Upper and Lower Wrangell Dams
Seismic Safety Report
Wrangell Alaska

FIGURES

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1 Introduction

This report provides the results of the Seismic Stability Study of the Upper and Lower Wrangell Dams, Wrangell, Alaska. This seismic analysis is being performed as part of the Special Conditions in Attachment A of the Certificate of Approval to Operate a Dam, granted by the State of Alaska, dated June 30, 2004. Specifically, the attachment required the owner to perform a detailed review of the static and seismic stability of each dam, including a seismic study, conducted in general accordance with Chapter 6 of the "Guidelines for Cooperation with the Alaska Dam Safety Program" (September 2003).

This review is in general accordance with a Phase I – Special Studies review as outlined in U.S. Army Corps of Engineers, Engineering Circular EC 1110-2-xxxx, Dynamic Stability of Existing Embankment Dams, dated 2 June 2003. It is noted that this document has an expiration date of 30 September 2005 and that an updated document was due on 1 November 2005, but is not yet available. Significant changes to the document are not expected.

The scope of this study included a site investigation that consisted of six SPT borings, three each in the Upper and Lower Dams. Laboratory data from the samples included moisture content, grain size analysis and some organic contents. Full surveys of the dams were performed and incorporated into a recent hydrographic survey of the reservoirs. Design earthquakes were developed for both the OBE and MCE events.

This information was used to analyze liquefaction of the dam embankment soils and foundation materials. Following liquefaction calculations, stability analyses were run on both dams using residual and reduced strengths to determine the potential for flow failure. Preliminary deformation analyses were performed to address the potential for large deformations leading to possible dam failure.

1.1 Background

The two dams, designated Upper and Lower, are situated on a single drainage way southeast of the city of Wrangell. They impound approximately 122 and 67 acre feet of water respectively. The original crib structures were constructed around 1900 for the Lower Dam and around 1935 for the Upper Dam. According to records and previous reports, the upper log crib structure leaked badly after construction and did not retain water until it was modified around 1958. Since initial construction, both dams were modified with new designs and raised by covering or partially covering these log structures with earthfill. Minimal records documenting these changes for both dams are available in the form of 1965 to 1967 design sheets and "as built" drawings generally showing what was to be done or supposedly what was done. Discrepancies between the design and as-built sheets and between as built data have been an issue in previous studies and still exist.

The original crib dams were constructed by a private company. The U.S. Forest Service obtained the land where the dams are located sometime around 1940 and maintained them under their inventory until ownership was transferred to the City of Wrangell in the late 1990's. There are drawings indicating the dams have been raised at least twice, sometime in the 1940's and at some time in the 1960's.

1.2 Previous Studies

Most of the information available about the dams has been generated from the 1960's drawings and information compiled during an investigation performed by Shannon and Wilson, Inc. (Shannon and Wilson) in 1985. Much of this work relied heavily on the assumption that the limited "as built" information was correct and adequately depicted actual conditions within the dam.

In May 1992, the dams were inspected by the U.S. Forest Service as part of their annual inspection program. During this and subsequent inspections, water seepage was observed coming from several feet above the toe of the Upper Dam, triggering concerns about piping and reduced overall dam stability. Shannon and Wilson was requested to perform a safety inspection of the dam in September 1992.

Shannon and Wilson conducted a study of the Upper Dam's toe area with test pits, toe clearing and installation of weirs in an effort to address stability and toe seepage concerns. These investigations revealed soft or loose foundation sediments in the toe area of the Upper Dam to depths greater than 8 feet. The conclusion was that additional studies were needed.

In May 1993 Shannon and Wilson completed a stability study of the Upper and Lower Wrangell dams. The study included nine modified SPT borings and five probes at the two sites. Laboratory tests included moisture contents, grain size analysis and R-tests on recompacted disturbed samples as they were unable to obtain satisfactory undisturbed samples. The SPT's were performed using the standard 140-lb hammer, but the sampler size was 2.5 inches and not the standard 2 inch sampler.

The information in the Shannon and Wilson report was used to support the finding of this investigation. The general soil profiles and soils data obtained in 1993 was used to augment data obtained during site investigations in August 2005.

In June 2004 Shannon and Wilson presented the results of the periodic safety inspection conducted for the Upper and Lower Wrangell Dams on April 15, 2003. The report reiterated the findings of the 1993 report that the dams are marginally stable under static and steady seepage conditions, but are not stable under seismic conditions. There were discussions of a rock buttress having been constructed at the toe of the Upper Dam but there are no construction records of this buttress and subsequent surveys indicate that the dam profile has not changed since 1993. Seepage monitoring weirs had been placed at the toe of each of the dams but have since been removed.

2 Project Description

2.1 General Site Description

The Wrangell Upper and Lower Dam System consists of two earthfill dams and reservoirs which provide for the main water supply to the City of Wrangell, Alaska. The two dams are located on Wrangell Island near the City of Wrangell as shown in Figure 2.1. The dams and reservoirs are both on Mill Creek about 1500 feet apart and are situated in a narrow drainage-way about ½ to 1 mile southeast of the City. Most of the

Figure 2.1 Locality Map for Upper and Lower Wrangell Dams

surface water in the watershed drains down steep rock slopes to the southeast and is collected in the reservoirs. The dams are owned by the City of Wrangell. The two dams are earthen structures approximately 28 feet high and 310 to 320 feet long. The elevation difference between the two dams is 64 feet.

The upper dam has a 22 to 28 foot wide crest, coarse granular slopes on both the up and downstream sides, and retains water via an internal crib core. Prior to 1958 the Crib was its own water retention structure, but leaked excessively and held little to no water. Available records suggest that about 1958 it was covered with fill on both the up and downstream slopes to impound water and was raised again in about 1967 with additional fill to create its current shape. Survey data of the upper dam is shown in Plate 1.

Figure 2.2 Upper Dam – Looking across spillway to upstream side of dam

The Lower Dam, shown in Plate 2, has a much smaller 12 foot crest width with coarse granular embankment slopes and an internal central sheet pile and treated timber core. A log crib dam once retained water at this site. The current dam was installed just upstream of the crib dam to retain a higher reservoir level and the middle of the crib dam was removed to install the present outlet works. The remains of the log crib dam are visible in the downstream toe of the current dam.

Figure 2.3 Lower Dam – Looking across the crest to the right abutment and spillway

2.2 Hazard Classification

At present the Dam Safety and Construction Unit (Dam Safety) of the Alaska Department of Natural Resources has assigned both the Upper and Lower Dams a Class I (high) hazard potential classification. The periodic dam safety report prepared by Shannon & Wilson Engineers dated June, 2004 concluded that both the Upper and Lower Dams should have a hazard rating of Class II. Dam Safety has indicated that a more detailed study will need to be performed in order to justify the lower classification.

3 Field Explorations and Laboratory Testing

3.1 Drilling

Six borings were advanced at the two dam sites to evaluate the subsurface conditions. Three holes were drilled in the Upper Dam and three holes were drilled in the Lower Dam. The borings were designated DH-1-05 thru DH-6-05 to distinguish them from the borings performed by Shannon & Wilson in 1993 (B-1 thru B-9). The locations of the 2005 and 1993 borings are shown in Plates 1 and 2.

The drilling work was accomplished between the 8th and the 16th of August 2005. The borings were advanced with SPT sampling every 5 feet to depths ranging from 20 feet to 70 feet. All holes were drilled with a Mobile B-61 drilling rig using hollow stem continuous flight (5 inch ID by 8 inch OD) augers to advance the borings. Water was used below the water table to help prevent up heave in the bottom of the hole. The individual logs of the borings are presented in Appendix A.

The drilling work was subcontracted to Denali Drilling, Anchorage, Alaska. Their operations were continuously observed by an experienced engineer and/or geologist from the Walla Walla District Army Corps of Engineers.

3.2 Sampling

Sampling of the embankment and foundation soils in all borings was accomplished at approximately 5 foot intervals. Samples for classification purposes were obtained by driving with a hammer a split spoon sampler into the undisturbed soil at the bottom of the advancing hole using standard penetration sampling procedures. A 140-pound hammer was used to drive the 2.0-inch O.D. samples, 18 or 24-inches into the undisturbed soil. The drill rig used the standard 2 wraps on the manually operated cat head. The samplers were not sleeved. The soil recovered in the sampler was placed in airtight containers and sent to the laboratory for detailed examination and classifications testing, as necessary.

3.3 Piezometers

Piezometers were installed in all six borings following their completion to measure depth to groundwater or piezometric pressures within the embankment or foundation materials. Each piezometer consists of a 2-inch diameter slotted plastic tip connected

Figure 3.1 Drill Rig Located on Upper Dam Crest

to 2-inch plastic riser pipe. In tip areas, the hole was backfilled with dry clean bag sand, while the remainder of the hole to the ground surface was sealed with a bentonite slurry seal. Piezometer depths are noted on the boring logs in Appendix A.

3.4 Laboratory Testing

Laboratory tests were performed on representative samples from the borings to confirm the field classification and to evaluate the engineering properties of the subsurface materials. The main goal of the tests was to evaluate the percentage of fines for liquefaction calculations. Strength tests had been performed on samples during the 1993 analysis and it was felt that they were appropriate for the level of investigation of this program. Test data is presented in Appendix A.

4 Subsurface Data

4.1 Typical Cross Sections

Both the Upper and Lower Dams were surveyed and this data, along with the field testing data, was used to prepare typical cross sections of the dams. The typical cross sections used in the stability analyses are shown in Figures 4.1 and 4.2. The cross sections are also presented in Plates 3 and 5, respectively, with boring log data and piezometer information attached. The cross section shows the general subsurface profiles estimated from the data obtained from both the 2005 and the 1993 investigations. Additionally, we incorporated the information contained in the 1993 Shannon and Wilson report on their findings and interpretation of previous drawings and reports. Longitudinal cross sections with boring log data are presented in Plates 4 and 6.

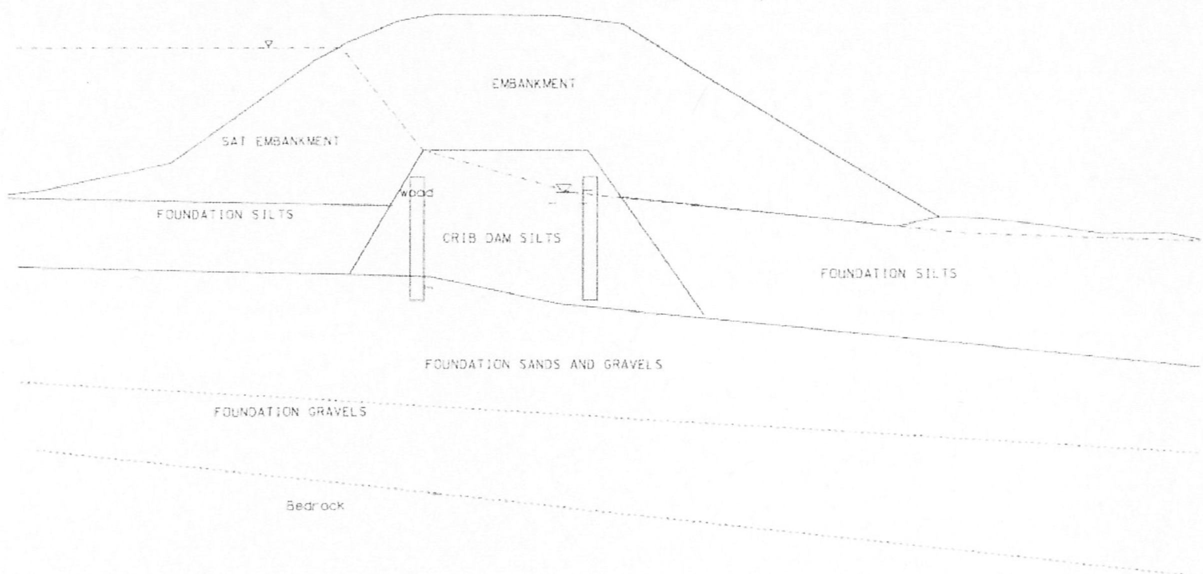

Figure 4.1 Upper Dam Cross Section for Stability Analyses

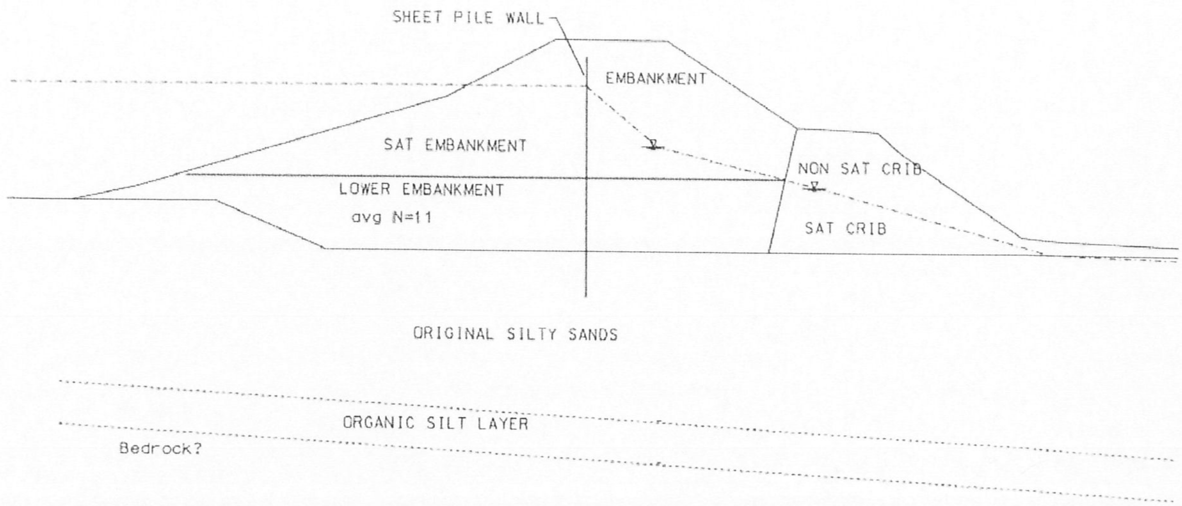

Figure 4.2 Lower Dam Cross Section for Stability Analyses

4.2 Foundation Materials

Shannon and Wilson covered the geology of the area in both their 1986 and 1993 reports. The steep rock slopes in the left abutment are covered with a thin veneer of surface organics, while the right abutment rock appears to be covered with a thin layer of glacial sediments, probably a glacial till consisting of equal parts sand, silt and gravels. Based on surface exposures and the boring data from both the 1993 and our 2005 investigations, the general slope or dip in the rock surface across the valley is depicted in Plates 3 and 5.

Boring data indicates that the valley floor materials are a mixture of sands, silts and gravels and likely represent a mixture of slide debris and alluvial materials deposited as stream sediments over time. Based on the gradation data from both the 1993 and the 2005 testing, the fines content in the foundation soils ranges from 10 to 60 percent, although more commonly in the range of 20 to 30 percent. The foundation materials also contained a significant amount of organics scattered throughout the samples. The organics were found in the borings near the crib dam and the silty foundation soils, with higher concentrations in the samples taken just above rock. Organic contents were in the range of 10 percent with a couple of samples in the 40% – 60% range.

Penetration resistance data taken in the foundation materials under both the Upper and Lower Dams averaged uncorrected values of about 18-25 blows per foot. Laboratory data indicated that the materials were non-plastic. For analysis purposes they are therefore treated as cohesionless, c=0, materials.

4.3 Crib Materials

Crib materials consisting of wood, silts and sandy gravels were encountered in all the borings. Previous drawings of the Upper Dam showed that the crib was initially about 164 feet long, 5 feet high, rectangular shaped, and extended below the original ground line an unknown distance. Shannon and Wilson reported that their borings revealed the crib was approximately 15 to 18 feet in vertical dimension, meaning it was originally buried 10 to 13 feet below the ground surface. We confirmed Shannon and Wilson's comment that the crib's top elevation has subsequently settled although it doesn't appear to be uniform.

The crib for the Lower Dam does not appear to extend below the groundline and the downstream side of the crib is visible as the dam was constructed upstream of the crib.

Shannon and Wilson discuss the crib dams in detail and that information won't be repeated here. The 2005 borings generally agreed with the data that Shannon and Wilson obtained. The cribs are wood with silty sands between the crib walls. The average blow counts range from 8 – 17 blows per foot and material is very similar to the foundation silts.

Loose low-plastic silt was encountered on top of the Upper Dam crib. It is thought that this was placed in an attempt to get the dam to hold water. This silt is not present on top of the crib at the Lower Dam. Design drawings for the Lower Dam also show a sheet pile, partial wood and partial steel, cutoff wall. Neither the 1993 nor the 2005 borings were able to determine the exact location of this wall. We did not include this wall in our stability analyses.

4.4 Outer Embankment

Both dams have an outer granular shell. This material is courser and more compact than the silty sands in the crib dam and the foundation. The average blow counts in this material in the 2005 explorations ranged from 22 to 25 blows per foot. The material contains approximately 20% fines. The outer embankment is likely the material placed during the final dam raise during the 1960's.

The Upper Dam's spillway was originally on the left embankment and in natural ground. When the dam was raised in the 1960's the spillway was moved to the right abutment and placed in rock. The left abutment was filled and raised an additional 5 to 8 feet to the current elevations. The one boring in this area, DH-3-05, does not contain any of the silts from the crib or subsequent raises. The material below the embankment is very similar to the crib silts, but is slightly more dense and more granular.

4.5 Groundwater

Shannon and Wilson installed nine piezometers in 1993. All piezometers in the Upper Dam, except one at the toe had the riser pipes broken off. We were not able to locate the three piezometers in the Lower Dam. Six new piezometers were installed in August, 2005. All of these were covered with a locking steel cap to prevent damage from recreational vehicles.

During drilling we were able to locate piezometer B-7. Table 4-1 presents the piezometer readings after drilling was complete. They indicate normal water levels thru the dam. The upstream piezometers read reservoir levels while the toe (only in the Upper Dam) reads just below the surface. There is a seep along the left abutment of the Upper Dam that has been a concern. Shannon and Wilson investigated this and concluded that, although it should be monitored, it is not piping fines, and there were no voids, sink holes or subsidence observed to indicate internal piping.

Table 4.1 Piezometer Readings – Upper and Lower Dams

	Install	Apr-93		Aug 8 - 16 2005			
Piezo	Elev.	Water Depth	Water Elev	BOH Depth	Elev. BOH	Water Depth	Water Elev
Upper Dam							
B-1	362.7	12	**350.7**	19.3	343.4	5.2	**357.5**
B-2	362.7	25	**337.7**	27	335.7	Dry	**DRY**
B-3	336	2	**334**	11	325	2	**334**
B-4	361	27	**334**	29.8	331.2	27.8	**333.2**
B-5	363	20	**343**	22.7	340.3	16.8	**346.2**
B-6	362	20	**342**	23.3	338.7	20.8	**341.2**
P-1-05	362			23.6	338.4	21.4	**340.6**
P-2-05	362			26	336	25.3	**336.7**
P-3-05	362.5			24	338.5	15.3	**347.2**
Lower Dam							
B-7	298	13	**285**	27.5	270.5	12.8	**285.2**
B-8	298	20	**278**				
B-9	287.9	12	**275.9**				
P-4-05	298			21.5	276.5	18	**280**
P-5-05	298			25	273	12.5	**285.5**
P-6-05	289			11.9	277.1	Dry	**dry**

5 Geology and Seismology

5.1 Purpose and Authority

This geological and seismological evaluation is being prepared in support of a Seismic Safety Analysis of the Wrangell Water Supply Dams located in Wrangell, Alaska. The analysis is being prepared for Alaska District Army Corps of Engineers who are assisting the City of Wrangell, Alaska under the authority of the planning assistance to the states.

The Wrangell Upper and Lower Dam System consists of two earthfill dams and reservoirs which provide for the main water supply to the City of Wrangell. The stability of the two earthen dams was called into question as a result of a stability study performed by Shannon & Wilson dated May 1993. The report indicates that "While stable under static load conditions, they do not even closely meet current design standards under dynamic loading and… both could fail under a future strong earthquake". The City is operating the dam under a conditional permit that requests the community perform a seismic study along with additional items listed as special conditions. The initial assessment as a result of visual inspection and review of the existing data is that the dam may be moderately unstable and could require some significant rehabilitation (Shannon & Wilson, 1993).

5.2 Project Description and Hazard Potential

The Wrangell Upper and Lower Dam System consists of two earthfill dams and reservoirs which provide for the main water supply to the City of Wrangell. The dams and reservoirs are both on Mill Creek about 1500-feet apart and are situated in a narrow drainage way about 1-mile southeast of the city. The dams are owned by the city of Wrangell with the land being under long term lease from the USDA Forest Service.

The two dams are both about 28-feet high from original ground surface and 320-feet wide with an elevation difference between the two structures of 64-feet. Both dams contain timber crib cores that were the original water retaining structures at the site. The embankment sides are steep and made up of course material. The original crib structures were covered with fill on the upstream and downstream slopes and the structures were raised with additional fill.

The Project Hazard Potential Classification as described in Appendix E of ER 1110-2-1155 rates Wrangell Dams as Significant due to rural location and disruption of essential facilities and access.

5.3 Geology

Glacier ice has advanced over the area at least once and probably several times during the Pleistocene Epoch. There presently are no glaciers on Wrangell Island but glaciers are present nearby on the mainland (Lemke, 1974).

The bedrock consists of a sequence of metamorphic rocks intruded by igneous rocks. The metamorphic and plutonic rocks are part of the Gravina belt of rocks that extends for many miles to the southeast and to the northwest (Brew, 1997). The igneous rocks are part of the Coast Range Batholith that forms the Coast Range Mountains of the adjacent mainland to the northeast. The primary bedrock in the vicinity of the dams is a shist or phyllite of Cretaceous age.

5.4 Southeast Alaska Seismicity and Faulting

Faulting and the associated seismic activity varies widely in southeast Alaska. The Fairweather/Queen Charlotte fault system borders southeast Alaska to the west and is associated with high magnitude earthquakes. Faulting and earthquake activity diminishes to the west to very little activity in the coastal range of Canada. Wrangell Island lies near the eastern border just off shore of the coast range. See Figure 5.1 that shows the location of Wrangell and faulting in Southeast Alaska. The following is a discussion of the prominent faults and earthquake activity in southeast Alaska.

<u>The Fairweather/Queen Charlotte Fault System</u>: The Queen Charlotte fault is the southern extension of the fault system and the portion of the fault that lies closest to Wrangell, AK. The fault is about 190-kilometers from Wrangell at its closest point. The Queen Charlotte and Fairweather faults are part of a long fault system that mark the eastern boundary of the Pacific plate and western boundary of the North American plate. The fault associated with this transform boundary is right-lateral strike-slip (Wesson and Others, 1999). This fault is capable of a M8+ (from length) has a slip rate of 58mm/yr and a recurrence time of 130 years for the characteristic earthquake (Nishenko and Jacob, 1990).

Four major earthquakes have been linked to the Queen Charlotte-Fairweather fault system. In 1927, a M7.1, in 1949, a M8.1, in 1958, a M7.9, and in 1972, a M7.4. All but the 1949 earthquake were felt in Wrangell Alaska. The shaking felt in Wrangell was from IV to V on the Modified Mercalli intensity scale.

<u>Denali Fault (Chatham Strait Fault)</u> is about 130-kilometers from Wrangell at its closest point. This is the second largest right lateral strike-slip fault in southeast Alaska. Geologic evidence shows that the fault was active as recently as the mid tertiary period and had a total right lateral displacement of up to 150-km. This fault is capable of a M8+ (from length) has a slip rate of 2-mm/yr and a recurrence time of 1900 years for characteristic earthquake (Plafker and others, 1993). Few earthquakes appear to be related to this fault in southeast Alaska.

Canoe Passage Fault While no historic or Quaternary record of movement has been recorded along this fault there are displacements mapped along the fault indicating offset of late Cenozoic units (less than 24 million years) Plafker and others, 1993. The movement was mapped about 50-km to the south (Koch and others, 1977, Plafker and others, 1993) and the fault is mapped as extending to the north to within 6 kilometers of Wrangell (Brew, 1997).

Tongass Narrows Fault is 75-kilometers from Wrangell and may have Quaternary movement associated with it (Plafker and others, 1993)..

Coast Range Megalineament Zone(CRML) is about 30-kilometers from Wrangell. The zone is defined locally by discontinuous and overlapping, en echelon topographic lineaments, is about 800-km long, and runs NNW-SSE (Brew and Ford, 1978, 1998). This zone is also referred to as the Coastal Range Lineament described by Twenhofel and Sainsbury (1958) and referred to by Lemke (1974). Movement within the zone has been noted in the vicinity of Juneau, where young glacial deposits have been offset. A random area earthquake source of M7.4 with a+-10,000-yr return and M6.2 with +-100-year return for the CRML was estimated by Davis and Pulpan, 1980 (R&M, 2004, CWDD, 1980)

Other faults that have been mapped in the vicinity of Wrangell, Alaska are the Fools Inlet Fault about 20-kilometers away and Virginia Lake Fault about 15-kilometers away. These faults, while near are not considered active.

In general, the only known active faults are greater than 150-kilometers from Wrangell, Ak. This makes for a low probability of significant seismic activity in the vicinity of Wrangell. However, the region is sparsely populated and there are few strong motion seismic stations in southeast Alaska, making for a limited seismic record in the vicinity of Wrangell.

5.5 Historical Seismicity

An intense seismic database search was performed using the AEIC (Alaska Earthquake Information Center) database to evaluate the seismic record at the site. Figure 5.1 shows all recorded earthquake epicenter greater than M-3. For this report M = Magnitude and MM = Modified Mercalli Intensity.

5.6 Design Earthquake and Motions

The site is in seismic zone 2B of the Uniform Building Code Seismic Zone Map and the dams have a hazard potential of significant. ER 1110-2-1806 , (5.c.) provides that detailed site explorations, site-specific ground motion studies, and structural analyses should be undertaken only for projects in zones 3 and 4, or for zone 2A and 2B projects when seismic loads control the design. It has not been determined weather seismic loads control the design for the Wrangell site. The stability study prepared by Shannon and Wilson in 1993 treated the site as being in seismic zone 3 according to Corps of Engineers criteria at the time. The analysis used a horizontal seismic coefficient of 0.10 based on being in zone 3.

Guidance provided in the WES Report 29: Selection of Earthquake Ground Motions For Engineering (Krinitzsky, 1996) suggests that "for noncritical structures, or for critical structures in those areas of low seismic threat (<0.15g), deterministic procedures can be used and may be preferred for the MCE but they are relatively expensive and they may not be warranted because of limited concerns for seismic hazards. For these circumstances, analyses based on published maps of probabilistic ground motions can be used". The Wrangell site is a noncritical structure and all indications are that the seismic threat is less than 0.15g.

A ground motion estimate was determined using maps prepared by the USGS under the National Seismic Hazard Mapping Project. These Probabilistic Seismic Hazard Maps of Alaska were prepared by Wesson and others in 1999. The seismic potential of Alaska is captured through consideration of earthquake sources that could be explicitly identified. Further, due to limited geologic data the recurrence assumptions are based on instrumental seismic data. The ground motion values are calculated for firm rock sites which correspond to a shear-wave velocity of 760 m/sec. in the top 30 m.

The values determined from the USGS Seismic Hazard Maps are as follows:

For the 2500-year event:
The 0.20 Sec. horizontal acceleration is 0.10-g with 2% probability of exceedance in 50-years.

For the 475 year event:
The peak horizontal acceleration is 0.060-g with 10% probability of exceedance in 50-years.

For the 145 year event:
The peak horizontal acceleration is reduced to 0.032-g (as shown on the attached work sheets)

Given these values the ground motion for the Maximum Credible Earthquake (MCE) would be the 2500-year event at 0.10-g. For the Operating Basis Earthquake (OBE) the ground motion would be the 145-year event of 0.032.

Given the limited amount of geologic and seismic data available for the Wrangell area these values may be too low and perhaps a deterministic method of evaluation should be performed for the site. However, a full site specific deterministic evaluation of the site is not within the scope of this evaluation and it may not be warranted as previously indicated. Given that the basic research for a deterministic study is presented in this report several values for ground motions are presented that reflect what a deterministic site specific study may determine.

There are three mapped faults in the vicinity of Wrangell Island the Canoe Passage Fault, Fools Inlet Fault, and the Virginia Lake Fault. Both the Fools Inlet and Virginia Lake Faults are inactive. The Canoe Passage fault has displacements mapped along the fault indicating offset of late Cenozoic units (less than 24 million years) Plafker and others, 1993. This fault extends to within 6 kilometers of Wrangell (Brew, 1997, B-1, B-2). While this fault has little evidence of any recent movement in the vicinity of Wrangell this could be a source for ground motion. A better choice, however, may be the Coast Range Megalineament (CRML).

The 800-kilometer long CRML is about 30 kilometers from Wrangell and late Pliestocene displacements have been mapped along the lineament in the vicinity of Juneau. Using the CRML as an earthquake source for the Wrangell area and the suggested magnitudes provided by Davis and Pulpan, 1980, the following horizontal accelerations were determined. Using a M6.2 floating earthquake for the OBE and a M7.4 for the MCE and using attenuation tables developed by Krinitzsky and others, (1988) a ground motion acceleration of 0.1g for the OBE and 0.23 for the MCE were estimated. These values are similar to ground motions used in other seismic evaluations performed in the area (R&M, 2004, CWDD, 1980). For critical features the Maximum Design Earthquake (MDE) is the same as the MCE.

5.7 Conclusions

The Wrangell Dams have a significant hazard potential classification. The site is in seismic zone 2B of the Uniform Building Code Seismic Zone Map. Probabilistic values for ground motion were determined using the Probabilistic Seismic Hazard Maps of Alaska. These values are extremely low and higher values of ground motion are recommended for the Wrangell Seismic evaluation using a direct seismic source. The Coast Range Megalineament is used for a seismic ground motion source and a MCE is estimated at 0.23g and an OBE is estimated at 0.1g.

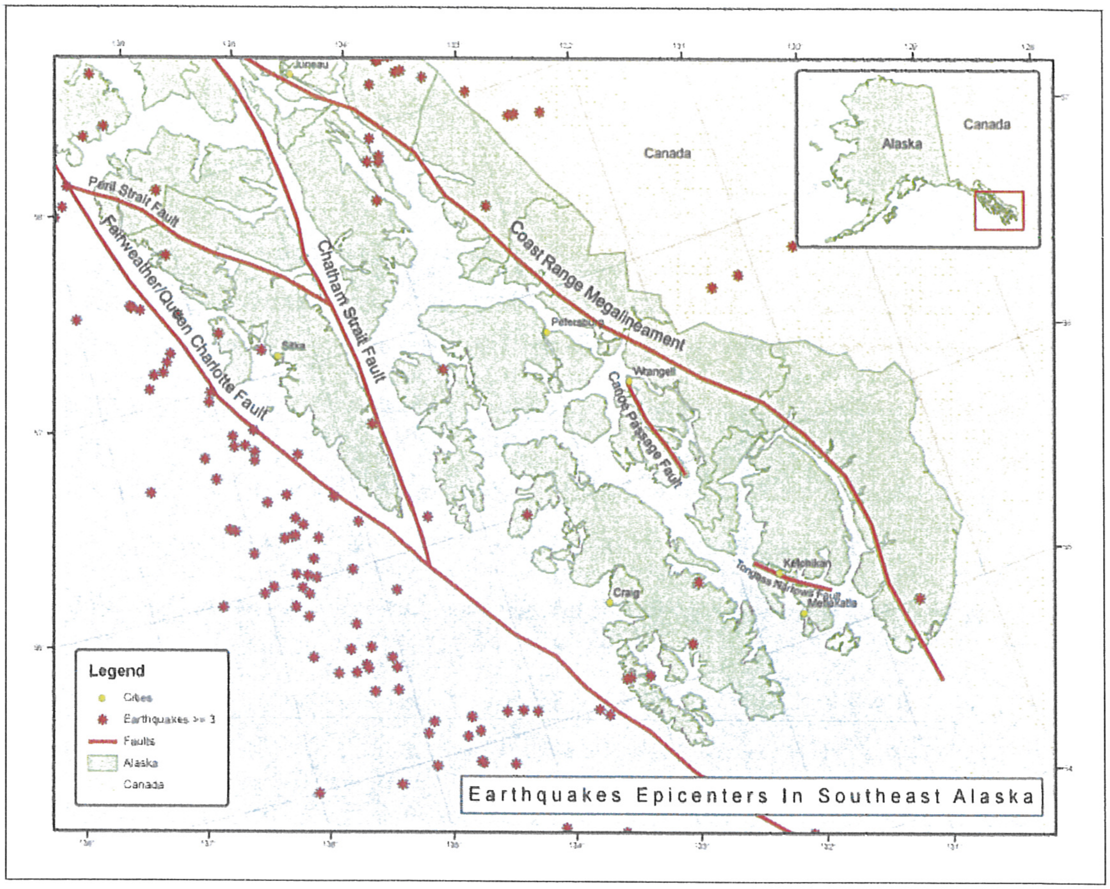

Figure 5.1 Earthquake Epicenters in Southeast Alaska

6 Liquefaction Potential

6.1 *Subsurface Information*

For the purpose of running the liquefaction analyses it was decided to only use the data obtained from the 2005 subsurface investigation. The data obtained during the 1993 investigations was not obtained using standard SPT equipment. Although there are correlations for larger samplers, the sampler and energy used in the 1993 investigations does not meet any of the published correction factors. Additionally, the blow count data from the 1993 samples was consistently lower in the silts and sands. It was felt that for this evaluation it was appropriate to remain with consistent soil investigation procedures for this part of the analysis.

6.2 *Liquefaction Analysis and Susceptibility*

The 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/NSF (National Science Foundation) Workshop on Evaluation of Liquefaction Resistance of Soils recommendations for liquefaction analysis were used to evaluate the soils under the Upper and Lower Wrangell Dams. A number of liquefaction calculations were run using the design ground motions and the data from soils borings performed in August 2005. The analysis was performed using both the Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE) earthquakes. Equations used for the liquefaction analysis are shown in Figure 6.1

6.3 *Cyclic Stress Ratio (CSR)*

The Cyclic Stress ratio was determined using the following equation in accordance with the 1998 NCEER/NSF Workshop (Workshop):

Cyclic Stress Ratio = CSR = $0.65(a_{max} / g)(\sigma_{vo} / \sigma'_{vo})r_d$

The value for r_d was determined using the equations suggested for routine practice as shown in Figure 6.1.

6.4 *Liquefaction Resistance (CRR)*

Liquefaction resistance was evaluated based on the SPT data obtained during the 2005 investigations. The value is based on the CRR versus $(N_1)_{60}$ plots reproduced in Figure 6.2. In this plot the SPT blow count is normalized to an overburden pressure of

Liquefaction Calculations

(based on 1996 NCEER and 1998 NCEER/NSF Workshops)

FSL = CRR/CSR*MSF*K_σ

Cyclic Stress Ratio = CSR = $0.65(a_{max} / g)(\sigma_{vo} / \sigma'_{vo})r_d$

$r_d = 1.0 - 0.00765z$ for $z < 9.15$ m
$r_d = 1.174 - 0.0267z$ for $9.15\text{m} < z < 23$ m

z = depth below the ground surface in meters

Cyclic Resistance Ratio = $CRR_{7.5} = \dfrac{1}{34-(N_1)_{60cs}} + \dfrac{(N_1)_{60cs}}{135} + \dfrac{50}{(10(N_1)_{60cs}+45)^2} - \dfrac{1}{200}$

$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$

$\alpha = 0$ for $FC \le 5\%$
$\alpha = \exp[1.76 - (190/FC^2)]$ for $5\% < FC < 35\%$
$\alpha = 5.0$ for $FC \ge 35\%$
$\beta = 1.0$ for $FC \le 5\%$
$\beta = [0.99 + (FC^{1.5}/1000)]$ for $5\% < FC < 35\%$
$\beta = 1.2$ for $FC \ge 35\%$

Magnitude Scaling Factor = MSF = $\dfrac{10^{2.24}}{M_w^{2.56}}$

$K_\sigma = (\sigma'_{vo} / P_a)^{(f-1)}$

Where σ'_{vo} and P_a (atmospheric pressure- approx. 1 tsf) are measured in the same units

$f = 0.8$ for $D_r \le 40\%$
$f = 0.7$ for $D_r \approx 60\%$
$f = 0.6$ for $D_r > 80\%$

Figure 6.1 NCEER/ NSF Equations for Liquefaction Potential Evaluation using the SPT approximately 1 ton/sq foot and a hammer energy ratio or hammer efficiency of 60%. This plot is represented by the following equations by A.F. Rauch, University of Texas:

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{(10(N_1)_{60cs}+45)^2} - \frac{1}{200}$$

It should be noted that this equation is valid for $(N_1)_{60} < 30$, For $(N_1)_{60} > 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable. During our evaluations we used a spreadsheet to determine the value of CRR. If the clean sand blow counts were above 30 the value returned was NA for not applicable.

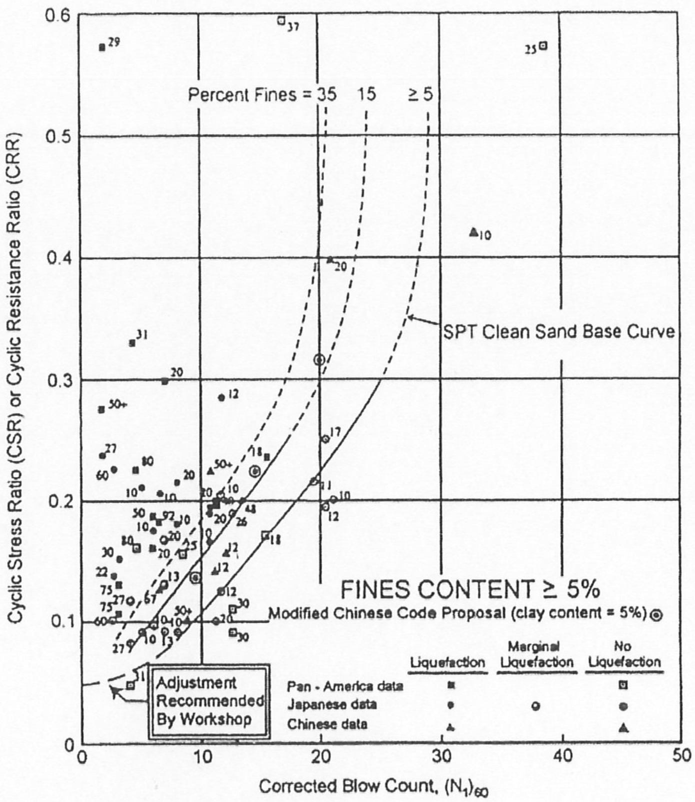

Figure 6.2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Histories (Modified from Seed, et al. 1985)

The workshop recommended the following equation to correct CRR for the influence of fines content as shown below. The equations for α and β are as shown in Figure 6.1.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$$

6.5 Other Corrections

In addition to normalizing the SPT $(N_1)_{60}$, the SPT blow counts were corrected for various factors such as overburden pressure, energy ratio, borehole diameter, rod length and samplers without liners. The general corrections are presented in Table 6.1.

The correction factor for overburden pressure, C_N used the following equation presented in the NCEER/NSF Workshop. This equation limits the maximum value of C_N to 1.7 as suggested by Workshop participants:

$$C_N = 2.2/(1.2 + \sigma'_{vo} / P_a)$$

TABLE 6.1 Corrections to SPT (Modified from Skempton 1986) as listed NCEER/NSF Workshop (1998)

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

6.6 Magnitude Scaling Factors (MSFs)

The clean-sand base, or CRR, curves in Figure 6.2 apply only to magnitude 7.5 earthquakes. To adjust the clean-sand curves to a magnitude small than 7.5, Seed and Idriss (1982) introduced correction factors termed "magnitude scaling factors (MSFs)." The workshop participants presented numerous scaling factors and an equation for the best fit line representing these values. This equation was used for ease of calculation and due to the level of investigation in this study. The equation is shown in Figure 6.1.

6.7 *Correction Factor K_σ and K_α*

Correction factors for high overburden and static shear stresses were developed by Seed (1983) to extrapolate the simplified procedure to larger overburden pressure and static shear stress conditions than those embodied in the case history data set from which the simplified procedure was derived. The Workshop had two main recommendations in these areas. The equation for K_σ is as presented in Figure 6.1. The recommendation is to limit the correction factor to a value of 1.0 for vertical effective stress less than 1 tsf. This is shown below in Figure 6.3.

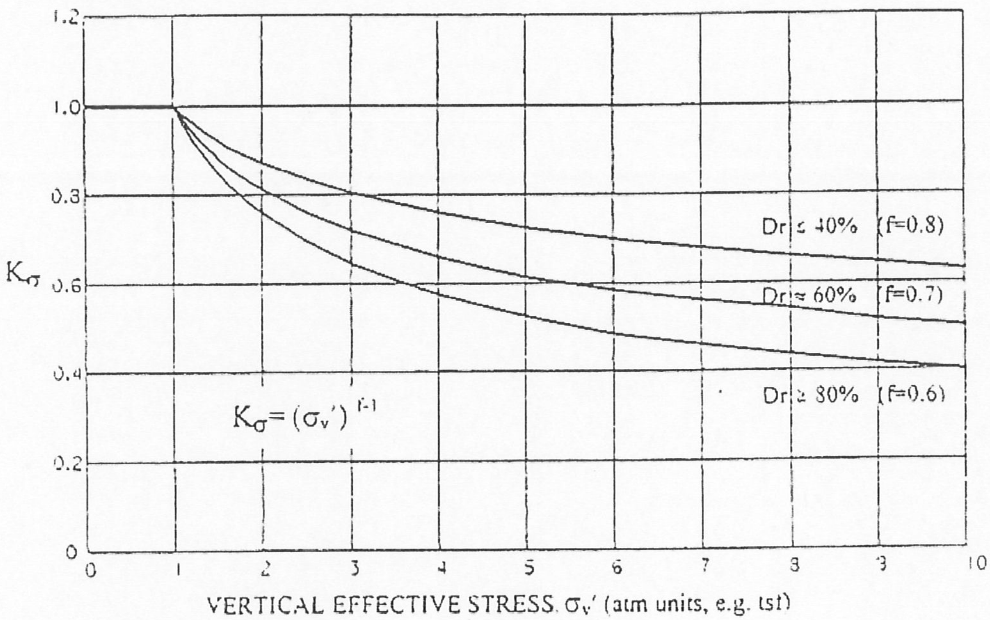

FIGURE 6.3 Recommended Curves for Estimating K_σ for Engineering Practice (NCEER/NSF 1998)

6.8 *Upper Dam Liquefaction Results*

Liquefaction potential results are presented in Table 6.2 for the Upper Dam for the OBE and the MCE. Each factor of safety was calculated separately for each SPT. The percent fines obtained during laboratory testing were used and the values were averaged or estimated where lab data was not obtained.

The Factors of Safety calculated in the silty sands of both the crib dam and the foundation materials for the OBE are well above values of concern for liquefaction. However, the values calculated for the MCE are well below safe values, and it can be assumed a significant portion of the dam and foundation will experience liquefaction given the magnitude of the MCE event.

TABLE 6.2 Upper Dam - Calculated Factors of Safety for Liquefaction

	OBE			MCE		
	DH-1-05	DH-2-05	DH-3-05	DH-1-05	DH-2-05	DH-3-05
Depth	FS_L	FS_L	FS_L	FS_L	FS_L	FS_L
5	--	--	--	--	--	--
10	--	--	--	--	--	--
15	--	--	6.15	--	--	1.36
20	2.93	--	2.93	0.72	--	0.81
25	2.28	--	4.23	0.69	--	1.29
30	2.37	2.77	3.70	0.82	0.73	1.03
35	2.28	3.91	2.22	0.60	1.03	0.65
40	4.39	2.44	2.77	1.21	0.67	0.73
45	3.09	4.72		0.89	1.36	
50	--	4.88		--	1.33	
55	4.52	1.99		1.29	0.57	
60	--	--		--	--	
65	6.10			1.72		

6.9 Lower Dam Liquefaction Results

Liquefaction potential results are presented in Table 6.3 for the Lower Dam for the OBE and the MCE. Each factor of safety was calculated separately for each SPT. The percent fines obtained during laboratory testing were used and the values were averaged or estimated in between where lab data was not obtained.

The Factors of Safety calculated for the OBE are well above values of concern for liquefaction. However, the values calculated for the MCE are less clear. The majority of the Lower Dam is above the FS_L = 1.4 considered for potential liquefaction (Seed and Harder, 1990) during the MCE event. There were two samples in DH-5-05 that had low blow counts. One was in the silty sands of the crib dam and the other was in the organic silt just above rock. These need to be investigated further before one can safely conclude that the dam is not susceptible to liquefaction. Additionally the soils at the toe of the dam have not been adequately sampled and may also be susceptible.

The excel spreadsheets of the liquefaction calculations for the six boreholes are located in Appendix B.

TABLE 6.3 Lower Dam – Calculated Factors of Safety for Liquefaction

	OBE			MCE		
Depth	DH-4-05 FS	DH-5-05 FS	DH-6-05 FS	DH-4-05 FS	DH-5-05 FS	DH-6-05 FS
5	--	--		--	N/A	
10	--	5.97		--	1.72	
15	8.14	--	4.88	2.22	--	1.79
20	6.04	3.46	--	1.68	0.93	--
25		6.69			1.91	
30		--			--	
35		--			--	
40		--			--	
45		2.17			0.62	

7 Existing and Post Earthquake Stability Analyses

According to the EC 1110-2-xxxx, after a determination has been made related to the potential of the soil to liquefy the engineer will either conclude that liquefaction will not occur in which case the analysis turns toward evaluating the dam for internal instabilities or the engineer will conclude that liquefaction will occur and the analysis turns toward evaluating the dam for weakening instabilities. The liquefaction analysis shows that we can expect liquefaction during the MCE in the Upper Dam and should also analyze the effects of liquefaction in the Lower Dam during the same event.

If liquefaction triggering analysis reveals that the site soils have the potential to liquefy, then further analyses should be aimed at determining values of deformation as a consequence of weakening instability. The major concerns in weakening instabilities are the generation of excess pore pressures that reduce the shear strength of the soil, and the concern that the earthquake will cause some type of structural disturbance reducing the soil strength. There are two main types of liquefaction related failures, flow failures and deformation failures. Flow failure analysis generally involves a significant reduction in soils strength producing large deformation and severe damage. The evaluation most typically consists of conventional static slope stability analyses using soil strengths based on conditions at the end of the earthquake (Marcuson and Hynes 1990). This type of analysis will only indicate if flow failure is possible but not provide any information on the amount of movement expected. Techniques for evaluating the extent of the zone influenced by a flow failure are extremely limited; however it is sufficient to assume that very large unacceptable deformations will occur

This section begins by discussing the stability of the two dams under existing conditions. It then addresses flow failure potential, in the form of static equilibrium analysis, after the OBE event and the MCE event. The issue of deformation due to earthquake forces will also be discussed.

7.1 Existing Conditions

Stability analyses were performed on each of the dams under existing conditions. UTEXAS4 was used to evaluate the stability of the two dams. UTEXAS4 is software for limit equilibrium slope stability computations, which computes a factor of safety with respect to shear strength. Analyses were run on both the upstream and downstream slopes using both circular and non-circular failure surfaces. All of the analyses were run using the Spencer method.

The soils strengths values used in the stability analyses were determined using the data from the 2005 explorations and in a limited case, Shannon & Wilson's 1993 stability study (1993 Study). The cross sections used are shown on Plate 1 (Upper Dam) and Plate 3 (Lower Dam). The material strengths and unit weights are shown in Table 7.1. This table also shows the values used in the 1993 analysis.

Table 7.1 Upper and Lower Wrangell Dams - Soil Parameters for Stability Analysis

Upper Dam Wrangell					
Material	Description	Assumed Properties		Shannon & Wilson	
		γ	ϕ	γ	ϕ
Embankment	medium to dense Silty Sand with Gravels (SM)	130	37*	135	39
Sat Embankment	" "	134	37	----	----
Foundation Silts	Silty Sands, loose to med, some gravels, (SM)	120	29	114 **	30
Crib Dam Silts	Fill btwn the wood crib. Silty Sands, ls-med, sm gravels, (SM) sm wood.	115	29	114	29
Foundation Sands and Gravels	Med dense Silty Sands and Gravels (SM and GM)	125	31	114	30
Foundation SM and GM	Med dense Sands and Gravel. Some organic layers above rock.	130	34	Lower Failure Boundary	

Lower Dam Wrangell					
Material	Description	Assumed Properties		Shannon & Wilson	
		γ	ϕ	γ	ϕ
Embankment	med to dense silty sand and gravels (SM/GM)	125	37	135	37
Upstream Sat Embankment	Same as above	135	37	125	33***
Sat lower embankment	med to dense silty sand and gravels (SM/GM)	125	31	125	30
Non Sat Crib Dam	Silty Sands, loose to med, some gravels, (SM)	119	31	119	31
Crib Dam Silts Saturated	Fill btwn wood crib. Silty Sands, med, some gravels, (SM) w/ wood	119	31	119	31
Foundation Silty Sand	Med dense Sands and Gravels, some silts. (SM, SP)	120	33	114	30
Foundation organic layer	med dense Gravel, some sands and silts. Organic just above rock.	114	30		

* Shannon and Wilson report indicated higher blow counts in this material. This report and this strength value only reflects the 2005 soils testing.
** Shannon and Wilson assumed that 25% of this soils mass was comprised of wood with a unit weight of 70 pcf. This report assumed the material was SM.
*** The 1993 soils boring information in the lower dam indicated that the embankment material upstream of the cut off wall was weaker than the downstream material. We did not encounter this and therefore used the same ϕ for both materials.

Existing steady seepage conditions were evaluated and factor of safety values compared with those obtained in the 1993 Study. This was used as a starting point in the stability analysis. The factors of safety obtained running various scenarios for upstream and downstream failures are located in Appendix C. Also in the same appendix are the graphics outputs for these stability runs. A summary of the results and a comparison with 1993 Study results is located in Table 7.2.

The calculated factors of safety for both dams were comparable to the 1993 Study results. Some of the calculated factors of safety differ due to the location of the failure planes. Shannon and Wilson forced the failure planes to be deep in some cases which accounts for the differences in both the Upper and Lower Dams upstream circular analyses. This study did not look at the sudden drawdown case as this was analyzed during the 1993 study and found to be adequate for both the Upper and Lower Dams.

Table 7.2 Stability Analysis Results for Existing Conditions

		Upper Dam - Existing		**Shannon and Wilson**	
LOCATION	FAILURE TYPE	FACTOR OF SAFETY	Comment	FACTOR OF SAFETY	Comment
Downstream	circular	1.718	Deep Failure	1.720	Deep Failure
Downstream	circular	1.450	Mid-Crest Failure		
Downstream	non-circular	1.586	Deep Failure	1.560	Deep Failure
Upstream	circular	2.182	Deep Failure	1.790	Deep Failure
Upstream	non-circular	2.080	Deep Failure		
Upstream	non-circular	1.873	shallow failure	1.540	Shallow Failure

		Lower Dam - Existing		**Shannon and Wilson**	
LOCATION	FAILURE TYPE	FACTOR OF SAFETY	Comment	FACTOR OF SAFETY	Comment
Downstream	circular	1.474	Deep Failure	1.510	Deep Failure
Downstream	non-circular	1.572	Deep Failure	1.350	Deep Failure
Upstream	circular	2.179	Deep Failure	2.160	Deep Failure
Upstream	non-circular	2.192	Deep Failure	2.160	Deep Failure

7.2 Residual Shear Strengths

Post Earthquake stability analyses were preformed on both dams using residual shear strengths for potentially liquefiable soils. The residual shear strengths were determined using both the Seed and Harder and Baziar charts shown in Figure 7.1. Both charts were used to develop acceptable residual strength values for those soils with low factors of safety (FS_L less than or equal to 1.1). The values calculated and used in the stability analyses are shown in Table 7.3.

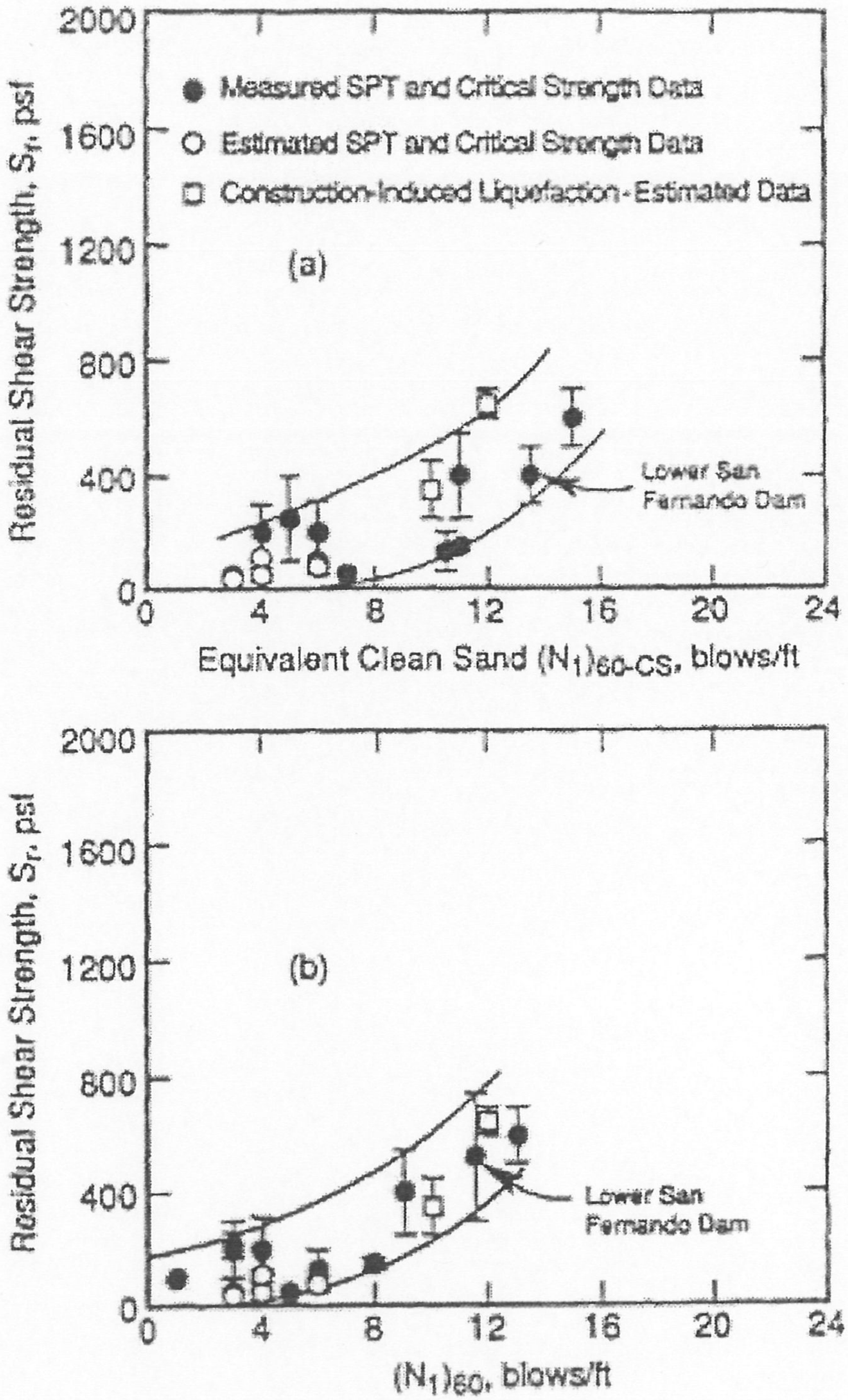

FIGURE 7.1 Proposed correlations between Residual Shear Strength and Normalized Standard Penetration Resistance for Clean and Silty Sands and Gravels: (a) with Correction for Fines (Seed and Harder, 1990): (b) without Correction for Fines, (Baziar, 1995)

7.3 *Residual Excess Pore Pressure*

Soils with intermediate factors of safety against liquefaction (FS = 1.1 to 1.4) should be assigned some strength value based on the expected residual excess pore pressure ratio (Marcuson and Hynes, 1989 and Seed and Harder) The effect of these earthquake-induced excess pore pressures was modeled as causing a reduction in available frictional strength (by means of reduced effective angle of internal friction, ϕ') for input into UTEXAS4. Material strengths were determined using a reduced effective angle of internal friction, calculated using the following expression (Hynes-Griffin, 1990):

$$\text{Sin } \phi'_{Ru\neq0} = (1-R_u) \text{ Sin } \phi'_{Ru=0}$$

R_u is the excess pore water pressure ratio, the ratio of earthquake–induced excess pore water pressure to pre-earthquake effective overburden stress. The residual excess pore pressures associated with factors of safety against liquefaction greater than 1.1 and less than 1.4 were estimated using Figure 7.2.

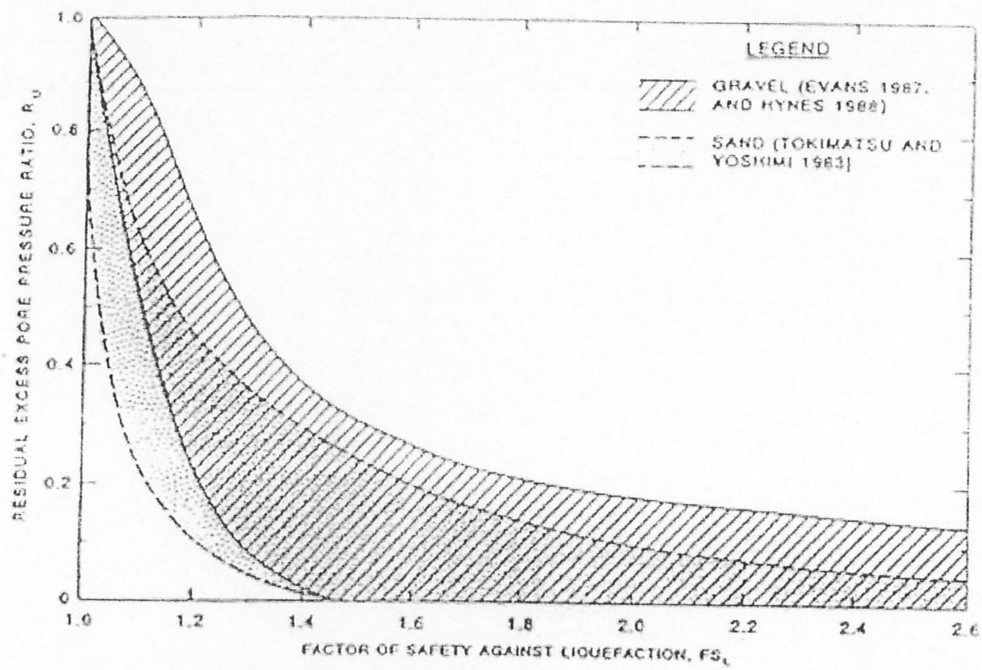

FIGURE 7.2 Typical relationships between residual excess pore pressure ratio and factor of safety against liquefaction for sand and gravel, from laboratory data (Hynes-Griffin, 1988)

It is noted here that the frictional strength relationship is as follows: $S = \sigma_n' \tan \phi'$. Dr. James Mitchell of Virginia Polytechnic University notes that to reduce tan ϕ' by $(1-R_u)$ is also an appropriate method for addressing the residual excess pore pressure.

However, Dr. Mitchell indicates that the use of the sin ϕ function is more conservative. The sin ϕ function was used here to be conservative.

Residual excess pore pressure ratios in saturated zones of embankment gravels and those foundation zones not considered susceptible to liquefaction ($FS_L > 1.4$) may still develop to about 10 percent, based on laboratory test data on dense gravels in other studies (Hynes-Griffin et al. 1988). The effective angle of internal friction for these soils was reduced from 10% to 25% depending on the FS_L using the following relationship.

$$\text{Tan}\ \phi'_{Ru\neq 0} = (1-R_u)\ \text{Tan}\ \phi'_{Ru=0}$$

In this case the less conservative Tan ϕ function was used as it was felt that the 10% to 25% R_u values were conservative enough. For a computed FS_L above 3.0 the effective strength was reduced using an R_u of 10%. For a computed FS_L between 1.5 and 3.0 the effective strength was reduced using an R_u of 25%. Values for R_u and the adjusted angles of internal friction for the Upper Dam and Lower Dam are shown in Table 7.3 and Table 7.4, respectively.

Table 7.3 – Properties for Limit Equilibrium Slope Stability Analysis – Residual Strengths and Reduced Angles of Internal Friction for the Upper Dam

Upper Dam Wrangell			OBE			MCE			
Material	Assumed Properties								
	γ	ϕ'	FS_L	R_u	ϕ'	Residual	FS_L	R_u	ϕ'
Embankment	125	37	-	--					37.0
Sat Embankment	130	37	5.60	.9 Tan ϕ'	34		1.4	0.25	26.8
Foundation Silts	120	29	2.80	.75 Tan ϕ'	22.5	S_r=434			
Crib Dam Silts	115	29	2.90	.75 Tan ϕ'	22.5	Sr=434			
Foundation SM/GM	125	31	3.80	.9 Tan ϕ'	28.0		1.25	0.35	19.6
Foundation SM/GM	130	34	4.50	.9 Tan ϕ'	31.0		1.7	.75 Tan ϕ'	27.0

Lower Dam Wrangell			OBE			MCE			
Material	Assumed Properties γ	ϕ'	FS_L	R_u	ϕ'	Residual	FS_L	R_u	ϕ'
Embankment	125	37			37.0				37.00
Sat embankment	135	37	3.00	.9 TAN ϕ	34.15		1.72	.75 TAN ϕ	29.47
Lower Embankment	125	31	3.00	.9 TAN ϕ	28.40	S_r=300	0.93		
Crib Silty Sand	119	31			31.0				31.00
Crib Silt Sand Sat	119	31		.9 TAN ϕ	28.40		2.4	.75 TAN ϕ	24.26
Foundation Silty Sand	125	33		.9 TAN ϕ	30.30		>3.0	.9 TAN ϕ	30.30
Foundation organic	114	30	2.17	.75 TAN ϕ	23.41	S_r=165			

7.4 Post Earthquake Slope Stability Analysis

Numerous stability analyses were performed on each of the two dams for both the upstream and downstream faces and with both circular and non-circular failure surfaces. The values for residual strength and for the adjusted phi angles were varied and checked for sensitivity to variations. The graphic results of the analyses are located in Appendix C. The following is a summary of the general findings.

Upper Dam

Using residual strengths for liquefied soils and reduced strengths for the remainder of the saturated soils in the Upper Dam the following factors of safety were obtained for the Post Earthquake Slope Stability Analysis. The Upper Dam is not stable for the MCE event. The values for the OBE event are above one. The results for the Upper Dam Stability Analysis are located in Table 7.5.

Lower Dam

The Factors of Safety for the Lower Dam are higher than the Upper Dam. The results for the downstream slope after the MCE event indicate that the dam should be further analyzed for that event. The results for the Lower Dam Stability Analysis under seismic conditions are located in Table 7.6.

Table 7.5 Upper Dam Seismic Stability Analysis Results

Upper Dam - MCE Stability Analysis			
ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
Seismic - MCE	Downstream	circular	0.066
Seismic - MCE	Downstream	non-circular	0.762
Seismic - MCE	Upstream	circular	1.364
Seismic - MCE	Upstream	non-circular	1.646
Upper Dam - OBE Stability Analysis			
ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
Seismic - OBE	Downstream	circular	1.170
Seismic - OBE	Downstream	non-circular	1.220
Seismic - OBE	Upstream	circular	1.794
Seismic - OBE	Upstream	non-circular	1.644

. **Table 7.** Lower Dam Seismic Stability Analysis Results

Lower Dam - MCE Stability Analysis			
ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
Seismic - MCE	Downstream	circular	1.061
Seismic - MCE	Downstream	non-circular	1.176
Seismic - MCE	Upstream	circular	1.686
Seismic - MCE	Upstream	non-circular	1.507
Lower Dam - OBE Stability Analysis			
ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
Seismic - OBE	Downstream	circular	1.316
Seismic - OBE	Downstream	non-circular	1.430
Seismic - OBE	Upstream	circular	1.956
Seismic - OBE	Upstream	non-circular	1.909

It is not possible to predict post-earthquake deformed shape with any certainty using limit equilibrium analysis. If limit equilibrium slope stability calculations, using post-earthquake material properties, do not clearly rule out development of deformations that threaten the reservoir retention capability of a high hazard dam, then more complex finite element analysis of the deformation potential of the dam is required. In the Upper

Dam, it is clear that more rigorous deformation analysis and likely, remediation are required.

7.5 Deformation Failure

Deformation failures as a consequence of liquefaction are smaller than those of flow failures and hence can be evaluated with finite element or finite difference programs and to some extent a suite of techniques that are largely empirical in nature.

One empirical technique is the approach presented by Baziar et al (1995). This was used for a preliminary analysis of deformation potential at both of the dams. The Baziar approach does not have a scale for earthquake magnitude but generally corresponds to earthquakes with moment magnitudes of $M_w \leq 8$. The right boundary in Figure 7.3 is the uppermost bound of the contractive behavior for the silty deposits in this study. Points that plot to the right of the Upper Bound will not have deformations greater than 3 feet. Those plotting within the Upper Bound have the potential of great than 3 feet of deformation. All of the borings in the Upper Dam had data that plotted within the Upper Boundary and one of the borings in the Lower Dam did. This indicates that Upper Dam has significant potential for large deformations that exceed three feet. The layer that the low blow counts were encountered in for the Lower Dam is presumed to be only 5 feet thick. If this layer were to deform it is unlikely that it would exceed 20% of its thickness or 1 foot. Although the Baziar method suggests potential for large deformations in the Lower Dam it appears unlikely. Graphs of this analysis are located in Appendix C.

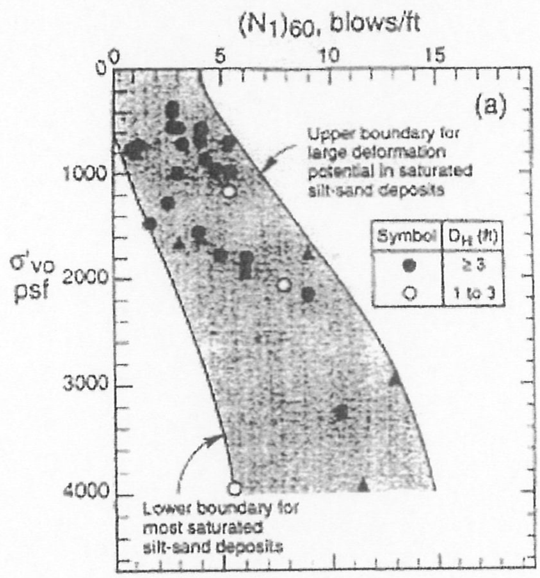

FIGURE 7.3 Normalized Standard Penetration Resistance $(N_1)_{60}$ to Vertical Effective Overburden Pressure, for Saturated Nongravelly Silt-Sand Deposits that have experienced Large Deformations (Baziar, 1995)

8 Conclusions and Recommendations

8.1 Conclusions

The scope of this study was to perform a detailed review of the seismic stability of each dam. This was accomplished by performing a Phase I – Preliminary Seismic Analysis of the Upper and Lower Wrangell Dams. Static stability of the dams was reviewed and compared to previous studies. The results confirmed the findings of the 1993 study that both dams are within the factor of safety guidelines for static stability.

The Preliminary Seismic Analysis involved a subsurface investigation consisting of 6 borings with SPT sampling, laboratory testing of disturbed samples and surveys of the dam topography. Design earthquakes were developed for both the OBE and MCE events.

This information was used to analyze liquefaction of the dam embankment soils and foundation materials. Following liquefaction calculations, stability analyses were run on both dams using residual and reduced strengths to determine the likelihood of flow failure. Preliminary deformation analyses were performed to address the potential for large deformations leading to possible dam failure.

<u>Design Earthquake</u>:

The design earthquakes for this analysis were developed using a direct seismic source. The estimated MCE uses a M7.4 floating earthquake with a ground acceleration of 0.23g. The estimated OBE is a M6.2 with ground acceleration of 0.10g. For the purpose of the study the MDE (Maximum Design Earthquake) was assumed to be the same as the MCE. If the dam structures are not high hazard then the MDE can be a fraction of the MCE. An updated Dam Break Analysis would need to be performed to determine the hazard classification of the dams, since this has not been done it is assumed they are Class I, High Hazard structures.

<u>Liquefaction Analysis:</u>

The liquefaction analysis of the Upper Dam for the MCE event indicates that significant areas of the silty sand center of the dam and the silty foundation materials are susceptible to liquefaction. The crib dam thru the center of the embankment contains loose silty sands in a layer at least 20 feet thick. Subsurface explorations indicate that the foundation materials on the downstream side of the dam consist of similar loose granular material and it is assumed that this same material is on the upstream side of the dam. The results for the OBE analysis indicate that the dam and foundation materials have a factor of safety well above 1.0 against liquefaction.

The liquefaction analysis for the Lower Dam for the MCE event indicates that the majority of the dam will not liquefy during this event. One of the SPT values had blow counts low enough to have a factor of safety against liquefaction less than 1.0. This layer was assumed to have a thickness of 5 feet and was treated as having residual strengths during the stability analysis. The liquefaction analysis during the OBE event indicates factors of safety significantly higher than 1.0.

<u>Post Earthquake Stability:</u>

The stability analyses using residual and reduced strengths for both dams and the OBE and MCE event indicates that both dams have the potential for failure under the MCE event on the downstream side of the dam. A factor of safety of 0.8 for the Upper Dam indicates failure would be likely given that the soils liquefy. A factor of safety of 1.09 for the Lower Dam would indicate that failure is also likely although the extent of deformation may be significantly less and only consist of cracking of the embankment with no loss of pool.

Loss of pool in the Upper Dam during the MCE is likely. The ability of the Lower Dam to pass this flow and not fail during this event was not in the scope of this study but is questionable and needs to be addressed during a Dam Break Analysis.

<u>Deformation Potential:</u>

A preliminary and empirical based deformation study was performed on both dams. The study does not differentiate between earthquake magnitudes but uses blow counts and overburden pressure relationships to address the potential for deformations greater than 3 feet. This preliminary analysis indicates that the Upper Dam has significant potential for greater than 3 feet of deformation. The Lower Dam has the potential for large deformations but it appears this would only occur in localized areas.

This type of deformation analysis does not differentiate between the OBE and the MCE events. Both the Upper and Lower Dams appear to be able to withstand the OBE event. A Newmark Analysis could provide additional information on potential deformations in both dams as well as potential in the Lower Dam during the MCE. However, a Newmark Analysis is not valid for liquefiable materials and not intended for silty sands. Additionally, it was felt that the MCE warrants a more rigorous analysis, such as FLAC, and therefore a Newmark Analysis was not performed.

8.2 Recommendations

Without further study of both dams and a dam break analysis it can not be said that the existing structures meet the seismic requirements of the State of Alaska. It is recommend that the City of Wrangell proceed on with an investigation into water supply alternatives to include remediation of the dams, construction of a new structure and alternative water supplies.

9 References

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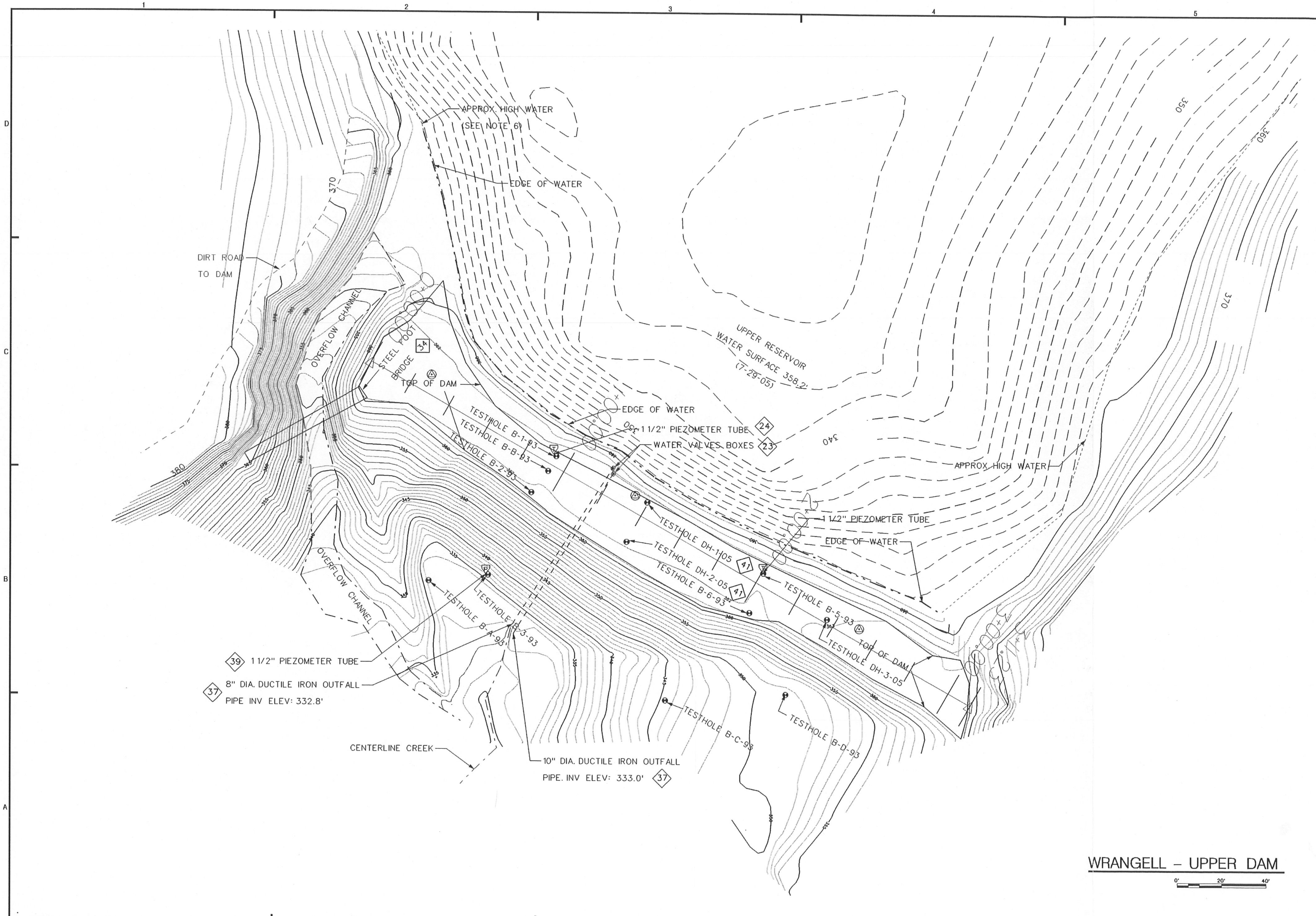
PLATES



WRANGELL - UPPER DAM

0' 20' 40'

US Army Corps
of Engineers
Walla Walla District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WALLA WALLA, WASHINGTON

DATE: DY MON YR
SOLICITATION NO. XX-B-XXXX

WRANGELL DAM
WRANGELL, ALASKA
SEISMIC STABILITY ANALYSIS

Sheet number:
PLATE 1

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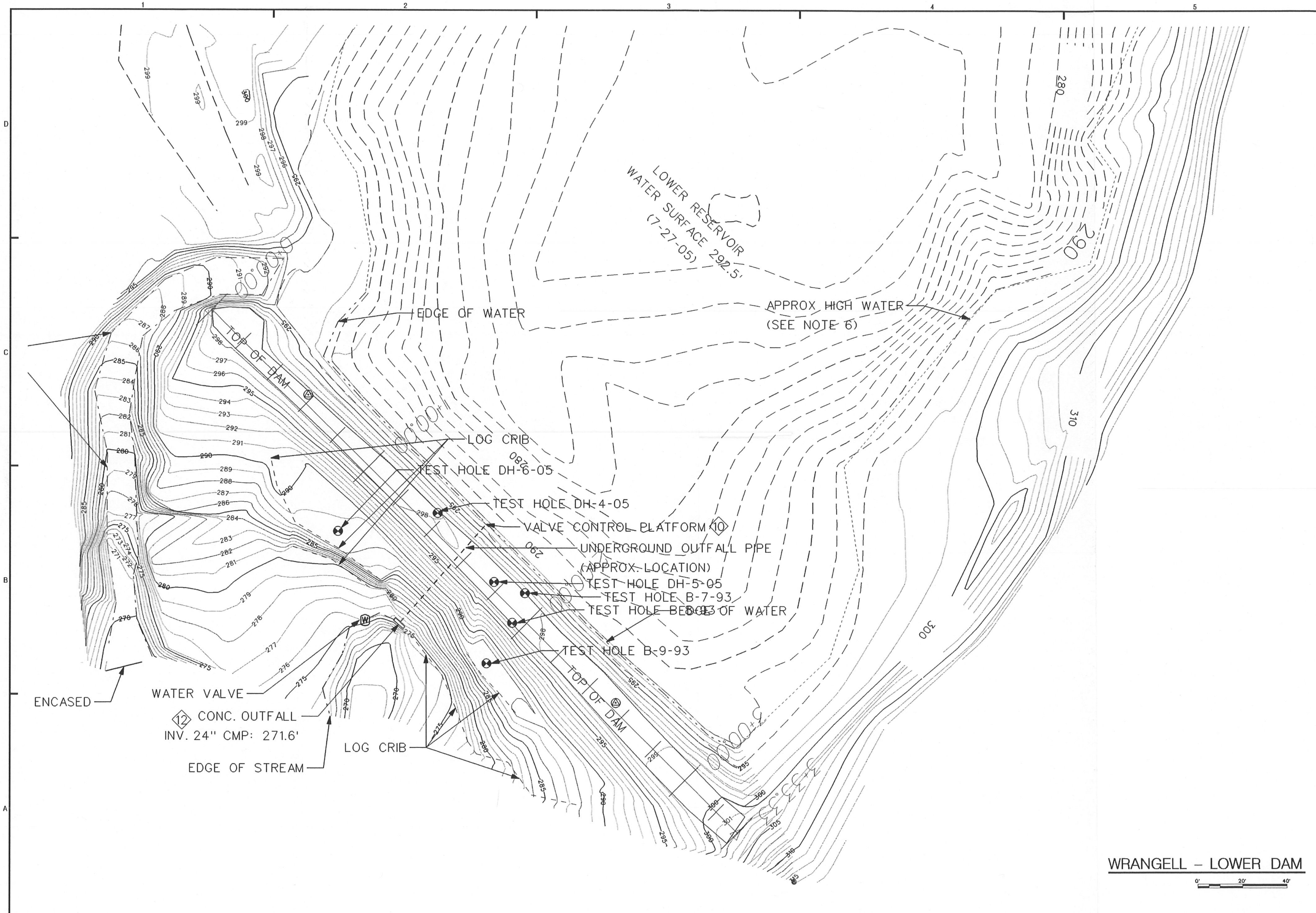

US Army Corps of Engineers
Walla Walla District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WALLA WALLA, WASHINGTON

SOLICITATION NO.: XX-B-XXXX

WRANGELL DAM
WRANGELL, ALASKA
SEISMIC STABILITY ANALYSIS

Sheet number:
PLATE 2

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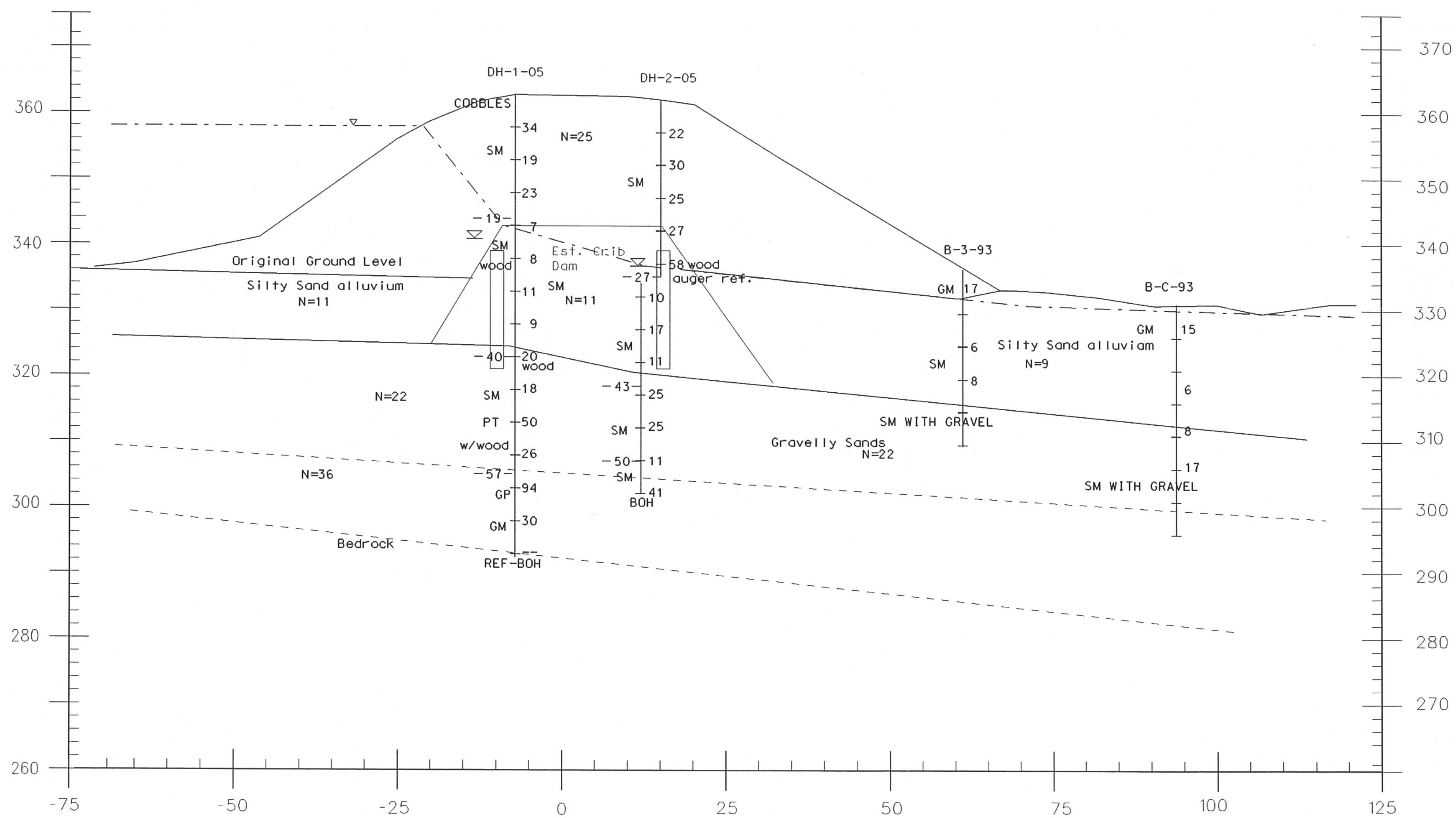

UPPER DAM - STA 1+20

US Army Corps of Engineers
Walla Walla District

DESIGNED BY: | DWN BY: | CHKD BY: | SUBMITTED BY:

DATE: DY MON YR
SOLICITATION NO.: XX-B-XXXX
CONTRACT NO.:
FILE NO.:

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WALLA WALLA, WASHINGTON

WRANGELL DAM
WRANGELL, ALASKA
SEISMIC STABILITY ANALYSIS
UPPER DAM STA 1+20

Sheet number:
PLATE 3

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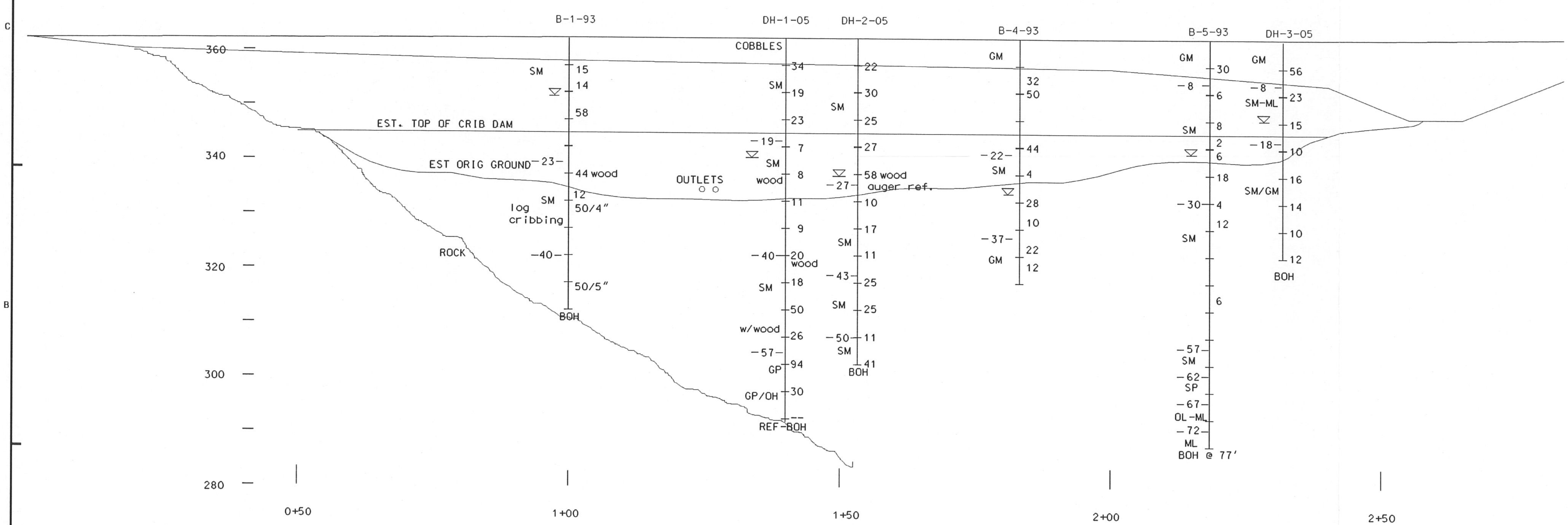

PROFILE - UPPER DAM

0' 10' 20'

US Army Corps of Engineers
Walla Walla District

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WALLA WALLA, WASHINGTON

DESIGNED BY: | DWN BY: | CHKD BY: | SUBMITTED BY: | DATE: DY MON YR | SOLICITATION NO. XX-B-XXXX | CONTRACT NO. | FILE NO.:

WRANGELL DAM
WRANGELL, ALASKA
SEISMIC STABILITY ANALYSIS
UPPER DAM PROFILE

Sheet number:
PLATE 4

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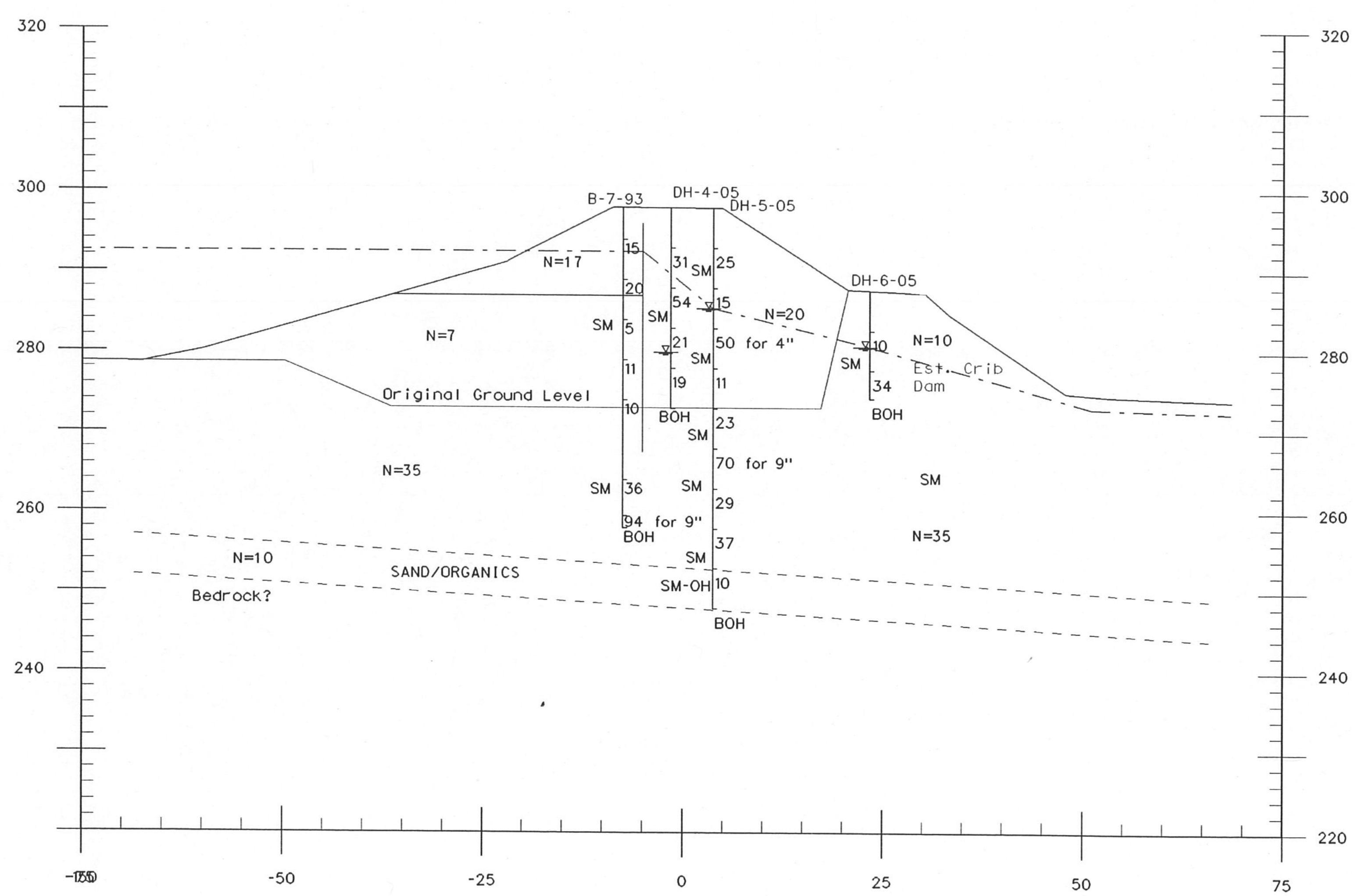


LOWER DAM – STA 2+00

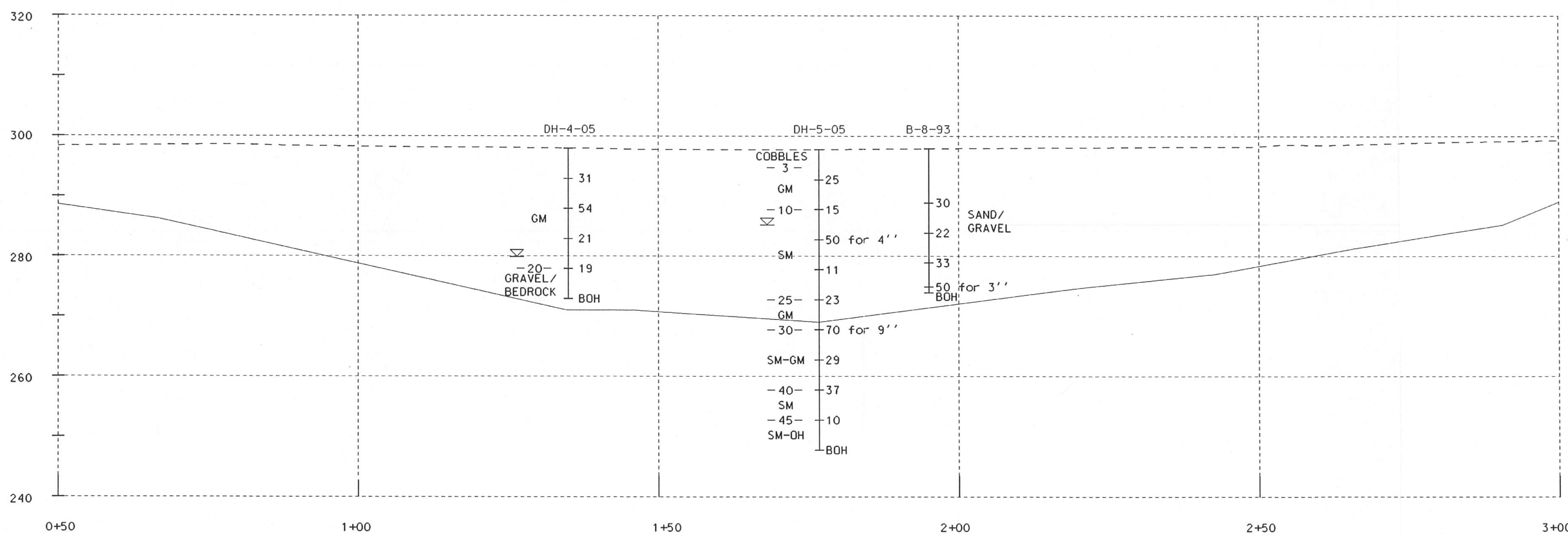
US Army Corps of Engineers
Walla Walla District

DATE: DY MON YR
SOLICITATION NO. XX-B-XXXX
CONTRACT NO.
DESIGNED BY: J SPEER
DWN BY: J SPEER
CHKED BY: Y GIBBONS
SUBMITTED BY:
FILE NO.:

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WALLA WALLA, WASHINGTON

WRANGELL DAM
WRANGELL, ALASKA
SEISMIC STABILITY ANALYSIS
LOWER WRANGELL DAM STA 2+00

Sheet number:
PLATE 5



PROFILE - LOWER DAM

0' 10' 20'

US Army Corps of Engineers
Walla Walla District

DATE: DY MON YR
SOLICITATION NO.: XX-B-XXXX
CONTRACT NO.:
DESIGNED BY:
DWN BY:
CHKD BY:
SUBMITTED BY:
FILE NO.:

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WALLA WALLA, WASHINGTON

WRANGELL DAM
WRANGELL, ALASKA
SEISMIC STABILITY ANALYSIS
LOWER DAM PROFILE

Sheet number:
PLATE 6

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Appendix A

Logs of Borings

and

Test Data

HOLE NO. DH-I-05

DRILLING LOG

WALLA WALLA DISTRICT

SHEET 1 OF 2 SHEETS

1. PROJECT WRANGELL UPPER DAM	10. CONTRACT NO. 05-Q-0093
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN
3. DRILLING CONTRACTOR DENALLI DRILLING, ANCHORAGE, AK	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 13; UNDISTURBED
4. HOLE NO. (As shown on drawing title and file number) DH-I-05	13. SAMPLER SIZE 2" SPT; 14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8/8/2005; COMPLETED 8/11/2005
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE 362
8. CATHEAD WRAPS/ENERGY: 2 WRAPS/ MANUAL	18. TOTAL DEPTH OF HOLE 68 FT
9. OTHER:	19. LOGGED BY: Y.GIBBONS

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
	5		(SM) SILTY SAND w/ sm GRAVEL, 50% sand, 33% angular gravels, 17% silt, med dense, moist, dk grey 10 % in place moisture	S-1	9-14-20-18	13"	
	10		(SM) SILTY SAND, w sm GRAVEL, same as above, med dense, wet, dk grey	S-2	7-10-9	4"	v. ltl recovery may be due to loss thru screen, new one added
	15		(SM) SILTY SAND, with GRAVEL med dense, wet, dk grey	S-3	10-5-11-12	6"	
	20	▽	(SM) SILTY SAND, ltl to no gravels, 47% med to fn sand, 50% silt, loose, wet, dk brown/grey. 53% in place moisture.	S-4	1-2-5	14"	
BOP	25		same as above bottom 2"-3" wood, some upheave	S-5	3-1-2-6	12"	BOP - Bottom of piezometer
	30		(SM) SILTY SAND, with GRAVEL, 20% silt, coarse to med sand, loose to med, sat., dk grey, 26% moisture	S-6	10-7-7-4	12"	6" upheave in sampler
	35		(SM) SILTY SAND, loose to med, sat, dk grey, sm organics	S-7	2-3-6	12"	

HOLE NO. DH-1-05

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 2 OF 2 SHEETS
1. PROJECT WRANGELL UPPER DAM	10. CONTRACT NO. 05-Q-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR Denalli Drilling, Anchorage, AK	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 13; UNDISTURBED	
4. HOLE NO. (As shown on drawing title and file number) DH-1-05	13. SAMPLER SIZE 2" SPT; 14. HAMMER WT. 140 LBS	
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8/8/2005; COMPLETED 8/11/2005	
7. HOLE SIZE AND TYPE: 6' HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE 362	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS/ MANUAL	18. TOTAL DEPTH OF HOLE 68 ft	
9. OTHER:	19. LOGGED BY: Y.GIBBONS	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
	40		(SM) SILTY SAND, with Gravel, med dense, wet, dk grey, some peat, wood pieces	S-8	5-11-9	14"	Lab data indicated 10% organic
	45		(SM) SILTY SAND and GRAVEL, med dense, sat., dk grey	S-9	11-11-7	8"	weathered bedrock?
	50		(pt) PEAT, with silty sand, dense, moist, dk grey; 4 % fines, 50% moisture content	S-10	8-7-50/5"	8"	Wood pieces in sample
	55		(SP) SAND with GRAVEL, course to fine sand and anguair gravel, med-dense, sat, dk grey	S-11	6-11-15	8"	~ 6" v.fine sand @ top of sample likely upheave in bottom of hole
	60		(GP) Weather bedrock, GRAVEL, sm silts, sands, dense, wet, dk grey	S-12	16-55-39	6"	
	65		(GM) Silty Gravel with Sand and PEAT, upper half of sample, peat; lower half apprx. 50% angular gravel, 30% sand, 20% fines, 26% moisture content; lower half peat, brn & grey, some wood	S-13	6-15-15	12"	
			BOH @ 68' - Rig busted drive shaft				

ENG FORM 1836 (REV NWW)
OCT 2005

PROJECT	HOLE NO.
WRANGELL UPPER DAM	DH-1-05

HOLE NO. DH-2-05

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 1 OF 1 SHEETS
1. PROJECT WRANGELL UPPER DAM	10. CONTRACT NO. 05-Q-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR DENALLI DRILLING, ANCHORAGE, AK	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 5; UNDISTURBED	
4. HOLE NO. (As shown on drawing title and file number) DH-2-05	13. SAMPLER SIZE 2" SPT; 14. HAMMER WT. 140 LBS	
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8/11/2005; COMPLETED 8/11/2005	
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE 362	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS/ MANUAL	18. TOTAL DEPTH OF HOLE 27 FT	
9. OTHER:	19. LOGGED BY: Y.GIBBONS	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
	5		(SM) SILTY SAND w/ GRAVEL, med dense, wet, lt grey	S-1	8-8-14	8"	Auger fast & easy
	10		(SM) SILTY SAND, w/Gravel, wet	S-2	48-18-12	8"	probably large frag. in sample in beginning
	15		(SM) SILTY SAND, with gravel. About 35% angular 3/4" minus gravel, 42% well graded sand and 23% non-plastic fines, dense, wet, dark grey. 12% moisture content	S-3	5-5-20	7"	fast easy auger in first segs
	20		(SM) SILTY SAND, with some gravel, med-dense to dense, wet dark grey.	S-4	27-59/ref	6"	refusal 2nd 6" - probably large fragment
BOP	25	▽	(SM) SILTY SAND, with gravel. About 20% 1/2" minus gravel, soft to dense, wet, dk grey.	S-5	2-4-54	7"	BOP - Bottom of piezometer
			BOH at 27 feet - Hit wood, refusal. Wood verified with drive sample				Moved hole over about 5 feet and augered to 25 feet New hole no. 2A
	30						

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 1 OF 1 SHEETS
1. PROJECT WRANGELL UPPER DAM	10. CONTRACT NO. 05-0-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR Denalli Drilling, Anchorage, AK	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 7; UNDISTURBED	
4. HOLE NO. (As shown on drawing title and file number) DH-2A-05	13. SAMPLER SIZE 2" SPT	14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8/11/2005; COMPLETED 8/11/2005	
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE 362	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS/ MANUAL	18. TOTAL DEPTH OF HOLE 60 ft	
9. OTHER:	19. LOGGED BY: Rich Young	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
			Drilled w/o sampling, same as DH-2-05. Mostly ML& SM, fine SANDY SILT w/GRAVEL. Material slighty more course below 20 ft, may be GM. All moist, dk grey and soft to med-dense. varies vertically.				Note: Hole DH-2-05 hit ref. at 25 feet, moved ~ 5 ft, augered to 30 ft and continued SPT's.
	30		(SM) SILTY SAND, with Gravel, loose to med, mst, dk grey	S-1	5-3-7	3"	easy drive
	35		(SM) SILTY SAND w/ GRAVEL. About 33% 1/2" minus, angular gravel, 42% sand, 17% non-plastic fines, loose to med, wet., dk grey	S-2	7-8-9	18"	very easy auger
	40		same	S-3	11-5-6	6"	extrememly fast and easy auger
	45		(SM) SILTY SAND, with coarse 2" minus GRAVEL, med dense, wet	S-4	12-13-56		refusal - no sample recovered
	50		(SM) SILTY SAND with GRAVEL, 41% fine gravel, 42% medium sand; 17% non-plastic fines; loose to med, in place moisture 16%.	S-5	7-15-10	8"	
	55		(SM) SILTY SAND with GRAVEL same as above	S-6	6-15-15	12"	fast easy auger
			BOH @ 60' - No recovery in sample				

HOLE NO. DH-3-05

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 1 OF 2 SHEETS
1. PROJECT WRANGELL UPPER DAM	10. CONTRACT NO. 05-Q-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR Denalli Drilling, Anchorage, AK	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 8; UNDISTURBED	
4. HOLE NO. (As shown on drawing title and file number) DH-3-05	13. SAMPLER SIZE 2" SPT; 14. HAMMER WT. 140 LBS	
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8/13/2005; COMPLETED 8/13/2005	
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE 363	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS/ MANUAL	18. TOTAL DEPTH OF HOLE 41 ft	
9. OTHER:	19. LOGGED BY: Rich Young	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
			Coarse GM for 1.5', with frags, cobbles, moist, med brn				
	5		(SM) SILTY SAND, with some Gravel, loose to med, mst, rounded frags to 1" dia, grey	S-1	2-16-40	12"	
	10		(SM) SILTY SAND w/ gravel. 20% fine Gravel; 62% coarse to fine sand; 18% non-plastic fines, 14% in place moisture content, dk grey; 14% moisture	S-2	8-5-18	8"	very easy auger
	15	▽	(SM) SILTY SAND fine sand and silt, slight plasticity, soft, loose, v. wet	S-3	3-4-11	9"	v. rapid, easy auger
	20		(SM) SILTY SAND with fine GRAVEL, loose, wet, shale frags	S-4	1-6-4	8"	fast easy auger
BOP	25		(SM) SILTY SAND w/ GRAVEL, mostly fine to med sand, little coarse to fine gravel and non-plastic fines, loose, wet, dk grey	S-5	9-4-12	8"	BOP - Bottom of piezometer
	30		(SM) SILTY SAND w/ gravel, 22% coarse to fine angular Gravel; 50% well graded sand; 28% fines, 28% in place moisture.	S-6	11-5-9	18	fast easy auger

HOLE NO. DH-3-05

DRILLING LOG

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 2 OF 2 SHEETS

1. PROJECT WRANGELL UPPER DAM	10. CONTRACT NO 05-0-0093
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN
3. DRILLING CONTRACTOR Denalli Drilling, Anchorage, AK	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 8; UNDISTURBED
4. HOLE NO. (As shown on drawing title and file number) DH-3-05	13. SAMPLER SIZE 2" SPT; 14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8/13/2005; COMPLETED 8/13/2005
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE 363
8. CATHEAD WRAPS/ENERGY: 2 WRAPS/ MANUAL	18. TOTAL DEPTH OF HOLE 41 ft
9. OTHER:	19. LOGGED BY: Rich Young

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
	35		(SM) SILTY SAND with GRAVEL, wet, loose.	S-7	7-3-7		
	40		(SM) SILTY SAND with GRAVEL, 19% fines, wet, loose: 25% moisture	S-8	10-7-5		
			BOH @ 40'				
	45						
	50						

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 1 OF 1 SHEETS
1. PROJECT LOWER WRANGELL DAM - WRANGELL, AK	10. CONTRACT NO. 05-Q-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR DENALI DRILLING	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 4	UNDISTURBED
4. HOLE NO. (As shown on drawing title and file number) DH-4-05	13. SAMPLER SIZE 2" SPT	14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8-14-05	COMPLETED 8-14-05
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS MANUAL	18. TOTAL DEPTH OF HOLE 25.5 FT	
9. OTHER:	19. LOGGED BY: R.YOUNG	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
			0' TO 5': GM- silty sandy and sandy silty GRAVEL; med. dense, moist, gray, rounded frags to 2" dia.				
	5		(SM) Silty Sand with Gravel, moist, med. dense, angular frags to 2" dia.	S-1	13-18-13	5	pene @ steady rate
	10		(SM) Silty Sand, with Gravel. 41% fine to coarse Sand; 34% coarse, angular gravel; 25% fines, 10% in place moisture. Water @ 13'	S-2	7-15-39	8	rock frag last 6", broke through last 5 blows
	15		(SM) silty sand with Gravel, med dense to loose, moist, gray, frags to 2-3" dia.	S-3	11-12-9	6	pene rate steady
		▽					
BOP	20		(SM) Silty Sand with Gravel. 64% coarse to fine Sand; 17% 3/4" minus Gravel; 19% non-plastic fines; 15% in place moisture, some angular fragments/bedrock	S-4	8-10-9	5	pene rate steady & smooth BOP - Bottom of piezometer
	25		BOH @ 25' REFUSAL; bogus sample material from slough		50-0-0	0	refusal first 6"

HOLE NO. DH-5-05

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 1 OF 2 SHEETS
1. PROJECT LOWER WRANGELL DAM - WRANGELL, AK	10. CONTRACT NO. 05-Q-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR DENALI DRILLING	12. TOTAL NO. OF SAMPLES TAKEN — DISTURBED 9	UNDISTURBED
4. HOLE NO. (As shown on drawing title and file number) DH-5-05	13. SAMPLER SIZE 2" SPT	14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE — STARTED 8-16-05	COMPLETED 8-16-05
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS MANUAL	18. TOTAL DEPTH OF HOLE 50 FT	
9. OTHER:	19. LOGGED BY: Y.GIBBONS	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
			(SM) Silty Sand with Gravel; loose, dk brown, moist, COBBLE and angular rock fragments in upper 3'				
	5		(SM)- silty SAND with coarse Gravel, dense, moist, dk grey	S-1	7-6-19	8"-12"	
	10		(SM) SILTY SAND, with GRAVEL; 28% angular gravel; 50% fine to coarse sand; 22% fines; 15% moisture content; med dense, moist, dk grey	S-2	8-7-8	8"-9"	
		▽					
	15		SM- silty SAND, with some GRAVEL, sampler hit cobble, low recovery	S-3	50 for 4"	4"	
	20		SM- SILTY SAND w/ sm GRAVEL, loose-med, wet dk grey	S-4	9-6-5		
BOP	25		(SM) Silty Sand with Gravel; med dense, sat, dk grey, upper 6-8" was coarse dk grey SAND (heave?)	S-5	6-8-15	10"-12"	BOP - Bottom of piezometer
	30		(SM) silty SAND with GRAVELS, 37% fine angular Gravel; 47% Sand; 16% slightly plastic fines, dense, moist, dk grey; 13% moisture content.	S-6	24-20-50 for 3"	18"	
	35						hard drilling @ 33'

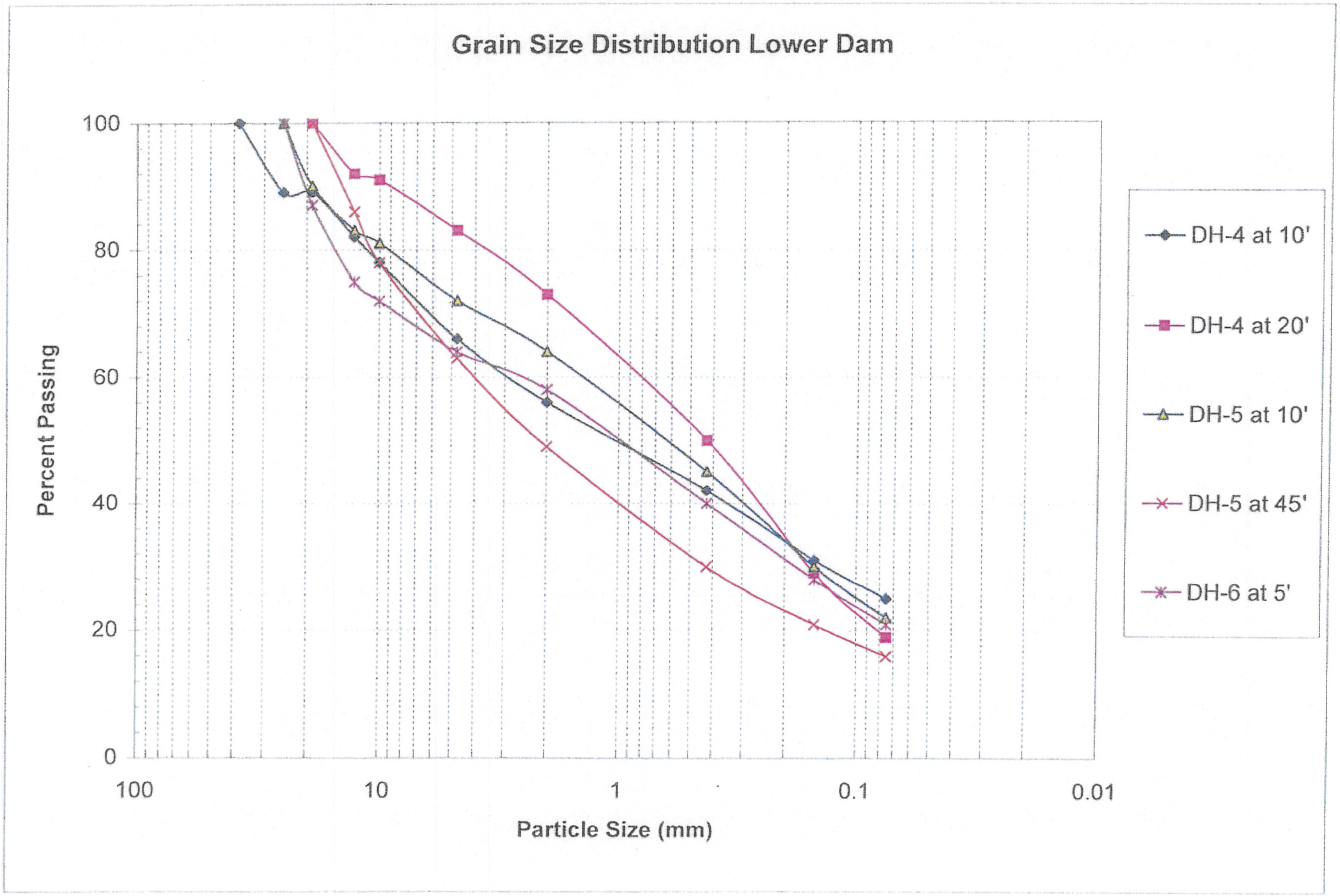
DRILLING LOG

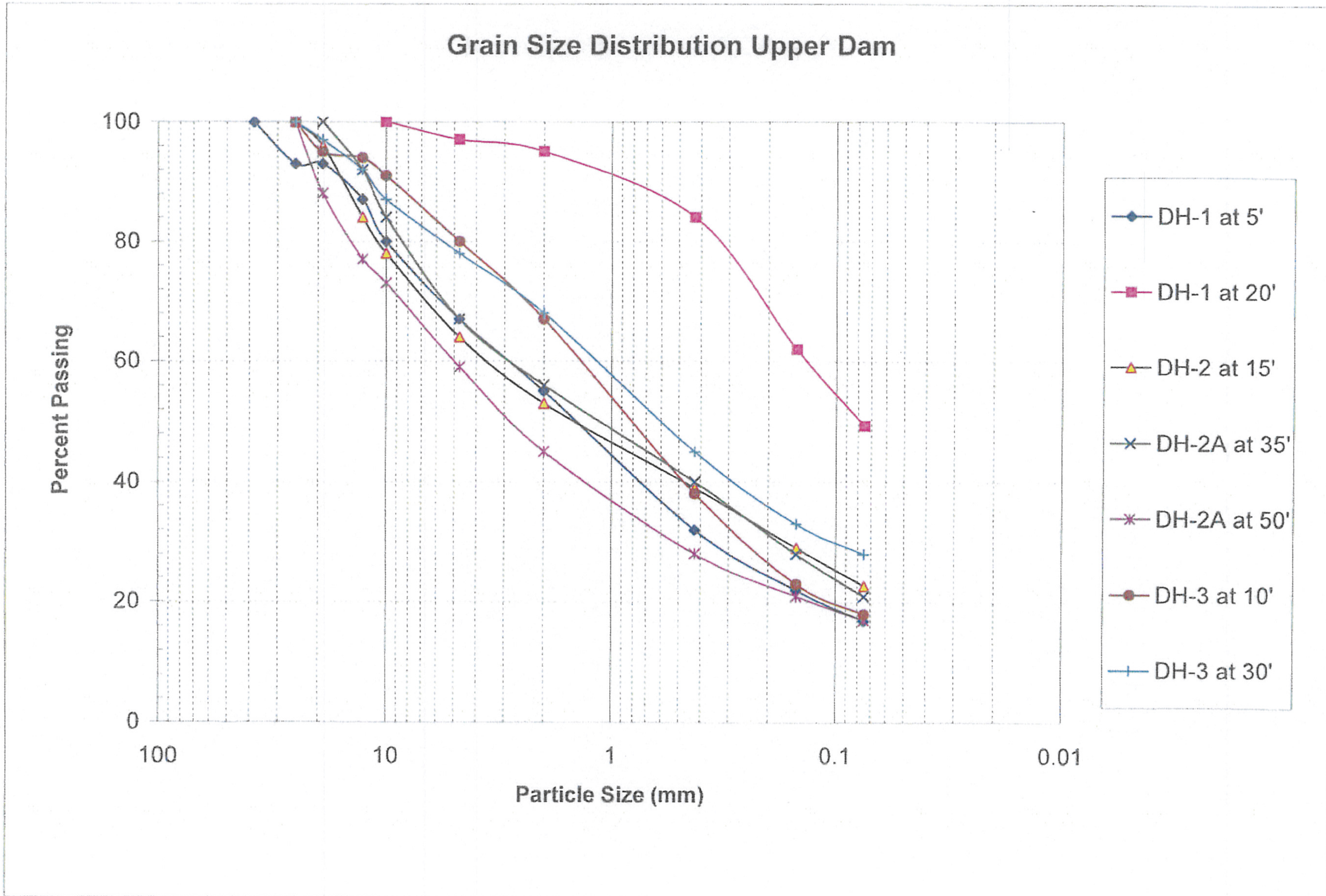
DRILLING LOG	WALLA WALLA DISTRICT	SHEET 2 OF 2 SHEETS
1. PROJECT LOWER WRANGELL DAM - WRANGELL, AK	10. CONTRACT NO. 05-Q-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR DENALI DRILLING	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 9	UNDISTURBED
4. HOLE NO. (As shown on drawing title and file number) DH-5-05	13. SAMPLER SIZE 2" SPT	14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8-16-05	COMPLETED 8-16-05
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS MANUAL	18. TOTAL DEPTH OF HOLE 50 FT	
9. OTHER: HOLE LOCATED 27.5' NORTH FROM MON #3	19. LOGGED BY: Y.GIBBONS	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
	35		(SM) Silty Sand with Gravel same as above	S-7	7-18-11		
	40		(SM) SILTY SAND with GRAVEL, less silts, med dense, sat, dk grey	S-8	13-15-22	18"	half was heave
	45		(SM) silty SAND and PEAT, 16% Organics; 99% moisture content, wood, med, wet, grey brown	S-9	5-5-5		
	50		hit REFUSAL - BOH @ 50'				

DRILLING LOG	WALLA WALLA DISTRICT	SHEET 1 OF 1 SHEETS
1. PROJECT LOWER WRANGELL DAM - WRANGELL, AK	10. CONTRACT NO. 05-0-0093	
2. LOCATION (Coordinates or Station)	11. DATUM FOR ELEVATION SHOWN	
3. DRILLING CONTRACTOR DENALI DRILLING	12. TOTAL NO. OF SAMPLES TAKEN: DISTURBED 2	UNDISTURBED
4. HOLE NO. (As shown on drawing title and file number) DH-6-05	13. SAMPLER SIZE 2" SPT	14. HAMMER WT. 140 LBS
5. NAME OF DRILLER MIKE STOCKTON	15. ELEVATION GROUND WATER	
6. DRILL RIG MOBILE B-61	16. DATE HOLE: STARTED 8-14-05	COMPLETED 8-14-05
7. HOLE SIZE AND TYPE: 6" HOLLOW STEM AUGER	17. ELEVATION TOP OF HOLE	
8. CATHEAD WRAPS/ENERGY: 2 WRAPS MANUAL	18. TOTAL DEPTH OF HOLE 13.5 FT	
9. OTHER: HOLE LOCATED 27.5' NORTH FROM MON #3	19. LOGGED BY: R.YOUNG	

ELEV. a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	SAMPLE TYPE-NO. e	BLOWS PER 6" f	RECOVERY (in) g	REMARKS (Drilling time, water loss, advance of boring, mechanical problems, etc., if significant)
			0' TO 5': SM-silty fine SAND w/ GRAVEL, loose dk brn, moist angular rock fragments. some boulder sized fragments on or near surface				
	5		SM: silty fine SAND w/ GRAVEL, wet. Water @ 7'	S-1	4-4-6		very rapid, easy pene
		▽					
	10		(SM) silty SAND w/ GRAVEL; 28% angular fine gravel; 47% fine to coarse Sand; 21% fines; loose, wet, dk grey.	S-2	1-12-22		rapid pene 1st 1/2
BOP							BOP - Bottom of piezometer
			BEDROCK - 13.5'				BOH
	15						


Grain Size Distribution Lower Dam
Percent Passing
100
80
60
40
20
0
100
10
1
0.1
0.01
Particle Size (mm)
DH-4 at 10'
DH-4 at 20'
DH-5 at 10'
DH-5 at 45'
DH-6 at 5'



Grain Size Distribution Upper Dam
Percent Passing
100
80
60
40
20
0
100
10
1
0.1
0.01
Particle Size (mm)
DH-1 at 5'
DH-1 at 20'
DH-2 at 15'
DH-2A at 35'
DH-2A at 50'
DH-3 at 10'
DH-3 at 30'

Appendix B

Liquefaction Analysis

Factor of Safety Against Liquefaction - OBE
Upper Wrangell Dam

DH-1-05 a_{max} / g = 0.1

DH-1-05	N_m	USCS	% fines	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft	σ_o' , lb/ft	CSR
S-1	34	SM-GM	*16*	125	5	1.52	0.988	625	625	0.06
S-2	19	"	16	130	10	3.05	0.977	1275	963	0.08
S-3	23	"	20	130	15	4.57	0.965	1925	1301	0.09
S-4	7	SM-ML	*50*	115	20	6.10	0.953	2500	1564	0.1
S-5	8	"	25	115	25	7.62	0.942	3075	1827	0.1
S-6	11	"	25	115	30	9.14	0.930	3650	2090	0.11
S-7	9	"	20	115	35	10.67	0.889	4225	2353	0.1
S-8	20	SM-OL	20	120	40	12.19	0.848	4825	2641	0.1
S-9	18	GM	15	125	45	13.72	0.808	5450	2954	0.1
S-10	50	OL-SP	*4*	125	50	15.24	0.767	6075	3267	0.09
S-11	26	SG	15	130	55	16.76	0.726	6725	3605	0.09
S-12	94	GP	15	130	60	18.29	0.686	7375	3943	0.08
S-13	30	GM-OL	*21*	130	65	19.81	0.645	8025	4281	0.08
							Eqn 2a or 2b			Eqn (1)

DH-1-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.06	N/A	6.2	1.63	N/A
10	0.08	N/A	6.2	1.63	N/A
15	0.09	N/A	6.2	1.63	N/A
20	0.1	0.18	6.2	1.63	2.93
25	0.1	0.14	6.2	1.63	2.28
30	0.11	0.16	6.2	1.63	2.37
35	0.1	0.14	6.2	1.63	2.28
40	0.1	0.27	6.2	1.63	4.39
45	0.1	0.19	6.2	1.63	3.09
50	0.09	N/A	6.2	1.63	N/A
55	0.09	0.25	6.2	1.63	4.52
60	0.08	N/A	6.2	1.63	N/A
65	0.08	0.30	6.2	1.63	6.10
				Eqn (24)	Eqn (23)

Determination of CRR (Cyclic Resistance Ratio)

DH-1-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	34	1.52	1.4712	0.75	1.1	41	*16*	2.767	1.054	46	0.256	0.31	1.42	1	N/A
S-2	19	3.05	1.3293	1	1.1	28	16	2.767	1.054	32	0.745	0.48	1.25	1	N/A
S-3	23	4.57	1.2123	1	1.1	31	20	3.615	1.079	37	-0.100	0.65	1.14	1	N/A
S-4	7	6.10	1.1346	1	1.1	9	*50*	5.000	1.200	15	0.165	0.78	1.08	1	0.18
S-5	8	7.62	1.0662	1	1.1	9	15	2.498	1.048	12	0.134	0.91	1.03	1	0.14
S-6	11	9.14	1.0057	1	1.1	12	15	2.498	1.048	15	0.163	1.05	0.99	1	0.16
S-7	9	10.67	0.9516	1	1.1	9	20	3.615	1.079	14	0.148	1.18	0.95	1	0.14
S-8	20	12.19	0.8987	1	1.1	20	20	3.615	1.079	25	0.291	1.32	0.92	1	0.27
S-9	18	13.72	0.8475	1	1.1	17	15	2.498	1.048	20	0.216	1.48	0.89	1	0.19
S-10	50	15.24	0.8018	1	1.1	44	*4*	0.000	1.000	44	0.223	1.63	0.86	1	N/A
S-11	26	16.76	0.7577	1	1.1	22	15	2.498	1.048	25	0.296	1.80	0.84	1	0.25
S-12	94	18.29	0.7182	1	1.1	74	15	2.498	1.048	80	0.569	1.97	0.82	1	N/A
S-13	30	19.81	0.6826	1	1.1	23	*21*	3.778	1.086	28	0.378	2.14	0.80	1	0.30
						Eqn (8)	lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/P_a)$

P_a = 2116.2 psf

Factor of Safety Against Liquefaction
Upper Wrangell Dam

DH-2-05

$a_{max} / g =$ 0.1

DH-1-05	N_m	USCS	% fines	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o, lb/ft^2	σ_o', lb/ft^2	CSR
S-1	22	SM-GM	15	125	5	1.52	0.988	625	625	0.06
S-2	30	"	15	130	10	3.05	0.977	1275	963	0.08
S-3	25	SM-GM	23	130	15	4.57	0.965	1925	1301	0.09
S-4	27	"	25	130	20	6.10	0.953	2575	1639	0.1
S-5	58	"	25	115	25	7.62	0.942	3150	1902	0.1
S-6	10	SM	25	115	30	9.14	0.930	3725	2165	0.1
S-7	17	SM	21	115	35	10.67	0.889	4300	2428	0.1
S-8	11	"	20	120	40	12.19	0.848	4900	2716	0.1
S-9	25	SM-GM	15	125	45	13.72	0.808	5525	3029	0.1
S-10	25	SM-GM	17	125	50	15.24	0.767	6150	3342	0.09
S-11	11	"	15	130	55	16.76	0.726	6800	3680	0.09
S-12	41	"	15	130	60	18.29	0.686	7450	4018	0.08
							Eqn 2a or 2b			Eqn (1)

DH-2-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.06	N/A	6.2	1.63	N/A
10	0.08	N/A	6.2	1.63	N/A
15	0.09	N/A	6.2	1.63	N/A
20	0.1	N/A	6.2	1.63	N/A
25	0.1	N/A	6.2	1.63	N/A
30	0.1	0.17	6.2	1.63	2.77
35	0.1	0.24	6.2	1.63	3.91
40	0.1	0.15	6.2	1.63	2.44
45	0.1	0.29	6.2	1.63	4.72
50	0.09	0.27	6.2	1.63	4.88
55	0.09	0.11	6.2	1.63	1.99
60	0.08	N/A	6.2	1.63	N/A
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-2-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o', tsf	K_σ	K_α	CRR
S-1	22	1.52	1.4712	0.75	1.1	27	15	2.498	1.048	30	0.506	0.31	1.42	1	N/A
S-2	30	3.05	1.3293	1	1.1	44	15	2.498	1.048	48	0.285	0.48	1.25	1	N/A
S-3	25	4.57	1.2123	1	1.1	33	23	4.059	1.100	41	0.149	0.65	1.14	1	N/A
S-4	27	6.10	1.1142	1	1.1	33	25	4.289	1.115	41	0.161	0.82	1.06	1	N/A
S-5	58	7.62	1.0482	1	1.1	67	25	4.289	1.115	79	0.557	0.95	1.02	1	N/A
S-6	10	9.14	0.9896	1	1.1	11	25	4.289	1.115	16	0.175	1.08	0.98	1	0.17
S-7	17	10.67	0.9372	1	1.1	18	21	3.778	1.086	23	0.254	1.21	0.94	1	0.24
S-8	11	12.19	0.8859	1	1.1	11	20	3.615	1.079	15	0.162	1.36	0.91	1	0.15
S-9	25	13.72	0.8361	1	1.1	23	15	2.498	1.048	27	0.328	1.51	0.88	1	0.29
S-10	25	15.24	0.7916	1	1.1	22	17	3.012	1.060	26	0.315	1.67	0.86	1	0.27
S-11	11	16.76	0.7486	1	1.1	9	15	2.498	1.048	12	0.131	1.84	0.83	1	0.11
S-12	41	18.29	0.71	1	1.1	32	15	2.498	1.048	36	-0.223	2.01	0.81	1	N/A
S-13															
			C_R = Correction for rod length											flat	
		$C_N = 2.2/(1.2 + \sigma_o'/P_a)$					lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	ground	

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - OBE
Upper Wrangell Dam

DH-3-05 $a_{max} / g =$ 0.1

DH-1-05	N_m	USCS	% fines	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft	σ_o' , lb/ft	CSR
S-1	56	SM-GM	15	125	5	1.52	0.988	625	625	0.06
S-2	23	SM-GM	15	130	10	3.05	0.977	1275	963	0.08
S-3	15	SM	18	115	15	4.57	0.965	1850	1226	0.09
S-4	10	"	20	115	20	6.10	0.953	2425	1489	0.1
S-5	16	"	25	115	25	7.62	0.942	3000	1752	0.1
S-6	14	SM	28	115	30	9.14	0.930	3575	2015	0.11
S-7	10	"	20	115	35	10.67	0.889	4150	2278	0.11
S-8	12	"	20	115	40	12.19	0.848	4725	2541	0.1
							Eqn 2a or 2b			Eqn (1)

DH-3-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.06	N/A	6.2	1.63	N/A
10	0.08	N/A	6.2	1.63	N/A
15	0.09	0.34	6.2	1.63	6.15
20	0.1	0.18	6.2	1.63	2.93
25	0.1	0.26	6.2	1.63	4.23
30	0.11	0.25	6.2	1.63	3.70
35	0.11	0.15	6.2	1.63	2.22
40	0.1	0.17	6.2	1.63	2.77
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-3-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	56	1.52	1.4712	0.75	1.1	68	15	2.498	1.048	74	0.516	0.31	1.42	1	N/A
S-2	23	3.05	1.3293	1	1.1	34	15	2.498	1.048	38	0.008	0.48	1.25	1	N/A
S-3	15	4.57	1.2364	1	1.1	20	18	3.234	1.066	25	0.292	0.61	1.16	1	0.34
S-4	10	6.10	1.1557	1	1.1	13	15	2.498	1.048	16	0.168	0.74	1.09	1	0.18
S-5	16	7.62	1.0849	1	1.1	19	15	2.498	1.048	23	0.249	0.88	1.04	1	0.26
S-6	14	9.14	1.0222	1	1.1	16	28	4.562	1.138	22	0.249	1.01	1.00	1	0.25
S-7	10	10.67	0.9664	1	1.1	11	20	3.615	1.079	15	0.161	1.14	0.96	1	0.15
S-8	12	12.19	0.9164	1	1.1	12	20	3.615	1.079	17	0.177	1.27	0.93	1	0.17
							lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction MCE
Upper Wrangell Dam

DH-1-05 a_{max} / g = 0.23

DH-1-05	N_m	USCS	% fines	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft	σ_o' , lb/ft	CSR
S-1	34	SM-GM	16	125	5	1.52	0.988	625	625	0.15
S-2	19	"	16	130	10	3.05	0.977	1275	963	0.19
S-3	23	"	20	130	15	4.57	0.965	1925	1301	0.21
S-4	7	SM-ML	50	115	20	6.10	0.953	2500	1564	0.23
S-5	8	"	25	115	25	7.62	0.942	3075	1827	0.24
S-6	11	"	25	115	30	9.14	0.930	3650	2090	0.24
S-7	9	"	20	115	35	10.67	0.889	4225	2353	0.24
S-8	20	SM-OL	20	120	40	12.19	0.848	4825	2641	0.23
S-9	18	GM	15	125	45	13.72	0.808	5450	2954	0.22
S-10	50	OL-SP	4	125	50	15.24	0.767	6075	3267	0.21
S-11	26	SG	15	130	55	16.76	0.726	6725	3605	0.2
S-12	94	GP	15	130	60	18.29	0.686	7375	3943	0.19
S-13	30	GM-OL	21	130	65	19.81	0.645	8025	4281	0.18
							Eqn 2a or 2b			Eqn (1)

DH-1-05 Depth	Factor of Safety CSR	CRR	M	MSF	FS
5	0.15	N/A	7.4	1.03	N/A
10	0.19	N/A	7.4	1.03	N/A
15	0.21	N/A	7.4	1.03	N/A
20	0.23	0.16	7.4	1.03	0.72
25	0.24	0.16	7.4	1.03	0.69
30	0.24	0.19	7.4	1.03	0.82
35	0.24	0.14	7.4	1.03	0.60
40	0.23	0.27	7.4	1.03	1.21
45	0.22	0.19	7.4	1.03	0.89
50	0.21	N/A	7.4	1.03	N/A
55	0.2	0.25	7.4	1.03	1.29
60	0.19	N/A	7.4	1.03	N/A
65	0.18	0.30	7.4	1.03	1.72
				Eqn (2	Eqn (2

Determination of CRR (Cyclic Resistance Ratio)

DH-1-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	34	1.52	1.4712	0.75	1.1	41	16	2.767	1.054	46	0.256	0.31	1.00	1	N/A
S-2	19	3.05	1.3293	1	1.1	28	16	2.767	1.054	32	0.745	0.48	1.00	1	N/A
S-3	23	4.57	1.2123	1	1.1	31	20	3.615	1.079	37	-0.100	0.65	1.00	1	N/A
S-4	7	6.10	1.1346	1	1.1	9	50	5.000	1.200	15	0.165	0.78	1.00	1	0.16
S-5	8	7.62	1.0662	1	1.1	9	25	4.289	1.115	15	0.158	0.91	1.00	1	0.16
S-6	11	9.14	1.0057	1	1.1	12	25	4.289	1.115	18	0.190	1.05	0.99	1	0.19
S-7	9	10.67	0.9516	1	1.1	9	20	3.615	1.079	14	0.148	1.18	0.95	1	0.14
S-8	20	12.19	0.8987	1	1.1	20	20	3.615	1.079	25	0.291	1.32	0.92	1	0.27
S-9	18	13.72	0.8475	1	1.1	17	15	2.498	1.048	20	0.216	1.48	0.89	1	0.19
S-10	50	15.24	0.8018	1	1.1	44	4	0.000	1.000	44	0.223	1.63	0.86	1	N/A
S-11	26	16.76	0.7577	1	1.1	22	15	2.498	1.048	25	0.296	1.80	0.84	1	0.25
S-12	94	18.29	0.7182	1	1.1	74	15	2.498	1.048	80	0.569	1.97	0.82	1	N/A
S-13	30	19.81	0.6826	1	1.1	23	21	3.778	1.086	28	0.378	2.14	0.80	1	0.30
			$C_N = 2.2/(1.2 + \sigma_o'/P_a)$	C_R = Correction for rod length		Eqn (8)	lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

P_a = 2116.2 psf

Factor of Safety Against Liquefaction - MCE
Upper Wrangell Dam

DH-2-05 $a_{max} / g =$ 0.23

DH-1-05	N_m	USCS	% fines	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o, lb/ft^2	σ_o', lb/ft^2	CSR
S-1	22	SM-GM	15	125	5	1.52	0.988	625	625	0.15
S-2	30	"	15	130	10	3.05	0.977	1275	963	0.19
S-3	25	SM-GM	23	130	15	4.57	0.965	1925	1301	0.21
S-4	27	"	25	130	20	6.10	0.953	2575	1639	0.22
S-5	58	"	25	115	25	7.62	0.942	3150	1902	0.23
S-6	10	SM	25	115	30	9.14	0.930	3725	2165	0.24
S-7	17	SM	21	115	35	10.67	0.889	4300	2428	0.24
S-8	11	"	20	120	40	12.19	0.848	4900	2716	0.23
S-9	25	SM-GM	15	125	45	13.72	0.808	5525	3029	0.22
S-10	25	SM-GM	17	125	50	15.24	0.767	6150	3342	0.21
S-11	11	"	15	130	55	16.76	0.726	6800	3680	0.2
S-12	41	"	15	130	60	18.29	0.686	7450	4018	0.19
							Eqn 2a or 2b			Eqn (1)

DH-2-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.15	N/A	7.4	1.03	N/A
10	0.19	N/A	7.4	1.03	N/A
15	0.21	N/A	7.4	1.03	N/A
20	0.22	N/A	7.4	1.03	N/A
25	0.23	N/A	7.4	1.03	N/A
30	0.24	0.17	7.4	1.03	0.73
35	0.24	0.24	7.4	1.03	1.03
40	0.23	0.15	7.4	1.03	0.67
45	0.22	0.29	7.4	1.03	1.36
50	0.21	0.27	7.4	1.03	1.33
55	0.2	0.11	7.4	1.03	0.57
60	0.19	N/A	7.4	1.03	N/A
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-2-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o', tsf	K_σ	K_α	CRR
S-1	22	1.52	1.4712	0.75	1.1	27	15	2.498	1.048	30	0.506	0.31	1.00	1	N/A
S-2	30	3.05	1.3293	1	1.1	44	15	2.498	1.048	48	0.285	0.48	1.00	1	N/A
S-3	25	4.57	1.2123	1	1.1	33	23	4.059	1.100	41	0.149	0.65	1.00	1	N/A
S-4	27	6.10	1.1142	1	1.1	33	25	4.289	1.115	41	0.161	0.82	1.00	1	N/A
S-5	58	7.62	1.0482	1	1.1	67	25	4.289	1.115	79	0.557	0.95	1.00	1	N/A
S-6	10	9.14	0.9896	1	1.1	11	25	4.289	1.115	16	0.175	1.08	0.98	1	0.17
S-7	17	10.67	0.9372	1	1.1	18	21	3.778	1.086	23	0.254	1.21	0.94	1	0.24
S-8	11	12.19	0.8859	1	1.1	11	20	3.615	1.079	15	0.162	1.36	0.91	1	0.15
S-9	25	13.72	0.8361	1	1.1	23	15	2.498	1.048	27	0.328	1.51	0.88	1	0.29
S-10	25	15.24	0.7916	1	1.1	22	17	3.012	1.060	26	0.315	1.67	0.86	1	0.27
S-11	11	16.76	0.7486	1	1.1	9	15	2.498	1.048	12	0.131	1.84	0.83	1	0.11
S-12	41	18.29	0.71	1	1.1	32	15	2.498	1.048	36	-0.223	2.01	0.81	1	N/A
S-13															
							lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - MCE
Upper Wrangell Dam

DH-3-05 $a_{max} / g =$ 0.23

DH-1-05	N_m	USCS	% fines	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft	σ_o' , lb/ft	CSR
S-1	56		15	125	5	1.52	0.988	625	625	0.15
S-2	23		15	130	10	3.05	0.977	1275	963	0.19
S-3	15	SM	18	115	15	4.57	0.965	1850	1226	0.22
S-4	10		20	115	20	6.10	0.953	2425	1489	0.23
S-5	16		25	115	25	7.62	0.942	3000	1752	0.24
S-6	14	SM	28	115	30	9.14	0.930	3575	2015	0.25
S-7	10		20	115	35	10.67	0.889	4150	2278	0.24
S-8	12		20	115	40	12.19	0.848	4725	2541	0.24
							Eqn 2a or 2b			Eqn (1)

DH-3-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.15	N/A	7.4	1.03	N/A
10	0.19	N/A	7.4	1.03	N/A
15	0.22	0.29	7.4	1.03	1.36
20	0.23	0.18	7.4	1.03	0.81
25	0.24	0.30	7.4	1.03	1.29
30	0.25	0.25	7.4	1.03	1.03
35	0.24	0.15	7.4	1.03	0.65
40	0.24	0.17	7.4	1.03	0.73
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-3-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	56	1.52	1.4712	0.75	1.1	68	15	2.498	1.048	74	0.516	0.31	1.00	1	N/A
S-2	23	3.05	1.3293	1	1.1	34	15	2.498	1.048	38	0.008	0.48	1.00	1	N/A
S-3	15	4.57	1.2364	1	1.1	20	18	3.234	1.066	25	0.292	0.61	1.00	1	0.29
S-4	10	6.10	1.1557	1	1.1	13	20	3.615	1.079	17	0.184	0.74	1.00	1	0.18
S-5	16	7.62	1.0849	1	1.1	19	25	4.289	1.115	26	0.304	0.88	1.00	1	0.30
S-6	14	9.14	1.0222	1	1.1	16	28	4.562	1.138	22	0.249	1.01	1.00	1	0.25
S-7	10	10.67	0.9664	1	1.1	11	20	3.615	1.079	15	0.161	1.14	0.96	1	0.15
S-8	12	12.19	0.9164	1	1.1	12	20	3.615	1.079	17	0.177	1.27	0.93	1	0.17
				C_R = Correction for rod length										flat	
		$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$					lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	ground	

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - MCE
Lower Wrangell Dam

DH-4-05 $a_{max} / g =$ 0.23

DH-4-05	N_m	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o, lb/ft²	σ_o', lb/ft²	CSR
S-1	31	135	5	1.52	0.988	675	675	0.15
S-2	54	135	10	3.05	0.977	1350	1350	0.15
S-3	21	135	15	4.57	0.965	2025	2025	0.14
S-4	19	135	20	6.10	0.953	2700	2388	0.16
					Eqn 2a or 2b			Eqn (1)

Determination of CRR (Cyclic Resistance Ratio)

DH-4-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o', tsf	K_σ	K_α	CRR
S-1	31	1.52	1.4484	0.75	1.1	37	20	3.615	1.079	44	0.214	0.34	1.00	1	N/A
S-2	54	3.05	1.197	0.8	1.1	57	20	3.615	1.079	65	0.444	0.68	1.00	1	N/A
S-3	21	4.57	1.02	0.85	1.1	20	20	3.615	1.079	25	0.297	1.01	1.00	1	0.30
S-4	19	6.10	0.9448	0.95	1.1	19	20	3.615	1.079	24	0.271	1.19	0.95	1	0.26
						Eqn (8)	lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/P_a)$

$P_a =$ 2116.2 psf

Factor of Safety

DH-4-05 Depth	CSR	CRR	M	MSF	FS
5	0.15	N/A	7.4	1.03	N/A
10	0.15	N/A	7.4	1.03	N/A
15	0.14	0.30	7.4	1.03	2.22
20	0.16	0.26	7.4	1.03	1.68
				Eqn (24)	Eqn (23)

Factor of Safety Against Liquefaction - MCE
Lower Wrangell Dam

DH-5-05 $a_{max} / g =$ 0.23

DH-5-05	N_m	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft^2	σ_o' , lb/ft^2	CSR
S-1	25	135	5	1.52	0.988	675	675	0.15
S-2	15	135	10	3.05	0.977	1350	1350	0.15
S-3	50	135	15	4.57	0.965	2025	1713	0.17
S-4	11	135	20	6.10	0.953	2700	2076	0.19
S-5	23	115	25	7.62	0.942	3275	2339	0.2
S-6	70	115	30	9.14	0.930	3850	2602	0.21
S-7	29	115	35	10.67	0.889	4425	2865	0.21
S-8	37	115	40	12.19	0.848	5000	3128	0.2
S-9	10	115	45	13.72	0.808	5575	3391	0.2
					Eqn 2a or 2b			Eqn (1)

DH-5-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.15	N/A	7.4	1.03	N/A
10	0.15	0.22	7.4	1.03	1.52
15	0.17	N/A	7.4	1.03	N/A
20	0.19	0.17	7.4	1.03	0.93
25	0.2	0.37	7.4	1.03	1.91
30	0.21	N/A	7.4	1.03	N/A
35	0.21	N/A	7.4	1.03	N/A
40	0.2	N/A	7.4	1.03	N/A
45	0.2	0.12	7.4	1.03	0.62
					Eqn (24)

Determination of CRR (Cyclic Resistance Ratio)

DH-5-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	25	1.52	1.4484	0.75	1.1	30	20	3.615	1.079	36	-0.277	0.34	1.00	1	N/A
S-2	15	3.05	1.197	0.8	1.1	16	20	3.615	1.079	21	0.224	0.68	1.00	1	0.22
S-3	50	4.57	1.0948	0.85	1.1	51	20	3.615	1.079	59	0.391	0.86	1.00	1	N/A
S-4	11	6.10	1.0087	0.95	1.1	12	20	3.615	1.079	16	0.172	1.04	0.99	1	0.17
S-5	23	7.62	0.9543	0.95	1.1	23	20	3.615	1.079	28	0.383	1.17	0.95	1	0.37
S-6	70	9.14	0.9055	0.95	1.1	66	20	3.615	1.079	75	0.527	1.30	0.92	1	N/A
S-7	29	10.67	0.8614	1	1.1	27	20	3.615	1.079	33	1.627	1.43	0.90	1	N/A
S-8	37	12.19	0.8215	1	1.1	33	20	3.615	1.079	40	0.114	1.56	0.87	1	N/A
S-9	10	13.72	0.785	1	1.1	9	20	3.615	1.079	13	0.140	1.70	0.85	1	0.12
							lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - MCE
Lower Wrangell Dam

DH-6-05 $a_{max} / g =$ 0.23

DH-6-05	N_m	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft^2	σ_o' , lb/ft^2	CSR
S-1	10	120	5	1.52	0.988	600	600	0.15
S-2	34	120	10	3.05	0.977	1200	888	0.2
					Eqn 2a or 2b			Eqn (1)

DH-6-05 Depth	Factor of Safety CSR	CRR	M	MSF	FS
5	0.15	0.18	7.4	1.03	1.24
10	0.2	N/A	7.4	1.03	N/A
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-6-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	10	1.52	1.483	0.75	1.1	12	20	3.615	1.079	17	0.179	0.30	1.00	1	0.18
S-2	34	3.05	1.3583	0.8	1.1	41	20	3.615	1.079	47	0.273	0.44	1.00	1	N/A
							lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - OBE
Lower Wrangell Dam

DH-4-05 $a_{max} / g =$ 0.1

DH-4-05	N_m	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft^2	σ_o' , lb/ft^2	CSR
S-1	31	135	5	1.52	0.988	675	675	0.06
S-2	54	135	10	3.05	0.977	1350	1350	0.06
S-3	21	135	15	4.57	0.965	2025	2025	0.06
S-4	19	135	20	6.10	0.953	2700	2388	0.07
					Eqn 2a or 2b			Eqn (1)

DH-4-05	Factor of Safety				
Depth	CSR	CRR	M	MSF	FS
5	0.06	N/A	6.2	1.63	N/A
10	0.06	N/A	6.2	1.63	N/A
15	0.06	0.30	6.2	1.63	8.14
20	0.07	0.26	6.2	1.63	6.04
				Eqn (24)	Eqn (23)

Determination of CRR (Cyclic Resistance Ratio)

DH-4-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	31	1.52	1.4484	0.75	1.1	37	20	3.615	1.079	44	0.214	0.34	1.00	1	N/A
S-2	54	3.05	1.197	0.8	1.1	57	20	3.615	1.079	65	0.444	0.68	1.00	1	N/A
S-3	21	4.57	1.02	0.85	1.1	20	20	3.615	1.079	25	0.297	1.01	1.00	1	0.30
S-4	19	6.10	0.9448	0.95	1.1	19	20	3.615	1.079	24	0.271	1.19	0.95	1	0.26
						Eqn (8)	lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - OBE
Lower Wrangell Dam

DH-5-05 $a_{max} / g =$ 0.1

DH-5-05	N_m	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft²	σ_o' , lb/ft²	CSR
S-1	25	135	5	1.52	0.988	675	675	0.06
S-2	15	135	10	3.05	0.977	1350	1350	0.06
S-3	50	135	15	4.57	0.965	2025	1713	0.07
S-4	11	135	20	6.10	0.953	2700	2076	0.08
S-5	23	115	25	7.62	0.942	3275	2339	0.09
S-6	70	115	30	9.14	0.930	3850	2602	0.09
S-7	29	115	35	10.67	0.889	4425	2865	0.09
S-8	37	115	40	12.19	0.848	5000	3128	0.09
S-9	10	115	45	13.72	0.808	5575	3391	0.09
					Eqn 2a or 2b			Eqn (1)

DH-5-05 Depth	Factor of Safety CSR	CRR	M	MSF	FS
5	0.06	N/A	6.2	1.63	N/A
10	0.06	0.22	6.2	1.63	5.97
15	0.07	N/A	6.2	1.63	N/A
20	0.08	0.17	6.2	1.63	3.46
25	0.09	0.37	6.2	1.63	6.69
30	0.09	N/A	6.2	1.63	N/A
35	0.09	N/A	6.2	1.63	N/A
40	0.09	N/A	6.2	1.63	N/A
45	0.09	0.12	6.2	1.63	2.17
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-5-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	25	1.52	1.4484	0.75	1.1	30	20	3.615	1.079	36	-0.277	0.34	1.00	1	N/A
S-2	15	3.05	1.197	0.8	1.1	16	20	3.615	1.079	21	0.224	0.68	1.00	1	0.22
S-3	50	4.57	1.0948	0.85	1.1	51	20	3.615	1.079	59	0.391	0.86	1.00	1	N/A
S-4	11	6.10	1.0087	0.95	1.1	12	20	3.615	1.079	16	0.172	1.04	0.99	1	0.17
S-5	23	7.62	0.9543	0.95	1.1	23	20	3.615	1.079	28	0.383	1.17	0.95	1	0.37
S-6	70	9.14	0.9055	0.95	1.1	66	20	3.615	1.079	75	0.527	1.30	0.92	1	N/A
S-7	29	10.67	0.8614	1	1.1	27	20	3.615	1.079	33	1.627	1.43	0.90	1	N/A
S-8	37	12.19	0.8215	1	1.1	33	20	3.615	1.079	40	0.114	1.56	0.87	1	N/A
S-9	10	13.72	0.785	1	1.1	9	20	3.615	1.079	13	0.140	1.70	0.85	1	0.12
				C_R = Correction for rod length										flat	
			$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$				lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	ground	

$P_a =$ 2116.2 psf

Factor of Safety Against Liquefaction - OBE
Lower Wrangell Dam

DH-6-05 $a_{max} / g =$ 0.1

DH-6-05	N_m	unit wt (pcf)	DEPTH (ft)	DEPTH (m)	r_d	σ_o , lb/ft^2	σ_o' , lb/ft^2	CSR
S-1	10	120	5	1.52	0.988	600	600	0.06
S-2	34	120	10	3.05	0.977	1200	888	0.09
					Eqn 2a or 2b			Eqn (1)

Factor of Safety

DH-6-05 Depth	CSR	CRR	M	MSF	FS
5	0.06	0.18	6.2	1.63	4.88
10	0.09	N/A	6.2	1.63	N/A
				Eqn (24)	

Determination of CRR (Cyclic Resistance Ratio)

DH-6-05	N_m	DEPTH (m)	C_N	C_R	C_S	$(N_1)_{60}$	% fines	α	β	$(N_1)_{60cs}$	CRR	σ_o' , tsf	K_σ	K_α	CRR
S-1	10	1.52	1.483	0.75	1.1	12	20	3.615	1.079	17	0.179	0.30	1.00	1	0.18
S-2	34	3.05	1.3583	0.8	1.1	41	20	3.615	1.079	47	0.273	0.44	1.00	1	N/A
							lab data	Eqn (6b)	Eqn (7b)	Eqn (5)	Eqn (4)		Fig. 15	flat ground	

C_R = Correction for rod length

$C_N = 2.2/(1.2 + \sigma_o'/ P_a)$

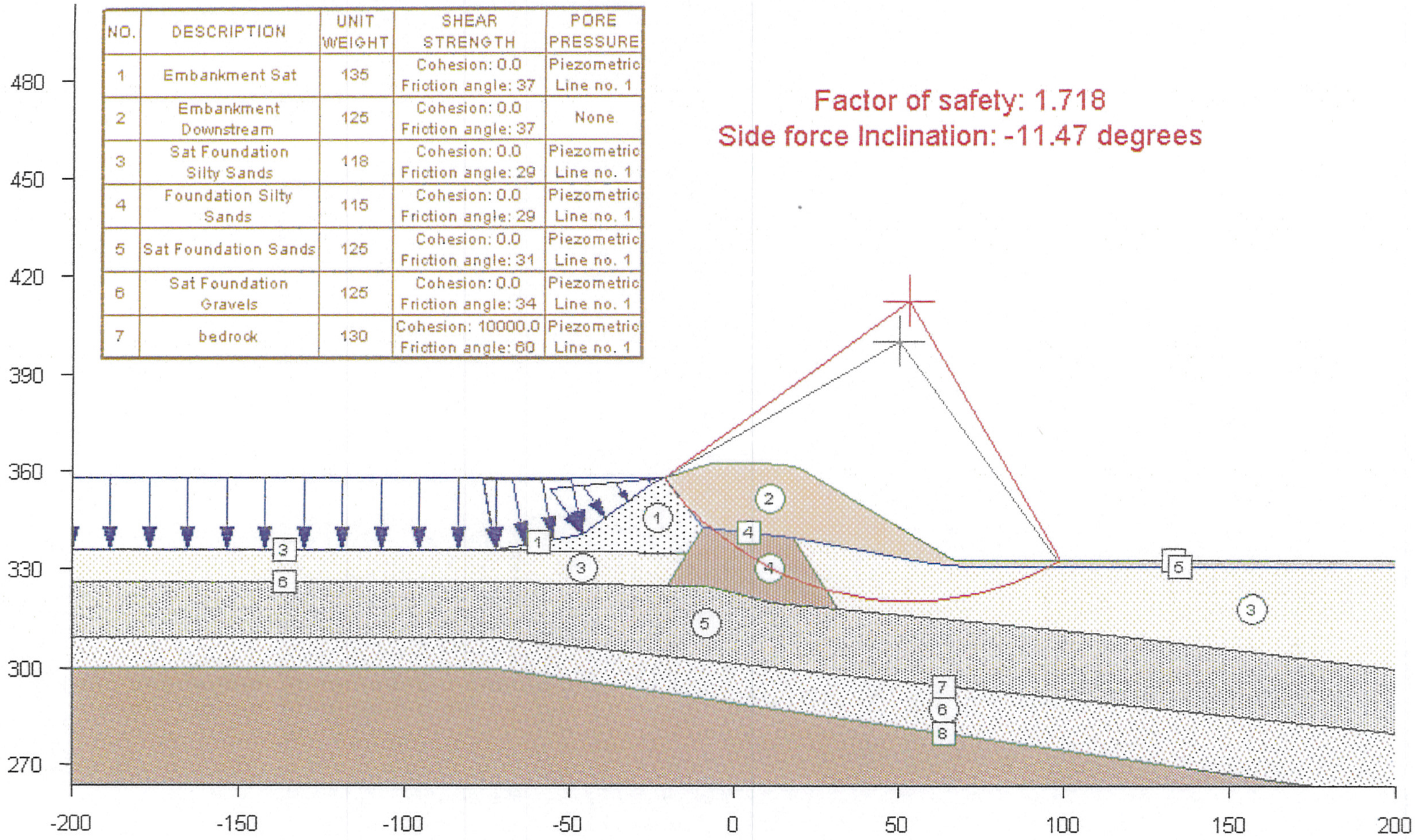
$P_a =$ 2116.2 psf

Appendix C

Stability Analysis

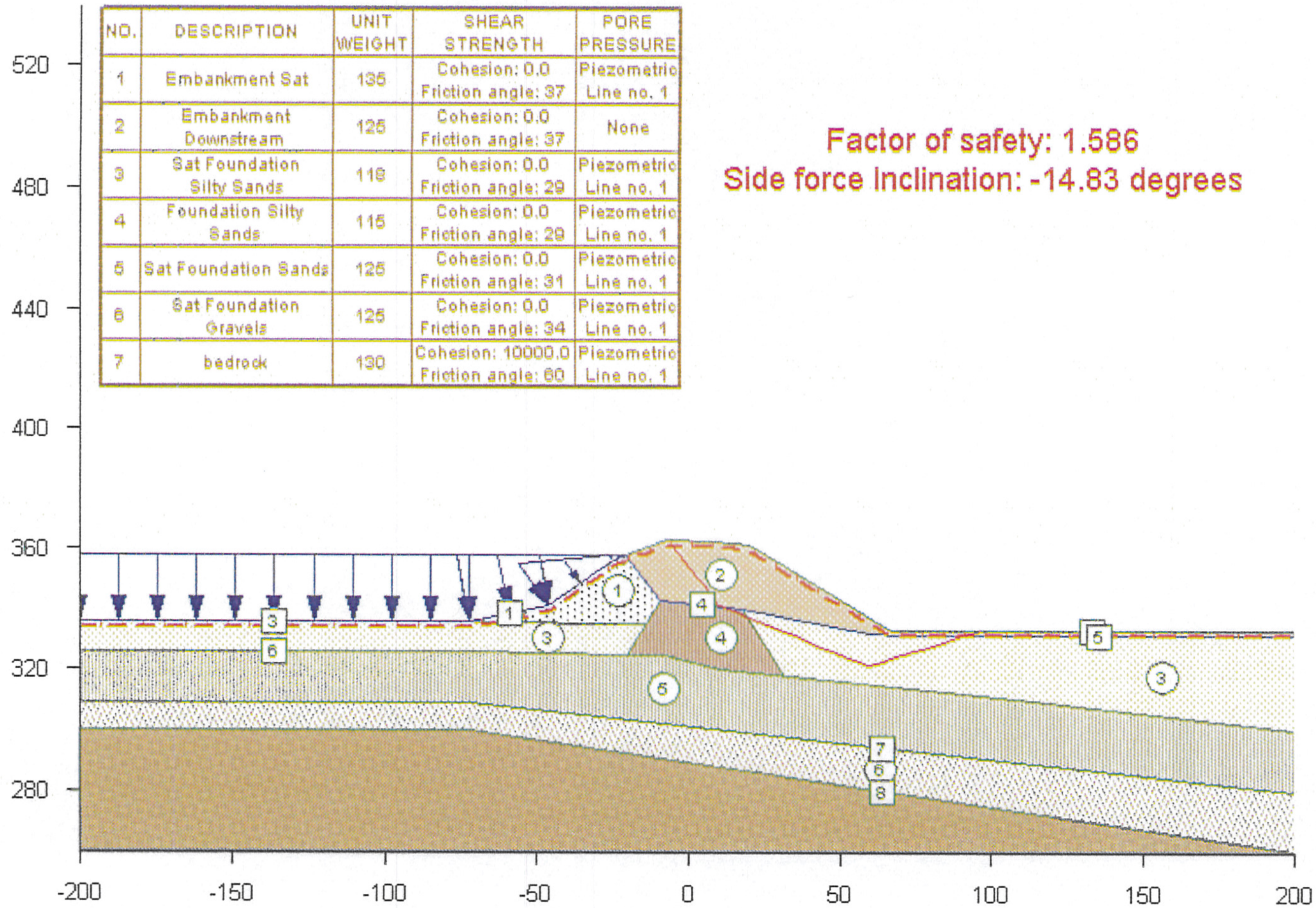
APPENDIX C – LIST OF FIGURES

	Upper Dam - Existing Conditions			
Figure	ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
C-1	Seismic - MCE	Downstream	circular	1.718
C-2	Seismic - MCE	Downstream	non-circular	1.450
C-3	Seismic - MCE	Upstream	circular	2.182
C-4	Seismic - MCE	Upstream	non-circular	2.080
	Upper Dam - MCE Stability Analysis			
Figure	ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
C-5	Seismic - MCE	Downstream	circular	0.066
C-6	Seismic - MCE	Downstream	non-circular	0.762
C-7	Seismic - MCE	Upstream	circular	1.364
C-8	Seismic - MCE	Upstream	non-circular	1.646
	Upper Dam - OBE Stability Analysis			
Figure	ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
C-9	Seismic - OBE	Downstream	circular	1.170
C-10	Seismic - OBE	Downstream	non-circular	1.220
C-11	Seismic - OBE	Upstream	circular	1.794
C-12	Seismic - OBE	Upstream	non-circular	1.644
	Upper Dam - Existing Conditions			
Figure	ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
C-13	Seismic - MCE	Downstream	circular	1.474
C-14	Seismic - MCE	Downstream	non-circular	1.572
C-15	Seismic - MCE	Upstream	circular	2.179
C-16	Seismic - MCE	Upstream	non-circular	2.192
	Lower Dam - MCE Stability Analysis			
Figure	ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
C-17	Seismic - MCE	Downstream	circular	1.061
C-18	Seismic - MCE	Downstream	non-circular	1.176
C-19	Seismic - MCE	Upstream	circular	1.686
C-20	Seismic - MCE	Upstream	non-circular	1.507
	Lower Dam - OBE Stability Analysis			
Figure	ANALYSIS	LOCATION	FAILURE TYPE	FACTOR OF SAFETY
C-21	Seismic - OBE	Downstream	circular	1.316
C-22	Seismic - OBE	Downstream	non-circular	1.430
C-23	Seismic - OBE	Upstream	circular	1.956
C-24	Seismic - OBE	Upstream	non-circular	1.909



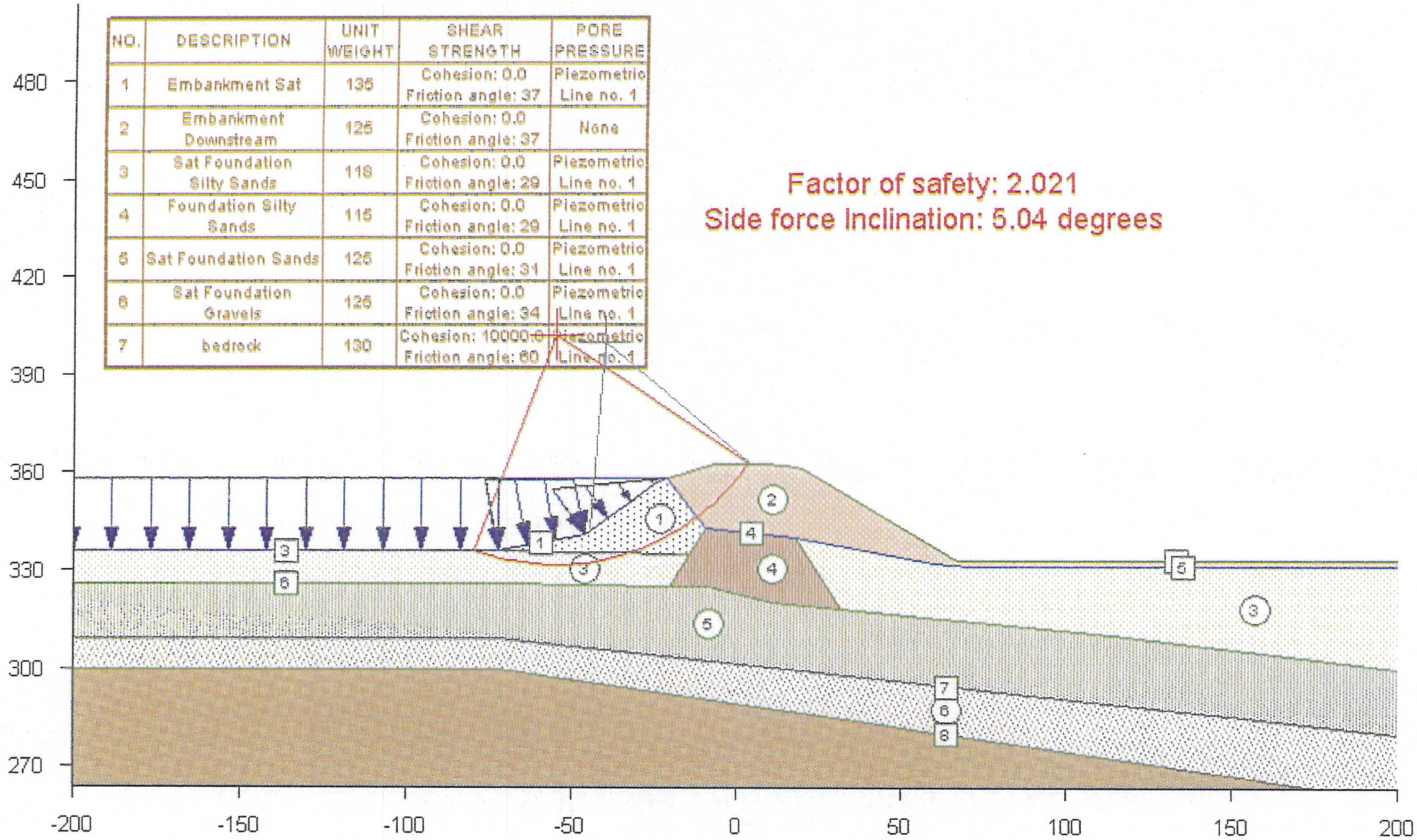
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Existing Upper Dam – Deep Failure to match Shannon & Wilson (1993) failure



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Existing Upper Dam

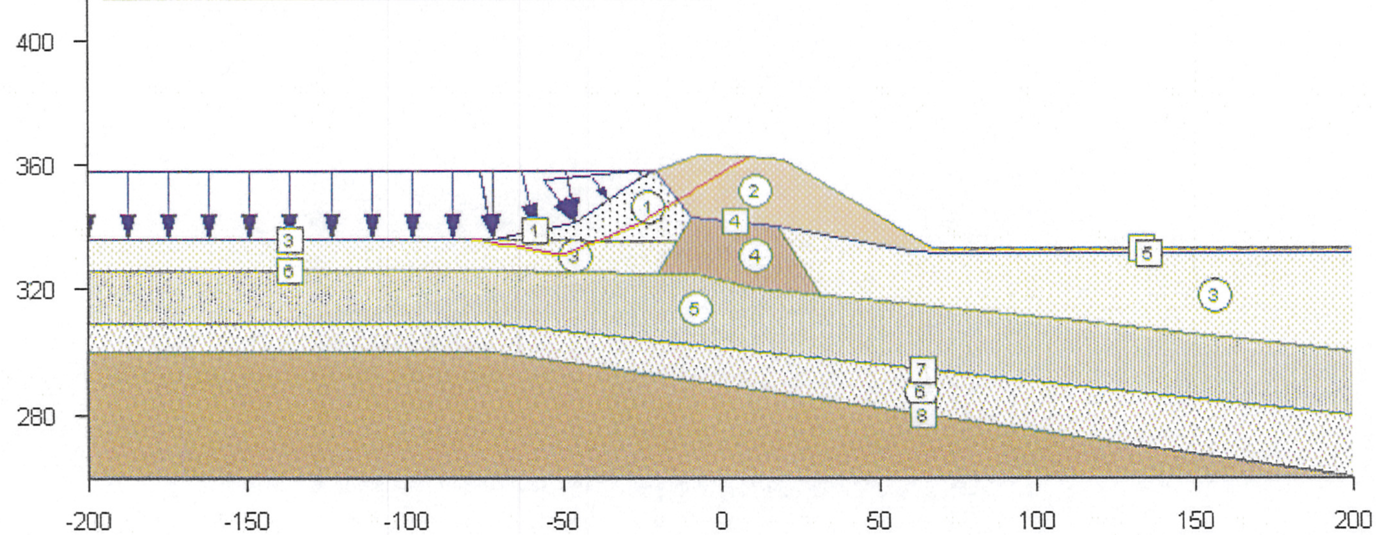


NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

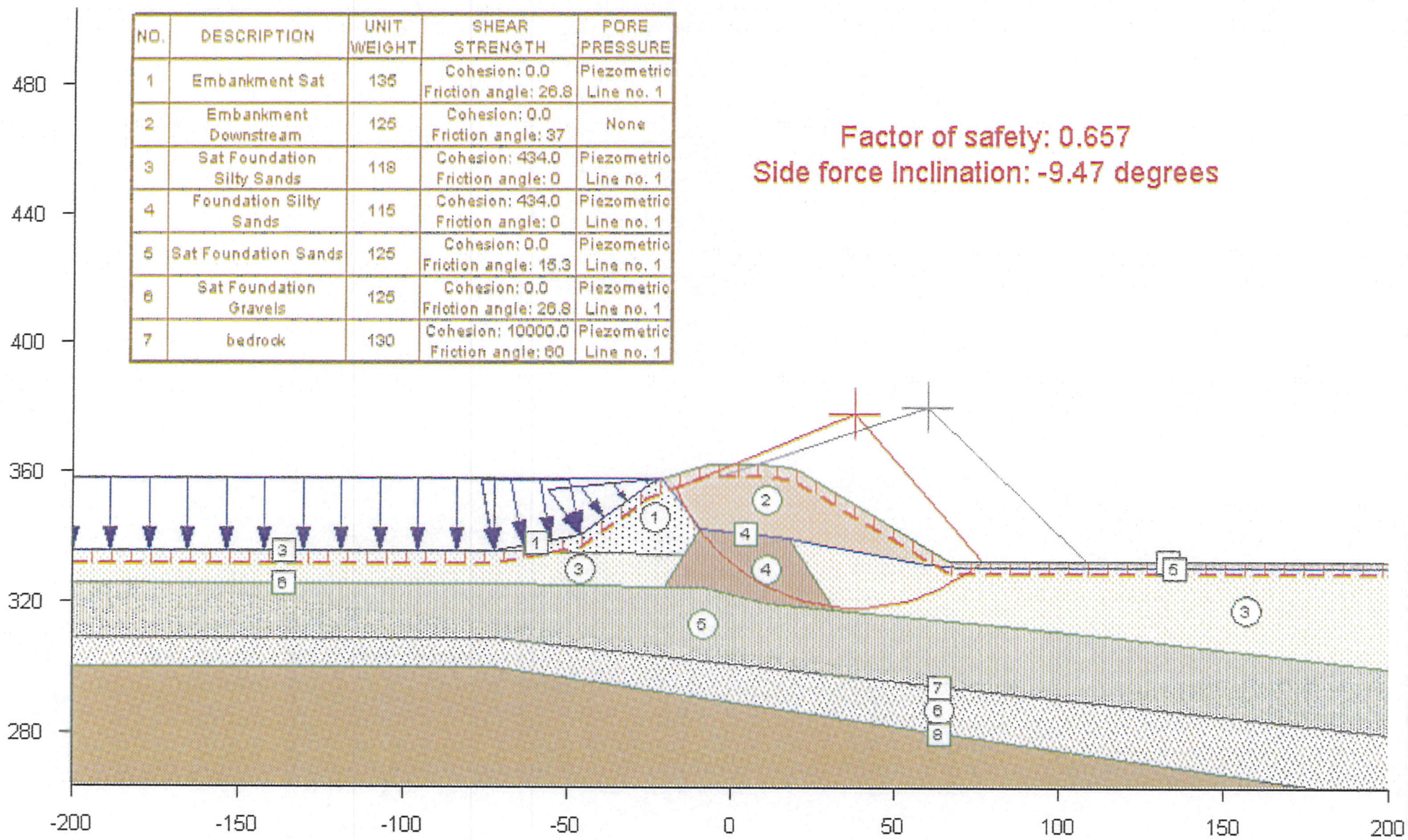
Existing Upper Dam

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Factor of safety: 2.087
Side force Inclination: 4.2 degrees

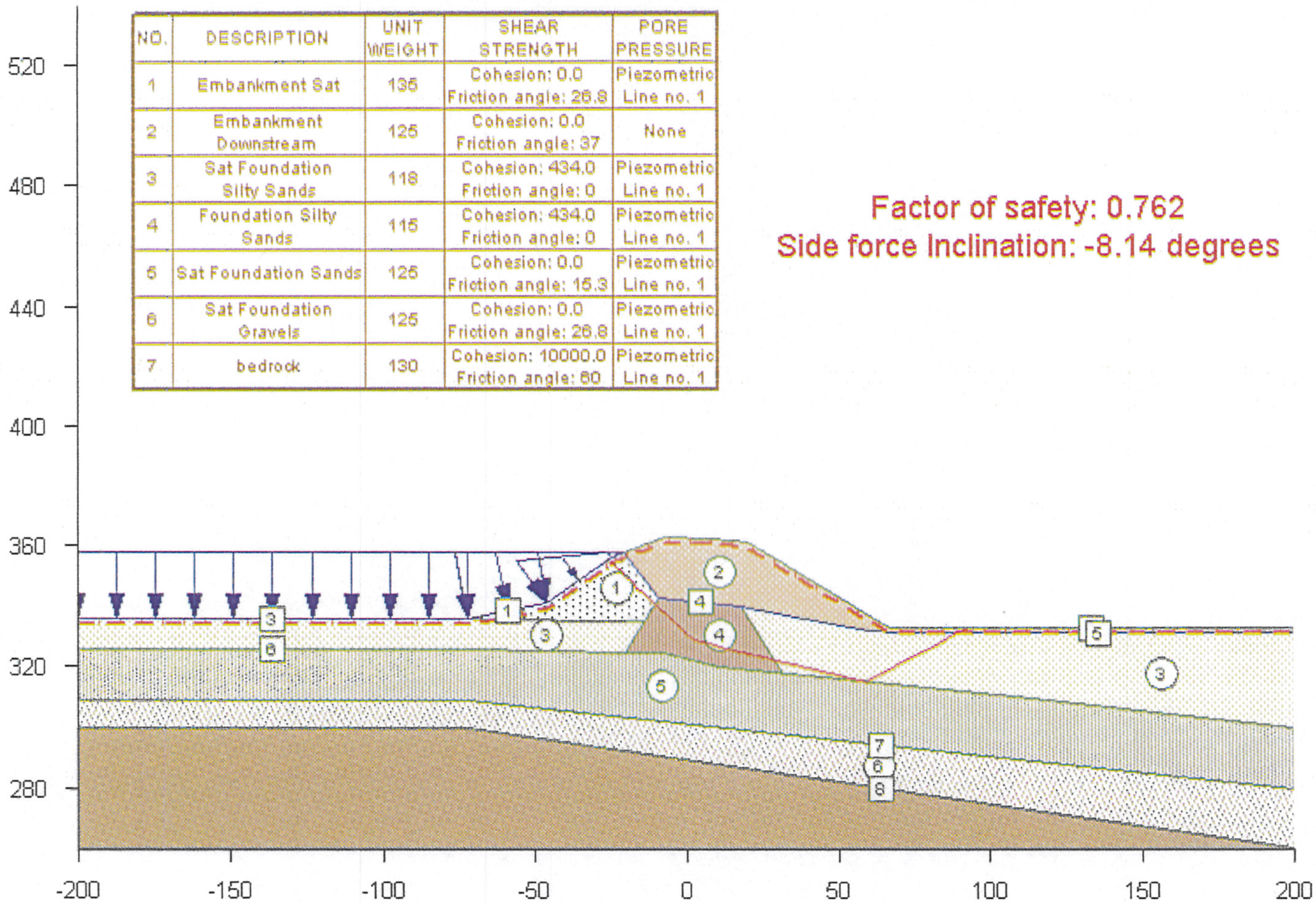


Existing Upper Dam



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 15.3	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Final MCE – Upper Dam



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 15.3	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

1 MCE – Upper Dam

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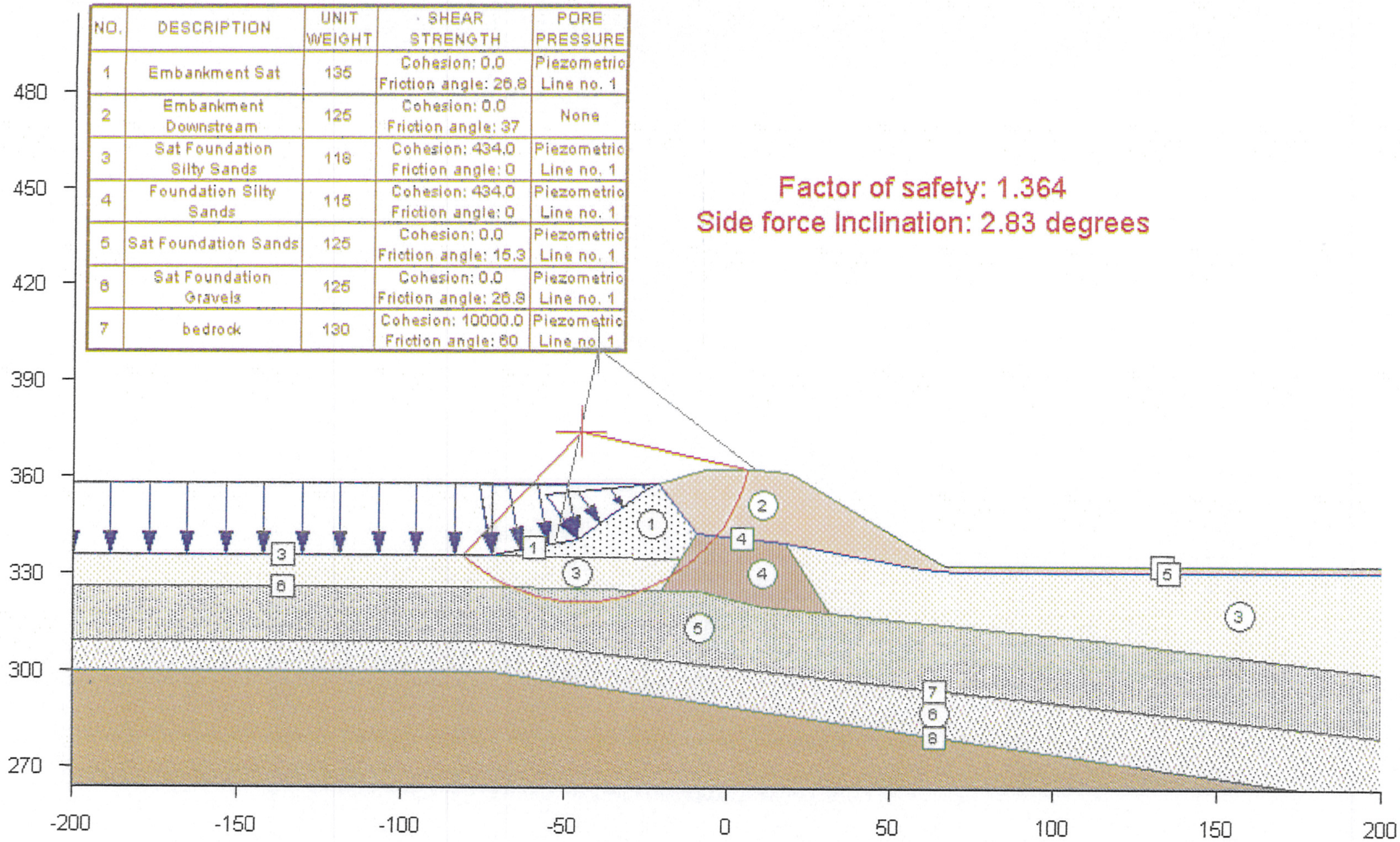
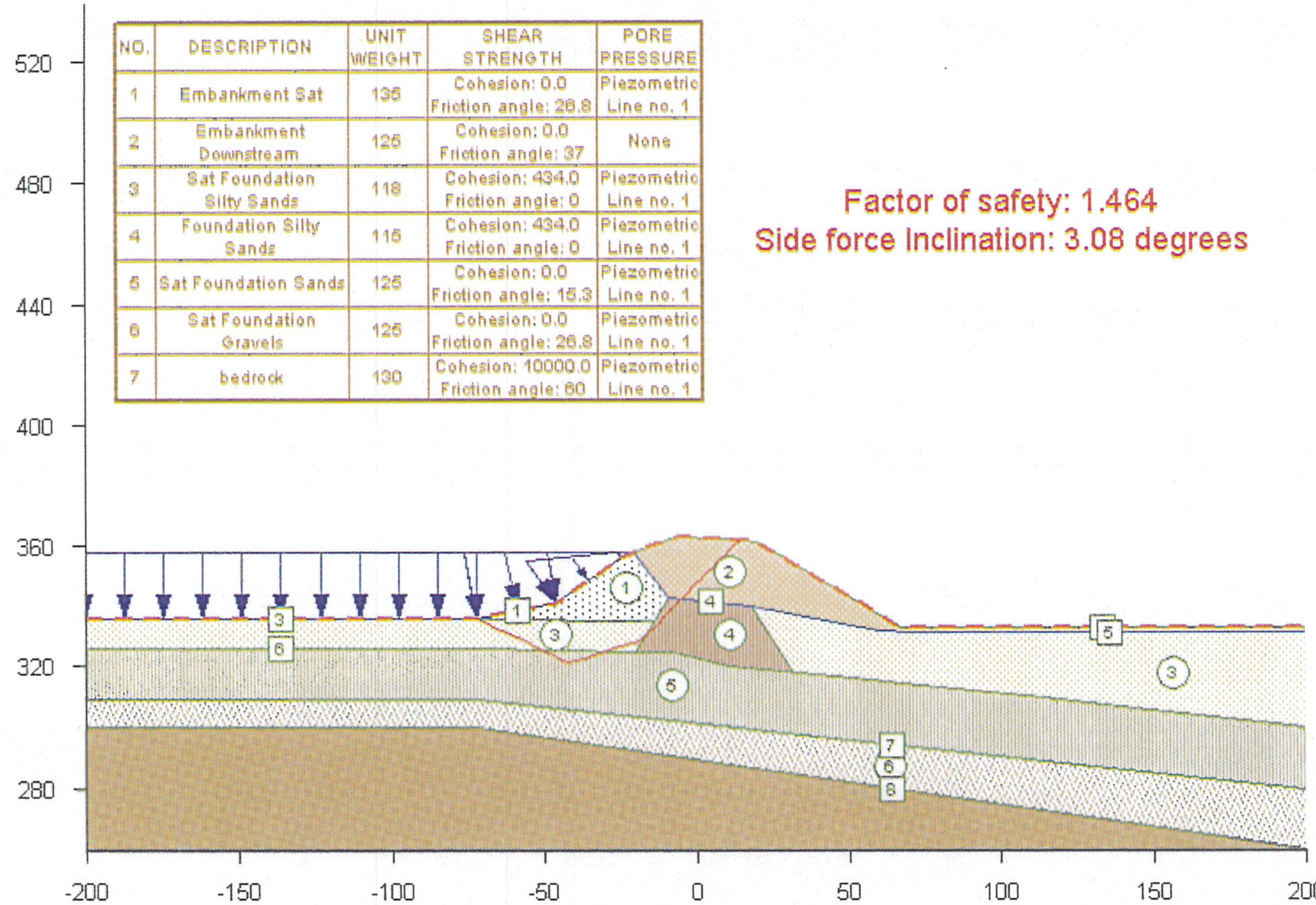
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 15.3	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Factor of safety: 1.364
Side force Inclination: 2.83 degrees

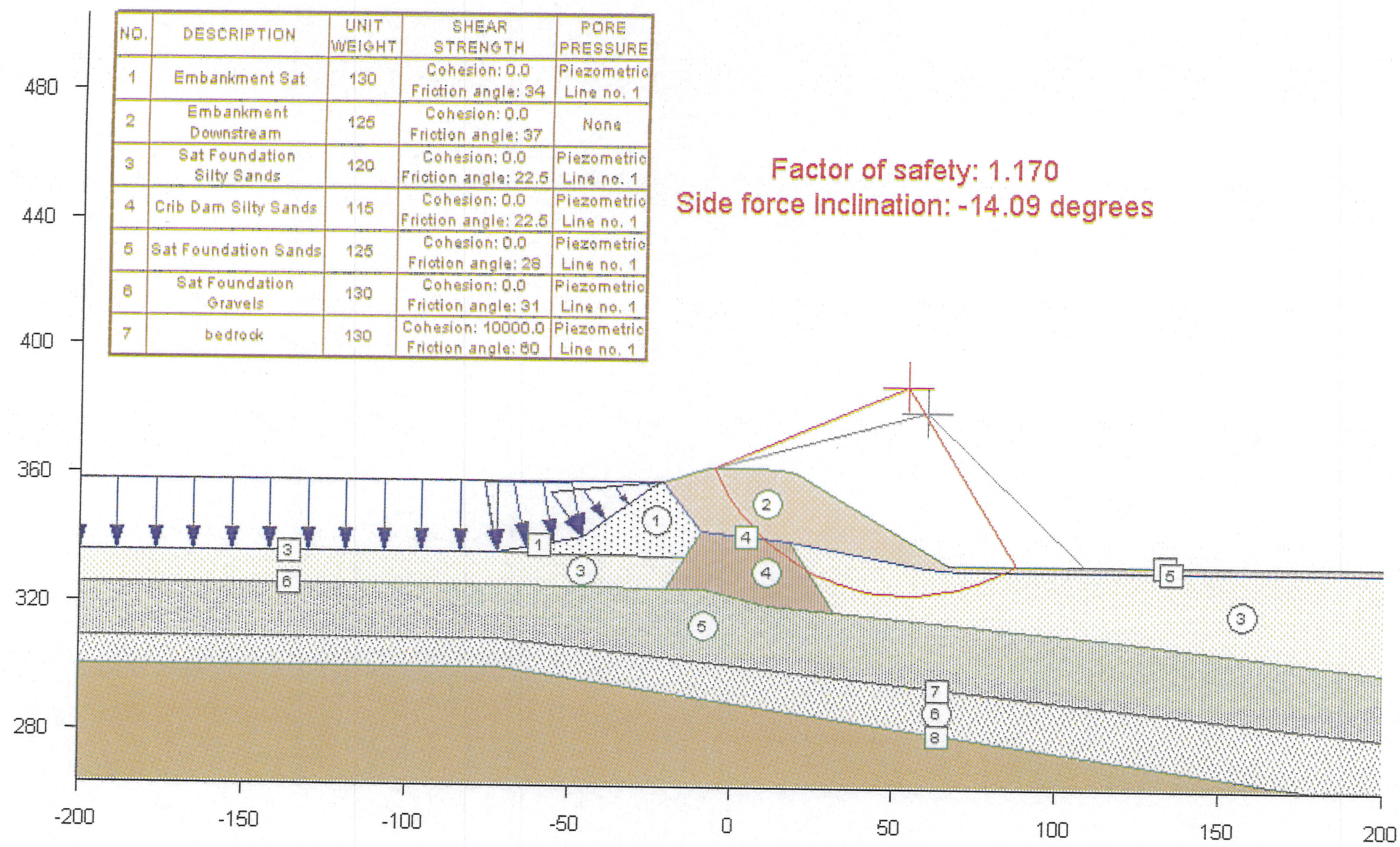


Final MCE – Upper Dam



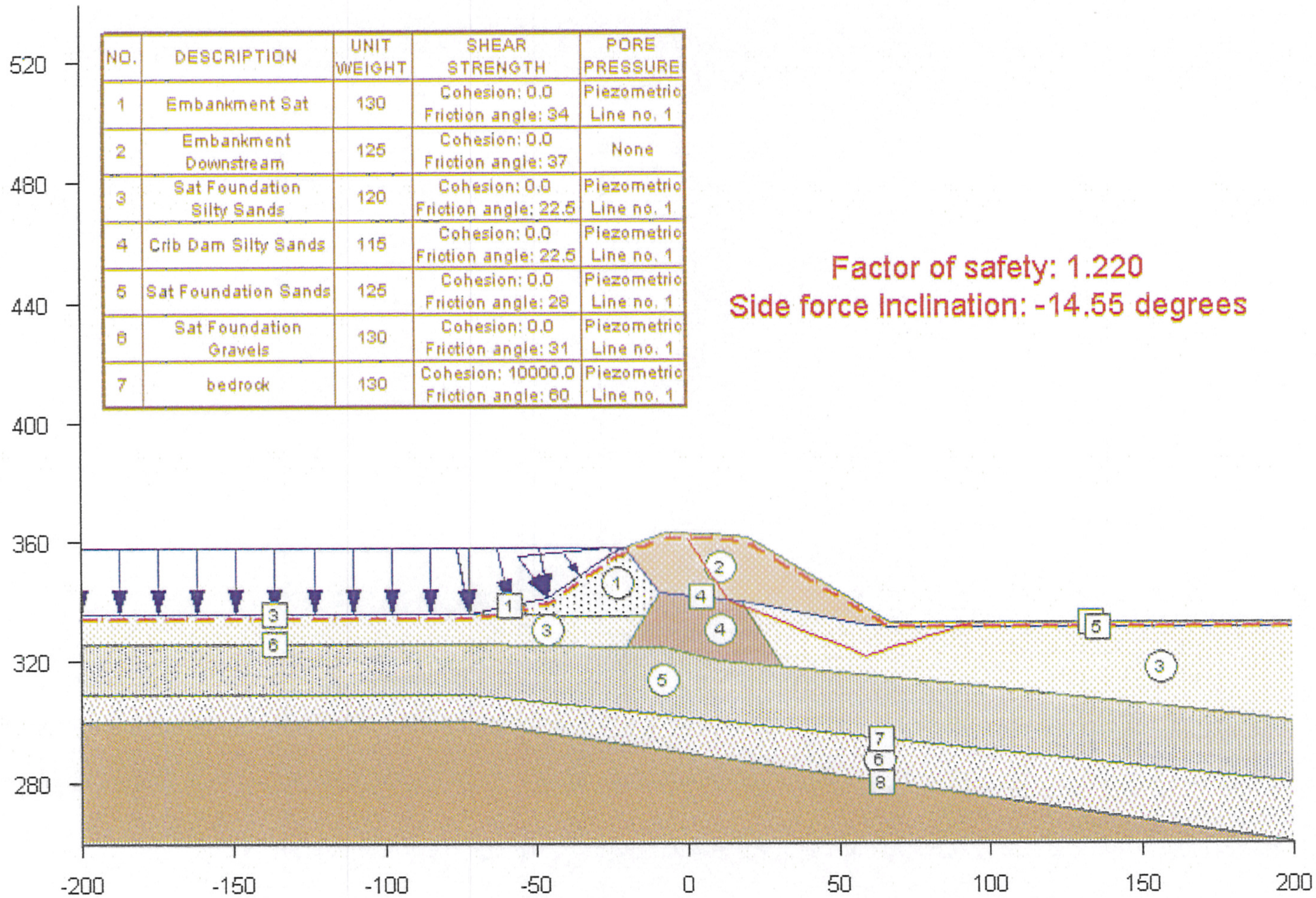
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	135	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	118	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
4	Foundation Silty Sands	115	Cohesion: 434.0 Friction angle: 0	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 15.3	Piezometric Line no. 1
6	Sat Foundation Gravels	125	Cohesion: 0.0 Friction angle: 26.8	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Final MCE – Upper Dam



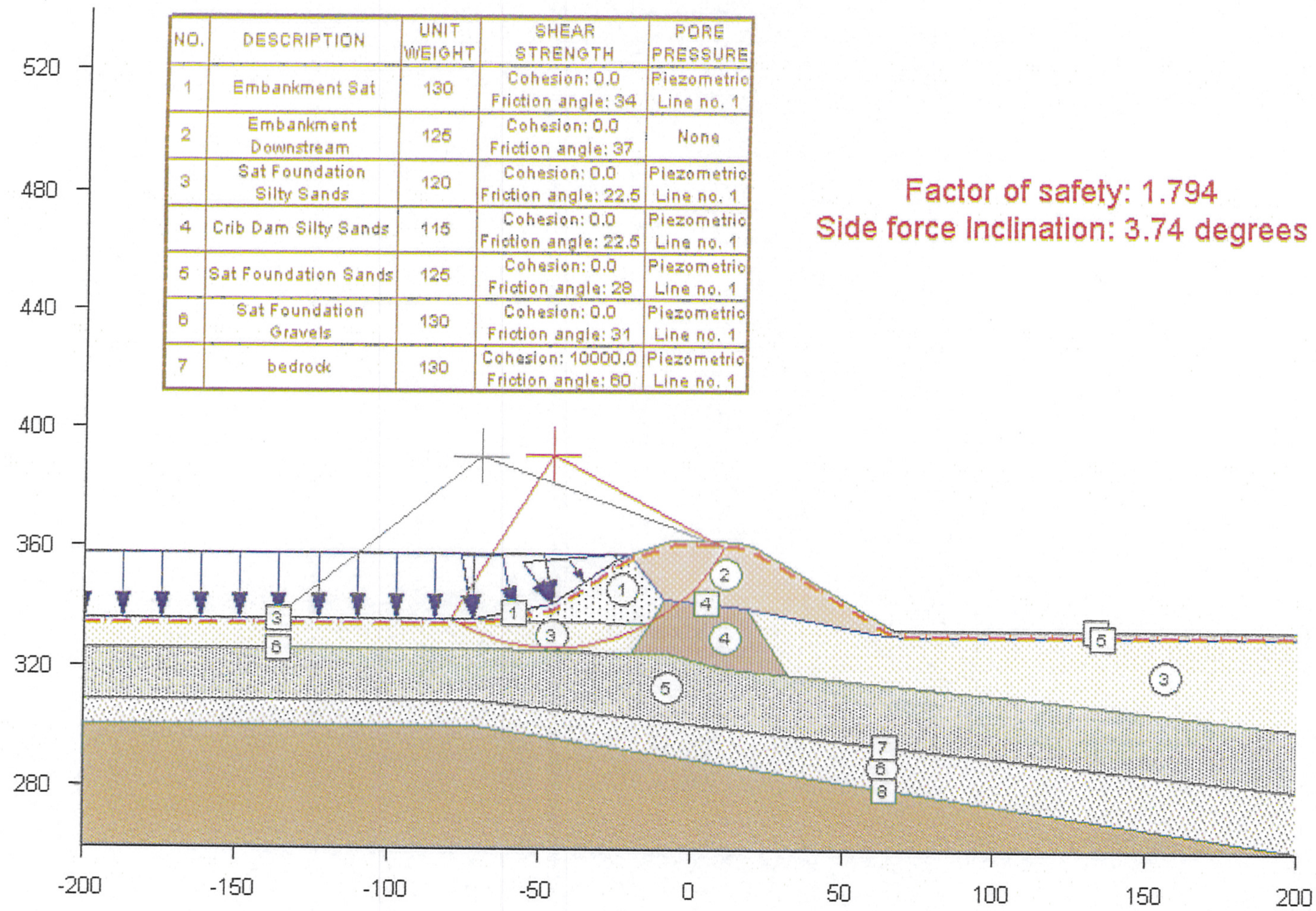
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	130	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	120	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
4	Crib Dam Silty Sands	115	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Sat Foundation Gravels	130	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Final OBE – Upper Dam



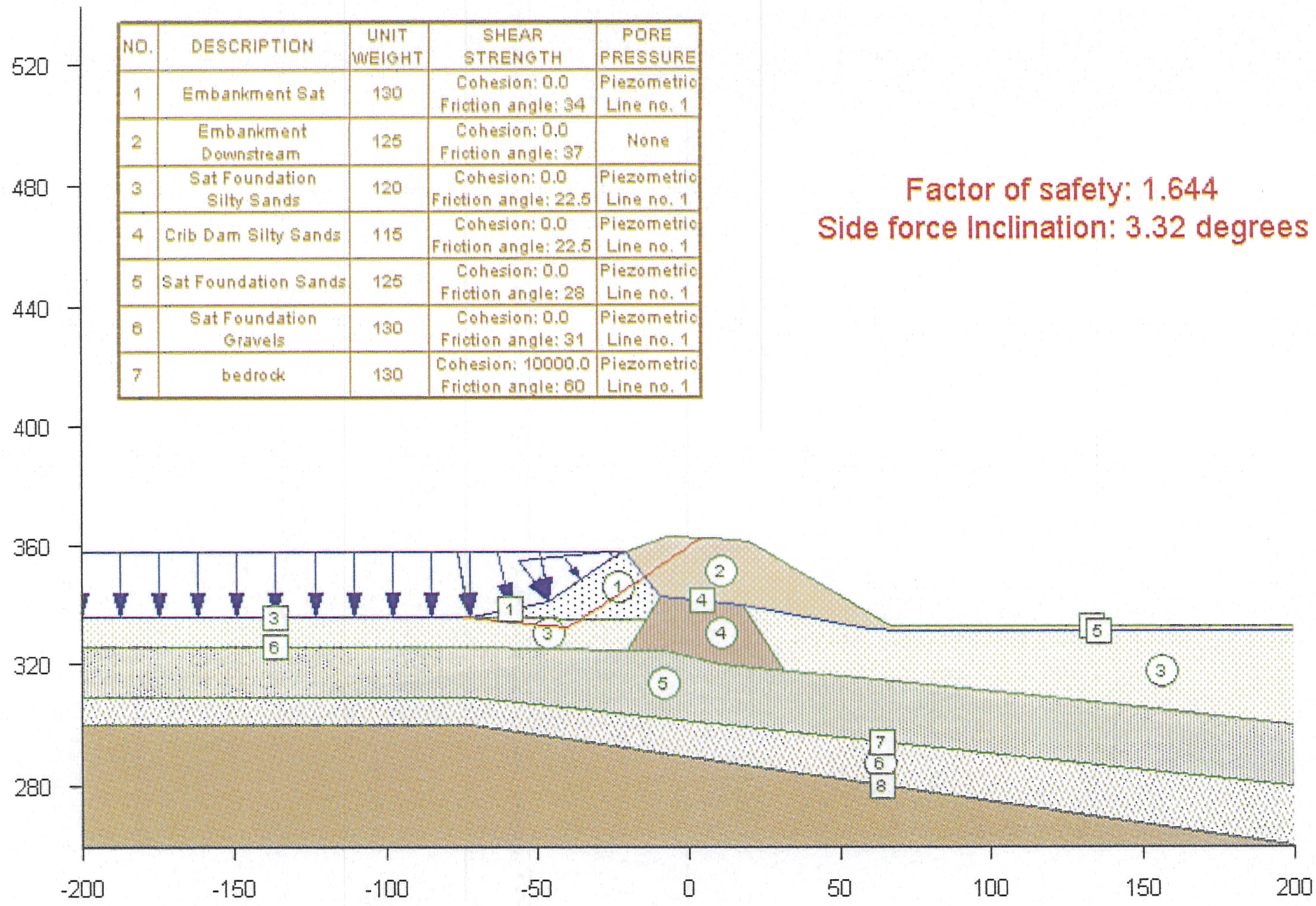
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	130	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	120	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
4	Crib Dam Silty Sands	115	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Sat Foundation Gravels	130	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Final OBE – Upper Dam



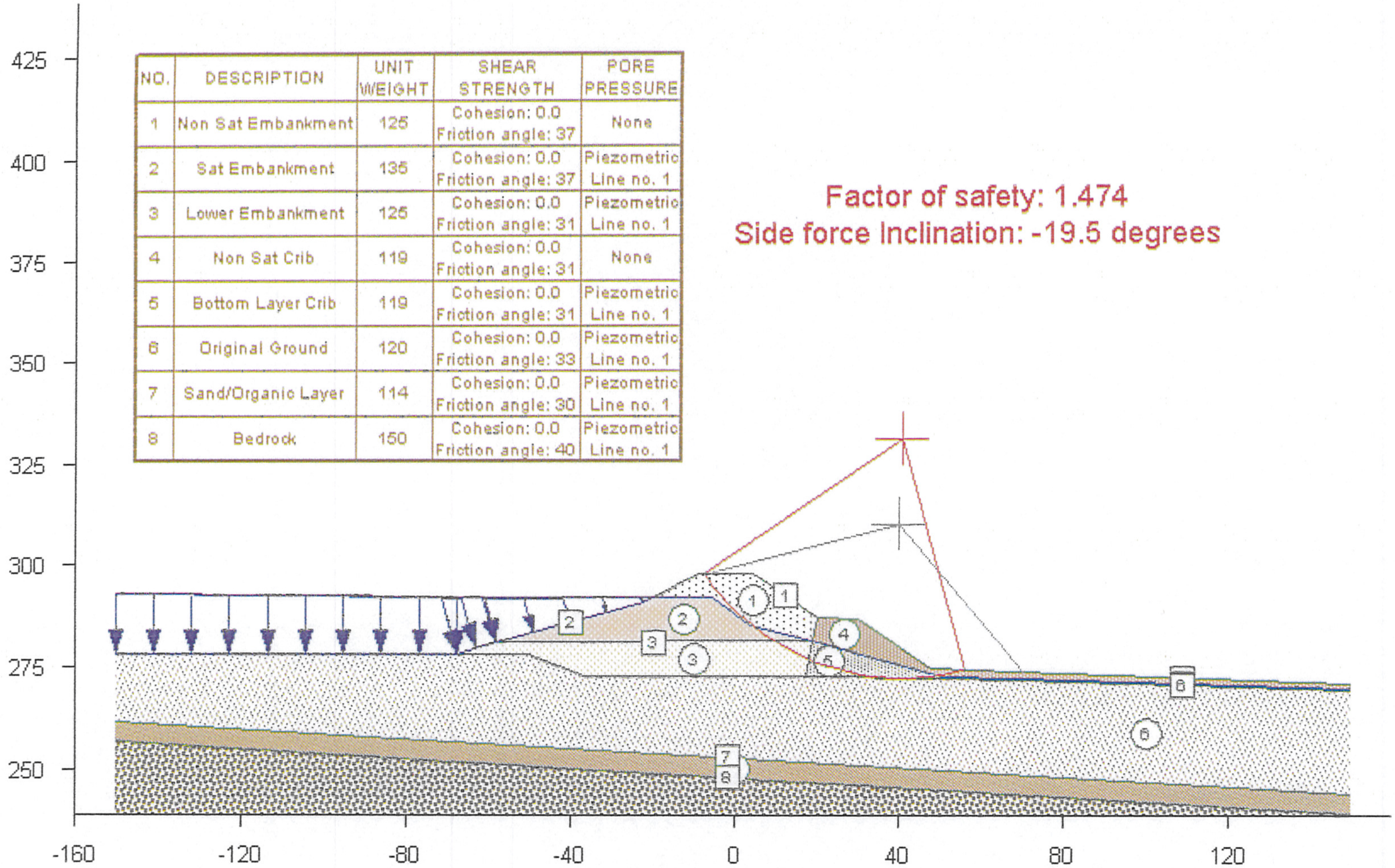
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	130	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	120	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
4	Crib Dam Silty Sands	115	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Sat Foundation Gravels	130	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Final OBE – Upper Dam



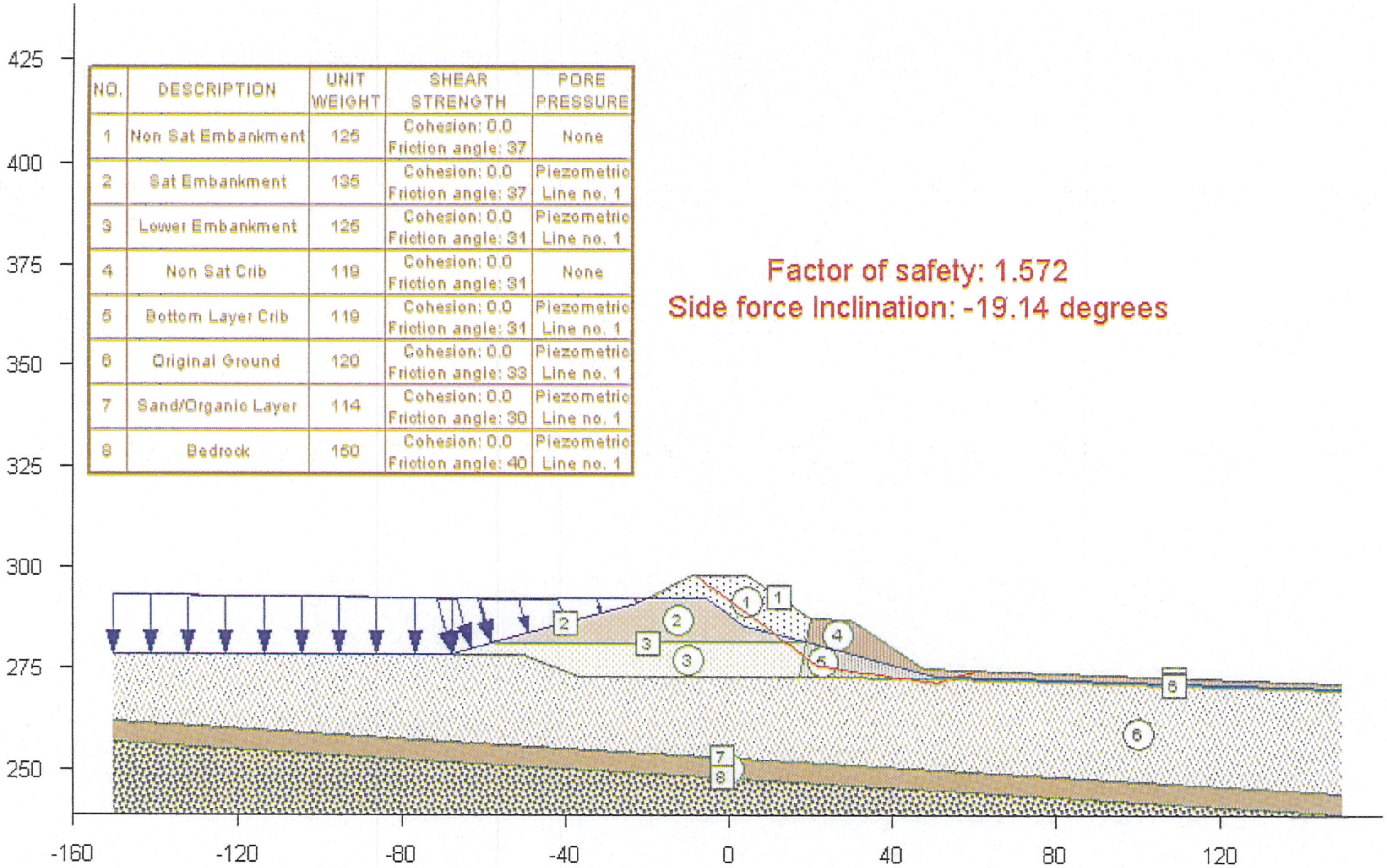
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Embankment Sat	130	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
2	Embankment Downstream	125	Cohesion: 0.0 Friction angle: 37	None
3	Sat Foundation Silty Sands	120	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
4	Crib Dam Silty Sands	115	Cohesion: 0.0 Friction angle: 22.5	Piezometric Line no. 1
5	Sat Foundation Sands	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Sat Foundation Gravels	130	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
7	bedrock	130	Cohesion: 10000.0 Friction angle: 60	Piezometric Line no. 1

Final OBE – Upper Dam



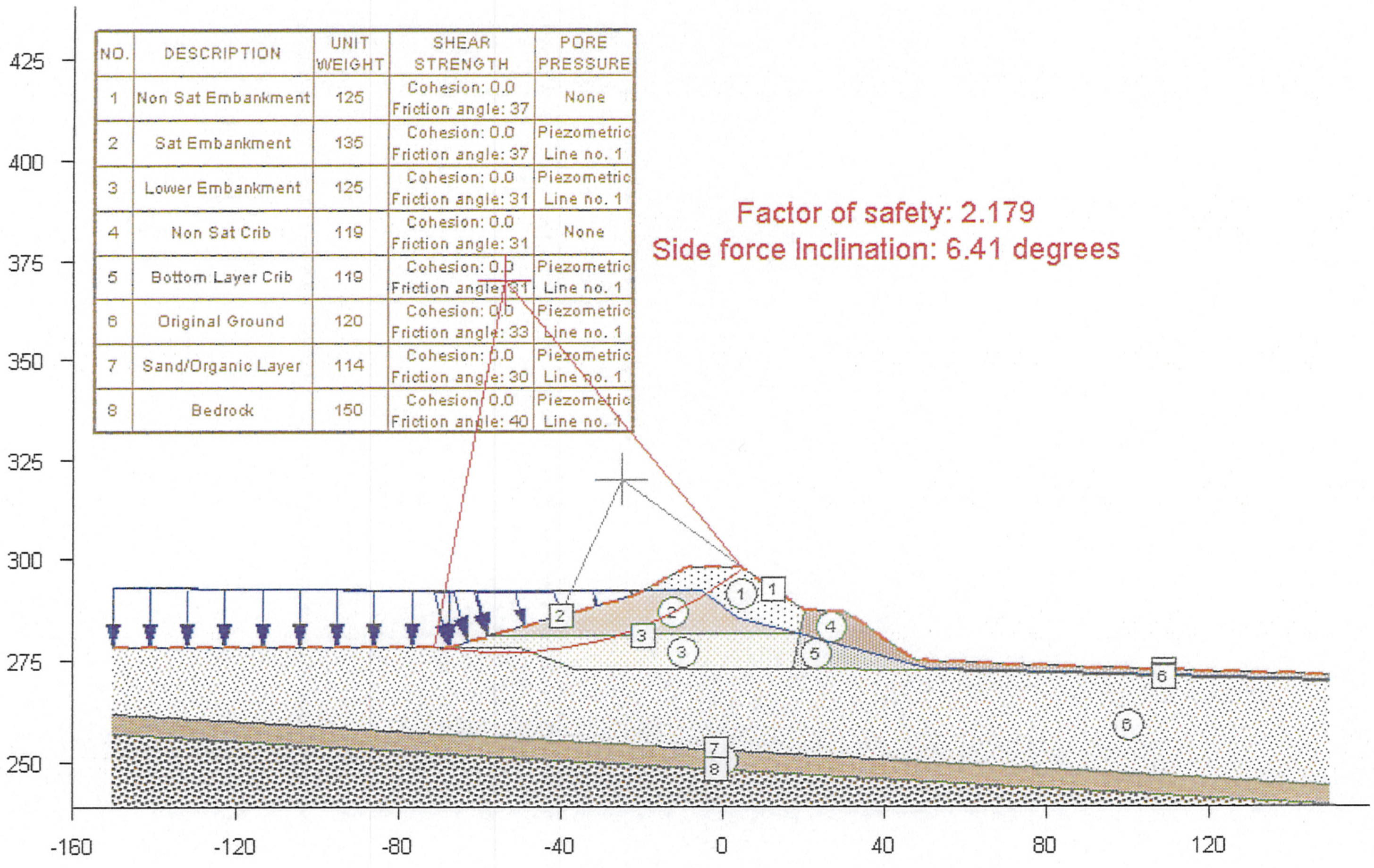
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1

Existing – Lower Dam



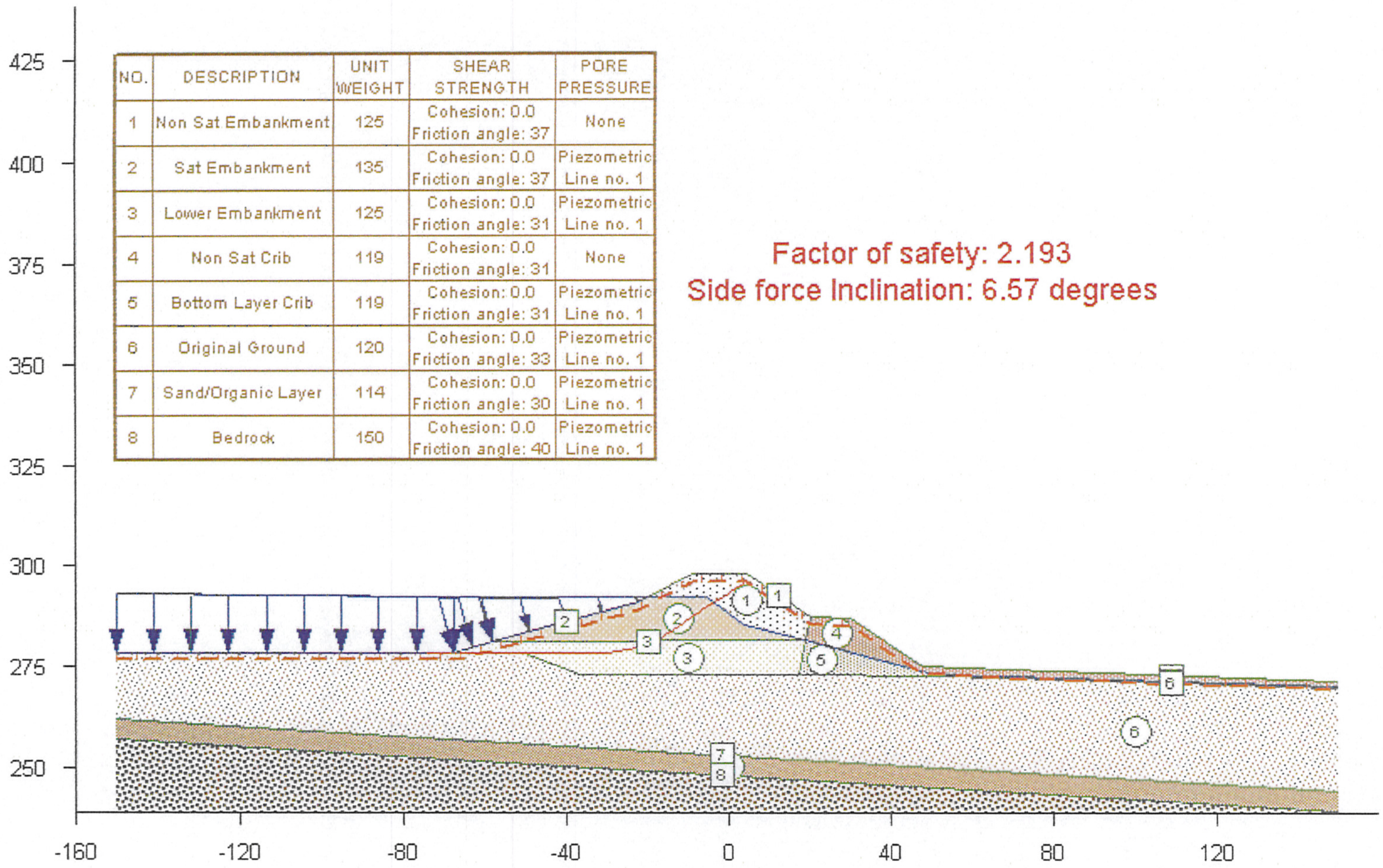
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1

Existing – Lower Dam



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1

Existing – Lower Dam

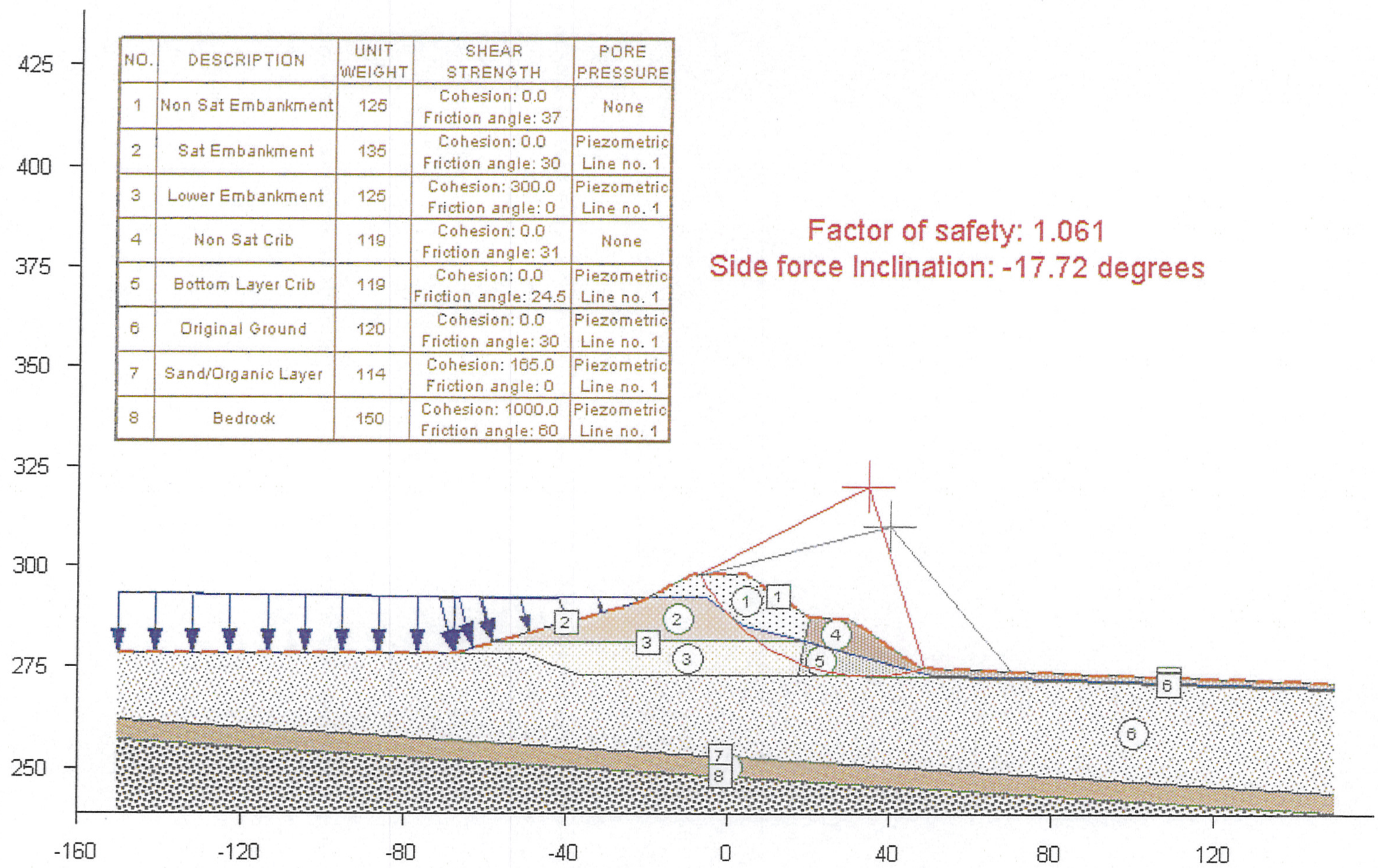


NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 37	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 33	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1

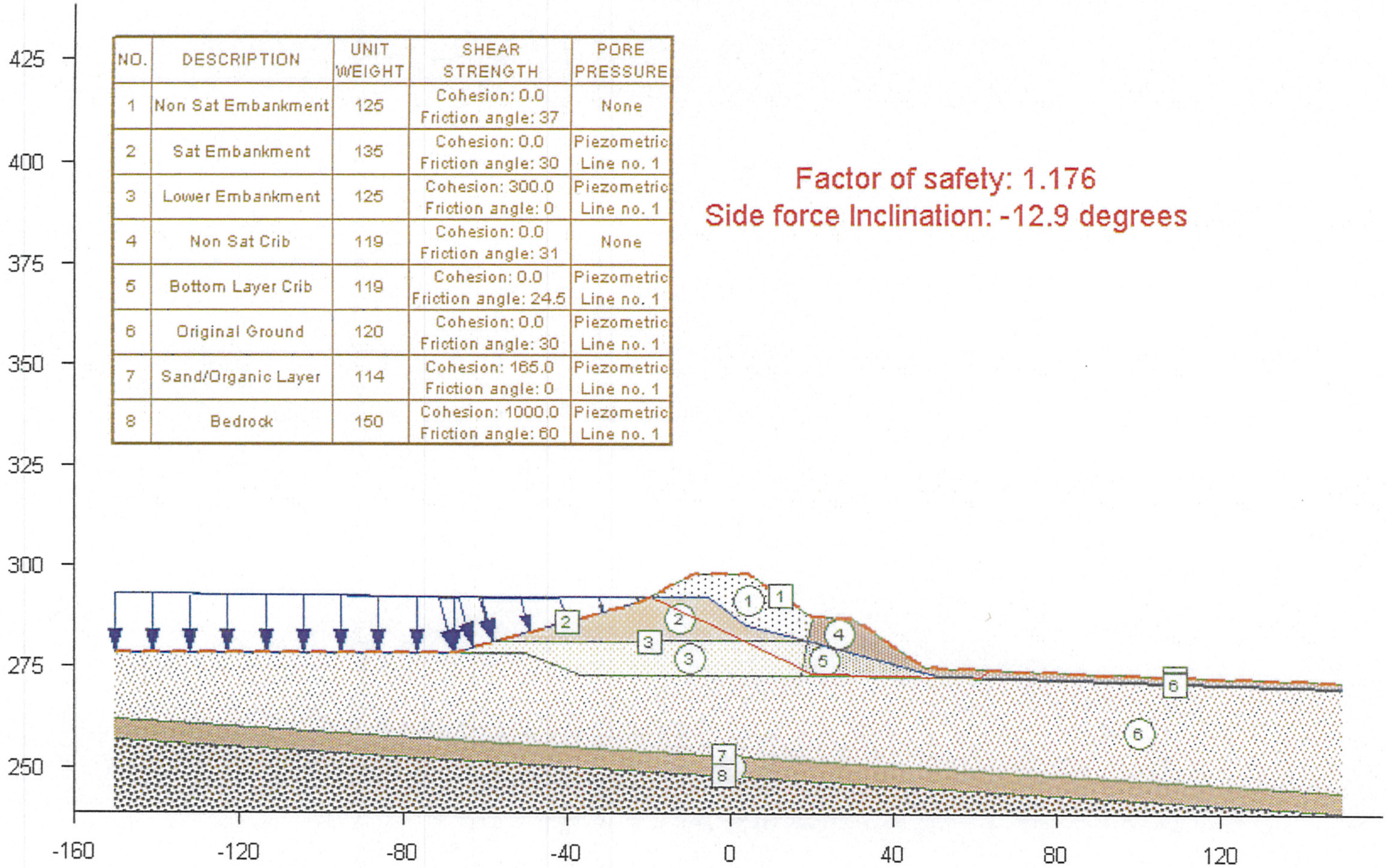
Factor of safety: 2.193
Side force Inclination: 6.57 degrees

Existing – Lower Dam

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 300.0 Friction angle: 0	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 24.5	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 165.0 Friction angle: 0	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 1000.0 Friction angle: 60	Piezometric Line no. 1

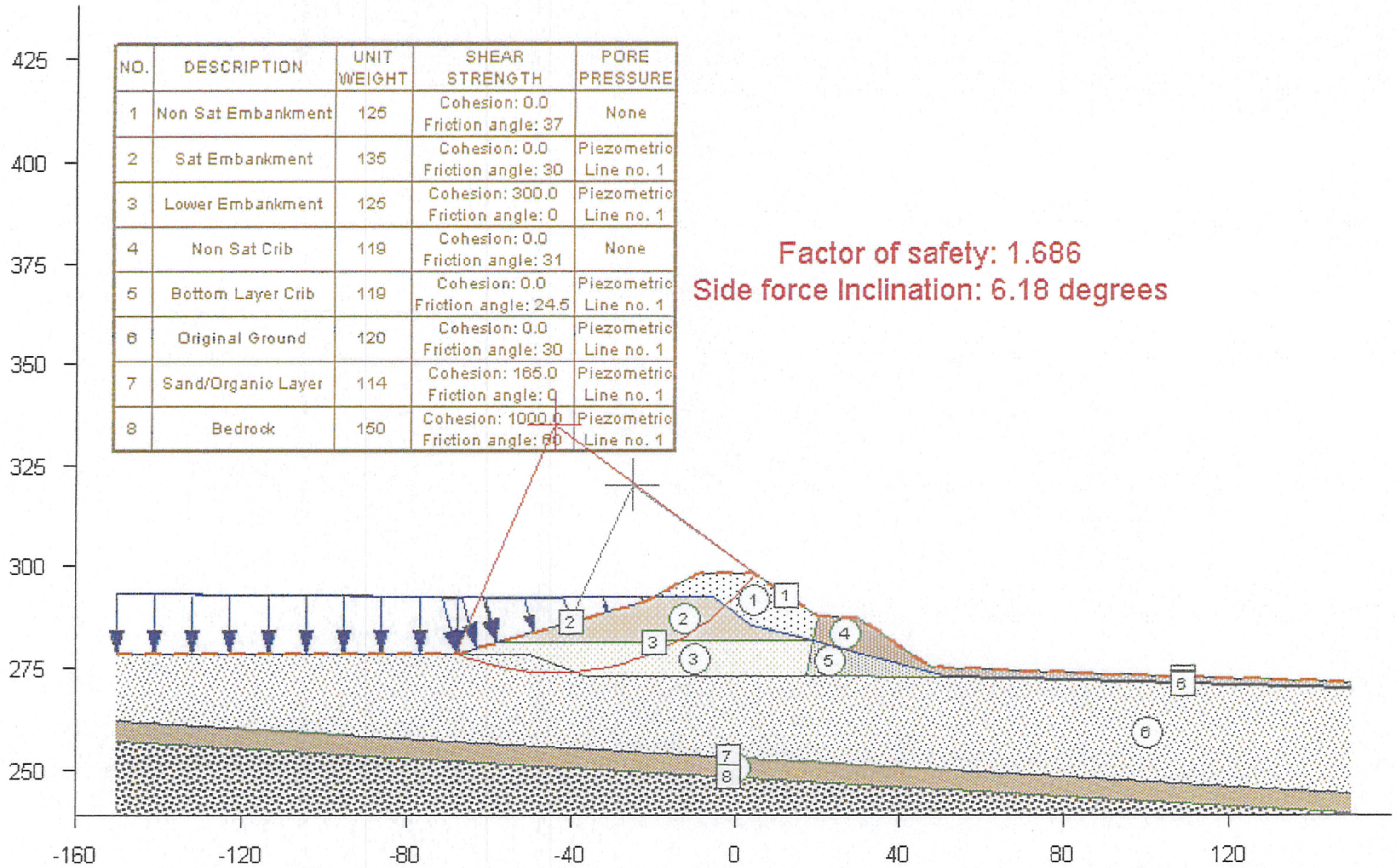


MCE – Lower Dam



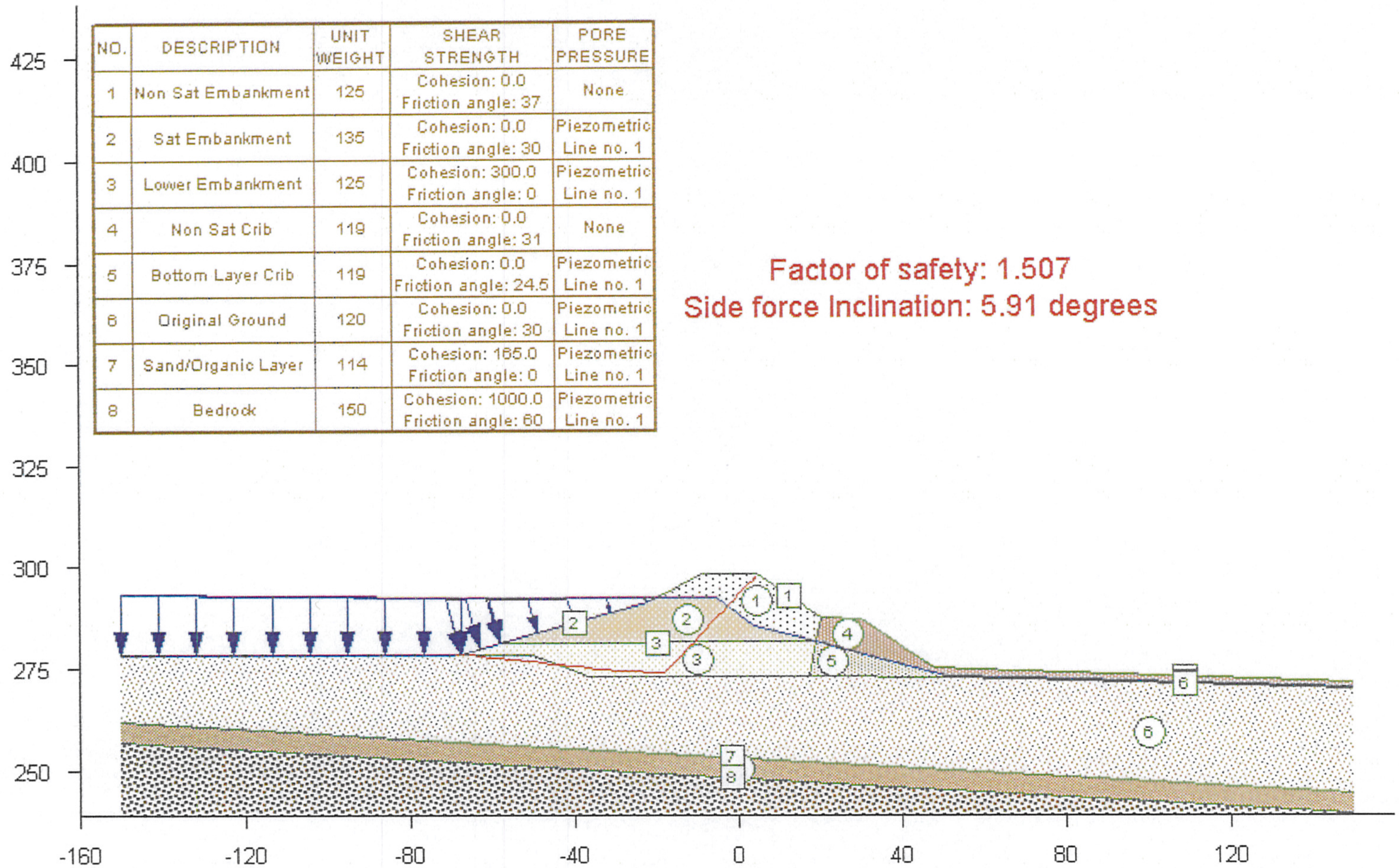
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 300.0 Friction angle: 0	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 24.5	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 165.0 Friction angle: 0	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 1000.0 Friction angle: 60	Piezometric Line no. 1

MCE – Lower Dam

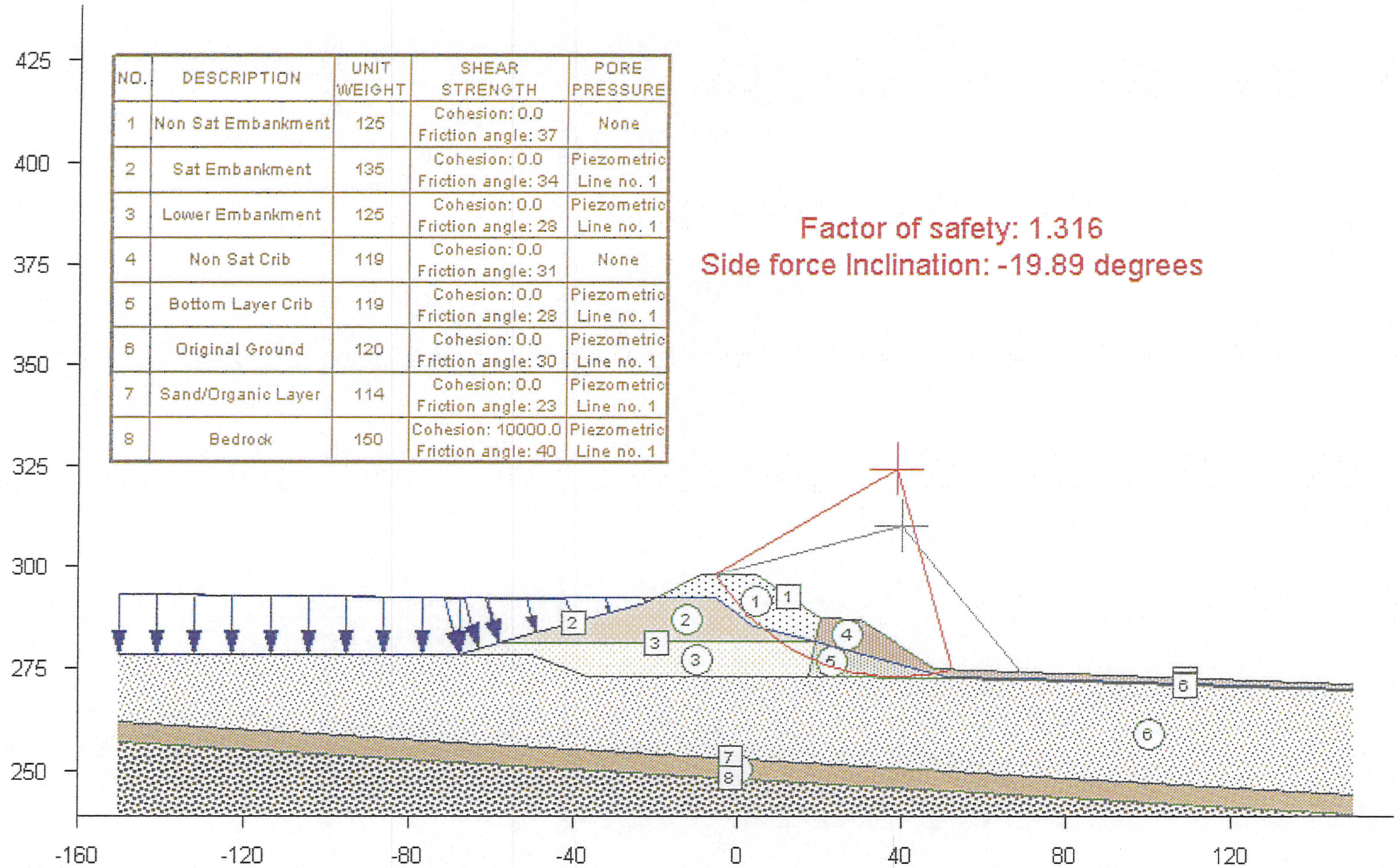


NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 300.0 Friction angle: 0	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 24.5	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 165.0 Friction angle: 0	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 1000.0 Friction angle: 60	Piezometric Line no. 1

MCE – Lower Dam

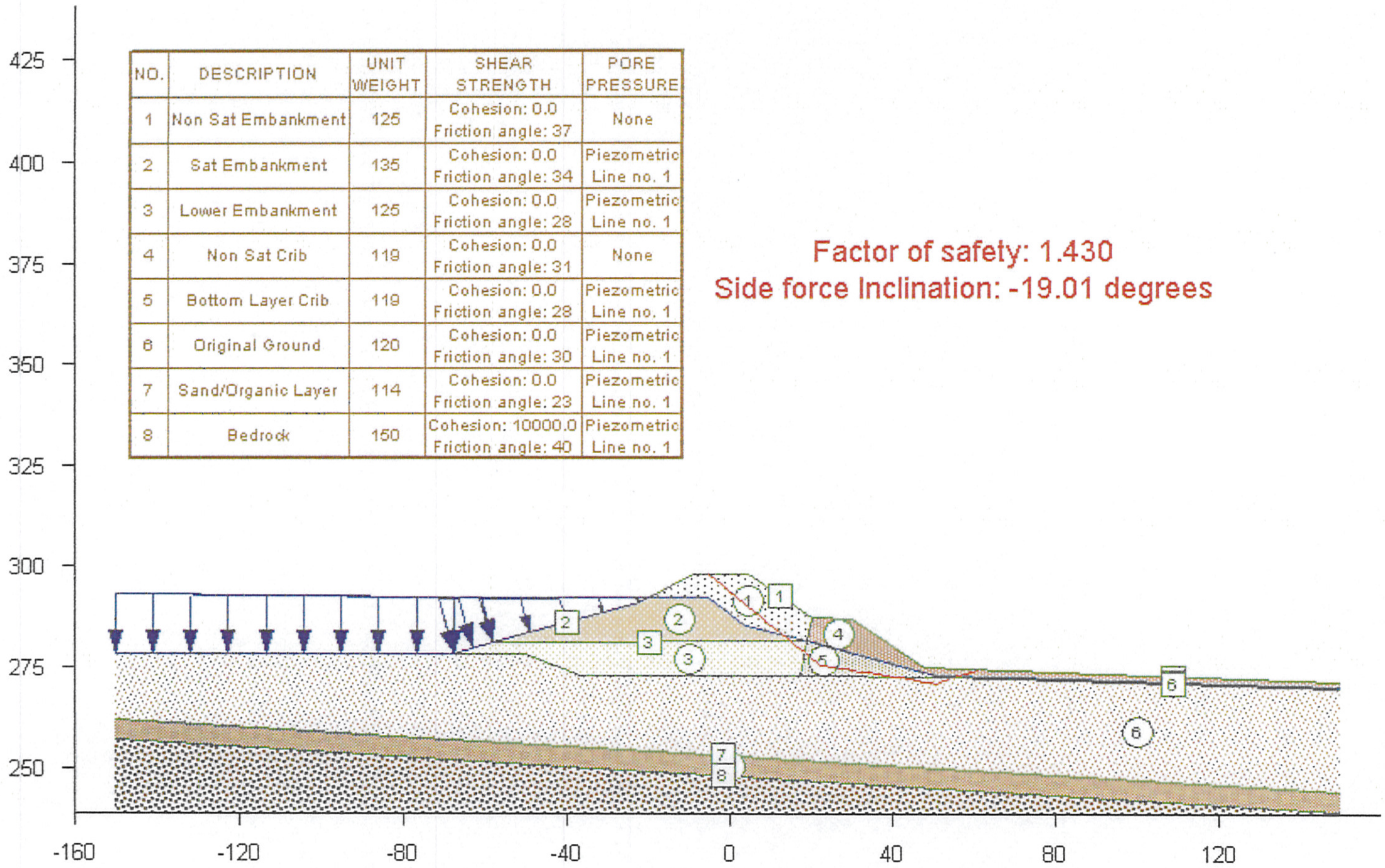


MCE – Lower Dam



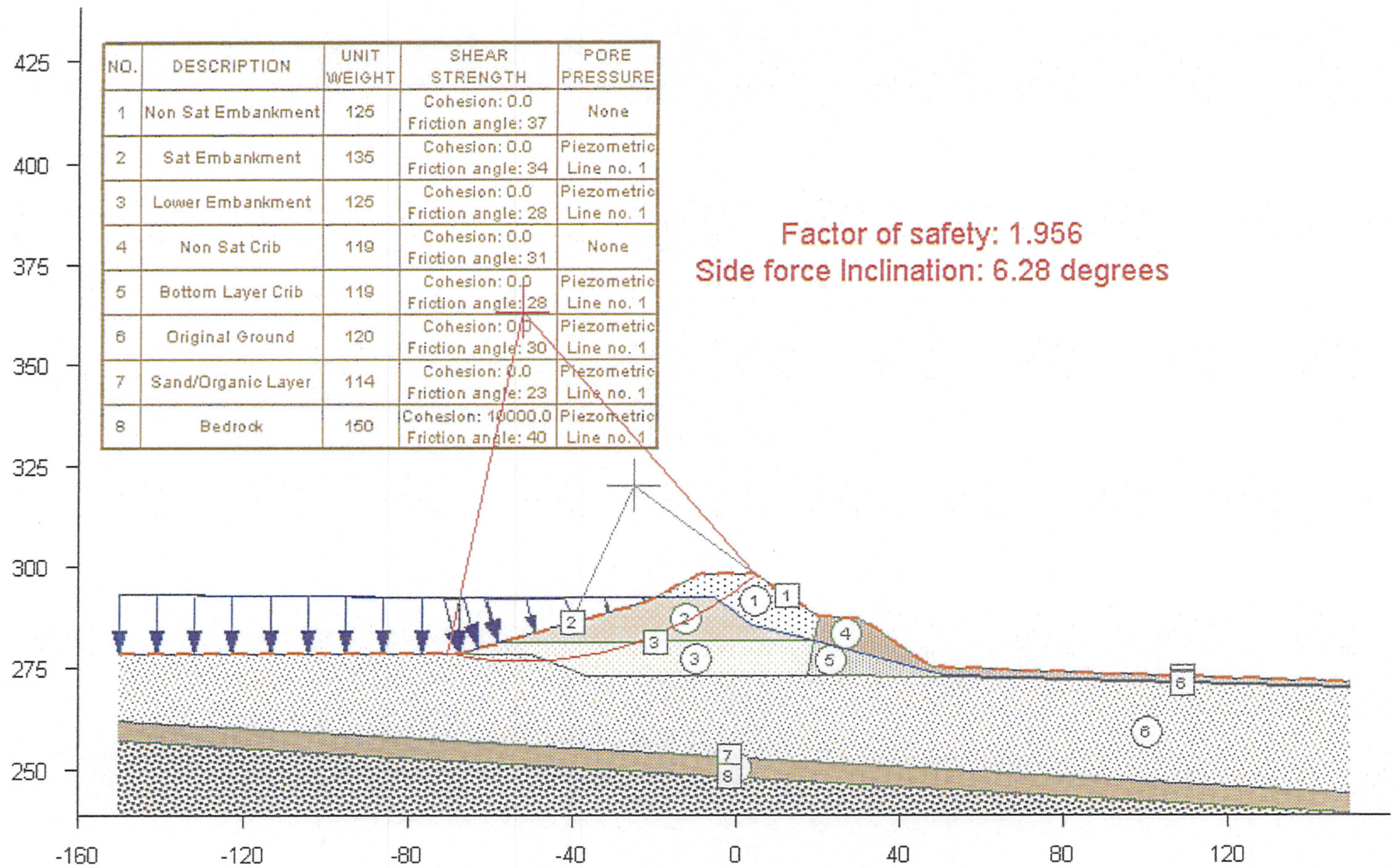
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 23	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 10000.0 Friction angle: 40	Piezometric Line no. 1

OBE – Lower Dam



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 23	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 10000.0 Friction angle: 40	Piezometric Line no. 1

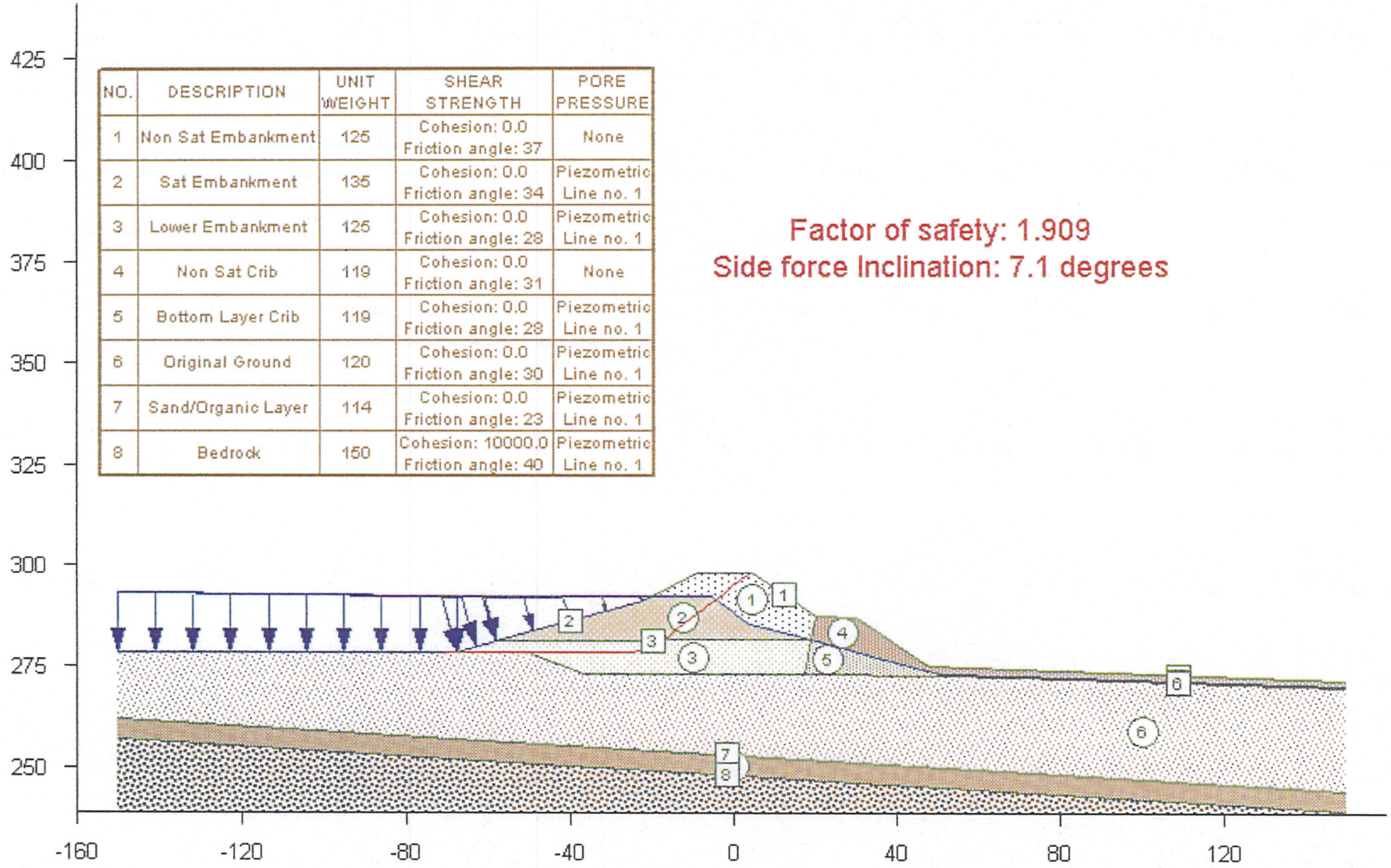
OBE – Lower Dam



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 23	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 10000.0 Friction angle: 40	Piezometric Line no. 1

OBE – Lower Dam

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Non Sat Embankment	125	Cohesion: 0.0 Friction angle: 37	None
2	Sat Embankment	135	Cohesion: 0.0 Friction angle: 34	Piezometric Line no. 1
3	Lower Embankment	125	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
4	Non Sat Crib	119	Cohesion: 0.0 Friction angle: 31	None
5	Bottom Layer Crib	119	Cohesion: 0.0 Friction angle: 28	Piezometric Line no. 1
6	Original Ground	120	Cohesion: 0.0 Friction angle: 30	Piezometric Line no. 1
7	Sand/Organic Layer	114	Cohesion: 0.0 Friction angle: 23	Piezometric Line no. 1
8	Bedrock	150	Cohesion: 10000.0 Friction angle: 40	Piezometric Line no. 1



OBE – Lower Dam

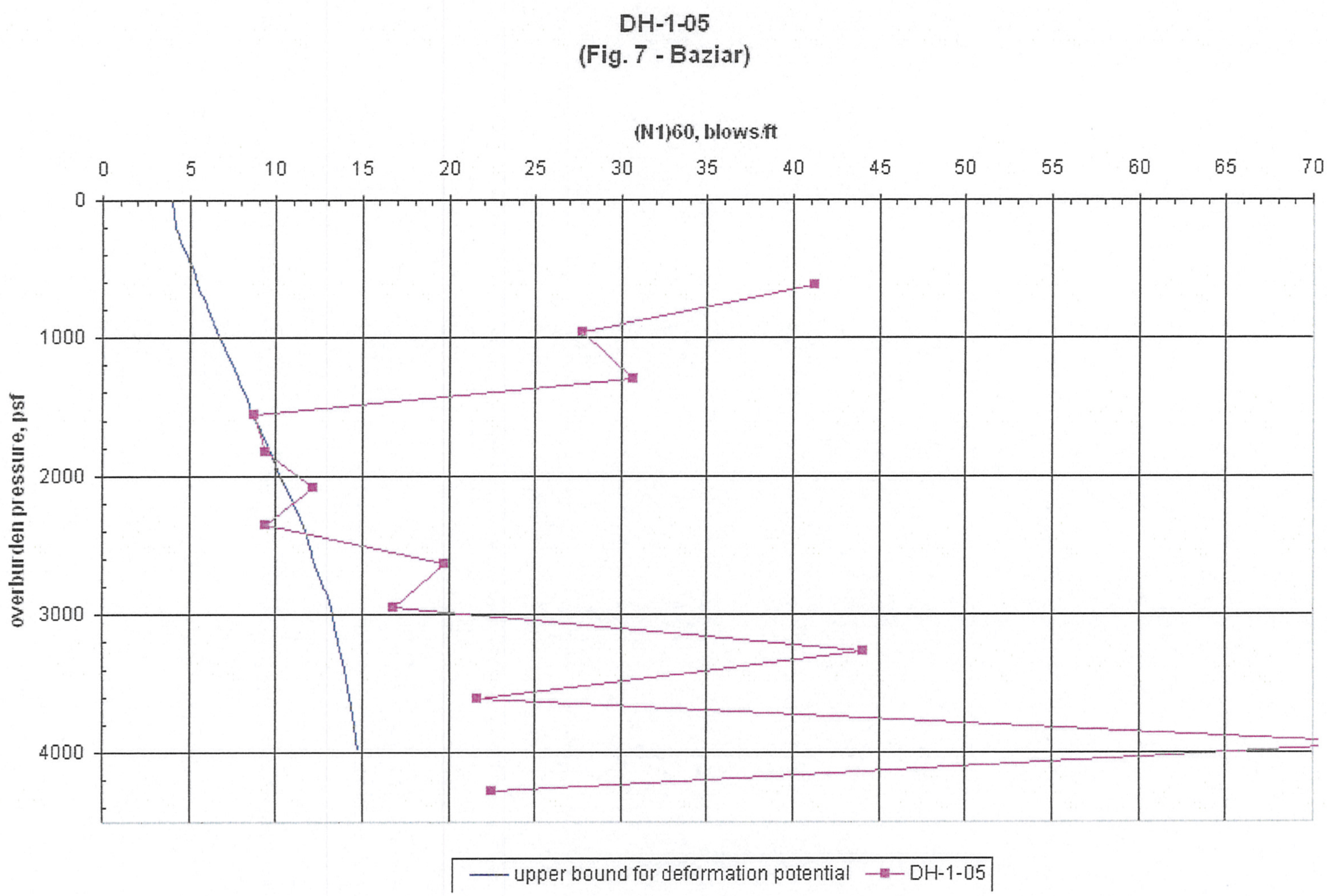


Figure C-19: Upper Wrangell Dam – Large Deformation Potential (DH-1-05)

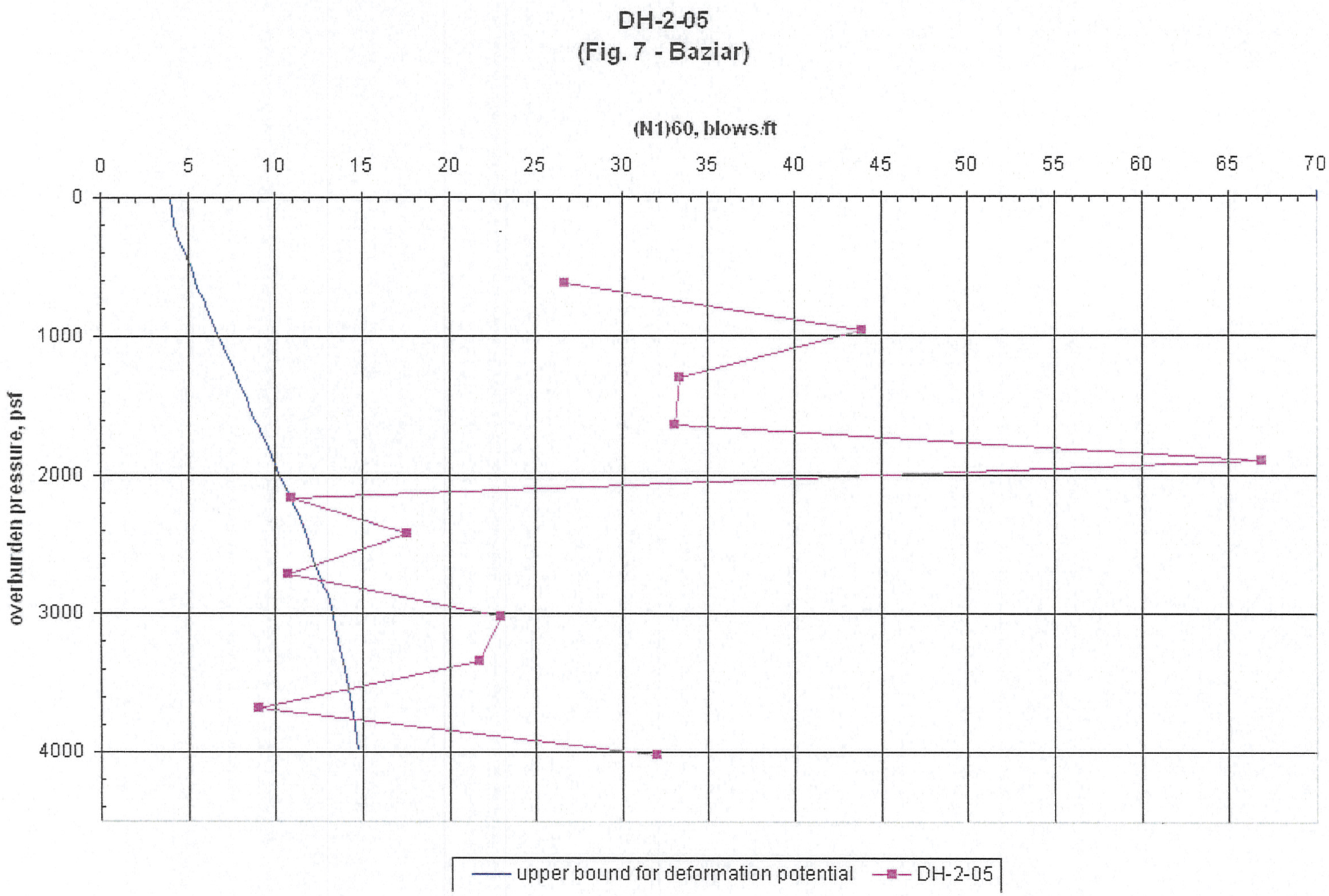


Figure C-20: Upper Wrangell Dam – Large Deformation Potential (DH-2-05)

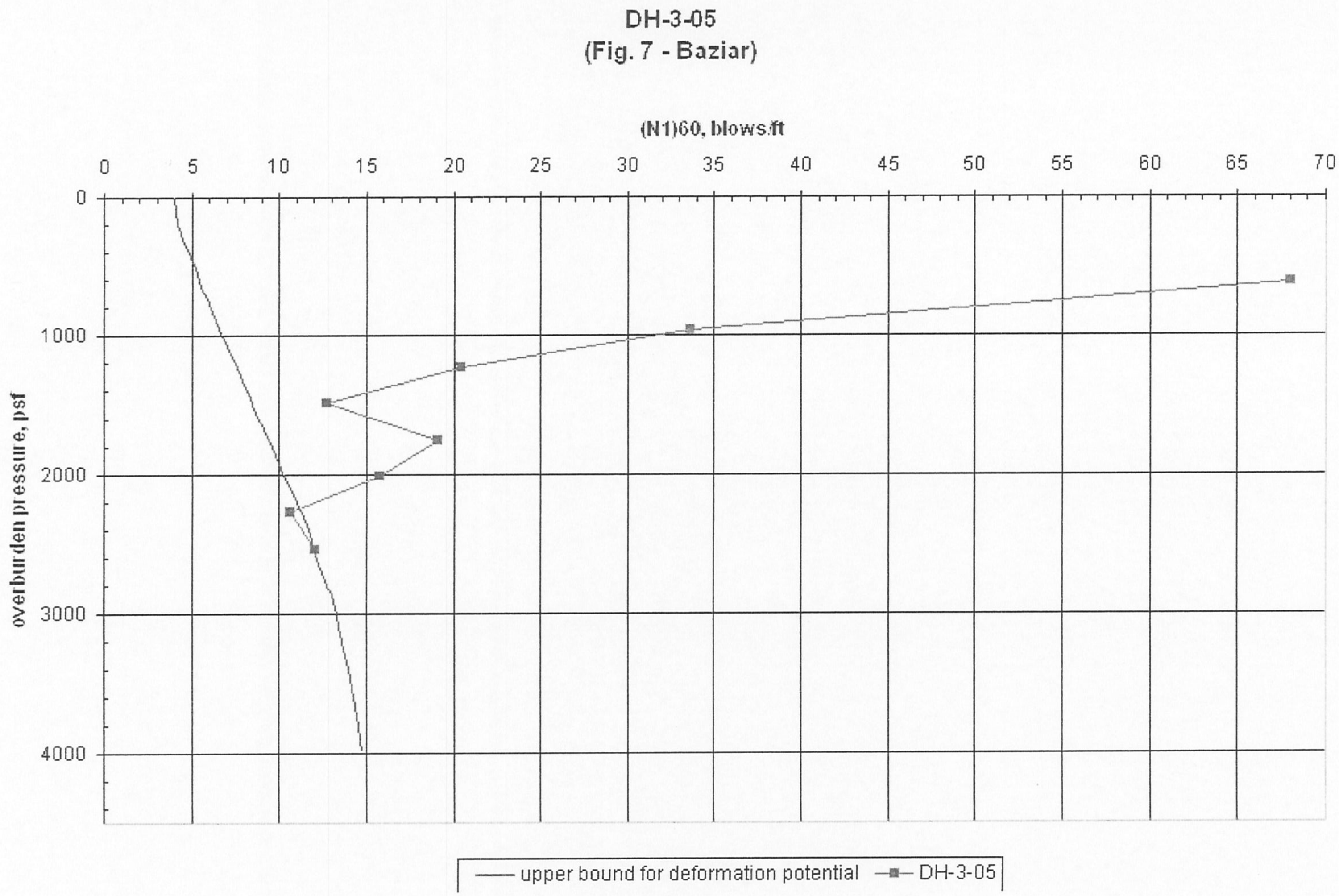


Figure C-21: Upper Wrangell Dam – Large Deformation Potential (DH-3-05)

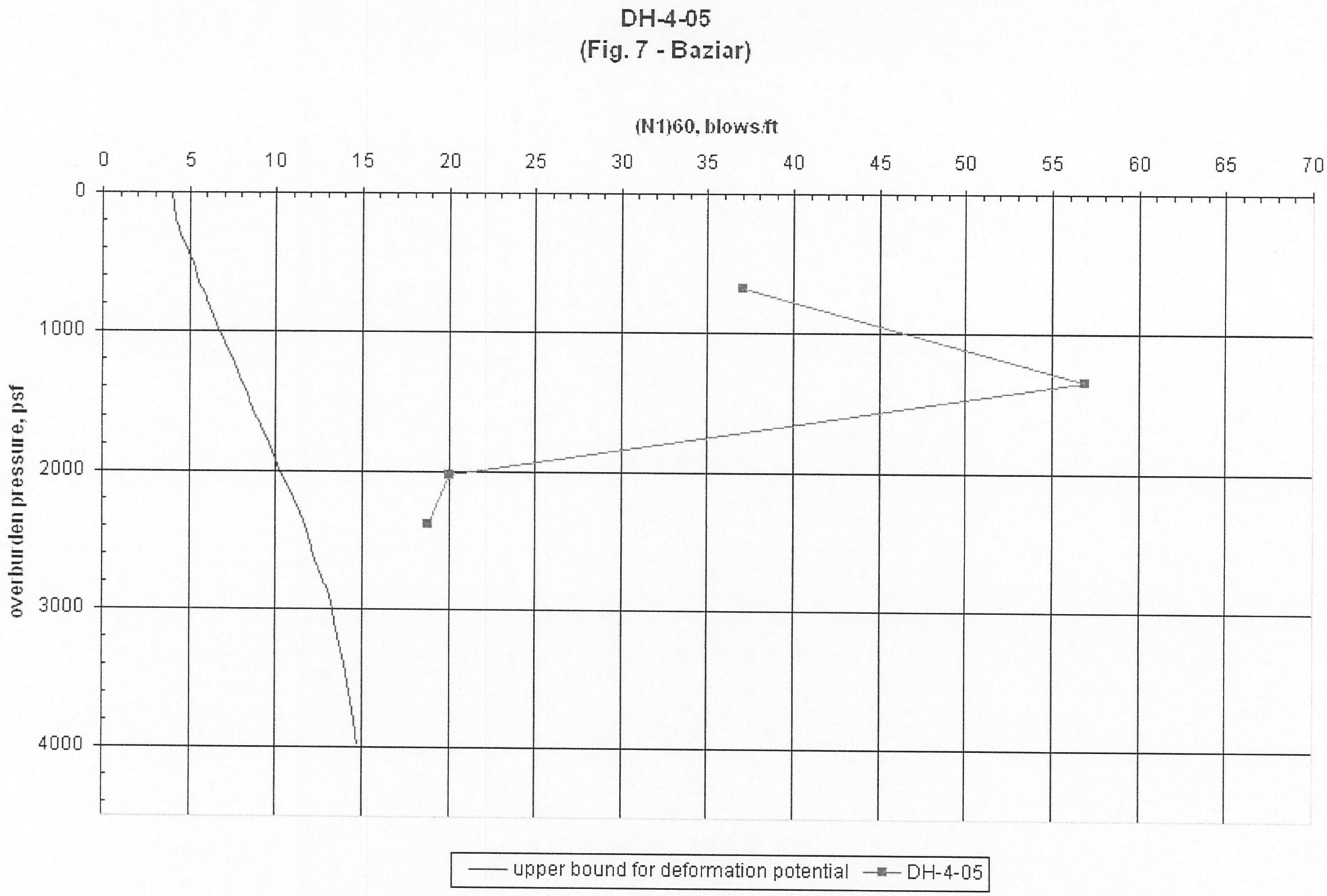


Figure C-22: Lower Wrangell Dam – Large Deformation Potential (DH-4-05)

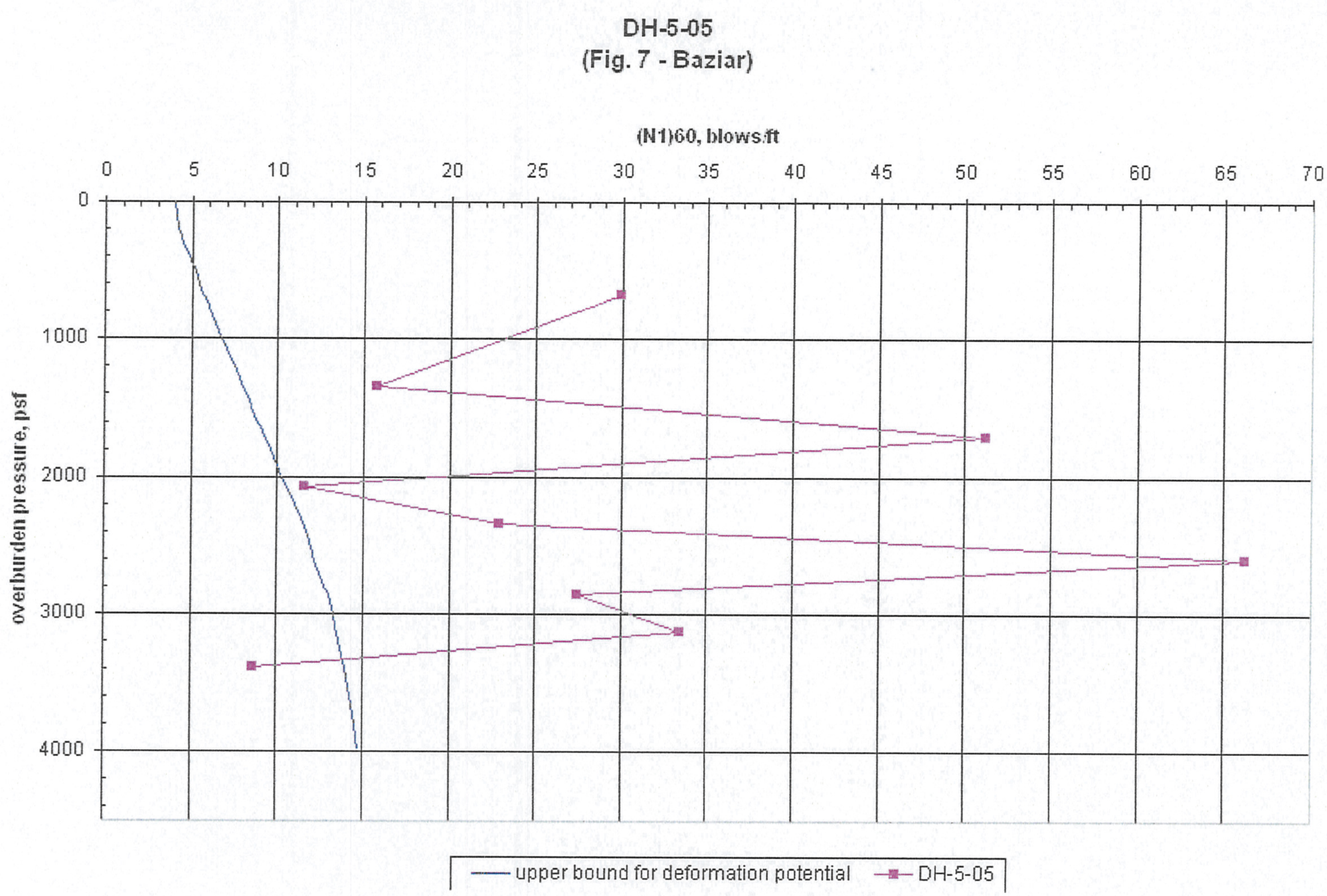


Figure C-23: Lower Wrangell Dam – Large Deformation Potential (DH-5-05)

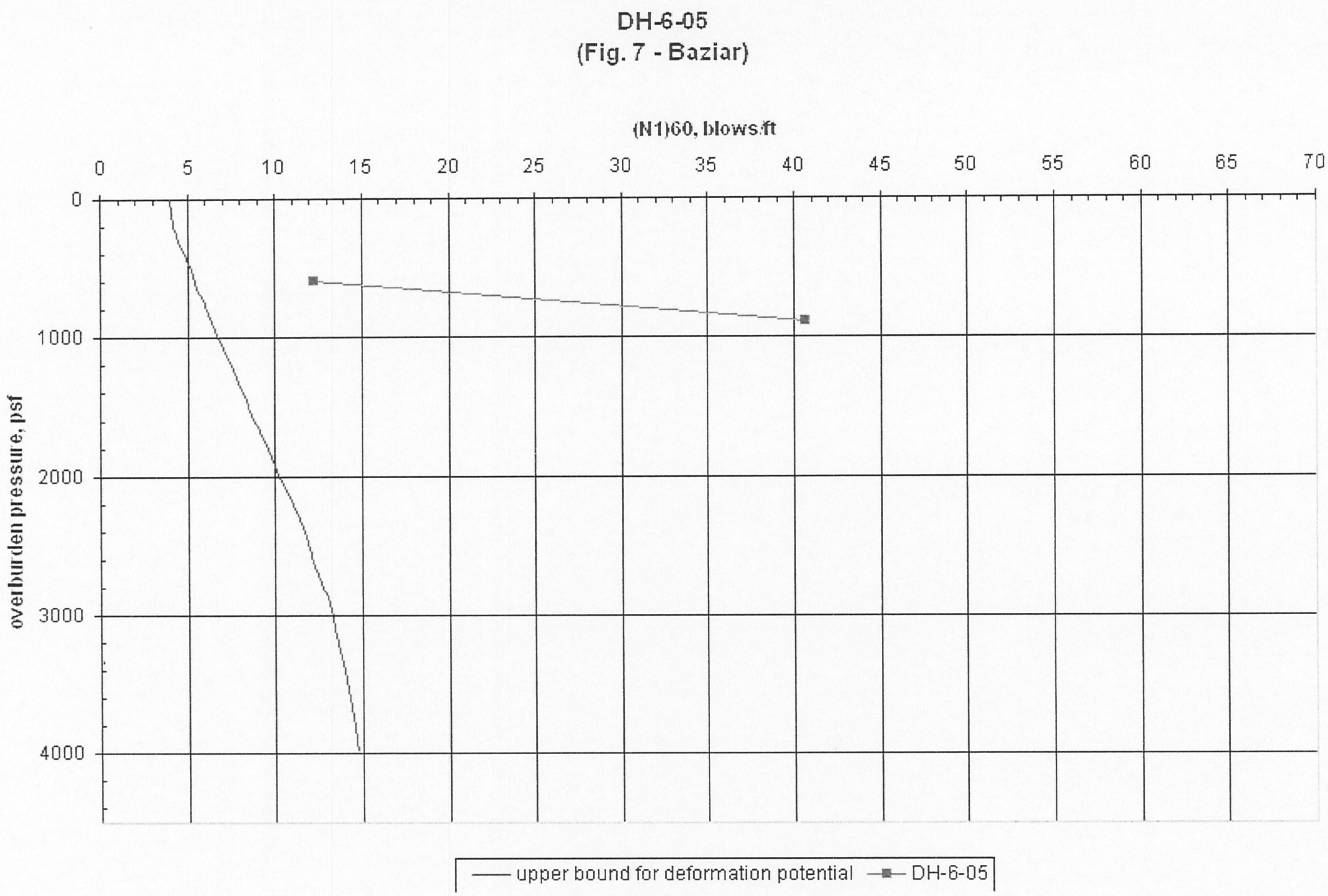


Figure C-24: Lower Wrangell Dam – Large Deformation Potential (DH-6-05)

Appendix D

Review Comments

and

Responses

Upper and Lower Wrangell Dams - Seismic Stability Report **Review Comments / Responses**

15-Mar-06

	Comments by Charlie Cobb	Response:
1	Please include a soil legend or make consistent with USCS terminology	Gibbons - Logs have been corrected to comply with USCS terminology.
2	The report initially states that the soil strengths from S&W, 1993 were "appropriate.. for this program", but then uses substantially different values for the static analysis without much discussion in Section and Table 7.1. Some additional detail or reference to the changes may be warranted	Gibbons - See response to Mierzejewski's Comment No. 4.
3	There was a M6.8 earthquake on the Queen Charlotte Fault on June 28, 2004 not mentioned as a "major" earthquake in Section 5.4 (maybe because its less than 7?).	Harrison - I listed several earthquakes that were very large and felt at Wrangell the list is not comprehensive. I assume there have been many M6 earthquakes on the Queen Charlotte Fault.
4	The report should reference a Maximum Design Earthquake (MDE) with respect to Maximum Credible Earthquake (MCE) to be consistent with ER 1110-2-1806; i.e., the MDE is the MCE	Harrison - For critical features the MDE is the same as the MCE. I will make that comment in the report.
5	If the MCE that produces a 0.23g ground motion is really an appropriate value, the MDE may be based on some proportion of the MCE in accordance with ER *1806, if the dams are not a Class I (high) hazard potential to the downstream development.	Gibbons - Correct, but until we perform a Dam Break analysis to determine the hazard classification it is assumed that the hazard classification is High due to the presence of residences in the down stream area. At this time we do not expect to perform a Dam Break analysis until a preferred alternative is selected from the Recon Report. Once an alternative is selected (i.e. new dam, remediate, etc) then a full analysis will be performed.
6	There are some conflicting statements on the critical nature of the structure; e.g., "The Wrangell site is a non-critical structure..." (p.18), and the hazard classification described as "significant due to...disruption of essential services" (Section 5.2). These conflicts may be creeping in due to inconsistency between the reference documents.	Harrison - From my evaluation of the site the dam fit the hazard potential of significant as defined in the Corps ER 1110-2-1806, Appendix B. I believe the state defines it differently and the state's evaluation is also included in the report.

	Comments by Marcus Palmer	**Response:**
1	Section 5.4, paragraph 1 and Section 5.5, paragraph 1 both refer to the "attached plate". I believe that they both refer to figure 5.1. Suggest that both be changed to reference figure 5.1.	Gibbons - Done.
2	On page 18 – fix typo, the 2500 year event should be 2% (not 10%) probability of exceedance in 50 years.	Gibbons - Done.
3	I reviewed your report and the report completed by Shannon and Wilson cited in your report. You addressed briefly the differences in assumptions for earthquake loadings; the loadings you chose are certainly more conservative. This more conservative approach is probably warranted in light of the NEHRP USGS map having very close acceleration contours in the area. ER 1110-2-1806 also states that the MCE is determined from the DSHA (Deterministic Seismic Hazard Analysis) and that the MDE is the same as the MCE for "critical features". It should be clarified if the analysis is performed using the MCE, MDE or if they are the same.	Agreed. In this case the MCE and MDE were considered to be the same. Mr. Cobb also commented on this subject. A further analysis would need to be performed to determine the critical nature and downstream hazards in order to determine if the MDE can be taken as less than the MCE. This would be performed during the feasibility and design phases of the project.
4	My biggest question is how critical these structures are. These structures are either in Seismic Use Group III or II depending on if the water is necessary to maintain pressure for local fire suppression according to FEMA 369. In addition, the Shannon and Wilson report doesn't seem to adequately address the downstream effects of a catastrophic failure. The hazard potential classification in ER 1110-2-1806 is dependent on these criteria as well. It should be clarified if the dams are critical in the sense of necessary for fire suppression, drinking water, and the downstream hazards (perhaps this is outside the scope and will be discussed later)	Gibbons - at present this is outside the scope, but will need to be addressed prior to finalizing an option for either remediation or removal of the dams.
5	The symbols and classifications do not conform to ASTM D2487 and ASTM D2488 standards. Every sample listed in the laboratory summary classifies as SM according to ASTM D2487. Further, all should be described as silty SAND with gravel except for DH-1, S-4 which should be described as silty SAND. Most of the tested samples are incorrectly labeled on the logs. It is also suggested that samples tested be reflected on the logs by adding the corresponding percentages of gravel, sand, and fines.	Gibbons - Logs have been corrected and lab data has been added.
6	ASTM D2488 does allow for "borderline symbols", but this provision has not been applied in the logs in accordance with the ASTM. It is suggested that ASTM D2488, Appendix X3 be consulted and followed for any soils that are near a border in classifications.	Gibbons - Concur. Logs are modified.

	Comments by Dave Mierzejewski	Response:
1.	Page 19, para.5.7 Report should explain in some detail why the values of ground motion from the seismic hazard maps of Alaska are to low and why values resulting from an earthquake on an inactive fault closer to Wrangell are more appropriate	Harrison - The ground motions are admittedly conservative. I am not comfortable using the Hazard Map values given the very limited amount of data they are based on in the vicinity of Wrangell. The nearest seismic station is in Juneau and there has been very limited geologic mapping done in the vicinity of Wrangell. While Pleistocene movement has been recorded on the CRML near Juneau, the CRML is not considered for the seismic hazard maps. Also, the CRML is of the same orientation and size as the Fairweather/Queen Charlotte Fault. Using the CRML as the source was a judgment call and is not necessarily the last word on the groundmotion. If Alaska District would prefer we use USGS Hazard Maps for the ground motion value we can.
2	What would be the results of the stability analysis if the Seismic Hazard Maps of Alaska data were used	Seismic Hazard Maps for Alaska indicated that the Maximum Credible Earthquake (MCE) would be the 2500-year event at 0.10-g. This is the same ground motion used in our analysis as the OBE. The results using the hazard maps and the OBE are the same. A comment has been added in the stability analysis section discussing this and noting the results using the Hazard Map values.
3	What sources of ground motion data does the AK. dam safety recommend?	Harrison - I contacted Charles Cobb the State Dam Safety Engineer early on in the study. He referenced: Chapter 6, Section 6.3 of the "Guidelines for Cooperation with the Alaska Dam Safety Program". These guidelines allow for the use of either probabilistic or deterministic methods and allow for and reference the USGS Hazard Maps. To my understanding the state did not make any recommendations. If the Corps were building a new Project at this site a deterministic study would be performed.
4	Page 29, table 7.1 Upper Dam. Why are the "assumed soil properties" in the table so different then the "Shannon & Wilson" properties?	Gibbons - One of the properties listed for Shannon & Wilson was a typo: specifically the phi angle of 21 degrees should have been 29 degrees. The new Table with comments follows:

Upper Dam Wrangell

Material	Description	Assumed Properties γ	ϕ	Shannon & Wilson γ	ϕ
Embankment	medium to dense Silty Sand with Gravels (SM)	130	37	135	39 *
Sat Embankment	" "	134	37	----	----
Foundation Silts	Silty Sands, loose to med, some gravels, (SM)	118	29	114 **	30
Crib Dam Silts	Fill material btwn the wood crib. Silty Sands, loose to med, sm gravels, (SM) sm wood chunks throughout	115	29	114	29
Foundation Sands and Gravels	Med dense Silty Sands and Gravels (SM and GM)	125	31	114	30
Foundation SM and GM	Dense to med dense Sand, Gravel. Some organic layers just above rock.	130	34	Lower Failure Boundary	

* Shannon and Wilson report indicated higher blow counts in this material. This report and this strength value only reflects the 2005 soils testing.

** Shannon and Wilson assumed that 25% of this soils mass was comprised of wood with a unit weight of 70 pcf. This report assumed the material was SM.

5	See comment 4: What would be the result of the stability analysis if the Shannon & Wilson properties were used?	Gibbons - I used the Shannon and Wilson (S&W) values with the Corps cross section at Sta 1+20. The S&W stability analyses were taken at approximately Sta 80+00 and Sta 1+50. So, the Corps cross section is in between these two locations. The results are in the following table. The values are similar for both the analysis using the Corps cross section and the S&W results from 1993. Note that in some cases the failure plane had to be restricted in order to develop the "deep failure/breach" condition that S&W used in their report.

Factor of Safety Comparison		**Upper Dam**		**Shannon and Wilson**	
LOCATION	FAILURE TYPE	FACTOR OF SAFETY	Comment	FACTOR OF SAFETY	Comment
Downstream	circular	1.746	Deep Failure/breach	1.720	Deep Failure/breach
Downstream	circular	1.479	Mid-Crest Failure		
Downstream	non-circular	1.574	Deep Failure/breach	1.560	Deep Failure/breach
Upstream	circular	1.890	Mid-Crest Failure	1.790	Deep Failure/breach
Upstream	non-circular	1.873	shallow failure	1.540	Shallow Failure

	Comments by Jeremy Britton	**Response:**
1	How did you decide what pool elevations to use for the analyses? Are the pool elevations in the analyses the same as those during drilling? On the upstream side, is it possible that the factor of safety is less if the pool is lower? There are competing factors: less support from the pool, but lower pre-earthquake pore pressures. Plus, perhaps the consequences of failure with a lower pool are less	Gibbons - Pool Elevations were the same as during drilling and are the conditions for the majority of the time. During dry summers the pool on both dams can be lower, but this is an "unusual condition". A quick change in the water level on the upper dam produced higher factors of safety so this condidition was not pursued further.
2	Table 7.1: Why is there a difference between the "Embankment" and "Sat Embankment" friction angles for the Upper Dam? Are the (N1)60 values different above and below the piezometric line?	There is not a significant change in blow counts in this material above and below the water table. The values have been changed and are now the same.
3	Why did you include a cohesion intercept for the cohesionless materials in the stability analyses? Were you trying to discourage shallow slip surfaces?	Cohesion intercept was to discourage shallow slip surfaces. The cohesion was removed and the limiting parameters in UTEXAS4 for the locations of both the center of the circle and the radius was used to prevent shallow failure surfaces. These numbers appear more valid.
4	Why were only non-circular slip surfaces examined for the upstream slopes, and not circular surfaces too? Shannon and Wilson had a much lower FS for the Upper Dam upstream slope using a circular surfac	The program was resulting in error messages using the circular surfaces. Input and analysis parameters were modified and the circular surfaces added to our analyses.

	Comments by Jeremy Britton (continued)	**Response:**
5	For the Lower Dam, I'm not sure where the four different embankment zones are located and why they have such different properties.	The four embankment conditions reflect the following: (1) the upper embankment and downstream 'dry' embankment, phi is based on blowcounts in the range of N=20; (2) the upstream material below the water table has lower blowcounts than the embankment on the downstream side of the cut off wall so a lower phi was assigned to this material (average equal to 17); (3) the bottom layer of the upstream embankment had average blowcounts of N= 7 hence the lower strength values (4) This layer is the same as the embankment (phi = 37) except that it is saturated.
6	Lower Dam - Looks like the friction angles were reduced due to liquefaction in the soils above the water table. Does this reduction in strength apply above the water table?	Gibbons - No, the materials above the water table should not have any strength reductions. These were changed for the Lower Dam - new values will be in the revised report.
7	In section 7.3 you describe how you reduced the strength of soils that do not liquefy but do build up excess pore pressure. This is the first time I've seen the Hynes-Griffin (1988) approach you used. Though I haven't seen the reference, I'm not sure about the method because it seems fundamentally wrong. The method reduces the friction angle when it's really the effective stress that is reduced when excess pore pressures build up. In other words, in the strength relationship $s = \sigma_n' \text{ Tan } \phi'$, the Hynes-Griffin approach reduces ϕ', but I think it's more fundamentally correct to reduce σ_n'.	The Hynes-Griffin equation has been used to reduce soil strengths in gravels and sands in the seismic analyses of Folsom Dam, Ririe Dam and Lucky Peak Dam. The reference is from the Folsom Dam analysis. Additionally, the equation was referenced in the Draft Seismic Stability of Earth and Rockfill Dams (EM 1110-2-6001, 1 Feb 1994). In the comments made on the Lucky Peak Dam analysis Dr. James K. Mitchell makes the following observations about the equation. "The use of a tan ϕ' function may be appropriate. However, use of the sin ϕ function is more conservative, with the differences in values of ϕ ranging from about 0.8 to 2.5 degrees less for the sine function than for the tangent function for a residual pore pressure ratio of Ru of 0.1 and ϕ ranging from 30 to 45."
		Per Frank Walberg, Kansas City District, while use of Ru is in accord with this, recent review shows that applying a constant pore pressure coefficient, say ru = 0.25, is incorrect. In UTEXAS this results in 75% of the effective strength parameter Tan ϕ being multiplied with the total normal stress, instead of the effective stress. Therefore, the calculated "reduced" strength is often greater than the whole effective strength corresponding to steady state seepage. A reduced effective strength applied to the non-liquefying saturated materials is appropriate and this can be done in UTEXAS by applying the reduced factor to Tan ϕ and ensuring the use of effective stress by making reference to the piezometric line. The non-liquefiable values were re-evaluated and both approaches were used. The calculated values for Ru were used and the Hynes-Griffin equation used to reduced the effective strength for calculated values of Ru above .25. For Ru less than or equal to .25 the effective strength was reduced using Tan ϕ. This provides a conservative approacl

	General Comments during Tele-Conference Review:	
	It was decided that the current report would be finalized by addressing the comments received including clarifying the classification assumptions and recommend the full deterministic evaluation. The next step would be to produce a recon style report by the end of July that would include costs to repair the existing structures, develop alternatives with costs that would remove, replace, repair, and increase the capacity of the water supply for the community.	Recommendation section of the report has been changed to indicate that without further study of both the dams and a Dam Breach Analysis it can not be said that the existing structures meet the seismic requirements of the State of Alaska. It is recommended that the City of Wrangell proceed on with an investigation into water supply alternatives to include remediation of the dams, construction of a new structure and alternative water supplies.