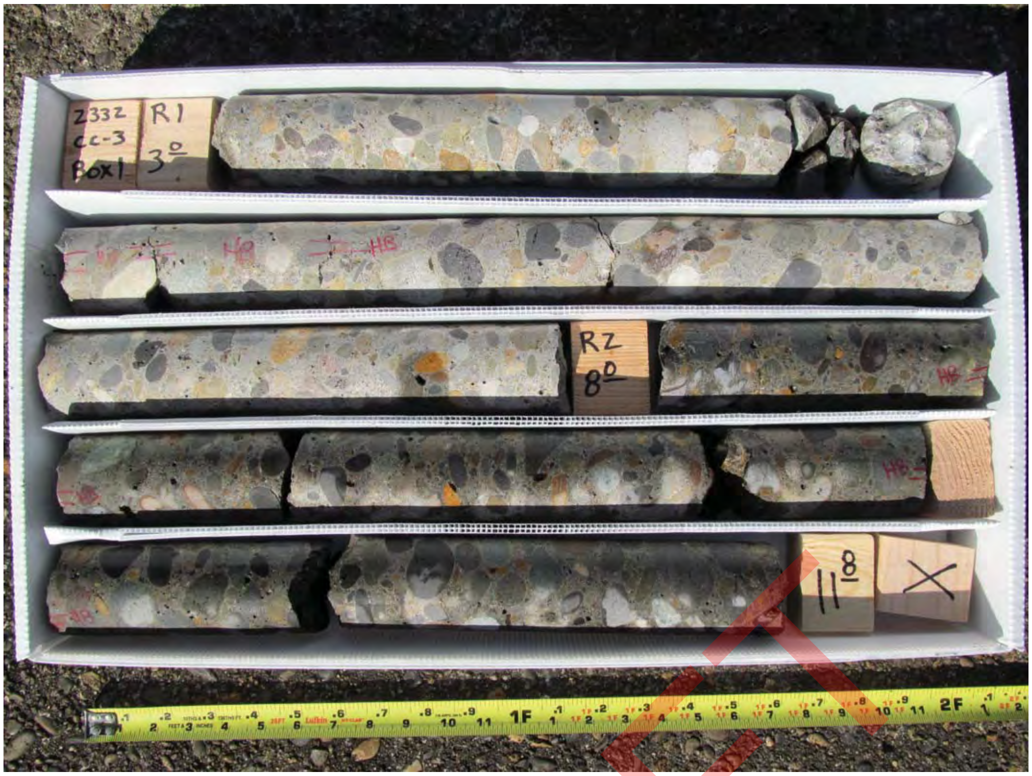
SUMMARY BORING LOG CC-3 (4 of 4)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014

PROJ 2332

FIG. 8



BOX 1 - 3.0 TO 11.8 FT

BOX 2 - 11.8 TO 19.9 FT

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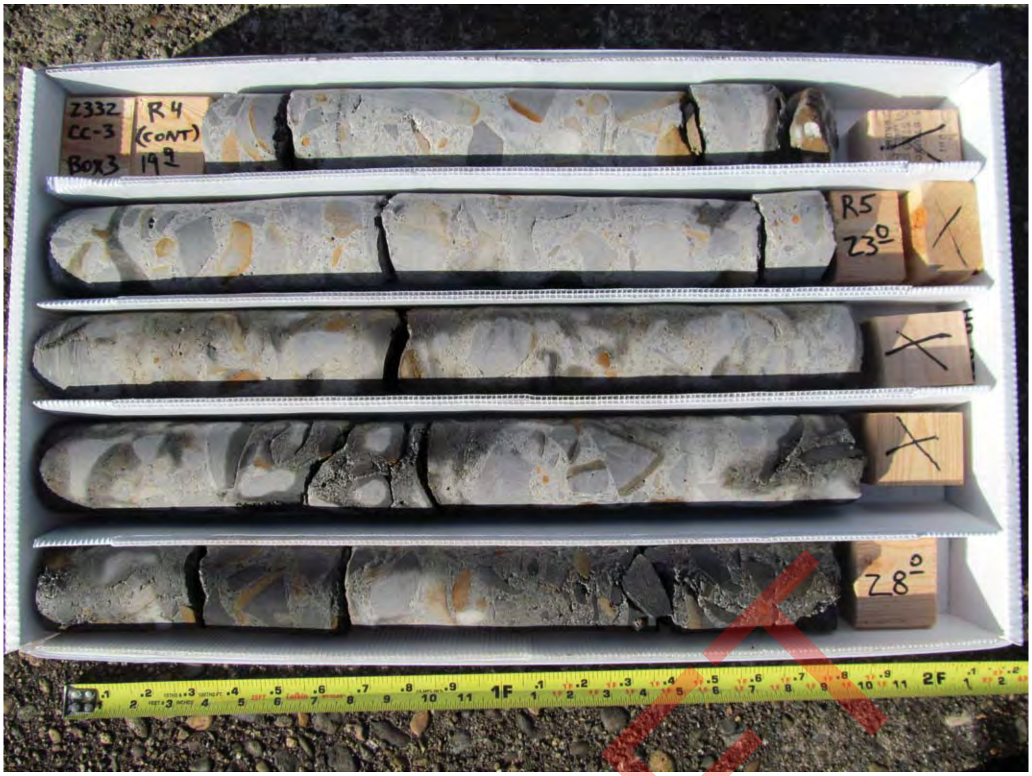
CORE BOX PHOTOS
CC-3 (1 OF 7)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

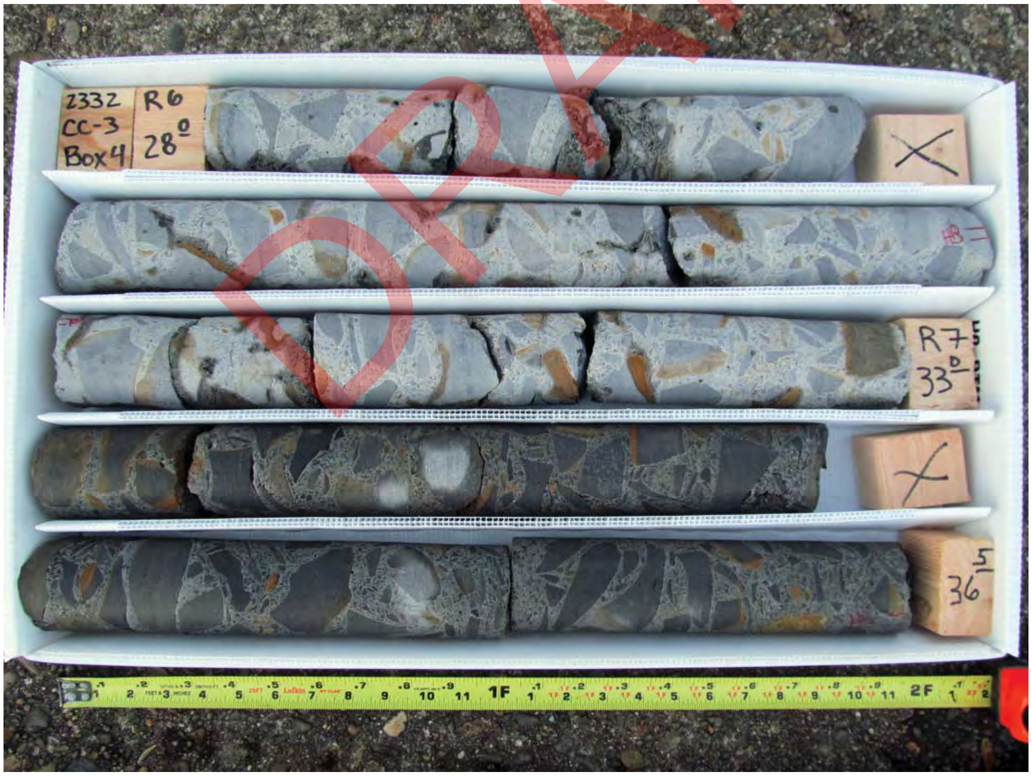
MAR 2014

PROJ. 2332

FIG. 9



BOX 3 - 19.9 TO 28.0 FT



BOX 4 - 28.0 TO 36.5 FT

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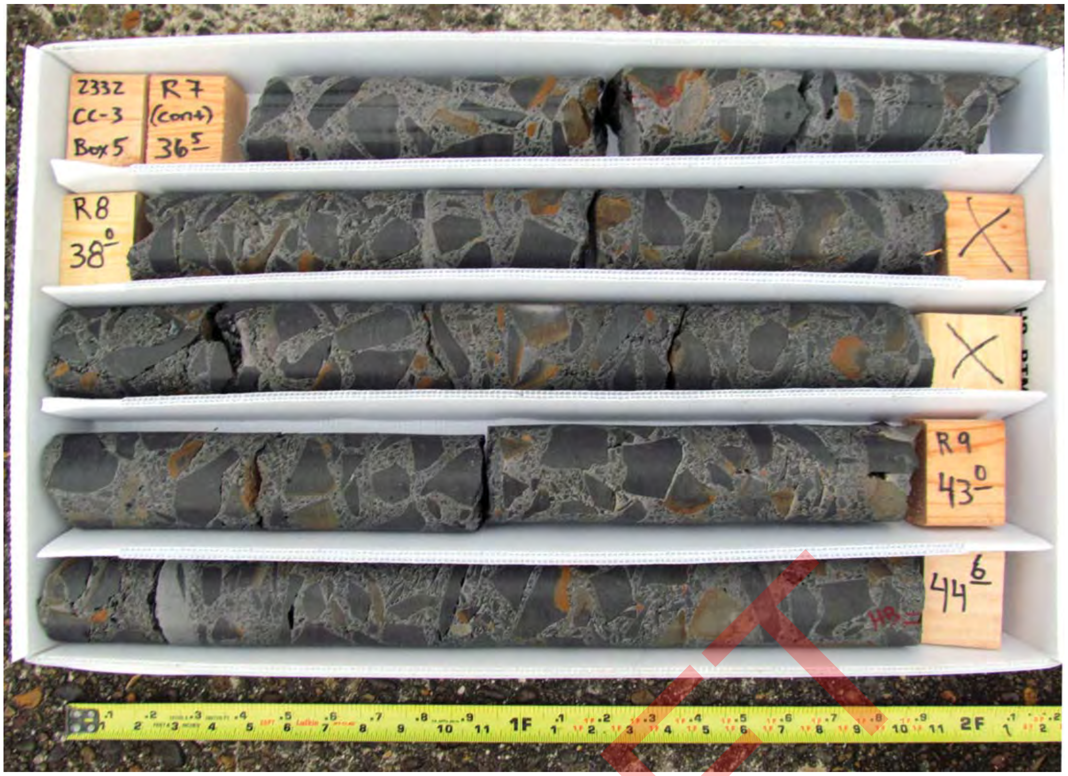
CORE BOX PHOTOS
CC-3 (2 OF 7)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

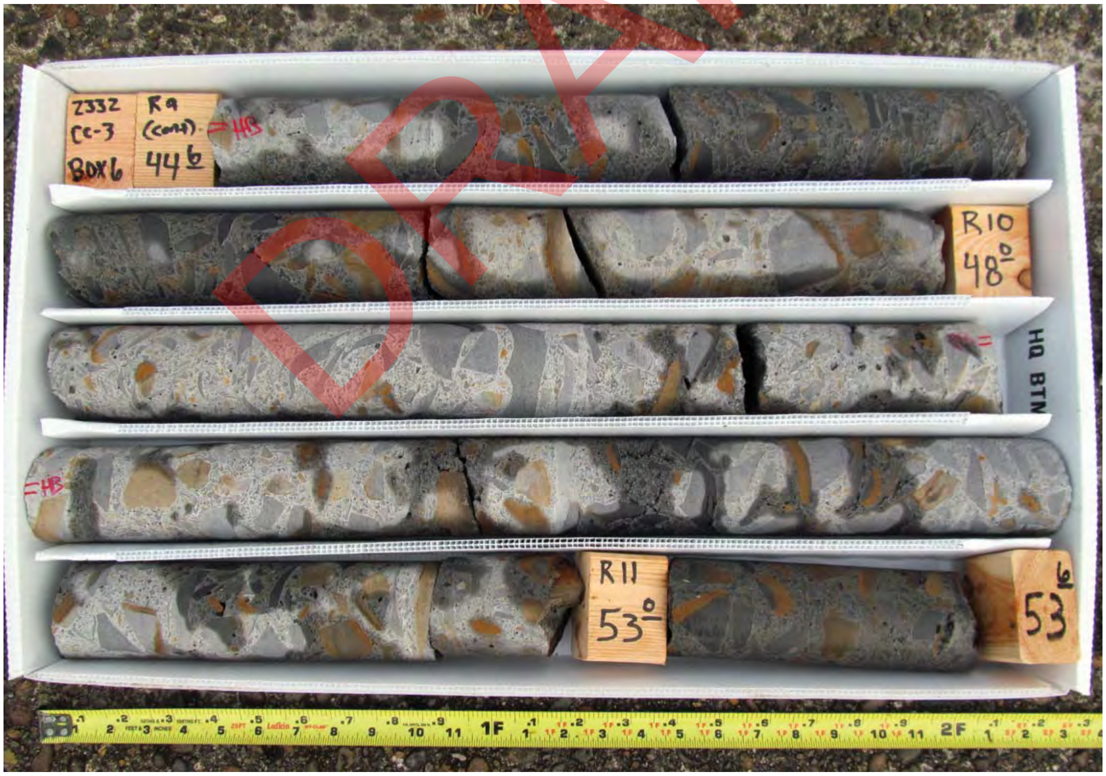
MAR 2014

PROJ. 2332

FIG. 9



BOX 5 - 36.5 TO 44.6 FT



BOX 6 - 44.6 TO 53.6 FT

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CORE BOX PHOTOS
CC-3 (3 OF 7)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 9

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BOX 7 - 53.6 TO 62.3 FT

BOX 8 - 62.3 TO 70.6 FT

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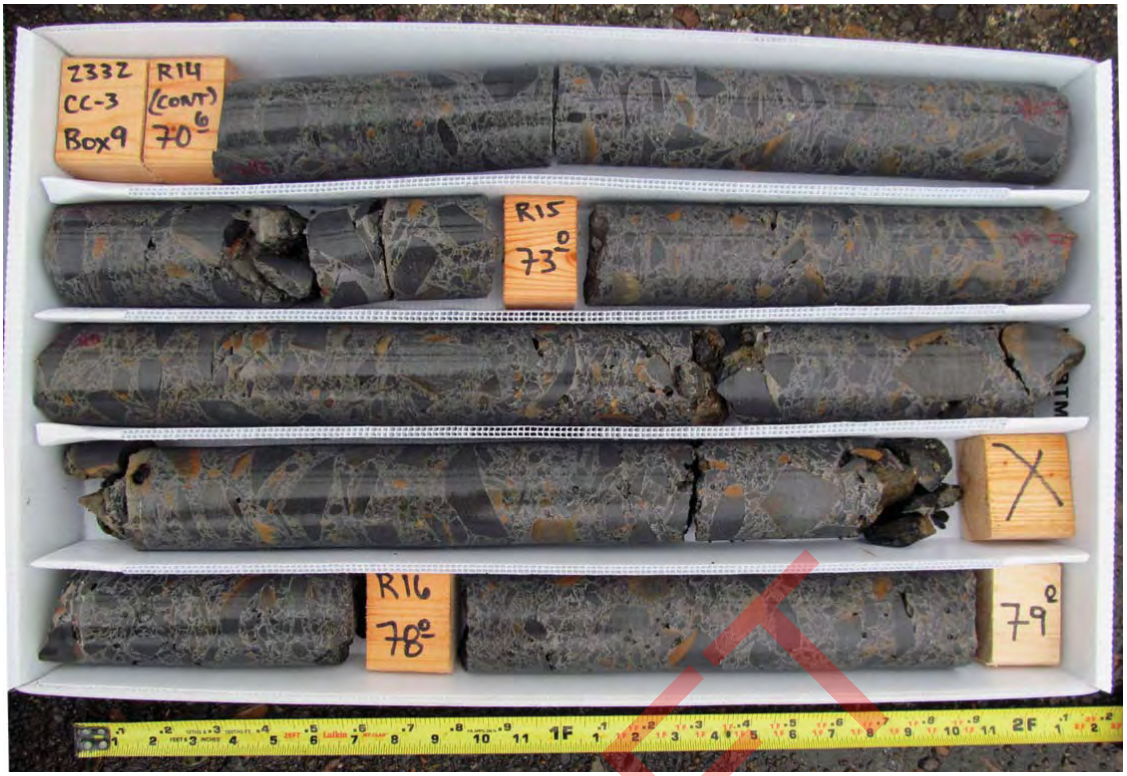
CORE BOX PHOTOS
CC-3 (4 OF 7)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 9



BOX 9 - 70.6 TO 79.0 FT

BOX 10 - 79.0 TO 88.0 FT

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CORE BOX PHOTOS
CC-3 (5 OF 7)

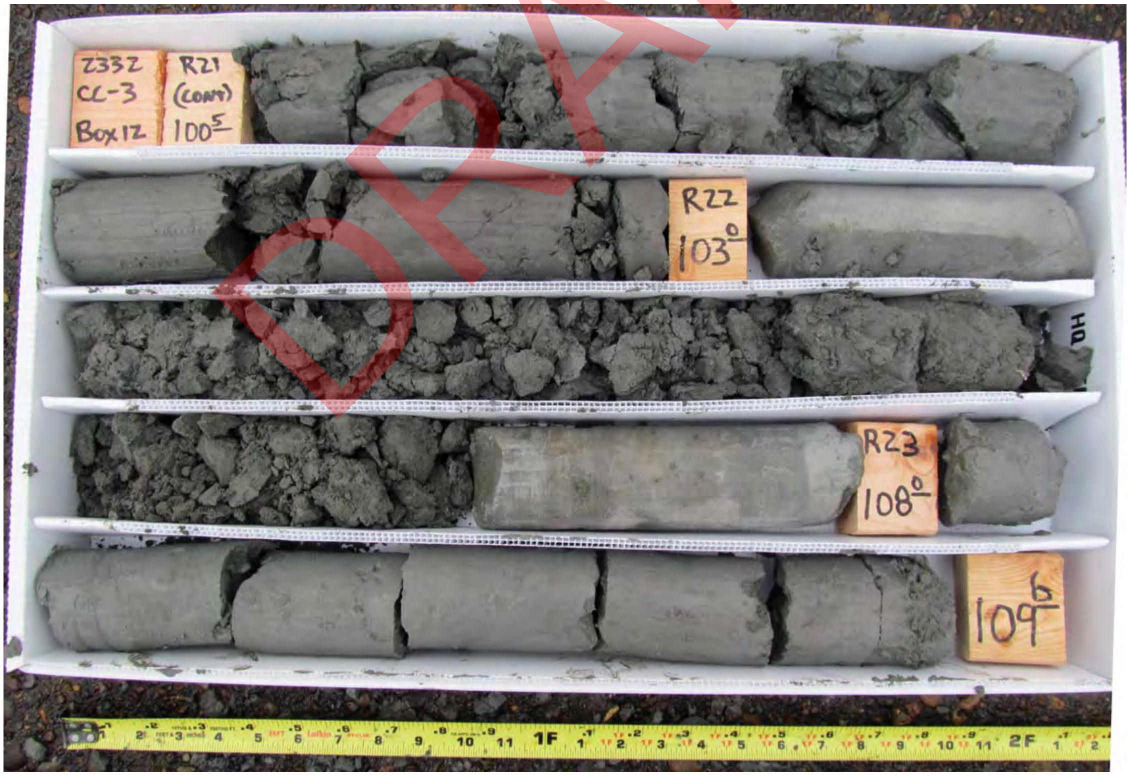
BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 9

BOX 11 - 88.0 TO 100.5 FT



BOX 12 - 100.5 TO 109.6 FT

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CORE BOX PHOTOS
CC-3 (6 OF 7)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

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FIG. 9

BOX 13 - 109.6 TO 118.0 FT

DRAFT

BOX 14 - 118.0 TO 123.0 FT

2332/CC-3.ai NAU

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CORE BOX PHOTOS
CC-3 (7 OF 7)

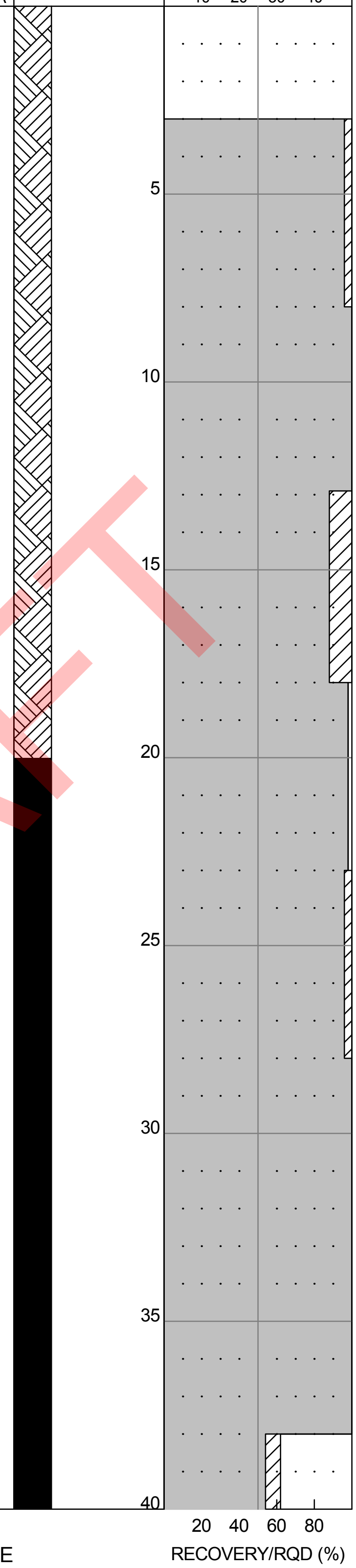
BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 9

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.0 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION	PENETRATION TEST (BLOWS PER FOOT) WATER CONTENT (%) 10 20 30 40
		Quick Drill to 3.0 feet to accommodate HQ3 core barrel - No Samples Taken					
662.0	3	Light gray CONCRETE; highly to moderately fractured, with scattered very highly fractured zones, dominant 0 to 20° (rough, planar), sand- to gravel-sized rounded river rock aggregate, scattered air voids up to 3/8-inch diameter, trace organics and woody debris, trace rebar (RAISED DAM SECTION) … 3/8-inch rebar at 3.6 feet … 1/2-inch woody debris at 5.6 feet	R-1				
		… small amount of drill water entering reservoir from 9.0 to 11.0 feet.	R-2				
		… 2-inch steel re-bar at 15.7 to 15.8 feet	R-3				
649.1	15.9	Light gray CONCRETE; highly to moderately fractured, with scattered very highly fractured zones, dominant 0 to 20° (rough, planar), sand- to cobble-sized angular basalt aggregate up to 5-inches maximum dimension, trace air voids up to ½-inch diameter, (ORIGINAL DAM SECTION)					
		... concrete cold joint/lift line at 20.4 feet ...drilling action indicates fracture, lost all circulation at 21.0 feet ... concrete cold joint/lift line at 22.4 feet ... concrete cold joint/lift line at 23.3 feet ... concrete cold joint/lift line at 24.2 feet	R-4				
		... concrete cold joint/lift line at 27.2 feet ... concrete cold joint/lift line at 28.1 feet	R-5				
		... concrete cold joint/lift line at 29.0 feet, drill water coming out of crack in downstream face of dam ... concrete cold joint/lift line at 30.8 feet ... concrete cold joint/lift line at 32.9 feet	R-6				
		... concrete cold joint/lift line at 34.2 feet ... concrete cold joint/lift line at 34.9 feet ... concrete cold joint/lift line at 35.8 feet	R-7				
625.0	40	... concrete cold joint/lift line at 37.7 feet ... concrete cold joint/lift line at 38.2 feet ... concrete cold joint/lift line at 38.7 feet	R-8				



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 96.0 FT (S/N: 1326216)
3. 20-50 FEET 50/50 MIXTURE OF BENTONITE AND SILICA SAND
4. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 10/7/2013 FINISH: 10/8/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

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SUMMARY BORING LOG CC-4 (1 of 3)

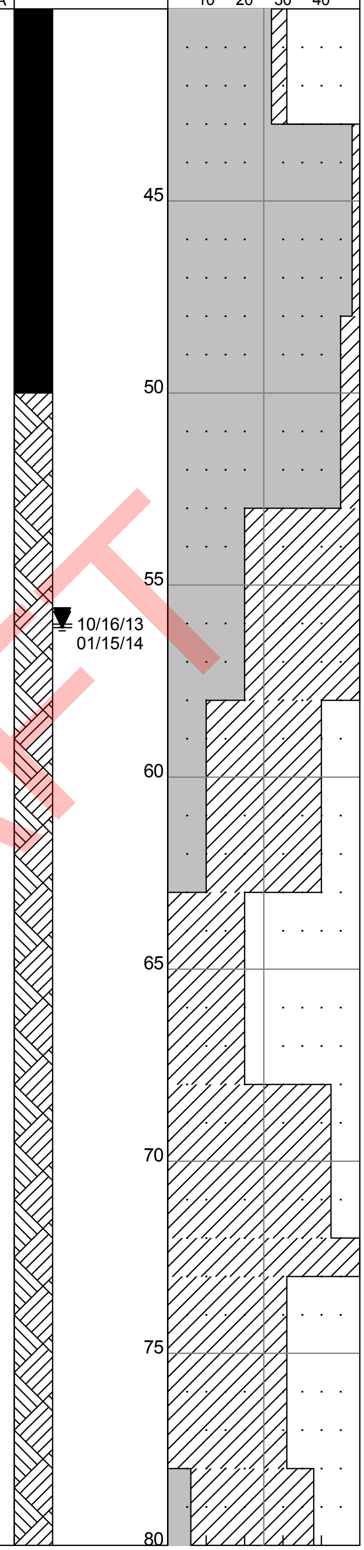
BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014

PROJ 2332

FIG. 10

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.0 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION	PENETRATION TEST (BLOWS PER FOOT) WATER CONTENT (%) 10 20 30 40
		(continued from previous page) ... concrete cold joint/lift line at 40.1 feet ... drill rate very fast from 41.5 to 43.0 feet	R-8				
		... zone of LOOSE, brown, sandy gravel-sized ROCK FRAGMENTS in a matrix of SOFT, brown, slightly clayey SILT from 43.0 to 43.2 feet	R-9				45
		... concrete cold joint/lift line at 49.0 feet ... concrete cold joint/lift line at 50.6 feet ... concrete cold joint/lift line at 52.0 feet	R-10				50
610.0	55	HARD to VERY HARD (R4-R5), gray, slightly weathered BASALT; very highly to highly jointed, dominant 30 to 50° (smooth, planar), numerous 0 to 20° and 70 to 90° joints, occasional clayey SILT infilling along joints up to 1/2-inch thick, medium hard (R3) weathered surfaces up to 1-inch thick, abundant iron staining and black mineralization along joints up to 1-inch thick, abundant cement-grout infilling in joints up to 6-inches thick (COLUMBIA RIVER BASALT?)	R-11			10/16/13 01/15/14	55
			R-12				60
		... cement-grout is not observed in fractures below 65.0 feet	R-13				65
			R-14				70
			R-15				
			R-16				75
585.0	80		R-17				80



20 40 60 80
RECOVERY/RQD (%)

LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 96.0 FT (S/N: 1326216)
3. 20-50 FEET 50/50 MIXTURE OF BENTONITE AND SILICA SAND
4. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE   SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE   BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 10/7/2013 FINISH: 10/8/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

CORNFORTH CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

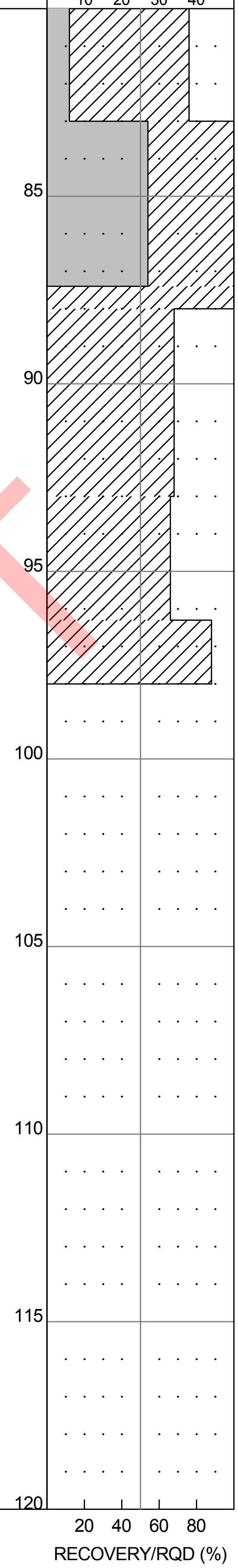
SUMMARY BORING LOG CC-4 (2 of 3)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 10

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 665.0 FT.	RUN NO.	SAMPLE NO.	PEN. DATA	GROUND WATER/ INSTRUMENT INSTALLATION
		(continued from previous page)	R-17			
			R-18			
			R-19			
			R-20			
			R-21			
567.0	98		R-22			
		Bottom of Boring: 98 FT				

PENETRATION TEST (BLOWS PER FOOT)
WATER CONTENT (%)



LEGEND

- 2-INCH O.D. SPLIT SPOON
- 3-INCH O.D. SPLIT SPOON
- 3-INCH O.D. THIN WALL SAMPLER
- 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- STANDARD PENETRATION TEST (BLOWS/FT.)
- WATER CONTENT IN PERCENT
- CORE RECOVERY IN PERCENT
- RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. VIBRATING WIRE PIEZOMETER INSTALLED ON OUTSIDE OF SACRIFICIAL 1" SCHEDULE 40 PVC IN GROUT AT 96.0 FT (S/N: 1326216)
3. 20-50 FEET 50/50 MIXTURE OF BENTONITE AND SILICA SAND
4. DISCONTIUITY ANGLES MEASURED ORTHOGONAL TO CORE AXIS. FOR A VERTICLE BORING, A HORIZONTAL DISCOUTINUITY MEASURES 0°.

HAMMER ASSEMBLY: NOT APPLICABLE SPT SAMPLER: NOT APPLICABLE
DRILL ROD USED: NOT APPLICABLE BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 10/7/2013 FINISH: 10/8/2013
DRILLING TECHNIQUE: MUD ROTARY / HQ3 CORING

CORNFORTH CONSULTANTS
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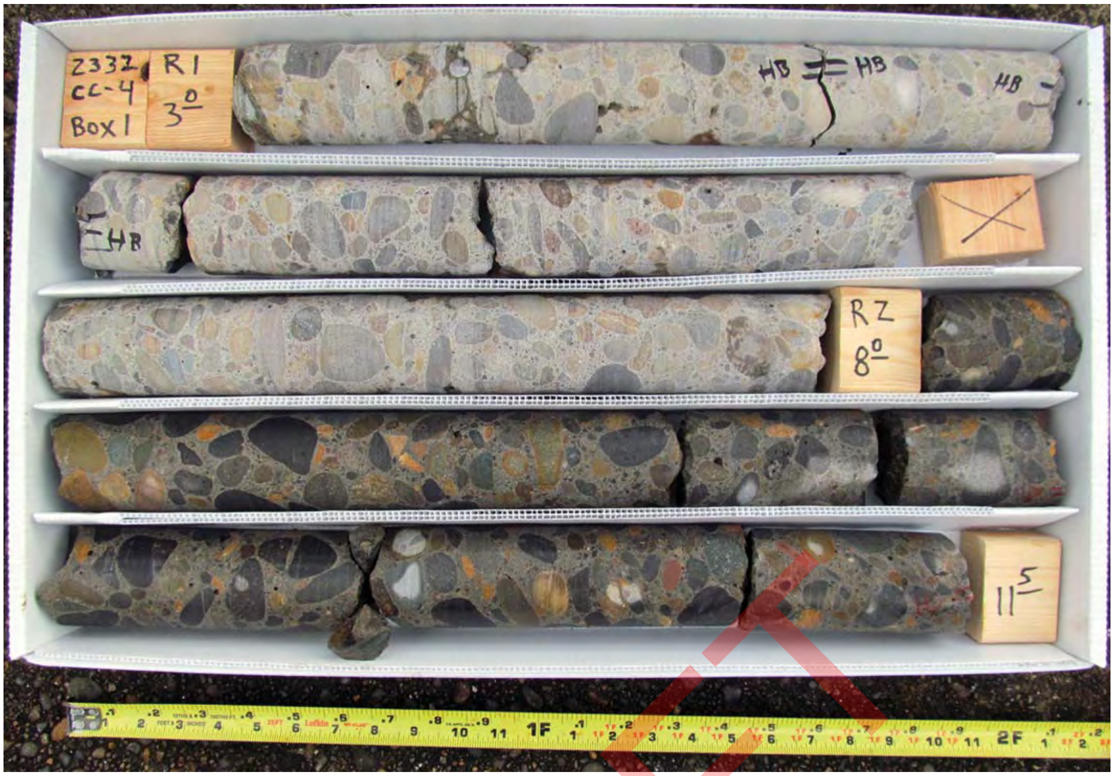
SUMMARY BORING LOG CC-4 (3 of 3)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OR

MAR 2014
PROJ 2332
FIG. 10



BOX 1 - 3.0 TO 11.5 FT

BOX 2 - 11.5 TO 20.2 FT

CORNFORTH
CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

2332/CC-4.ai NAU

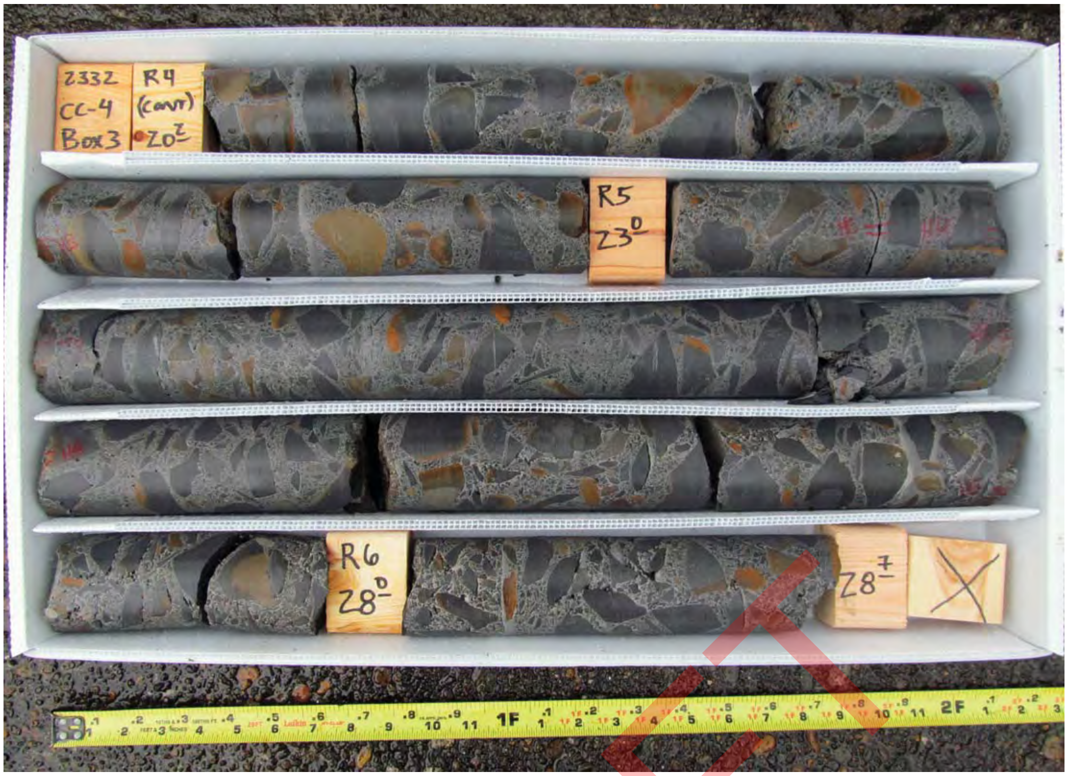
CORE BOX PHOTOS
CC-4 (1 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

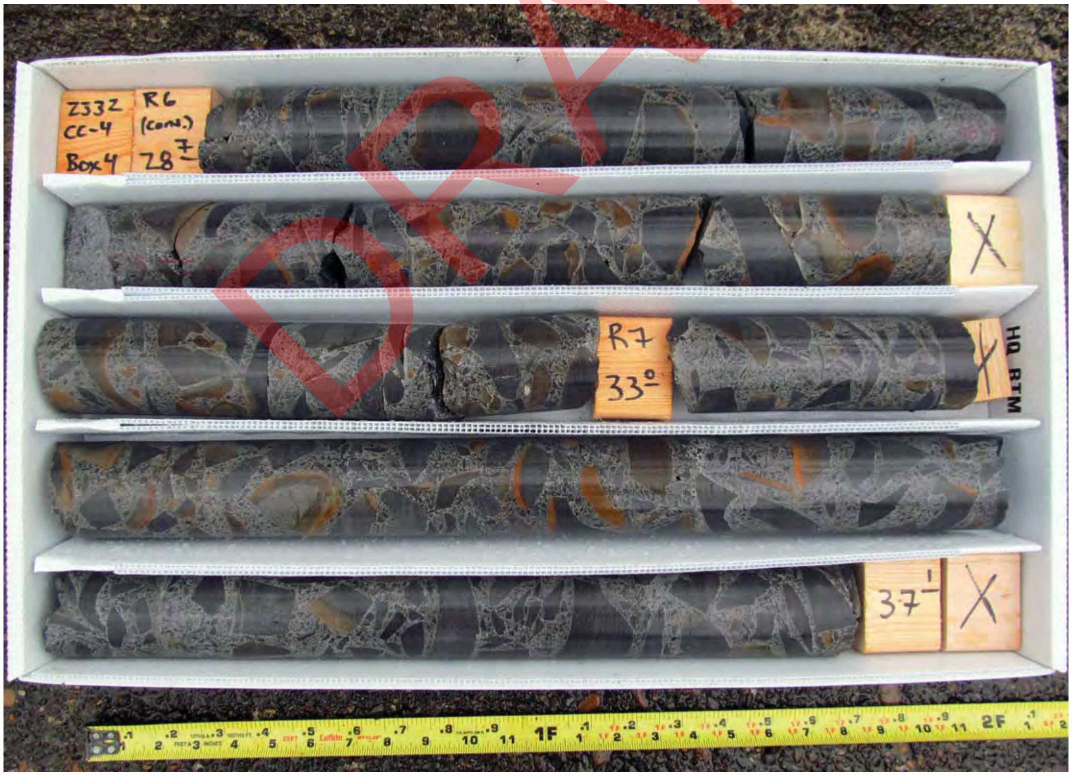
MAR 2014

PROJ. 2332

FIG. 11



BOX 3 - 20.2 TO 28.7 FT



BOX 4 - 28.7 TO 37.1 FT

CORNFORTH
CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

2332/CC-4.ai NAU

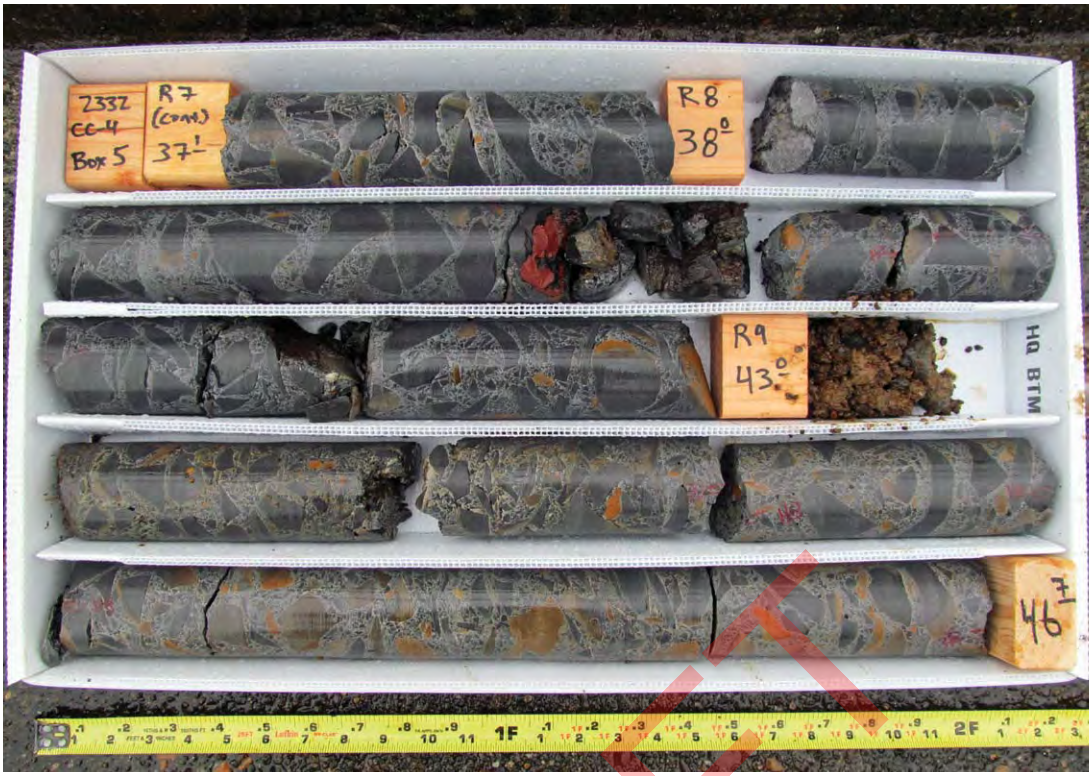
CORE BOX PHOTOS
CC-4 (2 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

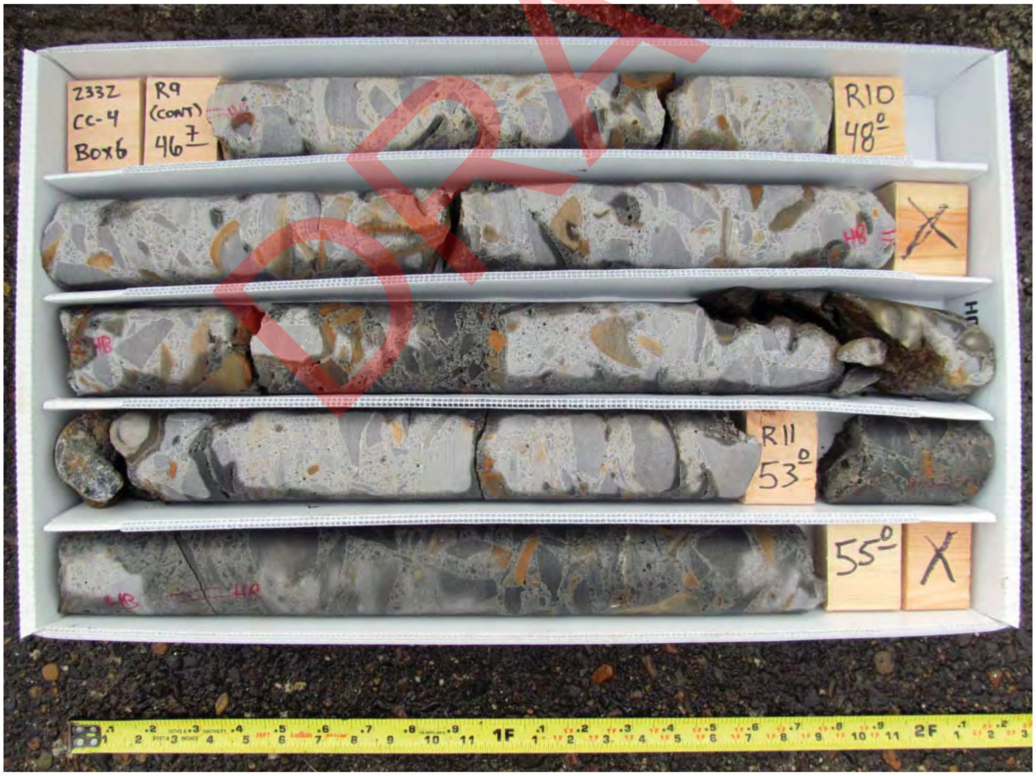
MAR 2014

PROJ. 2332

FIG. 11



BOX 5 - 37.1 TO 46.7 FT



BOX 4 - 46.7 TO 55.0 FT

CORNFORTH
CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

2332/CC-4.ai NAU

CORE BOX PHOTOS
CC-4 (3 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 11

BOX 7 - 55.0 TO 64.3 FT

BOX 8 - 64.3 TO 78.0 FT

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CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

CORE BOX PHOTOS
CC-4 (4 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

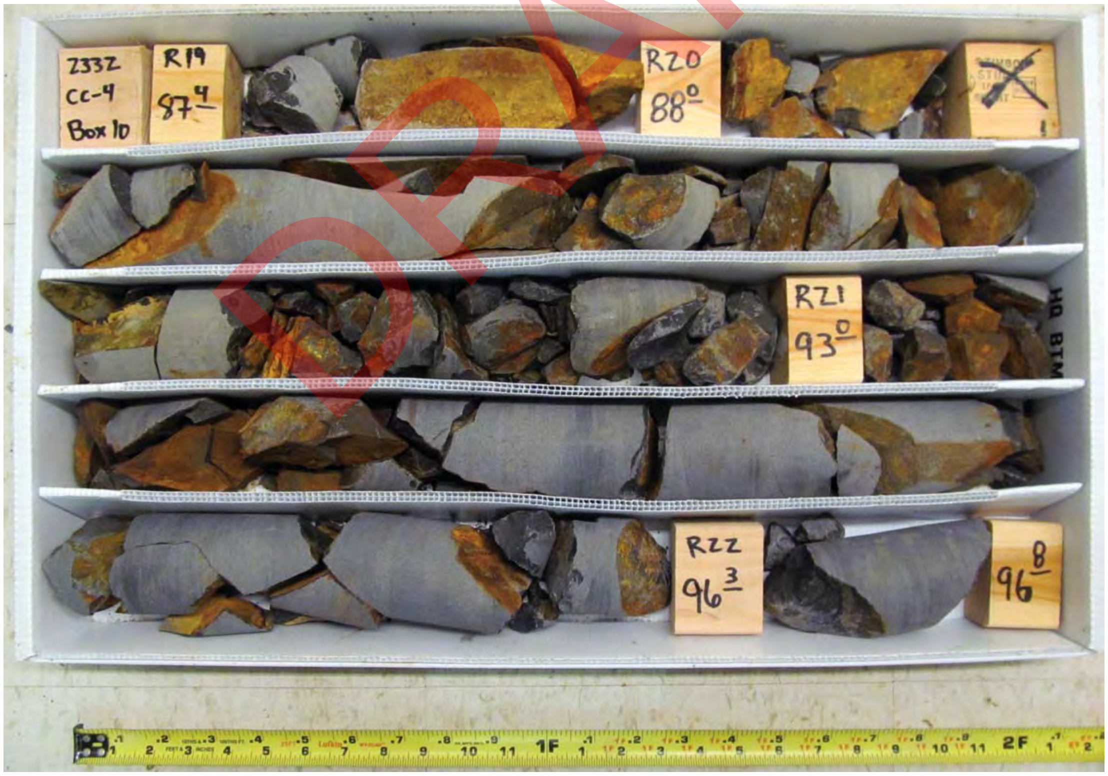
MAR 2014

PROJ. 2332

FIG. 11

2332/CC-4.ai NAU

BOX 9 - 78.0 TO 87.4 FT



BOX 10 - 87.4 TO 96.8 FT

CORNFORTH
CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

CORE BOX PHOTOS
CC-4 (5 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 11

2332/CC-4.ai NAU

BOX 11 - 96.8 TO 98.0 FT

CORNFORTH CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

2332/CC-4.ai NAU

CORE BOX PHOTOS
CC-4 (6 OF 6)

BEAR CREEK DAM SEISMIC STABILITY
ASTORIA, OREGON

MAR 2014

PROJ. 2332

FIG. 11

DRILLING METHOD	NX Wireline Core	SURFACE ELEVATION	665.0 feet	LOGGED BY	NTD
DEPTH TO GROUNDWATER	42.0 feet	BORING DIAMETER	1.75-inch	DATE DRILLED	12/11/92

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	DEPTH (FEET)	RECOVERY (%)	RQD (%)	REMARKS and OTHER TESTS
Concrete. aggregates are rounded and oversized	5	100	95	No core barrel samples to a depth of 5 feet due to clearance restriction. Drilling water observed on the upstream side
	10	100	100	
Good bonding between new (addition) and old concrete. aggregates are angular and oversized Aggregates oxidized and soft.	15	100	93	
Piece of wood at 23'7".	20	100	88	
26'10" to 27'highly fractured and porous. Seepage zone?	25	100	80	
	30	100	92	
At 35' highly oxidized aggregates. 37'1" to 37'4" highly fractured.	35	100	83	
At 41'6" fracture with crushed aggregates.	40	100	95	
	45	100	80	
	50	100	85	
	55	100	90	
	60	100	98	
1" fracture, poor aggregate-cement bonding 64'9" to 65'1" highly fractured	65	100	85	
72' to 72'4" highly fractured	70	87	48	
73' to 73'4" highly fractured	75	50	13	Loss of water circulation 2 feet of caving and heaving
Foundation contact at 73'4". 8 inches of rock sample lost during sampling. Material consists of highly crushed Basalt rock in a silty sandy matrix. 76' to 78', Basalt is highly fractured. 79' to 80', Basalt is very hard. 80' to 83', Basalt is highly fractured.	80	85	63	Falling Head Permeability k = 1.0 cm/sec
Bottom of borehole at 83 feet.				

HARZA NORTHWEST, INC.
Consulting Engineers & Scientists

EXPLORATORY BORING LOG

BEAR CREEK DAM
ASTORIA, OREGON

PROJECT NO.	DATE	BORING NO
7067G	August 27, 1993	B-1

DRILLING METHOD	NX Wireline Core	SURFACE ELEVATION	665.0 feet	LOGGED BY	NTD
DEPTH TO GROUNDWATER	48.0 feet	BORING DIAMETER	1.75-inch	DATE DRILLED	12/13/92

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	DEPTH (FEET)	RECOVERY (%)	RQD (%)	REMARKS and OTHER TESTS
Concrete aggregates are rounded and oversized Fractures in the upper 8 feet were the result of sample extrusion. most fractures appear at the aggregate/cement contact - poor bonding with the oversized aggregates. Contact between new and old section. aggregates are angular and oversized Fractures (3) 2-inches below contact, oxidized basalt aggregates At 21'5" crushed aggragates, oxidized, can scrape with knife. 23' to 24' highly fractured. 26'11" to 27'4" highly fractured.	5			No core barrel samples to a depth of 6 feet due to clearance restriction.
	10	100	95	
	15	100	93	
	20	100	96	
	25	100	62	
	30	100	67	Loss of all drilling water after core barrel was pulled out.
	35	100	35	
Foundation contact at 36'4". 7-inches of very hard BASALT, immediately below concrete. 12-inches highly fractured and oxidized basalt	40			Loss of water circulation. Falling Head Permeability k = 1.0 cm/sec
Concrete, from 37'10" to 41'3"	45	100	0	
		100	61	
BASALT, very poor bonding between basalt and overlying concrete Top 4 to 6 inches highly fractured and altered. 18 inches of fair to poor rock 43' to 46' highly fractured (crushed) basalt fractures oriented at 45 degrees fractures and joints are filled with calcium carbonate solution (reacts with (HCL). Basalt is oxidized along fractures. 51' to 54' basalt is very hard, unaltered, with fair to good quality.	50	88	76	
		100	69	
Bottom of borehole at 54 feet				

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EXPLORATORY BORING LOG

BEAR CREEK DAM
ASTORIA, OREGON

PROJECT NO.	DATE	BORING NO
7067G	August 27, 1993	B-2

DRILLING METHOD NX Wireline Core	SURFACE ELEVATION 665.0 feet	LOGGED BY NTD
DEPTH TO GROUNDWATER 5.0 feet	BORING DIAMETER 1.75-inch	DATE DRILLED 12/15/92

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	SOIL TYPE	DEPTH (FEET)	SAMPLE	RECOVERY (%)	RQD (%)	REMARKS and OTHER TESTS	WELL CONSTRUCTION
Concrete aggregates are rounded and oversized. fractures appear at cement - aggregate interface. ~1/2 -inch diameter piece of wood. fracture at same depth. 1/2-inch crack, may be connected to the crack on the downstream face at the same elevation. Contact between new and old concrete aggregates in old concrete are angular and oversized 17' to 17'4" highly fractured, crushed and oxidized basalt aggregates		5				No core barrel samples to a depth of 5 feet due to clearance retriction.	
		10		100	100		
		15		100	90	At 12 feet added dye to drilling water. Dye observed from crack on downstream face at El.653 feet.	
		20		100	100		
		25		100	0		
Sandstone and Basalt rock fragments, highly fractured, all pieces less than 4-inches in length. very poor rock / concrete contact. alternating sandstone and basalt fragments rock is crushed between 35 and 36 feet		30		35	0	Loss of water circulation. Could not fill boring with water. Loss of 27 foot water column in 30 seconds. Switched to low viscosity mud drilling. Loss of mud circulation. Drilling mud and grease observed bubbling on the upstream side. Thin grease sheen formed on the reservoir water surface. 2 feet of caving (34 to 36 feet).	
				50	0		
		35		80	0		
Bottom of borehole at 36.5 feet							

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EXPLORATORY BORING LOG

BEAR CREEK DAM
ASTORIA, OREGON

PROJECT NO.	DATE	BORING NO
7067G	August 27, 1993	B-3

DRILLING METHOD NX Wireline Coring	SURFACE ELEVATION 580.0 feet	LOGGED BY NTD
DEPTH TO GROUNDWATER 2.5 feet	BORING DIAMETER 1.75-inch	DATE DRILLED 5/18/93

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SOIL TYPE	DEPTH (FEET)	SAMPLE	RECOVERY (%)	RQD (%)	REMARKS and OTHER TESTS	WELL CONSTRUCTION
4 inches of concrete, angular basalt aggregates Basalt rock fragments, some sandstone fragments, part of the spillway channel lining. 7 to 8 feet, crushed basalt < 1-inch size		5				Coring with 3.5-inch bit. Borehole walls caving. Used powdered bentonite to stabilize caving.	
5 inches dark green, decomposed and soft clayey sandstone 8.5 to 12.5 feet, blue green, soft clayey medium to coarse sand, black staining, and thin layers of blue/green clayey silt and fine sand.		10		60	47	2 feet of caving (2 to 4 feet). Installed 5-foot 3.5-inch casing (0 to 5 feet).	
12.5 to 13 feet, blue / green soft, fine grained sandstone to siltstone with very thin layers of clay. at 13 feet, 2-inch layer of dense, medium to fine sand. 4-inch layer of hard sandstone. at 15 feet, 3-inch layer of hard sandstone		15		80	19	Coring with 2.5-inch bit. Caving at 7 feet.	
15 to 20 feet, very soft sandstone (more like a dense sand) 20 to 23 feet, hard sandstone layers of loose sand at 20.8 and 21.4 feet.		20		92	45	Switched to low-viscisity drilling mud (EZ mud).	
at 23 feet, 2-inch layer of sandstone 4-inch layer of grey / dark brown clayey sand, some decomposed organic debris and fine rock fragments. 5-inch layer of sandstone decomposed soft, grey blue sandstone with layers of clayey fine sand		25		60	0		
28 to 30 feet, most sample washed out (6 inches recovered), probably sand.				38	0		
5.5 inches grey blue hard sandstone 30 to 31 feet, highly fractured sandstone, < 2 inches in size. Fractured sandstone with thin layers of clayey fine sand.		30		100	39	Core barrel plugged at 30 feet. Water pressure rose to 200 psi.	
Bottom of borehole at 33 feet below existing grade.							

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EXPLORATORY BORING LOG

BEAR CREEK DAM
ASTORIA, OREGON

PROJECT NO.	DATE	BORING NO
7067G	August 27, 1993	B-101

DRILLING METHOD	NX Wireline Coring	SURFACE ELEVATION	582.0 feet	LOGGED BY	NTD
DEPTH TO GROUNDWATER	2.5 feet	BORING DIAMETER	1.75-inch	DATE DRILLED	5/19/93

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS

5 inches of basalt rock fragments with concrete filling, spillway channel lining. Basalt rock fragments < 3 inches in size, loose
1.5 to 2 feet concrete with angular basalt aggregates
2 to 4 feet, hard basalt, oxidized at fractures.
6 to 8 feet, Basalt fragments, 1 to 3 inches in size, oxidized.
8 to 10 feet, fractured basalt, one 7-inch piece at 8'6" to 9'1", clay filling at 9'1"
10 to 12 feet, Fractured and decomposed basalt, fractures every 2 to 3 inches, oxidized.
12 to 13 feet, fractured basalt, fractures at 45 degrees filled with clay.
13 to 20 feet, crushed basalt, fragments varying between 2 and 6 inches, fractures filled with green / black sandy clay

20 to 22.5 feet, soft shale, fractures every 2 to 3 inches, some black clay filling.

22.5 to 24 feet, soft sandstone, bottom one-foot of core sample washed out.

Bottom of borehole at 24 feet below existing grades.

DEPTH (FEET)	RECOVERY (%)	RQD (%)
5		
	29	0
	96	57
10	95	0
	100	73
15	95	0
	100	0
20	90	0
	67	40
	90	0
	0	0

REMARKS and OTHER TESTS

Upper 5 feet cored with 3.5-inch bit.
No core barrel sampling due to clearance restriction.
Removed coring bit and rod to install 5-foot casing.
1.5 feet caving at 5 feet.
No water return. Switched to low viscosity mud.
Coring bit plugging with rock fragments and caving sand.
2-foot core runs.
Caving at 8 feet.
Caving at 10 feet.
Caving at 12 feet.
Caving at 13 feet.
Caving at 14 feet.
Caving at 16 feet.
Caving at 18 feet.
Caving at 20 feet.
Caving at 23 feet.
Total loss of circulation.
Switched to thick (higher viscosity) mud.
Drilling mud observed seeping 35 feet downslope from borehole along base of right slope.
Water pressure rose to 250 psi.
Could not core through 5-foot of caved material.

WELL CONSTRUCTION

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Consulting Engineers & Scientists

EXPLORATORY BORING LOG

BEAR CREEK DAM
ASTORIA, OREGON

PROJECT NO.	DATE	BORING NO
7067G	August 27, 1993	B-102

Appendix B

Laboratory Test Results

Earth Mechanics Institute

Client: Cornforth Consultants, Inc.

Cornforth Job No.: 2392

Project Name: Bear Creek Dam

Colorado School of Mines

Mining Engineering Department

Date: 12/01/2014 Sample ID	Rock Type	Length (in)	Average Diameter (in)	Length to Diameter Ratio	Density (lbs/ft^3)	Failure Load (lbs)	Uniaxial Compressive Strength Failure Stress σ_c (psi)	Uniaxial Compressive Strength Failure Stress σ_c (MPa)	Notes *(Failure type)*
CC-3@6.9-8.0	Sedimentary	4.956	2.396	2.07	151	39,618	8,790	60.6	Type 5
CC-3@14.8-15.7	Sedimentary	4.945	2.398	2.06	154	40,989	9,079	62.6	Type 5
CC-3@23.8-24.8	Sedimentary	4.861	2.396	2.03	157	27,134	6,020	41.5	Type 5
CC-3@44.6-45.5	Sedimentary	4.932	2.386	2.07	153	21,956	4,910	33.9	Type 4
CC-3@65.6-66.5	Sedimentary	4.946	2.388	2.07	155	35,165	7,852	54.1	Type 5
CC-3@69.5-70.6	Sedimentary	5.169	2.392	2.16	151	28,380	6,318	43.6	Type 6
CC-3@84.1-85.5	Sedimentary	4.936	2.390	2.07	159	30,686	6,840	47.2	Type 2
CC-4@8.2-9.4	Sedimentary	4.866	2.385	2.04	155	34,053	7,622	52.6	Type 6
CC-4@30.7-31.4	Sedimentary	4.945	2.390	2.07	154	20,004	4,461	30.8	Type 5
CC-4@45.2-46.1	Sedimentary	4.916	2.385	2.06	152	30,760	6,888	47.5	Type 4

Dam Raise Section

Main Dam Section

Cutoff Trench Section

Earth Mechanics Institute

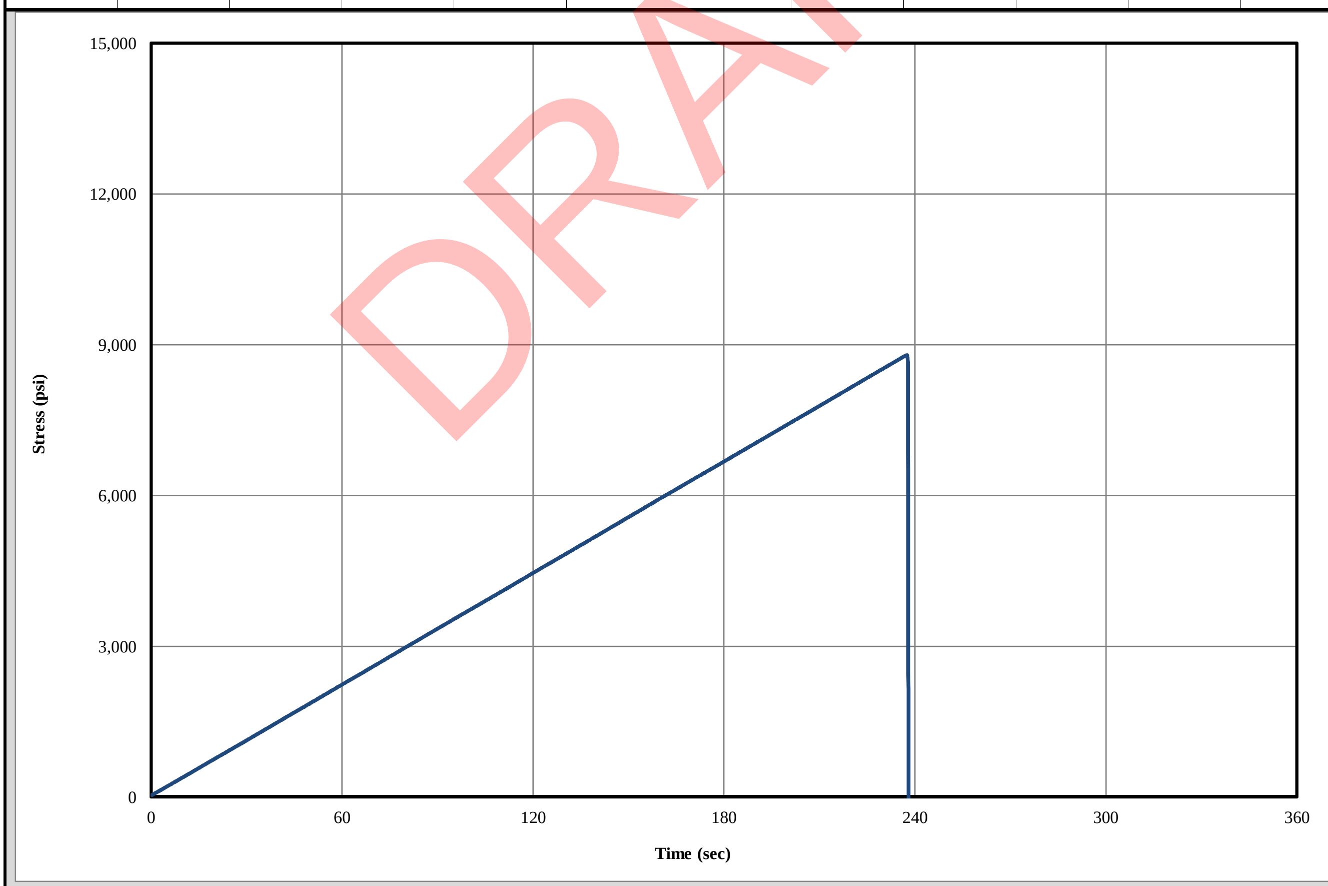
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@6.9-8.0
File Name: CC-3@6.9-8.0_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.956	12.588	2.396	6.085	2.07	39,618	8,790		60.6	Type 5

P-Wave		S-Wave		Dynamic E		Dynamic ν	Static E		Static ν	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	151	2.42



Earth Mechanics Institute

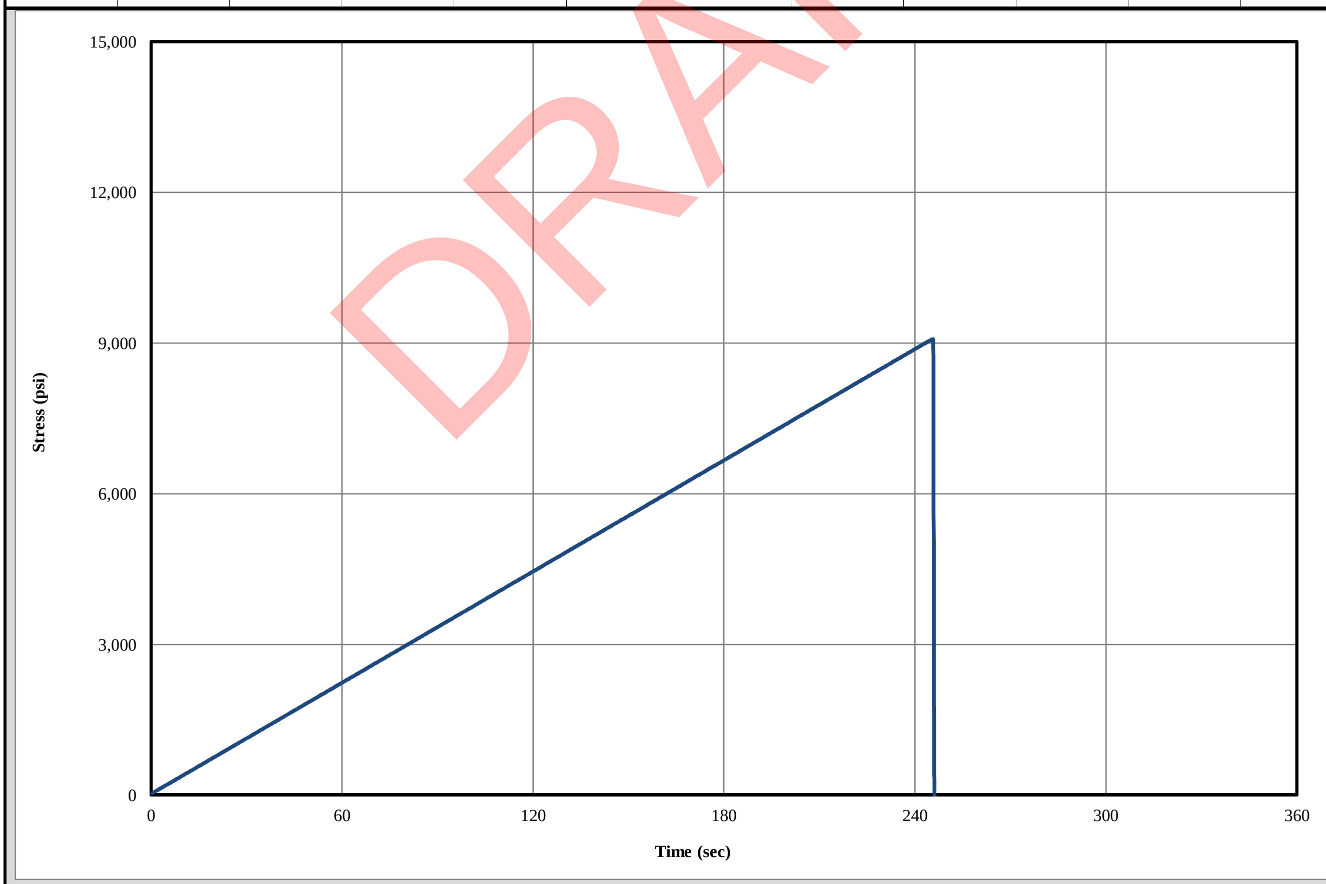
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@14.8-15.7
File Name: CC-3@14.8-15.7_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.945	12.560	2.398	6.090	2.06	40,989	9,079		62.6	Type 5

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	154	2.47



Earth Mechanics Institute

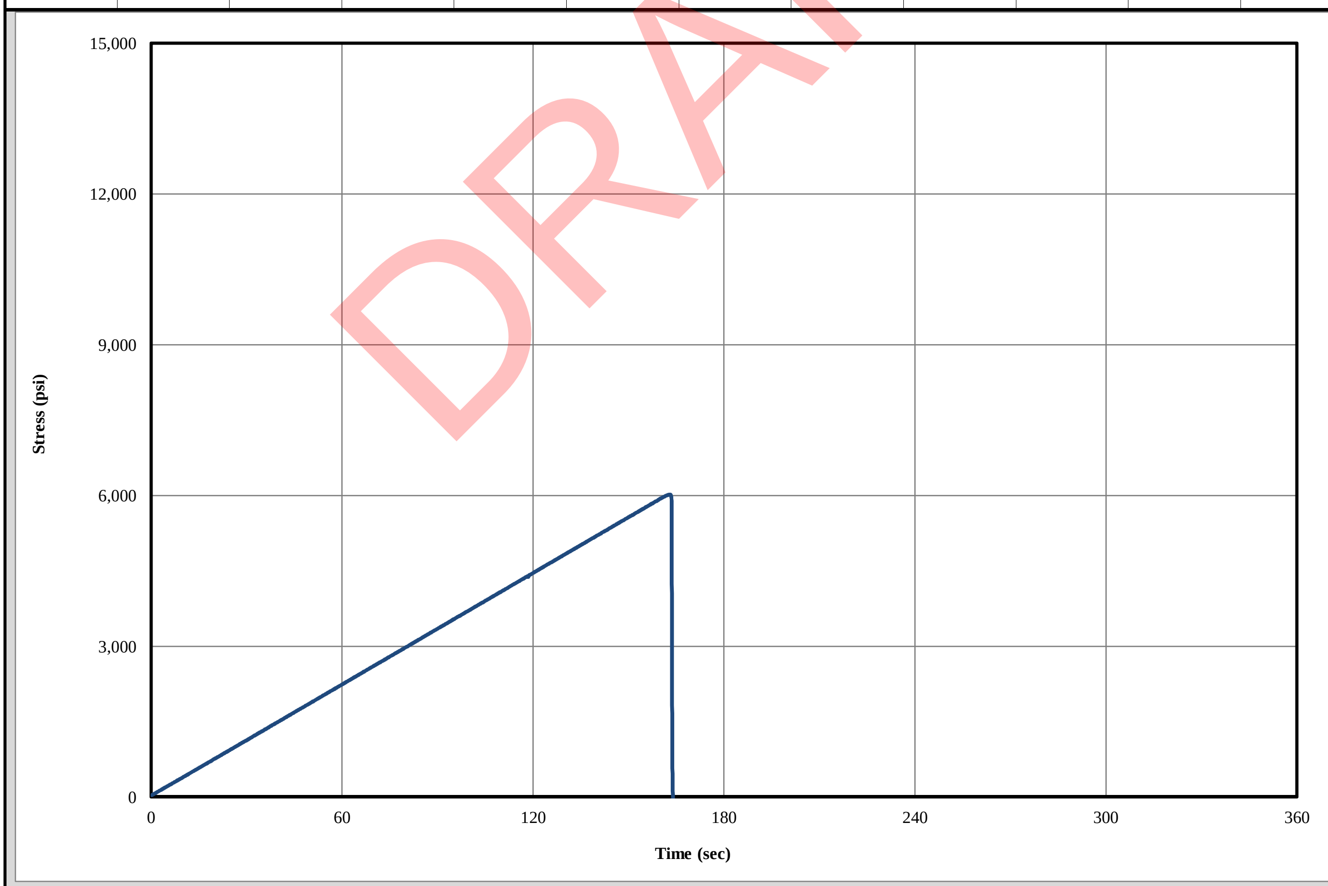
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@23.8-24.8
File Name: CC-3@23.8-24.8_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.861	12.347	2.396	6.085	2.03	27,134	6,020		41.5	Type 5

P-Wave		S-Wave		Dynamic E		Dynamic ν	Static E		Static ν	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	157	2.51



Earth Mechanics Institute

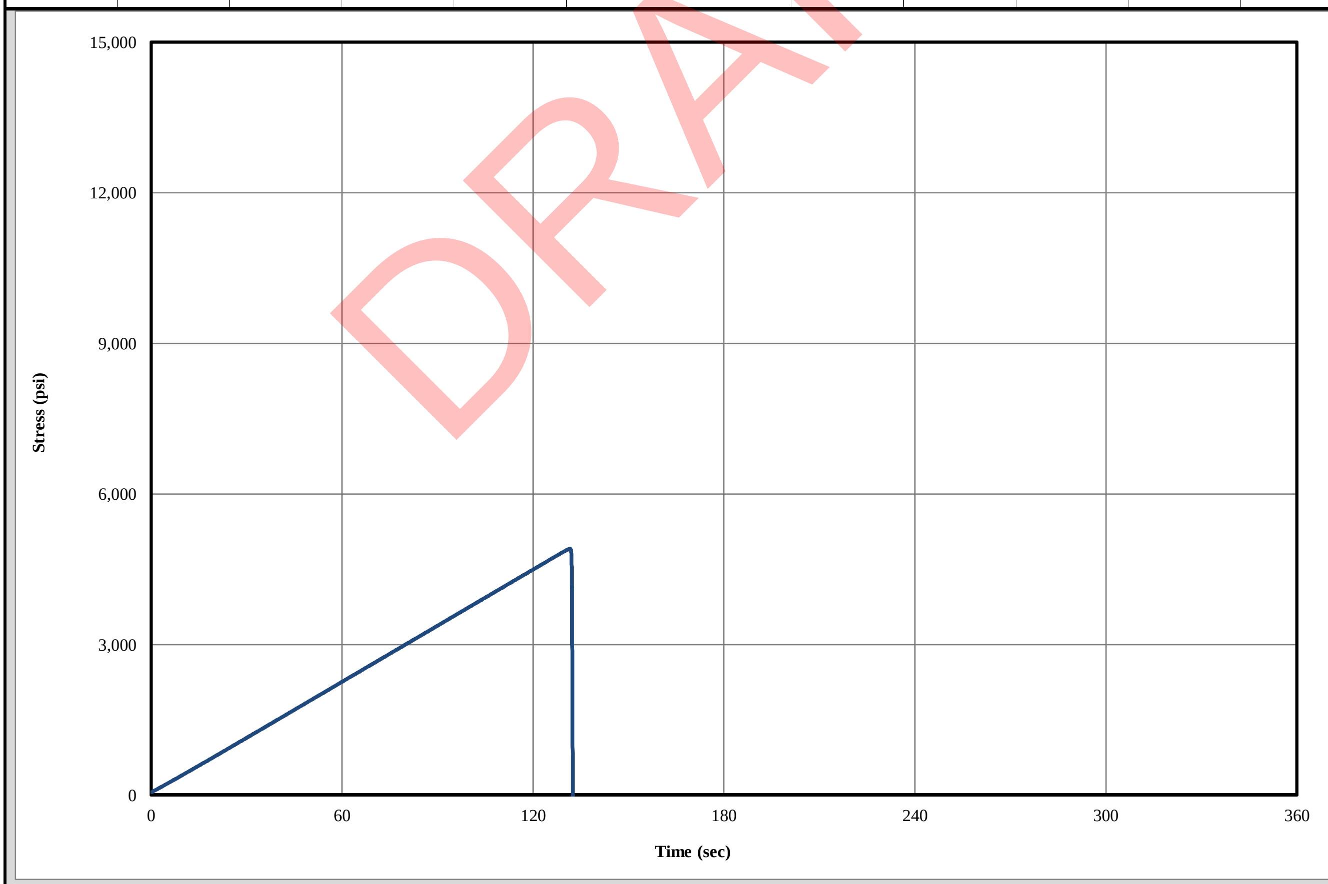
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@44.6-45.5
File Name: CC-3@44.6-45.5_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.932	12.527	2.386	6.060	2.07	21,956	4,910		33.9	Type 4

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	153	2.46



Earth Mechanics Institute

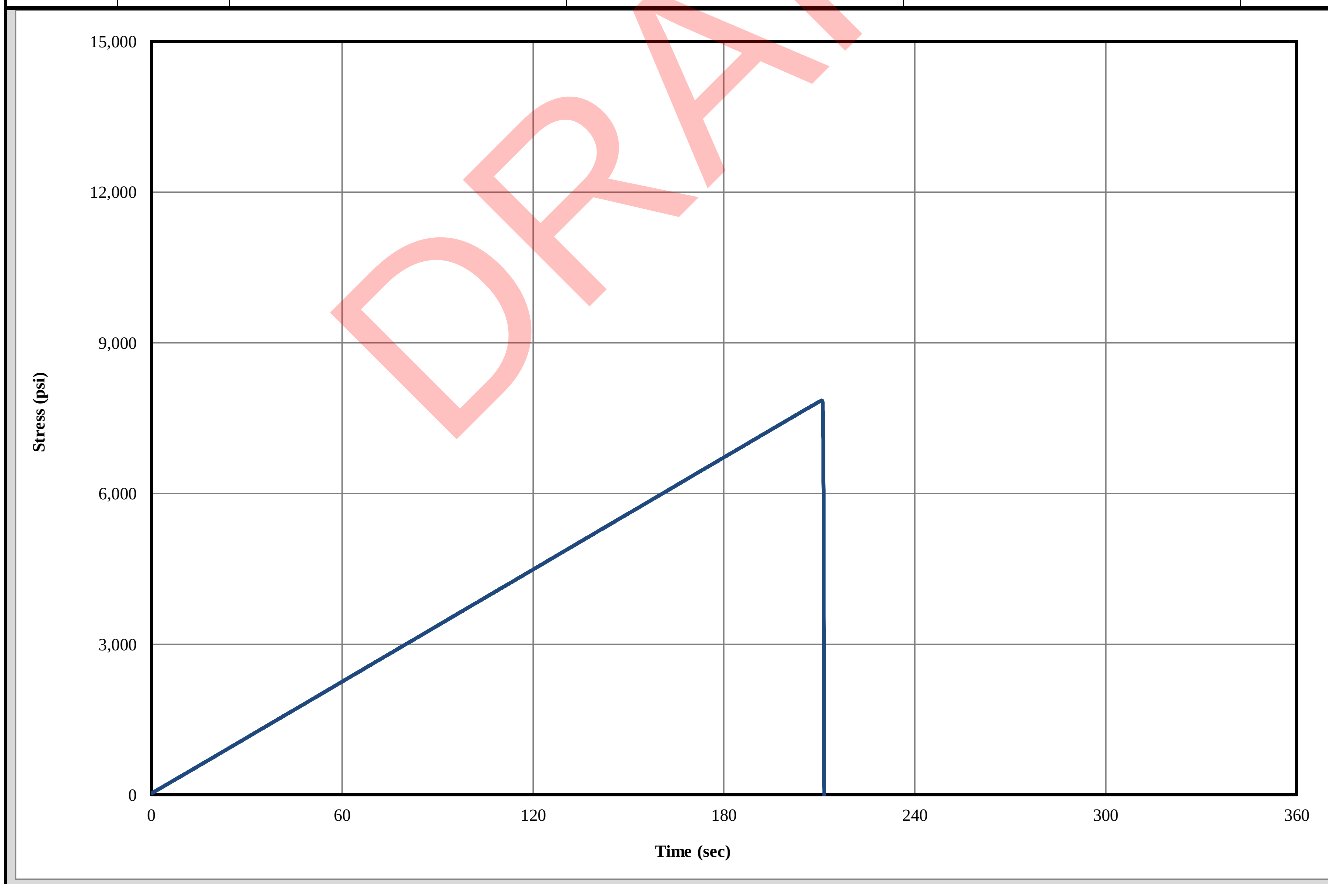
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@65.6-66.5
File Name: CC-3@65.6-66.5_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.946	12.563	2.388	6.066	2.07	35,165	7,852		54.1	Type 5

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	155	2.48



Earth Mechanics Institute

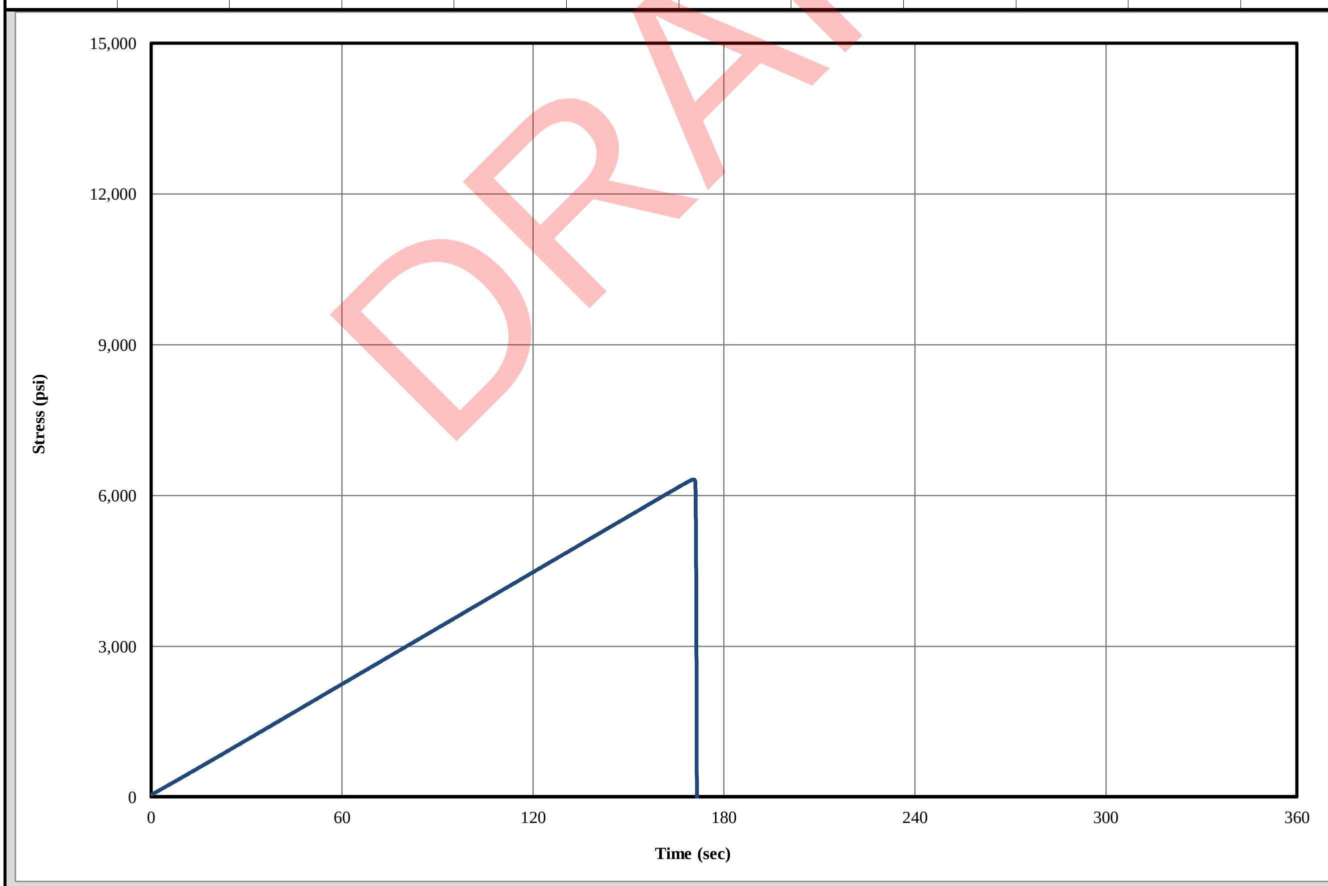
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@69.5-70.6
File Name: CC-3@69.5-70.6_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
5.169	13.129	2.392	6.074	2.16	28,380	6,318		43.6	Type 6

P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	151	2.42



Earth Mechanics Institute

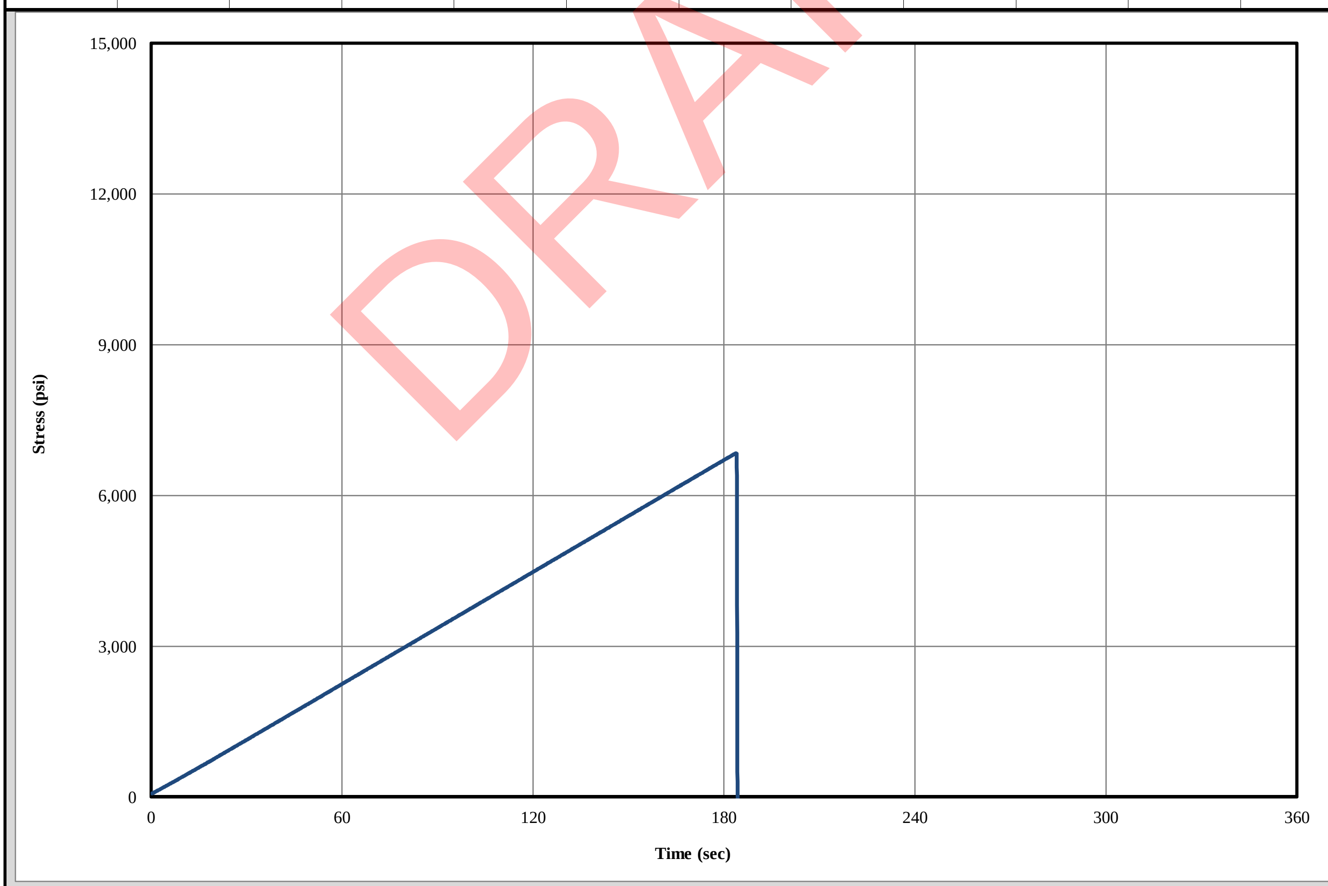
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-3@84.1-85.5
File Name: CC-3@84.1-85.5_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.936	12.537	2.390	6.071	2.07	30,686	6,840		47.2	Type 2

P-Wave		S-Wave		Dynamic E		Dynamic ν	Static E		Static ν	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	159	2.55



Earth Mechanics Institute

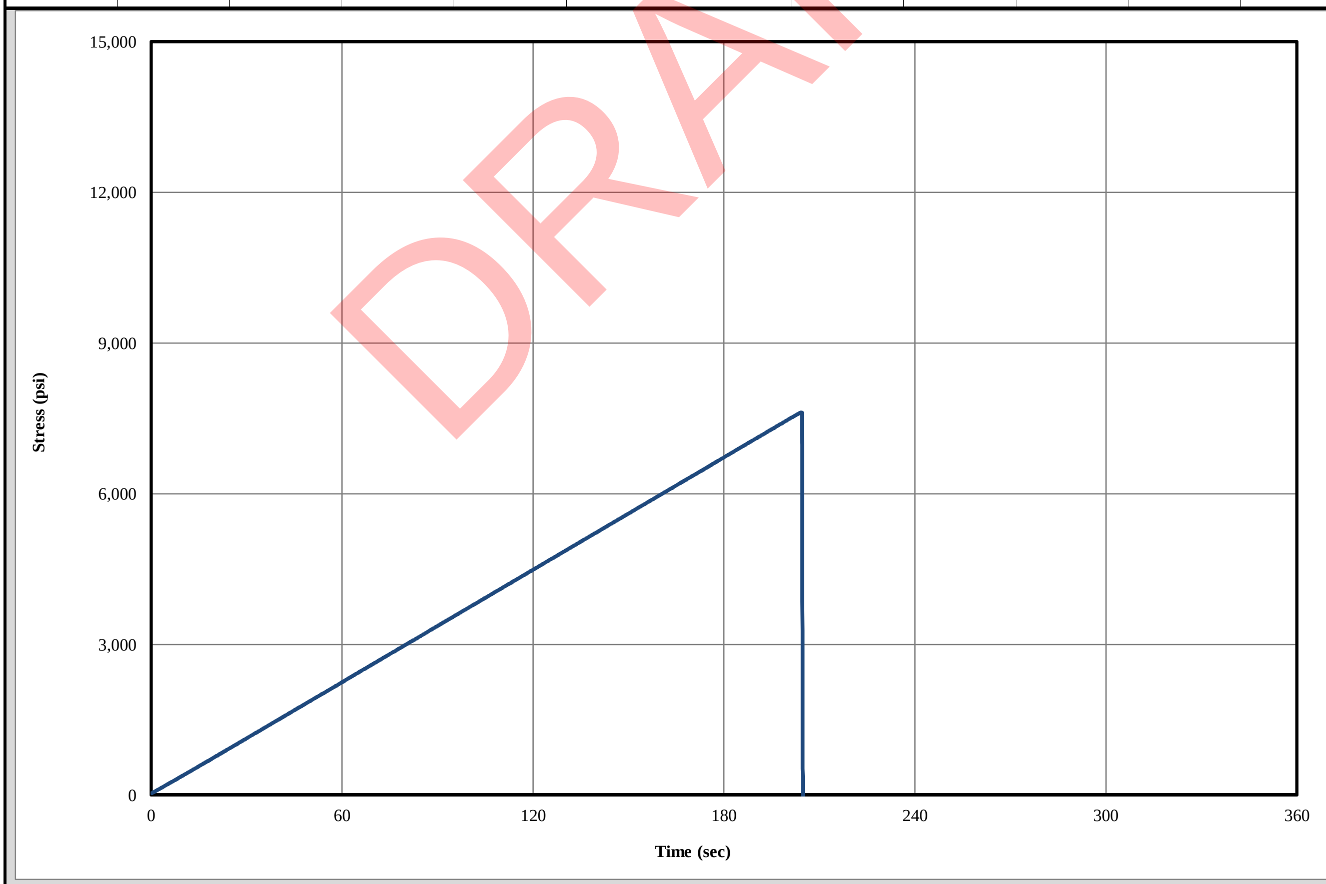
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-4@8.2-9.4
File Name: CC-4@8.2-9.4_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.866	12.360	2.385	6.058	2.04	34,053	7,622		52.6	Type 6

P-Wave		S-Wave		Dynamic E		Dynamic ν	Static E		Static ν	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	155	2.49



Earth Mechanics Institute

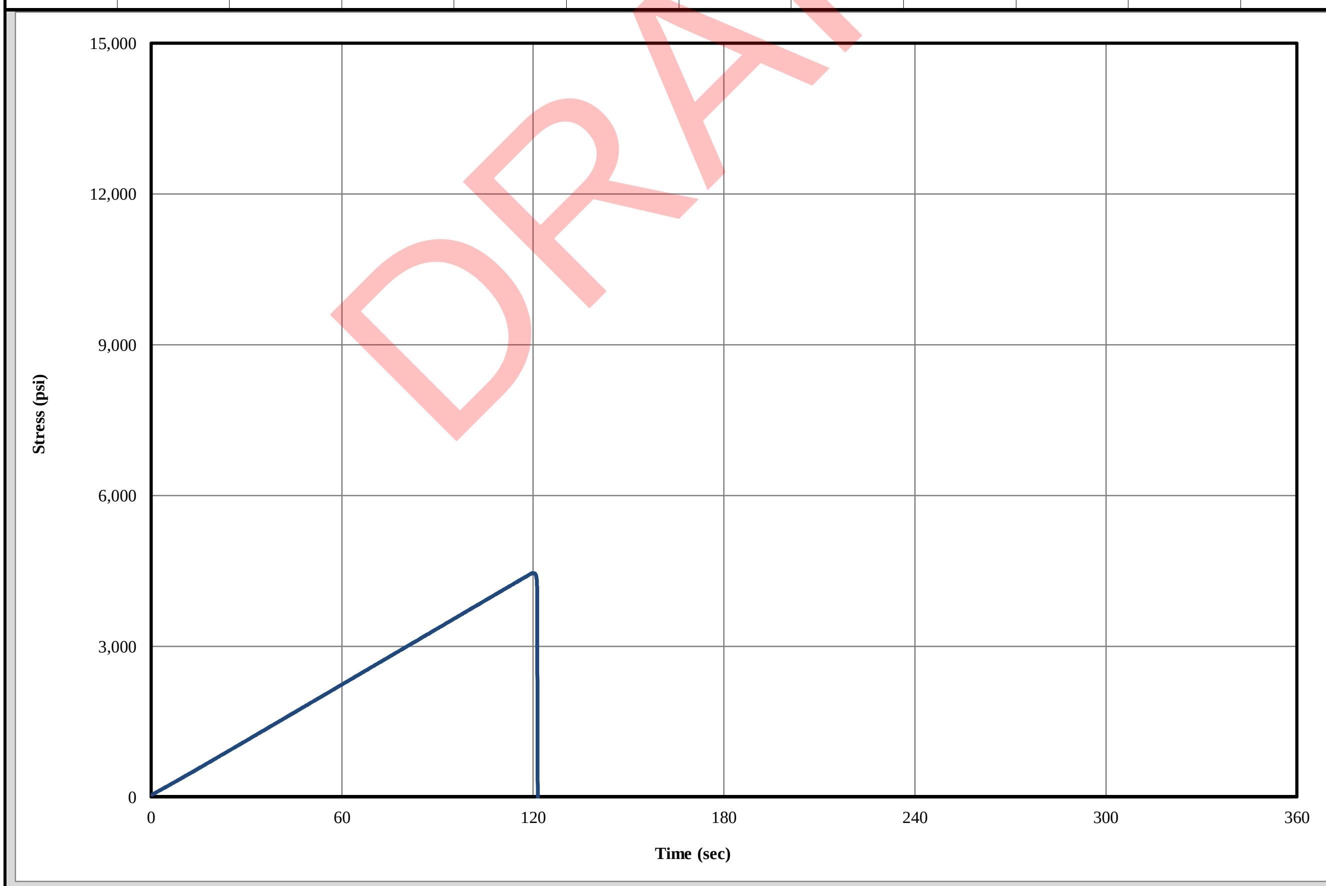
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-4@30.7-31.4
File Name: CC-4@30.7-31.4_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.945	12.560	2.390	6.069	2.07	20,004	4,461		30.8	Type 5

P-Wave		S-Wave		Dynamic E		Dynamic ν	Static E		Static ν	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	154	2.46



Earth Mechanics Institute

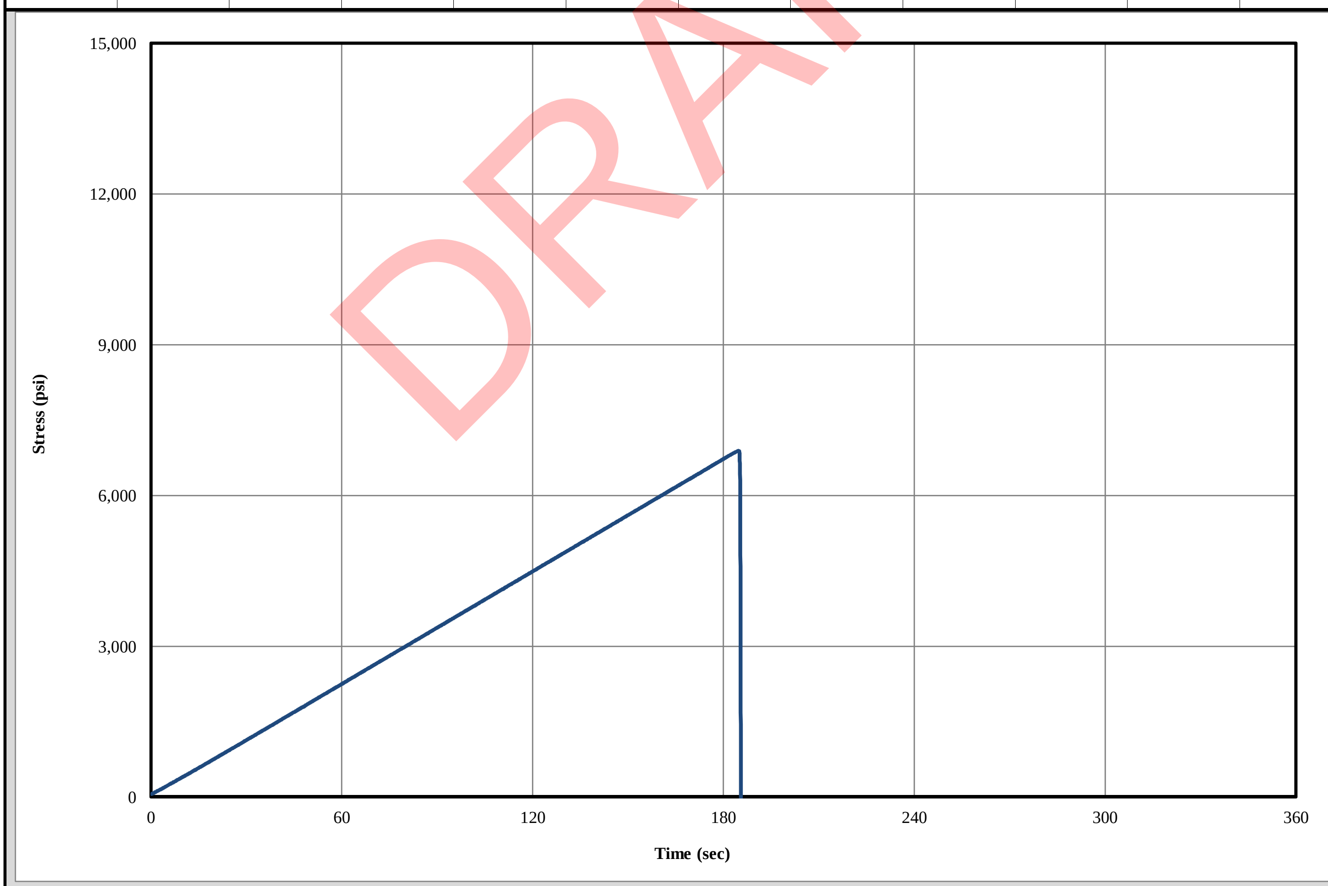
Mining Engineering Department, CSM

Uniaxial Compressive Strength Test Results

Client: Cornforth Consultants, Inc.
Project: Bear Creek Dam
Location: N/A
Rock Type: Concrete
Rock Name: N/A
Characteristics: N/A
Core ID: CC-4@45.2-46.1
File Name: CC-4@45.2-46.1_UCS
Test Performed By: BH
Date Tested: 11/25/2014
Data Reduced By: EAS
Date Reduced: 11/25/2014

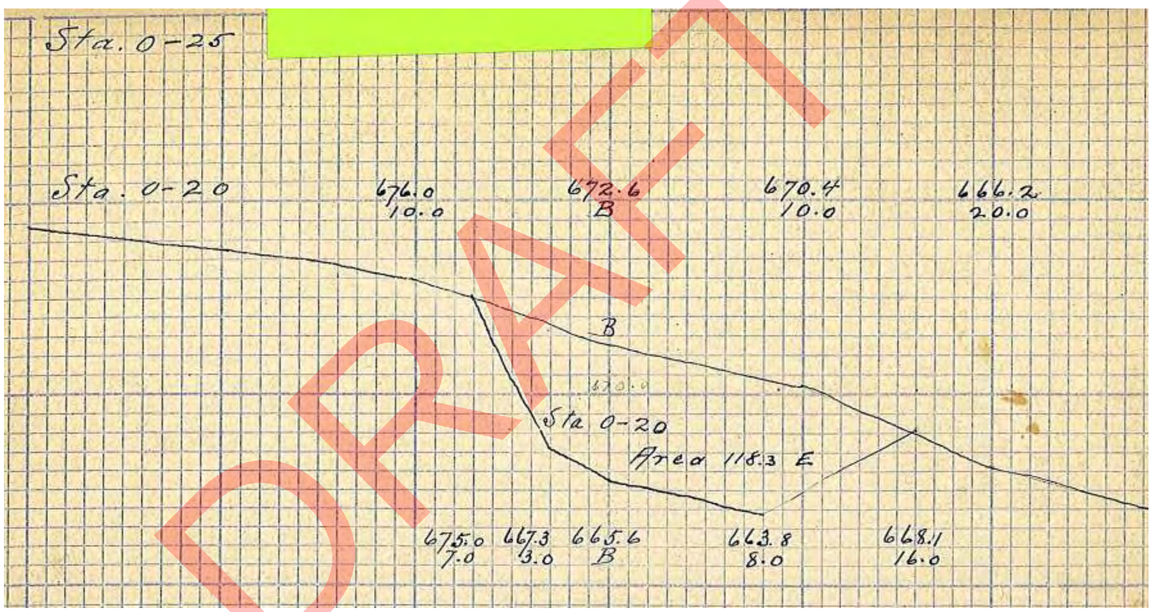
Core Length		Diameter		L/D Ratio	Failure Load	Failure Stress			Failure Mode
in	cm	in	cm		lbs	psi		MPa	
4.916	12.487	2.385	6.057	2.06	30,760	6,888		47.5	Type 4

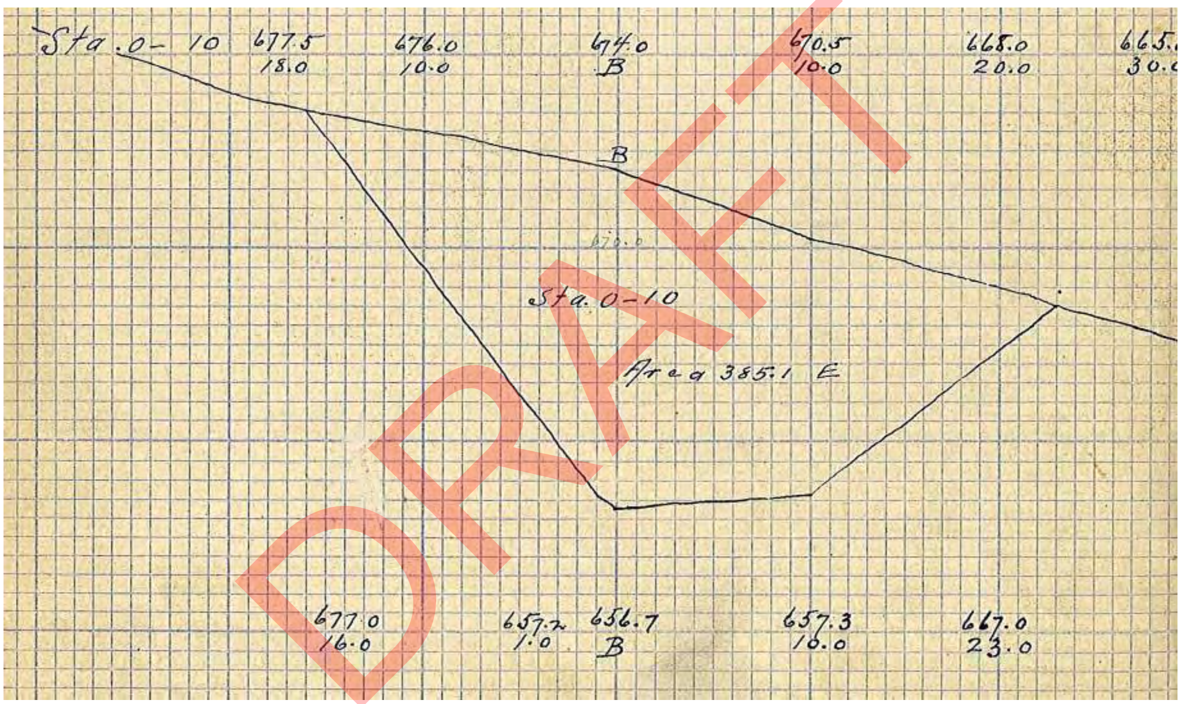
P-Wave		S-Wave		Dynamic E		Dynamic v	Static E		Static v	Density	
ft/sec	m/sec	ft/sec	m/sec	ksi	GPa		ksi	GPa		lb/ft^3	g/cm^3
N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	152	2.44

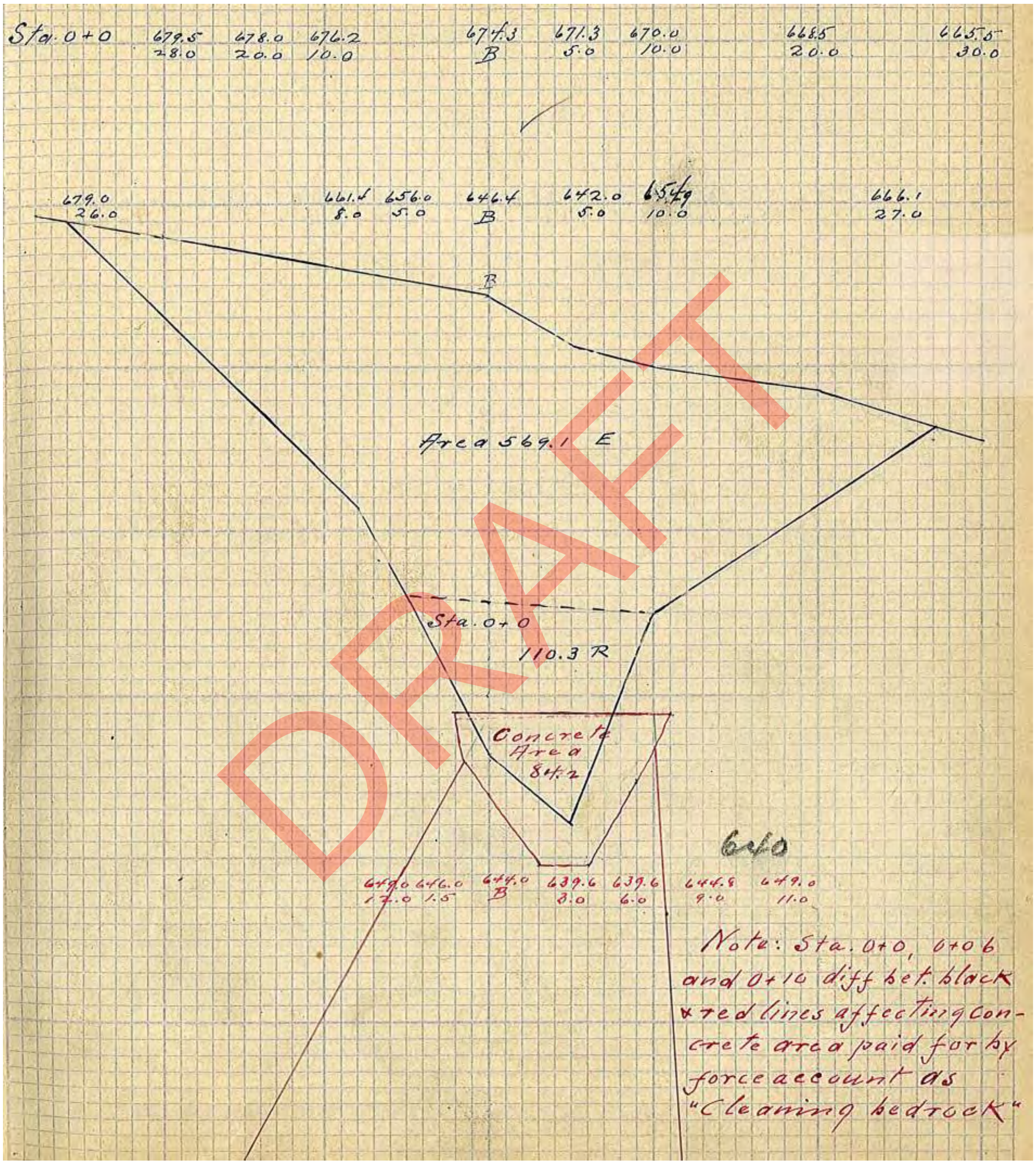


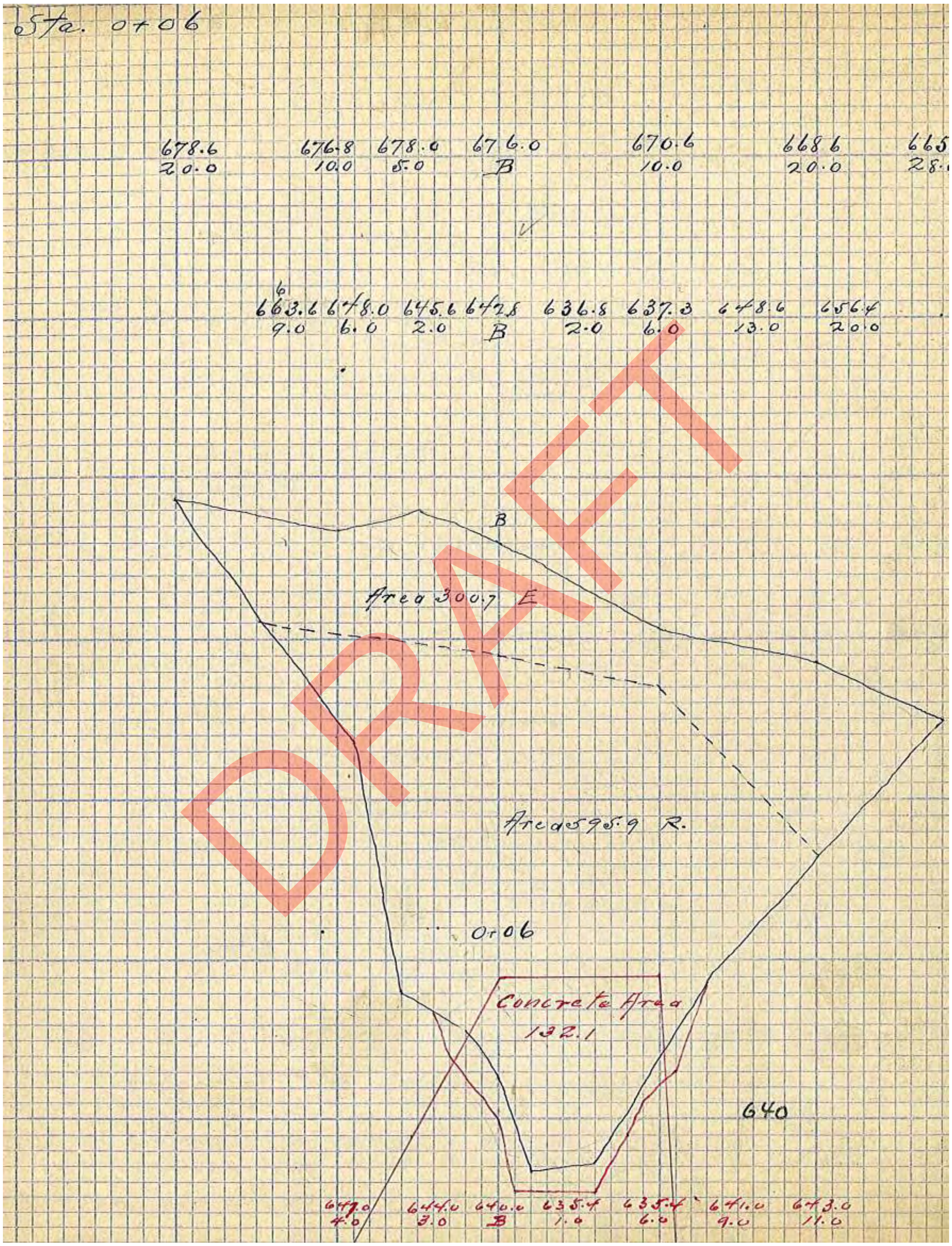
Appendix C

As-Built Cross Sections and Plans


Sta. 0-25
Sta. 0-20
676.0
10.0
672.6
B
670.4
10.0
666.2
20.0
B
Sta 0-20
Area 118.3 E
675.0
7.0
667.3
3.0
665.6
B
663.8
8.0
668.1
16.0


Sta. 0- 10
677.5
18.0
676.0
10.0
674.0
B
670.5
10.0
668.0
20.0
B
Sta. 0-10
Area 385.1 E
677.0
16.0
657.2
1.0
656.7
B
657.3
10.0
667.0
23.0


Sta. 0+0
679.5 28.0
678.0 20.0
676.2 10.0
674.3 B
671.3 5.0
670.0 10.0
668.5 20.0
665.5 30.0
679.0 26.0
661.4 8.0
656.0 5.0
646.4 B
642.0 5.0
654.9 10.0
666.1 27.0
B
Area 569.1 E
Sta. 0+0
110.3 R
Concrete Area 84.2
640
649.0 17.0
646.0 1.5
644.0 B
639.6 3.0
639.6 6.0
644.8 9.0
649.0 11.0
Note: Sta. 0+0, 0+06 and 0+10 diff bet. black & red lines affecting concrete area paid for by force account as "Cleaning bedrock"

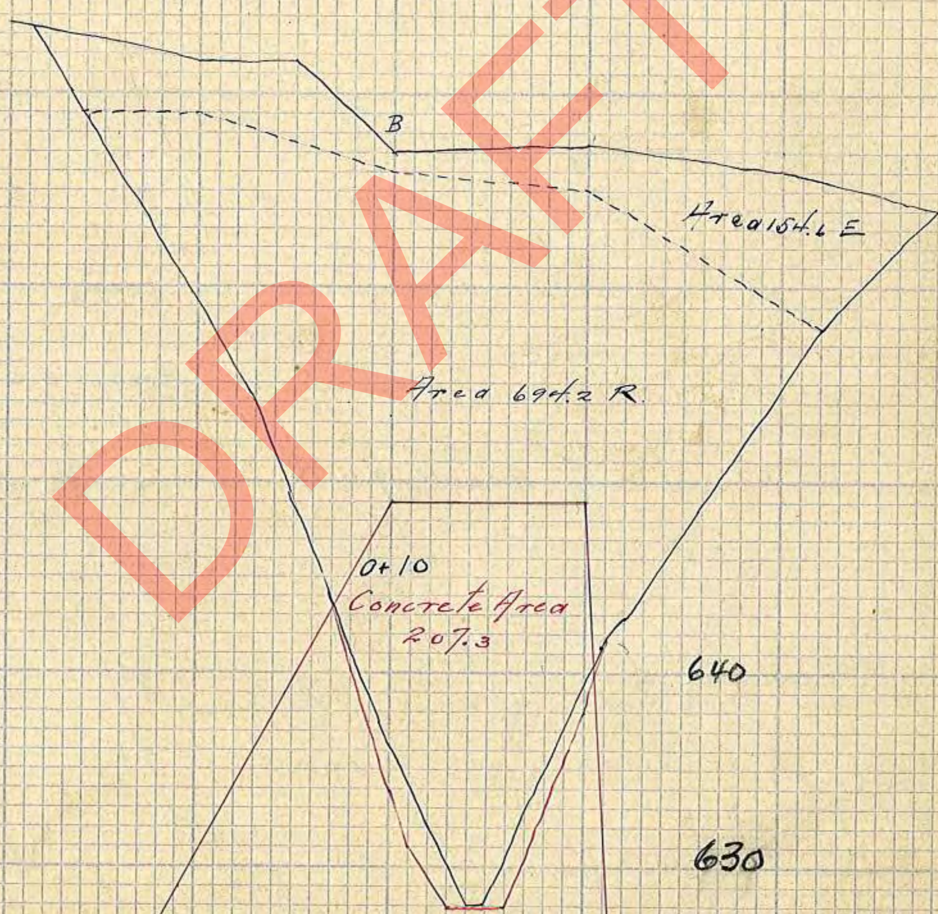

Sta. 0+06
678.6 20.0
676.8 10.0
678.0 5.0
676.0 B
670.6 10.0
668.6 20.0
663.6 9.0
648.0 6.0
645.6 2.0
642.8 B
636.8 2.0
637.3 6.0
648.6 13.0
656.4 20.0
B
Area 300.7 E
Area 595.9 R.
0+06
Concrete Area 132.1
640
647.0 4.0
644.0 3.0
640.0 B
635.4 1.0
635.4 6.0
641.0 9.0
643.0 11.0

Sta 0+10

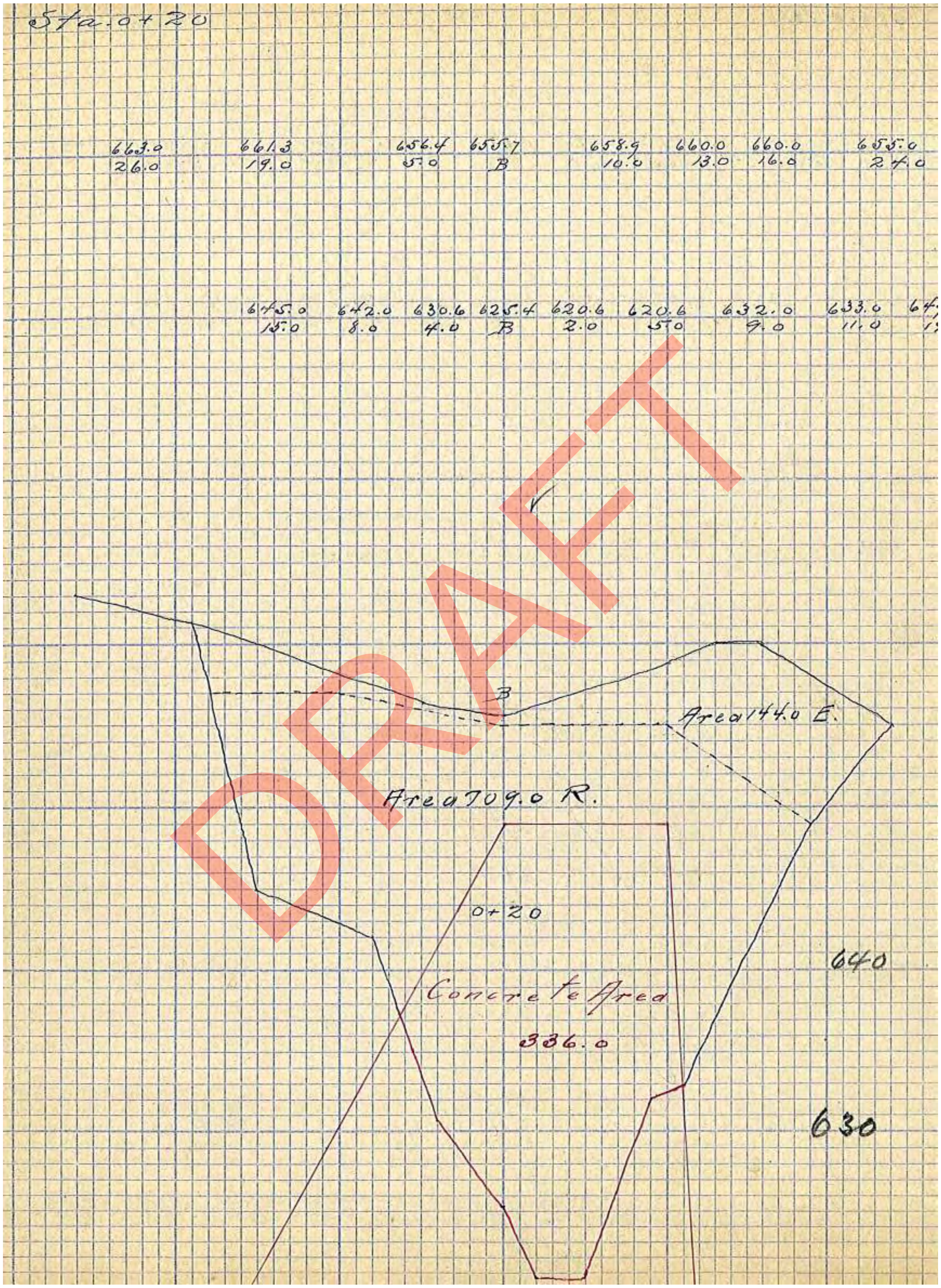
673.6	671.6	671.6	667.0	667.4	666.0	664.0
20.0	10.0	5.0	B	10.0	20.0	28.0

V

673.4	654.0	644.0	636.4	632.0	628.0	628.0	641.8	642.8	657.8
19.0	7.0	3.0	B	2.0	4.0	5.0	11.0	12.0	22.0



644.0	634.0	631.0	628.0	628.0	638.0	641.8
3.0	B	1.0	3.0	6.0	10.0	11.0

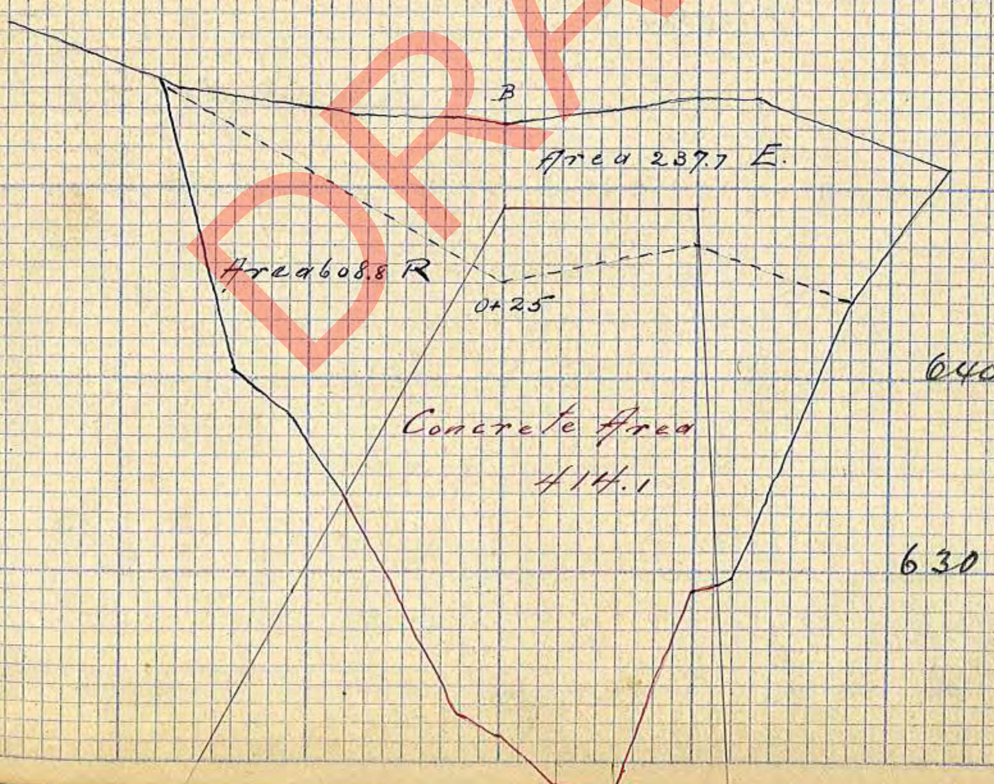


Sta. 0+20
663.0 / 26.0
661.3 / 19.0
656.4 / 5.0
655.7 / B
658.9 / 10.0
660.0 / 13.0
660.0 / 16.0
655.0 / 24.0
645.0 / 15.0
642.0 / 8.0
630.6 / 4.0
625.4 / B
620.6 / 2.0
620.6 / 5.0
632.0 / 9.0
633.0 / 11.0
B
Area 144.0 E.
Area 709.0 R.
0+20
Concrete Area
336.0
640
630

Sta. 0+25

658.4	655.0	653.6	653.6	653.2	654.5	654.5	651.0			
26.0	17.0	7.0	3.0	B	10.0	13.0	23.0			

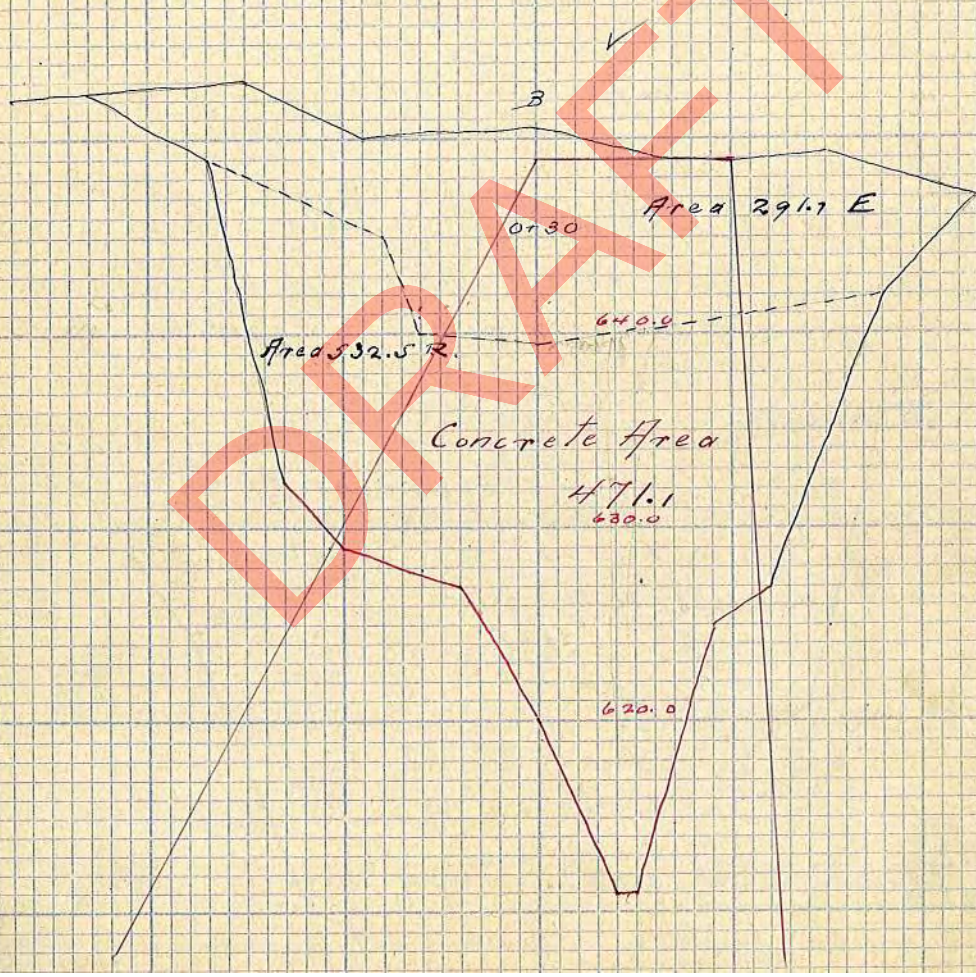
655.2	640.4	638.0	633.6	622.4	621.2	618.6	618.0	627.0	629.6	644.0
18.0	14.0	11.0	8.0	2.0	B	3.0	6.0	10.0	12.0	18.0



Sta 0+30

652.0	653.0	650.0	650.5	649.1	648.9	649.3	647.0
27.0	15.0	9.0	B	6.0	10.0	15.0	23.0

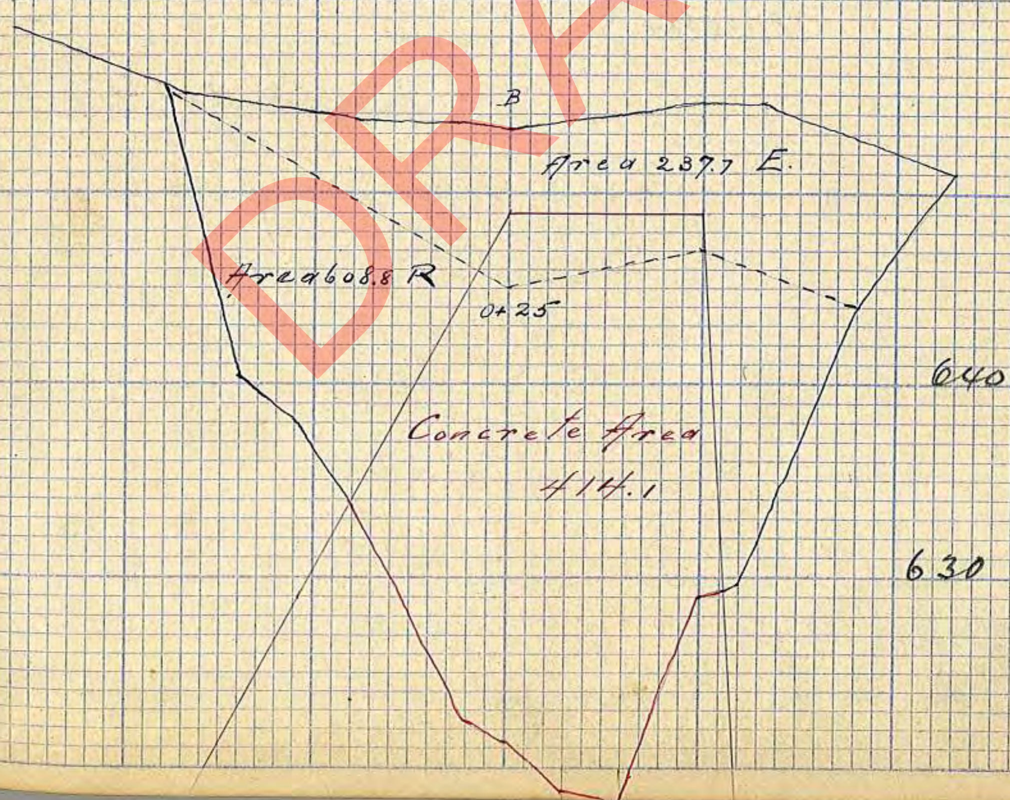
652.4	649.0	632.4	629.0	627.0	620.0	611.0	611.0	625.0	627.0	642.0
23.0	17.0	13.0	10.0	4.0	B	4.0	5.0	9.0	12.0	18.0



Sta. 0+25

658.4	655.0	653.6	653.6	653.2	654.5	654.5	651.0			
26.0	17.0	7.0	3.0	B	10.0	13.0	23.0			

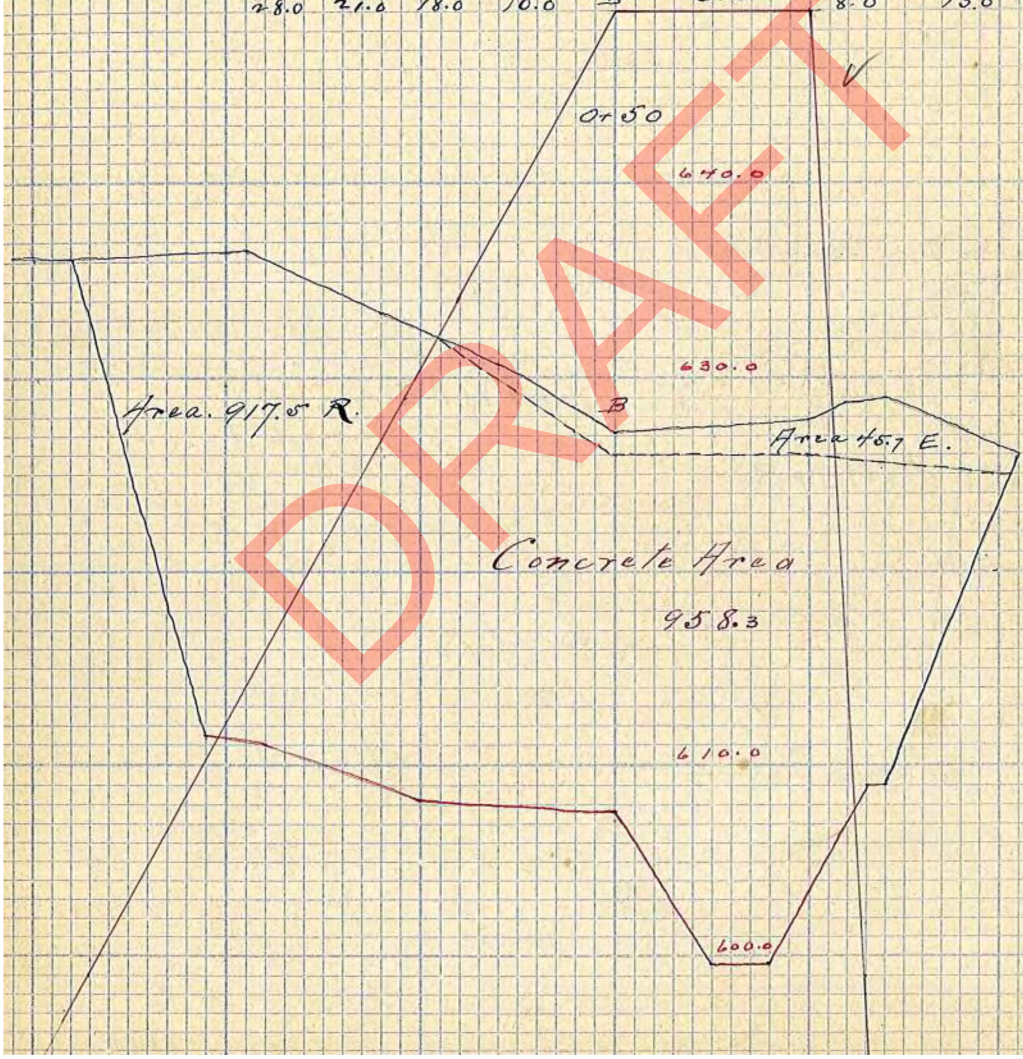
655.2	640.4	638.0	633.6	622.4	621.2	618.6	618.0	627.0	629.6	644.0
18.0	14.0	11.0	8.0	2.0	B	3.0	6.0	10.0	12.0	18.0



Sta. 0+50

636.1	636.1	636.5	633.3	630.7	627.2	627.9	628.8	629.0	626.5	626.
31.0	28.0	19.0	12.0	6.0	B	10.0	12.0	14.0	20.0	21.0

636.2	611.5	611.0	608.2	607.6	599.6	599.6	609.0	609.0	626.
28.0	21.0	18.0	10.0	B	5.0	8.0	13.0	14.0	21.0

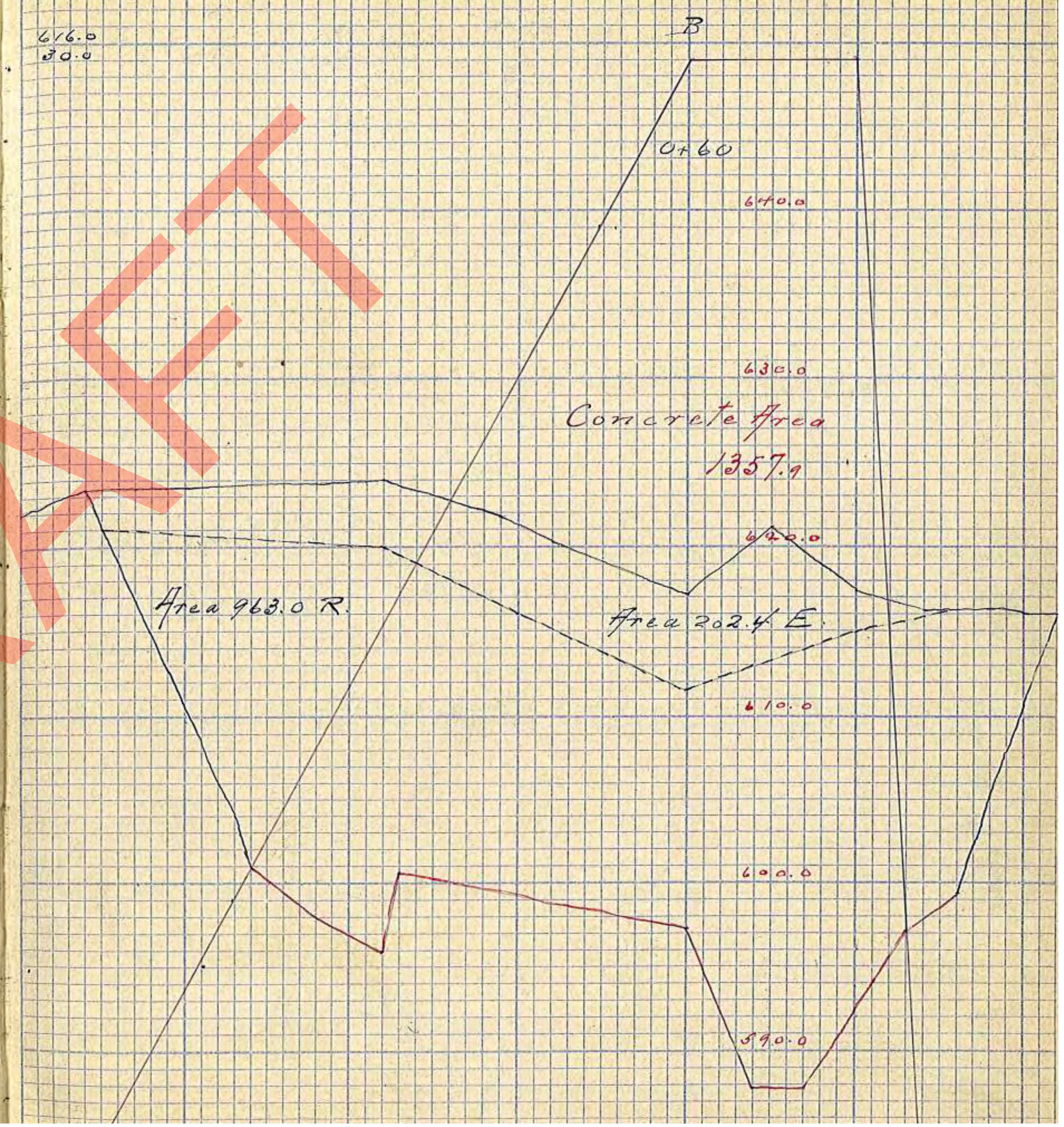


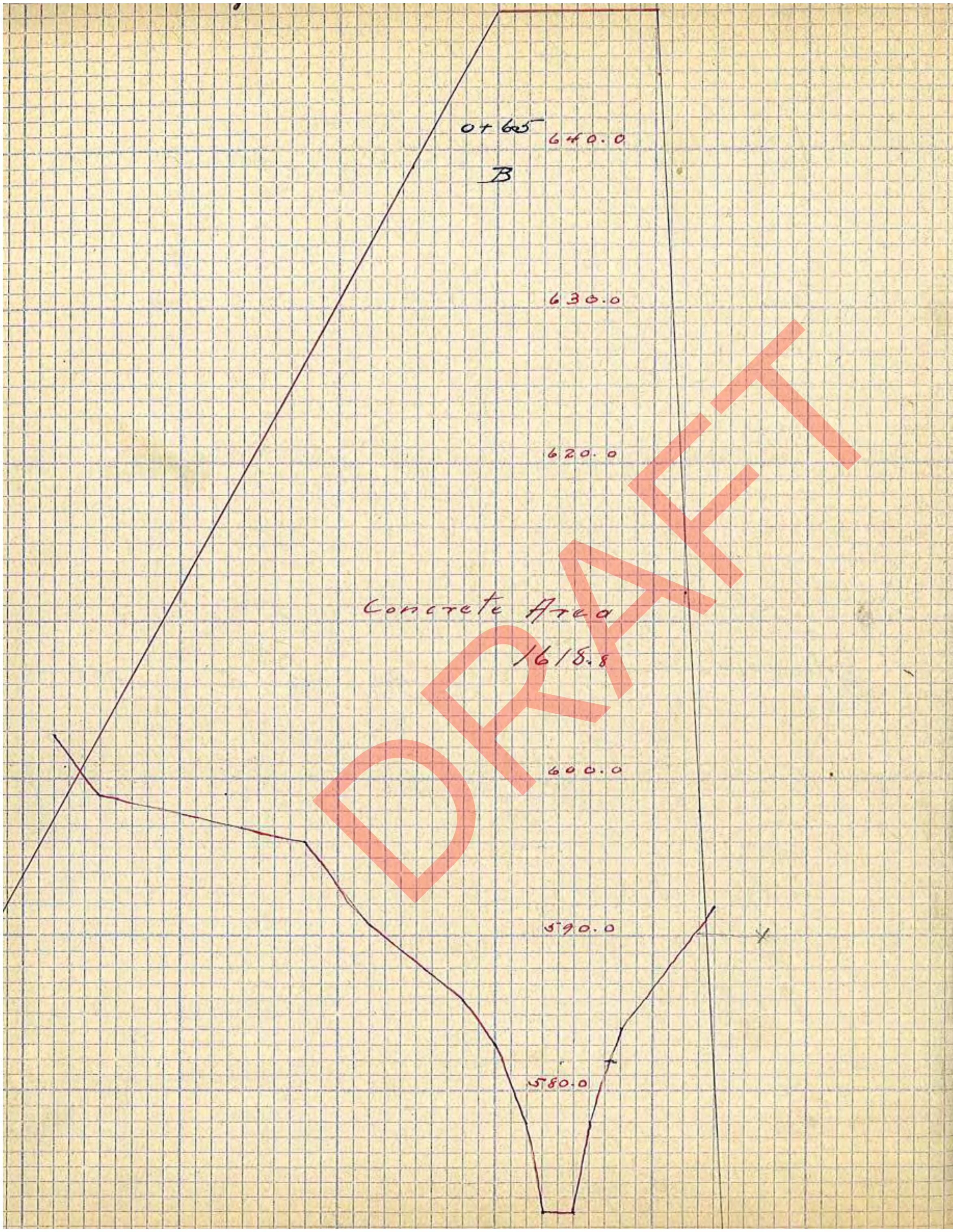
Sta. 0+60

621.0	623.2	624.0	621.7	620.3	617.2	621.1	617.4	616.3	616.3	616.0	616.0
42.0	36.0	18.0	12.0	8.0	B	5.0	10.0	14.0	18.0	20.0	30.0

✓

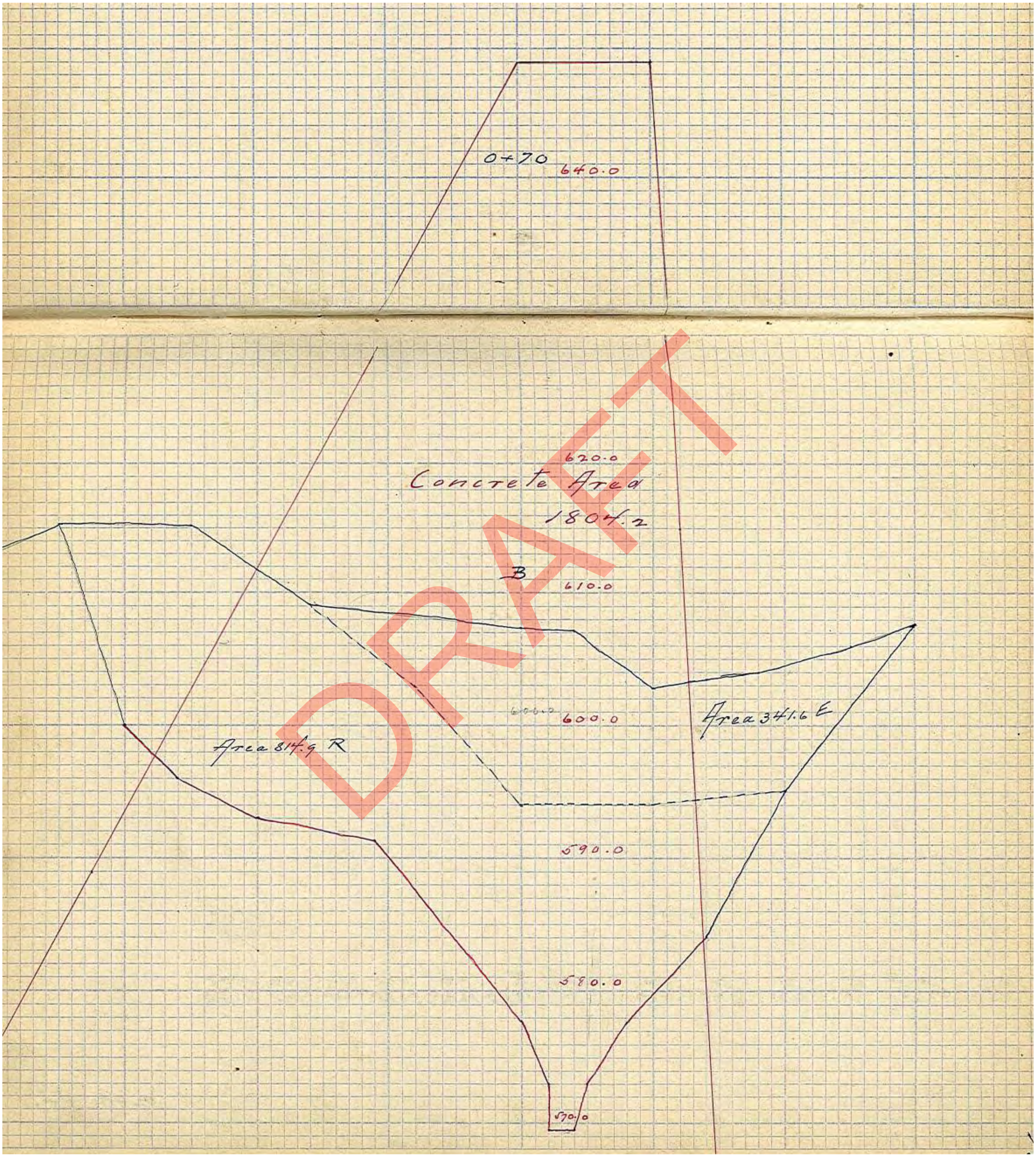
623.0	604.0	601.0	598.0	596.0	600.6	597.2	587.8	587.8	597.2	599.4	616.1
26.0	27.0	26.0	22.0	18.0	17.0	B	4.0	7.0	13.0	16.0	22.0

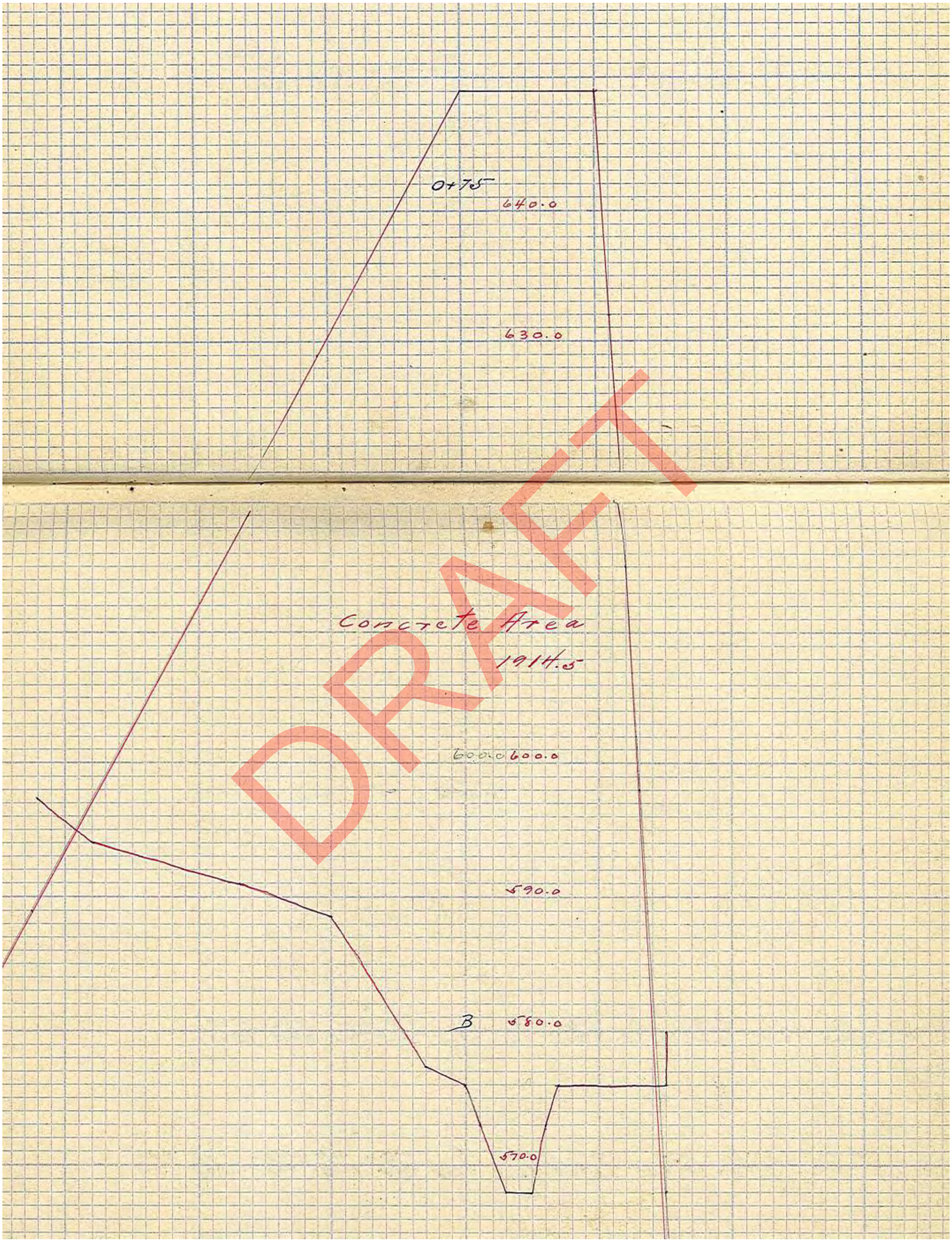


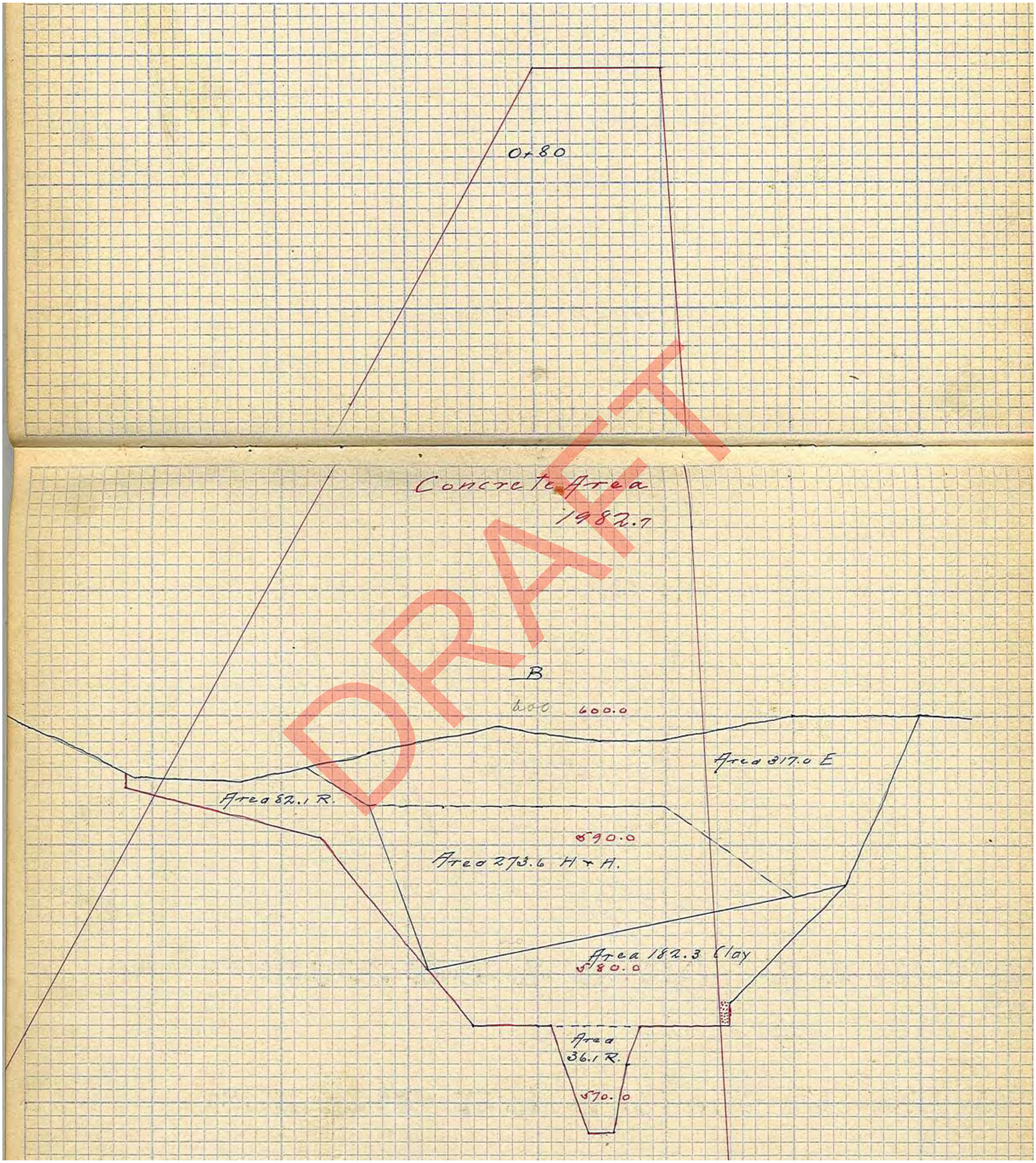


Sta 0+65

602.8 599.0
28.0 25.0


0+70
640.0
620.0
Concrete Area
1804.2
B
610.0
600.0
Area 814.9 R
Area 341.6 E
590.0
580.0
570.0

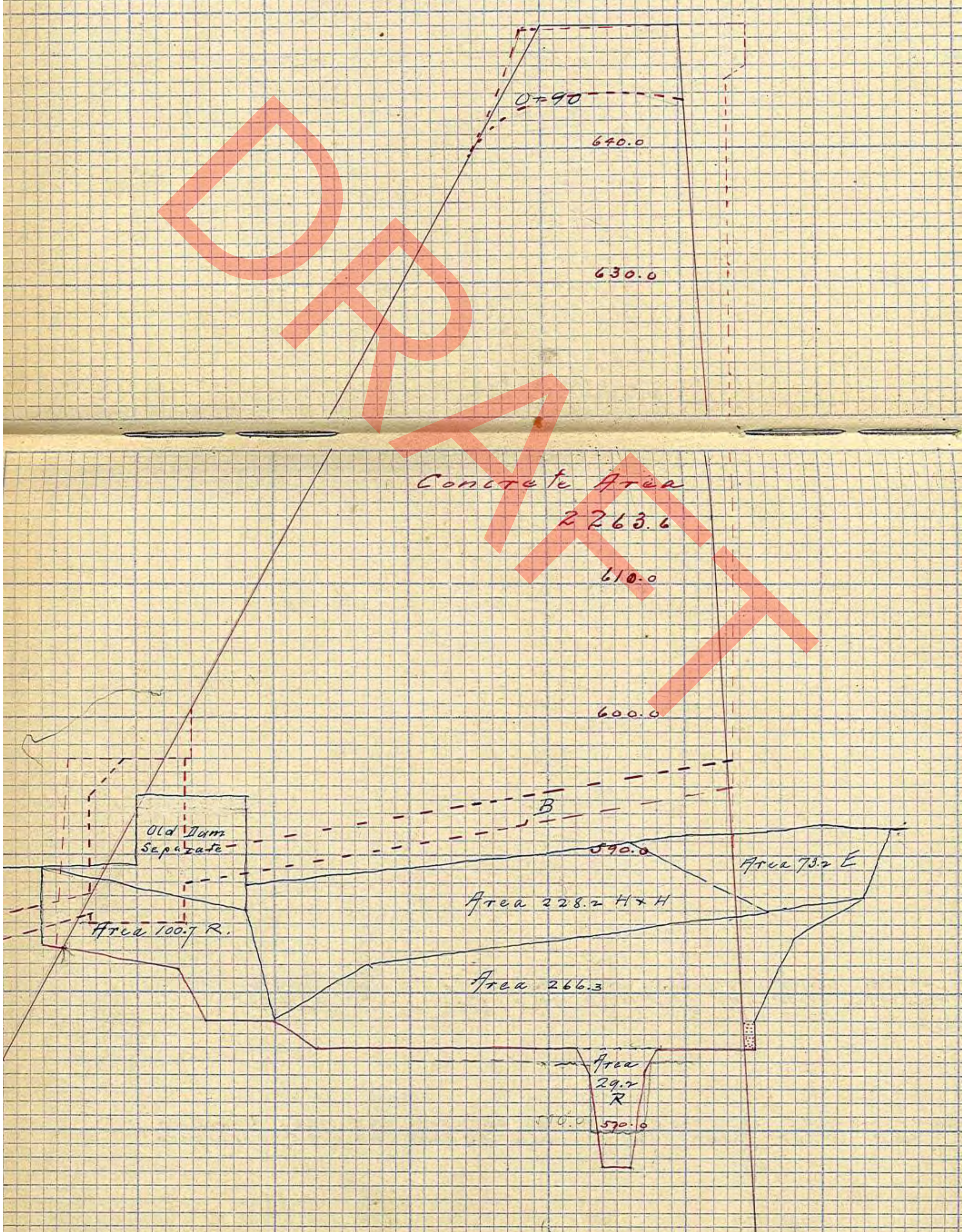

0+75
640.0
630.0
Concrete Area
1914.5
600.0 600.0
590.0
B
580.0
570.0


0+80
Concrete Area
1982.7
B
600
600.0
Area 317.0 E
Area 82.1 R.
590.0
Area 273.6 H + H.
Area 182.3 Clay
580.0
Area
36.1 R.
570.0

Sta. 0+90

588.8	589.4	594.3	594.3	587.9	590.1	591.5	591.7	592.0	592.
45.0	30.0	30.0	27.0	22.0	B	10.0	16.0	20.0	23.

583.5	581.7	578.0	578.0	576.0	576.0	576.0	574.0	567.4	567.4	574.0	576.0
37.0	27.0	25.0	20.0	17.0	B	2.0	3.0	4.0	6.0	7.0	8.0



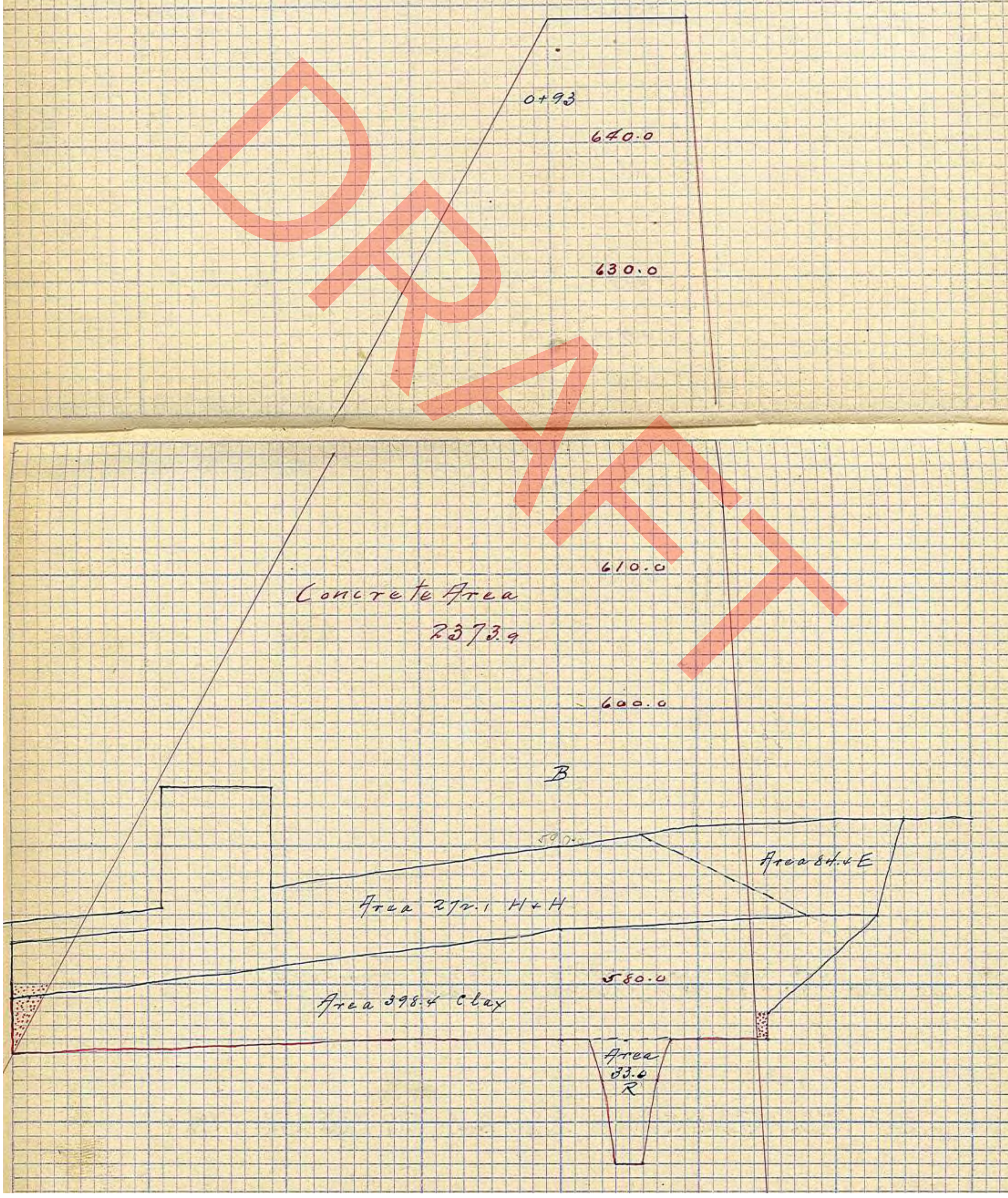
Sta. 0+93

587.5	584.3	585.5	594.3	594.3	587.0	590.0	591.5	591.7	592.0	592.0
50.0	42.0	29.0	29.0	21.0	21.0	B	10.0	16.0	20.0	30.0

V

585.0
23.0

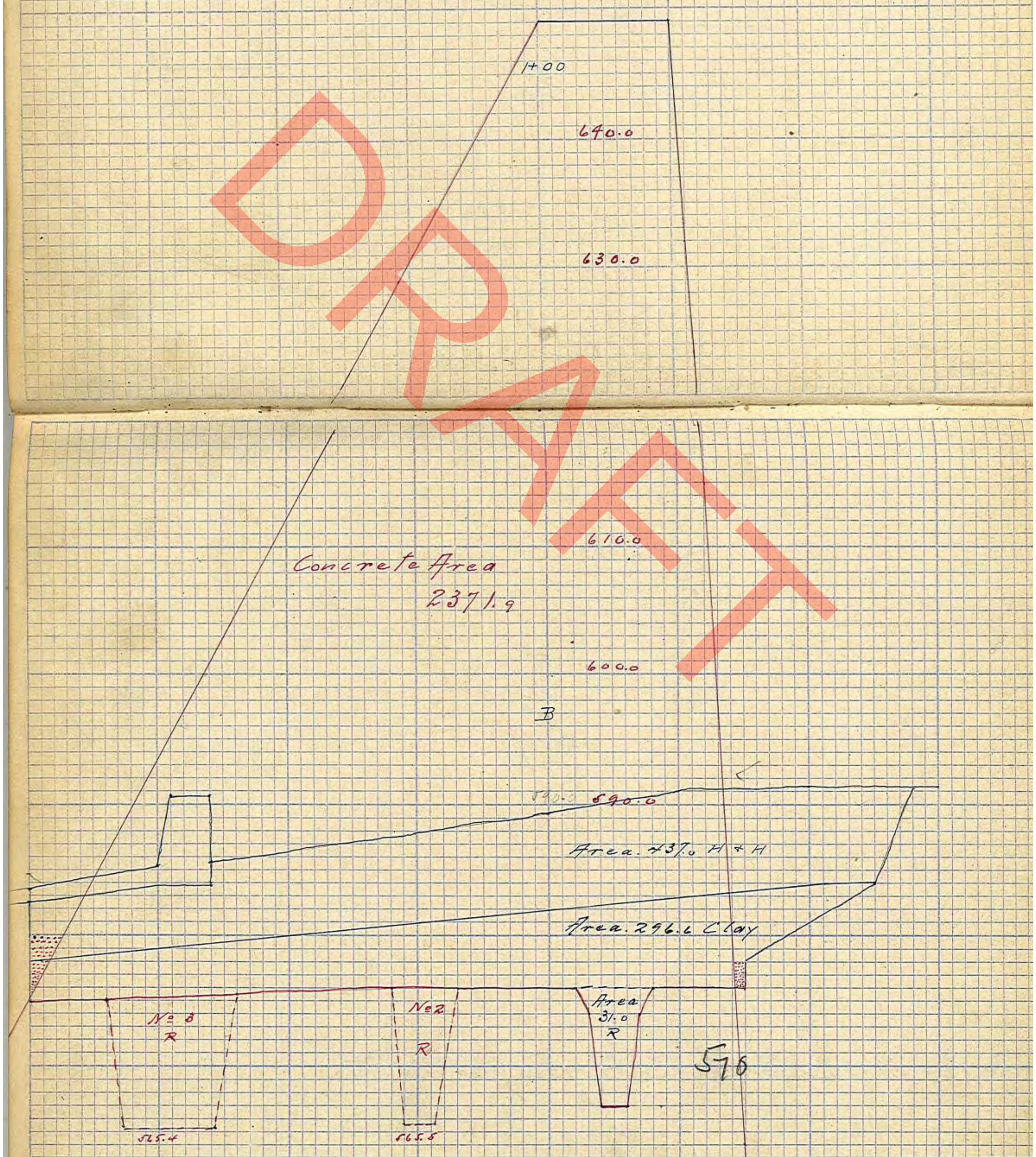
583.0	575.0	576.0	576.0	576.0	573.0	567.0	567.0	573.0	576.0	576.0
40.0	40.0	12.0	B	2.0	3.0	4.0	6.0	7.0	8.0	15.0



Sta 1+00

583.7	585.2	590.6	590.6	585.5	589.4	591.0	591.2	591.4	591.4
41.0	30.0	29.0	26.0	26.0	B	10.0	16.0	20.0	30.0

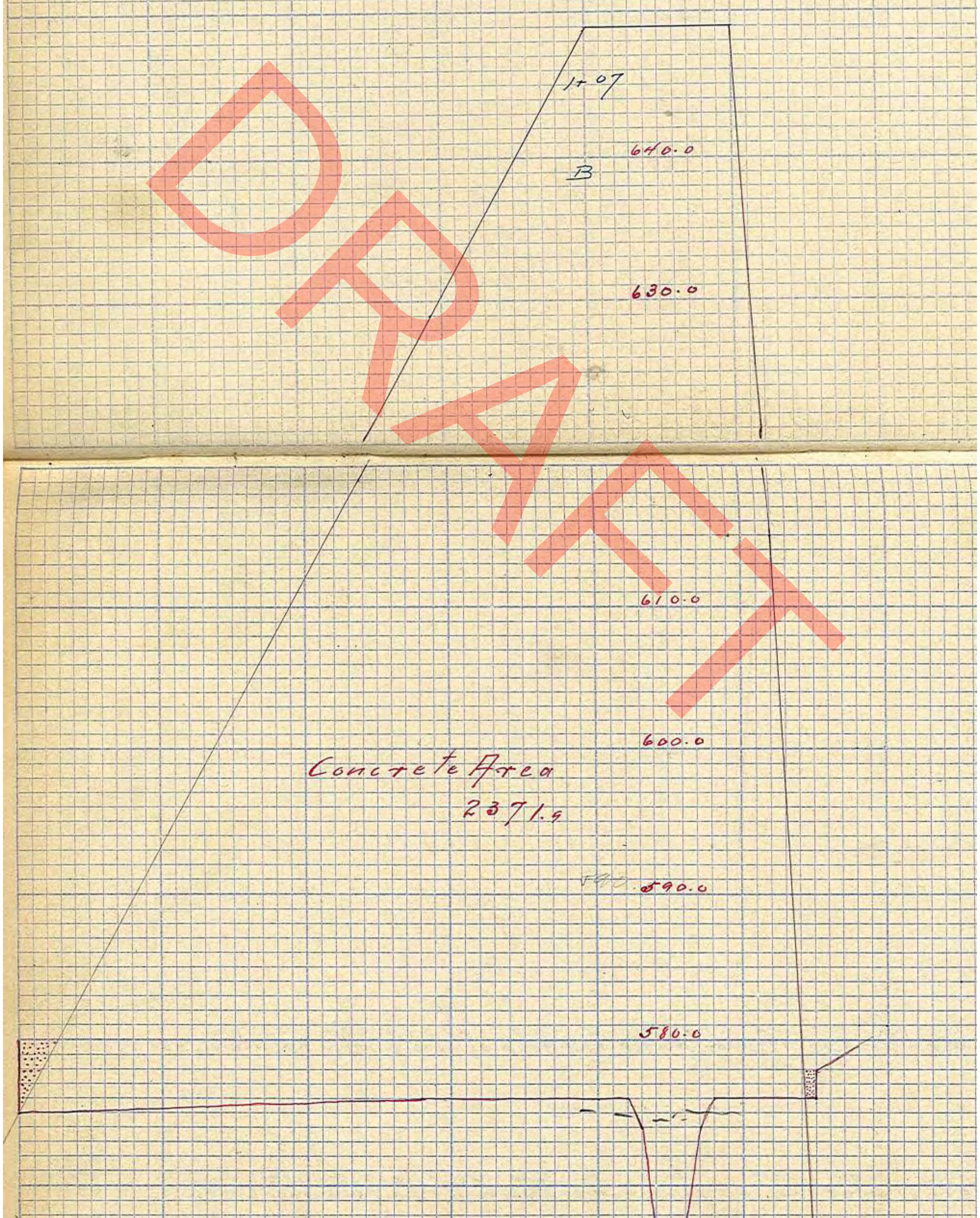
582.5	575.0	576.0	576.0	576.0	574.0	567.0	567.0	574.0	576.0	576.0	578.0
40.0	40.0	12.0	B	2.0	3	4.0	6.0	7.0	8.0	15.0	15.0



Trench No 2 & 3 Measured Separate. See page 38

Sta. 1+07

580.0	575.0	576.0	576.0	576.0	574.0	567.0	567.0	574.0	576
40.0	40.0	12.0	B	2.0	3.0	4.0	6.0	7.0	8.0

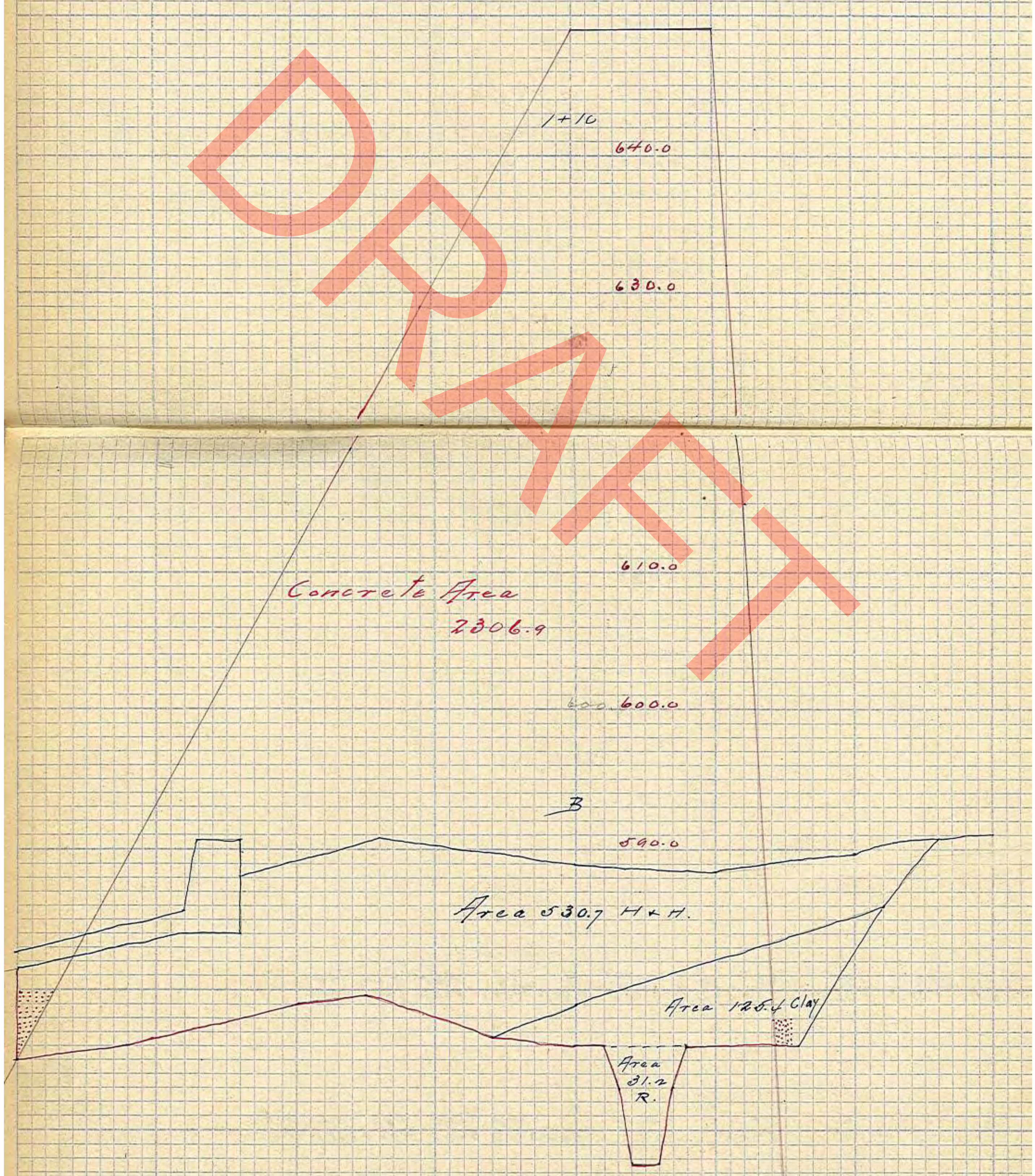


Sta. 1+10

582.4	585.5	590.6	590.6	588.1	590.7	589.0	588.4	589.4	589.8	590.5	591.0
40.0	28.0	27.0	24.0	24.0	14.0	B	10.0	6.0	20.0	23.0	30.0

✓

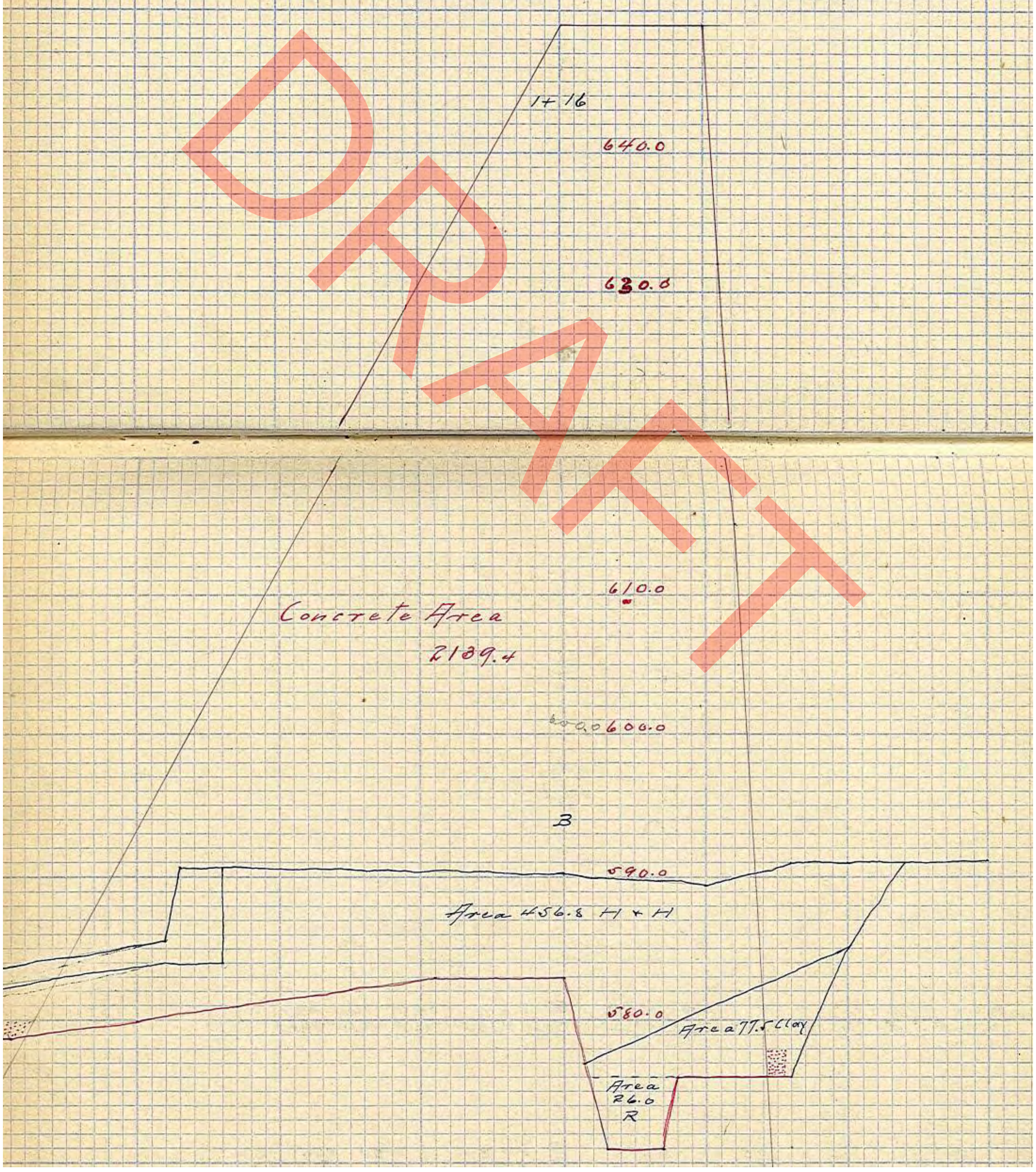
581.5	575.0	576.0	579.0	579.6	576.6	576.0	576.0	573.0	567.6	567.6	573.0	576.0	576.0	[illegible]
40.0	40.0	32.0	20.0	15.0	6.0	B	2.0	3.0	4.0	6.0	7.0	8.0	16.0	[illegible]



Sta 1+16

581.4	583.3	585.5	590.6	590.6	590.1	589.5	591.0	591.0	591.1
45.0	41.0	28.0	27.0	24.0		10.0	16.0	20.0	30.0

582.0	578.8	578.8	583.0	583.0	571.0	571.0	576.0	576.0	585.1	591.0
43.0	42.0	39.0	9.0	B	3.0	7.0	8.0	16.0	20.0	24.0

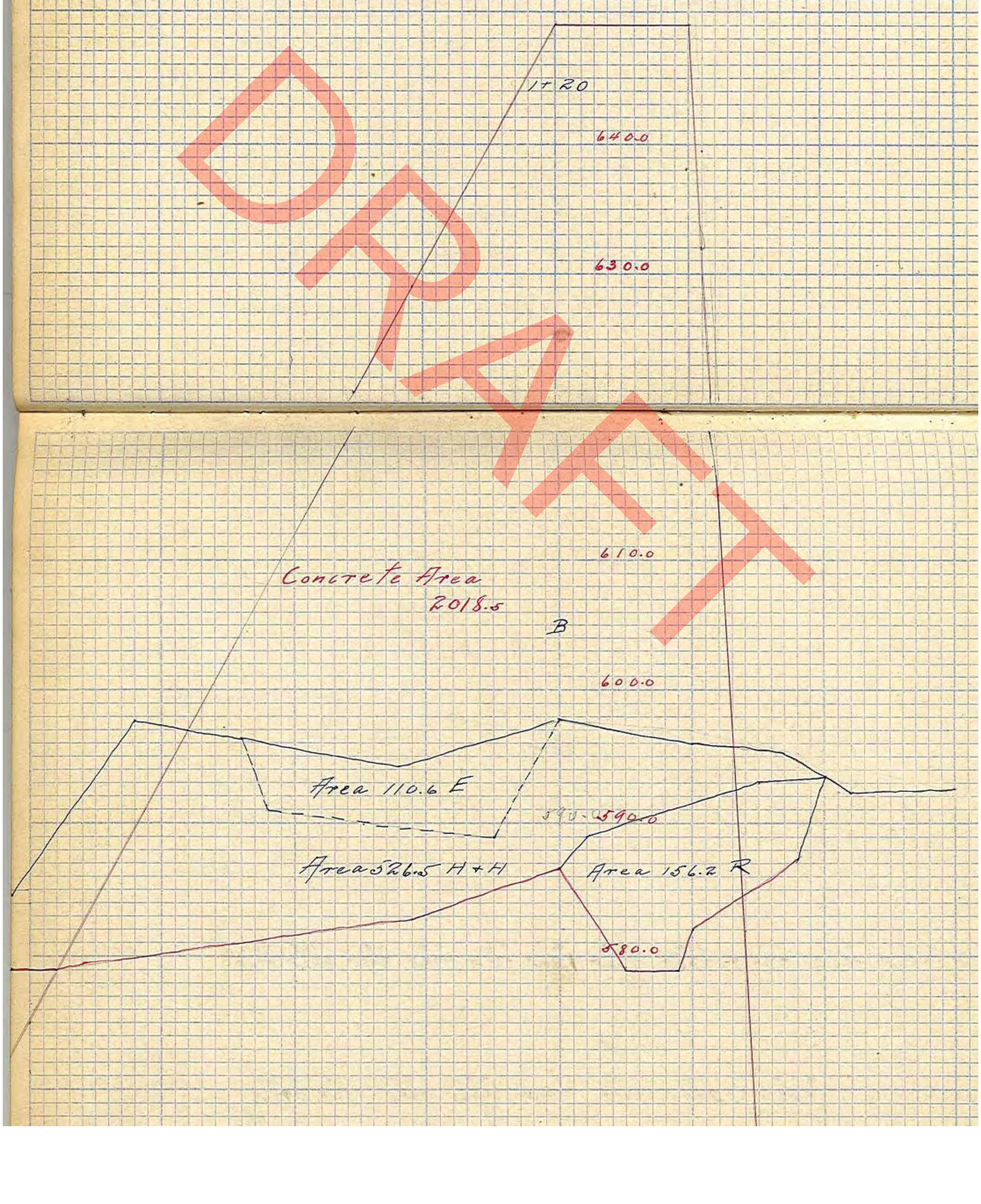


Sta. 1+20

583.0	584.6	597.7	594.2	597.8	596.0	595.2	592.2	592.5	
47.0	41.0	32.0	12.0	B	10.0	17.0	22.0	30.0	

V

584.4	579.0	579.0	582.8	586.6	579.0	579.0	582.0	581.4	593.5
42.0	41.0	38.0	11.0	B	5.0	9.0	10.0	18.0	20.0

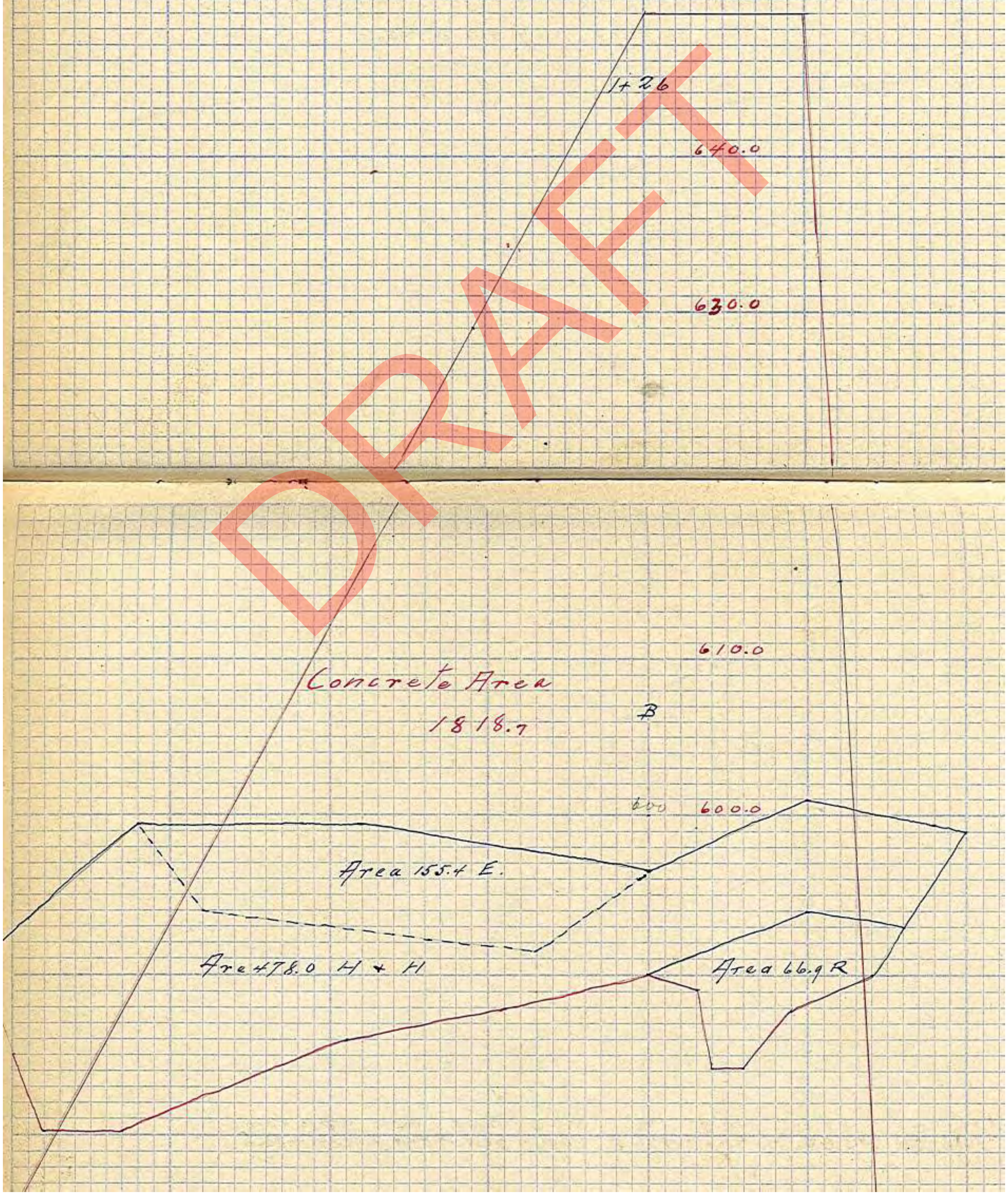


Sta 1+26

586.6	593.5	599.5	599.5	596.6	601.0	599.6	599.0
46.0	39.0	32.0	18.0	B	10.0	18.0	20.0

✓

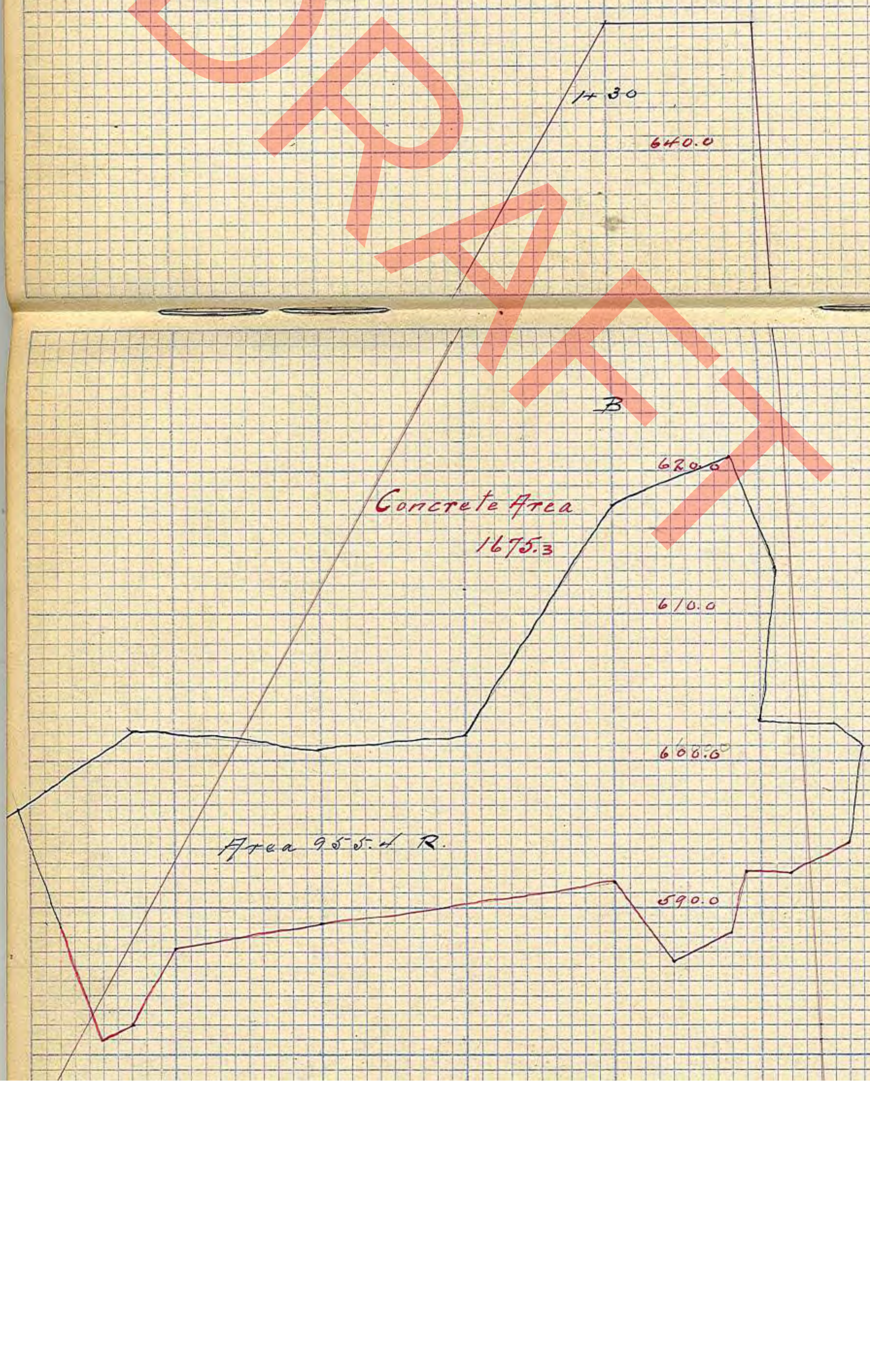
590.5	580.7	580.2	586.0	588.0	590.0	589.2	584.2	584.2	587.8
42.0	38.0	33.0	19.0	9.0	B	3.0	4.0	6.0	9.0



Sta. 1+30

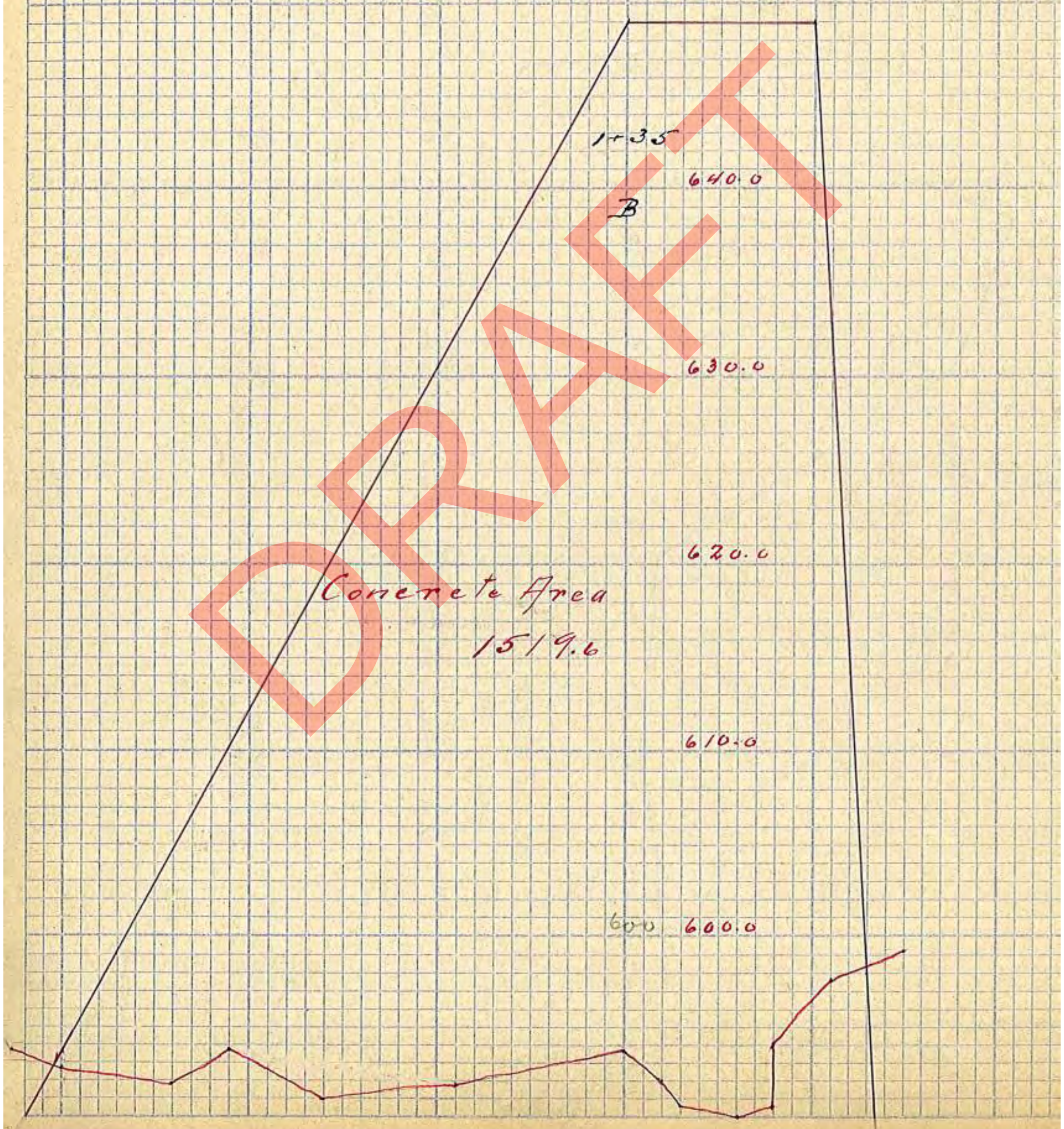
594.4	602.0	600.8	601.8	617.6	621.0	613.0	602.8	602.
44.0	33.0	20.0	10.0	B	8.0	11.0	10.0	15.

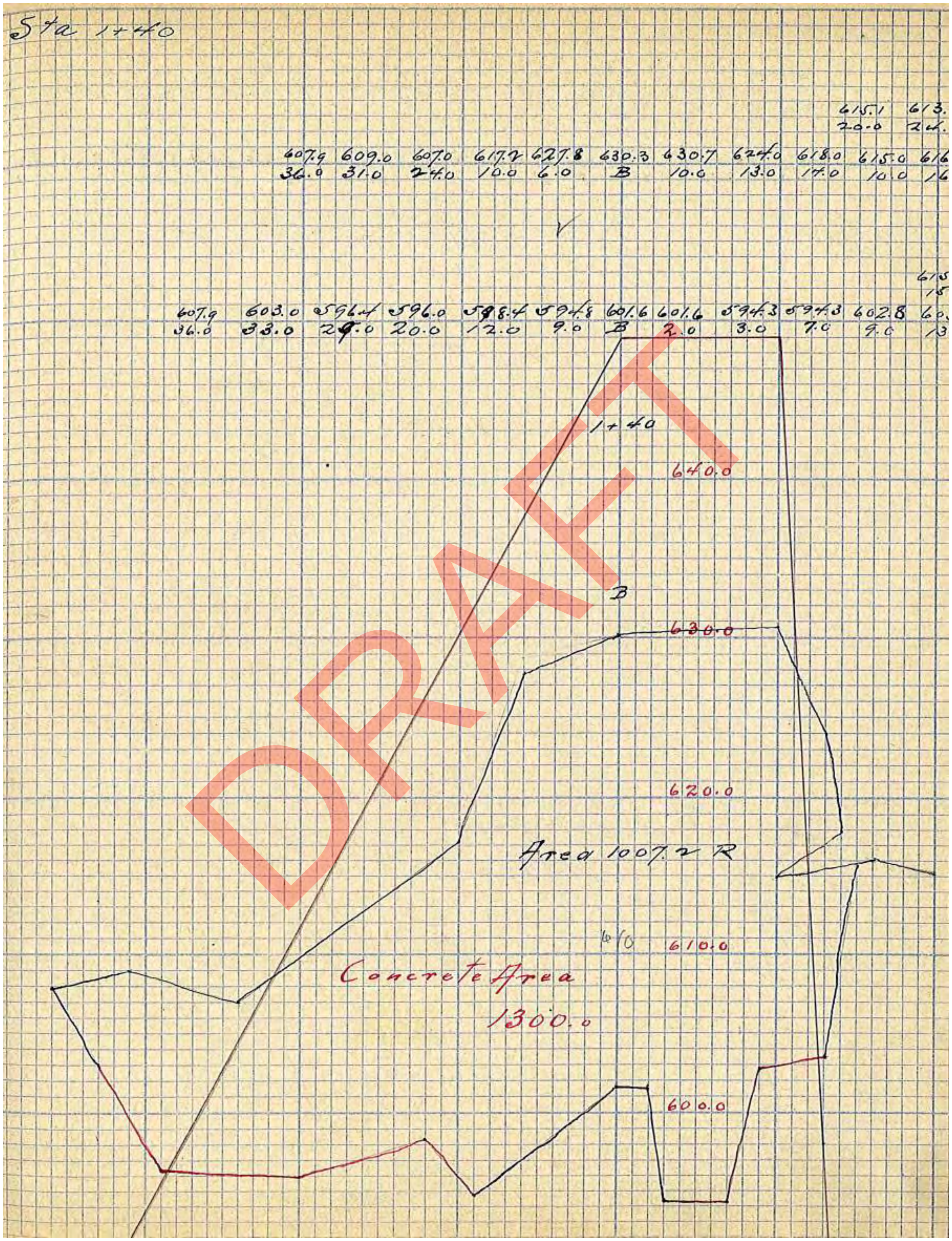
596.6	581.0	582.0	587.2	589.0	591.6	586.4	588.4	592.5	59
41.0	35.0	33.0	30.0	20.0	B	4.0	8.0	9.0	1



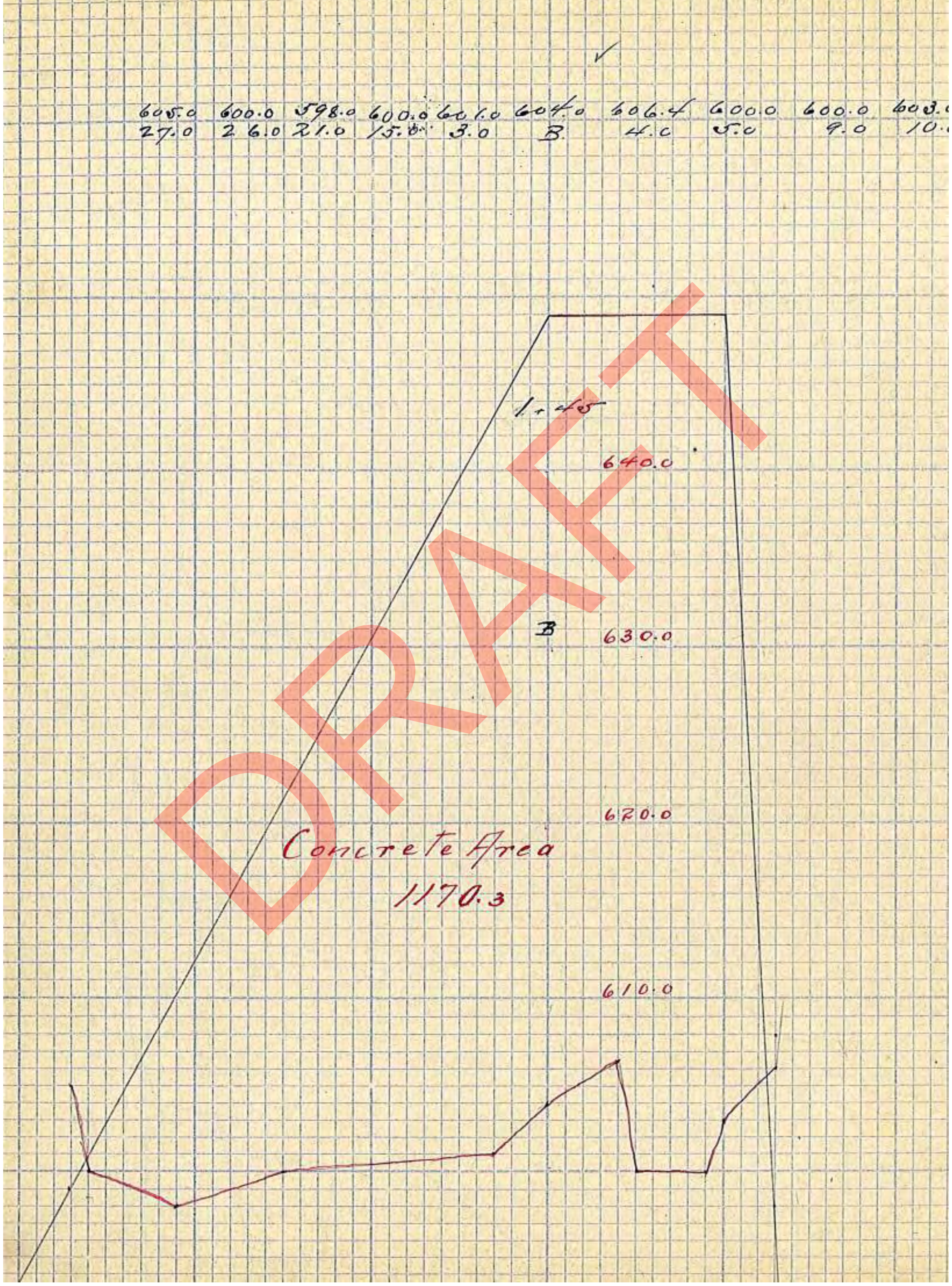
Sta 1+35

593.6	592.6	591.8	593.8	591.0	591.8	593.8	592.0	590.5	590.0	590.5
33.0	30.0	24.0	21.0	16.0	9.0	B	2.0	3.0	6.0	8.0




Sta 1+40
615.1 613.
20.0 24.
607.9 609.0 607.0 617.2 627.8 630.3 630.7 624.0 618.0 615.0 616
36.0 31.0 24.0 10.0 6.0 B 10.0 13.0 14.0 10.0 16
607.9 603.0 596.4 596.0 598.4 594.8 601.6 601.6 594.3 594.3 602.8 60
36.0 33.0 29.0 20.0 12.0 9.0 B 2.0 3.0 7.0 9.0 13
1+40
640.0
B
630.0
620.0
Area 1007.2 R
610.0
Concrete Area
1300.0
600.0

Sta. 1+45



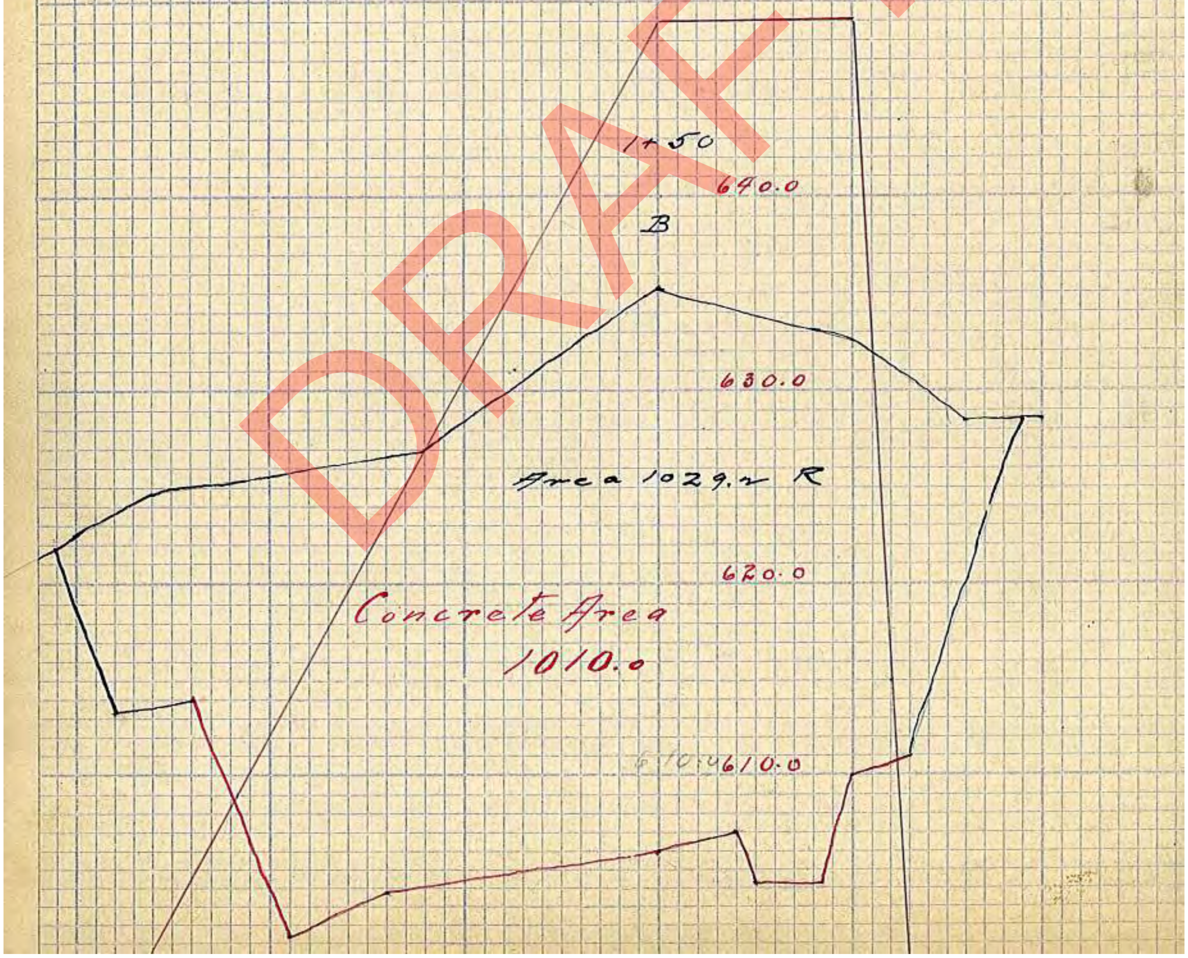
605.0 600.0 598.0 600.0 601.0 604.0 606.4 600.0 600.0 603.
27.0 26.0 21.0 15.0 3.0 B 4.0 5.0 9.0 10.
1+45
640.0
B
630.0
620.0
Concrete Area
1170.3
610.0

Sta 1+50

622.4	624.6	627.0	635.3	632.6	628.5	628.7
30.0	26.0	12.0	B	10.0	16.0	20.0

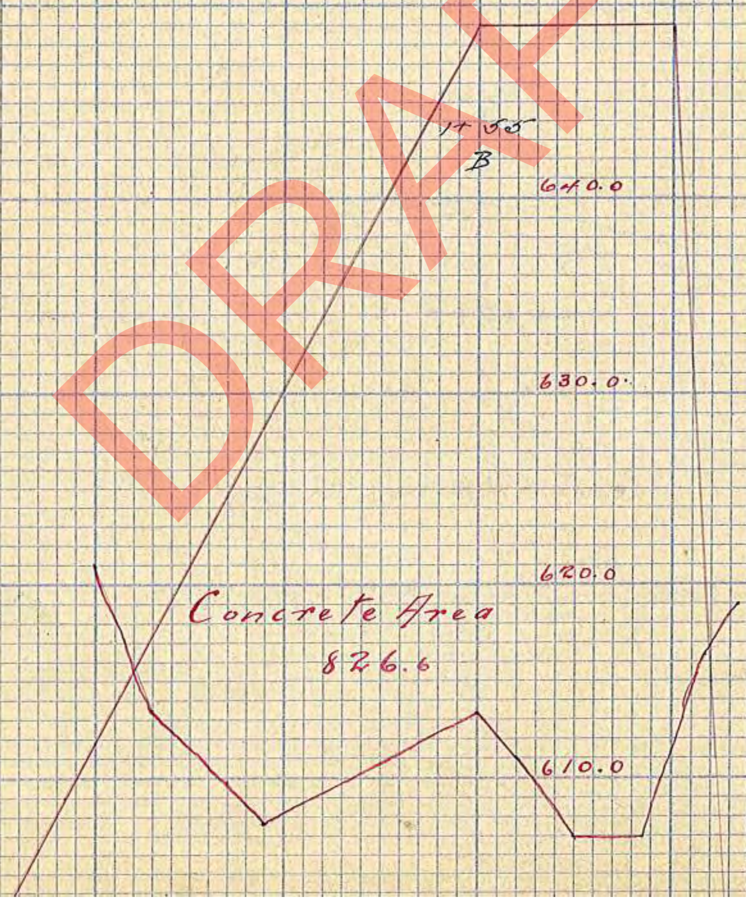
✓

618.8	613.3	614.0	601.4	603.7	606.0	607.0	604.3	604.3	610.0	611.0
30.0	28.0	24.0	19.0	14.0	B	4.0	5.0	8.5	10.0	13.0



Sta. 1+55

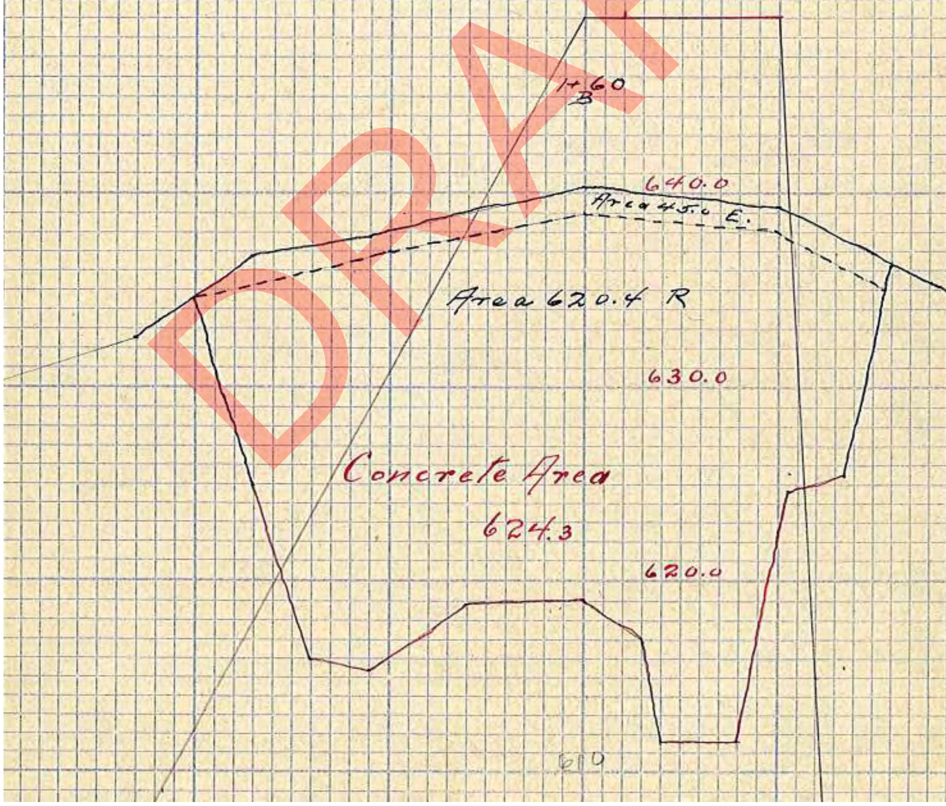
621.0	613.5	607.6	613.3	611.6	607.0	607.0	616.4
20.0	17.0	11.0	B	2.0	5.0	8.6	11.6



Sta. 1+60

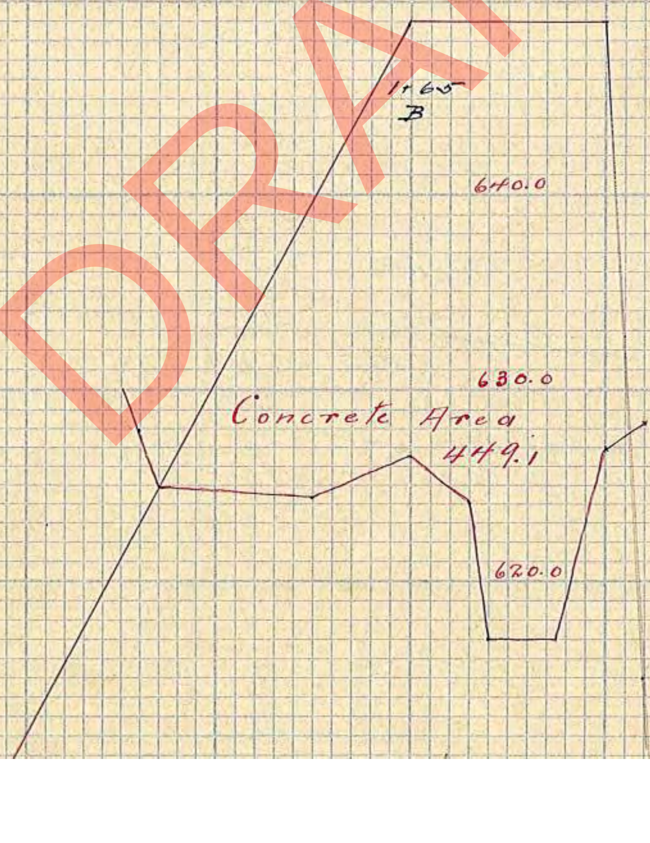
629.7	632.5	636.8	637.8	640.2	639.4	636.9	635.0
32.0	23.0	17.0	11.0	B	10.0	15.0	19.0

634.5	625.0	616.0	615.4	618.8	619.0	617.0	611.6	611.6	624.6
20.0	17.0	14.0	11.0	6.0		3.0	4.0	8.0	10.5



Sta 1+65

628.0	625.0	624.3	626.6	624.0	617.0	617.0	6[illegible]
14.0	13.0	5.0	B	3.0	4.0	7.5	1[illegible]

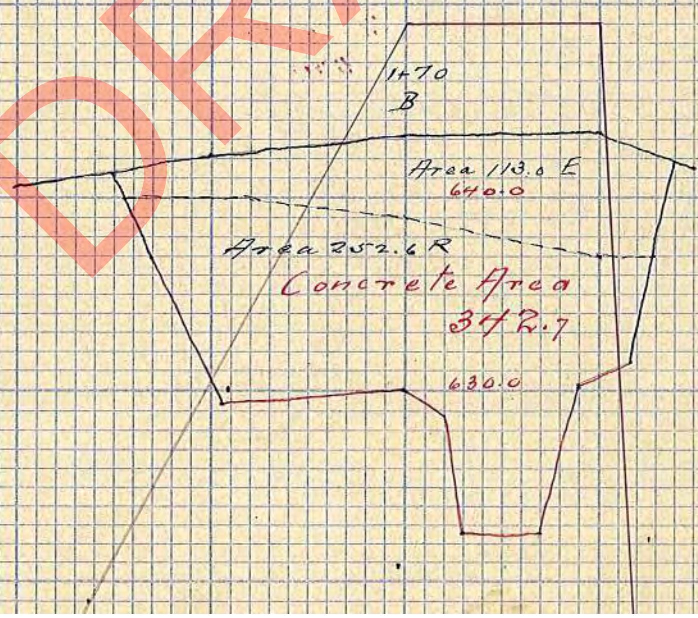


Sta 1+70

640.5	642.1	643.2	643.3	641.5
20.0	10.0	B	10.0	15.0

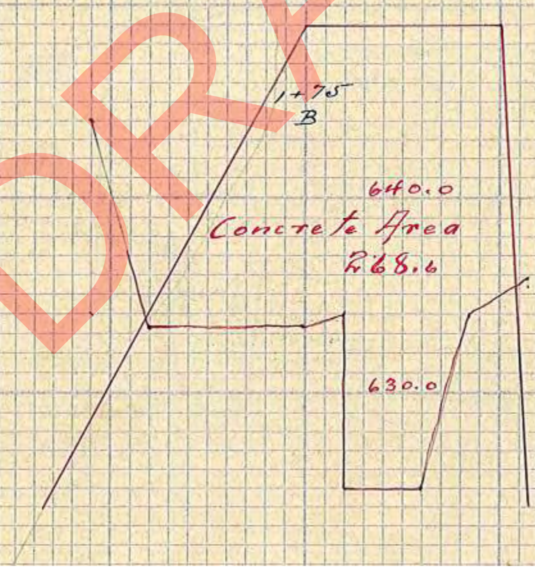
✓

641.2	637.0	629.4	634.0	628.7	622.5	622.5	623.0
15.0	13.0	9.5	B	2.0	3.0	7.0	9.



Sta 1+75

644.0	633.4	633.4	634.0	628.0	625.0	634.0	635.
11.0	8.0		2.0	2.0	6.0	8.5	11.3

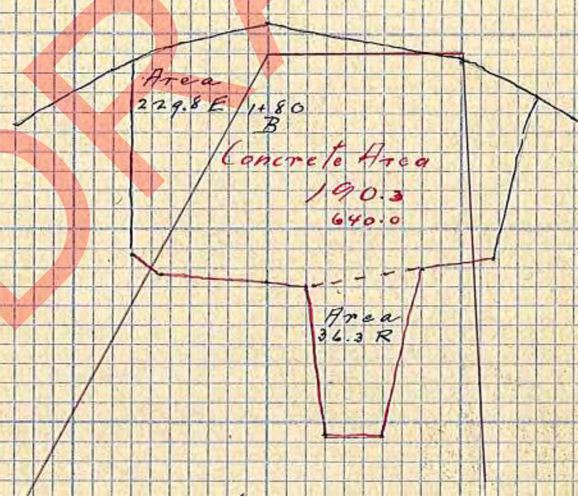


Sta 1+80

645.4	648.1	649.4	650.5	648.8	647.4	645.8
13.0	8.0	5.0	B	10.0	13.0	16.0

646.
14

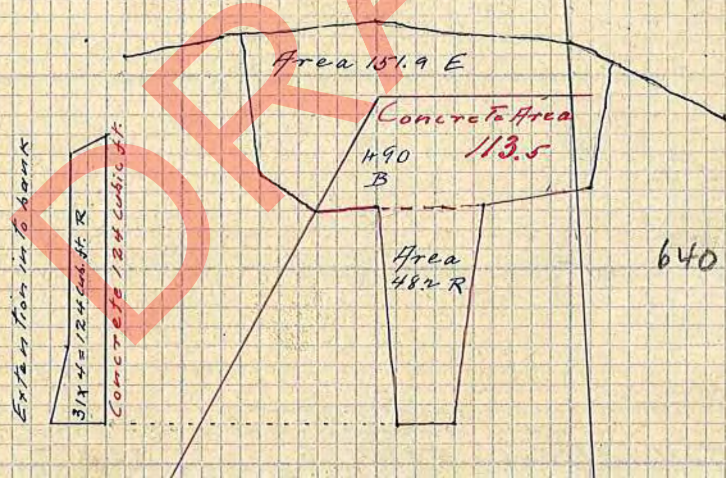
644.0	638.7	637.6	637.2	637.0	629.4	629.4	638.0	638.
7.0	7.0	5.5	B	2	3.0	6.0	8.0	11.6



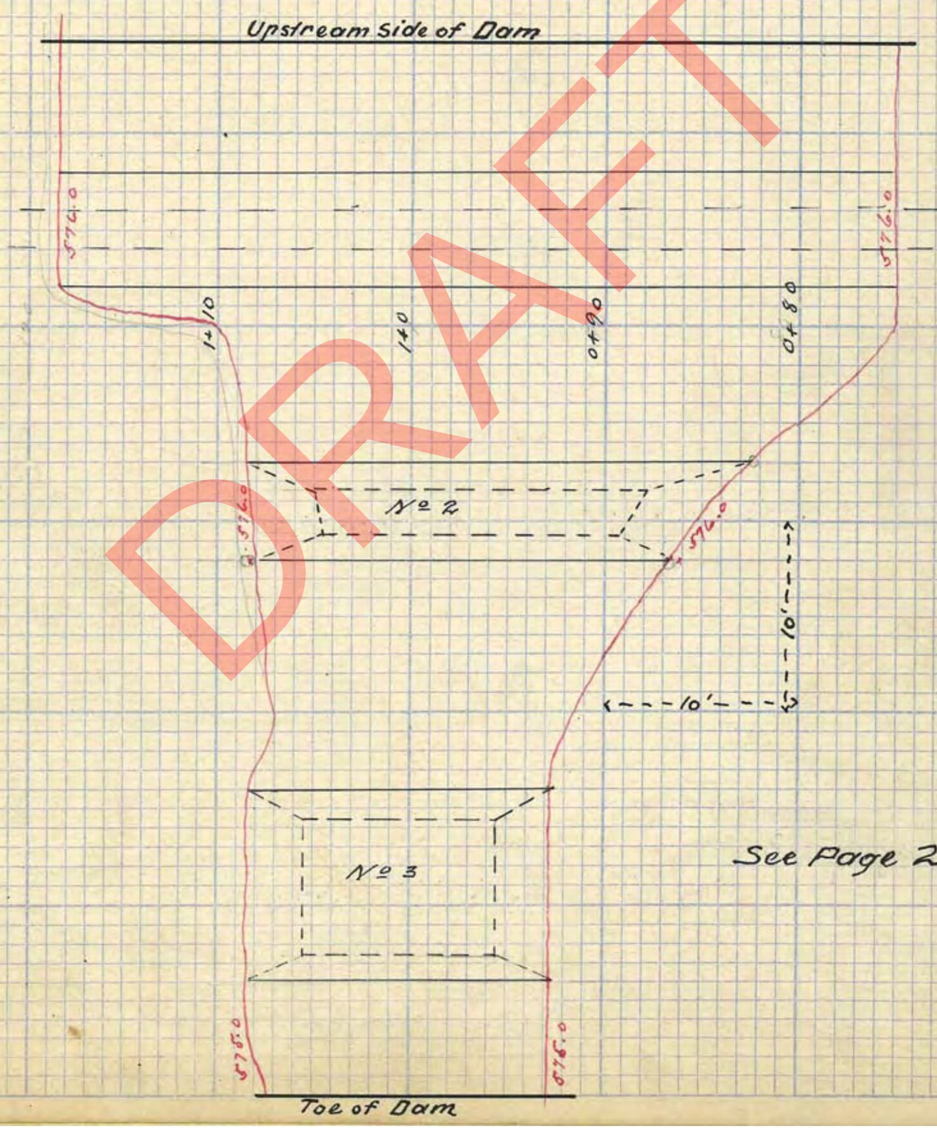
Sta 1+90

651.0	652.0	652.8	651.7	649.8	647.7
13.0	8.0	B	10.0	14.0	18.0

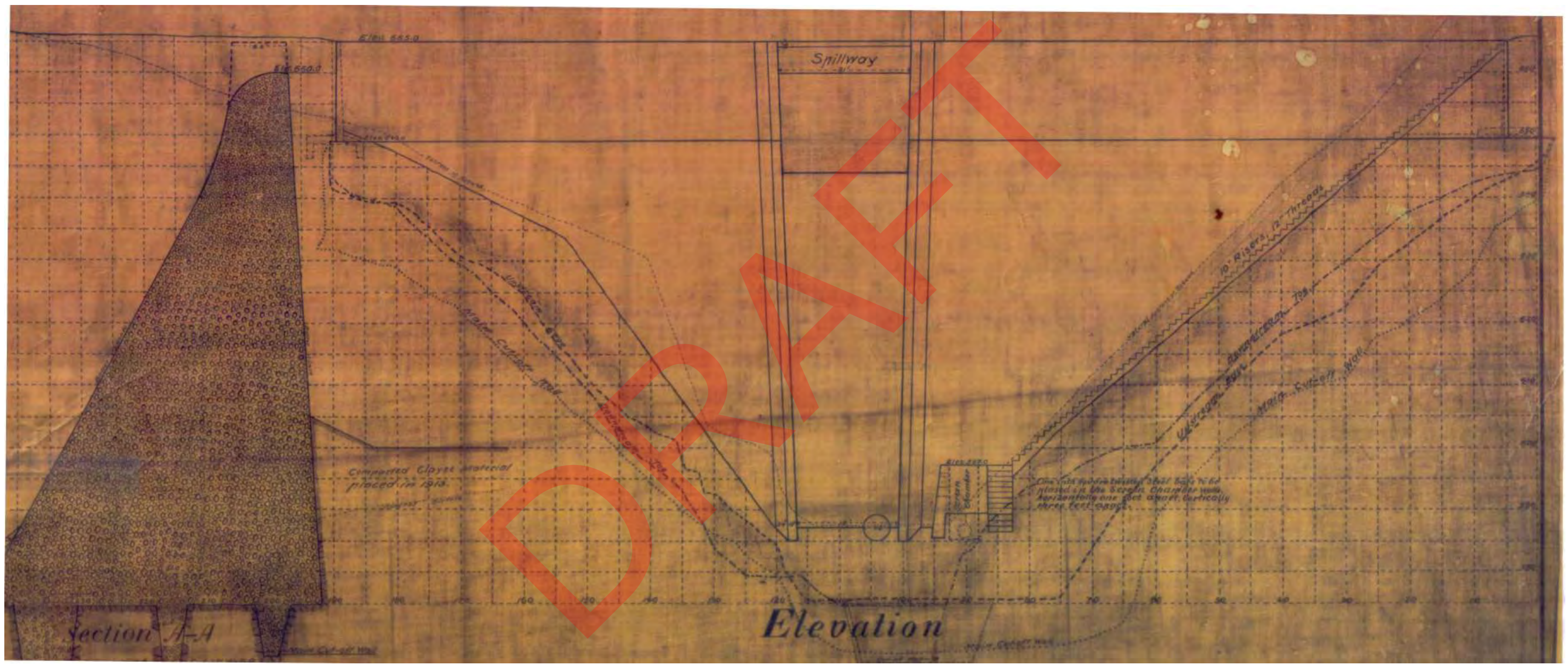
654.0	645.0	643.0	643.1	632.0	632.0	643.4	644.1	650.5
7.0	6.0	3.0	B	1.0	4.0	5.5	11.0	12.[illegible]

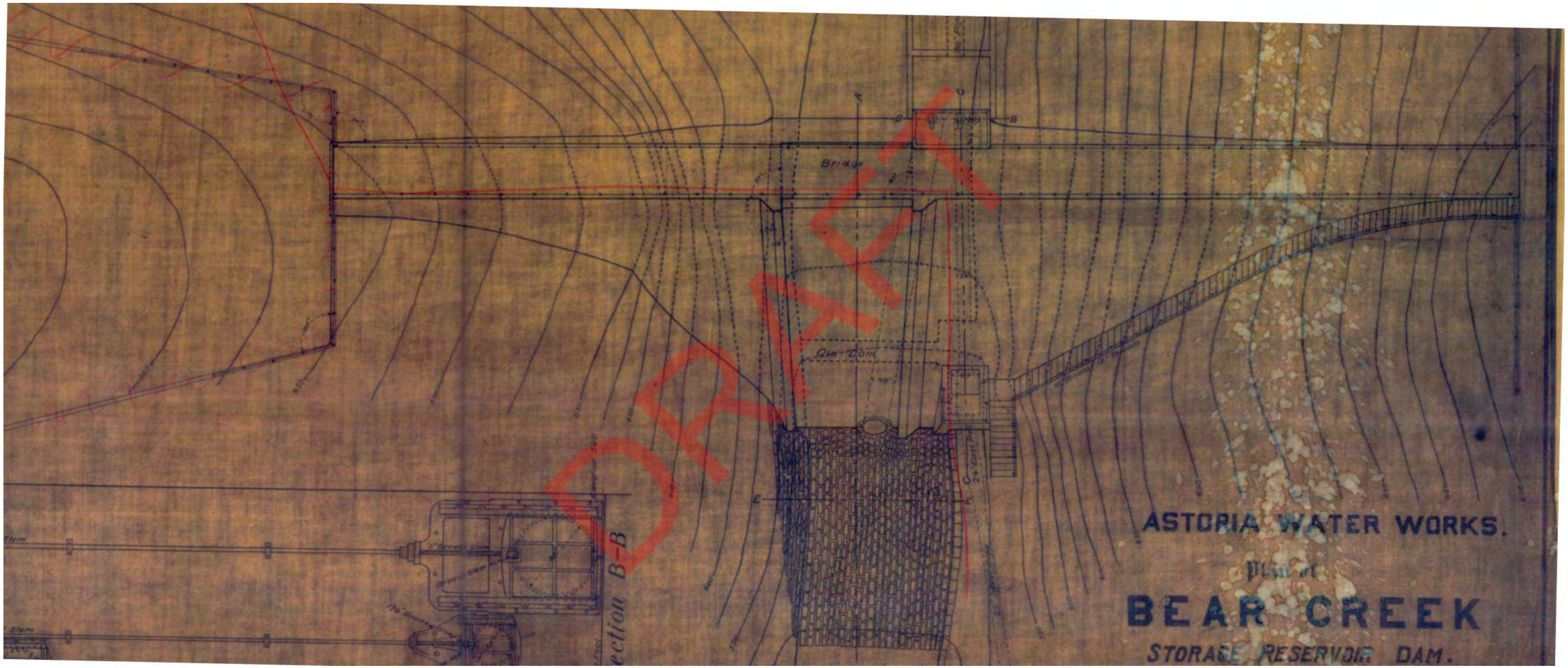


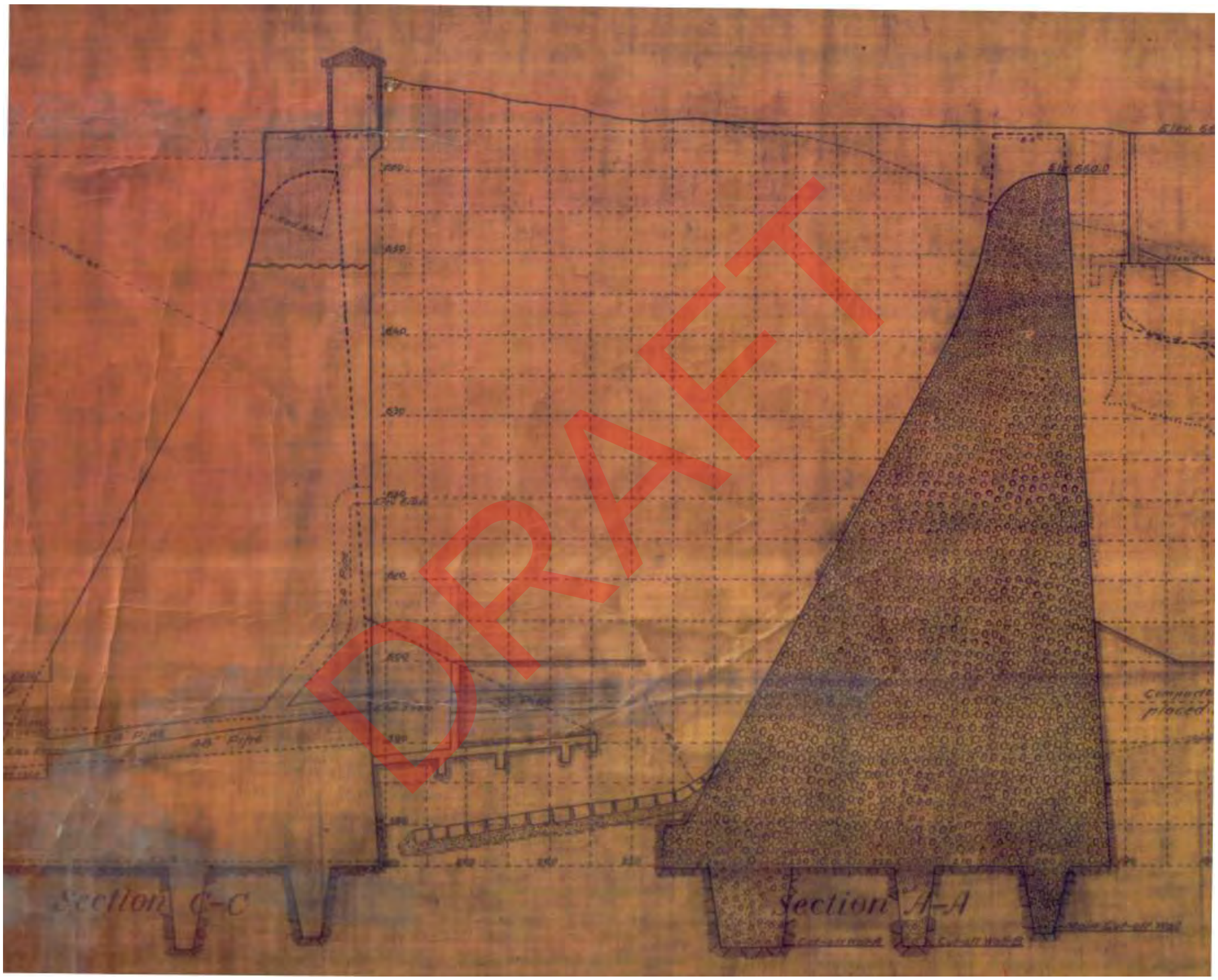
Plan Showing Area of the Sandstone Formation of the Dam Foundation. Also Showing location of the Cut-off Trenches.



See Page 23


Spillway
10' Risers, 12' Threads
Compacted Clayey material
placed in 1913
Section A-A
Elevation


ASTORIA WATER WORKS.
Plan of
BEAR CREEK
STORAGE RESERVOIR DAM.
Bridge
Section B-B
Stem

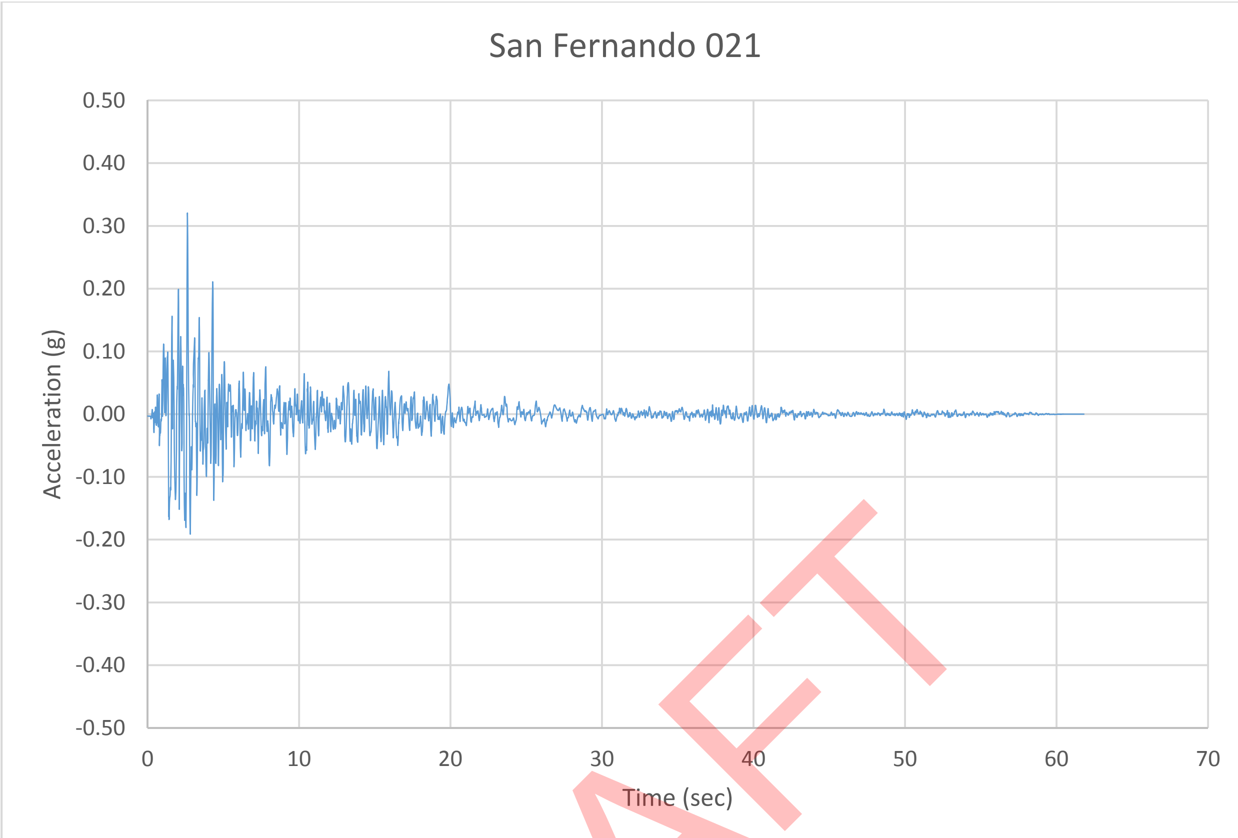

Section C-C
Section A-A
Main Cut-off Wall

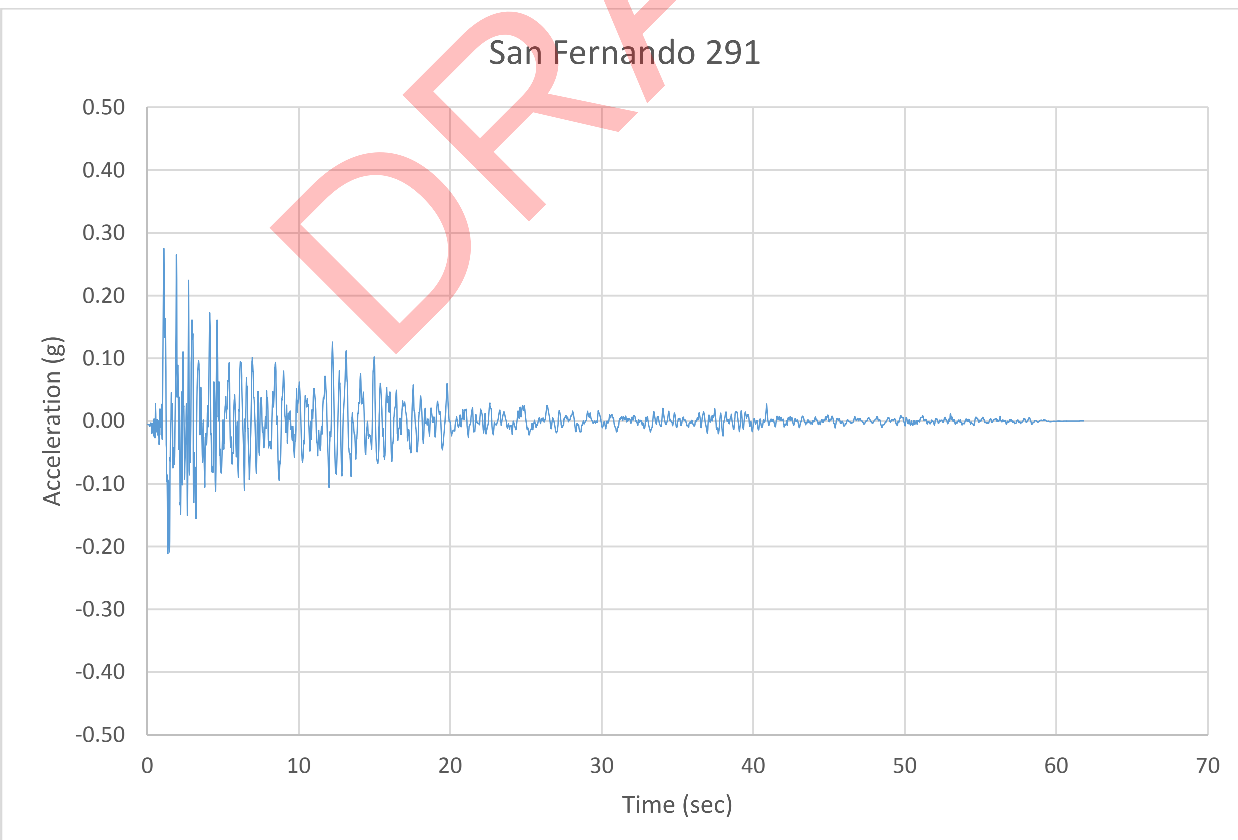
Appendix D

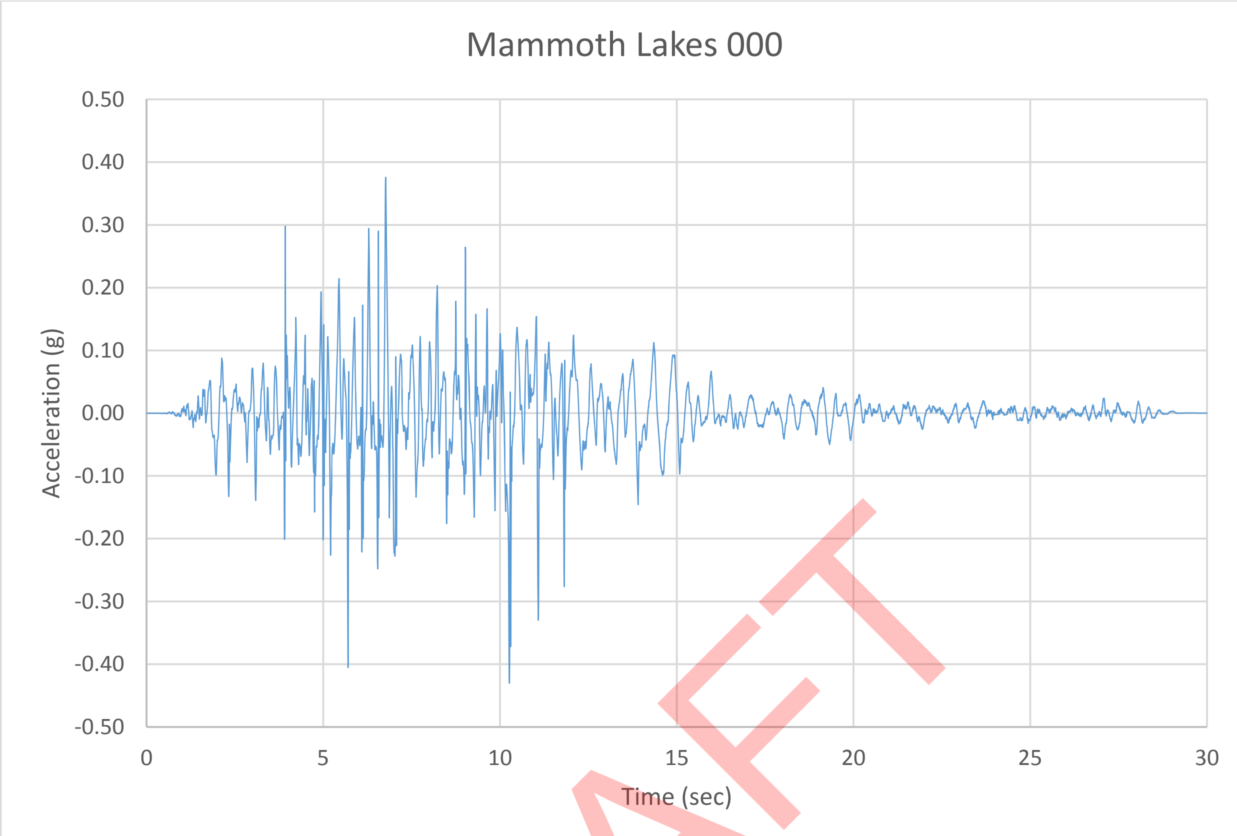
Earthquake Acceleration Time Histories
(Unscaled)

Appendix E

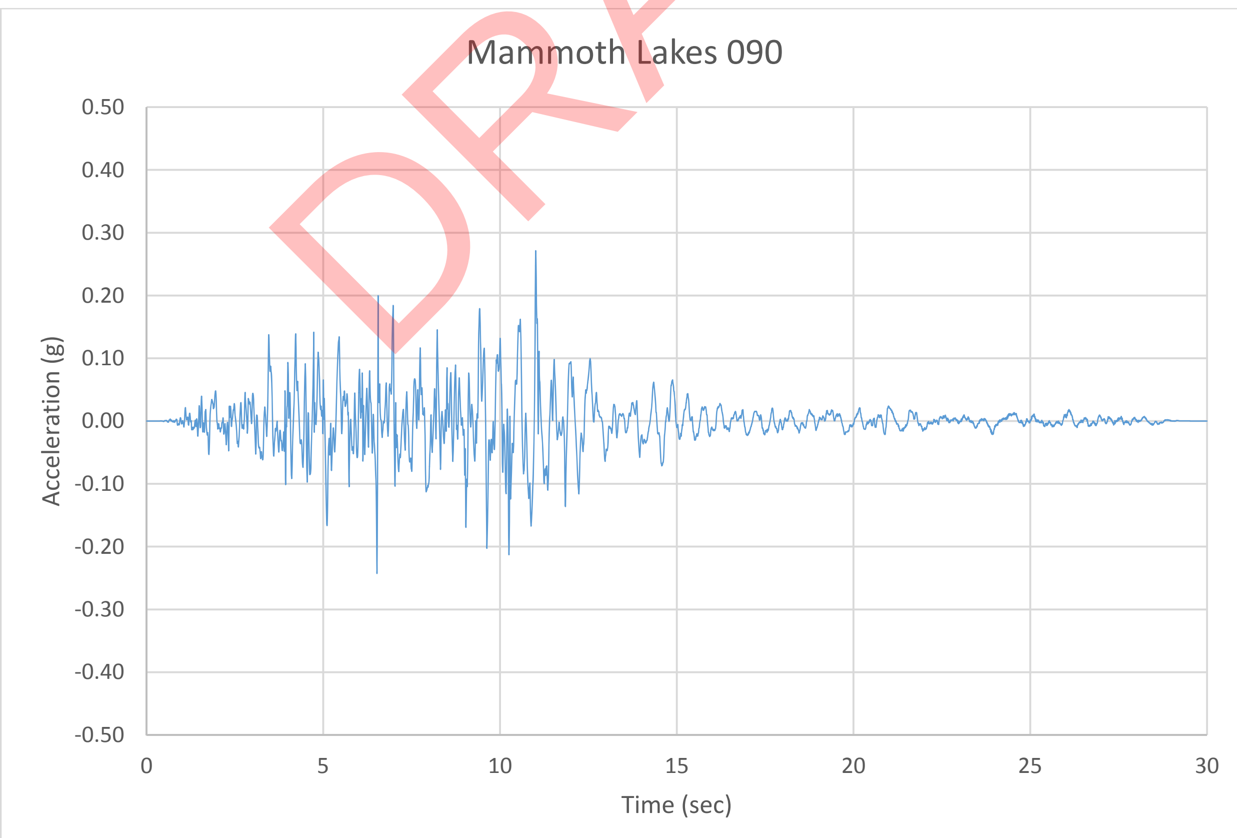
Multi-Wedge Analysis Equations


San Fernando 021
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
10
20
30
40
50
60
70
Time (sec)

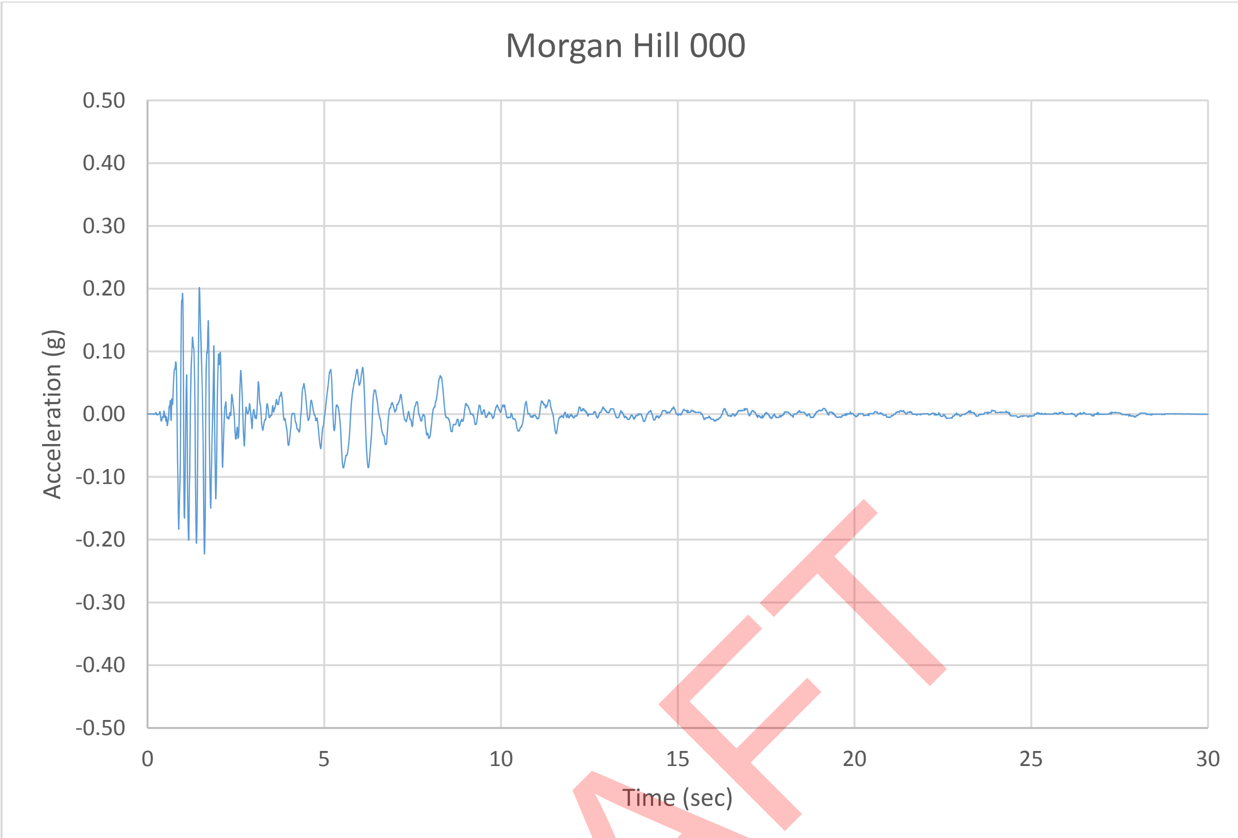

San Fernando 291
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
10
20
30
40
50
60
70
Time (sec)

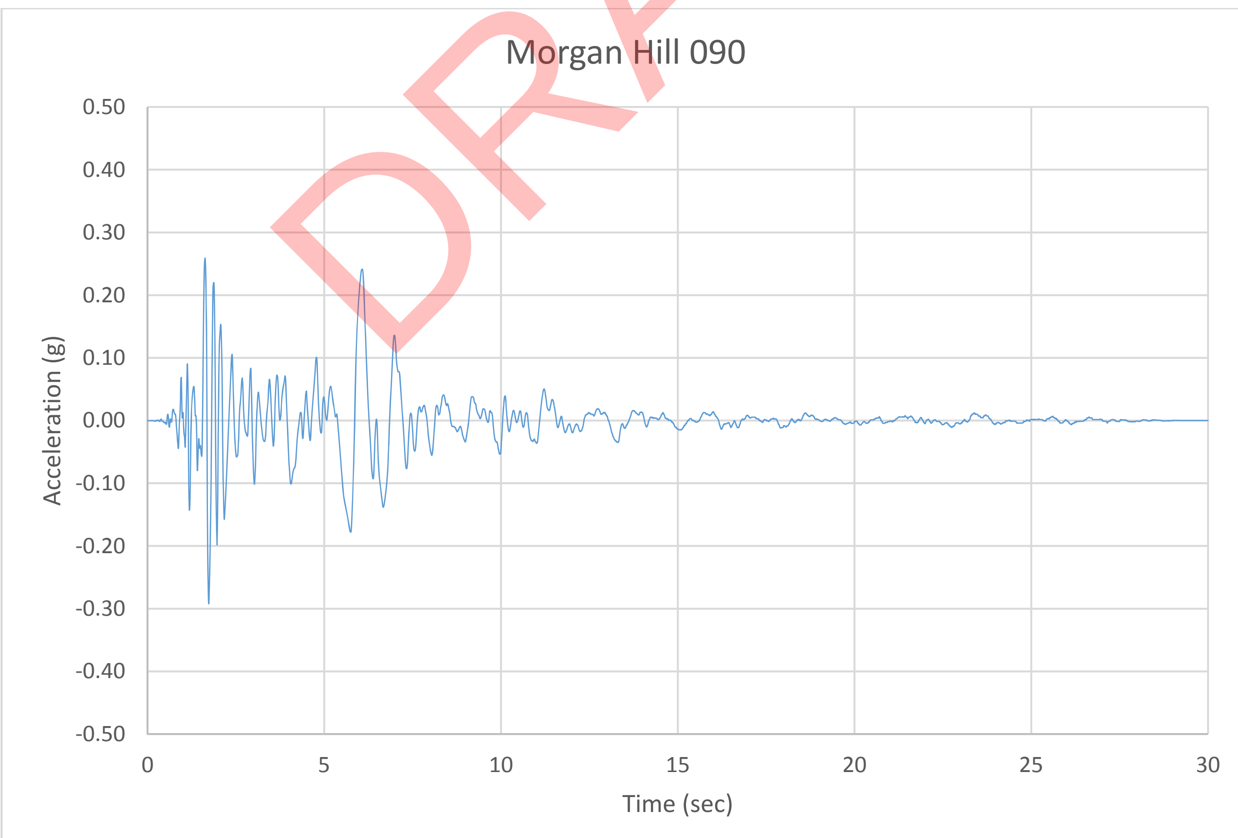


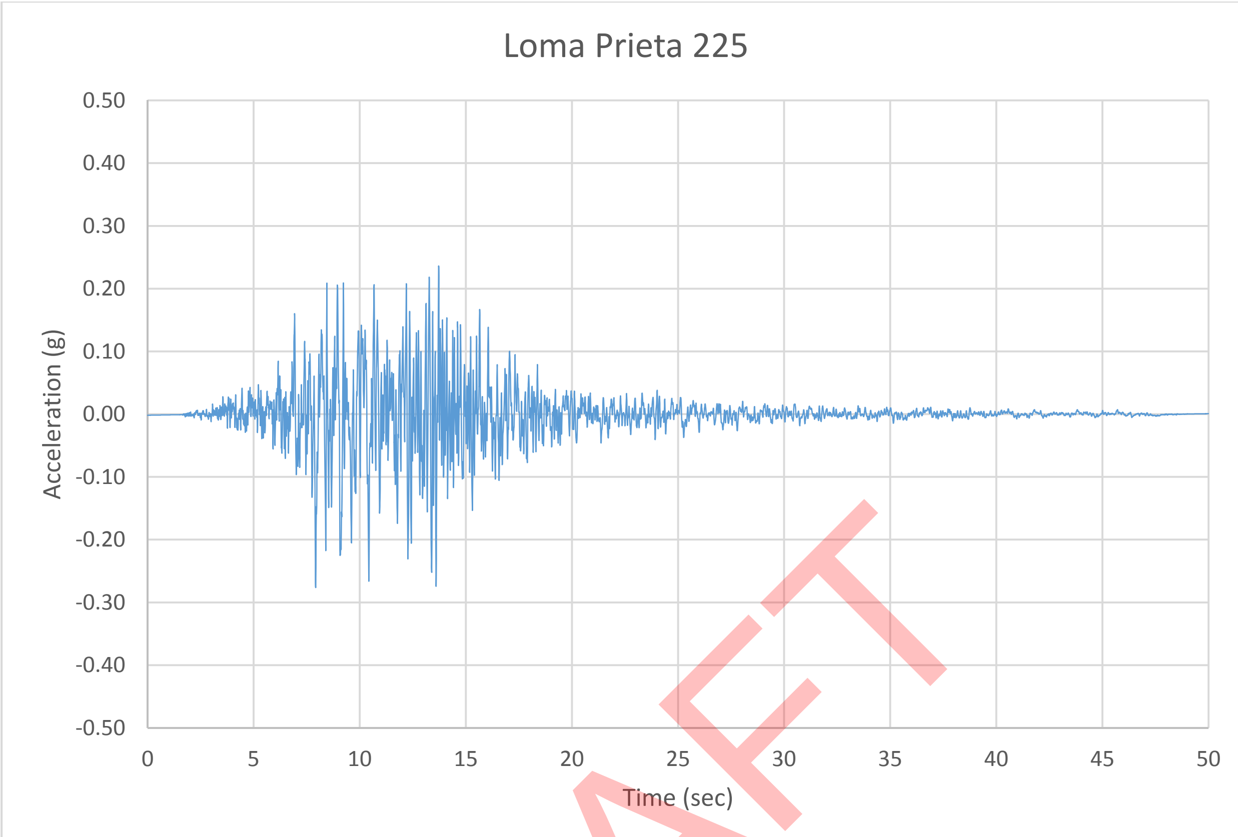
Mammoth Lakes 000
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
5
10
15
20
25
30
Time (sec)

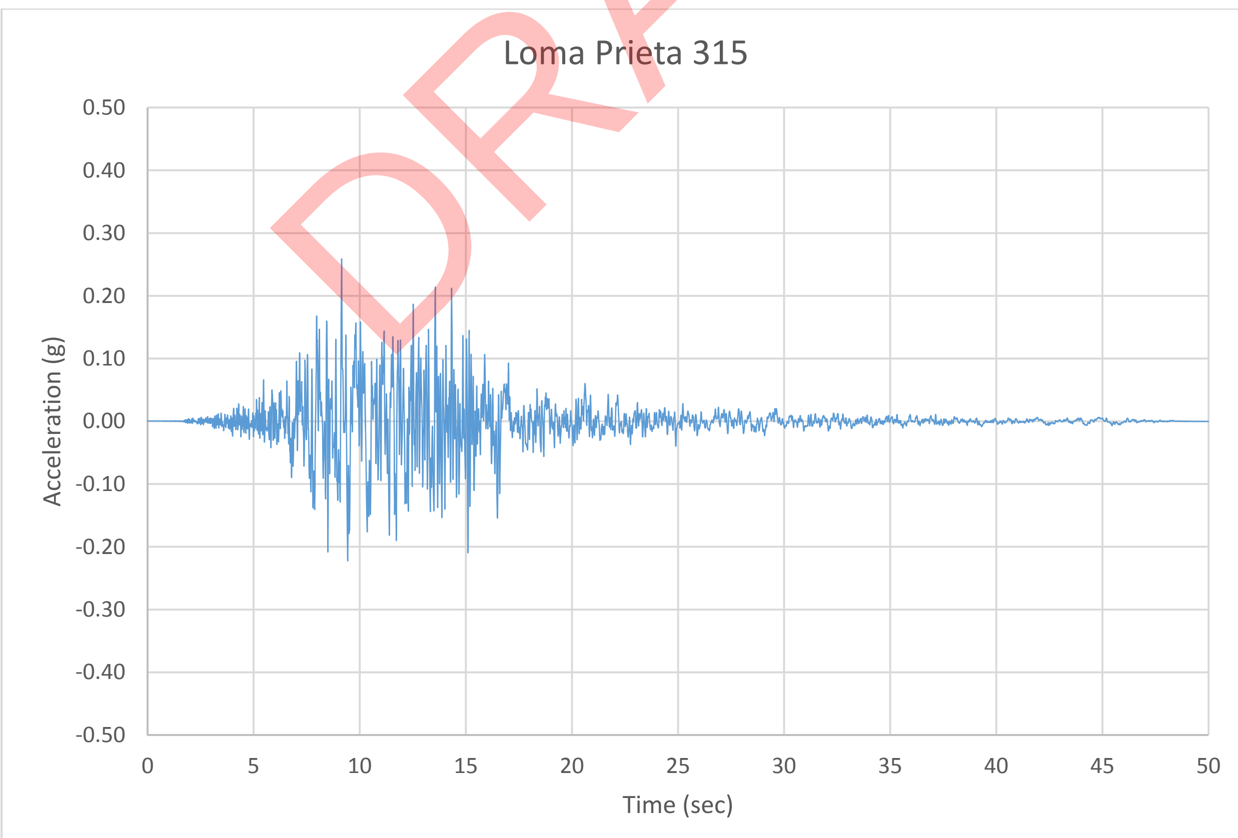


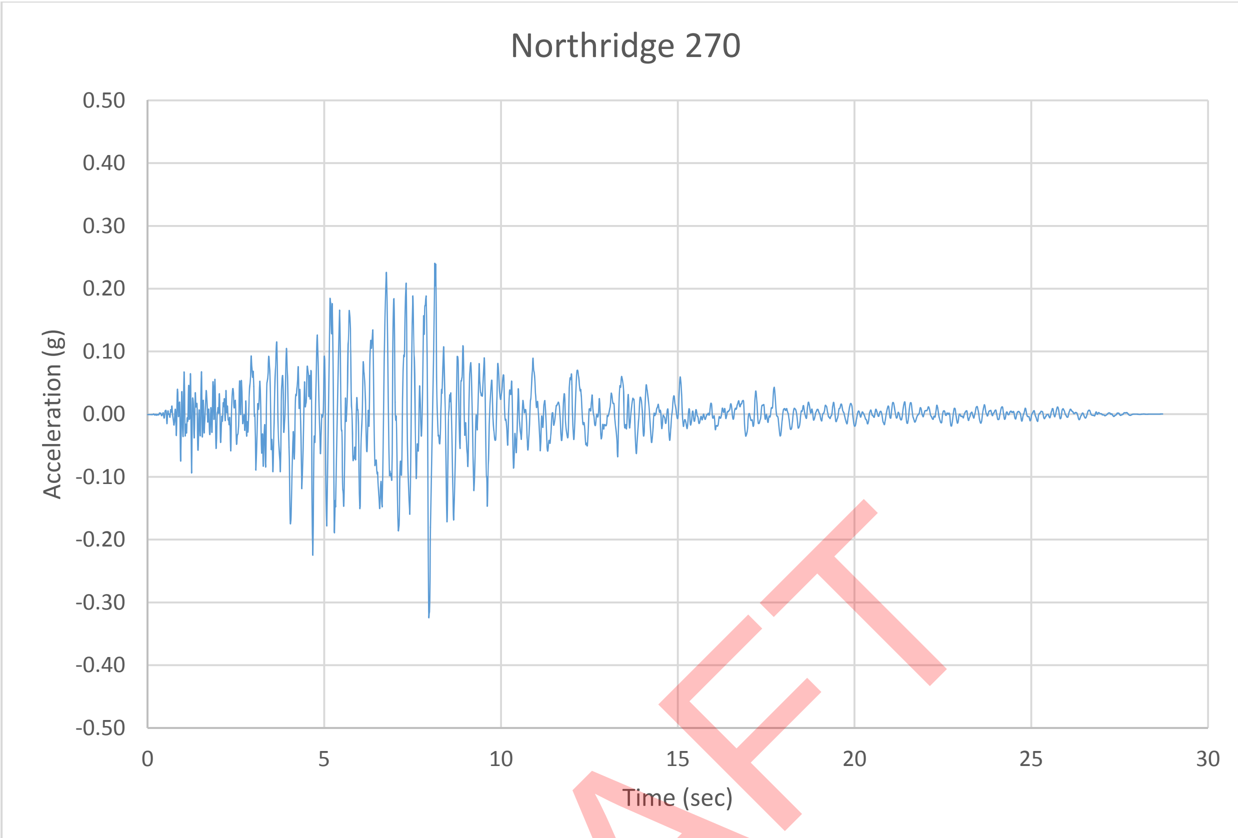
Mammoth Lakes 090
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
5
10
15
20
25
30
Time (sec)

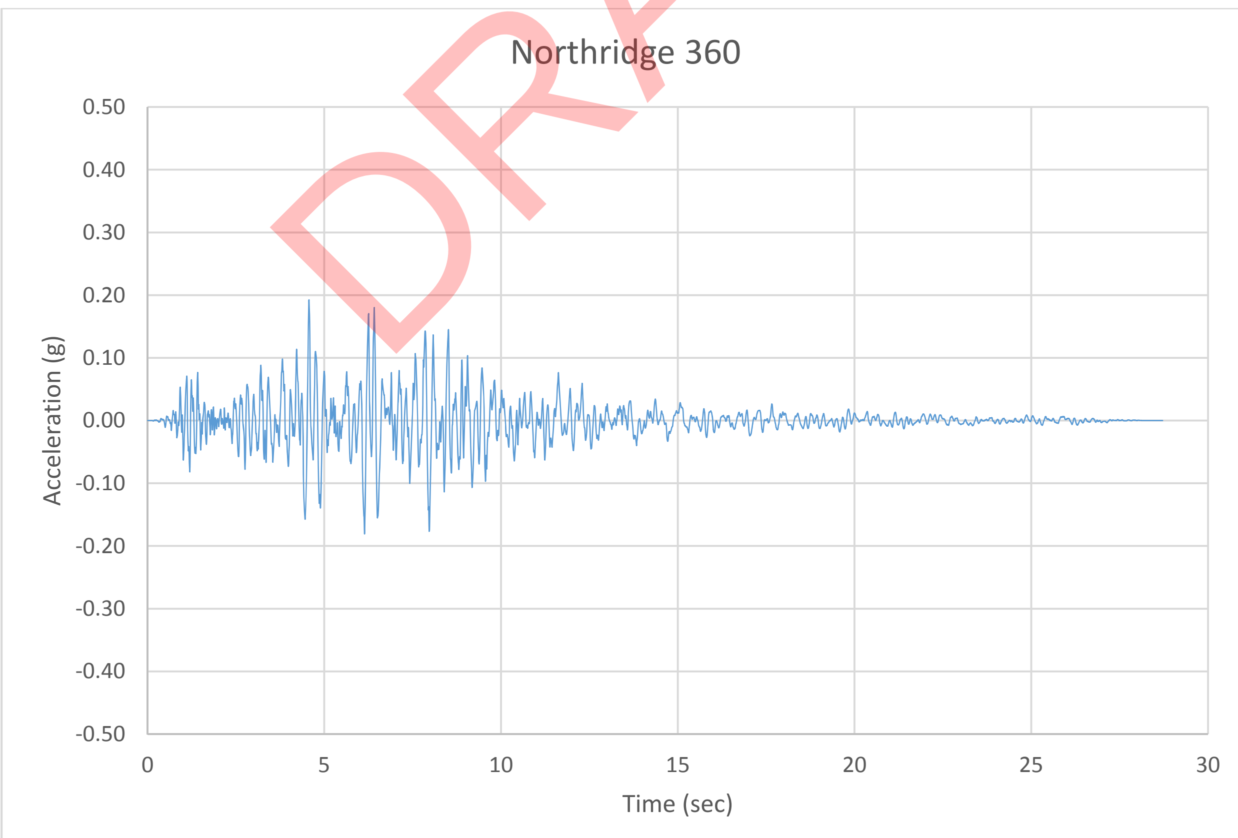

Morgan Hill 000
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
5
10
15
20
25
30
Time (sec)


Morgan Hill 090
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
5
10
15
20
25
30
Time (sec)

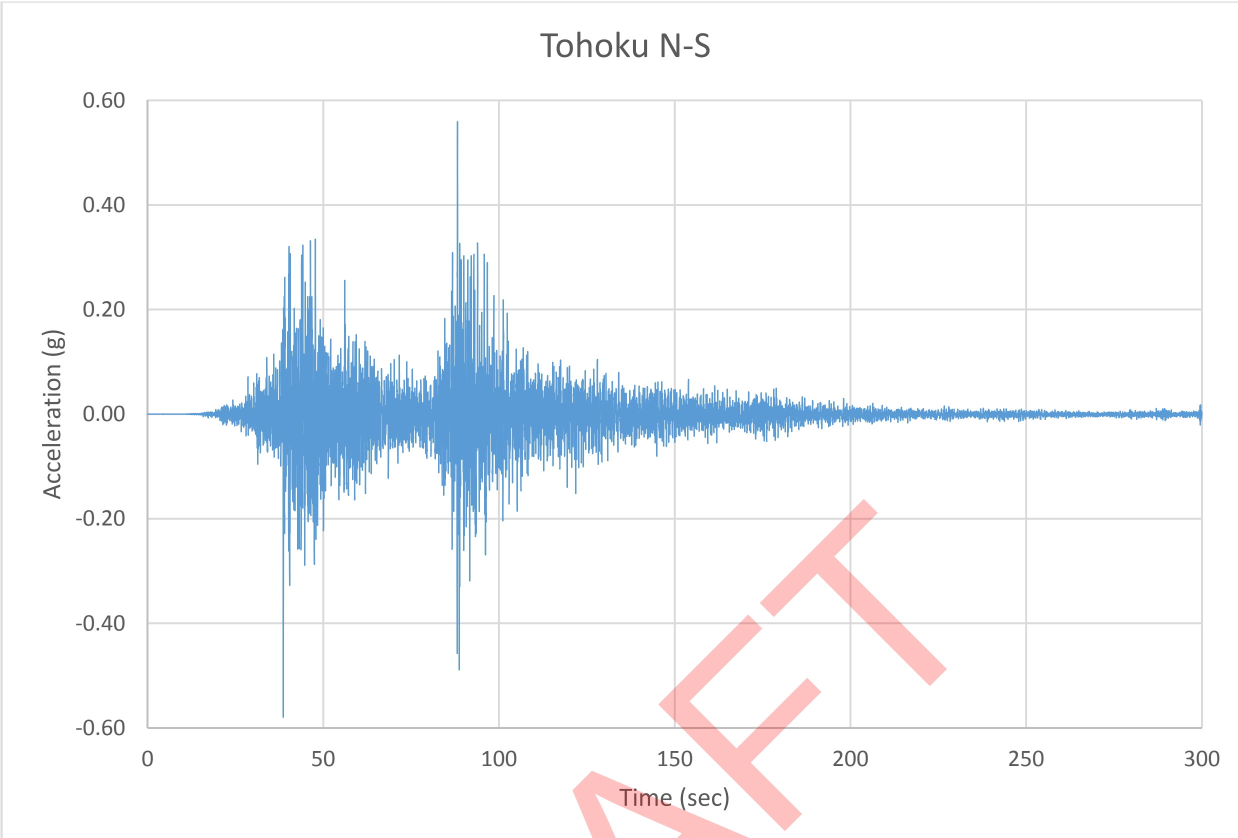

Loma Prieta 225
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
5
10
15
20
25
30
35
40
45
50
Time (sec)

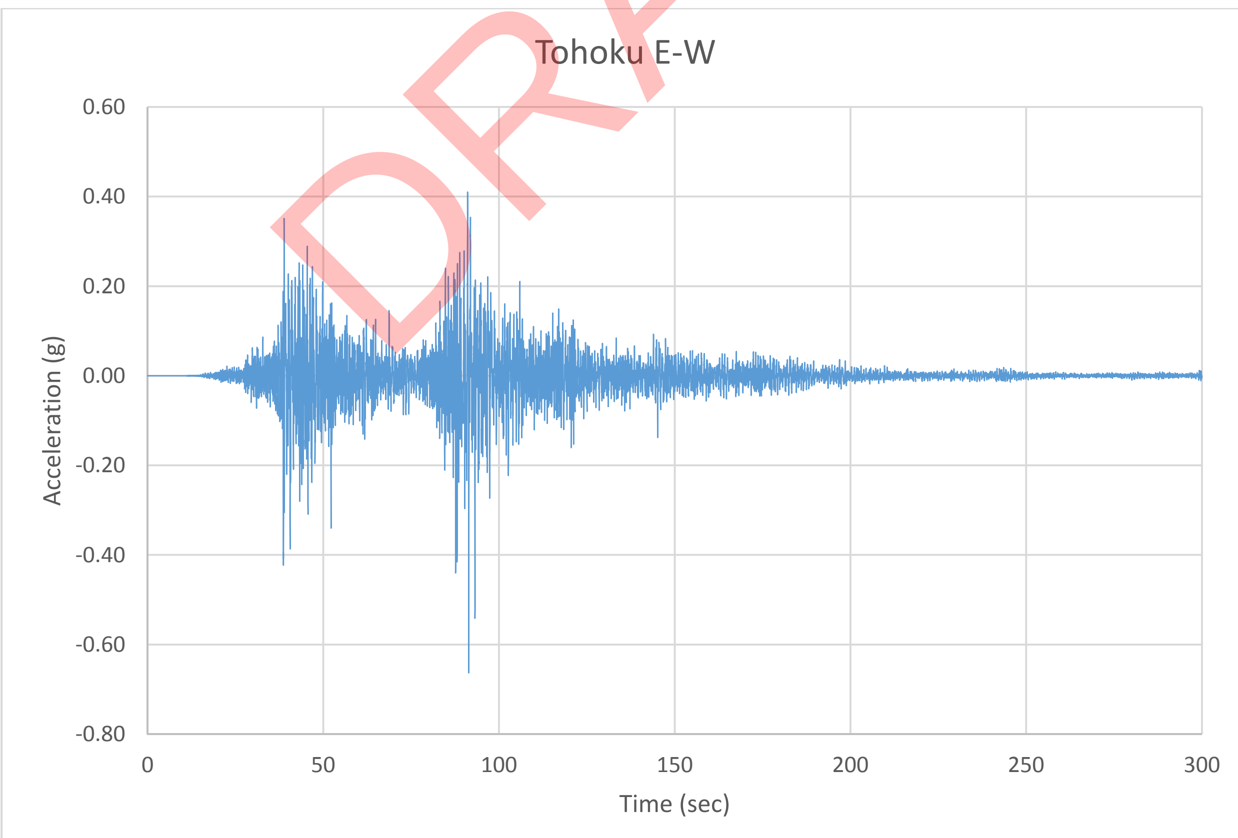

Loma Prieta 315
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
5
10
15
20
25
30
35
40
45
50
Time (sec)

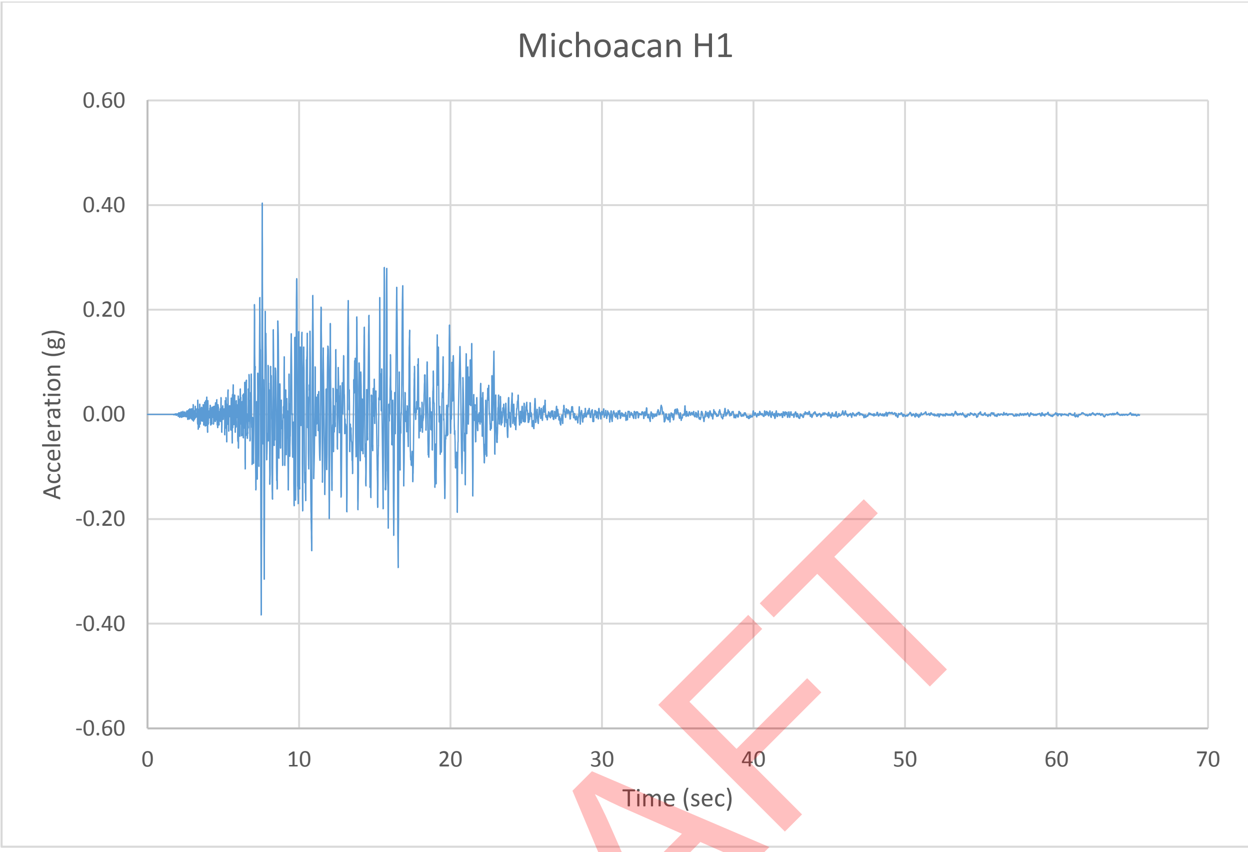

Northridge 270
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
5
10
15
20
25
30
Time (sec)

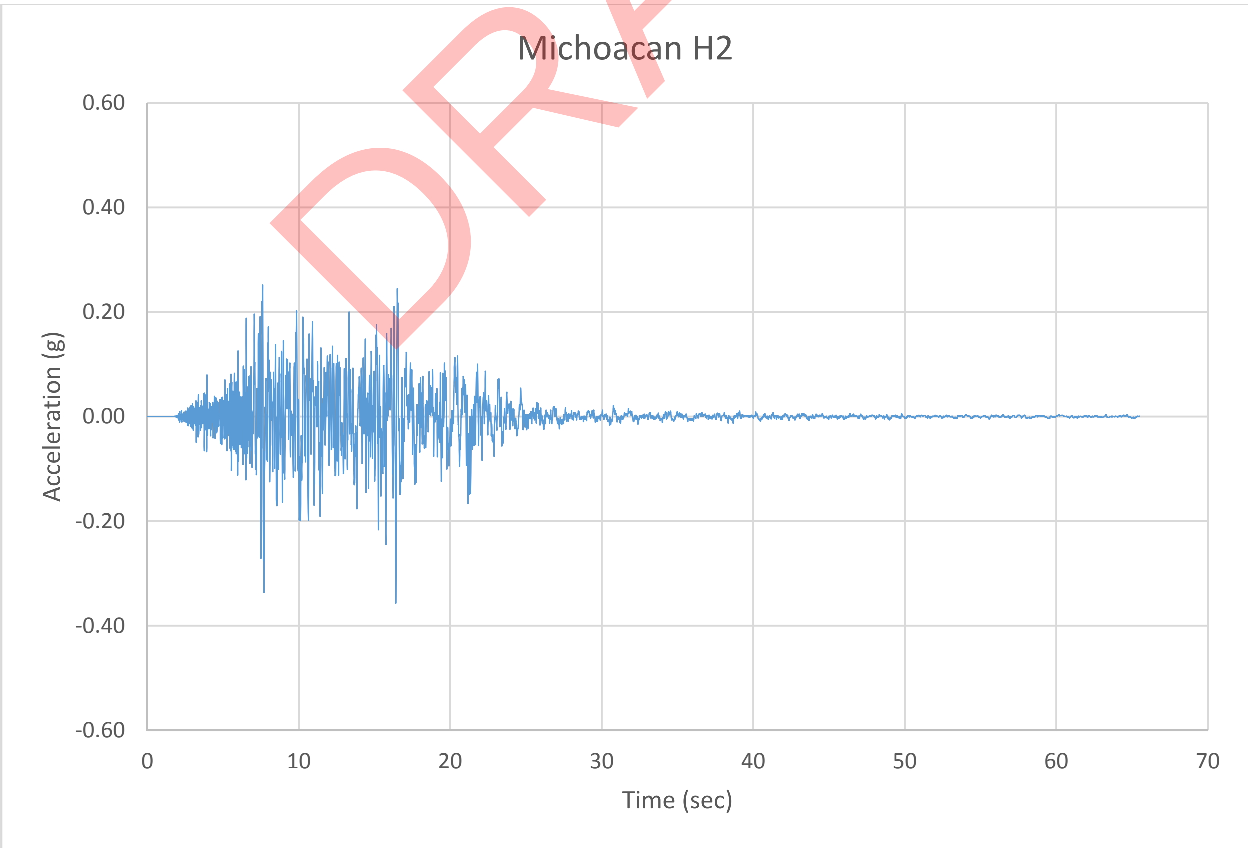

Northridge 360
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
5
10
15
20
25
30
Time (sec)

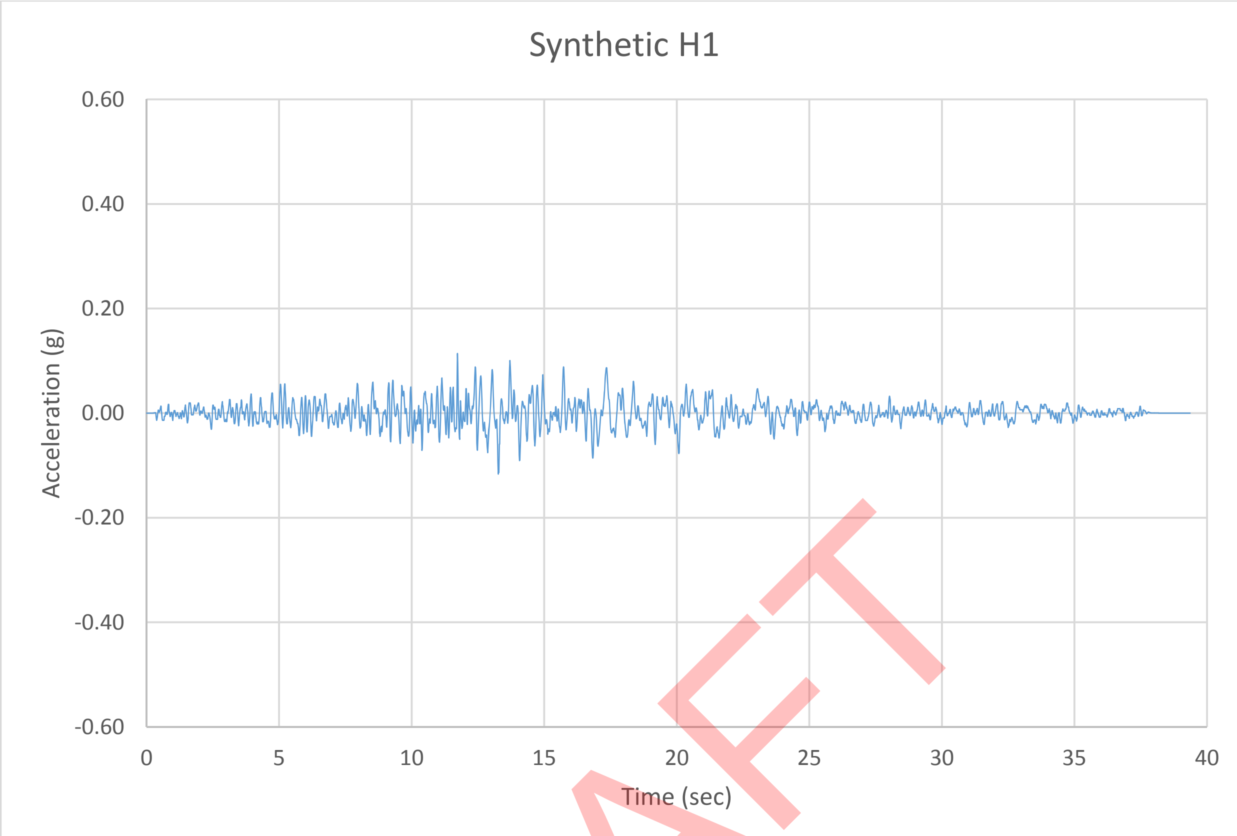
CSZ Interface Motions

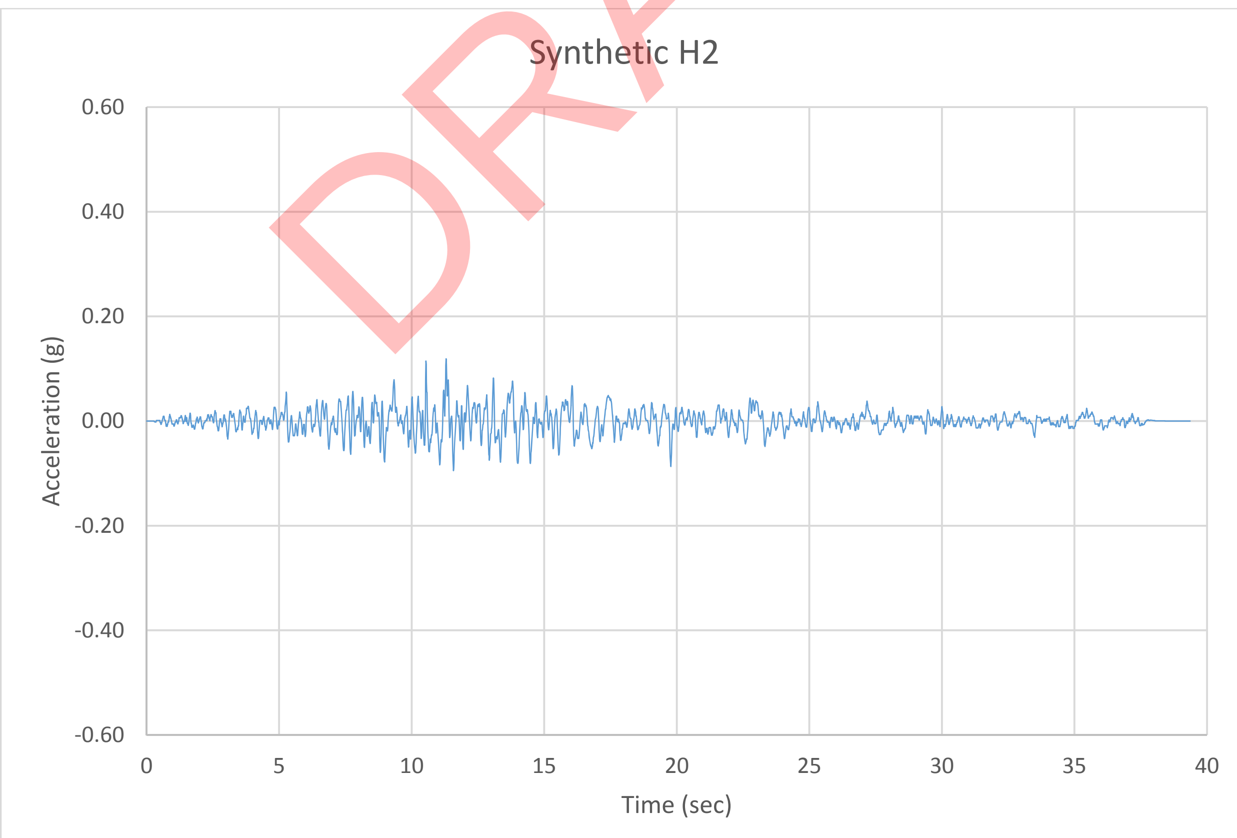

Tohoku N-S
Acceleration (g)
0.60
0.40
0.20
0.00
-0.20
-0.40
-0.60
0
50
100
150
200
250
300
Time (sec)


Tohoku E-W
Acceleration (g)
0.60
0.40
0.20
0.00
-0.20
-0.40
-0.60
-0.80
0
50
100
150
200
250
300
Time (sec)

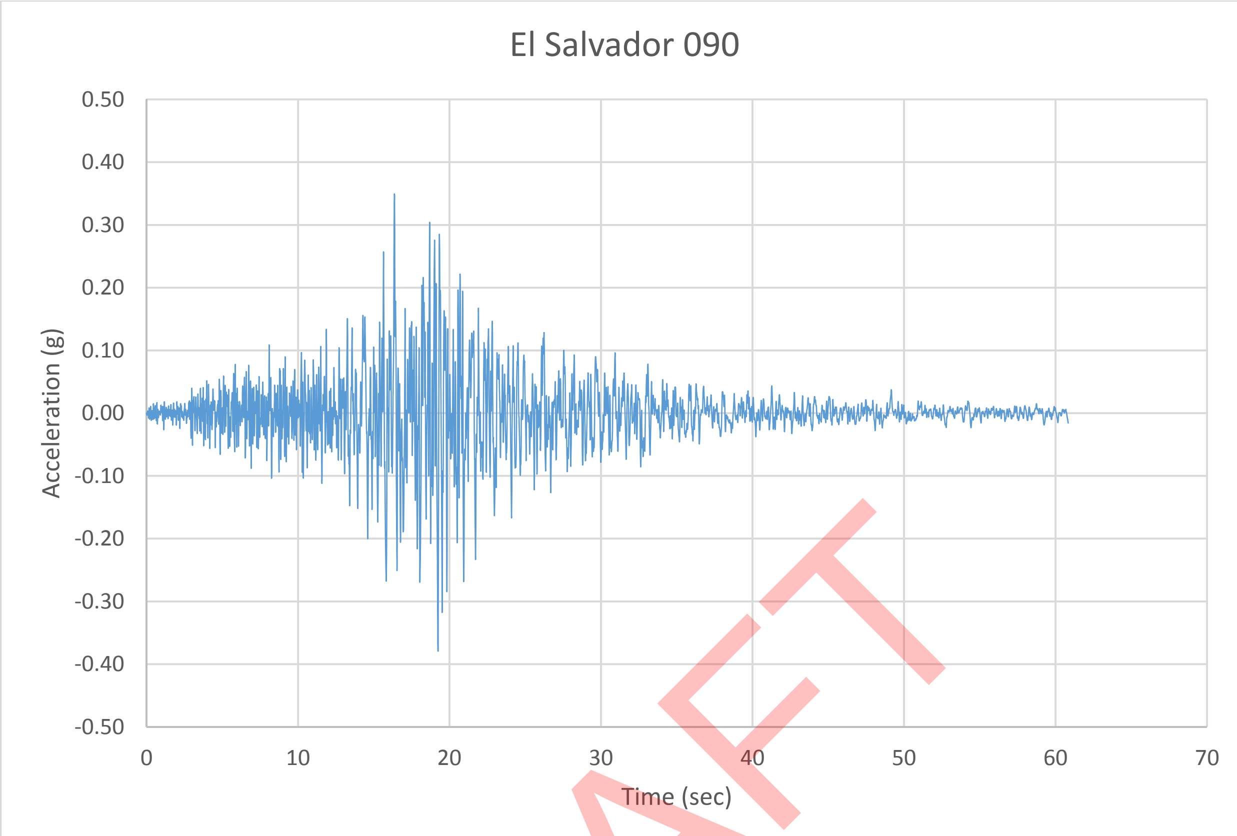

Michoacan H1
0.60
0.40
0.20
0.00
-0.20
-0.40
-0.60
Acceleration (g)
0
10
20
30
40
50
60
70
Time (sec)

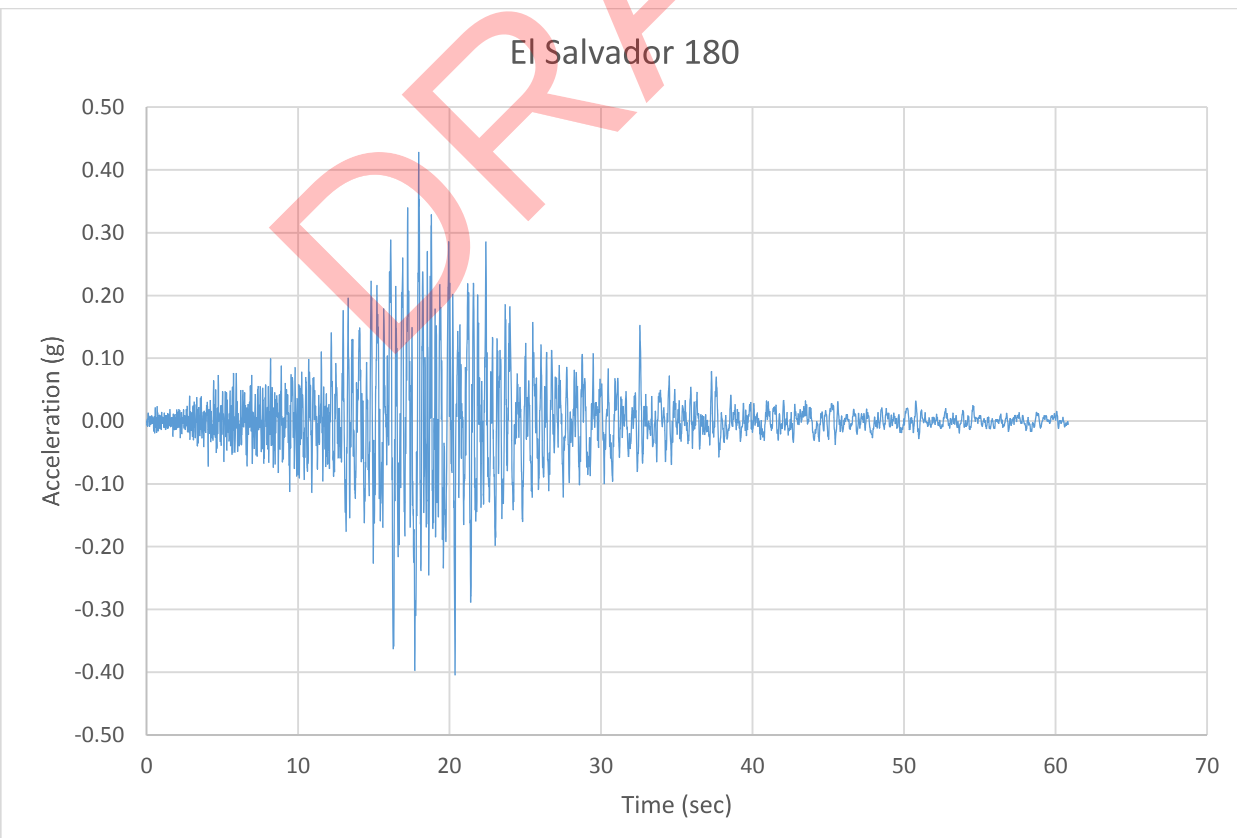

Michoacan H2
0.60
0.40
0.20
0.00
-0.20
-0.40
-0.60
Acceleration (g)
0
10
20
30
40
50
60
70
Time (sec)

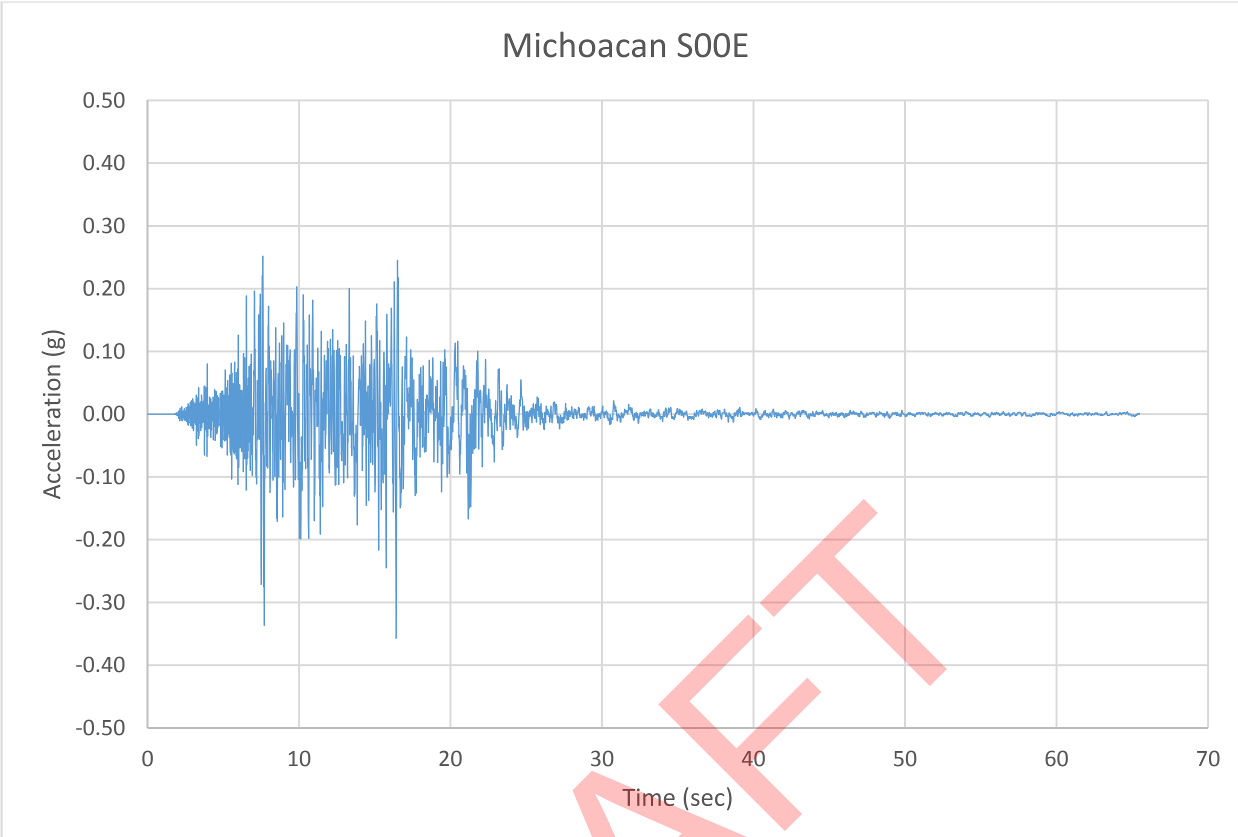

Synthetic H1
0.60
0.40
0.20
0.00
-0.20
-0.40
-0.60
Acceleration (g)
0
5
10
15
20
25
30
35
40
Time (sec)


Synthetic H2
0.60
0.40
0.20
0.00
-0.20
-0.40
-0.60
Acceleration (g)
0
5
10
15
20
25
30
35
40
Time (sec)

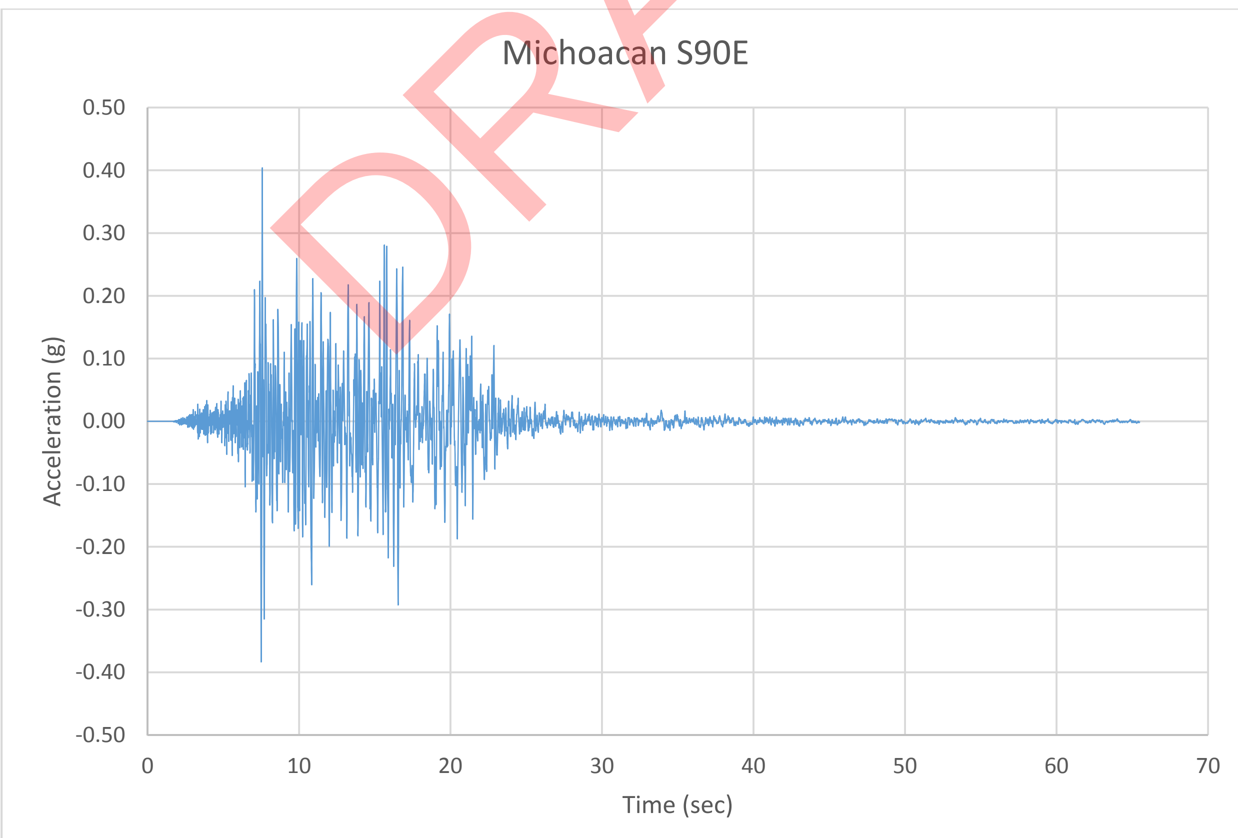
CSZ Intraslab Motions


El Salvador 090
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
10
20
30
40
50
60
70
Time (sec)

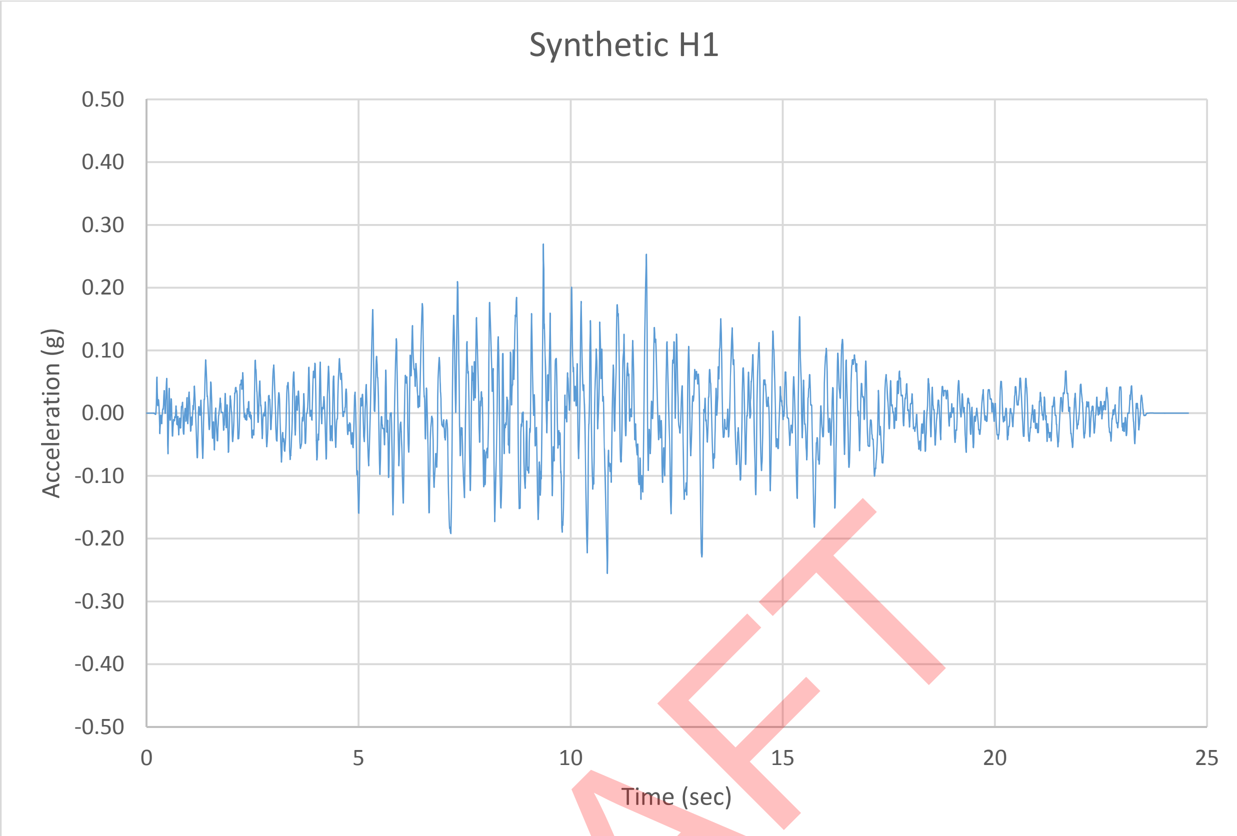

El Salvador 180
Acceleration (g)
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
0
10
20
30
40
50
60
70
Time (sec)

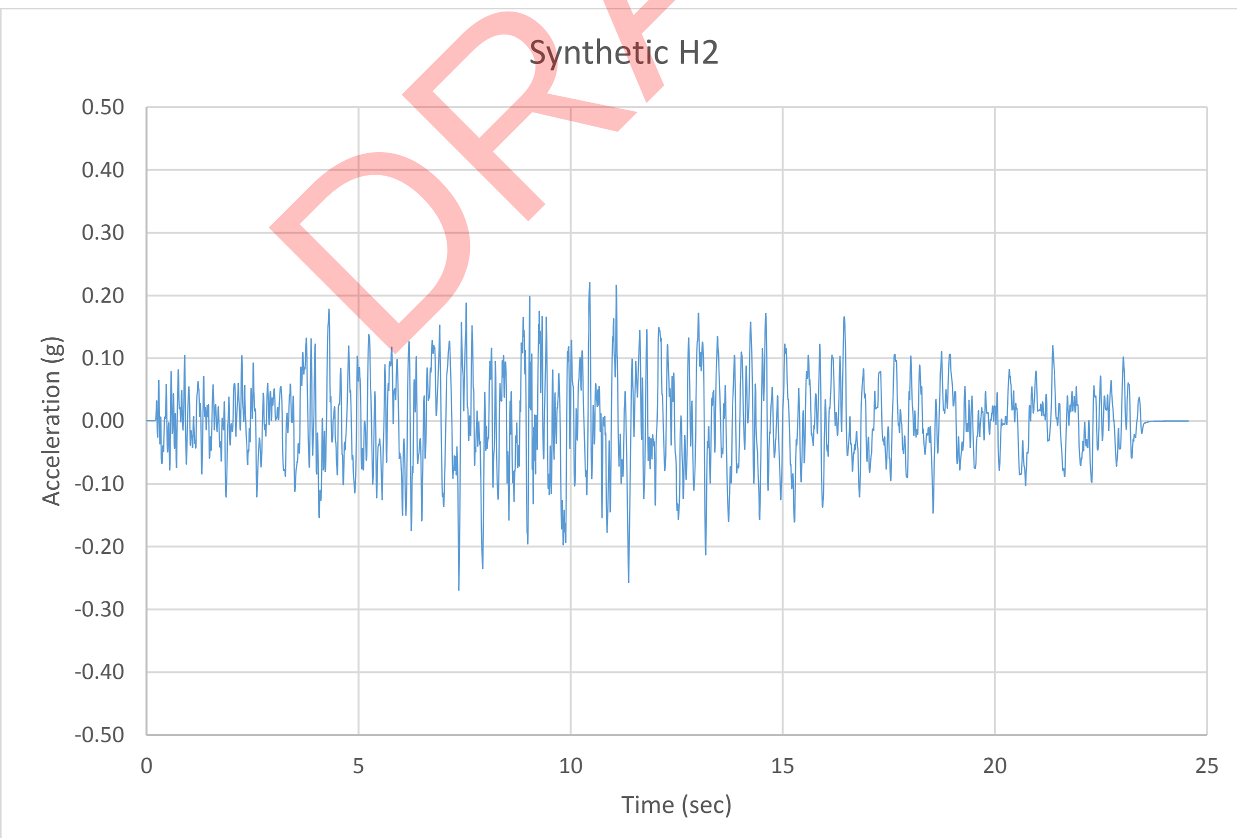


Michoacan S00E
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
10
20
30
40
50
60
70
Time (sec)



Michoacan S90E
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
10
20
30
40
50
60
70
Time (sec)


Synthetic H1
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
5
10
15
20
25
Time (sec)


Synthetic H2
0.50
0.40
0.30
0.20
0.10
0.00
-0.10
-0.20
-0.30
-0.40
-0.50
Acceleration (g)
0
5
10
15
20
25
Time (sec)

Appendix E

Multi-Wedge Analysis Equations

Summary of Known and Unknown Variables

For each wedge, the known variables are:

- Slice weight (W)
- Base length (L)
- Base inclination (α)
- Hydrostatic uplift force (U)
- Vertical and horizontal surcharges ($P_{sur,V}$ and $P_{sur,H}$)
- Seismic coefficient (k_h)
- Soil/rock cohesion (c')
- Soil/rock friction angle (ϕ')

For each wedge, the unknown variables are:

- Base normal force (N')
- Shear force along base (S)
- Interslice force (P)
- Interslice force inclination (δ)
- Factor of Safety (F)

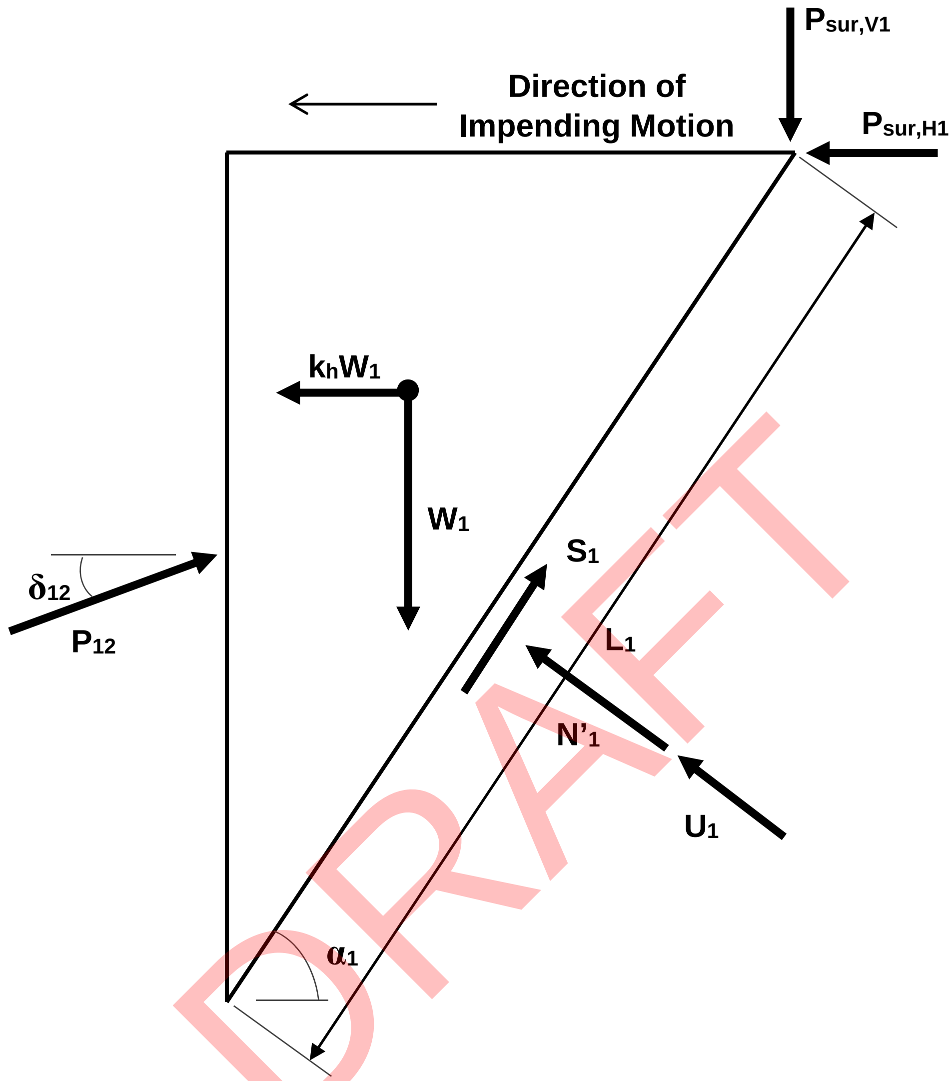
For this force equilibrium approach, the available equations to solve for these variables include:

- Sum of horizontal forces equals zero
- Sum of vertical forces equals zero

The shear force may be written in terms of N', c', ϕ', and F, eliminating it as an unknown variable. For these analyses, the horizontal interslice force was assumed to act horizontally, making δ a known variable. The factor of safety is taken to be the same for all slices, resulting in 1 more unknown variable than available equations.

The unknown variables are solved by selecting a trial value for F, resulting in an equal number of unknown variables and available equations. Equations are then solved simultaneously to determine N' and P for each slice. The factor of safety is the trial value of F that results in equilibrium for all slices.

Wedge 1 of 5

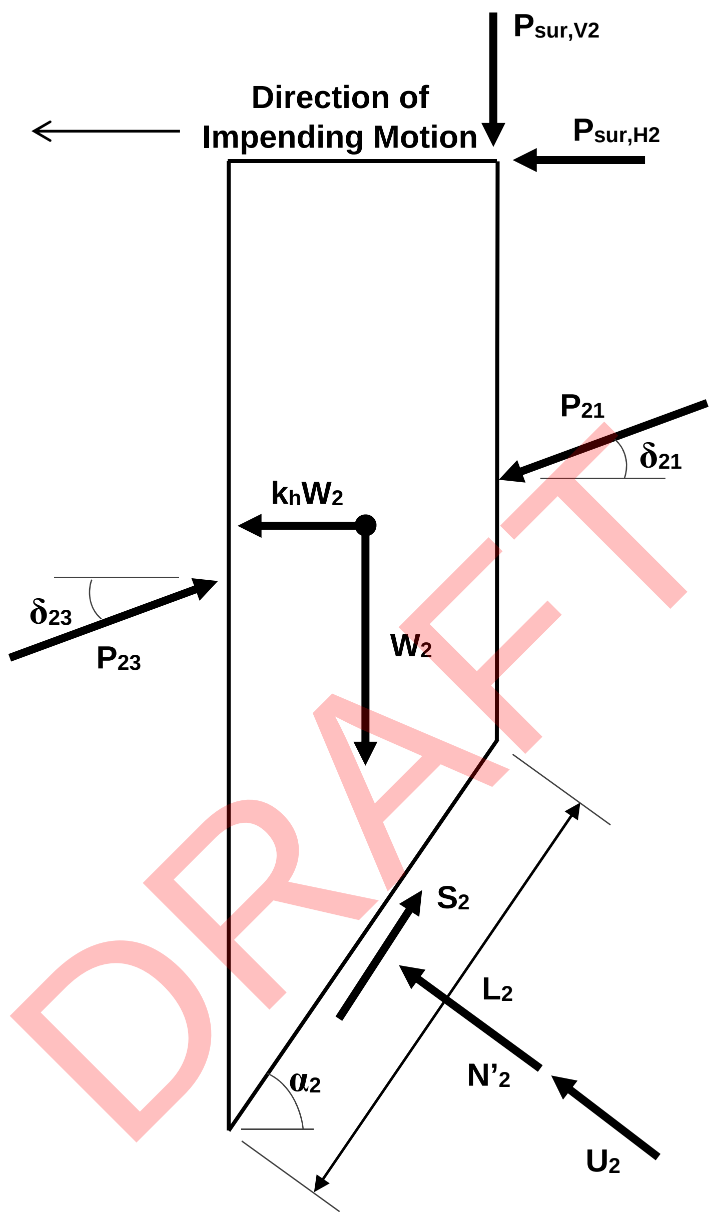


$$N'_1=\frac{W_1+P_{sur,V1}-\frac{c'_1L_1}{F}(\sin\alpha_1-\cos\alpha_1\tan\delta_{12})-U_1(\cos\alpha_1+\sin\alpha_1\tan\delta_{12})-\left(P_{sur,H1}+k_hW_1\right)\tan\delta_{12}}{\frac{\tan\phi'_1}{F}(\sin\alpha_1-\cos\alpha_1\tan\delta_{12})+\cos\alpha_1+\sin\alpha_1\tan\delta_{12}}$$

$$S_1=\frac{c'_1L_1+N'_1\tan\phi'_1}{F}$$

$$P_{23}=\frac{N'_1\sin\alpha_1+U_1\sin\alpha_1-S_1\cos\alpha_1+P_{sur,H1}+k_hW_1}{\cos\delta_{12}}$$

Wedge 2 of 5

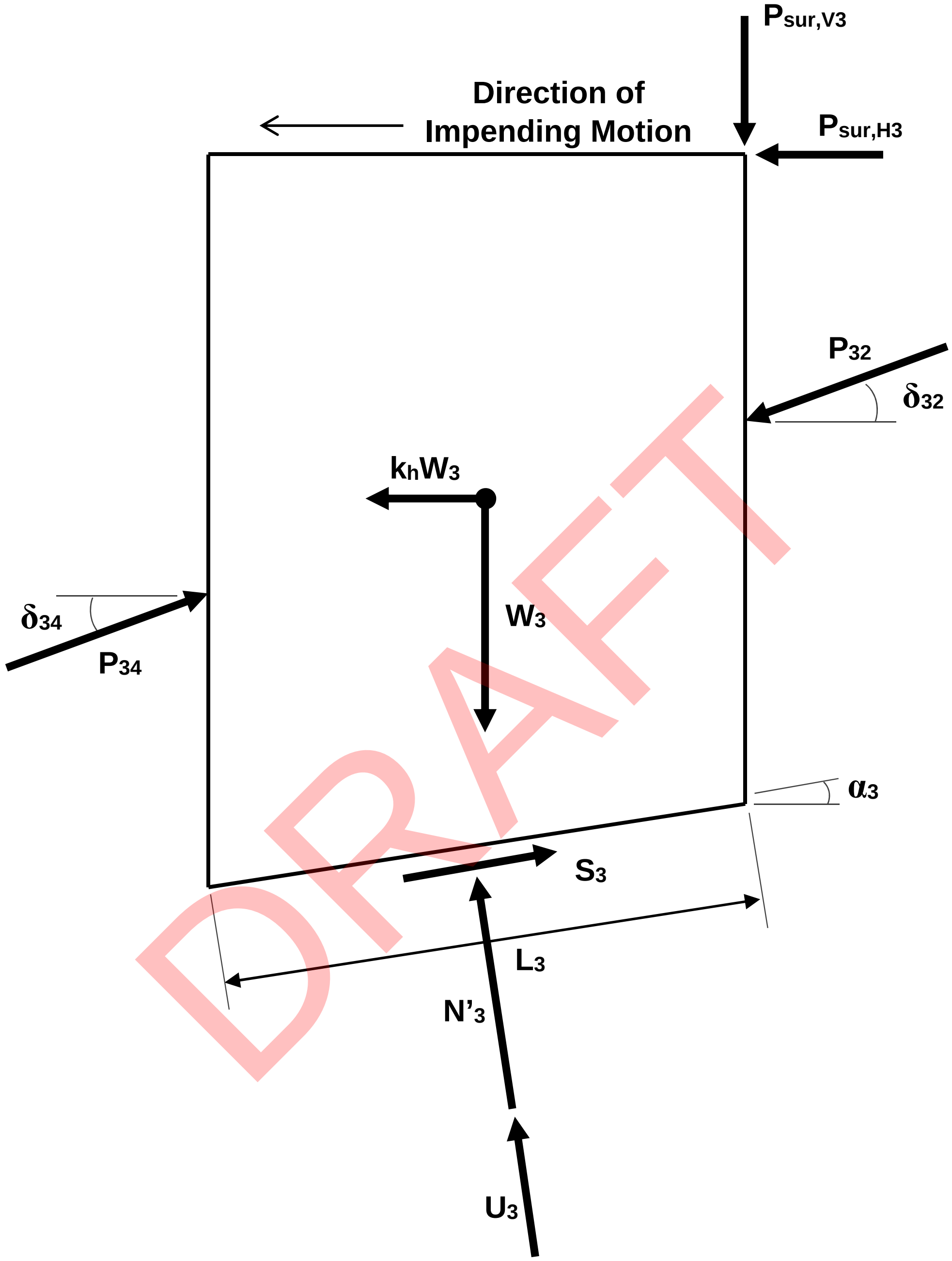


$$N'_2=\frac{W_2+P_{sur,V2}-\frac{c'_2 L_2}{F}(\sin\alpha_2-\cos\alpha_2\tan\delta_{23})-U_2(\cos\alpha_2+\sin\alpha_2\tan\delta_{23})+P_{21}(\sin\delta_{21}-\cos\delta_{21}\tan\delta_{23})-\left(P_{sur,H2}+k_h W_2\right)\tan\delta_{23}}{\frac{\tan\phi'_2}{F}(\sin\alpha_2-\cos\alpha_2\tan\delta_{23})+\cos\alpha_2+\sin\alpha_2\tan\delta_{23}}$$

$$S_2=\frac{c'_2 L_2+N'_2\tan\phi'_2}{F}$$

$$P_{23}=\frac{N'_2\sin\alpha_2+U_2\sin\alpha_2-S_2\cos\alpha_2+P_{21}\cos\delta_{21}+P_{sur,H2}+k_h W_2}{\cos\delta_{23}}$$

Wedge 3 of 5

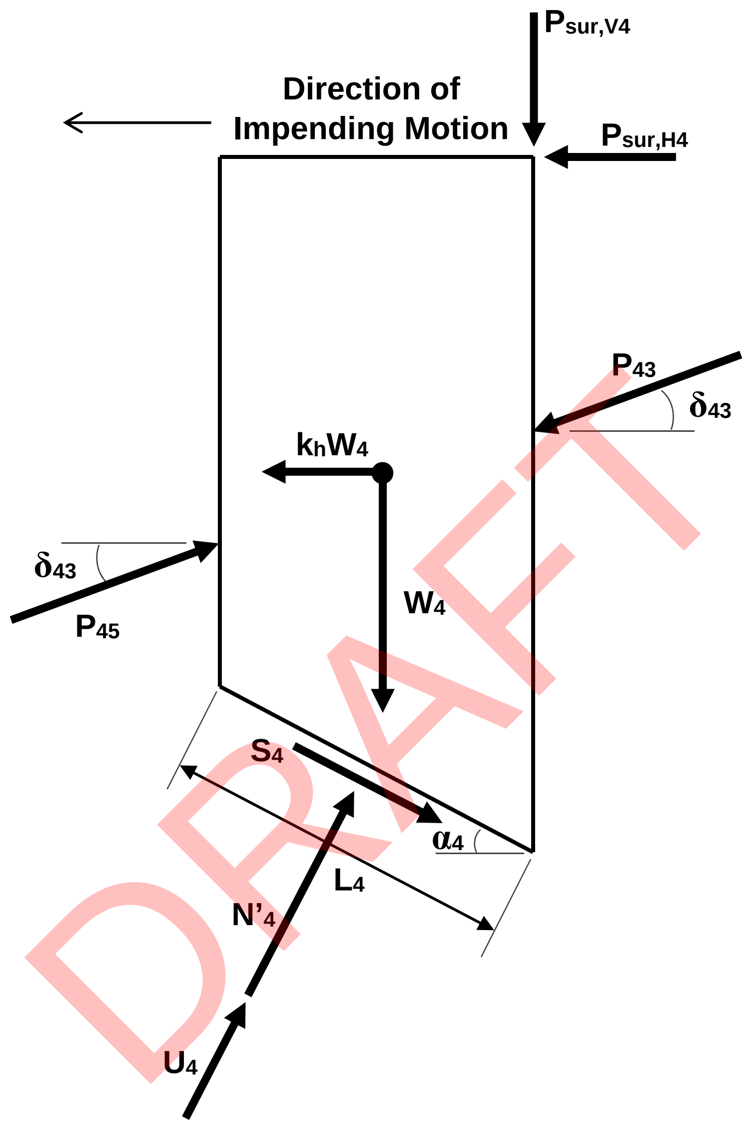


$$N'_3=\frac{W_3+P_{sur,V3}-\frac{c'_3L_3}{F}(\sin\alpha_3-\cos\alpha_3\tan\delta_{34})-U_3(\cos\alpha_3+\sin\alpha_3\tan\delta_{34})+P_{32}(\sin\delta_{32}-\cos\delta_{32}\tan\delta_{34})-\left(P_{sur,H3}+k_hW_3\right)\tan\delta_{34}}{\frac{\tan\phi'_3}{F}(\sin\alpha_3-\cos\alpha_3\tan\delta_{34})+\cos\alpha_3+\sin\alpha_3\tan\delta_{34}}$$

$$S_3=\frac{c'_3L_3+N'_3\tan\phi'_3}{F}$$

$$P_{34}=\frac{N'_3\sin\alpha_3+U_3\sin\alpha_3-S_3\cos\alpha_3+P_{32}\cos\delta_{32}+P_{sur,H3}+k_hW_3}{\cos\delta_{34}}$$

Wedge 4 of 5

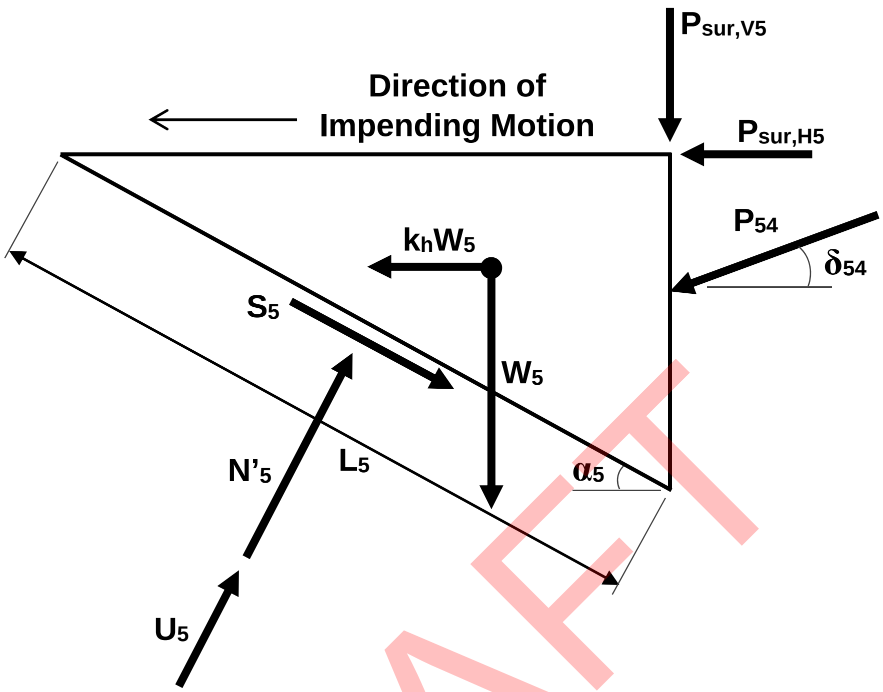


$$N'_4=\frac{W_4+P_{sur,V4}+\frac{c'_4L_4}{F}(\sin\alpha_4+\cos\alpha_4\tan\delta_{43})-U_4(\cos\alpha_4-\sin\alpha_4\tan\delta_{43})-P_{45}(\sin\delta_{45}-\cos\delta_{45}\tan\delta_{43})-\left(P_{sur,H4}+k_hW_4\right)\tan\delta_{43}}{\cos\alpha_4-\sin\alpha_4\tan\delta_{43}-\frac{\tan\phi'_4}{F}(\sin\alpha_4+\cos\alpha_4\tan\delta_{43})}$$

$$S_4=\frac{c'_4L_4+N'_4\tan\phi'_4}{F}$$

$$P_{43}=\frac{N'_4\sin\alpha_4+U_4\sin\alpha_4+S_4\cos\alpha_4+P_{45}\cos\delta_{45}-P_{sur,H4}-k_hW_4}{\cos\delta_{43}}$$

Wedge 5 of 5



$$N'_5 = \frac{W_5 + P_{sur,V5} + \frac{c'_5 L_5}{F}(\sin\alpha_5 + \cos\alpha_5 \tan\delta_{54}) - U_5(\cos\alpha_5 - \sin\alpha_5 \tan\delta_{54}) - \left(P_{sur,H5} + k_h W_5\right)\tan\delta_{54}}{\cos\alpha_5 - \sin\alpha_5 \tan\delta_{54} - \frac{\tan\phi'_5}{F}(\sin\alpha_5 + \cos\alpha_5 \tan\delta_{54})}$$

$$S_5 = \frac{c'_5 L_5 + N'_5 \tan\phi'_5}{F}$$

$$P_{54} = \frac{N'_5 \sin\alpha_5 + U_5 \sin\alpha_5 + S_5 \cos\alpha_5 - P_{sur,H5} - k_h W_5}{\cos\delta_{54}}$$

Appendix F

Example of Composite Factor of Safety Calculation

The following example illustrates the composite factor of safety approach. Consider an analysis of the left abutment for a seismic case with k_h = 0.20 and a trial factor of safety of 1.40. Driving and resisting forces that are specified for the analysis include the weight of the soil, uplift forces acting perpendicular to the base of the wedge, and vertical and horizontal surcharges due to water pressure. For a seismic case, the horizontal surcharge is the combined hydrostatic and hydrodynamic forces from the free water against the dam. An additional seismic inertial force is added, which corresponds to the seismic coefficient multiplied by the saturated weight of the wedge. The effective stress normal to the base, the shear stress, and interslice forces are then calculated. The factor of safety is defined as the ratio between the available shear strength and the shear force acting along the base.

Figure F-1 at the conclusion of this appendix shows the general calculation steps of the composite factor of safety approach. Under the seismic loads described above, the center analysis region of the dam has a factor of safety of 0.26. Load must be transferred to the adjacent section to reach the trial factor of safety value. Figure F-2 shows that Section C1 must transfer 257.3 kips/ft to reach a factor of safety of 1.40. The center analysis region is 43 feet long. One half of this length is assumed to be supported by the left abutment. Therefore, the load transferred to the adjacent analysis section (L1) is 257.3 kips/ft x 0.5 x 43 ft = 5,532.2 kips.

Section L1 represents a region of the dam that is 18 feet long. The corresponding load acting on Section L1 is then 5,532.2 kips/18 ft = 307.3 kips/ft. This horizontal load, in addition to other forces acting on the section, brings the factor of safety of the section to 1.24. Therefore, Section L1 also must transfer load to the adjacent section to reach the trial factor of safety value. Figure F-3 shows that this load is 125.6 kips/ft, which corresponds to a total load of 125.6 kips/ft x 18 ft = 2,261.3 kips transferred to Section L2.

Section L2 represents a region of the dam that is 20 feet long. The corresponding load acting on Section L2 is then 2,261.3 kips/20 ft = 113.1 kips/ft. This horizontal load, in addition to other forces acting on the section, brings the factor of safety of the section to 1.53 (Figure F-4). Since this value does not match the trial value of 1.40, a new trial value would be selected, and the process would be repeated until the factors of safety match.

If additional trial values of 1.45 and 1.50 are selected, the factor of safety of Section L2 is 1.40 and 1.31, respectively. The error for any trial is defined as the difference between the trial value and computed factor of safety of the end section. The errors associated with these three trials are summarized in Table F-1 below.

Table F-1 – Results for Example Composite Factor of Safety Calculations

Trial FS	FS of End	Error
1.40	1.53	-0.13
1.45	1.40	0.05
1.50	1.31	0.19

By fitting a curve through these three points and determining the value of the trial factor of safety for which the error is zero, as shown in Figure F-5, the composite factor of safety is found to be approximately 1.44.

To determine the composite yield acceleration, this analysis is repeated with various values of k_h to arrive at a plot of factor of safety versus k_h. The horizontal seismic coefficient for which the factor of safety is equal to 1.0 is the yield coefficient, k_y. The plot of factor of safety versus horizontal seismic coefficient for the left abutment is shown in Figure F-6. The yield coefficient is determined to be approximately 0.64.

COMPOSITE FACTOR OF SAFETY | **CORNFORTH CONSULTANTS, INC.**

Project: Bear Creek Dam | By: AK

Description: Composite Factor of Safety - Treat Left and Right Abutments Separately | Date: 04/30/15

Pseudostatic Analysis with k_h = 0.20

Left Side Trial Factor of Safety = 1.40 | Right Side Trial Factor of Safety =

Center Analysis Section (STA 0+93)			
Length of Analysis Section =	43.0 ft		
Percentage of Load to Left =	50 %	Percentage of Load to Right =	50 %
Controlling Analysis Case =	2(c)	Controlling Analysis Case =	
S_{max} =	268.9 k/ft	S_{max} =	k/ft
For the Trial FS, $S_{mob} = S_{max}/FS$ =	192.1 k/ft	For the Trial FS, $S_{mob} = S_{max}/FS$ =	0.0 k/ft
Load Shared =	257.3 k/ft	Load Shared =	k/ft
Contributing Length of Section =	21.5 ft	Contributing Length of Section =	21.5 ft
Load Shared =	5532.2 k	Load Shared =	0.0 k

Left Abutment Section 2 (STA 0+65)		Right Abutment Section 2 (STA 1+40)	
Additional Load =	5532.2 k	Additional Load =	0.0 k
Length of Analysis Section =	18.0 ft	Length of Analysis Section =	37.0 ft
Additional Load =	307.3 k/ft	Additional Load =	0.0 k/ft
Controlling Analysis Case =	3(c)	Controlling Analysis Case =	
S_{max} =	712.5 k/ft	S_{max} =	k/ft
For the Trial FS, $S_{mob} = S_{max}/FS$ =	508.9 k/ft	For the Trial FS, $S_{mob} = S_{max}/FS$ =	0.0 k/ft
Load Shared =	125.6 k/ft	Load Shared =	k/ft
Length of Analysis Section =	18.0 ft	Length of Analysis Section =	37.0 ft
Load Shared =	2261.3 k	Load Shared =	0.0 k

Left Abutment Section 1 (STA 0+50)		Right Abutment Section 1 (STA 1+70)	
Additional Load =	2261.3 k	Additional Load =	0.0 k
Length of Analysis Section =	20.0 ft	Length of Analysis Section =	34.0 ft
Additional Load =	113.1 k/ft	Additional Load =	0.0 k/ft
Controlling Analysis Case =	2(a)	Controlling Analysis Case =	
S_{max} =	343.3 k/ft	S_{max} =	k/ft
S_{mob} =	223.7 k/ft	S_{mob} =	k/ft
FS of Section =	1.53	FS of Section =	

Definitions:
Load Shared = Horizontal load on dam passed to adjacent analysis section(s) to meet trial Factor of Safety
Additional Load = Horizontal load on dam received from adjacent analysis section

Figure F-1. Composite Factor of Safety Calculations Summary

SLIDING STABILITY ANALYSIS - WEDGE METHOD — **CORNFORTH CONSULTANTS, INC.**

Project:	Bear Creek Dam	By:	AK
Description:	Center Section, STA 0+93	Date:	04/30/15
	Failure surface through sandstone beneath cutoff trench (3)		
	Pseudostatic Analysis with k_h = 0.20		

Materials

Material 1: Overburden			Material 2: Sandstone			Material 3: Concrete		
Unit Wt	[pcf]	125	Unit Wt	[pcf]	140	Unit Wt	[pcf]	154
Cohesion	[psf]	0	Cohesion	[psf]	150	Cohesion	[psf]	20160
Friction	[deg]	30	Friction	[deg]	35	Friction	[deg]	57

Geometry

	Wedge No:	1	2	3	4	5
Area of Material 1	[sf]	301.0	34.1	38.8	186.1	125.8
Area of Material 2	[sf]	0.0	28.8	521.4	68.9	0.0
Area of Material 3	[sf]	0.0	263.0	2263.3	0.0	0.0
Base Length	[ft]	36.2	11.6	48.2	17.8	24.1

Slip Surface Inclination		
Wedge 1	[deg]	60
Wedge 2	[deg]	60
Wedge 3	[deg]	0
Wedge 4	[deg]	30
Wedge 5	[deg]	30

Forces

	Wedge No:	1	2	3	4	5
Weight	[k/ft]	37.6	48.8	426.4	32.9	15.7
Uplift Force	[k/ft]	159.8	65.4	148.8	17.3	7.6
Vertical Surcharge	[k/ft]	61.2	12.6	0.0	0.0	0.0
Horizontal Surcharge	[k/ft]	5.5	110.9	0.0	0.0	0.0
Seismic Load, k_h = 0.20	[k/ft]	7.5	9.8	85.3	6.6	3.1
Load Received from Adj	[k/ft]	0.0	0.0	0.0	0.0	0.0
Load Shared with Adj	[k/ft]	0.0	0.0	257.3	0.0	0.0

Interslice Force Inclination		
Wedge 1-2	[deg]	0
Wedge 2-3	[deg]	0
Wedge 3-4	[deg]	0
Wedge 4-5	[deg]	0

		FS = 1.30	FS = 1.35	FS = 1.40	FS = 1.45	FS = 1.50	FS = 1.55	FS = 1.60	FS = 1.65	FS = 1.70
Wedge 1 — Material for Strength Parameters: 1; Cohesion: 0 psf; Friction: 30 deg										
N'_1	[k/ft]	21.4	21.7	22.1	22.4	22.7	23.0	23.3	23.6	23.8
S_1	[k/ft]	9.5	9.3	9.1	8.9	8.7	8.6	8.4	8.2	8.1
$P_{12} = P_{21}$	[k/ft]	165.2	165.6	166.0	166.4	166.7	167.1	167.4	167.7	168.0
Wedge 2 — Material for Strength Parameters: 2; Cohesion: 150 psf; Friction: 35 deg										
N'_2	[k/ft]	28.5	29.1	29.6	30.1	30.6	31.1	31.6	32.0	32.5
S_2	[k/ft]	16.7	16.4	16.0	15.7	15.5	15.2	14.9	14.6	14.4
$P_{23} = P_{32}$	[k/ft]	358.8	359.9	360.9	361.9	362.8	363.7	364.6	365.4	366.2
Wedge 3 — Material for Strength Parameters: 2; Cohesion: 150 psf; Friction: 35 deg										
N'_3	[k/ft]	277.6	277.6	277.6	277.6	277.6	277.6	277.6	277.6	277.6
S_3	[k/ft]	155.1	149.3	144.0	139.0	134.4	130.1	126.0	122.2	118.6
P_{34}	[k/ft]	31.7	38.5	44.9	50.8	56.4	61.6	66.5	71.2	75.6
Wedge 4 — Material for Strength Parameters: 2; Cohesion: 150 psf; Friction: 35 deg										
N'_4	[k/ft]	31.8	31.2	30.7	30.2	29.7	29.3	29.0	28.7	28.3
S_4	[k/ft]	19.2	18.1	17.2	16.4	15.7	15.0	14.4	13.8	13.2
P_{43}	[k/ft]	47.8	46.2	44.9	43.6	42.5	41.4	40.5	39.6	38.8
Wedge 5 — Material for Strength Parameters: 1; Cohesion: 0 psf; Friction: 30 deg										
N'_5	[k/ft]	14.2	14.0	13.9	13.7	13.6	13.5	13.3	13.2	13.1
S_5	[k/ft]	6.3	6.0	5.7	5.5	5.2	5.0	4.8	4.6	4.5
$P_{54} = P_{45}$	[k/ft]	13.2	12.9	12.5	12.2	12.0	11.7	11.5	11.3	11.1

Solving for Equilibrium between Wedges 3 and 4, FS = 1.40

Figure F-2. Section C1 Sample Calculations

SLIDING STABILITY ANALYSIS - WEDGE METHOD **CORNFORTH CONSULTANTS, INC.**

Project: Bear Creek Dam By: AK

Description: Left Abutment Section, STA 0+65 Date: 04/30/15

Failure surface through marine sediments, passive resistance from rock (3)

Pseudostatic Analysis with k_h = 0.20

Materials

Material 1: Overburden			Material 2: Basalt			Material 3: Marine Sediments		
Unit Wt	[pcf]	125	Unit Wt	[pcf]	160	Unit Wt	[pcf]	125
Cohesion	[psf]	0	Cohesion	[psf]	1210	Cohesion	[psf]	150
Friction	[deg]	30	Friction	[deg]	52	Friction	[deg]	26

Material 4: Concrete		
Unit Wt	[pcf]	154

Geometry

	Wedge No:	1	2	3	4	5
Area of Material 1	[sf]	242.6	25.9	60.0	0.0	111.4
Area of Material 2	[sf]	0.0	22.6	651.3	0.0	1134.2
Area of Material 3	[sf]	0.0	0.0	0.0	0.0	0.0
Area of Material 4	[sf]	0.0	638.7	1126.6	0.0	0.0
Base Length	[ft]	34.3	22.9	30.3	0.0	65.7

Slip Surface Inclination		
Wedge 1	[deg]	63
Wedge 2	[deg]	65
Wedge 3	[deg]	0
Wedge 4	[deg]	0
Wedge 5	[deg]	27

Forces

	Wedge No:	1	2	3	4	5
Weight	[k/ft]	30.3	105.2	285.2	0.0	195.4
Uplift Force	[k/ft]	120.6	90.1	94.1	0.0	64.5
Vertical Surcharge	[k/ft]	38.8	6.7	0.0	0.0	0.0
Horizontal Surcharge	[k/ft]	0.8	64.0	0.0	0.0	0.0
Seismic Load, k_h = 0.20	[k/ft]	6.1	21.0	57.0	0.0	39.1
Load Received from Adj	[k/ft]	0.0	0.0	307.3	0.0	0.0
Load Shared with Adj	[k/ft]	0.0	0.0	125.6	0.0	0.0

Interslice Force Inclination		
Wedge 1-2	[deg]	0
Wedge 2-3	[deg]	0
Wedge 3-4	[deg]	0
Wedge 4-5	[deg]	0

Wedge 1										
Material for Strength Parameters:			1	Cohesion:	0 psf		Friction:	30 deg		
		FS = 1.30	FS = 1.35	FS = 1.40	FS = 1.45	FS = 1.50	FS = 1.55	FS = 1.60	FS = 1.65	FS = 1.70
N'_1	[k/ft]	16.9	17.2	17.5	17.8	18.0	18.3	18.5	18.8	19.0
S_1	[k/ft]	7.5	7.4	7.2	7.1	6.9	6.8	6.7	6.6	6.5
$P_{12} = P_{21}$	[k/ft]	126.0	126.3	126.6	126.9	127.2	127.5	127.8	128.1	128.3
Wedge 2										
Material for Strength Parameters:			2	Cohesion:	1210 psf		Friction:	52 deg		
		FS = 1.30	FS = 1.35	FS = 1.40	FS = 1.45	FS = 1.50	FS = 1.55	FS = 1.60	FS = 1.65	FS = 1.70
N'_2	[k/ft]	41.5	43.1	44.7	46.2	47.7	49.2	50.7	52.1	53.5
S_2	[k/ft]	62.1	61.4	60.6	59.9	59.2	58.5	57.8	57.2	56.5
$P_{23} = P_{32}$	[k/ft]	304.0	306.1	308.2	310.2	312.2	314.1	316.0	317.8	319.6
Wedge 3										
Material for Strength Parameters:			3	Cohesion:	150 psf		Friction:	26 deg		
		FS = 1.30	FS = 1.35	FS = 1.40	FS = 1.45	FS = 1.50	FS = 1.55	FS = 1.60	FS = 1.65	FS = 1.70
N'_3	[k/ft]	191.1	191.1	191.1	191.1	191.1	191.1	191.1	191.1	191.1
S_3	[k/ft]	75.2	72.4	69.8	67.4	65.2	63.1	61.1	59.2	57.5
P_{34}	[k/ft]	467.6	472.5	477.1	481.6	485.8	489.8	493.6	497.3	500.8
Wedge 4										
Material for Strength Parameters:			0	Cohesion:	0 psf		Friction:	0 deg		
		FS = 1.30	FS = 1.35	FS = 1.40	FS = 1.45	FS = 1.50	FS = 1.55	FS = 1.60	FS = 1.65	FS = 1.70
N'_4	[k/ft]	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
S_4	[k/ft]	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
P_{43}	[k/ft]	541.5	507.0	477.1	451.0	428.0	407.5	389.2	372.7	357.8
Wedge 5										
Material for Strength Parameters:			2	Cohesion:	1210 psf		Friction:	52 deg		
		FS = 1.30	FS = 1.35	FS = 1.40	FS = 1.45	FS = 1.50	FS = 1.55	FS = 1.60	FS = 1.65	FS = 1.70
N'_5	[k/ft]	373.2	357.5	344.0	332.1	321.6	312.4	304.0	296.6	289.8
S_5	[k/ft]	428.6	397.8	371.2	348.0	327.5	309.2	292.9	278.2	265.0
$P_{54} = P_{45}$	[k/ft]	541.5	507.0	477.1	451.0	428.0	407.5	389.2	372.7	357.8
Solving for Equilibrium between Wedges 3 and 4, FS = 1.40										

Figure F-2. Section L1 Sample Calculations

SLIDING STABILITY ANALYSIS - WEDGE METHOD **CORNFORTH CONSULTANTS, INC.**

Project:	Bear Creek Dam	By:	AK
Description:	Left Abutment Section, STA 0+50	Date:	04/30/15
	Failure surface through basalt beneath cutoff trench, passive resistance from soil weight only		
	Pseudostatic Analysis with k_h = 0.20		

Materials

	Units	Material 1: Overburden	Material 2: Basalt	Material 3: Concrete
Unit Wt	[pcf]	125	160	154
Cohesion	[psf]	0	1210	20160
Friction	[deg]	30	52	57

Geometry

Wedge No:		1	2	3	4	5
Area of Material 1	[sf]	92.7	47.6	0.0	88.4	39.0
Area of Material 2	[sf]	0.0	4.4	0.0	62.0	0.0
Area of Material 3	[sf]	0.0	187.0	198.8	726.3	0.0
Base Length	[ft]	24.5	11.1	3.0	28.7	17.3

Slip Surface Inclination		
Wedge 1	[deg]	68
Wedge 2	[deg]	57
Wedge 3	[deg]	0
Wedge 4	[deg]	24
Wedge 5	[deg]	74

Forces

Wedge No:		1	2	3	4	5
Weight	[k/ft]	11.6	35.5	30.6	132.8	4.9
Uplift Force	[k/ft]	65.3	39.1	7.5	36.4	1.4
Vertical Surcharge	[k/ft]	18.6	7.7	0.0	0.0	0.0
Horizontal Surcharge	[k/ft]	-3.2	0.0	43.2	0.0	0.0
Seismic Load, k_h = 0.20	[k/ft]	2.3	7.1	6.1	26.6	1.0
Load Received from Adj	[k/ft]	0.0	0.0	113.1	0.0	0.0
Load Shared with Adj	[k/ft]	0.0	0.0	0.0	0.0	0.0

Interslice Force Inclination		
Wedge 1-2	[deg]	0
Wedge 2-3	[deg]	0
Wedge 3-4	[deg]	0
Wedge 4-5	[deg]	0

Wedge 1

Material for Strength Parameters: 1 Cohesion: 0 psf Friction: 30 deg

		FS = 2.80	FS = 2.90	FS = 3.00	FS = 3.10	FS = 3.20	FS = 3.30	FS = 3.40	FS = 3.50	FS = 3.60
N'_1	[k/ft]	10.1	10.2	10.4	10.5	10.6	10.7	10.8	10.9	10.9
S_1	[k/ft]	2.1	2.0	2.0	1.9	1.9	1.9	1.8	1.8	1.8
$P_{12} = P_{21}$	[k/ft]	68.3	68.4	68.5	68.6	68.7	68.9	69.0	69.1	69.1

Wedge 2

Material for Strength Parameters: 2 Cohesion: 1210 psf Friction: 52 deg

		FS = 2.80	FS = 2.90	FS = 3.00	FS = 3.10	FS = 3.20	FS = 3.30	FS = 3.40	FS = 3.50	FS = 3.60
N'_2	[k/ft]	19.2	19.6	20.1	20.5	20.8	21.2	21.6	21.9	22.2
S_2	[k/ft]	13.6	13.3	13.0	12.8	12.5	12.3	12.1	11.8	11.6
$P_{23} = P_{32}$	[k/ft]	116.9	117.5	118.1	118.7	119.3	119.8	120.3	120.8	121.3

Wedge 3

Material for Strength Parameters: 2 Cohesion: 1210 psf Friction: 52 deg

		FS = 2.80	FS = 2.90	FS = 3.00	FS = 3.10	FS = 3.20	FS = 3.30	FS = 3.40	FS = 3.50	FS = 3.60
N'_3	[k/ft]	23.1	23.1	23.1	23.1	23.1	23.1	23.1	23.1	23.1
S_3	[k/ft]	11.9	11.5	11.1	10.7	10.4	10.1	9.8	9.5	9.2
P_{34}	[k/ft]	267.4	268.4	269.4	270.4	271.3	272.1	273.0	273.7	274.5

Wedge 4

Material for Strength Parameters: 2 Cohesion: 1210 psf Friction: 52 deg

		FS = 2.80	FS = 2.90	FS = 3.00	FS = 3.10	FS = 3.20	FS = 3.30	FS = 3.40	FS = 3.50	FS = 3.60
N'_4	[k/ft]	143.8	142.3	140.9	139.6	138.5	137.4	136.4	135.5	134.6
S_4	[k/ft]	78.1	74.8	71.7	68.9	66.2	63.8	61.6	59.5	57.5
P_{43}	[k/ft]	134.1	130.4	127.1	124.0	121.1	118.5	116.0	113.7	111.5

Wedge 5

Material for Strength Parameters: 0 Cohesion: 0 psf Friction: 0 deg

		FS = 2.80	FS = 2.90	FS = 3.00	FS = 3.10	FS = 3.20	FS = 3.30	FS = 3.40	FS = 3.50	FS = 3.60
N'_5	[k/ft]	16.3	16.3	16.3	16.3	16.3	16.3	16.3	16.3	16.3
S_5	[k/ft]	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$P_{54} = P_{45}$	[k/ft]	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0

Solving for Equilibrium between Wedges 3 and 4, FS = 1.53

Figure F-4. Section L2 Sample Calculations

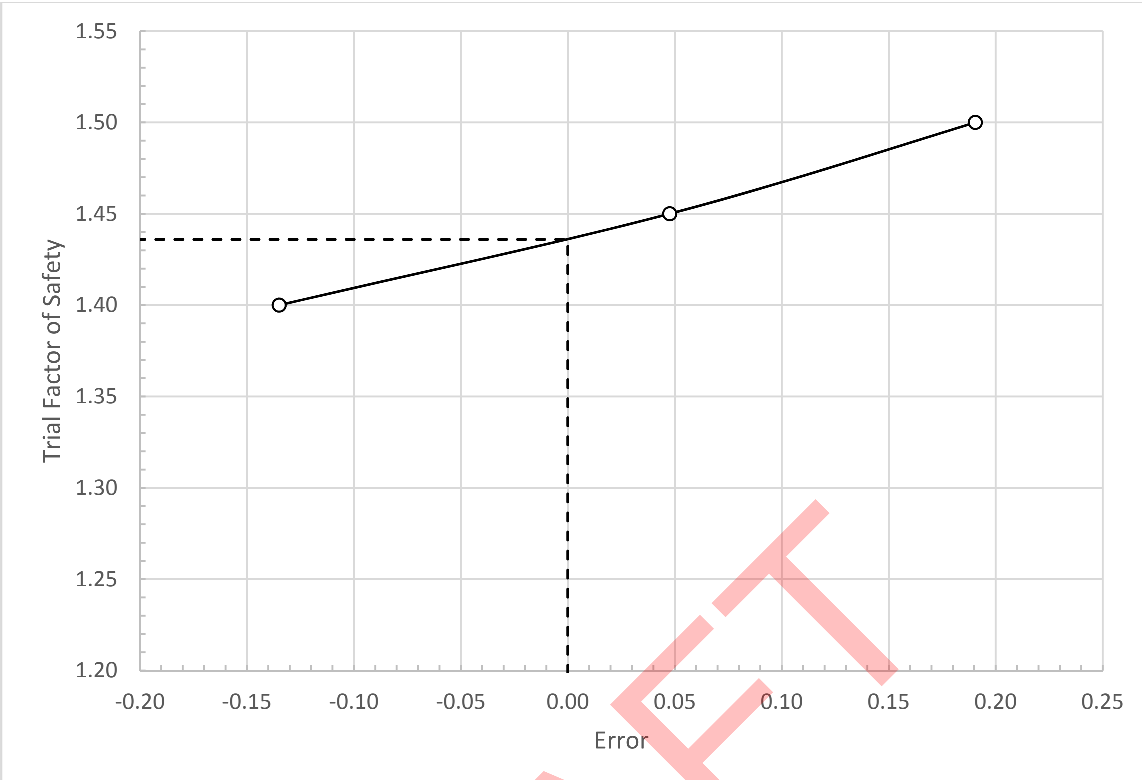


Figure F-5. Trial Factor of Safety vs. Error

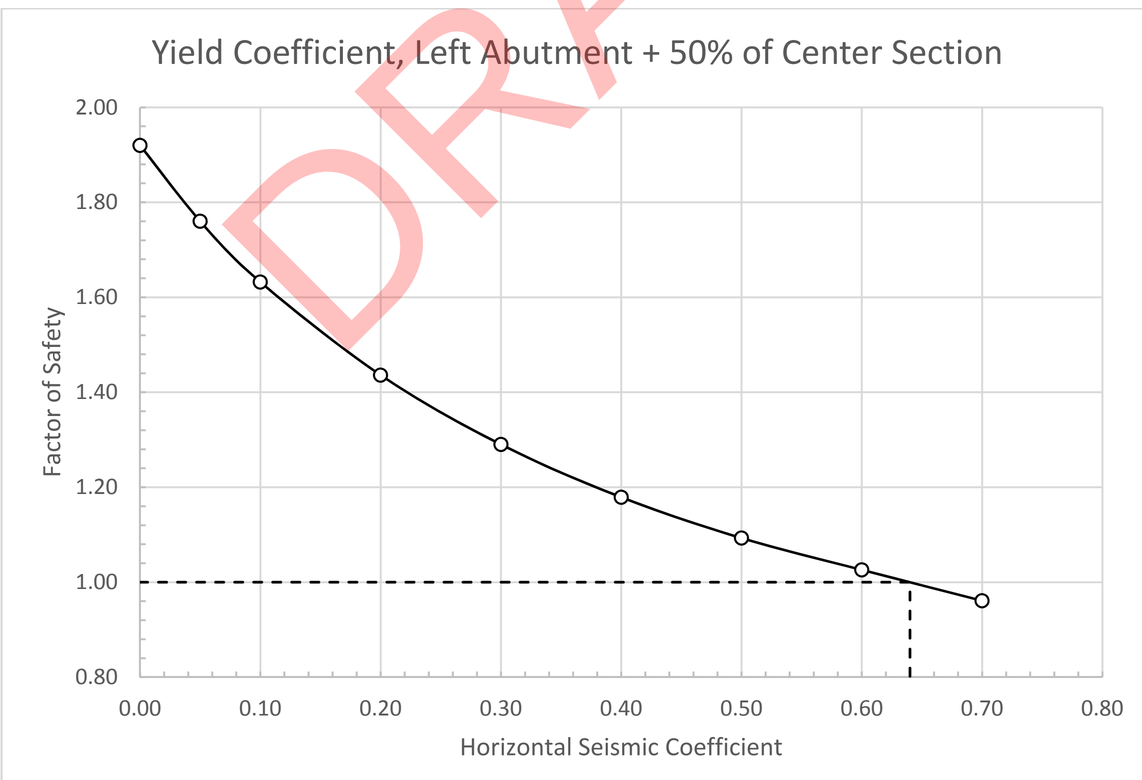


Figure F-6. Example of Yield Coefficient Determination for the Left Abutment