STABILITY STUDY
UPPER AND LOWER WATER SUPPLY DAMS
WRANGELL, ALASKA

May 1993

City of Wrangell
P.O. Box 531
Wrangell, Alaska 99929

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
5430 Fairbanks Street ▪ Suite 3
Anchorage, Alaska 99518
907 ▪ 561 ▪ 2120

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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FAIRBANKS
ANCHORAGE
SAINT LOUIS

May 20, 1993

City of Wrangell
P.O. Box 531
Wrangell, Alaska 99929

Attn: Mr. Jim Pung, City Engineer

RE: STABILITY STUDIES FOR WRANGELL UPPER AND LOWER DAMS, WRANGELL, ALASKA

Gentlemen:

We are pleased to submit herewith our stability study report for Wrangell's two water supply dams, AK-00013 and 00014. The purpose of this study was to further explore the subsurface conditions in and below both dams and to conduct stability studies of these dams to evaluate if they possess adequate static and dynamic stability to meet currently accepted design standards. To conform to US Forest Service requests and provide for efficient review, we have also attempted to discuss each significant parameter including our interpretation of each value as well as additional concerns that were raised during the course of our work.

In summary, we found that actual conditions within the dams do not always conform to the as-built information. As a result, considerable matching of information labeled "existing" on prior design documents had to be accomplished with actual conditions in order to develop reasonable sections for stability analyses. Even then exact boundary definitions were not always possible, forcing us to make some interpretations in order to complete the sections and perform the analysis. We learned from our field explorations and stability studies that the dams were not as well constructed as probably they could have been. While stable under static load conditions, they do not even closely meet current design standards under dynamic loading and, in fact, both could fail under a future strong earthquake. If, therefore, these dams are to continue to store water for the City's needs, repairs appear necessary to improve stability. The actual details and factors of safety for both dams are discussed and presented in the pages which follow.

We appreciate the opportunity to be of service to you. Your confidence in our firm is appreciated.

Sincerely,

SHANNON & WILSON, INC.

Fred R Brown

Fred R. Brown, P.E.
Vice President

5430 FAIRBANKS STREET • SUITE 3
ANCHORAGE, ALASKA 99518
907 • 561 • 2120 FAX 907 • 561 • 4483

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STABILITY STUDY
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1.0 INTRODUCTION

This report presents the results of static and dynamic stability studies of the Upper and Lower Water Supply dams in Wrangell, Alaska. The two dams, designated, Upper and Lower Dams, are situated on U.S. Forest Service land on a single drainage way southeast of the city and impound about 122 and 67 acre feet of water respectively. Attempts to retain water at these two locations date back as far as about 1900 and 1935 for the Lower and Upper dams respectively, with the construction of log crib structures. The upper log crib structure leaked badly after construction and did not retain water until it was modified in about 1958. Since initial construction, both dams were modified with new designs and raised by covering or partially covering these log structures with earthfill. 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 was done. Discrepancies between the design and as-built sheets and between as built data and newly excavated test pits in the toe area, led to further confusion and questions as to what the dams were comprised of and the nature of the foundation support materials. The unknowns and questions about overall stability led the City, with Forest Service urging, to conduct explorations at both dams and evaluate the stability of these dams. This report presents the results of these explorations and stability studies.

1.1 Previous Studies

Prior to the current studies, most of the information available about the dams has been generated from the above 1960's drawings and information compiled during our 1985 inspection of these dams. Much of this work relied heavily on the assumption that the limited "as built" information was correct and adequately depicted actual conditions within the dams.

In May 1992, the dams were inspected by the U.S. Forest Service as a 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. Two seepage collars with weirs were installed by the City to monitor flow. The Forest Service also pulled old file documents from their Archives which suggested even more discrepancies with the "as-built" documents. As the property owners, they additionally requested that the City have an engineer perform a safety inspection of the dams which was conducted by Shannon & Wilson in September 1992.

The current study is a follow-on study to our September safety inspection and the City's test pits, toe clearing, and weir installation efforts in October to address stability and toe seepage concerns. Our inspection report, dated November 1992, followed both the inspection, test pit and clearing efforts in order to incorporate these results into simplified stability calculations. In addition to a number of predefined deficiencies, the City's test pit studies revealed soft or loose foundation sediments in the toe area of the Upper Dam to depths greater than 8 feet. Based heavily on assumptions about the embankment materials and water levels in the dam, these studies also revealed static factors of safety of 1.2 to 1.4, values less than the 1.5 number used to define minimum acceptable stability for dams.

While the above inspection/simplified stability work was being completed, a second independent study was undertaken to develop an inundation map of the downstream channel identifying areas that would be affected should the Upper Dam fail, taking with it the Lower Dam. This analysis is presented in our October 2, 1992, report and was performed working with the best available topography, a 12 year old 1"=100 foot map with 5 foot contours. From this map and a more recent check of new inhabitants in the area, the Forest Service determined that about 13 homes fall in the inundation zone and re-classified the Upper Dam as high hazard, pending further studies. At that time, it was acknowledged that the contour map was based on aerial photo interpretation, may be inaccurate, and probably should be redone with a ground survey and new inundation map.

Based on the above information, more studies were needed as it was felt that the soft or loose materials in the toe of the Upper Dam may reflect organic stripping and that foundation conditions below most of the dam may be much better (i.e. resting on rock or firm glacial materials). This assumption was based heavily on nearby rock exposures in the spillway and abutments. As there was concern that the Lower Dam may have similar unknown conditions, the follow-on scope was conducted to address both dams.

1.2 Current Studies

The detailed scope of the current studies is defined in our proposal to the City of Wrangell dated January 22, 1993. It involves four parts; stability studies of each dam, downstream inundation mapping studies and miscellaneous meetings.

Since there were essentially no exploratory borings drilled into the dams other than the previously identified test pits (3) in the toe of the Upper Dam, soil strengths of the various embankment and foundation zones and water levels in the two dams have never been measured and used to confirm the current stability of the dam. For lack of this basic data, boring explorations, laboratory testing and water level measurements were accomplished to evaluate current conditions. For consistency, our approach to these stability studies followed procedures normally used to perform a Phase II study under the National Dam Safety Program, with modifications tailored to the needs of the project.

After the field program started at the Upper Dam, it became immediately apparent that "as-built" drawings did not match the new borehole information. Borings were having to be extended to considerable depths below the dam bottom to reach firm support. Additionally, one of the October 1992 test pits at the toe of the dam was depicting what was thought to be bedrock, but was discovered with the drilling work, to be a boulder. Crib depths and conditions were also inconsistent. Based on this initial effort, it was decided that the field work scope should be expanded at the Upper Dam to include additional borings/probes to further identify the nature and general density of the foundation materials in the toe area and below the dam. This follow-on scope change was outlined in our February 18, 1993 letter and approved at Wrangell City Council's regular meeting on February 23, 1993. Laboratory tests were conducted on selected soil samples and included mostly index and strength testing. These field and laboratory results are presented in Appendices A and B respectively.

From these new data, studies were conducted to evaluate the stability of the two dams under steady seepage and rapid drawdown conditions. Pseudo-static (simplified dynamic) stability analyses were also performed, treating the site as being in Seismic Zone 3 according to U.S. Army Corps of Engineer Criteria. In this analysis, a horizontal seismic coefficient of 0.10 was used. Special strength reductions to residual values were also applied to the poorer foundations soils, recognizing that liquefaction could occur. Based on these findings, embankment stability was checked to determine whether the dams meet minimum recommended stability standards provided in the Corp's guideline (U.S. Army Corps of Engineers, 1979). The end product of this study is a final report documenting the results of our field and laboratory efforts and stability analyses.

1.3 Limitations

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist and further assume that the results of the boreholes are representative of the subsurface conditions throughout the two sites, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the exploration. If there is a substantial lapse of time between submission of this report and the start of repair work, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples. In this instance, even with the new data, considerable interpretation was required to complete these studies. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

2.0 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. The two dams are located on Wrangell Island near the City of Wrangell. As shown in Figure 1, the dams and reservoirs are both on Mill Creek about 1500 feet apart and are situated in a narrow drainage-way about 1/2 to 1 mile southeast of the City. Most of the surface water in the watershed drains down steep rock slopes to the southeast and the reservoirs. 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 approximately 28 feet high, 315 to 320 feet long earth structures located on the same stream. The elevation difference between the two dams is 64 feet. The upper dam has a 25 to 33 foot wide crest, coarse granular slopes on both the up and downstream sides, and retains water via an internal crib core which prior to 1958 was it's own water retention structure, but leaked excessively and held little or no water. Available records suggest that in about 1958 it was covered with fill on both the up and downstream slopes to support water and then raised again in about 1967 with additional fill to create it's current shape. The approximate current shape of the dam is shown in Figure 2.

The lower dam, shown in Figure 3, has a much smaller 12 foot crest width with similar embankment slopes to the upper dam and an internal central sheet and treated timber core. A log crib dam once retained water at this site only the current dam was installed just upstream of the crib dam to retain a higher reservoir level. The remains of this log dam, shown in Figure 3, are visible in the downstream toe of the current dam.

The Upper and Lower Reservoirs supported by these dams are shown in Figure 1 and have a normal storage capacity of 122 and 67.5 Acre Feet (AF) respectively. The maximum storage capacities are 190 and 102 AF. For these capacities and heights, both dams are classified by the State of Alaska as small, the minimum defined size category. Using USDA Forest Service criteria, the size classification for both dams would be a C based on the above height and impoundment capacities, or a B if an exception to spillway design capacities is not obtained from the Regional Director of Engineering.

The two dams and reservoirs on Mill Creek form the water supply for the City of Wrangell. Water is taken to the treatment plant from the smaller Lower Dam reservoir via a pipeline. This reservoir is maintained full by manually adding water from the larger Upper Dam reservoir, when necessary. In the event of a failure of the Upper Dam, the Lower dam would be overtopped, washed out locally and possibly drained. An October 1992 inundation map prepared for rapid drainage of these dams indicated that the treatment plant would be approached but not flooded. Follow-on studies of the map with current conditions indicated that about a dozen homes fell in the inundation zone. The loss of storage water would also impact major industry and to a lesser extent, the community's water supply system and fire fighting capabilities. Based on this information and pending receipt of more accurate survey data and a better inundation map, the Forest Service determined that this dam should be considered high hazard. Based on the above map, the hazard classification of the Lower Dam would be Class 3 (low) as it's failure would not cause loss of the complete water supply system namely the Upper Dam and water supply. Sudden release of this lower and smaller (67.5 AF) reservoir by itself would also keep the inundation area within its main drainage channel where it would not likely impact downstream homes. With a general lack of confidence in the old survey map, it was felt that with new survey data, the above conclusions and the hazard classification of both dams should be confirmed as a part of follow-on studies.

3.0 FIELD EXPLORATIONS AND LABORATORY TESTING

Nine borings and five probes were advanced at the two dam sites to evaluate the subsurface conditions. Of these holes, all but three were drilled at the Upper Dam. The borings were designated B-1 through 9, while the five probe holes were labeled B-A, C, D, E and G. These borings and probes were advanced and sampled with a track-mounted Mobile B-47 drilling rig and extended to depths ranging between 24 and 77 feet. The locations of these borings and probes are shown on Figures 2 and 3.

The individual logs of the borings are presented in Appendix A. Also presented in Appendix A is a detailed description of the drilling and sampling procedures and the installation methods for the piezometers installed in each of the nine borings. Results of the groundwater readings, taken by our firm during and after drilling and by City personnel on April 27, 1993, about 70 days after drilling, are summarized on the logs and in Table 1.

As discrepancies were noted in much of our exploratory work, it was felt that survey support of key surface features was necessary including the elevations and locations of our borings and probes as well as current up and downstream slope profiles for both dams. In addition to the boring locations, longitudinal sections A-A and B-B were developed for each dam, Figures 4 and 5, three profiles were developed for the Upper Dam, Figures 6, 7 and 8, and one was developed for the Lower Dam, Figure 9. The locations of these profiles are shown in plan in Figures 2 and 3. This survey was conducted by City staff and furnished to us for our studies.

Laboratory tests were performed on representative soil samples from the borings to confirm the field classifications and to evaluate the engineering properties of the subsurface materials. Emphasis of the program was directed toward evaluating the index and strength characteristics of the embankment and foundation materials for stability analyses. The laboratory results are presented in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 Foundation Materials

The geology of this area is generally covered in our 1986 initial safety inspection report for these dams. This northeast trending valley is thought to have been carved or modified by glacial ice that advanced over the area at least once and probably several times during the Pleistocene Epoch. Bedrock exposed in the steep slopes and the spillway sidewalks consist of metamorphic rock including graywackes, schists, phyllites and slates that have been intruded by igneous rock. 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, the general slope or dip in the rock surface across the valley is depicted in Figures 4 and 5. The two Figure 4 profiles are a projection of boring data at the Upper Dam, the upper profile reflecting natural vertical and horizontal scales. The lower plot in Figure 4 has an exaggerated scale to depict the details of the foundation soils in the valley. While bedrock was not reached in many borings, a projection of the rock surface in Figure 4 suggests that the original sediments that infilled the steep sided valley following glacial carving may approach 60 feet in the bottom.

The boring data indicates that these 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. The more gravelly foundation samples in Appendix B likely depict slope debris while the sand and silt dominant soils with little gravels are probably alluvial in origin. Based on these gradation curves, the fines content in the foundation soils ranges from 10 to 60 percent, although in most instances it is more commonly in the 15 to 35 percent range. The mixing of the foundation materials is evident in part due to presence of organics scattered randomly within the foundation materials. Based on the results in Appendix B, Table B-1, the organics typically vary from 2 to 8 percent, but in many cases are not present. In one instance, Boring B-5 encountered silty, fibrous organics at a depth of 70 feet. The measured organic content was 64 percent.

The penetration resistance data taken during sampling in the borings show uncorrected values between 2 and 24 blows per foot with an average value of about 10 of 11 blows per foot. This generally corresponds to a loose to medium dense unconsolidated foundation material. The inability to perform plastic limit measurements on most of the soils indicates that even with a high fines content the foundation soils possess only a small amount of cohesion. For analysis purposes they are therefore are treated as cohesionless $c = 0$ materials.

As depicted in a section across the centerline of the Lower Dam (Figure 5), similar deep sediments also lie below in this part of the valley. The bedrock is exposed in the spillway and right abutment. Based on "as built" drawings, this rock is shallow below the right half of the dam and in the right abutment area, but then plunges near the center of the dam. None of our borings at the Lower Dam reached bedrock. The borings that extended into these foundation materials disclosed that these sediments are similar in gradation and density properties to those below the Upper Dam.

4.2 Embankment Materials

The embankment conditions encountered in the borings and probes are depicted generally in three subsurface profiles, Profiles 1, 2 and 3, for the Upper Dam, and one profile (Profile 4) for the Lower Dam, Figures 6 through 9. The locations of these profiles are shown in Figures 2 and 3. Detailed conditions are also shown on the boring logs, Appendix A.

Crib Material

In general, three basic embankment materials were encountered, 1) a wood/soil crib, 2) a silt over the Upper Dam, and 3) the upstream and downstream embankment shell coverings. Because of the general discrepancies between as-built, design, and actual conditions; considerable difficulty was encountered in developing the three Upper Dam profiles and distinguishing the size, depth and locations of the internal elements in the dam. Our approach therefore started by extrapolating pertinent information from old plan sheets, which depicted existing conditions focusing on the location and elevation of the crib structure. It was felt that "existing" information on the older sheets should be reasonably correct as the documents provided for changes to real conditions, whether the proposed changes were or were not actually constructed. This information was placed on the valley sections (Figures 4 & 5) along with the new borehole data for comparisons. The drawings for the Upper Dam showed that the crib was initially about 164 feet long, 5 feet high, rectangular shaped, and extended below the original groundline an unknown amount. The borings revealed that the crib was about 15 to 18 feet in vertical dimension, meaning it initially was buried 10 to 13 feet below the original groundline. This data also indicated that the crib is now five to seven feet lower than the old crest of the crib, meaning that considerable settlement of the crib likely occurred under the added weight of 23 feet of fill placed on top of the crib during 1968 raising efforts. The data also suggests that the crib members are not continuous as the crib was likely cut in the middle area to upgrade the water supply/drawdown piping through the dam.

In general, our borings at the Upper Dam encountered solid wood within the crib, although the Forest Service archives data indicates exterior deterioration and rotting of the outer members.

From this, we conclude that the integrity of the exposed outer logs in the upper five feet of the Upper Dam may be questionable.

Much of the crib in the Lower Dam is exposed and can be seen in photographs taken during earlier safety inspections. The face of wood is badly rotted, and is likely near failure where local downslope slumping of the crib materials is possible. The center area of the crib structure is missing, meaning it was breached with the raising effort

Both cribs for the two dams are filled with earth of similar density and composition. Our borings reveal that the Upper Dam's crib soils are classified as silty gravelly sands and silty, sandy gravels, while the Lower Dam materials are slightly siltier. Of the ten penetration resistance tests in the Upper Crib zone, two are high (45 and 50 blows/foot) while eight are low (less than 15 blows/foot). Discounting the two high values as being on rock materials or logs, the average uncorrected penetration resistance of the remaining eight tests is 10 blows per foot or borderline between loose and medium dense. This is depicted in Figure 10. In the Lower Dam, six penetration tests within the crib depicted values between 4 and 18 blows per foot with an average of 11 blows per foot.

In developing the average shape of the crib in the Upper Dam, it is known that the log crib was rectangular and buried roughly 10 to 13 feet below the groundline. To bury the crib therefore means an open excavation had to be constructed with backfill placed around the outside of the excavation to restore the groundline to it's original grade. Since no borings were drilled in this area, we have therefore assumed conservatively that the backfill was in a limited work space and therefore compacted to the same low density as the crib materials. This is reflected as wedges on either side of the crib in the profiles, Figs. 6, 7 and 8. The use of these low densities in this area is justified based on Old Forest Service archives for this dam which indicates re-use of excavated materials was permitted.

As there is no evidence that the crib extended below the groundline in the Lower Dam (Figure 9), this wedge was not added to Section 4.

Concern was expressed that the logs in the Upper Dam may have buoyancy and their lighter weight than soil may produce reduced lower stability than analysis would suggest. For analysis, buoyancy was ignored as a reasonable assumption, as the logs have been submerged or partially saturated under the same environment for at least 25 years. Conditions have therefore probably stabilized in this period where buoyancy should, under a worst case scenario, be very small. For weight considerations, the wood was assumed to represent 25 percent of the crib section shown in the profiles, with a density of 70 pcf. For analysis purposes, this would make the average wet unit weight of crib area in the profiles about 114 pcf.

Silt

Directly on top of the wood crib, a low plasticity silt unit was encountered in all three borings drilled in Section 1 of the Upper Dam (Figure 6). This silt was locally gravelly and sandy and three penetration tests produced an average driving resistance of 7 blows per foot. This corresponds to a medium stiff cohesive material or a loose granular soil. For purposes of analysis, this material is treated as a cohesionless (c = 0) material or the more conservative of the two possible assumptions. The top of this unit is thought to represent the sloping and flat top zone of material depicted on the as built documents. The documents suggest that this silt was likely placed here as an extension of the impervious core as part of early (pre 1967) attempts to seal the leaking Upper Dam and raise the water level. These documents also suggest that sloping fills were placed on each side of the crib and at the higher elevations to form what is referred to as the top of old dam in Figure 4. The survey information and current observations suggest that this dam was later covered with another granular shell to create the current slopes in Figures 6, 7 and 8.

The silt cap was not present on top of the crib at the Lower Dam. In addition to this crib, the lower dam is reported to have a sheet pile (partial wood and partial steel) cut off wall to control seepage through the dam. Its exact location could not be determined from our explorations, however, this is not considered critical to the stability. For the purpose of the stability analysis, the strength influence of the wall was ignored as it was judged to be small. It's lateral position is also within the center crest area where significant material changes on either side of the cutoff will not, in our opinion, greatly alter the calculated factor of safety for either the up or downstream slopes. The water level readings in Boring B-8 and 9 should provide adequate information for evaluating it's current effectiveness at maintaining low hydrostatic forces in the downstream embankment.

Outer Embankment

Both dams possess similar outer granular shells consisting of till-like materials of about equal parts of silts, sands and gravels. They therefore are not rockfill dams, as originally suspected, but rather earthfill dams with central cores for water control. The grain size results in Appendix B indicate that the embankment shell materials are only slightly less silty than the crib soils. The penetration resistance values show that these embankment soils are consistently much denser. Twelve penetration resistance values for the Upper Dam range from a low of 14 to a high of 77 with an average uncorrected value of 43 blows per foot. These values and Figure 10 suggest a moderate compactive effort and a generally dense consistency. Based on the compaction test in Appendix B, Figure B-9, the shell material for the Upper Dam has a maximum dry density of about 127 pounds per square foot and an optimum water content of 9.4 percent. The natural water contents in Table B-1 also reflect that most of the materials in the Upper Dam embankment are wet of optimum meaning, if excavated, they will be difficult to replace in a compact state.

The granular materials at the Lower Dam have similar gradation and compaction properties to the Upper Dam only they are less dense and therefore have slightly lower in place wet unit weights and higher water contents. Based on limited boring data, the section in Figure 9 reflects three basic shell units differentiated largely by density. The upstream shallow and deep embankment materials have average uncorrected penetration resistance values of 17 and 9 blows/foot respectively. The lower upstream zone also contains less gravel and is mostly a silty sand. The downstream embankment is more compact with average uncorrected penetration values of 34 blows per foot.

The sheet pile wall in the Lower Dam presumably falls between borings B-7 and B-8 in Figure 9. It's integrity cannot be evaluated by boring explorations nor was it considered necessary as it's effectiveness at sealing off water should be reflected in the piezometers and signs of toe seepage. In both instances, low water levels in the dam and no apparent seepage suggest that hydrostatic pressures are not large in this dam. The sheet piles structural integrity was ignored in the follow-on stability analysis.

4.3 Voids/Piping

During the drilling operations, concern was expressed that voids or extremely soft materials may exist below the crib structures of the Upper Dam and may have led to piping out at the toe of the dam from the toe seepage that is occurring. Concern was also expressed that the materials may be so poor that after drilling and prior to sampling the auger may have settled through any soft material meaning that only the better quality, not the poorer materials, were sampled. In our opinion, voids were not encountered during drilling and the low blow counts that were recorded (as low as 2 blows per foot) are a reflection that loose materials are present and were not missed. Also with these loose materials being accounted for, it is likely that any voids that are present below the logs are small (i.e. the low angle of repose of the loose saturated materials would have filled most voids, limiting their size).

Referring to Figure 6, it is our opinion that the risk of piping of loose crib materials from the Upper Dam is low for the following reasons.

1. The gravelly silty sands in the foundation are too well graded and while loose have too many fines for migration below the dam.

2. The dam is not a rockfill dam, but rather an embankment dam with 60 to 70 percent sand and silt size particles with little or no apparent cohesion. The void space or porosity of this material is small or low compared to a rockfill material, providing limited paths for piping. As long as the flow or seepage through these materials is not large, excessive internal piping cannot occur. Before internal piping can occur, the fines at the toe would have to be washed out first causing signs of ground subsidence in this region as an early warning sign.

3. With the crib structure and soil materials set well into the foundation materials (Figure 6), piping of the lower two-thirds of crib materials up to the original ground level and through a pervious zone of embankment is unlikely. Based on the low water levels in the foundation area, the hydraulic gradient in the zone of piping (from the crib to the toe) is very low and not conducive to extensive piping.

4. Voids, sink holes, crest subsidence or silt deposition in the toe were not observed on the crest surface, the slopes, or in the borings that would imply internal piping.

5. The flow over the weir (7 gpm) while indicative of only partial seepage is clear water and does not appear to be transporting fines.

6. The one void noted in one of the City's test pits at the toe of the Upper Dam is likely the result of a decaying log at this location, according to Jim Pung,City Engineer. Also, if piping was occurring in the denser shell materials at the toe of the dam, piping within the dam and the loose inner crib materials would likely be widespread and would have shown some signs of distress by now in the form of crest subsidence or sinkholes.

4.4 Groundwater

Groundwater conditions at the two dams were determined by measuring the water level in new piezometers installed in each of the nine borings after the drilling work was completed. Piezometers were not installed in the probes, as most of these holes were drilled in the toe area of the Upper Dam in a low lying area of the drainage-way where the water level was known to be at or near the ground surface. To aid in evaluating these results, water level measurements were also noted in the field during drilling. The water level taken both during drilling in the borings and later in the piezometers are tabulated in Table 1 and summarized in the profiles, Figures 6 through 9. In a few instances, water data is not shown and was not taken during drilling because of the drilling method. Once water is introduced as part of the drilling operation, these measurements can no longer be taken until after the piezometers have been installed and water levels have stabilized.

During drilling, one boring in the Upper Dam experienced excess hydrostatic pressures (about a 6 foot head above the reservoir level) which resulted in water surge flow out the top of the augers as the hole extended from a depth of 5 to 11 feet. This boring (B-1) is located about 7 feet horizontal distance from the reservoir near the upstream crest edge of the dam. Since this boring is positioned on the crest and is surrounded by the reservoir on the upstream side, the downstream dam face and no water on the opposite side, and the spillway on a third side, this pressure under natural conditions can only be charged by water flow from the left abutment 300 feet across the 35 foot crest to this area. Since these levels are not reflected in the 3 crest borings (B-4, 5 and 6) spaced between this boring and the left abutment, we conclude that this flow is likely not a reflection of real conditions that would affect stability, but induced pressures by drilling operations.

Locally during drilling, water levels higher than depicted in the piezometers after drilling were recorded in the downstream area of the Upper Dam, namely B-2. This suggests that water may be perched locally on impervious zones within the fill and may now be draining into the borehole and out the hole at a lower elevation where more pervious and better draining materials exist. This condition is not judged to be adverse to the overall performance of the dam as it indicates that 1) the pore pressure in the deeper weaker materials (the crib and foundation) are low and 2) there are pervious layers in the downstream embankment providing for relief and drainage of any excess pressures before they can develop. The hydrostatic pressures due to these perched conditions while they may reduce local stability are not considered significant to mass instability as they must be very low or a reflection of water ponded zones. If they were more significant and charged by seepage through the dam they would be evidenced as springs on the embankment slope. With the exception of the one seep at the toe, no obvious springs or wet zones are evidenced on the embankment slope of the Upper Dam.

5.0 STABILITY STUDIES

5.1 Approach to Analyses

Stability analyses were performed on each of the four sections of the dam embankments in Figures 6 through 9 using the strength and unit weight properties in Table 2 and both circular arc and random irregular failure surfaces. Both up and downstream slopes were studied in each section using the 2 dimensional limit equilibrium slope stability program, PCSTABL5M. This program was developed at Purdue University and has been upgraded over at least 13 years to handle general slope stability problems by the Simplified Janbu, Simplified Bishop, and Spencer method of slices with varying options to pick the type of failure surface as well as other features or corrections to improve the calculated results.

Both upstream and downstream slopes were first studied under steady seepage static loading conditions to determine 1) the location of the critical failure surface or arcs and the factor of safety that would lead to a breach of the dam. In many instances, the failure area was expanded to allow shallow failure on the embankment slopes to have a minimum calculated factor of safety to compare with the value that would result in breaching of the dam. This information provides insight into the mode of failure. This effort was followed by an analysis of the upstream embankment section under sudden (or rapid) drawdown conditions. Finally, the most critical embankment section (both up and downstream) in both dams were evaluated using pseudo-static methods to determine the change in factor of safety due to simulated earthquake loading conditions. Because liquefaction was likely in some materials under strong shaking, strength reductions to undrained residual values had to be accommodated in the analysis to simulate this condition where appropriate. The factors of safety determined under each load condition where breaching of the dam would result were then compared with that recommended by the Corps of Engineers (Table 3) to determine if they exceeded the suggested values. As a part of the National Dam Safety Program, it was considered desirable to have a minimum factor of safety of 1.5 for steady seepage conditions under the maximum normal pool, 1.2 for sudden drawdown conditions and 1.0 for earthquake. For the sudden drawdown study, full slope drawdown is assumed to occur so rapidly that drainage or reduction in pore pressure cannot occur.

Under steady seepage conditions, the reservoir level was placed at Elevation 360 feet for the Upper Dam and 295.5 feet for the Lower Dam (the maximum normal pool elevation). This condition is close to the maximum level that develops under normal loading conditions and assumes that the spillway is overflowing about 1.5 feet above the normal levels of Elevation 358.5 and 294 feet respectively. This condition closely follows the criteria suggested by the Corps of Engineers in Table 3 (Case III). The actual phreatic surface used in our analyses is shown in the profiles (Fig. 6 through 9) relative to the water level measurements in the piezometers.

Under sudden drawdown conditions, the reservoir level is assumed to drop instantly from the maximum normal pool elevation to the lowest feasible level with the water surface in the drawdown area being placed at the upstream embankment surface. Our stability of the upstream embankment was based on dropping the reservoir at the Upper and Lower Dams from the maximum normal pool to the bottom of the reservoirs. In reality, the chances of this occurring are small, as the drawdown values and piping are too small to allow it to happen this rapidly.

Under earthquake loading conditions, the same conditions used for steady seepage loadings apply with adjustments in strength to accommodate strength loss due to local liquefaction. This reduction is discussed in the next section. In addition, a suitable horizontal seismic coefficient is added to the driving forces on each slice analyzed. In previous studies, this site was judged to be in Seismic Zone 2 and the dams were evaluated using the Corps recommended 0.05 g coefficient. We understand that the Corps has recently revised their seismic map placing Wrangell on the boarder line between Seismic Zone 2 and 3. From Forest Service review comments, the higher more conservative Zone 3 seismic coefficient (0.1 g) were used in this analysis.

5.2 Analyses Procedures

To evaluate the stability of the two dams, two basic programs were used from the previously referenced computer program; the simplified Bishop method applicable to circular shaped failure surfaces and the simplified Janbu method applicable to failure surfaces of irregular random shapes. Both analyses calculate the factor of safety of a slope by the method of slices. In a calculation, a failure surface is determined starting on the ground surface at the toe of the slope and generating trial failure surfaces composed of a series of straight line segments of equal length that extend through or up the slope to the crest. In most cases, a 10 foot line segment was specified for this project to form the chord of a circular arc or a random surface. The search is controlled only by specifying the horizontal width of the search area at the toe and crest of the slope. In the case of these dams, deep seated sliding and breaching of the dam was controlled by specifying that the failure plane occur below the entire crest width of the slope or further upslope if calculations so indicated. To seek the minimum value or a shallow sliding condition, this width limit was generally specified near the top of the down slope crest, allowing the search routine wider boundaries.

Through repetitive iterations over 100 random or circular surfaces are studied, accumulated and sorted by values of their factors of safety. The ten most critical (lowest) surfaces are then plotted and numbered so that the pattern may be evaluated visually on the profile being studied and modified and re-run if necessary. Examples of these plots are presented as Figures 11 and 12 for random and circular failure surfaces respectively. Both circular and random irregular surfaces were studied for each section to be sure that the lower values were being calculated.

A single phreatic surface, comprising a line 1.5 feet above the reservoir level and the piezometer readings was used in the analyses to approximate hydrostatic conditions in the dams. In Janbu's random search program, the new perpendicular method was used to approximate the pore pressure, reducing the conservatism in the analysis where the pore pressure is normally taken as the vertical distance from the base of the slide to the phreatic surface immediately above. The new method approximates the equipotential line as a straight line from the base of the slide perpendicular to the line through the piezometer surface bounding the top of that slice.

It was originally intended to take relatively undisturbed samples of the soils for triaxial testing for comparison with the penetration resistance values for developing soil strength parameters. Unfortunately, the materials both in the dam and foundations were gravelly, greatly limiting the ability to recover these samples. Of the two samples attempted, both were of poor quality. It was therefore decided to recompact disturbed samples and test them for strength at their natural moisture contents. The lack of material often required preparing composites of several consecutive samples from a boring to have enough material for testing. Because the materials were largely granular with little cohesion, consolidated undrained triaxial tests were performed with pore pressure measurements in order to define effective and total stress envelopes and identify for analysis the shear strength parameters specified in Table 3 (the average between total and effective stresses). While the triaxial results in Appendix B confirmed that the material possesses little cohesion; the high gravel content in the composited samples, the presence of local organics and the inability to reproduce realistic densities led to strength results which in most cases were greatly in excess of the values that would be indicated by the penetration tests. This problem was particularly evident when the same composited sample (B-C, S-2 to 4) was prepared twice and produced widely varying results. Confidence in the triaxial results was therefore low and on the unconservative side, making the penetration test results the best information for assessing strength for our analysis.

The strength values were therefore determined by selectively deleting unusually high penetration values (N values) in each major unit and averaging the remaining values. These average N values were converted to angles of internal friction, ϕ, under undrained conditions by empirical relationships established by Peck, Hanson & Thornburn in their text book titled Foundation Engineering. This relationship is included as Figure 10. The N values were uncorrected for overburden efforts to offset the use of the slightly larger (2.5 inch OD) sampler over the more standard (2 in. OD) size. All strength values in Table 2 therefore are based on no apparent cohesion, the above ϕ (Fig. 10) relationship and undrained conditions with one exception. This one special condition is discussed below and was applied where the N value in the foundation material is less than 10 blows per foot and the slope is being evaluated under earthquake loading conditions.

The dynamic stability of the embankment and foundation soils was evaluated using a pseudostatic method of analysis applied to the critical arcs or failure surfaces determined from steady seepage loading conditions. As implied in the name, pseudo-static analyses are an extension of the conventional static method of analysis and is provided for in the above computer program. The earthquake force is included in the analysis as an equivalent static lateral force applied to all the elements in the shear slice. The magnitude of the force is defined by a seismic coefficient expressed as a fraction of gravity, multiplied by the weight of the slice. The pseudo-static analysis provides the factor of safety for the failure surface for a given seismic coefficient.

A seismic coefficient of 0.10 g was used in the analyses to represent the lateral earthquake forces. As indicated previously, this coefficient corresponds to that recommended by the Army Corps of Engineers for analyses of dams in Seismic Zone 3.

Pseudo-static analyses are appropriate to evaluate the seismic stability of slopes provided that the soils do not liquefy or experience a major loss of strength during the earthquake. The outer embankment materials of the Upper Dam and the downstream section of the Lower Dam are generally considered to be dense where pore pressure build up and liquefaction is not considered possible even under the most severe earthquake shaking. Granular materials such as 1) in the cribs, 2) in the lower upstream area of the Lower Dam and 3) particularly in the foundation soils below and on either side of both dams are much less dense with low blow counts where they may be susceptible to these conditions in the event of a design earthquake typical of those experienced in a Seismic Zone 3 region.

The empirical procedures of Seed (Seed, 1979 and SW-AA 1976) were used to account for the liquefaction potential of these marginal materials in our analyses. In Seed's procedure, liquefaction potential is evaluated using Standard Penetration Resistance N-values taken from the borings. The first step in the analysis involves the differentiation between nonliquefiable and potentially liquefiable materials. The loose materials identified above are considered potentially in this category. The next step is to correct the N values for overburden effects and then compare them and cyclic stress ratios that would be anticipated for Seismic Zone 3 with published field case histories and large scale laboratory tests where liquefaction has both occurred and not occurred. This data is summarized in Figure 13. Based on this comparison, granular soils in Seismic Zone 3 would likely liquefy if the average corrected N value were 10 blows per foot or less. Using this value (10) as a threshold value, procedures recommended by Seed and Harder (1990) were used to determine the amount of strength loss that should be applied to these zones. The next step involves correcting the N values for overburden and correcting the N value a second time for the amount of fines. In our study, our N values were in fact corrected a third time to account for the larger sampler used in the field study (2.5" OD vs. 2" OD). In this correction, the N value was reduced an additional 20 percent for the larger sampler. If the final N value is equal to or less than 10 blows per foot, it should be assumed for analysis that the soils at that point would liquefy and drop to residual undrained strengths which for the corrected N value of less than 10 blows can be taken as 200 psf ($\phi = 0$, c = 200 psf). The published relationship upon which this value is based is shown in Figure 14. From these results, and based on the geology or our knowledge of general construction procedures, areas where liquefaction can be expected can be identified on the profiles for use in the analysis.

For purposes of analysis, it was assumed for both dams that the crib soil materials may liquefy and lose some strength, however, the crib's wood framing in all likelihood would hold it together and remain intact for several reasons. Logs are extensive in the crib zone based on the drilling, and substantial area wide strength losses would need to occur along the dam axis before enough volume change can take place and cause the entire crib to distort excessively. Further, the crib on the Upper Dam in Figure 6 is deeply embedded entirely within dense embankment fill on the top part and seated well into the relatively low permeability foundation soils where it would be difficult for a liquid material within the crib to reach the slope face where it can flow out. In the Lower Dam, the crib is close to the face of the slope, however, liquefaction is only common in saturated materials (i.e. below the water table). Based on Figure 9 the water level encompasses only the lower third of the crib structure and is in an area where the N-values are somewhat higher.

A second area identified where liquefaction may occur is the upstream fill of the Lower Dam, Figure 9. Only one data point fell into this range, suggesting local loose pockets, however, the overall database in this region is weak. For lack of more definitive information and the fact that the hazard rating is thought to be low for this dam, we have assumed that it would not liquefy.

Substantial areas of the foundation materials were found to be potentially liquefiable below both dams and in the downstream areas immediately below the dams. By compositing all N-values for each dam and plotting the results with elevation on the more critical slopes (Figures 6 and 9) it appears that the shallow soils in the upper 15 to 20 feet in the valley area were liquefiable diminishing to 5 feet or less deep below the dam. This boundary is represented by the bold dashed line in Figures 6 and 9. Above this line, it was assumed in the analysis that liquefaction would occur and shear strengths in this area would drop to a residual value (200 psf). Below or outside this boundary, the foundation soils would not liquefy and the static strength would remain unchanged.

5.3 Analyses Results

The results of our stability analyses are tabulated in Table 4 for varying load conditions, for both circular arc and irregular (non circular) failure surfaces and for both shallow failures that would not breach the dam and deeper failure surfaces that would. From the Table 4 results, a number of trends are evident:

1. Analyses using irregular random failure surfaces produce factors of safety in this case which were consistently below those analyses assuming circular arc failure surfaces.

2. Analyses where shallow failure surfaces were permitted always resulted in lower factors of safety than deep seated failure and breaching of the dams. This is because the materials are largely granular (ϕ materials) where the shear strength is, by definition, lower where the normal forces or overburden pressures are low. This means that the mode of failure would be shallow initially and progress deeper.

3. With the exception of the smaller Section 3 at the Upper Dam, the factors of safety for the downstream slopes are lower than those for the upstream slopes.

4. Under steady seepage, the factors of safety against breaching of the Upper Dam (deep seated sliding) on both up and downstream faces are above 1.5 for all sections or above the minimum value recommended by the Corp of Engineers for static stability. The same may also be said for the Lower Dam, only the random search analysis of the downstream face indicates that the factor of safety drops slightly below the recommended 1.5 to 1.35.

5. Under steady seepage and sudden drawdown conditions, lower than suggested factors of safety are calculated for shallow failure surfaces that would not jeopardize the safety of the dam. This is judged to be a reflection of steep embankment slopes and weak materials in the face of the dam slopes.

6. Under earthquake loading conditions, factors of safety of much less than 1 were calculated for the downstream slopes of both dams. This means that failure with breaching is highly possible under a future strong earthquake.

7. Under earthquake loading conditions, the upstream slopes of both dams have factors of safety of greater than 1 against deep failures and breaching of the dam. It drops below 1 for shallow failure surfaces of the larger Upper Dam meaning slumping of the upstream face may occur during a future strong earthquake.

As indicated above, low factors of safety were calculated for shallow sliding or ravelling of the steep up and downstream slopes under many loading conditions. The failure surfaces in these instances are not, however, deep enough to breach the dam and pose a safety hazard to downstream inhabitants. To evaluate safety conditions for each slope, the minimum factors of safety were calculated for arcs or failure surfaces that were deeper and would encompass, as a minimum, the entire crest of the slope or likely breaching and water release.

For the Upper Dam, the results in Table 4 generally indicate that the Corps recommended minimum factors of safety (1.5 for steady seepage, 1.2 for sudden drawdown and 1.0 for earthquake) are nearly met for all loading conditions except the downstream slope under earthquake loading. Factors of safety of 0.44 for shallow sliding and 0.73 for actual breaching suggest that the Upper Dam is vulnerable to complete downslope failure during a strong earthquake. This failure would be induced by liquefaction and strength loss of the loose foundation soils both at the toe and below the dam. In order to improve this stability condition to acceptable minimum standards some form of buttress needs to be installed at and below the toe of the Upper Dam.

In several instances, factors of safety in Table 4 for the Upper Dam dropped slightly below the recommended 1.2 value under sudden drawdown conditions. This is attributed to the steep upstream slopes, the lack of consideration of riprap on the face, and the severity of the loading assumed. Full drawdown from maximum normal pool to the bottom of the reservoir is probably an excessive loading condition for this dam because short of complete failure of the dam, the maximum feasible drawdown is from the normal pool to the outlet pipe. Assuming no recharge from the creeks and slope runoff, it would take the two drawdown pipes (8 and 10" in diameter) several weeks or more running full to create this much drawdown. In this time period, face drainage would in reality have time to occur where a higher factor of safety than calculated probably exists.

Similar findings were also reached for the Lower Dam, although it should be understood that the database for the study is much weaker than at the Upper Dam. Therefore the analysis and conclusions had to be derived based on limited information and extrapolations, particularly in the upstream area. The fact that these results produced similar factors of safety as the Upper Dam give some confidence that additional studies would likely produce overall results not too different than reached herein. In summary, they show low factors of safety of 0.4 to 0.5 for the downstream face under earthquake loading. These results too are a reflection of the liquefaction of the foundation soils. The smaller difference in the shallow and deep seated results and the lower results reflect a much smaller crest width and a likely more rapid slope deterioration and breaching than would take place at the Upper Dam. This also implies that the Lower Dam would likely fail first, followed by the Upper Dam.

When compared with the Corps suggested factors of safety, factors of safety for the Lower Dam under steady seepage and sudden drawdown fall close to or below this criteria. This is in part attributed to a weak database for the lower dam and likely a marginal design. A toe buttress would also need to be installed to raise the static stability and at the same time provide the much needed additional dynamic stability.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our slope stability studies reveal that the Upper and Lower Dams both have marginal but adequate stability under static load conditions but poor overall stability under earthquake loading conditions. From the assumptions and analyses presented, the low factors of safety of about 0.4 to 0.8 suggest that both dams could fail under a future strong earthquake. We therefore conclude that remedial treatment of both dams is appropriate to protect any residents that may lie in the inundation zone during a breaching and to assure a continuous supply of water to the City. Without knowledge of all the alternatives possible for serving the long term water supply needs for Wrangell, at first review, rock or granular filled toe buttresses appear to be the more attractive method for improving the stability of the dams.

We understand that dams and most other important structures on U.S. Forest Service lands must go through an orderly process in order to receive approval to implement such repairs. The policies and procedures adopted by the Forest Service, referred to as NEPA (National Environmental Policy Act), requires follow-on studies 1) to justify the feasibility of the repair compared to other alternatives, 2) to conduct an environmental impact statement for the treatment or selected alternative and 3) to design and construct the repair with the Forest Service's review and approval of all phases of the work. This can be a long term process and can take several years or more, particularly if funding is critical. We recommend that this work be implemented as soon as possible as a strong earthquake could potentially create extensive damage and loss of the existing water supply system.

In addition, short term measures should be implemented on an emergency basis to provide for the immediate safety of residents that may lie in the inundation zone should one or both dams fail. We understand that recent checks of residents in the preliminary inundation zone prepared by our firm in October 1992 reveal that 12 or 13 homes/trailers may lie in the flood zone. This according to a 1988 publication by the U.S. Department of Interior, Bureau of Reclamation and titled Downstream Classification Guidelines would make this a high hazard dam. This has also been confirmed in recent Forest Service correspondence for these dams. A March 25th ground survey performed by Forest Service Engineer, John Bowman, resulted in a memo dated March 31, 1993, suggesting that 1) the contours in the October 1992 inundation map were based on aerial photo interpretation and are likely in considerable error and 2) the probability of homes lying in the inundation zone is low, but should be checked. We concur with this finding and recommend that the City immediately obtain the needed survey information and complete an accurate new inundation map for failure of both dams. The time from breach to flooding of the developed area should also be addressed. If homes or habited trailers lie in the new inundation, a plan should be immediately implemented to protect these residents. This may include temporary evacuation, moving of trailers and/or installation of an early warning system, which could be made a part of the Emergency Action Plan, which we understand is in draft form. If homes fall in this zone, one alternative may include constructing a small berm or dike above the homes or in the channel area to divert the water back to the main channel and/or around the homes. This construction work could be implemented quickly without a complete design as it would likely fall on State of Alaska land where the NEPA process can be avoided. With the safety of the residents assured by the above short term measures, the focus would be directed more toward completing the NEPA Process and implementing the chosen alternative for providing water to the City on a long term basis.

7.0 MISCELLANEOUS CONSIDERATIONS

We understand that plans initially included breaching the center of the Upper Dam to install a new water supply pipe through its center and downslope to the treatment plant. Our studies revealed that most of the material, except for the upper 5 to 10 feet, will be wet of it's optimum moisture content for suitable compaction and marginally, if at all, reusable as backfill in the excavation. Our stability studies also reveal that the factor of safety of the upstream face is low for slumping under sudden drawdown conditions. Therefore if the reservoir is drained to install the pipe, the drawdown should be conducted slowly such that slope face drainage can occur and truly rapid drawdown conditions do not occur.

As a result of the above explorations and studies, nine observation wells have been installed, six in the Upper Dam and three in the Lower Dam. These instruments provide a data source for monitoring the short and long term performance of the dams. Our stability studies have relied heavily on readings taken roughly 70 days after the wells were installed and not necessarily maximum levels that could occur with seasonal melting, extended rainfalls, or unusual conditions. To confirm that the above readings are a reasonable representation of actual conditions and not subject to large fluctuations, we recommend that until a repair is implemented, all observation wells should be read at least twice monthly and compared with past readings. If the levels fluctuate less than 2 feet from previous readings, no action is considered necessary and the data should be entered as a permanent record under the operations and maintenance plan. Separate extra readings should also be taken following unusual events such as an earthquake or following periods of extended rainfall or rapid snow melt. If large rises in the water level are noted (greater than 2 feet) particularly from observation wells on the dam crests, the data should be reviewed by an experienced engineer to assess its impact on overall stability.

A crude weir was installed in October, 1992, without Forest Service approval of the plans. Now that it is in place, it should also be monitored as a part of the long term O&M Plan. The frequency of readings required by the Forest Service has been specified as daily in previous correspondence, largely because of the dam's current high hazard rating. After the above short term measures are implemented, the long term measures should provide for a better devise for monitoring future total toe seepage, if a buttress repair option is selected as the preferred long term alternative to provide for the existing dams and water storage for the City of Wrangell.

SHANNON & WILSON, INC.
Fred R. Brown
Fred R. Brown, P.E.
Vice President

STATE OF ALASKA
FRED R. BROWN
CE 5469
REGISTERED PROFESSIONAL ENGINEER

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TABLE 1
WATER LEVEL MEASUREMENTS

Boring	Date Installed	During Drilling Depth, Ft.	Date 1 Depth, Ft.	4/27/93 Depth, Ft. (Elev.)
B-1	2/7/93	-2	11.05	11.6 (351.1)
B-2	2/5/93	20	25.4	26.9 (335.8)
B-3	2/8/93	3	-0.33	2.3 (334)
B-4	2/9/93	31	28.2	28.3 (333)
B-5	2/11/93	23	20.4	20.0 (342.8)
B-6	2/14/93	5	none	20.1 (341.6)
B-7	2/16/93	8	15.2	13.5 (284.6)
B-8	2/17/93	none	none	20.5 (277.1)
B-9	2/18/93	18	15.5	12.0 (275.9)
Probes				
B-A			No	
B-C		18	piezometers	
B-D		7	installed	
B-E		7.5	in	
B-G		none	probes	

5/20/93

TABLE 2
PARAMETERS FOR STABILITY STUDIES ****

UPPER DAM PARAMETERS

	Wet Unit * Wt., pcf	Rapid Drawdown		Steady Seepage		Earthquake	
Embankment		ϕ, deg.	c, pcf	ϕ, deg.	c, pcf	ϕ, deg.	c, pcf
Outer Shell	135	39	0	30	0	30	0
Crib	119*	31	0	31	0	31	0
Silt Cap	110	29	0	20	0	29	0
Foundation (N>10 to 25)**	114	30	0	30	0	30	0
Foundation (N≤10)**	114	30	0	30	0	0	200***
Firm Base Foundation (N>25)	Lower Failure Boundary						

LOWER DAM PARAMETERS

	Wet Unit * Wt., pcf	Rapid Drawdown		Steady Seepage		Earthquake	
Embankment		ϕ, deg.	c, pcf	ϕ, deg.	c, pcf	ϕ, deg.	c, pcf
Downstream	135	37	0	37	0	37	0
Upper Upstream	125	33	0	33	0	33	0
Lower Upstream	125	30	0	30	0	30	0
Crib	119	31	0	31	0	31	0
Foundation (N>10)	114	30	0	30	0	30	0
Foundation (N ≤10)**	114	30	0	30	0	0	200
Firm Base Foundation	135	39	0	39	0	30	0

* Extrapolated from triaxial results in Appendix B with adjustments, as appropriate. Crib weight assumes 25% of mass is wood with average unit weight of 70 pcf.

** N values corrected for overburden and fines effects per Seed and Harder, 1990 for evaluating undrained residual strengths plus reduced 20% more for use of large (2.5" O.D.) sampler over normal (2" O.D.) sampler.

*** Undrained Residual strength taken from empirical data in Seed and Harder 1990.

**** Most of strength values above based on penetration resistance values and undrained (total) strengths rather than triaxial results and procedures suggested in Table 3. This modification in use of strength parameters is on the conservative side (see Appendix B for discussion).

TABLE 3
FACTORS OF SAFETY (1)
CORPS OF ENGINEER CRITERIA

Case	Loading Condition	Factor of Safety	Shear Strength(2)	Remarks
I	Sudden drawdown from spillway crest or top of minimum drawdown elevation	1.2 (3)	Minimum composite of R & S shear strengths	Within the drawdown zone, submerged unit weights of materials are used for computing forces resisting sliding, and saturated unit weights are used for computing forces contributing to sliding
II	Partial pool assumed horizontal steady seepage saturation	1.5	$\frac{R+S}{2}$ for R < S	Composite intermediate envelope of R and S shear strength
III	Steady seepage from spillway crest or top of gates with $K_h/K_v = 9$ assumed (4)	1.5	Same as Case II	
IV	Earthquake)Cases II and III with seismic loading)	1.0	(5)	Use .10 for Seismic Coefficients in Zone 3

Notes

(1) Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths of clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

(2) Other strength assumptions may be used if in common usage in the engineering profession.

(3) The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

(4) K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

(5) Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled damage conditions maintained during the test. 'R' tests are those in which specimen drainage is allowed during consolidation according (or swelling) under initial stress conditions, but specimen drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

Criteria taken from U.S. Army Corps of Engineers (1979)

TABLE 4
SUMMARY OF STABILITY ANALYSES

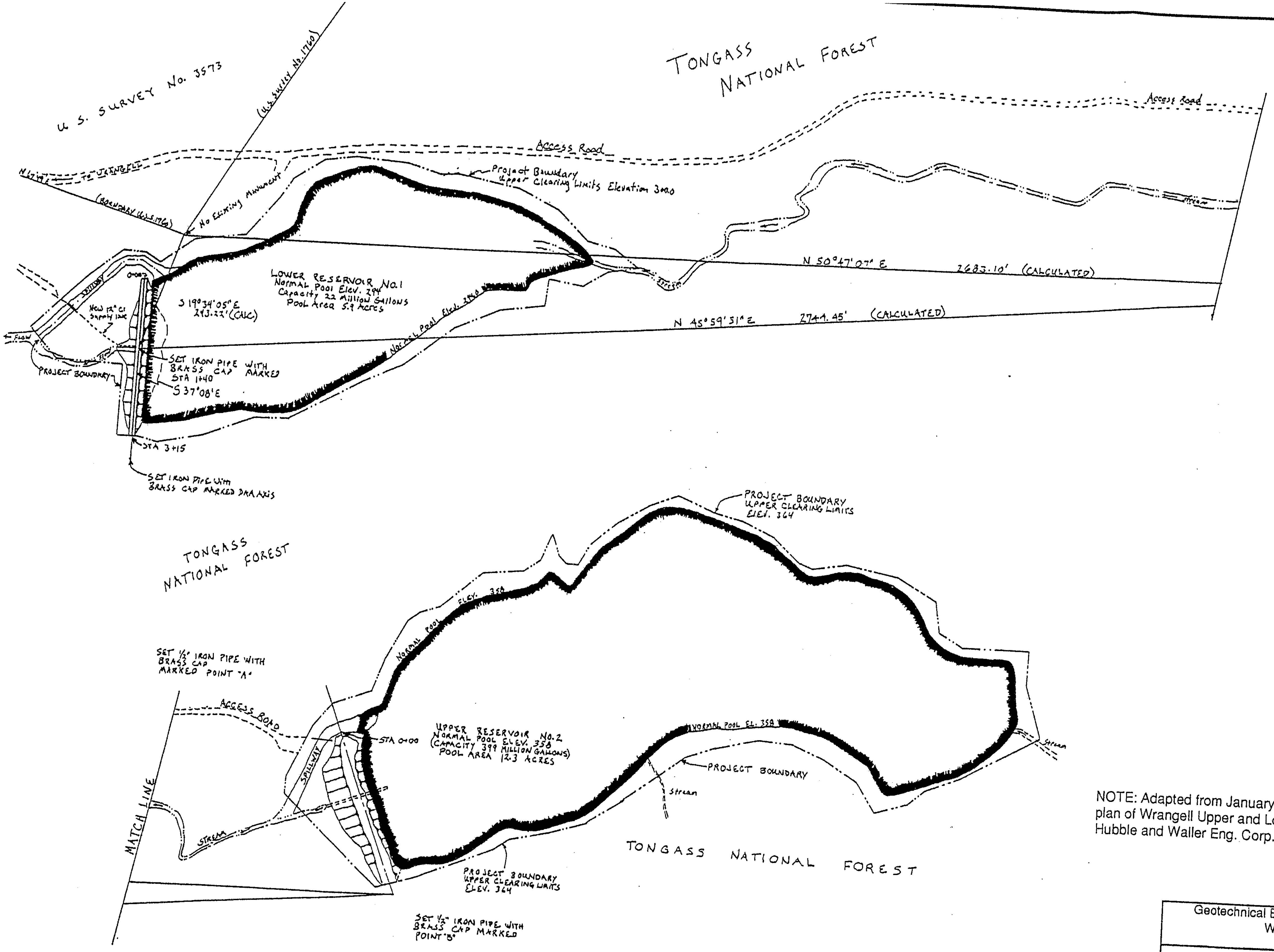
UPPER DAM

SECTION	SLOPE	LOADING	FACTOR OF SAFETY	FAILURE	COMMENT
1	Downstream	Steady seepage	1.16	Circular arc	Shallow failure (no breach)
1	Downstream	Steady seepage	1.72	Circular arc	Deep failure/breach
1	Downstream	Steady seepage	1.56	Random Surface	Deep failure/breach
1	Downstream	Earthquake	0.44	Random Surface	Shallow failure
1	Downstream	Earthquake	0.73	Random Surface	Deep failure/breach
1	Downstream	Earthquake	0.78	Circular arc	Shallow failure
1	Upstream	Steady seepage	1.44	Circular arc	Shallow failure
1	Upstream	Steady seepage	1.79	Circular arc	Deep failure/breach
1	Upstream	Steady seepage	1.54	Random Surface	Shallow failure
1	Upstream	Earthquake	1.11	Circular arc	Shallow failure
1	Upstream	Earthquake	0.90	Random Surface	Shallow failure
1	Upstream	Earthquake	1.36	Circular arc	Deep failure/breach
1	Upstream	Earthquake	1.18	Random Surface	Deep failure/breach
1	Upstream	Sudden Drawdown	0.85	Circular arc	Shallow failure
1	Upstream	Sudden Drawdown	1.22	Random Surface	Deep failure/breach
2	Downstream	Steady Seepage	2.06	Circular arc	Deep failure/breach
2	Downstream	Steady Seepage	1.79	Random Surface	Deep failure/breach
2	Upstream	Steady Seepage	2.50	Circular arc	Deep failure/breach
2	Upstream	Steady Seepage	2.18	Random Surface	Deep failure/breach
2	Upstream	Sudden Drawdown	1.32	Circular arc	Deep failure/breach
2	Upstream	Sudden Drawdown	1.11	Random Surface	Deep failure/breach
3	Downstream	Steady Seepage	2.78	Circular arc	Deep failure/breach
3	Downstream	Steady Seepage	2.49	Random Surface	Deep failure/breach
3	Downstream	Steady Seepage	2.21	Circular arc	Deep failure/breach
3	Downstream	Steady Seepage	2.08	Random Surface	Deep failure/breach
3	Upstream	Sudden Drawdown	1.22	Circular arc	Deep failure/breach
3	Upstream	Sudden Drawdown	1.15	Random Surface	Deep failure/breach

LOWER DAM

4	Downstream	Steady Seepage	1.51	Circular arc	Deep failure/breach
4	Downstream	Steady Seepage	1.35	Random Surface	Deep failure/breach
4	Downstream	Earthquake	0.52	Circular arc	Deep failure/breach
4	Downstream	Earthquake	0.43	Random Surface	Deep failure/breach
4	Downstream	Earthquake	0.52	Circular arc	Shallow failure
4	Upstream	Steady Seepage	2.16	Circular arc	Deep failure/breach
4	Upstream	Steady Seepage	2.16	Random Surface	Deep failure/breach
4	Upstream	Earthquake	1.33	Circular arc	Deep failure/breach
4	Upstream	Earthquake	1.17	Random Surface	Deep failure/breach
4	Upstream	Sudden Drawdown	1.16	Circular arc	Deep failure/breach
4	Upstream	Sudden Drawdown	1.13	Random Surface	Deep failure/breach
4	Upstream	Sudden Drawdown	0.76	Random Surface	Shallow failure

5/20/93



NOTE: Adapted from January, 1968 revised construction plan of Wrangell Upper and Lower Dams, General Plan by Hubble and Waller Eng. Corp., Seattle, Washington

Geotechnical Evaluation of Wrangell Dams
Wrangell, Alaska

UPPER AND LOWER DAM AND RESERVOIR

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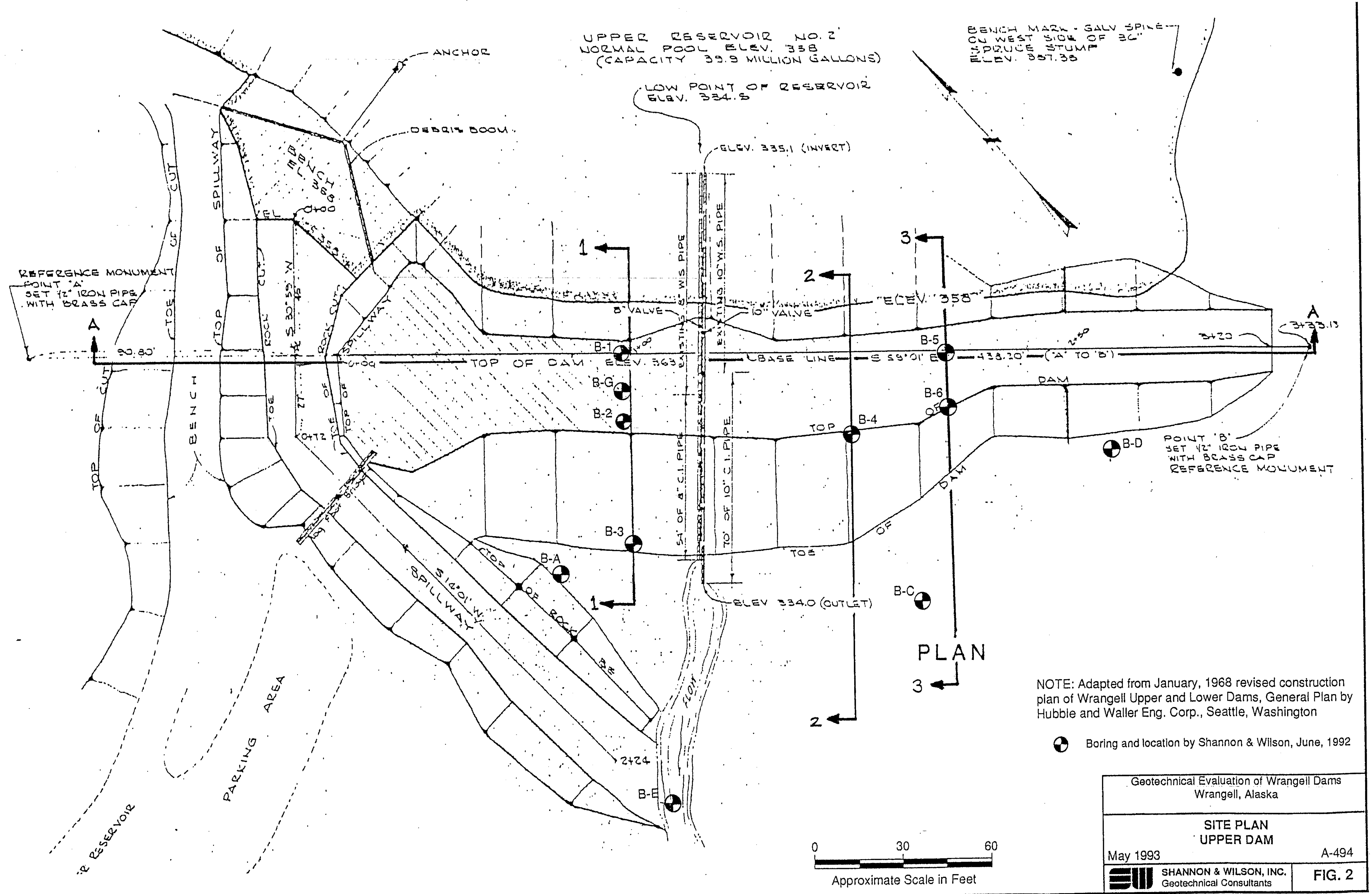
FIG. 1



NOTE: Adapted from January, 1968 revised construction plan of Wrangell Upper and Lower Dams, General Plan by Hubble and Waller Eng. Corp., Seattle, Washington

● Boring and location by Shannon & Wilson, June, 1992

Geotechnical Evaluation of Wrangell Dams
Wrangell, Alaska

SITE PLAN
UPPER DAM

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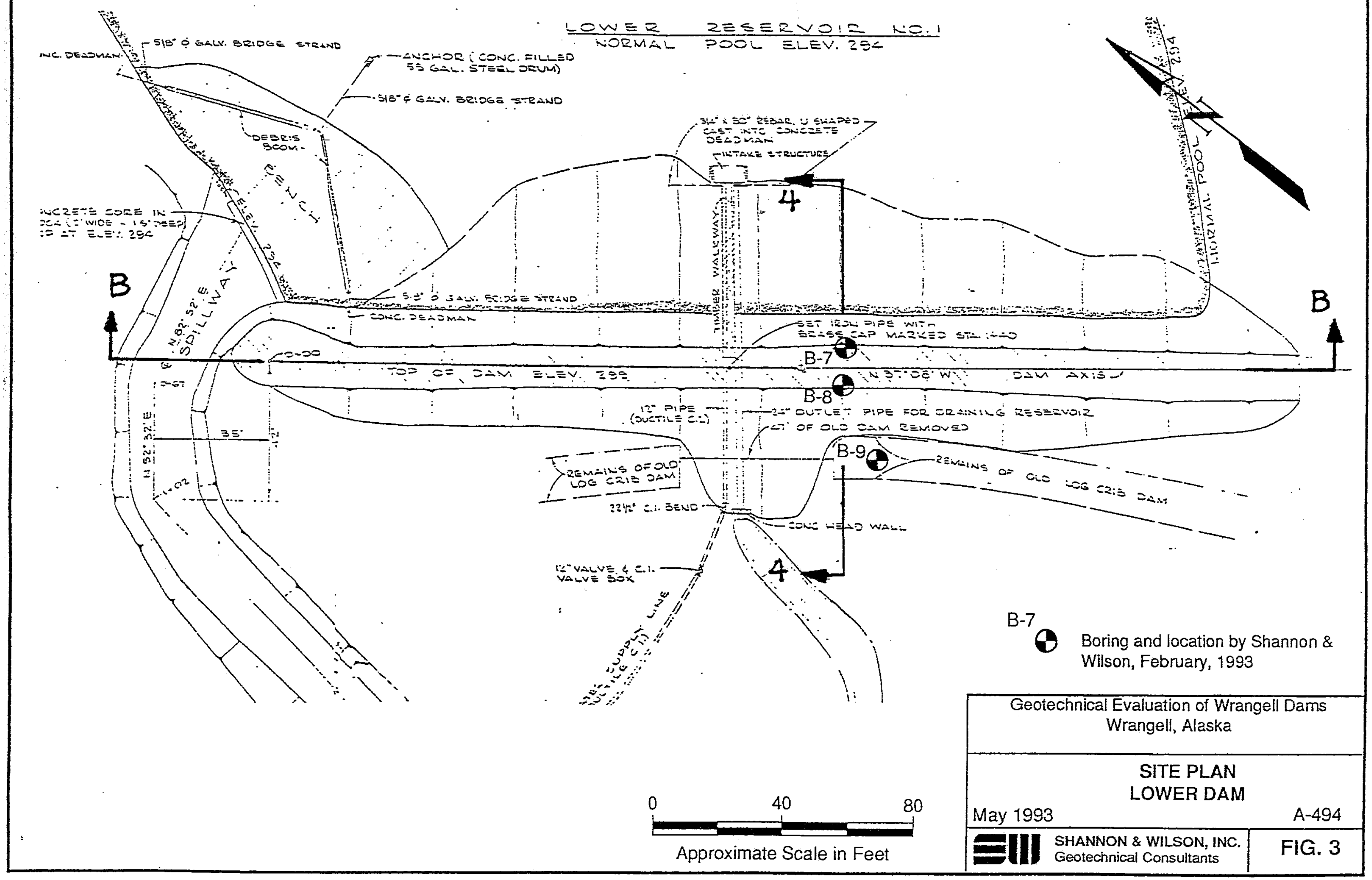
FIG. 2

Adapted from January 1968 as-built plan of Wrangell Lower Dam, Lower Dam Plan and Sections by Hubble and Waller Eng. Corp., Seattle, Washington



Geotechnical Evaluation of Wrangell Dams
Wrangell, Alaska

SITE PLAN
LOWER DAM

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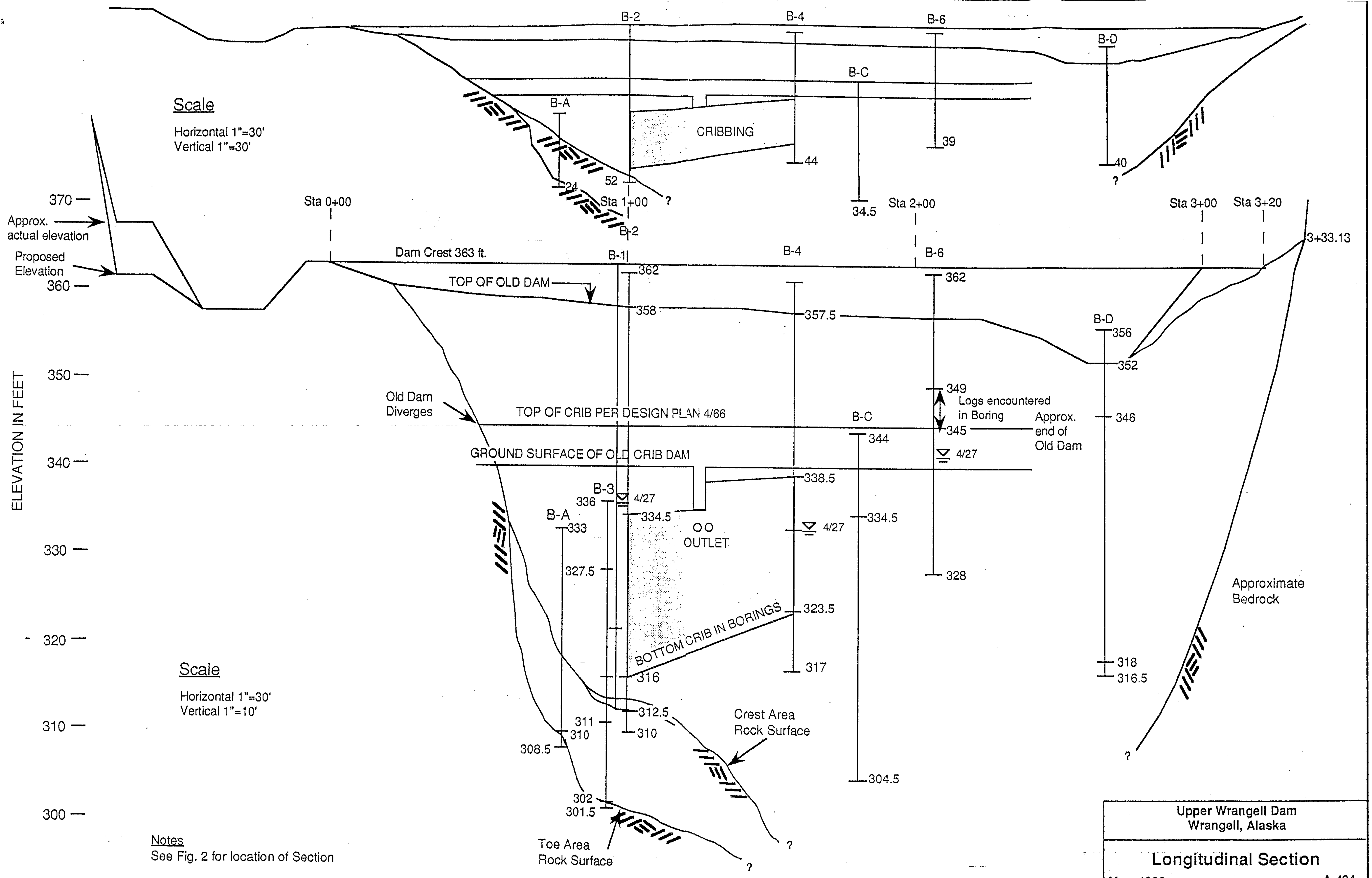
FIG. 3



Notes
See Fig. 2 for location of Section

Upper Wrangell Dam
Wrangell, Alaska

Longitudinal Section

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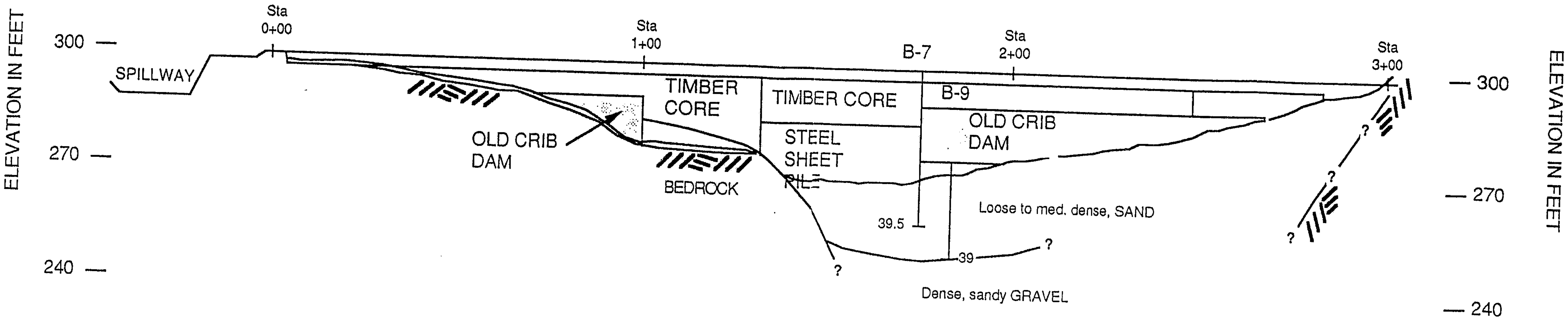
FIG. 4



NOTE

1. Section based heavily on existing information
2. See Fig. 3 for location of section

SCALE 1"=30'

Lower Wrangell Dam
Wrangell, Alaska

Longitudinal Section

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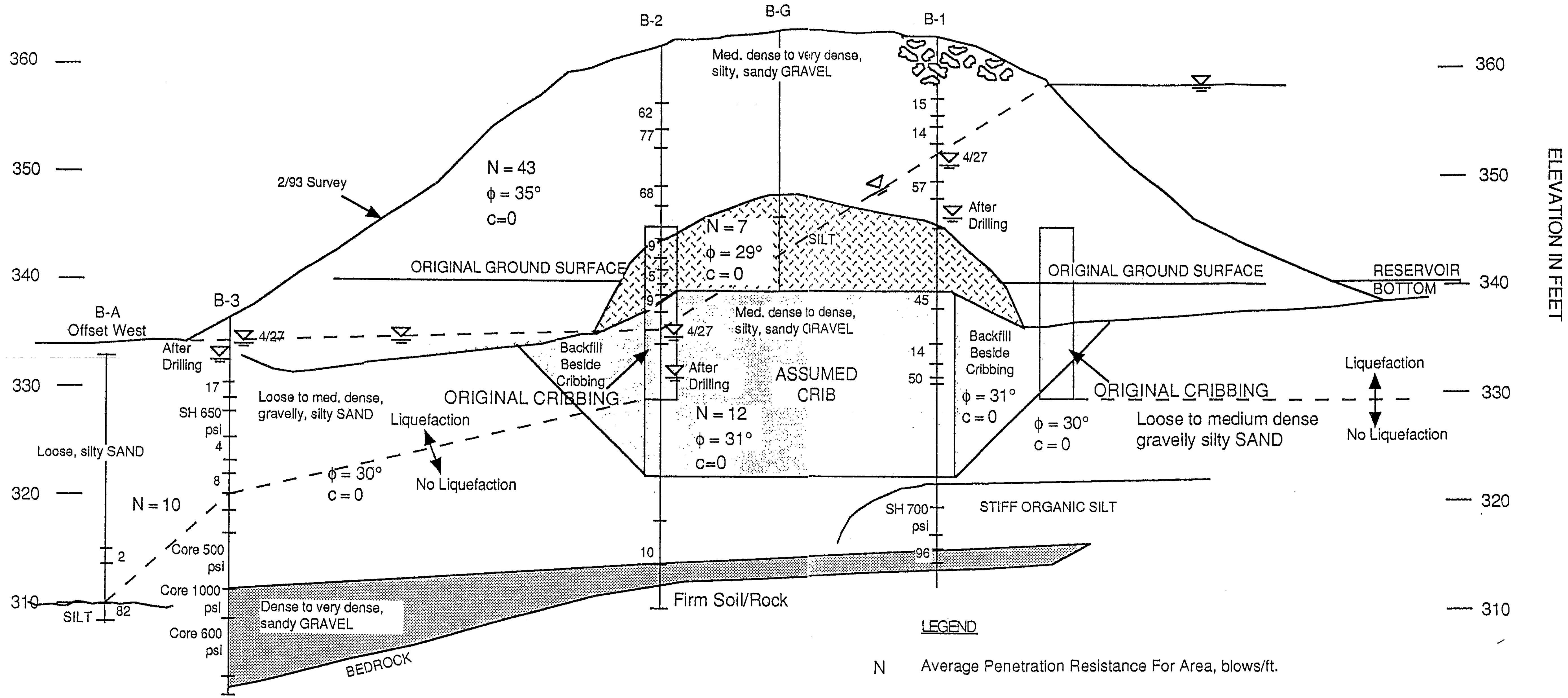
FIG. 5



LEGEND

N	Average Penetration Resistance For Area, blows/ft.
ϕ	Angle of Internal Friction
c	Cohesion Intercept
SH	Undidsturbed Sample Locations
▽	Water Level Measurements
4/27	Date of Water Level Reading
Core 500	Core Pressure During Sampling, psi
9 ┼	Sample Penetration Resistance, blows/ft.

NOTE
1. See Fig. 2 for location of section
2. Liquefaction boundary assumed for dynamic stability studies only. In liquefaction zone, strength was reduced to residual value of c=200psf

SCALE 1"=10'

Upper Wrangell Dam
Wrangell, Alaska

Section 1

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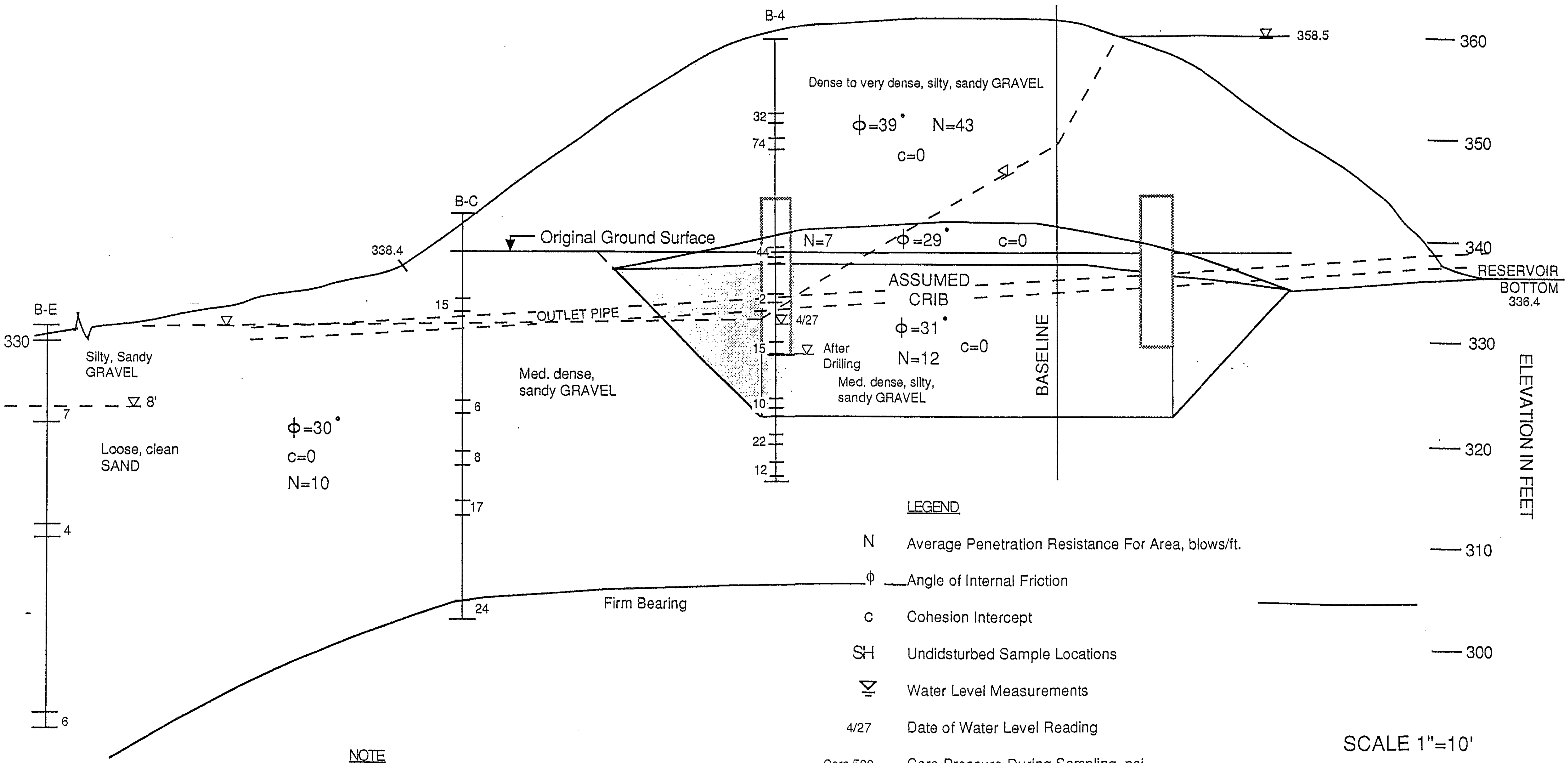
FIG. 6



LEGEND

N	Average Penetration Resistance For Area, blows/ft.
φ	Angle of Internal Friction
c	Cohesion Intercept
SH	Undidsturbed Sample Locations
▽	Water Level Measurements
4/27	Date of Water Level Reading
Core 500	Core Pressure During Sampling, psi
9	Sample Penetration Resistance, blows/ft.
☐	Cribbing Per Design Plans

NOTE
See Fig. 2 for location of section

SCALE 1"=10'

Upper Wrangell Dam
Wrangell, Alaska

Section 2

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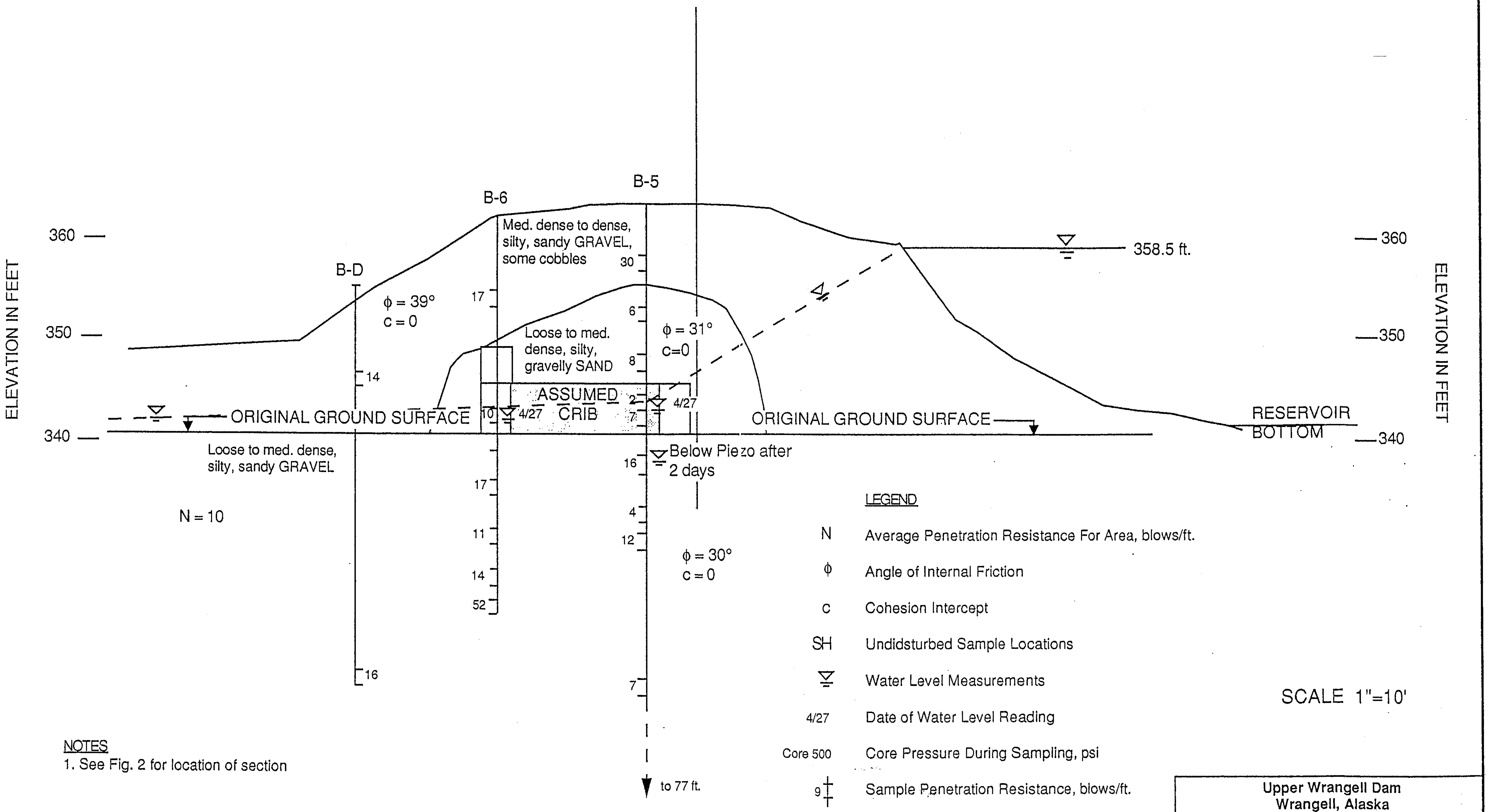
FIG. 7



LEGEND

N	Average Penetration Resistance For Area, blows/ft.
ϕ	Angle of Internal Friction
c	Cohesion Intercept
SH	Undidsturbed Sample Locations
▽	Water Level Measurements
4/27	Date of Water Level Reading
Core 500	Core Pressure During Sampling, psi
9 ┼	Sample Penetration Resistance, blows/ft.

NOTES

1. See Fig. 2 for location of section

Upper Wrangell Dam
Wrangell, Alaska

Section 3

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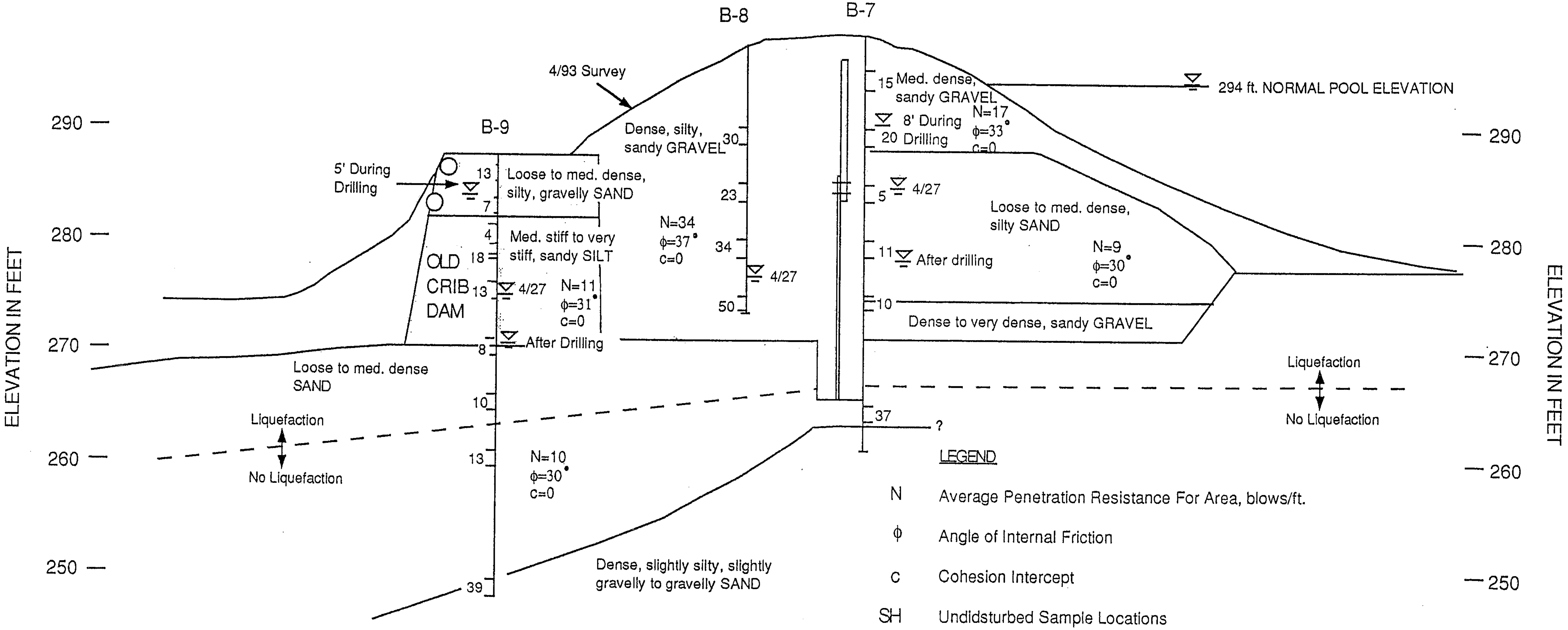
FIG. 8



LEGEND

N	Average Penetration Resistance For Area, blows/ft.
ϕ	Angle of Internal Friction
c	Cohesion Intercept
SH	Undidsturbed Sample Locations
▽	Water Level Measurements
4/27	Date of Water Level Reading
Core 500	Core Pressure During Sampling, psi
9 ┼	Sample Penetration Resistance, blows/ft.

SCALE 1"=10'

NOTE
1. See Fig. 3 for location of section
2. Liquefaction boundary assumed for dynamic stability studies only. In liquefaction zone, strength was reduced to residual value of c=200 psf

Lower Wrangell Dam
Wrangell, Alaska

Section 4

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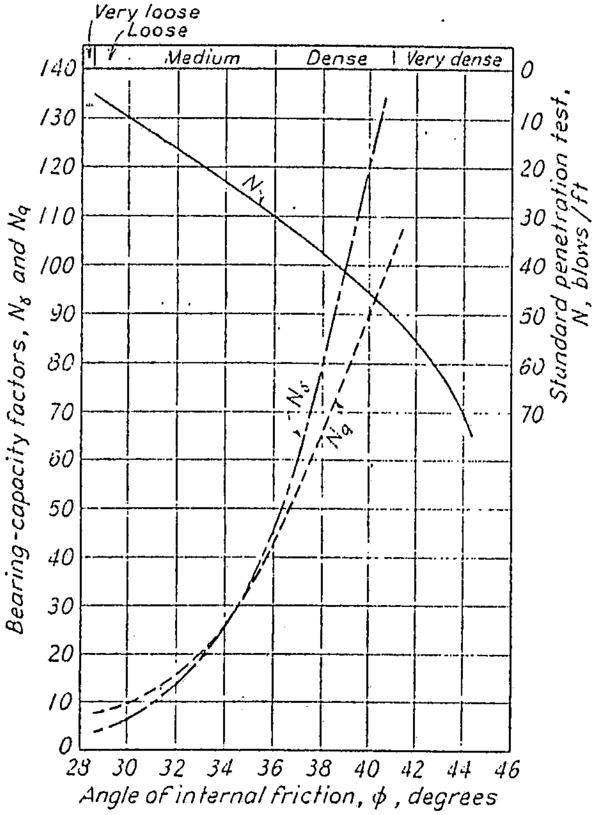
FIG. 9



Curves showing the relationship between φ, bearing capacity factors, and values of N from the standard penetration test.

Upper & Lower Dams Evaluation
Wrangell, Alaska

N VALUE VS. FRICTION ANGLE

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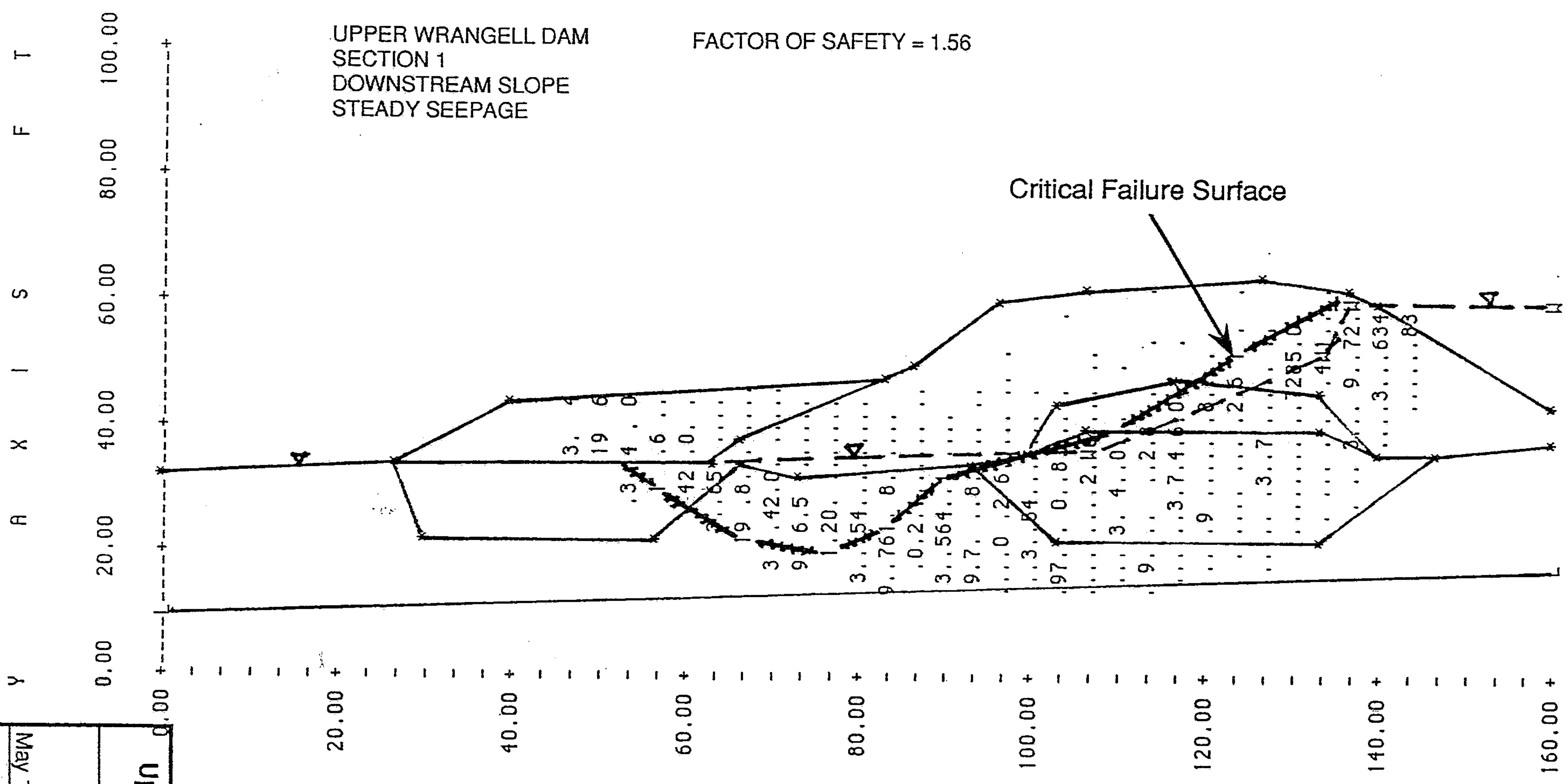
FIG. 10


UPPER WRANGELL DAM
SECTION 1
DOWNSTREAM SLOPE
STEADY SEEPAGE

FACTOR OF SAFETY = 1.56

Critical Failure Surface

LEGEND

++++++++ Critical Failure Surface

Notes:

1. Buttress modeled at toe of slope was assigned either zero strength/weight properties or the same properties as the materials that will be replaced in order to calculate the factor of safety without a buttress

Upper & Lower Dams Evaluation
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RANDOM PLOT

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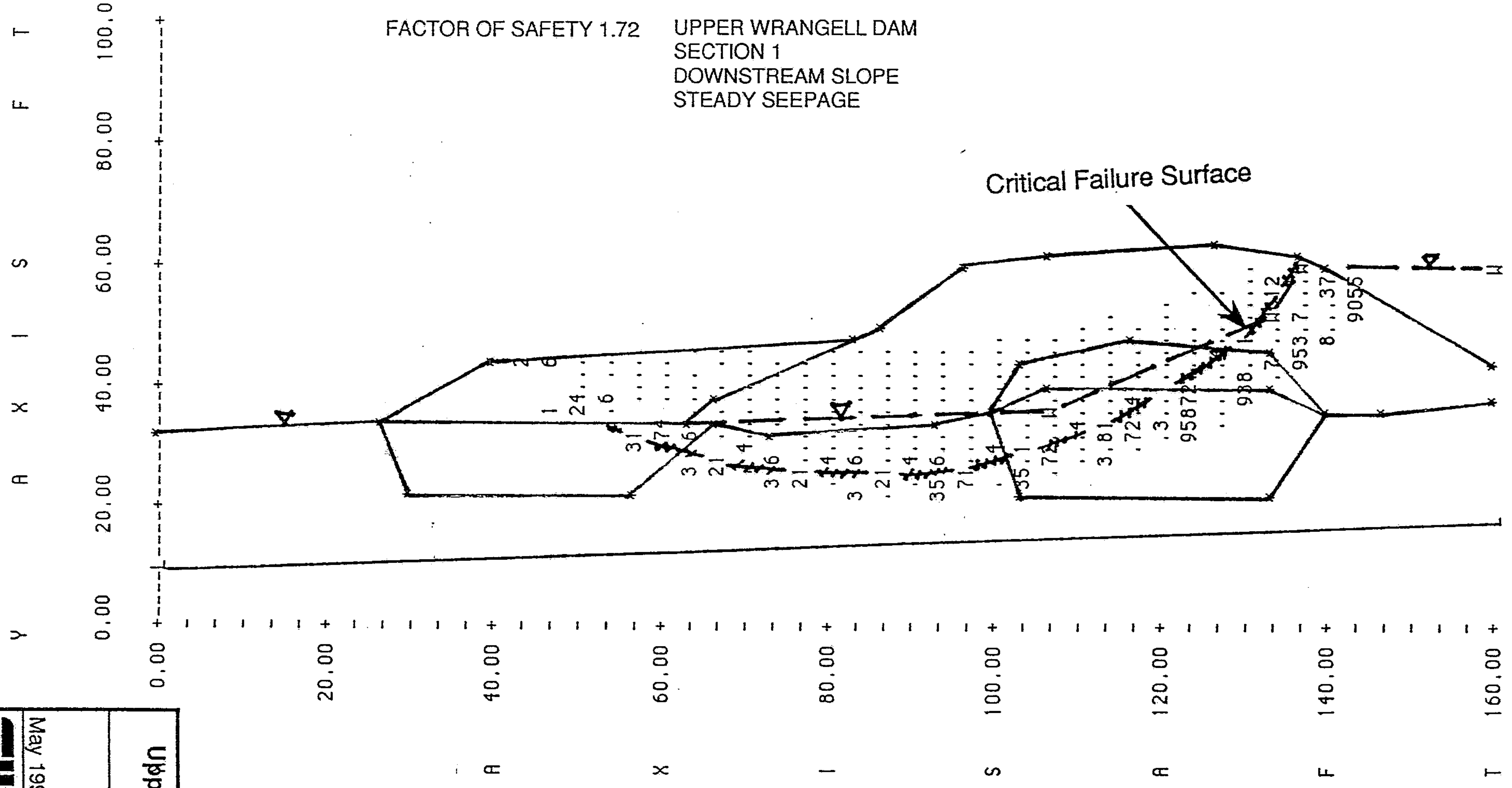
FIG. 11



LEGEND

Critical Failure Surface

Notes:

1. Buttress modeled at toe of slope was assigned either zero strength/weight properties or the same properties as the materials that will be replaced in order to calculate the factor of safety without a buttress

Upper & Lower Dams Evaluation
Wrangell, Alaska

CIRCULAR PLOT

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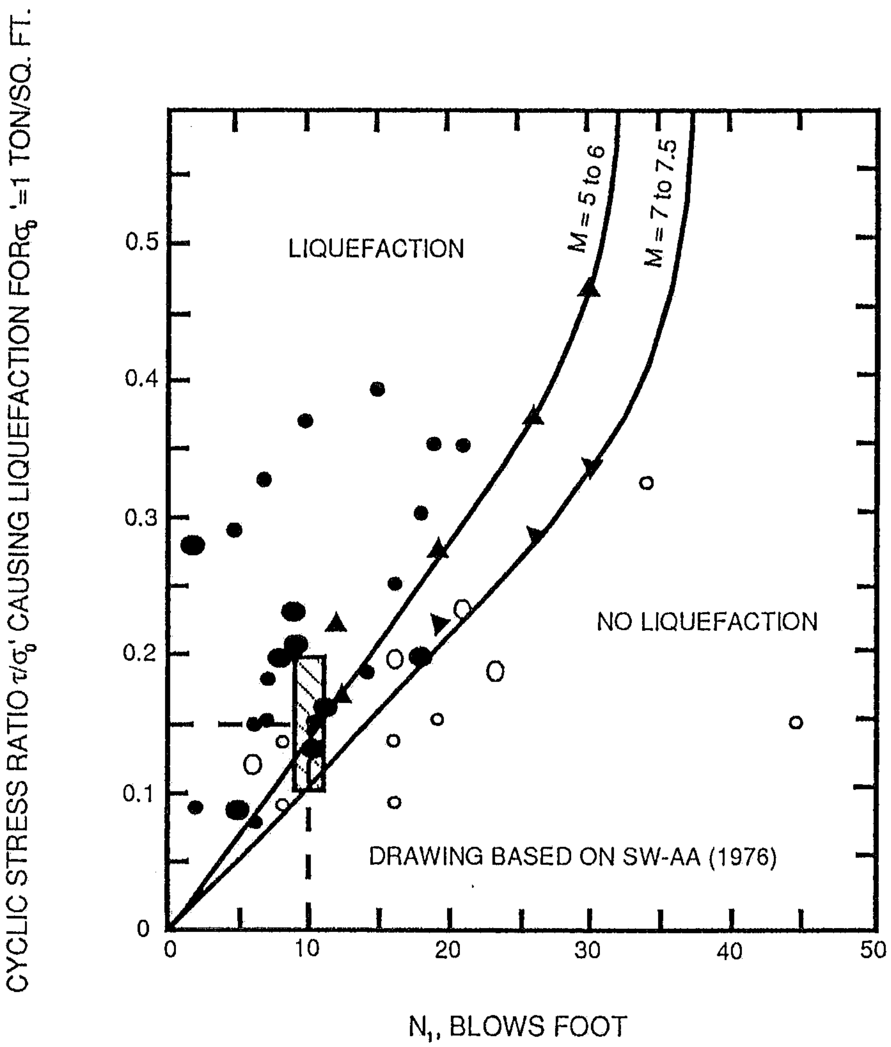
FIG. 12



LEGEND

● ○ BASED ON FIELD DATA } DATA FROM SW-AA (1976)

▲ ▼ } EXTRAPOLATED FROM RESULTS OF LARGE SCALE LABORATORY TESTS } DATA FROM SW-AA (1976)

▨ RANGE OF VALUES OF STRESS RATIO AND N$_i$ FOR MATERIALS WITHIN AND UNDERLYING DAM

M EARTHQUAKE MAGNITUDE

NOTE

SOLID POINTS ON PLOT INDICATE SITES AND TEST CONDITIONS SHOWING LIQUEFACTION. OPEN POINTS INDICATE SITES WHERE NO LIQUEFACTION OCCURRED.

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LIQUEFACTION EVALUATION

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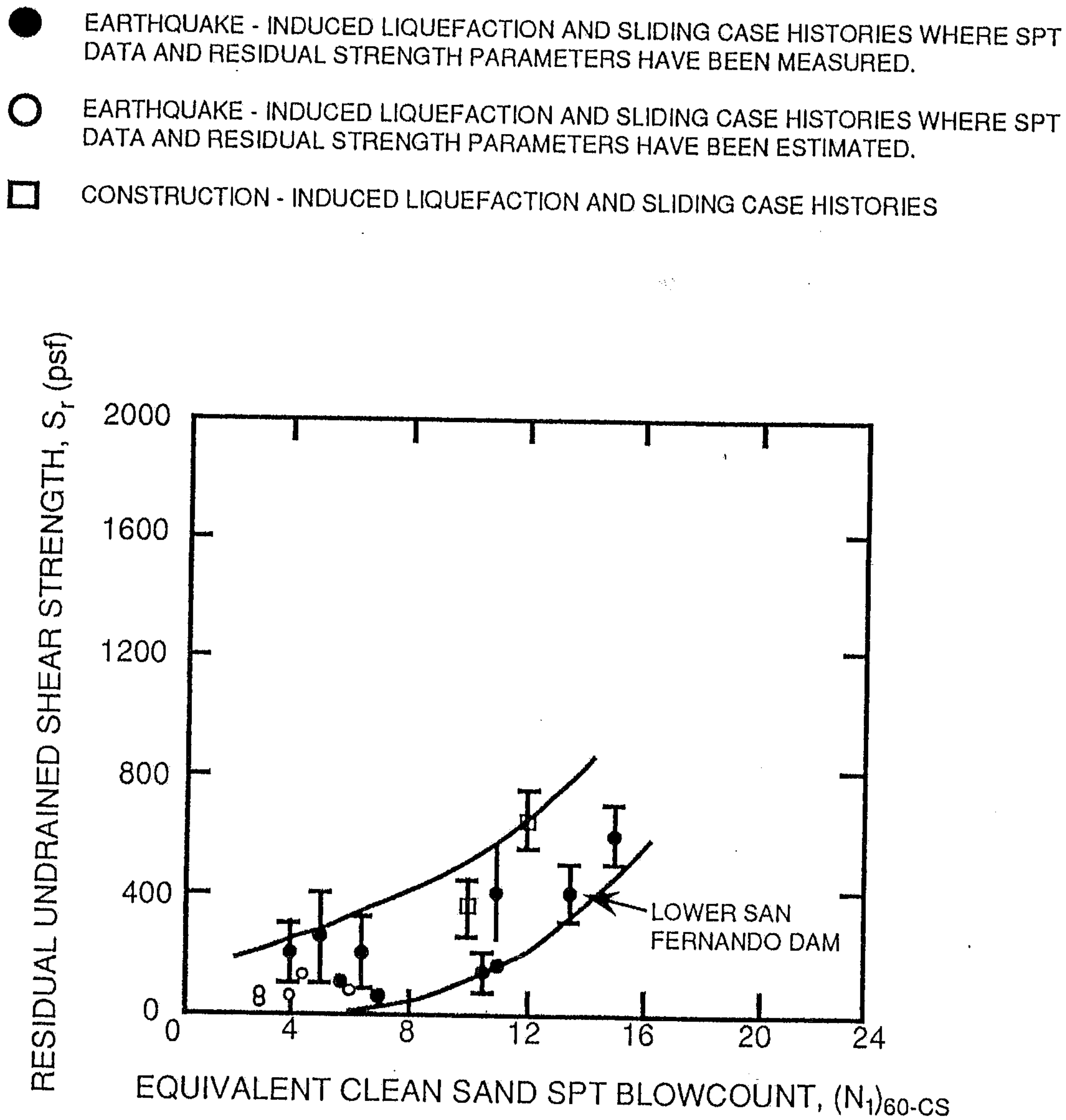
FIG. 13



Taken from:
Memorial Symposium
Proceedings, May 1990
Vancouver, B.C.
R.B. Seed & L.F. Harder

Upper & Lower Dams Evaluation
Wrangell, Alaska

EQUIVALENT CLEAN SAND
SPT BLOWCOUNT

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FIG. 14

APPENDIX A

FIELD EXPLORATION PROCEDURES AND RESULTS

APPENDIX A

FIELD EXPLORATION PROCEDURES AND RESULTS

A-1 Drilling

Our subsurface explorations at the two dams consisted of drilling nine borings and five probes at the locations shown on Figures 2 and 3. This drilling work was accomplished between the 5th and 18th of February, 1993. The borings, designated B-1 through B-9, were advanced with sampling to depth ranging between 27 and 77 feet. The probes, designated B-A, C, D, E and G were essentially borings with limited sampling. These probes were drilled in the toe area of the Upper Dam to define the depth to firm materials with the initial thought that rock varied considerably across the valley section. All holes were drilled with a track mounted Mobile B-47 drilling rig using hollow-stem continuous flight (3-3/8 inch ID by 6 inch OD) augers to advance the borings. In some instances, refusal of the auger was encountered or all 40 feet of the available auger was used requiring alternate drilling methods to continue advance of the deeper borings. When additional advance of the boring was necessary, rotary wash drilling techniques were supplemented with a tricone bit.

The drilling work was subcontracted to Wink Brothers Drilling of Juneau, Alaska. Their operations were continuously observed by an experienced engineer from Shannon & Wilson, Inc. The logs of these borings are presented as a part of this appendix, Figures A-1 through A-14.

The locations of all borings are shown on Figures 2 and 3 of the main text. These locations were determined by City Engineer, Jim Pung, after the borings were completed. Also referenced with these locations are the existing piezometers, one at each of the nine borings and four profiles of the slopes generally through most of the boreholes on the slope. These profiles shown on Figures 6 through 9 represent 1993 slope surface conditions. To obtain upslope measurements, holes were cut in the ice and soundings made. In all instances, these water depth soundings detected generally firm materials within a few inches of the surface (i.e. sedimentation was likely minimal).

A-2 Sampling

Sampling of the embankment and foundation soils in all borings was accomplished at regular depth intervals, typically every 2.5 to 5 feet. 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 modified penetration sampling procedures. In this test, a 140-pound hammer was used to drive the 2.5-inch O.D. samples, 18 or 24-inches into the undisturbed soil. The number of blows required to drive the samplers 12-inches after achieving the first 6-inch penetration is recorded on the logs. These values provide a means for estimating consistency (strength) for cohesive soils and density or compactness for granular soils. The soil recovered in the sampler was placed in airtight containers and brought to our laboratory for detailed examination and classifications testing, as necessary.

Two relatively undisturbed samples were obtained with 2.5-inch diameter thin-wall steel tubes, which were pushed into the soil at the bottom of the advancing boring by means of the hydraulic ram on the drill rig. The tube samples were sealed at the ends with plastic caps and delivered to our laboratory where the samples were extruded, classified and tested. The location and depth of these samples are shown on the boring logs.

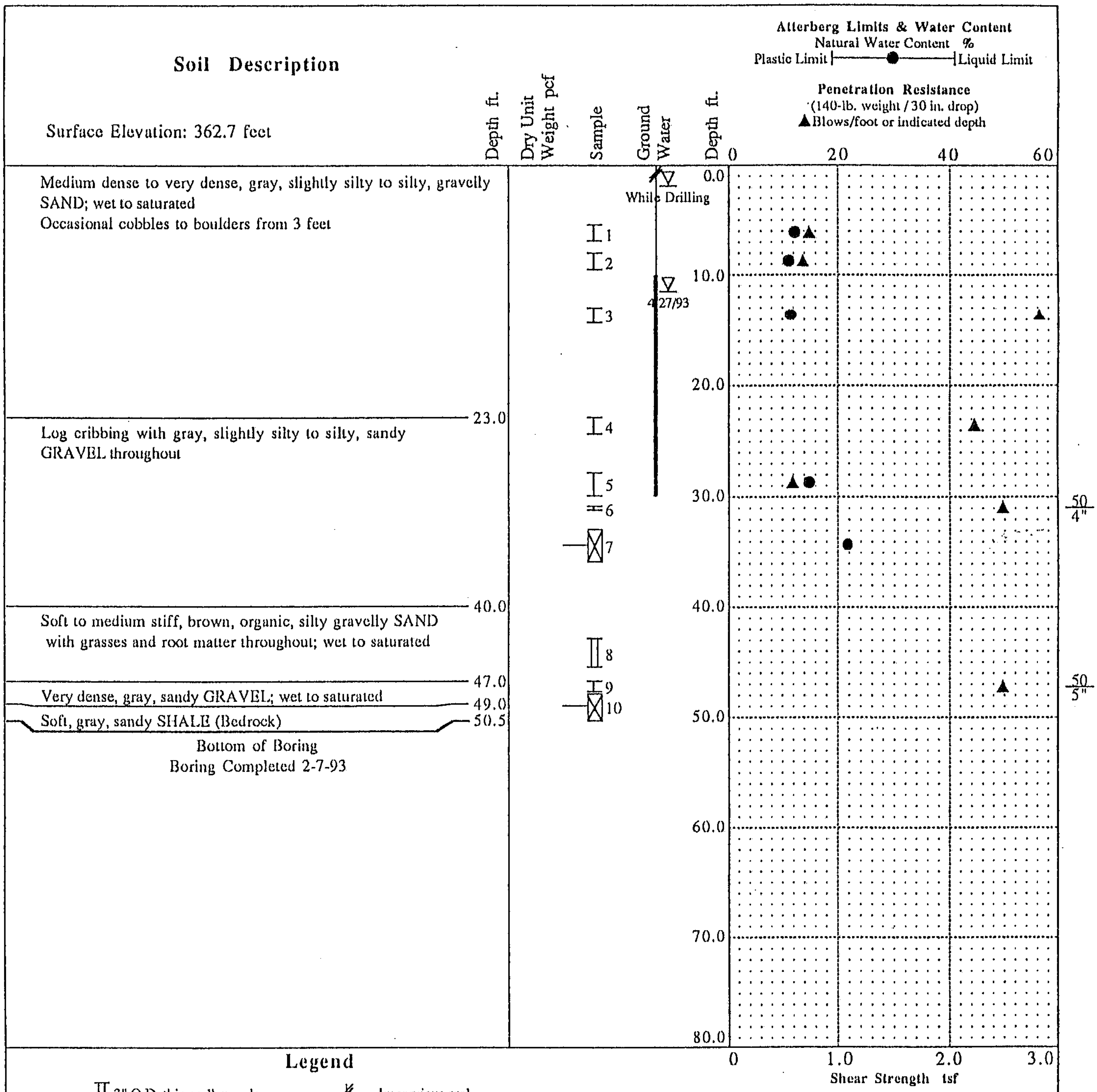
A-3 Piezometers

Piezometers were installed in all nine borings following their completion to measure depth to groundwater or piezometric pressures within the embankment or foundation materials. Each piezometer consists of a 1-inch diameter slotted plastic tip connected to 1-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 native materials and/or an impervious bentonite seal. The length of each porous tip and the locations of the different sealed zones relative to the tip for each installation are indicated on the boring logs.

The water level in these piezometers is measured by lowering an electrical water level reading device into the riser pipe until contact with the water is made. Water level readings from these piezometers were taken by Shannon & Wilson during the field effort and by City of Wrangell personnel on April 27, 1993. These data are summarized in Table 1, shown on the respective boring logs and included on the profiles, Figures 6 through 9.

Soil Description

Surface Elevation: 362.7 feet



Medium dense to very dense, gray, slightly silty to silty, gravelly SAND; wet to saturated
Occasional cobbles to boulders from 3 feet

23.0 Log cribbing with gray, slightly silty to silty, sandy GRAVEL throughout

40.0 Soft to medium stiff, brown, organic, silty gravelly SAND with grasses and root matter throughout; wet to saturated

47.0 Very dense, gray, sandy GRAVEL; wet to saturated

49.0 Soft, gray, sandy SHALE (Bedrock)

50.5

Bottom of Boring
Boring Completed 2-7-93

Legend

3" O.D. thin-wall sample	Impervious seal
2.5" O.D. split-spoon sample	Water level at indicated number of hours after drilling
▨ continuous sample	Piezometer tip
REC% / RQD% ☒ Rock core sample	
■ Grab sample	

Method of Measurement
- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-1

SHANNON & WILSON, INC.
Geotechnical Consultants

Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

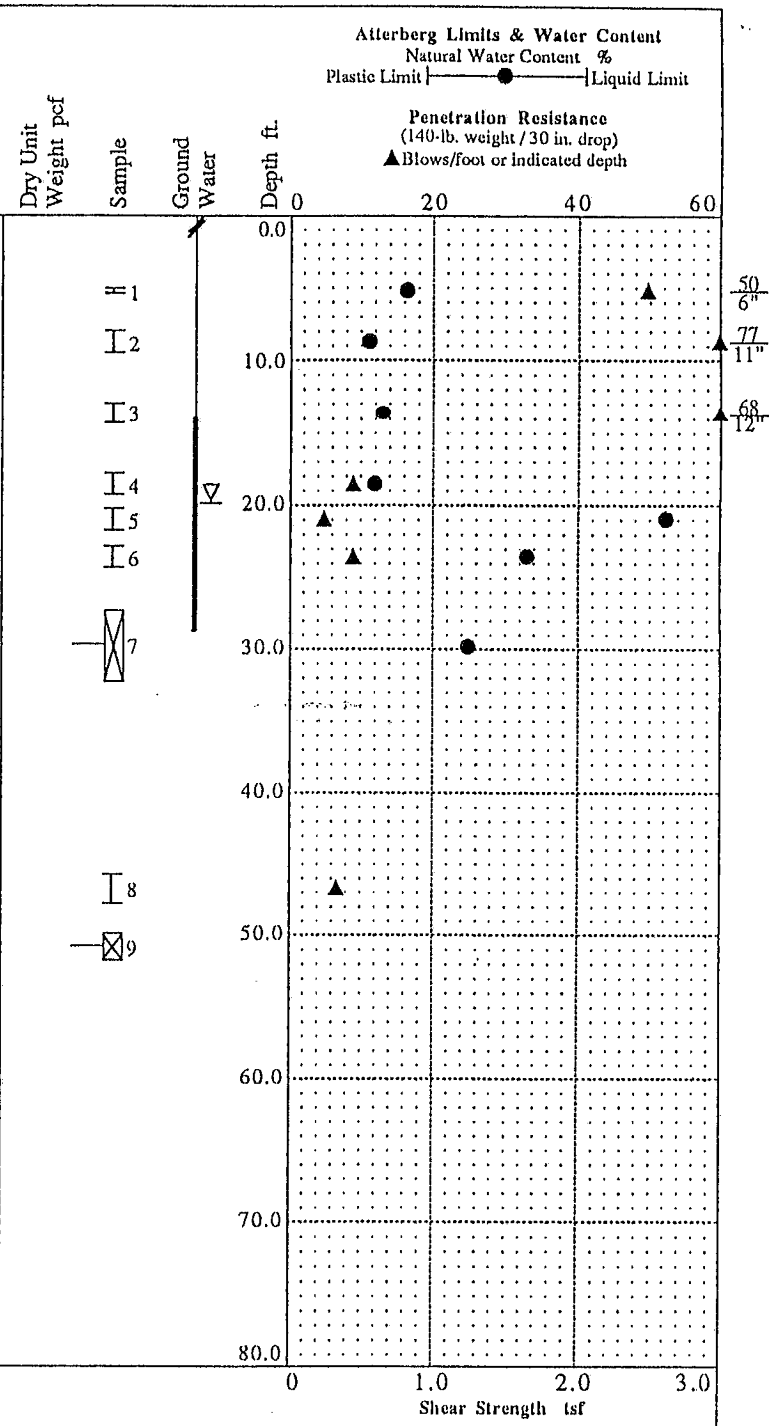
Fig. A1

Soil Description

Surface Elevation: 362.7 feet

Soil Description	Depth ft.
Dense to very dense, gray, silty, sandy GRAVEL to gravelly SAND; moist to wet	18.0
Medium stiff, brown to gray, slightly gravelly silty SAND to sandy SILT; moist to wet Organics (root matter, woodchips, etc., from 20.5 to 24.5)	25.5
Dense, gray, silty, gravelly SAND; moist to wet	27.5
Log cribbing	49.5
Soft, gray, sandy, SHALE (Bedrock)	52.0

Bottom of Boring
Boring Completed 2-5-93



Legend

3" O.D. thin-wall sample
2.5" O.D. split-spoon sample
▨ continuous sample
REC% RQD% ☒ Rock core sample
■ Grab sample

Impervious seal
∇ 0hr Water level at indicated number of hours after drilling
Piezometer tip

Method of Measurement
○ Unconfined Compression
△ Unconsolidated - Undrained triaxial compression
◇ Torvane
☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-2

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Geotechnical Consultants

Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

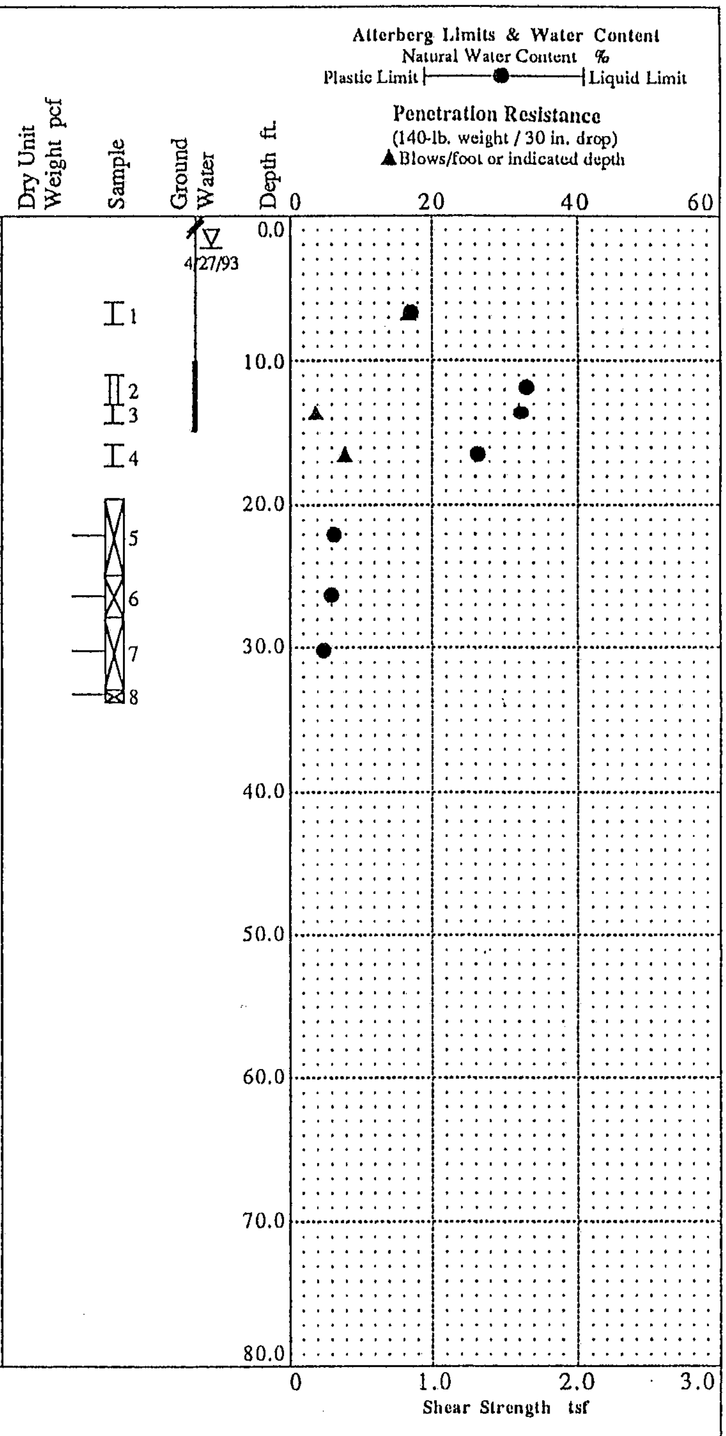
Fig. A2

Soil Description

Surface Elevation: 336.3 feet

Soil Description	Depth ft.
Medium dense, gray, sandy, silty GRAVEL, with some cobbles to boulders; wet	8.5
Loose, gray, slightly gravelly, slightly silty to silty SAND; wet to saturated	16.0
Loose to dense, gray, gravelly, silty SAND; wet to saturated Organics mixed into matrix in upper part of zone	19.8
Dense to very dense, gray, sandy GRAVEL [TILL];	25.0
Soft, gray SLATE (highly weathered bedrock)	34.0
Soft, sandy SHALE or SLATE (Bedrock)	34.8

Bottom of Boring
Boring Completed 2-8-93



Legend

- 3" O.D. thin-wall sample
- 2.5" O.D. split-spoon sample
- ▨ continuous sample
- REC% / RQD% ☒ Rock core sample
- ■ Grab sample
- Impervious seal
- ▽ 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- □ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-3

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Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A3

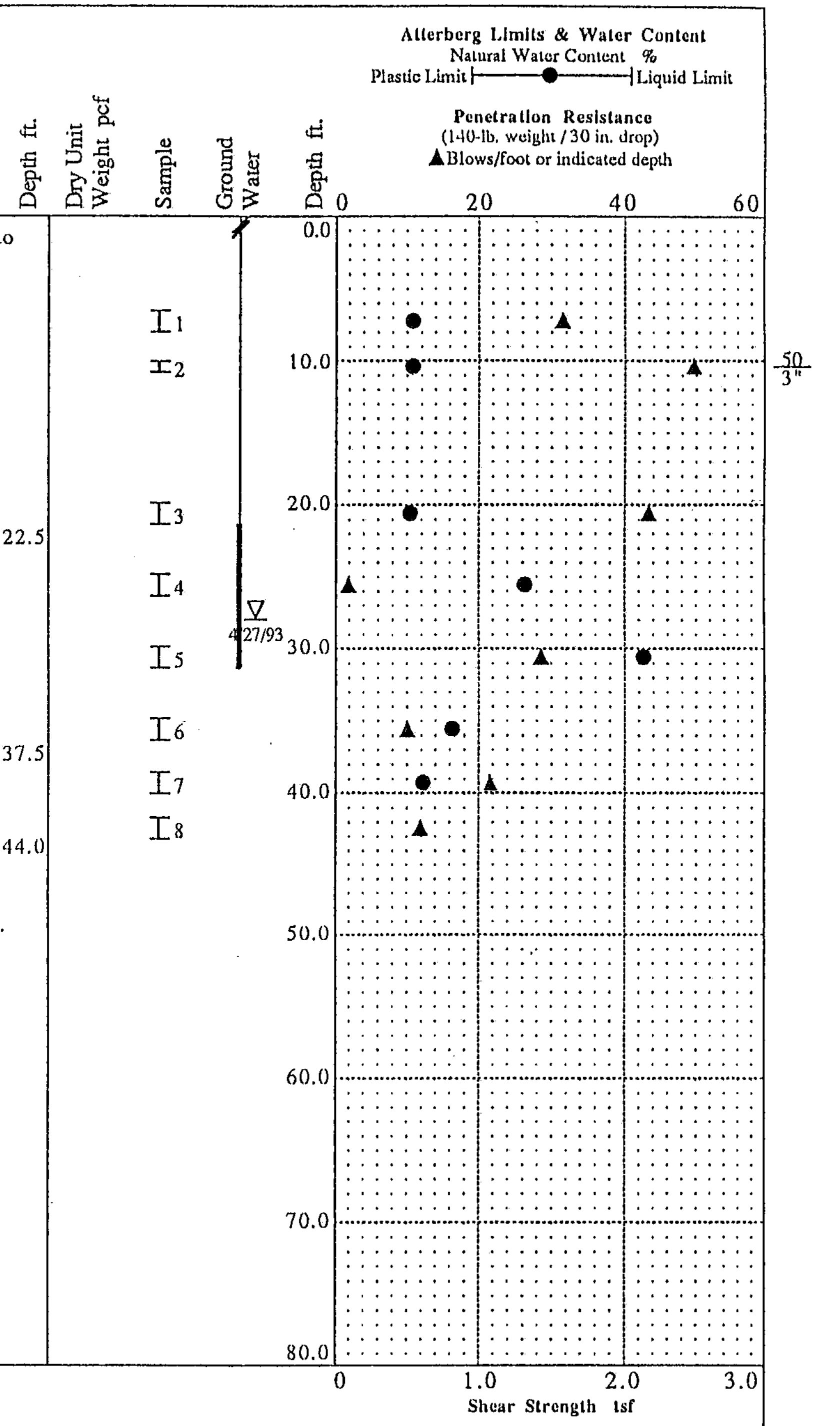
Soil Description

Surface Elevation: 361.3 feet

Soil Description	Depth ft.
Dense, gray, slightly silty to silty, sandy GRAVEL with some to many cobbles to boulders; moist or wet	22.5
Log cribbing with variable silty, sandy GRAVEL to gravelly SAND or organic SILT intermixed in soil matrix	37.5
Medium dense, gray, slightly silty, sandy GRAVEL; wet to saturated (extremely weathered bedrock)	44.0

Bottom of Boring
Boring Completed 2-9-93

Ground water contact not certain\drilling with water from 9.5 ft.



Legend

- Ⅱ 3" O.D. thin-wall sample
- I 2.5" O.D. split-spoon sample
- ▨ continuous sample
- REC% RQD% ☒ Rock core sample
- ■ Grab sample
- Impervious seal
- ∇ 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-4

SHANNON & WILSON, INC.
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Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

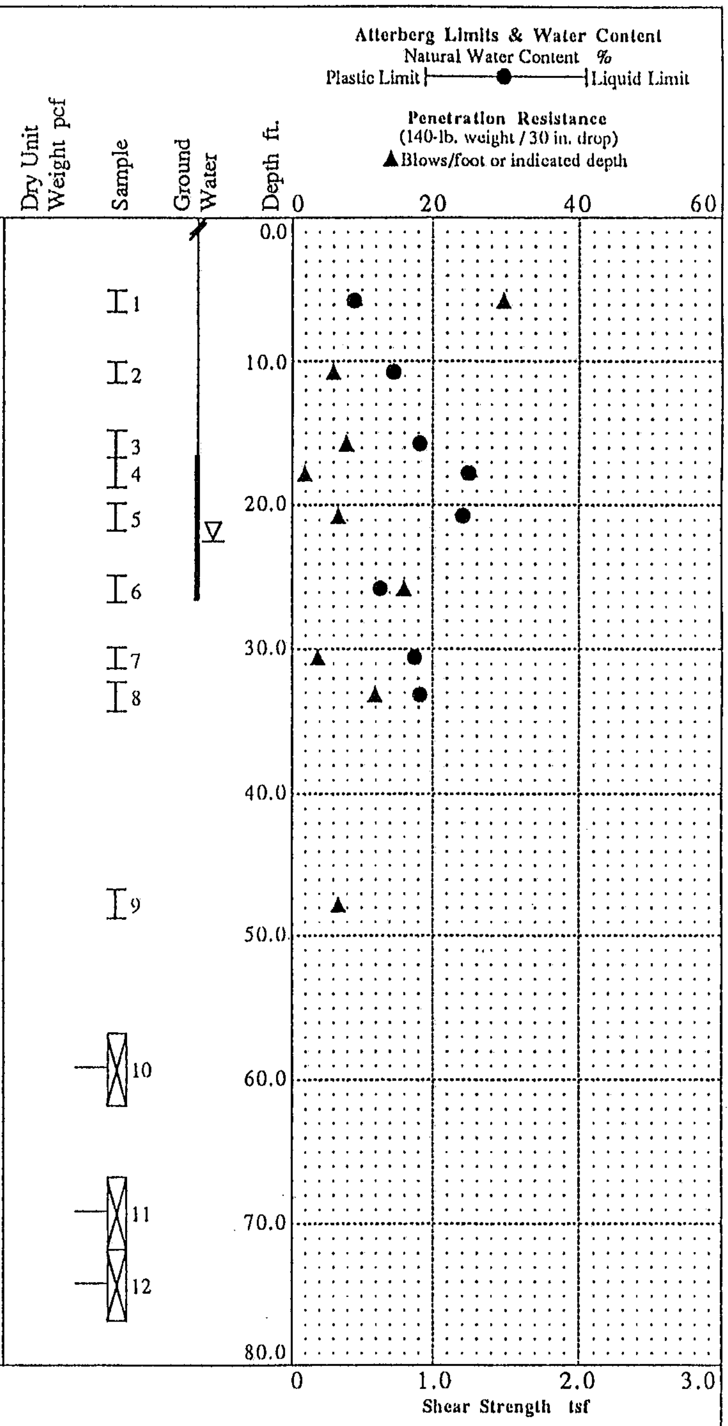
Fig. A4

Soil Description

Surface Elevation: 362.8 feet

Soil Description	Depth ft.
Dense, gray, slightly silty, sandy GRAVEL with some to many cobbles to boulders; wet	8.0
Loose to medium dense, gray, slightly silty to silty, sandy GRAVEL, to silty, gravelly SAND, with some cobbles to boulders throughout	57.0
Very dense, gray, silty, gravelly SAND, with cobbles to boulders throughout	62.0
Medium dense, gray, clean SAND	67.0
Medium stiff, brown, organic SILT; moist to wet	72.0
Stiff, gray SILT; moist to wet	77.0

Bottom of Boring - Boring Completed 2-11-93



Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

Legend

3" O.D. thin-wall sample	Impervious seal
2.5" O.D. split-spoon sample	Water level at indicated number of hours after drilling
continuous sample	Piezometer tip
REC% / RQD% Rock core sample	
Grab sample	

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-5

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Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

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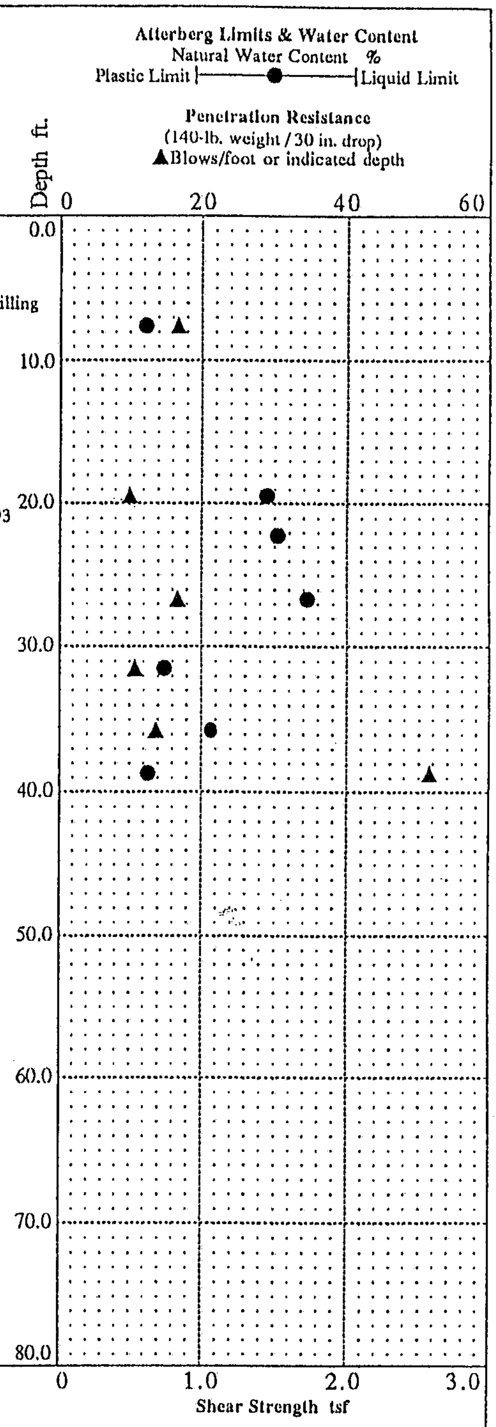
Fig. A5

Soil Description

Surface Elevation: 361.7 feet

Soil Description	Depth ft.	Dry Unit Weight pcf	Sample	Ground Water
Surface - 1' of snow Medium dense, gray, slightly silty to silty, sandy GRAVEL with some to many cobbles to boulders; wet			Ɪ1	∇ While Drilling
Log cribbing at 13' to 14' and 17' - 18' with soft soils from 14' to 17'	13.0			
Medium dense to very dense, gray, slightly silty to silty, sandy GRAVEL; wet to saturated	17.0		Ɪ2 II3 Ɪ4 Ɪ5 Ɪ6 Ɪ7	∇ 4/27/93
Bottom of Boring Boring Completed 2-14-93	39.8			



Legend

- II 3" O.D. thin-wall sample
- Ɪ 2.5" O.D. split-spoon sample
- ▨ continuous sample
- RQC% RQD% ☒ Rock core sample
- ■ Grab sample
- ⫽ Impervious seal
- ∇ 0hr Water level at indicated number of hours after drilling
- │ Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-6

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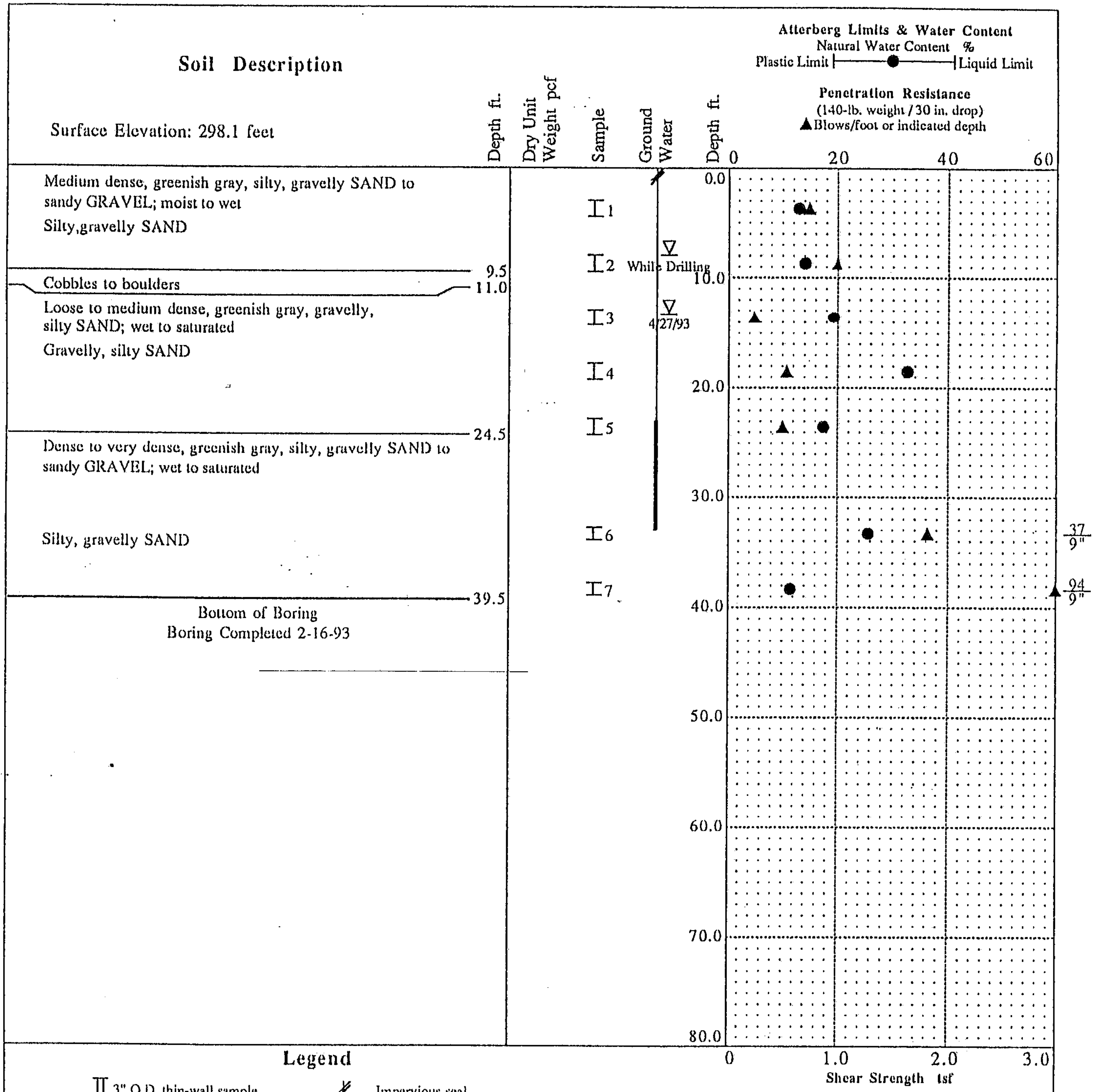
Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A6



Legend

- Ⅱ 3" O.D. thin-wall sample
- I 2.5" O.D. split-spoon sample
- ▨ continuous sample
- REC% RQD% ☒ Rock core sample
- ■ Grab sample
- Impervious seal
- ▽ 0hr Water level at indicated number of hours after drilling
- | Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-7

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Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A7

Soil Description

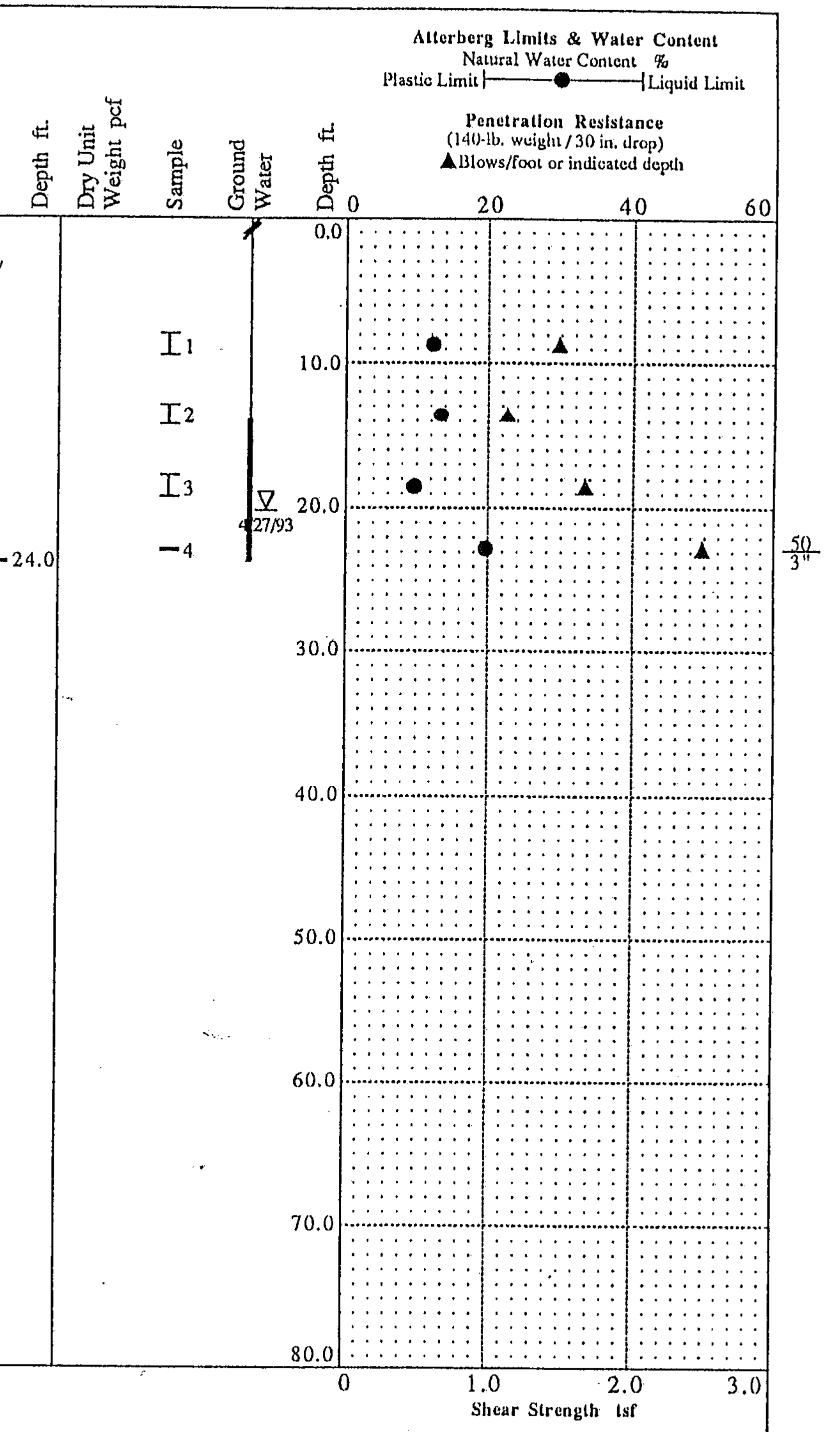
Surface Elevation: 297.6 feet

Surface - 1' of snow
Dense, brown to greenish gray, silty, gravelly SAND to sandy GRAVEL; moist to wet
Occasional cobbles to boulders throughout

Auger and sampler refusal at 24.0 ft.

Bottom of Boring
Boring Completed 2-17-93

No groundwater encountered during drilling



Legend

Ⅱ 3" O.D. thin-wall sample
I 2.5" O.D. split-spoon sample
▨ continuous sample
REC% / RQD% ☒ Rock core sample
■ Grab sample

Impervious seal
▽ 0hr Water level at indicated number of hours after drilling
Piezometer tip

Method of Measurement
○ Unconfined Compression
△ Unconsolidated - Undrained triaxial compression
◇ Torvane
☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-8

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Upper and Lower Dams
Wrangell, Alaska

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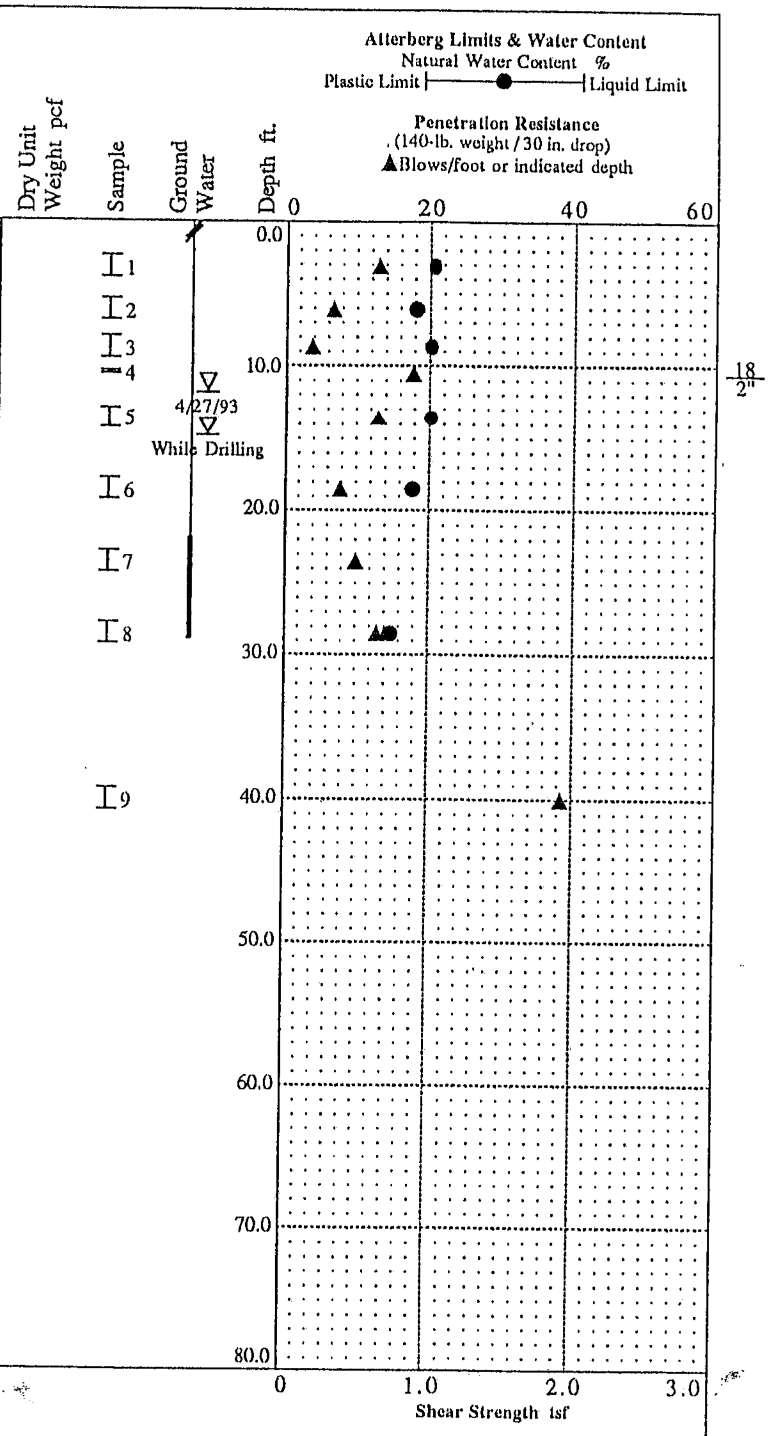
Fig. A8

Soil Description

Surface Elevation: 287.9 feet

Soil Description	Depth ft.
Brown to gray, loose, silty, gravelly SAND; moist to wet	
Soft to stiff, gray, sandy SILT; moist to wet Some organics throughout	7.0
Log cribbing with medium stiff, gray, sandy SILT in between logs	14.0
Loose to dense, gray, silty, gravelly SAND; wet to saturated	18.5
Bottom of Boring Boring Completed 2-18-93	41.0



Legend

- 𝙸 3" O.D. thin-wall sample
- I 2.5" O.D. split-spoon sample
- ▨ continuous sample
- RBC%/RQD% ☒ Rock core sample
- ■ Grab sample
- Impervious seal
- ▽ 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- □ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-9

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Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A9

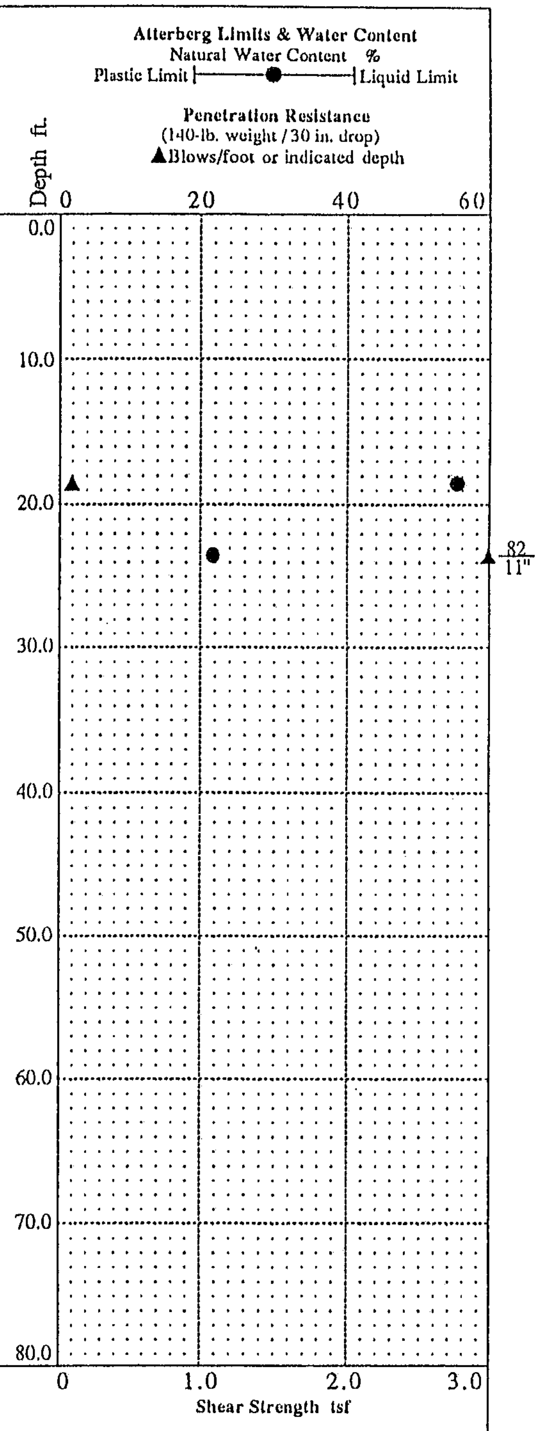
Soil Description

Surface Elevation: 333.0 feet

Soil Description	Depth ft.	Dry Unit Weight pcf	Sample	Ground Water
Boulders	5.0			
Very loose to loose, brown, silty SAND Cobbles to boulders throughout	18.0			
Very loose, gray, slightly gravelly, silty SAND Woodpieces at 19.0 feet	23.0		I 1	
Very dense, gray, sandy SILT to silty SAND with fractured rock; soft, gray slate (highly weathered bedrock)	24.5		I 2	

Bottom of Boring
Boring Completed 2-13-93

Depth to groundwater not noted during drilling



Legend

- 3" O.D. thin-wall sample
- 2.5" O.D. split-spoon sample
- continuous sample
- REC% / RQD% Rock core sample
- Grab sample
- Impervious seal
- ▽ 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-A

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Wrangell, Alaska

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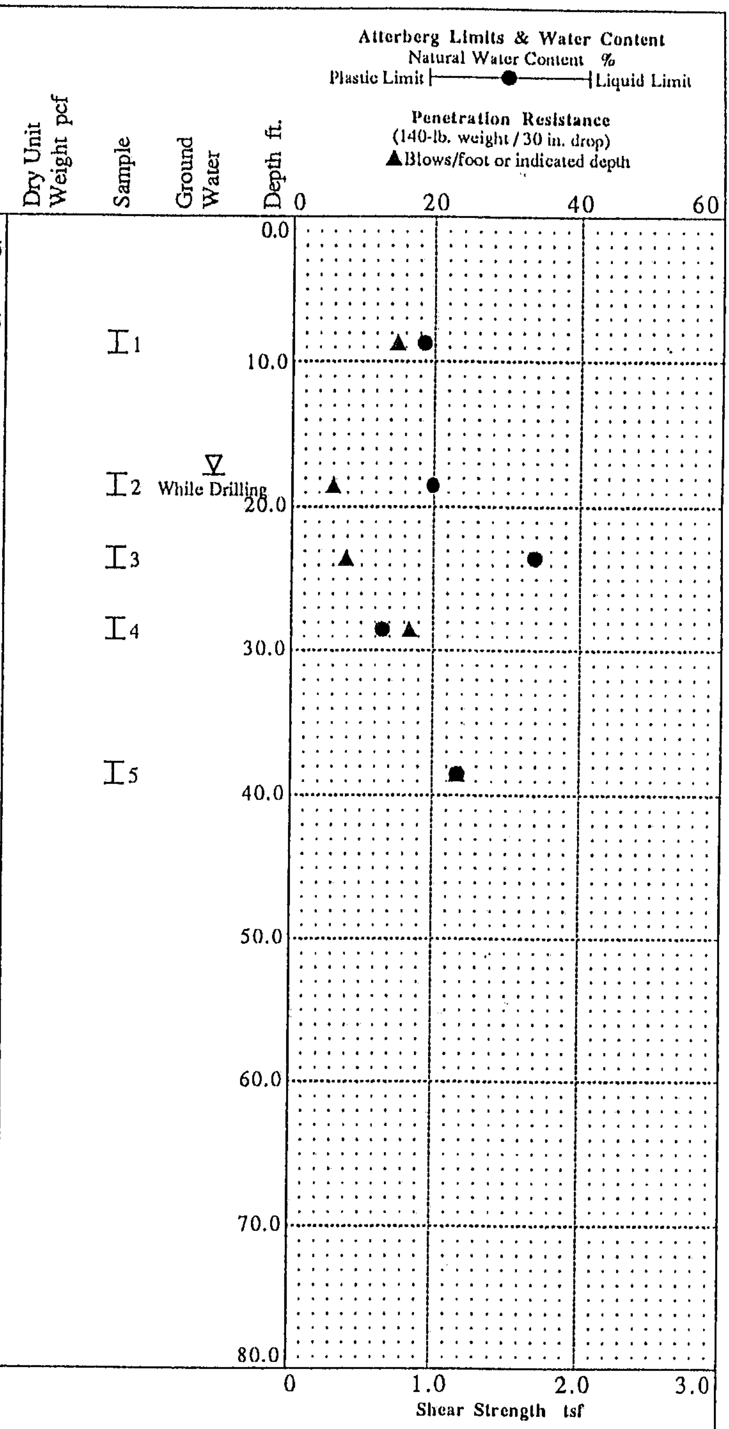
Fig. A10

Soil Description Surface Elevation: 343.7 feet	Depth ft.
Loose, silty sandy GRAVEL with organics	2.5
Cobbles to boulders	7.5
Loose to medium dense, silty, sandy GRAVEL to gravelly SAND with occasional cobbles to boulders Trace organics at 23' to 24.5' Fractured rock at 29.5'	39.5
Bottom of Boring Boring Completed 2-13-93	



Legend

- 3" O.D. thin-wall sample
- 2.5" O.D. split-spoon sample
- continuous sample
- REC% / RQD% Rock core sample
- Grab sample
- Impervious seal
- 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-C

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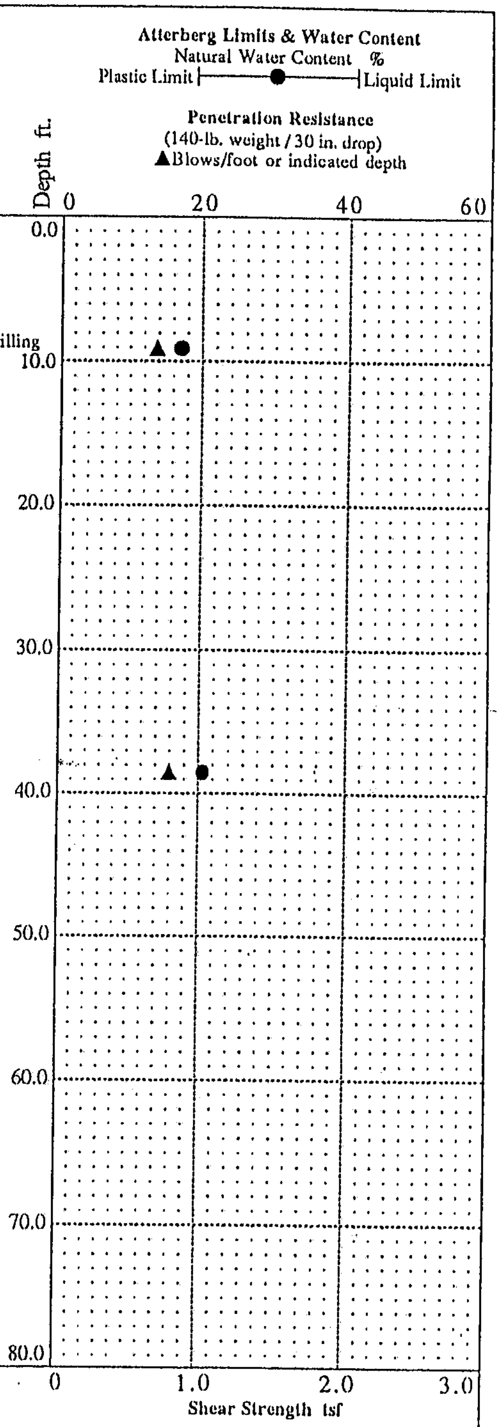
Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A11

Surface Elevation: 355.9 feet

Soil Description	Depth ft.	Dry Unit Weight pcf	Sample	Ground Water
Loose, brown, sandy, organic SILT; wet	1.5			
Cobbles to boulders	4.0			
Medium dense, gray, silty, sandy GRAVEL to silty, gravelly SAND; wet Occassional cobbles and boulders from 10' to 38'	39.5		I 1 I 2	∇ While Drilling
Bottom of Boring Boring Completed 2-13-93				



Legend

- Ⅱ 3" O.D. thin-wall sample
- I 2.5" O.D. split-spoon sample
- ▨ continuous sample
- REC% / RQD% ☒ Rock core sample
- ■ Grab sample
- Impervious seal
- ∇ 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-D

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Upper and Lower Dams
Wrangell, Alaska

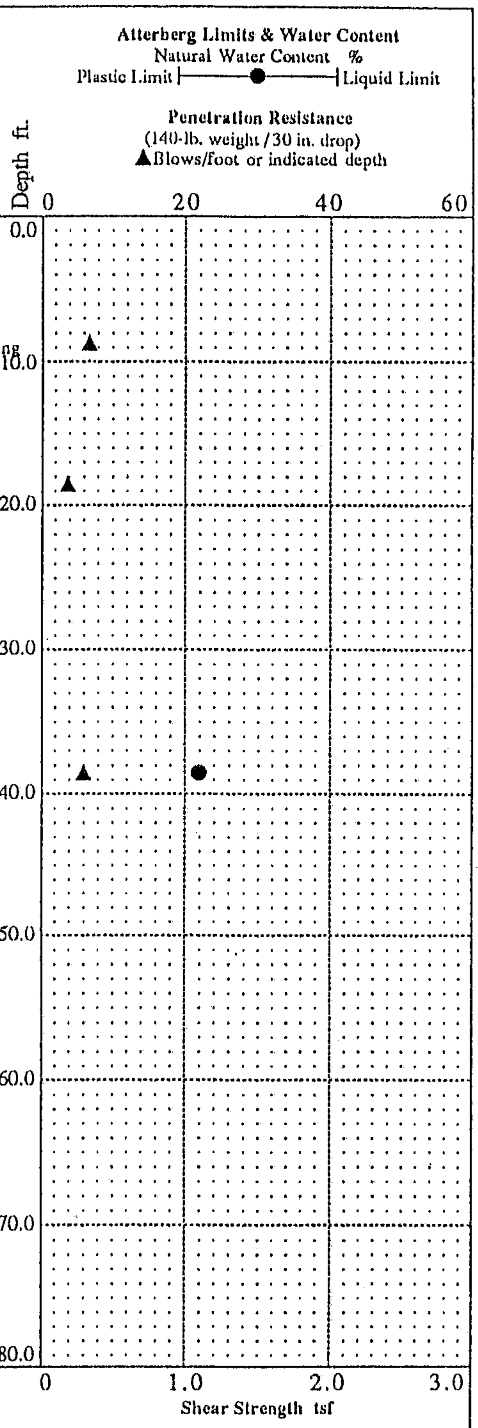
A-494

Fig. A12

Soil Description

Surface Elevation: 333.0 feet

Soil Description	Depth ft.	Dry Unit Weight pcf	Sample	Ground Water
Loose to medium dense, gray, silty, sandy GRAVEL; moist to wet Cobbles on surface				
Loose, gray, slightly gravelly, silty SAND to sandy SILT; wet Wood from 9' to 9.5' and 19.2' to 19.5' Cobbles to boulders at 17' and 23'	8.0		1 2	∇ While Drilling
Bottom of Boring Boring Completed 2-14-93	39.5		3	



Legend

3" O.D. thin-wall sample	Impervious seal
2.5" O.D. split-spoon sample	∇ 0hr Water level at indicated number of hours after drilling
▨ continuous sample	Piezometer tip
REC% RQD% ☒ Rock core sample	
■ Grab sample	

Method of Measurement
- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-E

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Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A13

Soil Description

Surface Elevation: 362.7 feet

Depth ft. | Dry Unit Weight pcf | Sample | Ground Water

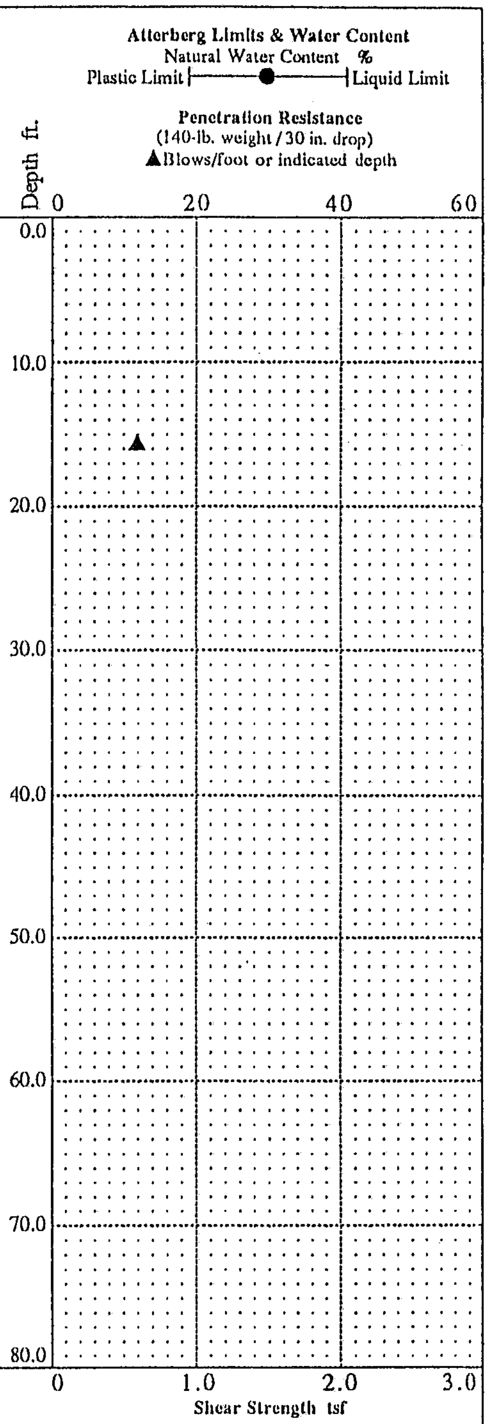
Loose to medium dense, gray, silty, sandy GRAVEL to gravelly SAND
Cobbles and boulders at 12' and 14'
Wood/logs encountered at 15' to 16.5', 20' to 20.8' and 23.5' to 24'

I 1

24.0

Bottom of Boring
Boring Completed 2-15-93

Depth to groundwater not noted during drilling



Legend

- Ⅱ 3" O.D. thin-wall sample
- I 2.5" O.D. split-spoon sample
- ▨ continuous sample
- REC% RQD% ☒ Rock core sample
- ■ Grab sample
- Impervious seal
- ∇ 0hr Water level at indicated number of hours after drilling
- Piezometer tip

Method of Measurement

- ○ Unconfined Compression
- △ Unconsolidated - Undrained triaxial compression
- ◇ Torvane
- ☐ Pocket Penetrometer

1. Groundwater levels may vary with time, precipitation, infiltration, and other factors
2. The stratification lines represent approximate soil boundaries. Actual boundary may be transitional.

Log of Boring B-G

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Stability Evaluation
Upper and Lower Dams
Wrangell, Alaska

A-494

Fig. A14

APPENDIX B

LABORATORY TEST PROCEDURES AND RESULTS

Laboratory tests were performed on selected soil samples from the borings and probes to determine those physical characteristics and engineering properties pertinent to the stability studies. The following sections discuss each of the tests performed for the various properties required.

B-1 Classification Tests

Many select samples were carefully examined and classified in the laboratory and their descriptions were checked against those in the field. From this information and from observed changes in drilling characteristics, the detailed boring logs were prepared which show a generalized description of each material encountered. Following the visual classification of each sample, a portion of the material was then weighed and oven-dried to determine its natural water content. Water contents are presented graphically on the boring logs, Figures A-1 through 14. They are also summarized in Table B-1. Samples that appear to possess organics were tested further as a follow-on effort to water contents measurements. The dry specimen from the water contents were burned where the organics were turned to ash. From this change in weight, the organic content was calculated. These results are also summarized in Table B-1.

Grain-size analyses were conducted on 28 selected samples. The soil specimen were tested to obtain estimates of the percent fines or permeability and the general gradation characteristics of the coarser soils in the various zones of the two dams and foundations. These tests were performed in accordance with the test method described in Laboratory Soils Testing, Department of the Army, Pages V-1 through V-25 (EM 1110-2-1906). The results of these tests are presented in Figures B-1 through B-8. The Figure B-8 grain size curves are bulk samples of the embankments for each dam and represent support results for the follow-on compaction curves in Figure B-9.

Compaction tests were conducted on two granular bulk samples of the embankment materials for each dam. These tests were conducted to determine the optimum moisture content of the embankment soils for comparison with the natural moisture contents of the in situ soils. If the Upper Dam is breached to install a new pipe, the suitability of excavated materials for re-use can be evaluated from these results. Both of these tests were conducted using procedures described in AASHTO T99-70 using the three point method. The results of these tests are presented in Figure B-9.

B-2 Strength Tests

Triaxial compression tests were performed on recompacted specimen from the embankment and foundation materials to determine strength characteristics for stability analyses.

Consolidated Undrained Triaxial Compression Tests

Consolidated undrained triaxial compression tests (R-test) were accomplished on eight specimens from the various foundation and embankment materials present at the dam sites. For lack of sufficient material from the sampler and the inability to recover undisturbed samples, testing had to be limited to recompacted samples prepared from like materials from several samples in a given boring. Each cylindrical specimen was prepared by tamping the soil at the natural water content in 5 layers to fill a cylinder of predetermined dimensions. The ends were then squared off and the final dimensions determined. Next, each specimen was encased in a rubber membrane and placed in a triaxial chamber. The specimen was saturated with back pressure and allowed to consolidate under preselected confining pressures. After consolidation, the drain valves were closed and each specimen was slowly loaded to failure while pore pressure measurements are taken. The results of these tests are summarized as Mohr's Circles in Figures B-10 and B-11.

Since the crib and foundation materials have similar properties, these specimens were placed on Figure B-10 for comparison. Similarly, the denser embankment materials are summarized in Figure B-11. The details of each of these tests are presented in Figures B-12 through 20.

Table B-1
Moisture and Organic Contents

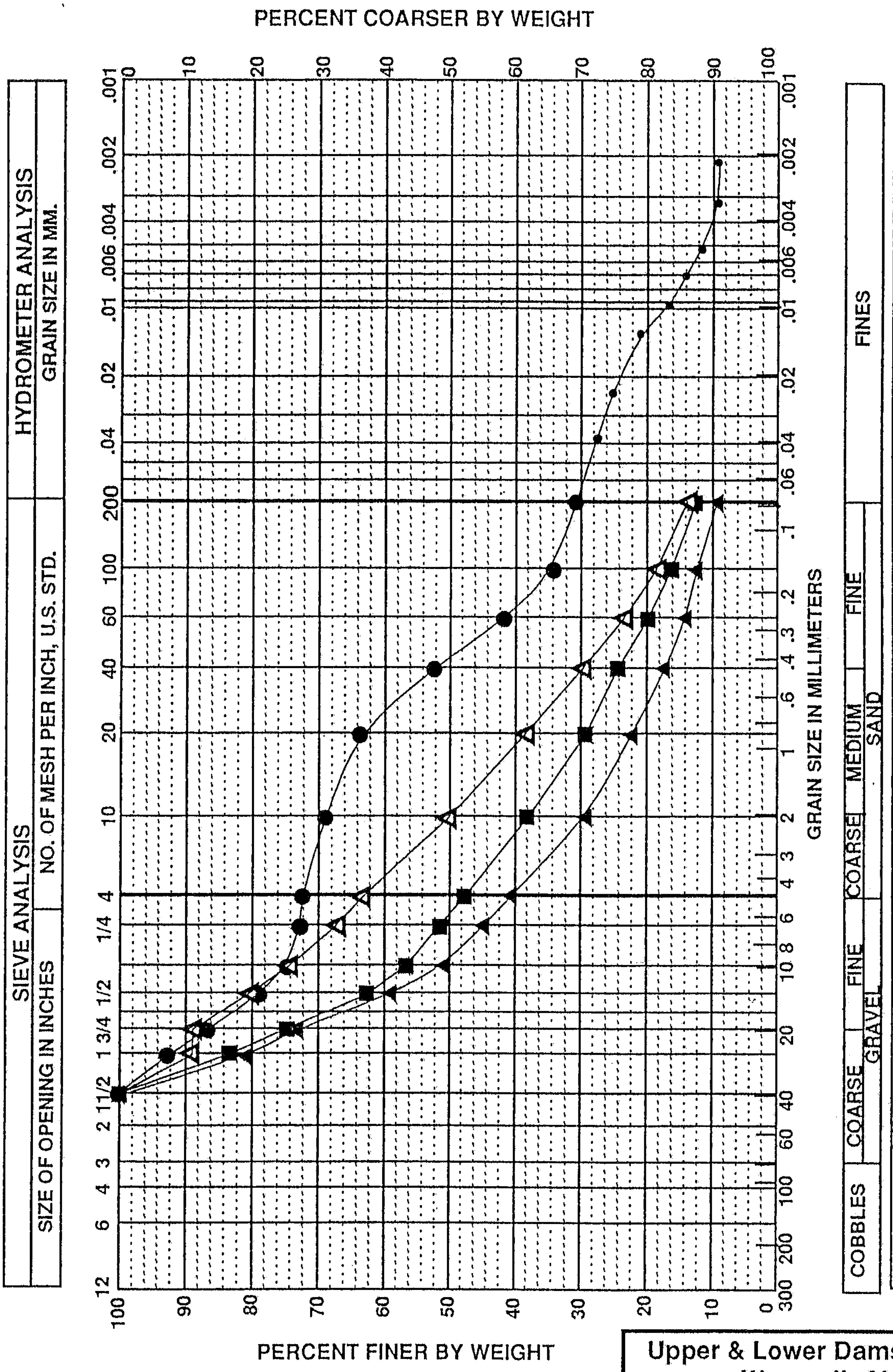
Boring No.	Sample No.	Natural Water Content %	Remarks
B-1	S1	12.4	
	S2	11.3	
	S3	11.6	
	S4	82	Sample of Log Crib
	S5	14.9	
	S6	80.4	Sample of Log Crib
	S7	22.2	
	S9	103.8	Moss and Wood in sample
	S9	5.8	
B-2	S1	16.4	
	S2	11.4	
	S3	13	
	S4	12.2	
	S5	52.8	Organic Content of 7.6%
	S6	33.7	
	S7	25.5	
	S8	18.5	
B-3	S1	17.1	
	S2	33.4	
	S3	32.7	Organic Content of 2.3%
	S4	26.8	
	S5	6.6	
	S6	6.2	
	S7	5.1	
B-4	S1	10.8	
	S2	10.9	
	S3	10.5	
	S4	26.8	
	S5	43.3	Organic Content of 7.3%
	S6	16.6	Organic Content of 3.7%
	S7	12.4	
	S8	15.2	
B-5	S1	9	
	S2	14.8	
	S3	18.2	
	S4	25.2	
	S5	24.8	
	S6	12.6	
	S7	17.7	
	S9	18.5	
	S9	20.3	
	S10	NA	
	S11	102.8	Organic Content of 64.2%
	S12	3.8	

Table B-1
Moisture and Organic Contents

Boring No.	Sample No.	Natural Water Content %	Remarks
B-6	S1	12.5	
	S2	29.4	
	S3	30.9	Organic content of 1.6%
	S4	35.1	
	S5	15.1	
	S6	21.8	
	S7	12.7	
B-7	S1	13.2	
	S2	14.4	
	S3	19.4	
	S4	33.2	
	S5	17.5	
	S6	26	Wood in Sample
	S7	11.7	
B-8	S1	12.4	
	S2	13.6	
	S3	9.8	
	S4	19.7	
B-9	S1	20.8	
	S2	18.5	
	S3	20.6	
	S4	167.7	Sample of Log Crib
	S5	20.6	
	S6	17.8	
	S7	76.6	Sample of Log Crib
	S8	15	
B-A	S1	55.9	Organic Content of 7.1%
	S2	22.2	
B-C	S1	18.8	
	S2	20	
	S3	34.4	
	S4	13.2	
	S5	23.9	
B-D	S1	17.4	
	S2	20.8	
B-E	S1	60.2	Sample contained wood
	S2	73.8	Sample contained wood
	S3	22.3	
B-G	S1	143.2	Sample of Log Crib

APPENDIX B

LABORATORY TEST PROCEDURES AND RESULTS



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-1, S-2	8.0 - 9.5	SM	△ Gray, silty, gravelly SAND (embankment)	11.3			
B-1, S-5	28.0 - 30.0	GW-GM	◀ Gray, slightly silty, sandy GRAVEL (crib)	14.9			
B-1, S-8	43.0 - 45.6	GM	● Brown, silty, gravelly SAND, w/ moss & wood (foundation)	103.8			
B-2, S-3	13.0 - 14.5	GM	■ Gray, silty, sandy GRAVEL (embankment)	13.0			

Upper & Lower Dams Evaluation
Wrangell, Alaska

GRAIN SIZE CLASSIFICATION

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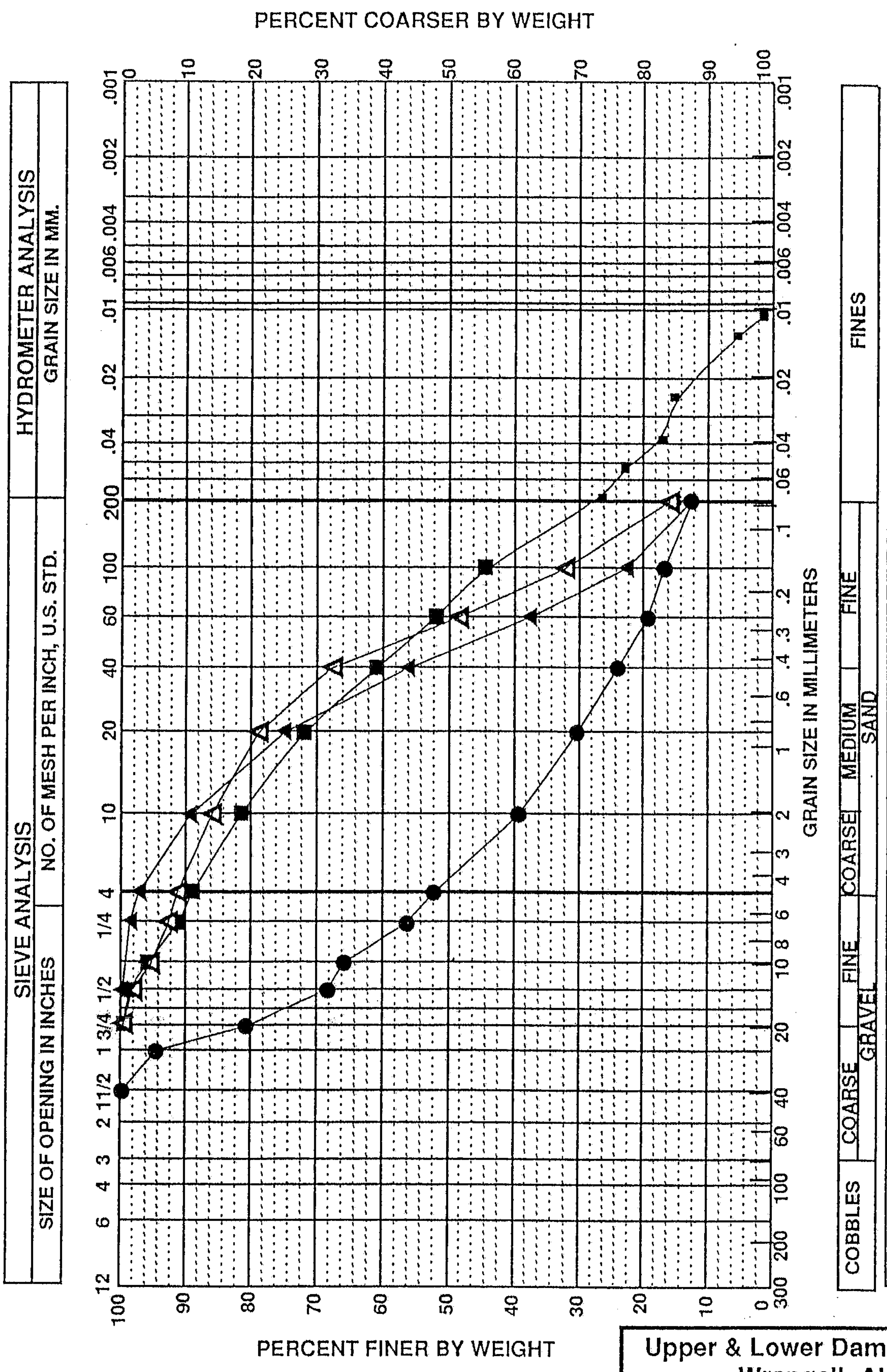
Fig. B-1



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-2, S-5	20.5 - 22.0	SM	■ Brown, slightly gravelly, silty SAND (crib)	52.8			
B-3, S-2	11.0 - 13.0	SM	△ Gray, slightly gravelly, silty SAND (foundation)	33.4			
B-3, S-3	13.0 - 14.5	SW-SM	▲ Gray, slightly silty SAND (foundation)	32.7			
B-4, S-3	20.0 - 21.5	GW-GM	● Gray, slightly silty, sandy GRAVEL (embankment)	10.5			

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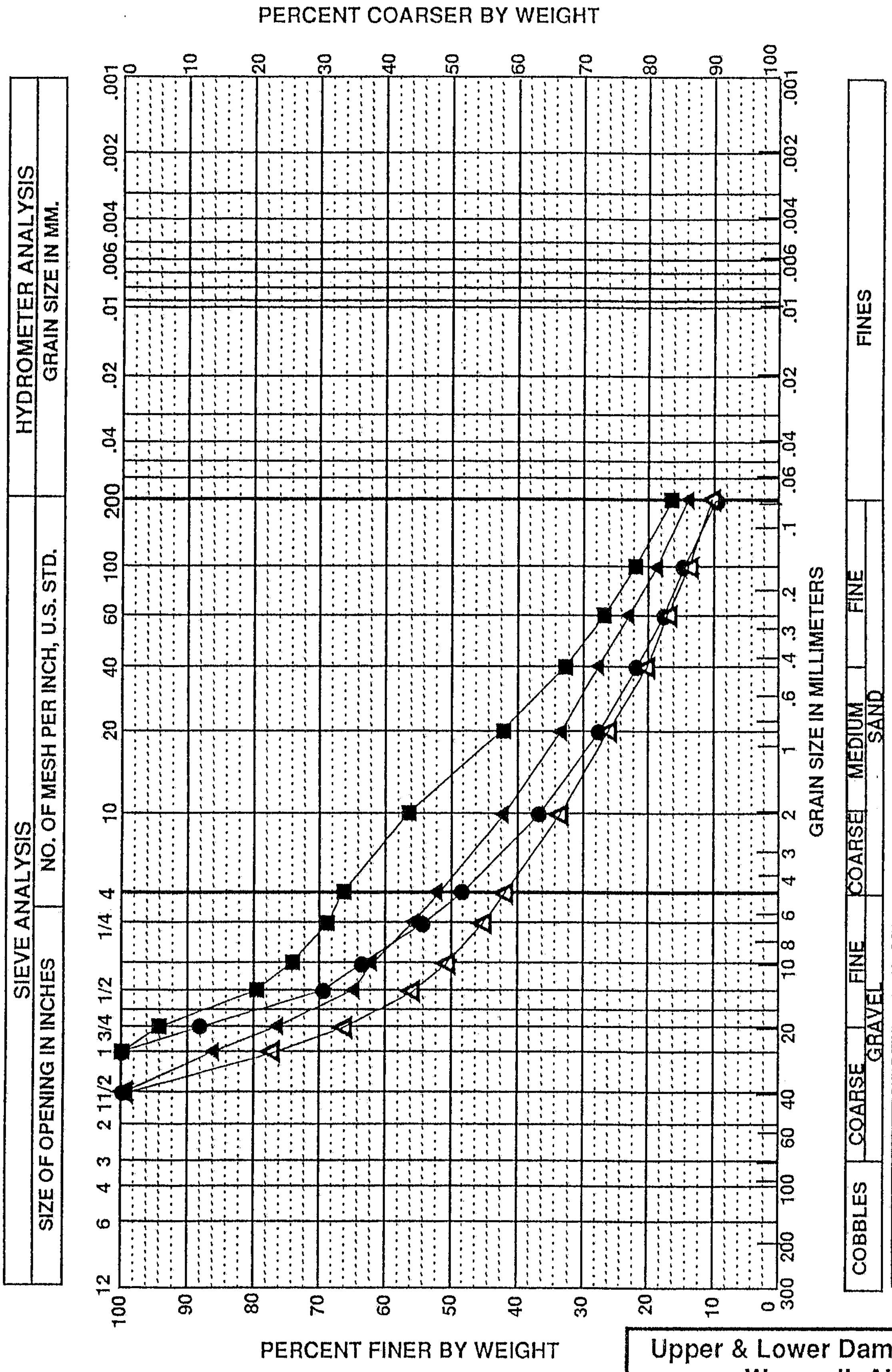
Fig. B-2



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-4, S-5	30.0 - 31.5	SW	■ Gray, silty, gravelly SAND (crib)	43.3			
B-4, S-7	38.8 - 40.3	GW-GM	△ Gray, slightly silty, sandy GRAVEL (foundation)	12.4			
B-5, S-2	10.0 - 11.5	GW-GM	● Gray, slightly silty, sandy GRAVEL (embankment)	14.8			
B-5, S-5	20.0 - 22.0	GW	▲ Gray, silty, sandy GRAVEL (crib)	24.8			

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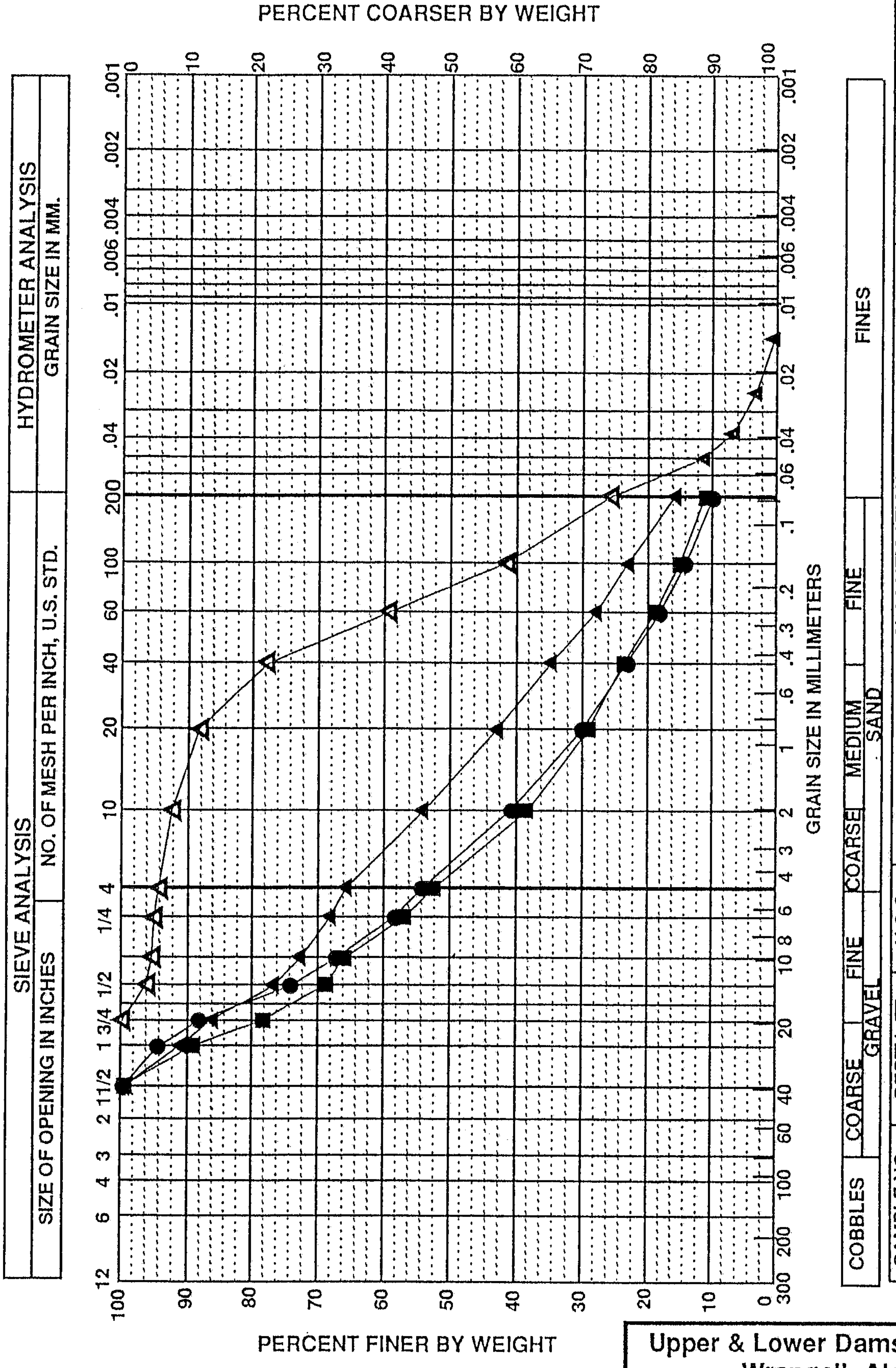
Fig. B-3



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-5, S-9	47.0 - 49.9	SM	▲ Gray, silty, gravelly SAND (foundation)	20.3			
B-6, S-1	7.0 - 8.5	GW-GM	● Gray, slightly silty, sandy GRAVEL (embankment)	12.5			
B-6, S-5	31.0 - 32.5	GW-GM	■ Gray, slightly silty, sandy GRAVEL (foundation)	15.1			
B-A, S-1	18.0 - 19.5	SM	△ Gray, slightly gravelly, silty SAND (foundation)	55.9			

Upper & Lower Dams Evaluation
Wrangell, Alaska

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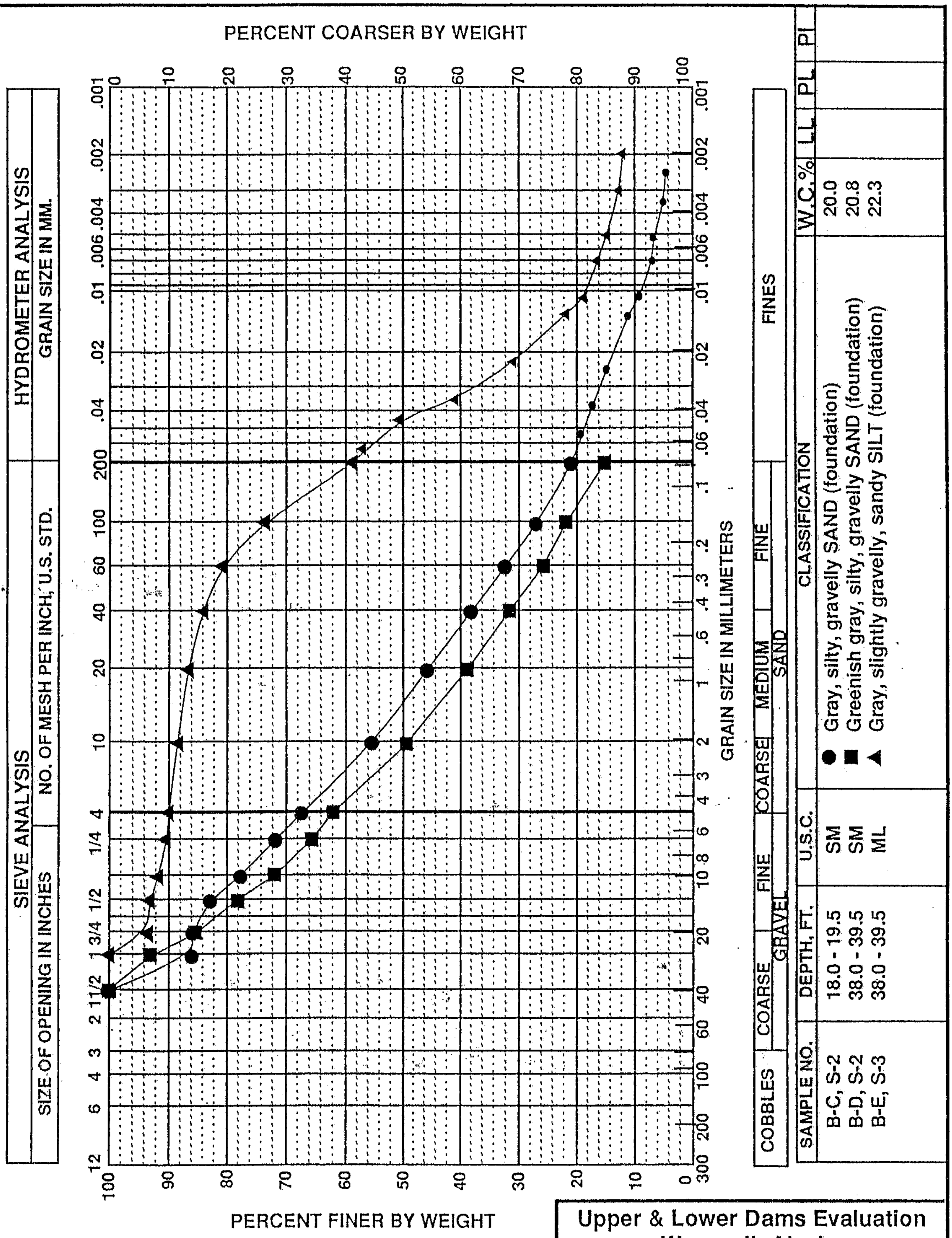
Fig. B-4



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-C, S-2	18.0 - 19.5	SM	● Gray, silty, gravelly SAND (foundation)	20.0			
B-D, S-2	38.0 - 39.5	SM	■ Greenish gray, silty, gravelly SAND (foundation)	20.8			
B-E, S-3	38.0 - 39.5	ML	▲ Gray, slightly gravelly, sandy SILT (foundation)	22.3			

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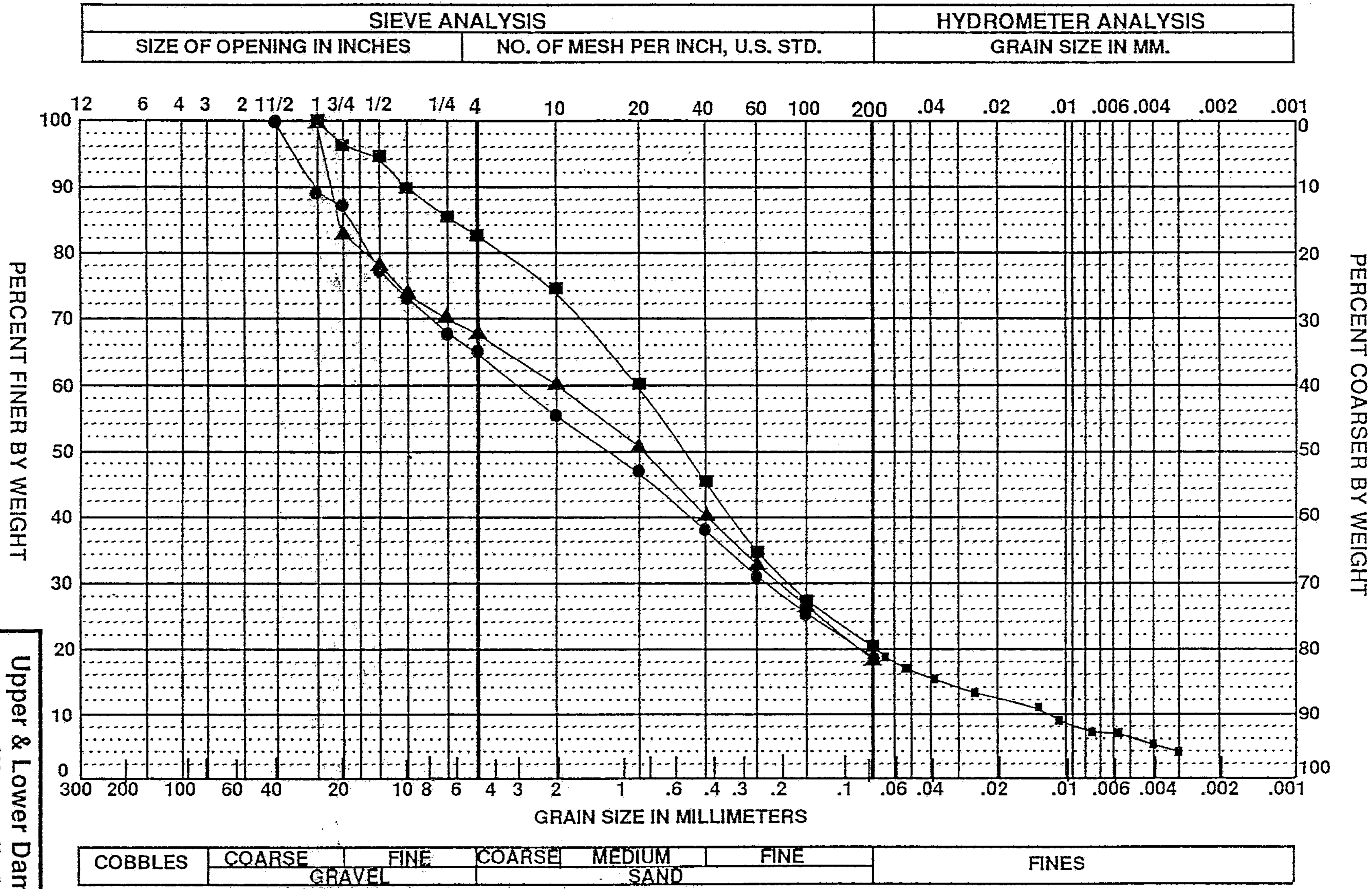
Fig. B-5



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-7, S-1	3.0 - 4.5	SM	● Greenish gray, silty, gravelly SAND (embankment)	13.2			
B-7, S-3	13.0 - 14.5	SM	■ Greenish gray, gravelly, silty SAND (embankment)	19.4			
B-7, S-6	33.0 - 34.2	SM	▲ Greenish gray, silty, gravelly SAND (foundation)	26.0			

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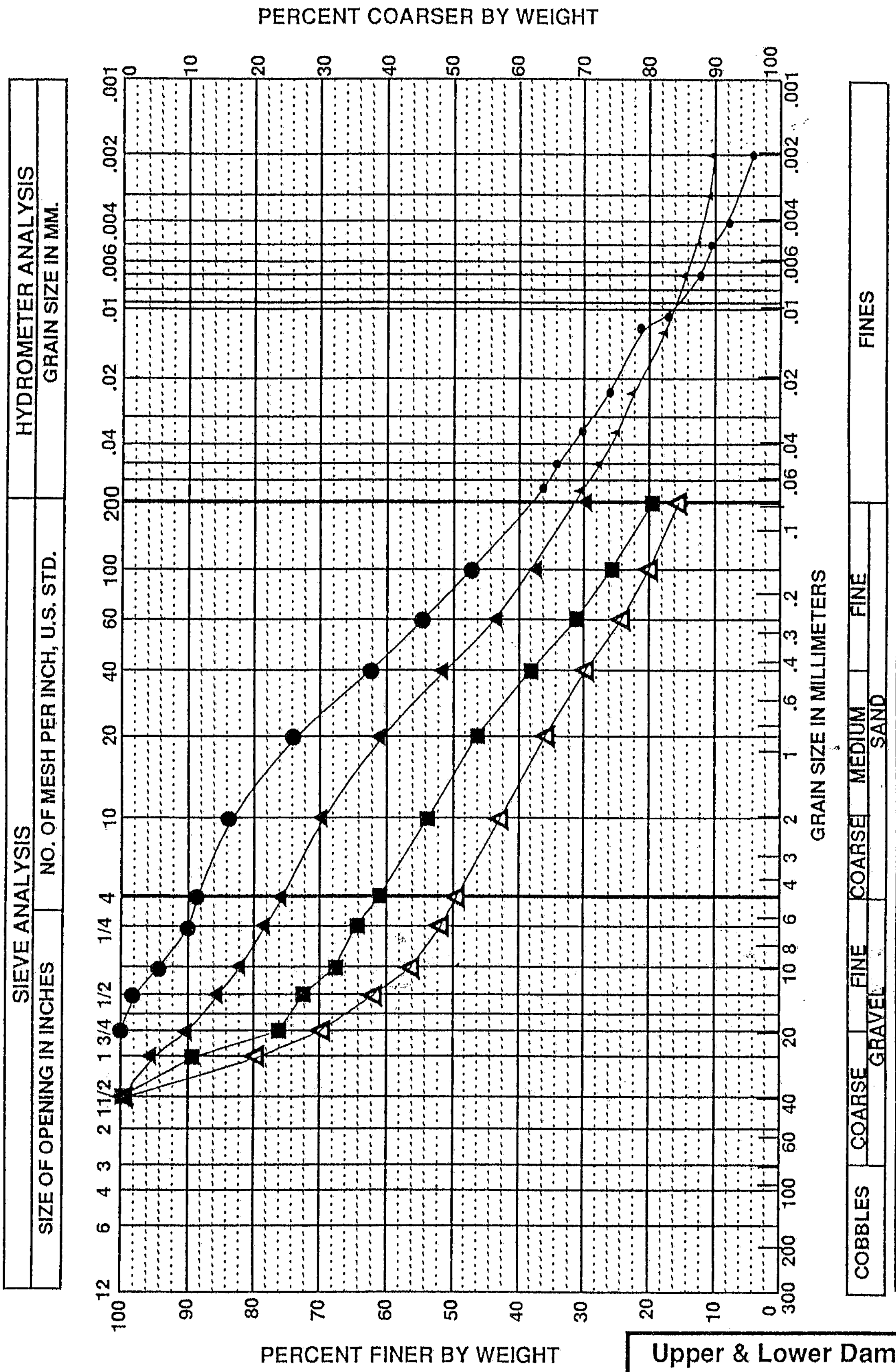
Fig. B-6



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
B-8, S-2	13.0 - 14.5	SM	■ Greenish gray, silty, gravelly SAND (embankment)	13.6			
B-8, S-3	18.0 - 19.5	GM	△ Greenish gray, silty, sandy GRAVEL (embankment)	9.8			
B-9, S-3	8.0 - 9.5	SM	● Gray, gravelly, silty SAND (crib)	20.6			
B-9, S-6	18.0 - 19.5	SM	▲ Gray, gravelly, silty SAND (crib)	17.8			

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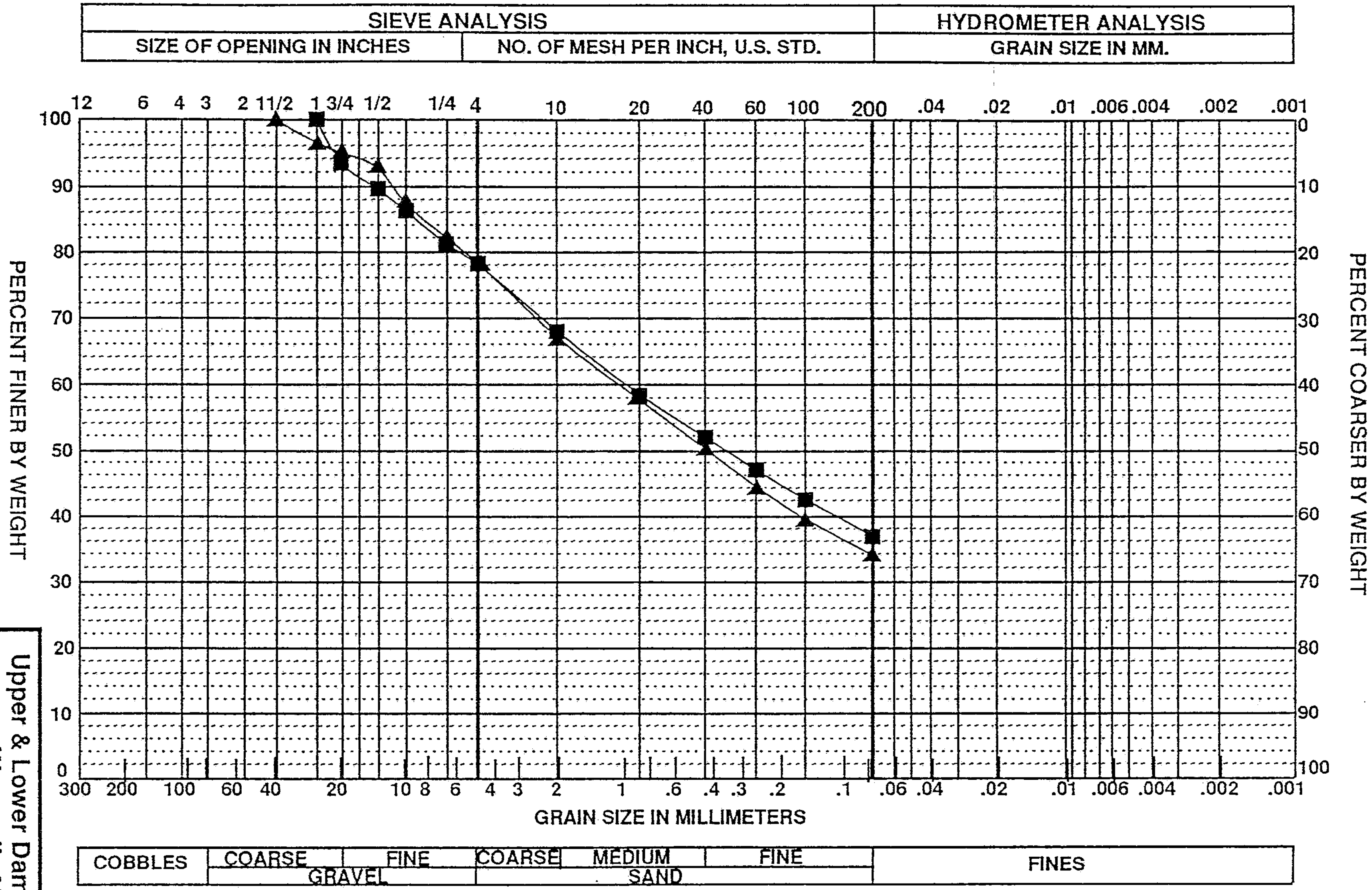
Fig. B-7



SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	W.C.%	LL	PL	PI
Bulk Sample B-1		SM	Typical Embankment Material in Upper Dam ■ Gray, gravelly, silty SAND				
Bulk Sample B-8		SM	▲ Gray, gravelly, silty SAND				

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Fig. B-8

COMPACTION TEST

SHANNON & WILSON , INC

☐ STANDARD AASHO (AASHO T-99, ASTM D698)
☒ MODIFIED AASHO (AASHO T180, ASTM D1557)

PROJECT Wrangell Dam
JOB NUMBER A-494 DATE 3/31/93
HAMMER WT.,LBS 10
DROP, IN. 18
NO. LAYERS 5
NO. BLOWS/LAYER 5
DIA. MOLD IN. 6
HEIGHT MOLD, IN. 5
VOL., MOLD CU. FT. .075
COMPACTIVE EFFORT-FT. LBS/CU. FT. 56300

SAMPLE CLASSIFICATION

BORING	SAMPLE	MAX DRY WT.	OPT. W.C.
B1	BULK	126.9	9.4
B8	BULK	126.5	10.6

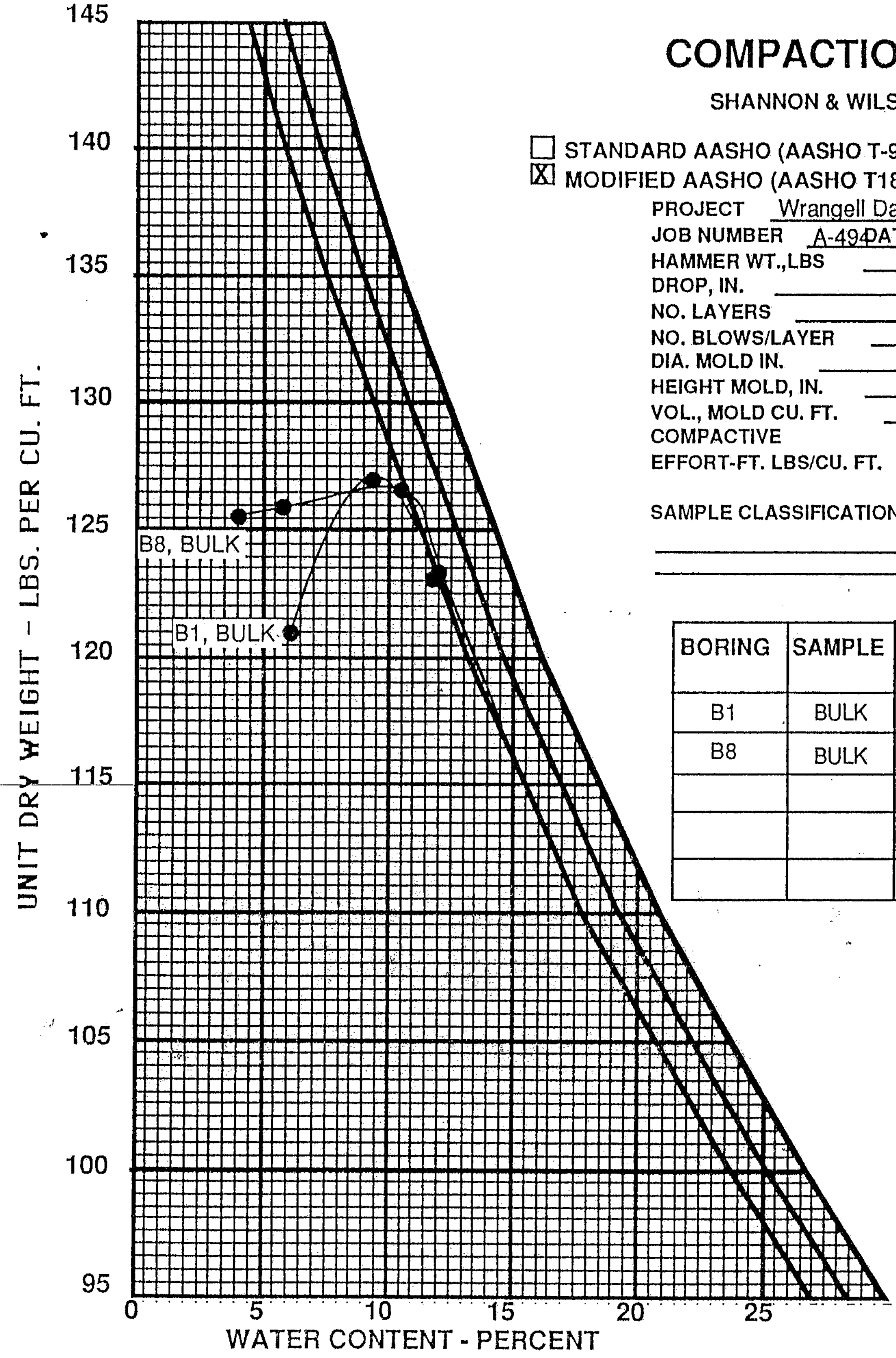
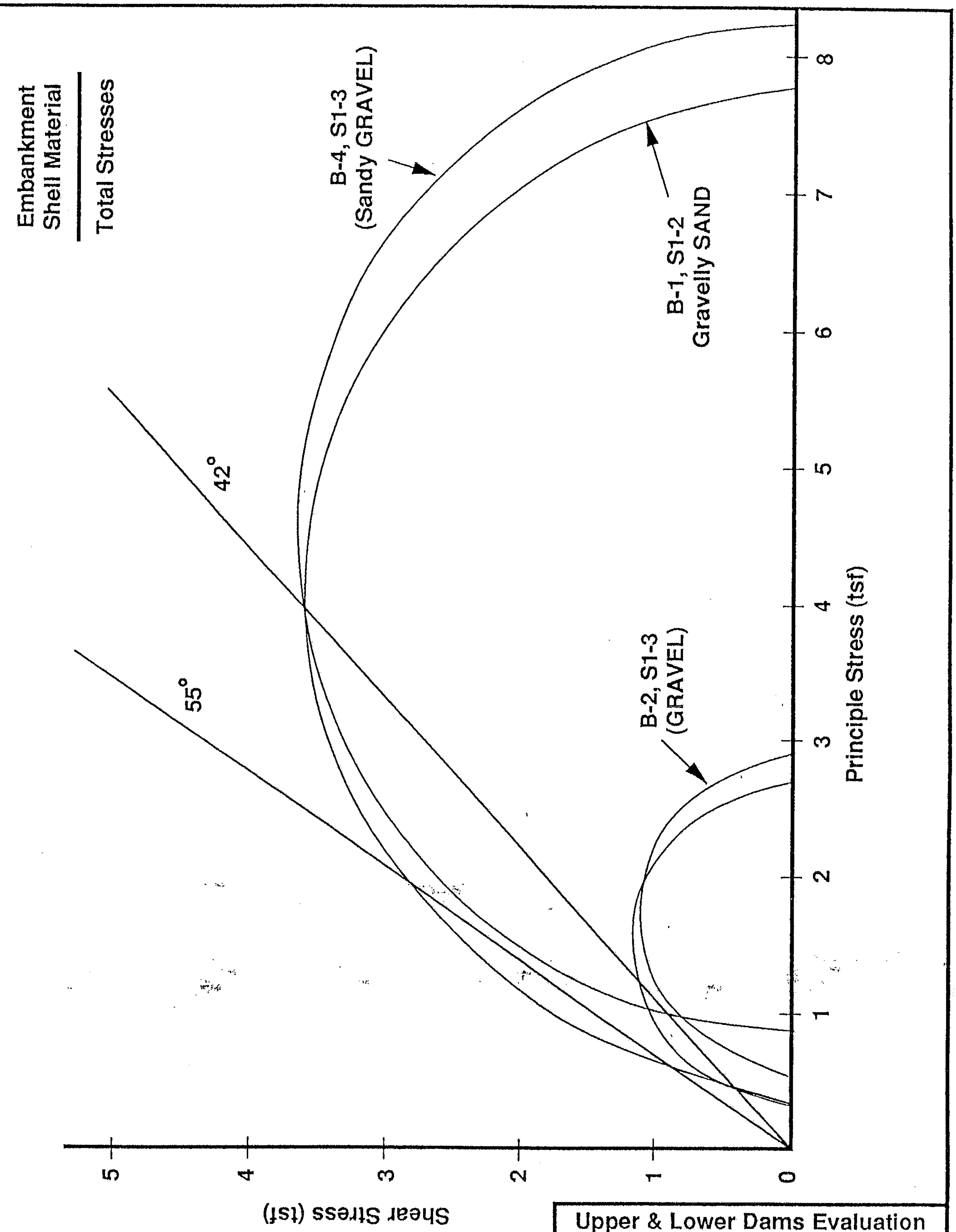


FIG. B-9



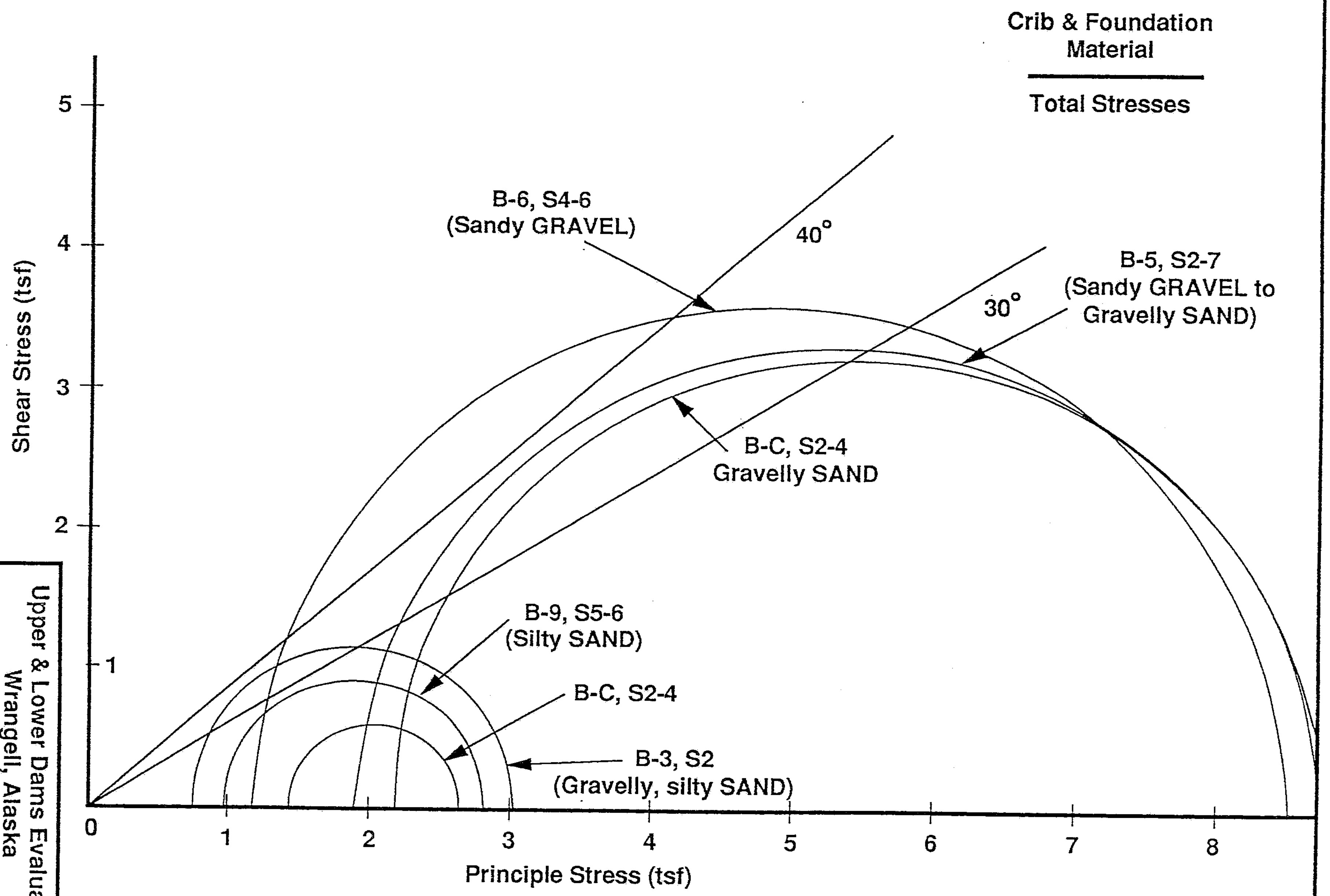
Upper & Lower Dams Evaluation
Wrangell, Alaska

EMBANKMENT SHELL MATERIAL

May 1993 A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

Fig. B-10



Upper & Lower Dams Evaluation
Wrangell, Alaska

CRIB & FOUNDATION MATERIAL

May 1993
A-494

SHANNON & WILSON, INC.
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Fig. B-11

Specimen Data

Height, in.	5.10	
Diameter, in.	2.45	
Hgt./Dia. Ratio	2.08	
Sample Weight	895.29	891.68
Bulk Dens., pcf	141.85	
Pan No.	**H-18**	
Wet + Tare	203.68	
Dry + Tare	187.46	
Tare	11.39	
% Water	9.21%	

Boring:	B1
Sample:	S1-2
Depth:	

Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	6.50	0.47	

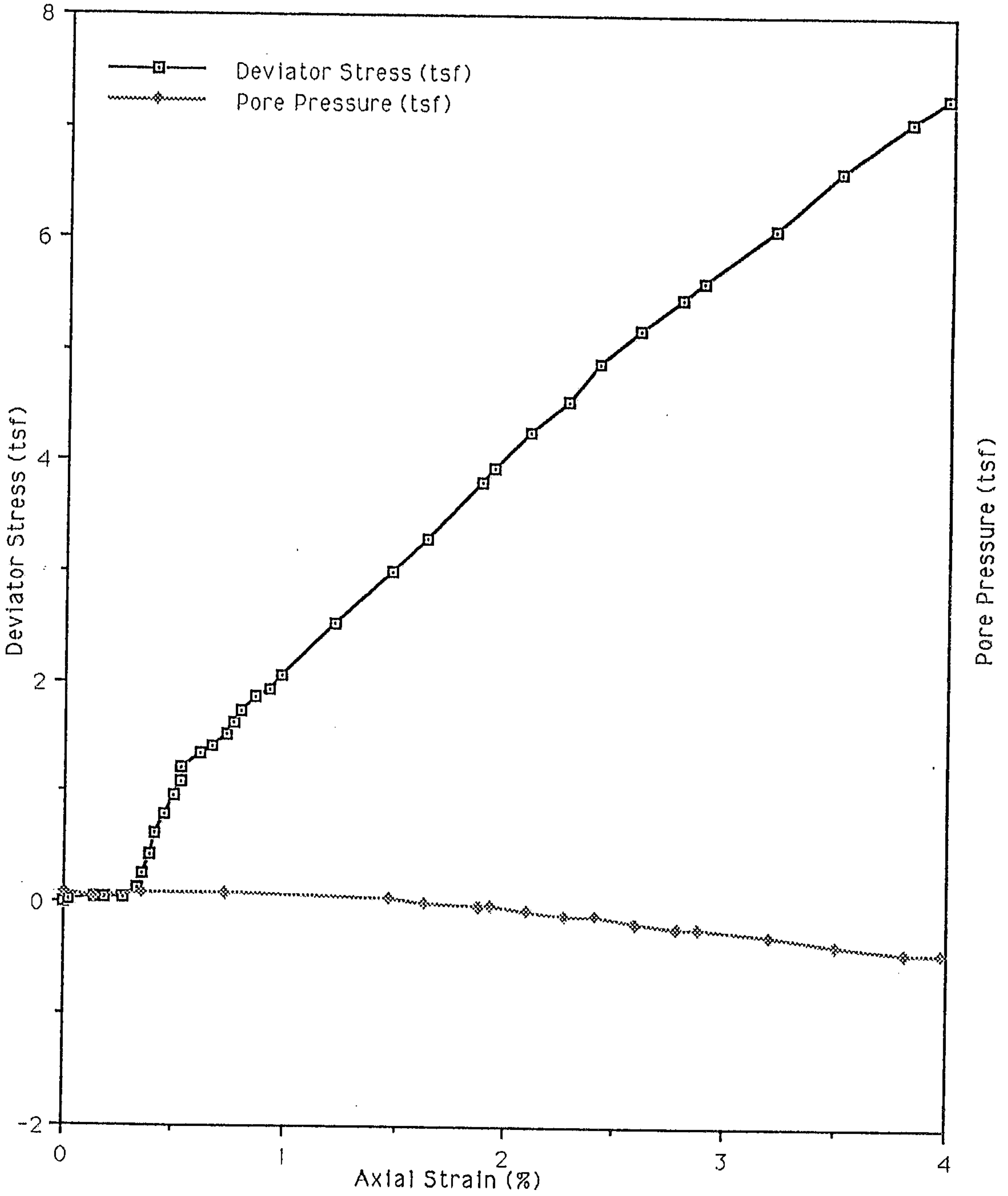
Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	39	0.00%	0.00	4.71	0.00	0.00	0.072
1	2	39	0.02%	1.30	4.72	0.28	0.02	0.072
7	3	38.5	0.14%	1.96	4.72	0.41	0.03	0.036
9	3	38.5	0.18%	1.96	4.72	0.41	0.03	0.036
14	3	38.5	0.27%	1.96	4.73	0.41	0.03	0.036
17	12	38.5	0.33%	7.82	4.73	1.65	0.12	0.036
18	25	38.5	0.35%	16.30	4.73	3.45	0.25	0.036
20	43	39	0.39%	28.04	4.73	5.92	0.43	0.072
21	63	39	0.41%	41.08	4.73	8.68	0.62	0.072
23	80	39	0.45%	52.16	4.74	11.01	0.79	0.072
25	98	39	0.49%	63.90	4.74	13.49	0.97	0.072
27	111	39	0.53%	72.37	4.74	15.27	1.10	0.072

Fig. B-12

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
29	123	39	0.57%	80.20	4.74	16.91	1.22	0.072
31	135	39	0.61%	88.02	4.74	18.56	1.34	0.072
34	144	39	0.67%	93.89	4.75	19.78	1.42	0.072
37	155	39	0.73%	101.06	4.75	21.28	1.53	0.072
39	165	39	0.76%	107.58	4.75	22.65	1.63	0.072
41	176	39	0.80%	114.75	4.75	24.15	1.74	0.072
44	188	39	0.86%	122.58	4.76	25.78	1.86	0.072
47	196	39	0.92%	127.79	4.76	26.86	1.93	0.072
50	208	39	0.98%	135.62	4.76	28.48	2.05	0.072
62	256	39	1.22%	166.91	4.77	34.97	2.52	0.072
75	307	38.5	1.47%	200.16	4.78	41.83	3.01	0.036
83	338	38	1.63%	220.38	4.79	45.98	3.31	0
96	391	37.5	1.88%	254.93	4.80	53.06	3.82	-0.036
99	403	37.5	1.94%	262.76	4.81	54.65	3.94	-0.036
107	437	37	2.10%	284.92	4.82	59.17	4.26	-0.072
116	467	36.5	2.27%	304.48	4.82	63.12	4.54	-0.108
123	502	36.5	2.41%	327.30	4.83	67.75	4.88	-0.108
132	534	35.5	2.59%	348.17	4.84	71.94	5.18	-0.18
142	561	35	2.78%	365.77	4.85	75.43	5.43	-0.216
147	581	35	2.88%	378.81	4.85	78.04	5.62	-0.216
163	632	34	3.20%	412.06	4.87	84.61	6.09	-0.288
179	688	33	3.51%	448.58	4.89	91.81	6.61	-0.36
195	737	32	3.82%	480.52	4.90	98.03	7.06	-0.432
203	760	32	3.98%	495.52	4.91	100.93	7.27	-0.432

Fig. B-12



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B1, S1-2

May 1993

A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-12

Specimen Data

Height, in.	5.95	
Diameter, in.	2.41	
Hgt./Dia. Ratio	2.47	
Sample Weight	890.85	929.27
Bulk Dens., pcf	125.04	
Pan No.	**h18**	**H6**
Wet + Tare	254.09	290.05
Dry + Tare	235.40	258.62
Tare	11.36	11.25
% Water	8.34%	12.71%

Boring:	**B2**
Sample:	**S1-3**
Depth:	

Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	8.00	0.58	

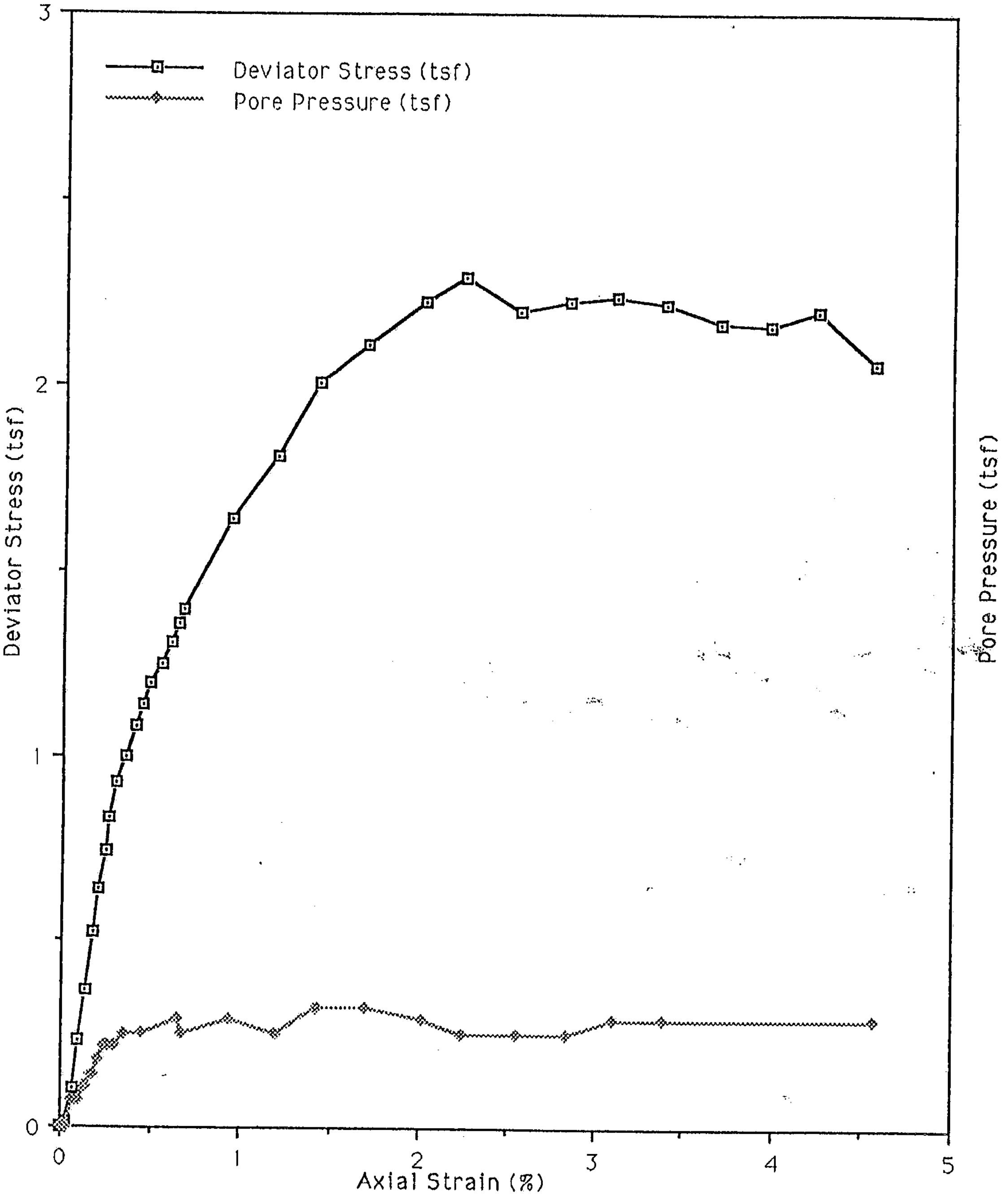
Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	30	0.00%	0.00	4.56	0.00	0.00	0
2	1	30	0.03%	0.65	4.56	0.14	0.01	0
4	10	31	0.07%	6.52	4.56	1.43	0.10	0.072
6	22	31	0.10%	14.34	4.57	3.14	0.23	0.072
8	36	31.5	0.13%	23.47	4.57	5.14	0.37	0.108
10	51	32	0.17%	33.25	4.57	7.28	0.52	0.144
12	62	32.5	0.20%	40.42	4.57	8.84	0.64	0.18
14	72	33	0.24%	46.94	4.57	10.27	0.74	0.216
15	81	33	0.25%	52.81	4.57	11.55	0.83	0.216
18	91	33	0.30%	59.33	4.58	12.97	0.93	0.216
21	98	33.5	0.35%	63.90	4.58	13.96	1.00	0.252
24	105	33.5	0.40%	68.46	4.58	14.95	1.08	0.252

Fig. B-13

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
26	111	33.5	0.44%	72.37	4.58	15.80	1.14	0.252
29	117	33.5	0.49%	76.28	4.58	16.64	1.20	0.252
32.5	122	33.5	0.55%	79.54	4.59	17.34	1.25	0.252
35.5	128	33.5	0.60%	83.46	4.59	18.19	1.31	0.252
38	133	34	0.64%	86.72	4.59	18.89	1.36	0.288
40	137	33.5	0.67%	89.32	4.59	19.45	1.40	0.252
56	161	34	0.94%	104.97	4.61	22.80	1.64	0.288
71	178	33.5	1.19%	116.06	4.62	25.14	1.81	0.252
85	198	34.5	1.43%	129.10	4.63	27.90	2.01	0.324
101	209	34.5	1.70%	136.27	4.64	29.37	2.11	0.324
120	221	34	2.02%	144.09	4.66	30.95	2.23	0.288
134	228	34	2.25%	148.66	4.67	31.85	2.29	0.288
152	219	33.5	2.55%	142.79	4.68	30.50	2.20	0.252
169	223	33.5	2.84%	145.40	4.70	30.97	2.23	0.252
185	225	33.5	3.11%	146.70	4.71	31.16	2.24	0.252
202	223	34	3.39%	145.40	4.72	30.79	2.22	0.288
220	219	34	3.70%	142.79	4.74	30.14	2.17	0.288
237	219	34	3.98%	142.79	4.75	30.05	2.16	0.288
253	223	34	4.25%	145.40	4.76	30.52	2.20	0.288
272	210	34	4.57%	136.92	4.78	28.64	2.06	0.288

Fig. B-13



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B2, S1-3

May 1993 A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-13

Specimen Data

Height, in.	5.04	
Diameter, in.	2.49	
Hgt./Dia. Ratio	2.02	
Sample Weight	657.14	617.86
Bulk Dens., pcf	102.00	
Pan No.	**CC4**	
Wet + Tare	136.81	
Dry + Tare	95.90	
Tare	8.74	
% Water	46.94%	

Boring:	B3
Sample:	S2
Depth:	

Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	10.00	0.72	

Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	40.5	0.00%	0.00	4.87	0.00	0.00	0.036
3	13	40.5	0.06%	8.48	4.87	1.74	0.13	0.036
6	25	40.5	0.12%	16.30	4.88	3.34	0.24	0.036
11	39	41	0.22%	25.43	4.88	5.21	0.38	0.072
15	50	41.5	0.30%	32.60	4.88	6.67	0.48	0.108
20	59	41.5	0.40%	38.47	4.89	7.87	0.57	0.108
26	66	41.5	0.52%	43.03	4.89	8.79	0.63	0.108
36	77	42	0.71%	50.20	4.90	10.24	0.74	0.144
42	81	42	0.83%	52.81	4.91	10.75	0.77	0.144
47	85	42	0.93%	55.42	4.92	11.27	0.81	0.144
59	93	42.5	1.17%	60,64	4.93	12.31	0.89	0.18
71	98	42.5	1.41%	63.90	4.94	12.94	0.93	0.18

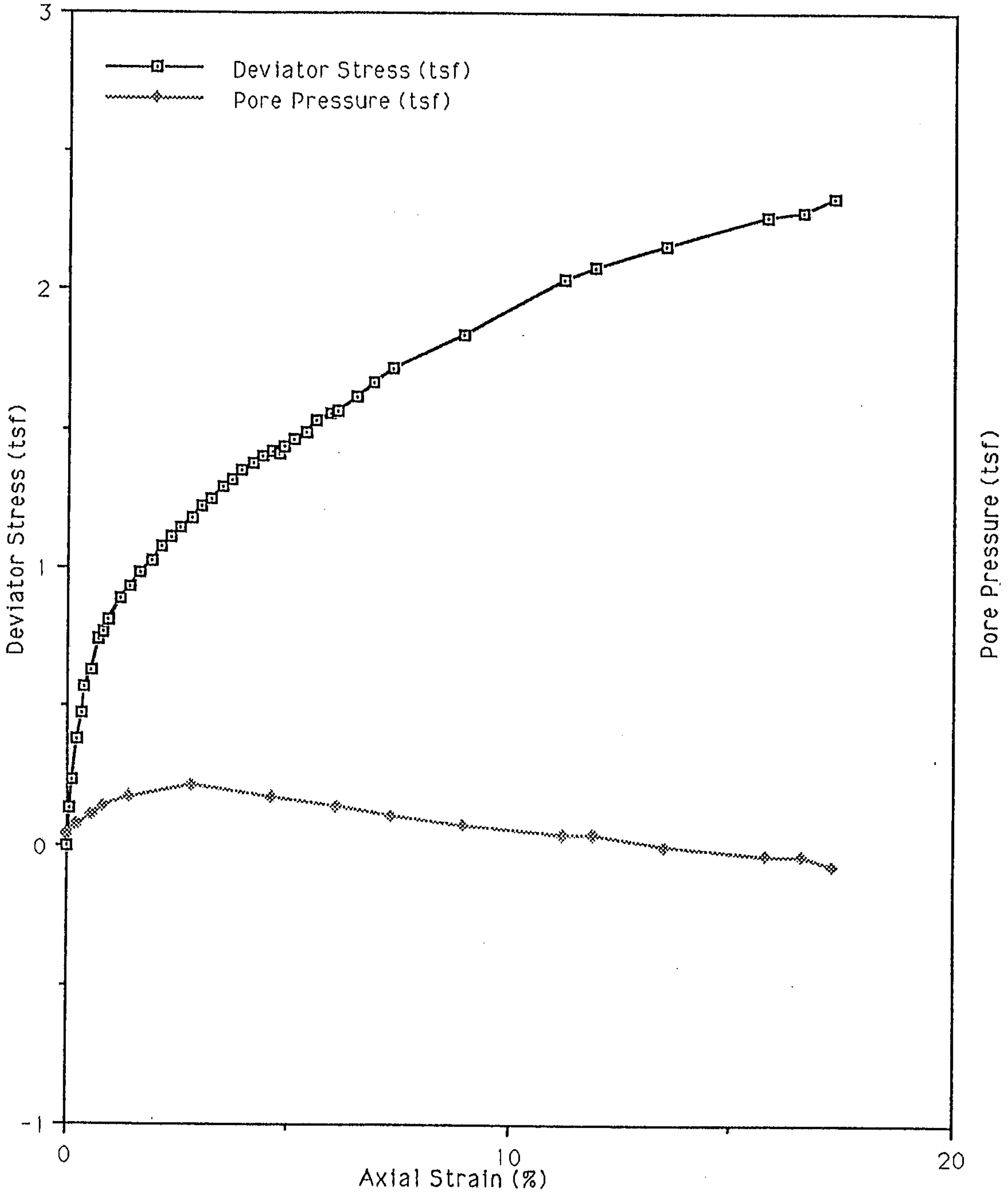
Fig. B-14

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
81	103	42.5	1.61%	67.16	4.95	13.57	0.98	0.18
94	109	42.5	1.87%	71.07	4.96	14.32	1.03	0.18
106	114	43	2.10%	74.33	4.97	14.94	1.08	0.216
117	118	43	2.32%	76.94	4.99	15.43	1.11	0.216
128	122	43	2.54%	79.54	5.00	15.92	1.15	0.216
141	126	43	2.80%	82.15	5.01	16.40	1.18	0.216
151	130	43	3.00%	84.76	5.02	16.88	1.22	0.216
163	134	43	3.23%	87.37	5.03	17.36	1.25	0.216
175	139	43	3.47%	90.63	5.04	17.96	1.29	0.216
187	142	42.5	3.71%	92.58	5.06	18.31	1.32	0.18
199	146	42.5	3.95%	95.19	5.07	18.78	1.35	0.18
211	149	42.5	4.19%	97.15	5.08	19.11	1.38	0.18
223	152	42.5	4.42%	99.10	5.09	19.45	1.40	0.18
234	155	42.5	4.64%	101.06	5.11	19.79	1.42	0.18
241	154	42.5	4.78%	100.41	5.11	19.63	1.41	0.18
246	157	42.5	4.88%	102.36	5.12	20.00	1.44	0.18
259	160	42.5	5.14%	104.32	5.13	20.32	1.46	0.18
271	163	42.5	5.38%	106.28	5.15	20.65	1.49	0.18
285	168	42	5.65%	109.54	5.16	21.22	1.53	0.144
298	172	42	5.91%	112.14	5.18	21.67	1.56	0.144
310	174	42	6.15%	113.45	5.19	21.86	1.57	0.144
327	180	42	6.49%	117.36	5.21	22.54	1.62	0.144
346	186	42	6.87%	121.27	5.23	23.19	1.67	0.144
368	192	41.5	7.30%	125.18	5.25	23.83	1.72	0.108
446	209	41	8.85%	136.27	5.34	25.51	1.84	0.072
565	238	40.5	11.21%	155.18	5.48	28.29	2.04	0.036
600	245	40.5	11.90%	159.74	5.53	28.90	2.08	0.036
679	259	40	13.47%	168.87	5.63	30.01	2.16	0
796	278	39.5	15.79%	181.26	5.78	31.34	2.26	-0.036
835	284	39.5	16.57%	185.17	5.84	31.73	2.28	-0.036
874	292	39	17.34%	190.38	5.89	32.32	2.33	-0.072

A-494 Wrangell Dam

Sample B3 S2 Page 2 of 2

Fig. B-14



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B3, S2

May 1993

A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-14

Specimen Data

Height, in.	5.59	
Diameter, in.	2.40	
Hgt./Dia. Ratio	2.33	
Sample Weight	907.46	
Bulk Dens., pcf	136.70	
Pan No.	**A100**	**CC1**
Wet + Tare	195.21	268.65
Dry + Tare	188.28	242.93
Tare	11.43	8.63
% Water	3.92%	10.98%

Boring:	**B4**
Sample:	**S1-3**
Depth:	14

Equipment Constants

Axial Def. Dial: 0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure psi., tsf	12.00	0.86	

Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	40	0.00%	0.00	4.52	0.00	0.00	0
7	5	40	0.13%	3.26	4.53	0.72	0.05	0
9	44	41	0.16%	28.69	4.53	6.33	0.46	0.072
11	71	40.5	0.20%	46.29	4.53	10.21	0.74	0.036
13	94	40.5	0.23%	61.29	4.53	13.52	0.97	0.036
15	116	41	0.27%	75.63	4.54	16.67	1.20	0.072
17	134	41	0.30%	87.37	4.54	19.25	1.39	0.072
20	162	41.5	0.36%	105.62	4.54	23.26	1.68	0.108
24	186	41.5	0.43%	121.27	4.54	26.69	1.92	0.108
27	206	41.5	0.48%	134.31	4.55	29.55	2.13	0.108
32	225	41.5	0.57%	146.70	4.55	32.24	2.32	0.108
35	243	41.5	0.63%	158.44	4.55	34.80	2.51	0.108

Fig. B-15

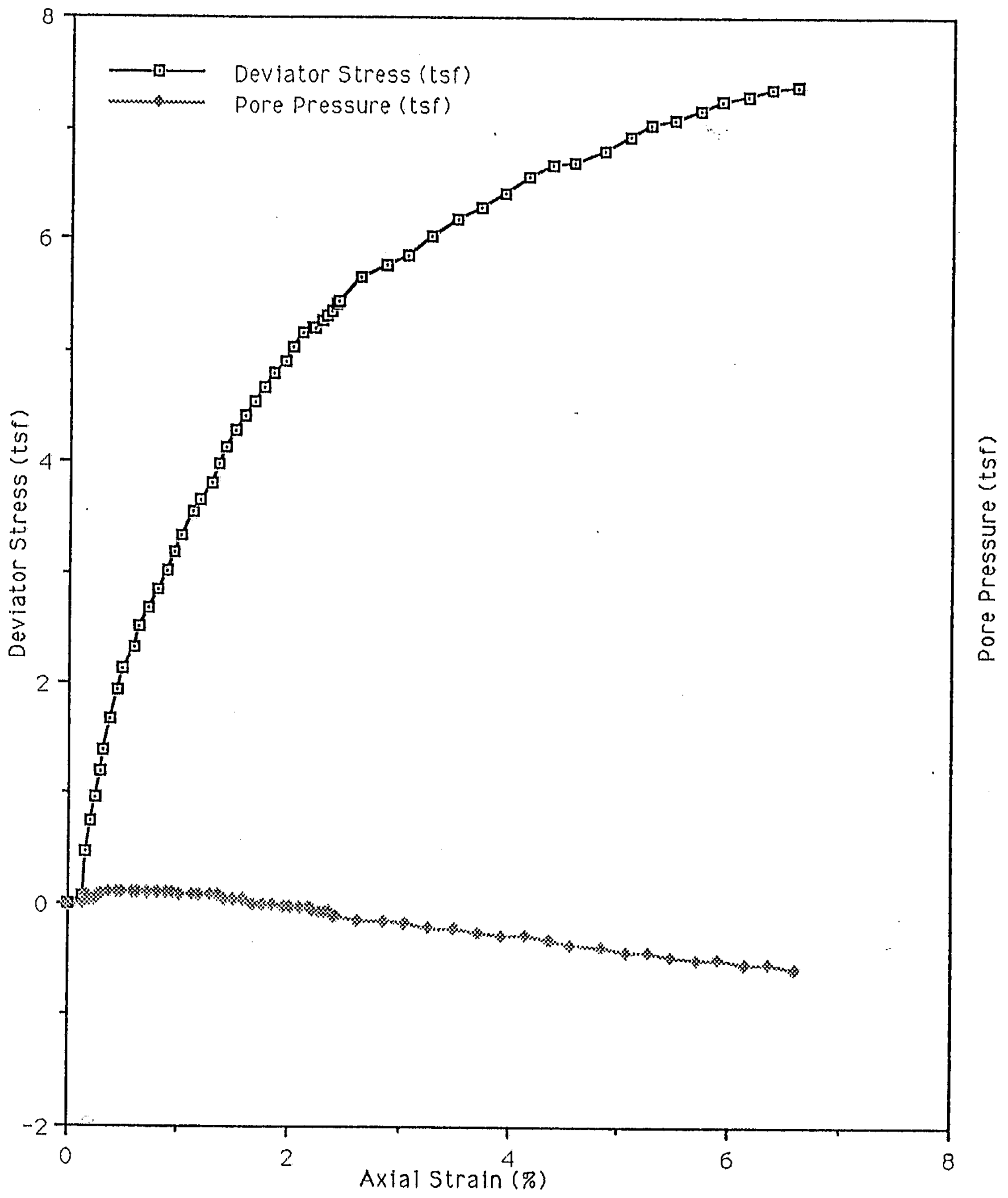
Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
40	259	41.5	0.72%	168.87	4.56	37.06	2.67	0.108
44	276	41.5	0.79%	179.95	4.56	39.47	2.84	0.108
49	294	41.5	0.88%	191.69	4.56	42.00	3.02	0.108
53	310	41.5	0.95%	202.12	4.57	44.25	3.19	0.108
57	325	41	1.02%	211.90	4.57	46.36	3.34	0.072
62	346	41	1.11%	225.59	4.57	49.31	3.55	0.072
66	357	41	1.18%	232.76	4.58	50.84	3.66	0.072
72	372	41	1.29%	242.54	4.58	52.92	3.81	0.072
76	389	41	1.36%	253.63	4.59	55.30	3.98	0.072
80	404	40.5	1.43%	263.41	4.59	57.39	4.13	0.036
84	419	40.5	1.50%	273.19	4.59	59.48	4.28	0.036
89	432	40.5	1.59%	281.66	4.60	61.27	4.41	0.036
94	445	40	1.68%	290.14	4.60	63.06	4.54	0
99	459	40	1.77%	299.27	4.61	64.98	4.68	0
104	471	40	1.86%	307.09	4.61	66.62	4.80	0
109	484	39.5	1.95%	315.57	4.61	68.40	4.92	-0.036
113	496	39.5	2.02%	323.39	4.62	70.04	5.04	-0.036
118	508	39.5	2.11%	331.22	4.62	71.67	5.16	-0.036
123	512	39.5	2.20%	333.82	4.63	72.17	5.20	-0.036
124	513	39	2.22%	334.48	4.63	72.30	5.21	-0.072
127	521	39	2.27%	339.69	4.63	73.38	5.28	-0.072
130	525	39	2.33%	342.30	4.63	73.91	5.32	-0.072
132	530	39	2.36%	345.56	4.63	74.58	5.37	-0.072
134	535	38.5	2.40%	348.82	4.64	75.26	5.42	-0.108
136	538	38.5	2.43%	350.78	4.64	75.65	5.45	-0.108
147	561	38	2.63%	365.77	4.65	78.73	5.67	-0.144
160	571	38	2.86%	372.29	4.66	79.94	5.76	-0.144
171	582	37.5	3.06%	379.46	4.67	81.31	5.85	-0.18
183	600	37	3.27%	391.20	4.68	83.64	6.02	-0.216
196	617	37	3.51%	402.28	4.69	85.81	6.18	-0.216
208	629	36.5	3.72%	410.11	4.70	87.28	6.28	-0.252

Fig. B-15

A-494 Wrangell Dam

Sample B4 S1-3 Page 2 of 3

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
220	643	36	3.94%	419.24	4.71	89.02	6.41	-0.288
232	660	36	4.15%	430.32	4.72	91.17	6.56	-0.288
244	673	35.5	4.36%	438.80	4.73	92.76	6.68	-0.324
255	677	35	4.56%	441.40	4.74	93.12	6.70	-0.36
270	688	34.5	4.83%	448.58	4.75	94.37	6.79	-0.396
284	704	34	5.08%	459.01	4.77	96.31	6.93	-0.432
294	716	34	5.26%	466.83	4.78	97.77	7.04	-0.432
307	721	33.5	5.49%	470.09	4.79	98.21	7.07	-0.468
319	732	33	5.71%	477.26	4.80	99.48	7.16	-0.504
331	742	33	5.92%	483.78	4.81	100.61	7.24	-0.504
344	749	32.5	6.15%	488.35	4.82	101.31	7.29	-0.54
356	756	32.5	6.37%	492.91	4.83	102.02	7.35	-0.54
369	761	32	6.60%	496.17	4.84	102.44	7.38	-0.576



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B4, S1-3

May 1993 A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-15

Specimen Data

Height, in.	5.74	
Diameter, in.	2.41	
Hgt./Dia. Ratio	2.38	
Sample Weight	933.19	913.35
Bulk Dens., pcf	135.77	
Pan No.	**T65**	**He21**
Wet + Tare	197.12	235.24
Dry + Tare	174.27	211.05
Tare	11.36	11.35
% Water	14.03%	12.11%

Boring:	B5
Sample:	S2-7
Depth:	20.5

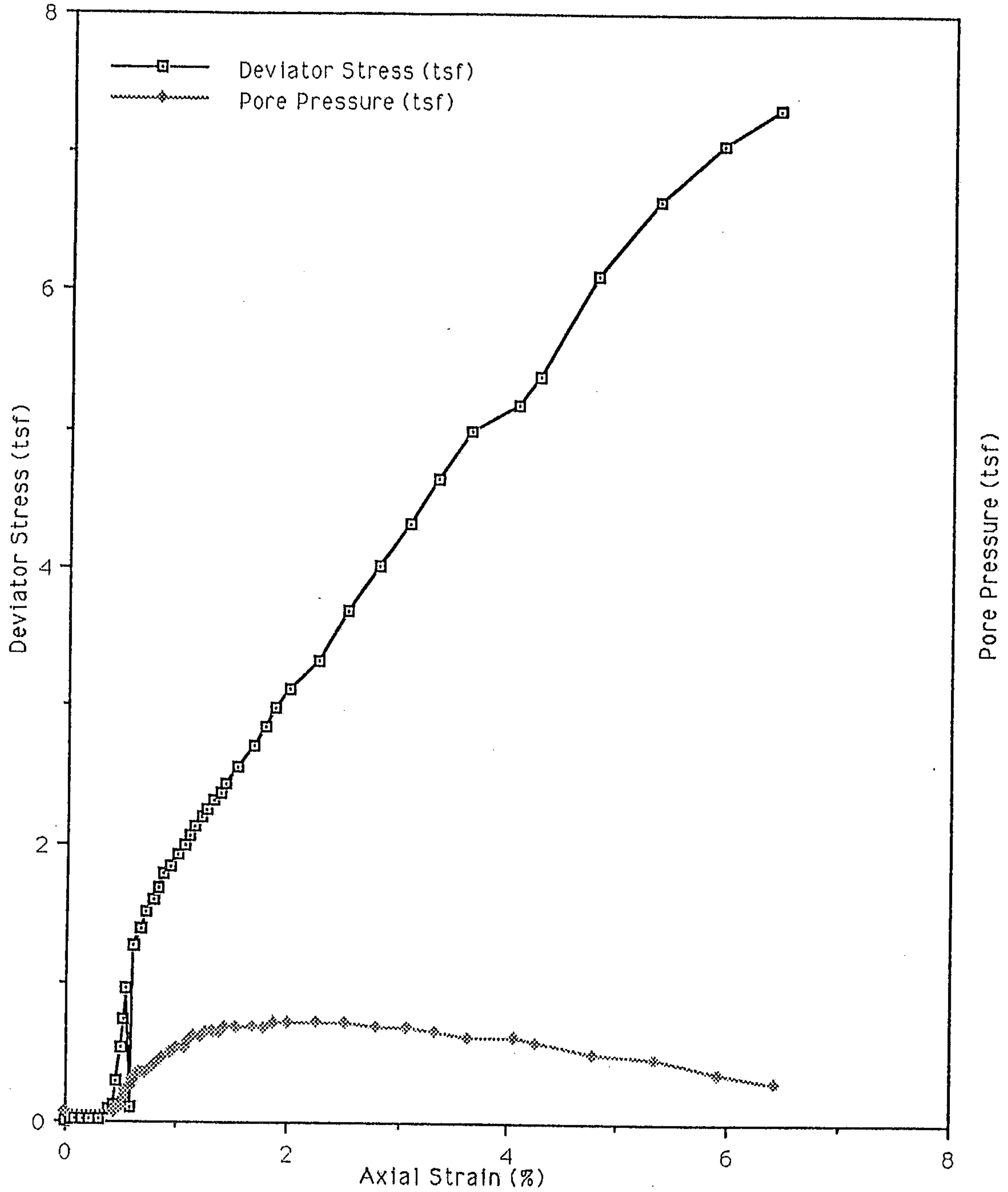
Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	17.00	1.22	

Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	46	0.00%	0.00	4.56	0.00	0.00	0.07
3	2	46	0.05%	1.30	4.56	0.29	0.02	0.07
5	2	46	0.09%	1.30	4.57	0.29	0.02	0.07
9	2	46	0.16%	1.30	4.57	0.29	0.02	0.07
12.5	2	46	0.22%	1.30	4.57	0.29	0.02	0.07
17	2	46	0.30%	1.30	4.58	0.29	0.02	0.07
22	8	46	0.38%	5.22	4.58	1.14	0.08	0.07
24	12	46	0.42%	7.82	4.58	1.71	0.12	0.07
26	28	46.5	0.45%	18.26	4.58	3.98	0.29	0.11
28	52	46.5	0.49%	33.90	4.58	7.40	0.53	0.11
29.5	72	47.5	0.51%	46.94	4.59	10.24	0.74	0.18
31	94	48	0.54%	61.29	4.59	13.36	0.96	0.22
33	111	48.5	0.57%	72.37	4.59	15.77	1.14	0.25
35	124	49.3	0.61%	80.85	4.59	17.62	1.27	0.31
38	136	50	0.66%	88.67	4.59	19.31	1.39	0.36

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
41	148	50	0.71%	96.50	4.59	21.00	1.51	0.36
44	156	50.5	0.77%	101.71	4.60	22.13	1.59	0.40
47	165	51	0.82%	107.58	4.60	23.39	1.68	0.43
50	174	51.5	0.87%	113.45	4.60	24.65	1.78	0.47
53	180	52	0.92%	117.36	4.60	25.49	1.84	0.50
56	188	52.5	0.98%	122.58	4.61	26.61	1.92	0.54
60	196	52.5	1.05%	127.79	4.61	27.72	2.00	0.54
63	202	53	1.10%	131.70	4.61	28.55	2.06	0.58
66	209	53.5	1.15%	136.27	4.61	29.53	2.13	0.61
69	215	53.5	1.20%	140.18	4.62	30.36	2.19	0.61
72	221	54	1.25%	144.09	4.62	31.19	2.25	0.65
76	228	54	1.32%	148.66	4.62	32.16	2.32	0.65
79	234	54	1.38%	152.57	4.63	32.99	2.37	0.65
82	241	54.5	1.43%	157.13	4.63	33.95	2.44	0.68
88	253	54.5	1.53%	164.96	4.63	35.61	2.56	0.68
96	268	54.5	1.67%	174.74	4.64	37.66	2.71	0.68
102	282	54.5	1.78%	183.86	4.64	39.59	2.85	0.68
108	295	55	1.88%	192.34	4.65	41.37	2.98	0.72
115	309	55	2.00%	201.47	4.65	43.28	3.12	0.72
129	331	55	2.25%	215.81	4.67	46.25	3.33	0.72
144	368	55	2.51%	239.94	4.68	51.28	3.69	0.72
161	402	54.5	2.80%	262.10	4.69	55.85	4.02	0.68
176	433	54.5	3.07%	282.32	4.71	59.99	4.32	0.68
192	467	54	3.34%	304.48	4.72	64.52	4.65	0.65
209	503	53.5	3.64%	327.96	4.73	69.28	4.99	0.61
233	525	53.5	4.06%	342.30	4.75	71.99	5.18	0.61
244	547	53	4.25%	356.64	4.76	74.86	5.39	0.58
274	623	52	4.77%	406.20	4.79	84.79	6.11	0.50
306	683	51.5	5.33%	445.32	4.82	92.42	6.65	0.47
340	729	50	5.92%	475.31	4.85	98.02	7.06	0.36
369	760	49	6.43%	495.52	4.88	101.64	7.32	0.29



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B5, S2-7

May 1993

A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-16

Specimen Data

Height, in.	5.74	
Diameter, in.	2.41	
Hgt./Dia. Ratio	2.39	
Sample Weight	883.97	884.50
Bulk Dens., pcf	129.14	
Pan No.	**A47**	**A49**
Wet + Tare	146.02	300.65
Dry + Tare	129.54	264.03
Tare	11.52	11.44
% Water	13.96%	14.50%

Boring:	B6
Sample:	S4-6
Depth:	31

Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	26.00	1.87	

Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	30	0.00%	0.00	4.54	0.00	0.00	0.00
2	5	30	0.03%	3.26	4.54	0.72	0.05	0.00
2	21	30.5	0.03%	13.69	4.54	3.01	0.22	0.04
3	35	30.5	0.05%	22.82	4.55	5.02	0.36	0.04
4	40	30.5	0.07%	26.08	4.55	5.74	0.41	0.04
4	65	31	0.07%	42.38	4.55	9.32	0.67	0.07
5	79	31.5	0.09%	51.51	4.55	11.33	0.82	0.11
5.5	94	32	0.10%	61.29	4.55	13.48	0.97	0.14
6	107	32	0.10%	69.76	4.55	15.34	1.10	0.14
8	121	32.5	0.14%	78.89	4.55	17.34	1.25	0.18
8	133	33	0.14%	86.72	4.55	19.06	1.37	0.22
9	143	33	0.16%	93.24	4.55	20.49	1.48	0.22
10	156	33.5	0.17%	101.71	4.55	22.35	1.61	0.25

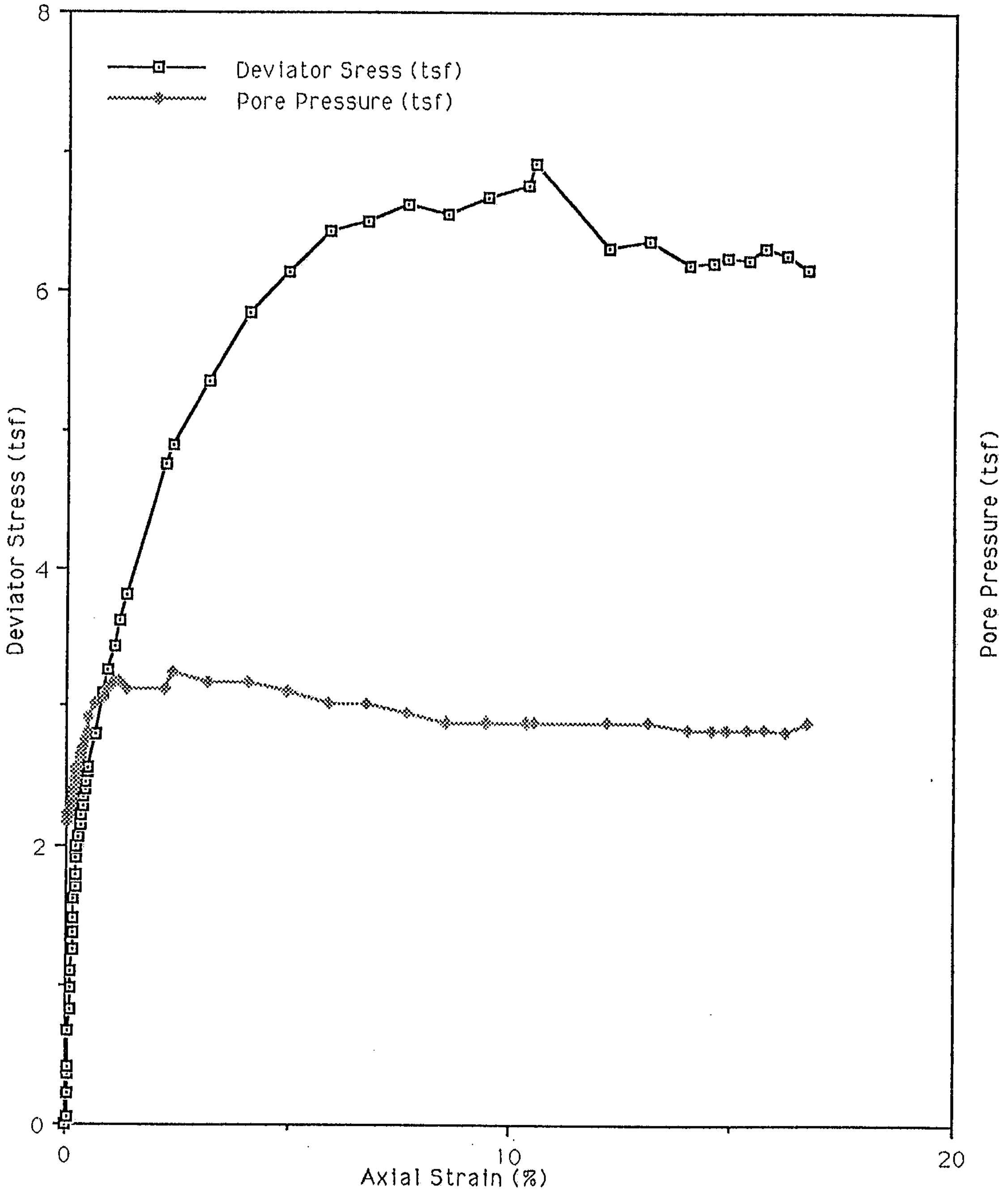
Fig. B-17

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
11	165	34	0.19%	107.58	4.55	23.64	1.70	0.29
12	174	34.5	0.21%	113.45	4.55	24.92	1.79	0.32
13	185	35	0.23%	120.62	4.55	26.49	1.91	0.36
14	193	35.5	0.24%	125.84	4.55	27.63	1.99	0.40
15.5	200	35.5	0.27%	130.40	4.56	28.63	2.06	0.40
17	209	36.5	0.30%	136.27	4.56	29.91	2.15	0.47
18.5	215	36.5	0.32%	140.18	4.56	30.76	2.21	0.47
20	221	37	0.35%	144.09	4.56	31.61	2.28	0.50
22	228	37.5	0.38%	148.66	4.56	32.60	2.35	0.54
24	233	37.5	0.42%	151.92	4.56	33.30	2.40	0.54
25	239	38	0.44%	155.83	4.56	34.15	2.46	0.58
27	245	38.5	0.47%	159.74	4.56	35.00	2.52	0.61
28.5	249	39	0.50%	162.35	4.57	35.56	2.56	0.65
36	272	40.5	0.63%	177.34	4.57	38.79	2.79	0.76
45	301	42	0.78%	196.25	4.58	42.86	3.09	0.86
53	318	42.5	0.92%	207.34	4.59	45.22	3.26	0.90
61.5	335.5	43.5	1.07%	218.75	4.59	47.64	3.43	0.97
69	355	44	1.20%	231.46	4.60	50.34	3.62	1.01
78	374	44	1.36%	243.85	4.61	52.95	3.81	1.01
128	470	43.5	2.23%	306.44	4.65	65.95	4.75	0.97
136	485	43.5	2.37%	316.22	4.65	67.96	4.89	0.97
181	536	45	3.15%	349.47	4.69	74.50	5.36	1.08
235	590	44	4.09%	384.68	4.74	81.21	5.85	1.01
285	625	44	4.97%	407.50	4.78	85.25	6.14	1.01
335	662	43	5.84%	431.62	4.82	89.47	6.44	0.94
387	675	42	6.74%	440.10	4.87	90.35	6.51	0.86
439	694	42	7.65%	452.49	4.92	91.99	6.62	0.86
492	694	41	8.57%	452.49	4.97	91.07	6.56	0.79
543	714	40	9.46%	465.53	5.02	92.78	6.68	0.72
594	731	40	10.35%	476.61	5.07	94.06	6.77	0.72

Fig. B-17

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
605	747	40	10.54%	487.04	5.08	95.91	6.91	0.72
700	696	40	12.20%	453.79	5.17	87.71	6.32	0.72
752	709	40	13.10%	462.27	5.23	88.43	6.37	0.72
804	697	40	14.01%	454.44	5.28	86.02	6.19	0.72
835	703	39.5	14.55%	458.36	5.32	86.22	6.21	0.68
855	711	39.5	14.90%	463.57	5.34	86.85	6.25	0.68
882	712.5	39.5	15.37%	464.55	5.37	86.55	6.23	0.68
904	726	39.5	15.75%	473.35	5.39	87.79	6.32	0.68
933	725	39	16.25%	472.70	5.42	87.14	6.27	0.65
959	717	40	16.71%	467.48	5.45	85.71	6.17	0.72

Fig. B-17



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B6, S4-6

May 1993 A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-17

Specimen Data

Height, in.	5.18	
Diameter, in.	2.48	
Hgt./Dia. Ratio	2.09	
Sample Weight	890.60	886.00
Bulk Dens., pcf	136.14	
Pan No.	**Q17**	
Wet + Tare	166.39	
Dry + Tare	147.12	
Tare	11.45	
% Water	14.20%	

Boring:	B9
Sample:	S5-6
Depth:	16

Equipment Constants

Axial Def. Dial:	0.001	In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure psi., tsf	13.50	0.97	

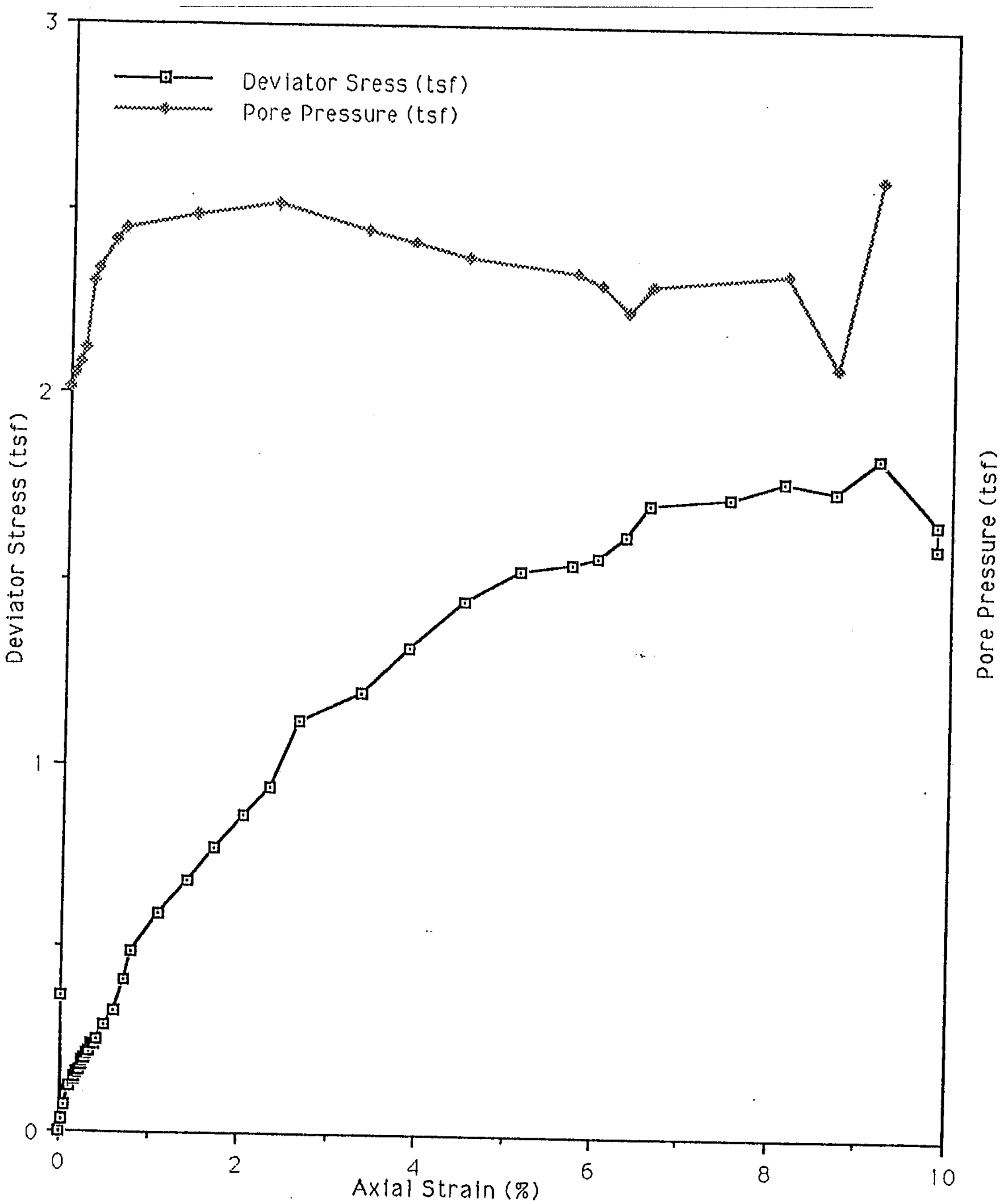
Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	28	0.00%	0.00	4.81	0.00	0.00	0
1	3	28	0.02%	1.96	4.81	0.41	0.03	0
3	7	28.5	0.06%	4.56	4.81	0.95	0.07	0.036
5	12	29	0.10%	7.82	4.82	1.62	0.12	0.072
8	14	29	0.15%	9.13	4.82	1.89	0.14	0.072
9	15	29.5	0.17%	9.78	4.82	2.03	0.15	0.108
9.5	16	29.5	0.18%	10.43	4.82	2.16	0.16	0.108
11	17	29.5	0.21%	11.08	4.82	2.30	0.17	0.108
12	18	29.5	0.23%	11.74	4.82	2.43	0.18	0.108
13	20	32	0.25%	13.04	4.82	2.70	0.19	0.288
14	21	32	0.27%	13.69	4.82	2.84	0.20	0.288
15	21.5	32	0.29%	14.02	4.83	2.91	0.21	0.288

Fig. B-18

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
15.5	22	32.5	0.30%	14.34	4.83	2.97	0.21	0.324
16.5	23	32.5	0.32%	15.00	4.83	3.11	0.22	0.324
17.5	24.5	32.5	0.34%	15.97	4.83	3.31	0.24	0.324
19	25	32.5	0.37%	16.30	4.83	3.38	0.24	0.324
20	26	32.5	0.39%	16.95	4.83	3.51	0.25	0.324
25	30	33.5	0.48%	19.56	4.83	4.05	0.29	0.396
30	34	34	0.58%	22.17	4.84	4.58	0.33	0.432
36	38	34	0.69%	24.78	4.84	5.11	0.37	0.432
41	42	34	0.79%	27.38	4.85	5.65	0.41	0.432
56	51	34	1.08%	33.25	4.86	6.84	0.49	0.432
72	61	34	1.39%	39.77	4.88	8.15	0.59	0.432
88	71	34.5	1.70%	46.29	4.89	9.46	0.68	0.468
104	81	34.5	2.01%	52.81	4.91	10.76	0.77	0.468
120	90	34.5	2.32%	58.68	4.93	11.91	0.86	0.468
137	99	35	2.64%	64.55	4.94	13.06	0.94	0.504
173	119	35	3.34%	77.59	4.98	15.59	1.12	0.504
200	128	34	3.86%	83.46	5.00	16.68	1.20	0.432
233	142	33.5	4.50%	92.58	5.04	18.38	1.32	0.396
264	157	33	5.10%	102.36	5.07	20.19	1.45	0.36
296	166	33	5.71%	108.23	5.10	21.21	1.53	0.36
311	169	32.5	6.00%	110.19	5.12	21.53	1.55	0.324
327	172	32	6.31%	112.14	5.14	21.84	1.57	0.288
341	179	31	6.58%	116.71	5.15	22.66	1.63	0.216
389	189	32	7.51%	123.23	5.20	23.69	1.71	0.288
420	193	32	8.11%	125.84	5.24	24.03	1.73	0.288
451	200	32.5	8.71%	130.40	5.27	24.74	1.78	0.324
476	197	29	9.19%	128.44	5.30	24.24	1.75	0.072
509	209	36	9.83%	136.27	5.34	25.54	1.84	0.576
510	190	36	9.85%	123.88	5.34	23.21	1.67	0.576
511	182	36	9.86%	118.66	5.34	22.23	1.60	0.576

Fig. B-18



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample B9, S5-6

May 1993
A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-18

Specimen Data

Height, in.	5.47	
Diameter, in.	2.47	
Hgt./Dia. Ratio	2.21	
Sample Weight	944.43	912.03
Bulk Dens., pcf	137.27	
Pan No.	**P84**	**T14**
Wet + Tare	112.69	166.26
Dry + Tare	107.69	145.92
Tare	11.35	11.44
% Water	5.19%	15.12%

Boring:	**BC**
Sample:	**S2-4**
Depth:	24

Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	30.00	2.16	

Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	28	0.00%	0.00	4.79	0.00	0.00	-0.144
10	1	28	0.18%	0.65	4.80	0.14	0.01	-0.144
200	1	28	3.66%	0.65	4.97	0.13	0.01	-0.144
25	1	28.5	0.46%	0.65	4.81	0.14	0.01	-0.108
30	1	28.5	0.55%	0.65	4.82	0.14	0.01	-0.108
40	1	28.5	0.73%	0.65	4.83	0.14	0.01	-0.108
50	1	28.5	0.91%	0.65	4.84	0.13	0.01	-0.108
60	1	28.5	1.10%	0.65	4.84	0.13	0.01	-0.108
70	7	28.5	1.28%	4.56	4.85	0.94	0.07	-0.108
80	38	28.5	1.46%	24.78	4.86	5.10	0.37	-0.108
90	79	28	1.65%	51.51	4.87	10.57	0.76	-0.144
100	105	28.5	1.83%	68.46	4.88	14.03	1.01	-0.108

Fig. B-19

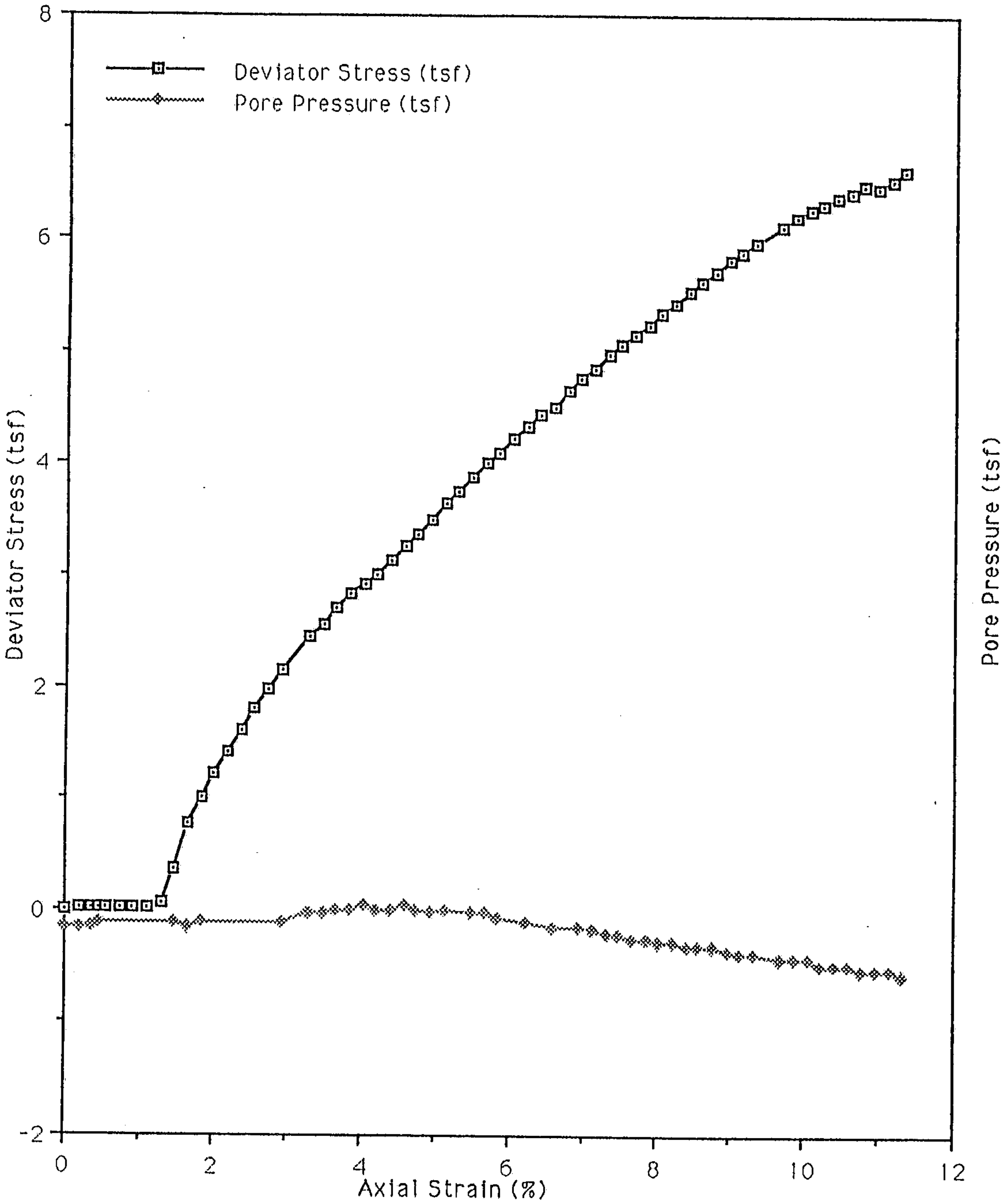
Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
110	126	28.5	2.01%	82.15	4.89	16.80	1.21	-0.108
120	147	28.5	2.19%	95.84	4.90	19.56	1.41	-0.108
130	167	28.5	2.38%	108.88	4.91	22.18	1.60	-0.108
140	187	28.5	2.56%	121.92	4.92	24.79	1.79	-0.108
150	208	28.5	2.74%	135.62	4.93	27.53	1.98	-0.108
160	226	28.5	2.93%	147.35	4.94	29.85	2.15	-0.108
170			3.11%	0.00	4.95	0.00	0.00	
180	259	29.5	3.29%	168.87	4.95	34.08	2.45	-0.036
190	271	29.5	3.47%	176.69	4.96	35.59	2.56	-0.036
200	285	30	3.66%	185.82	4.97	37.36	2.69	0
210	299	30	3.84%	194.95	4.98	39.12	2.82	0
220	309	30.5	4.02%	201.47	4.99	40.35	2.91	0.036
230	320	30	4.20%	208.64	5.00	41.71	3.00	0
240	334	30	4.39%	217.77	5.01	43.45	3.13	0
250	348	30.5	4.57%	226.90	5.02	45.19	3.25	0.036
260	361	30	4.75%	235.37	5.03	46.79	3.37	0
270	376	29.5	4.94%	245.15	5.04	48.64	3.50	-0.036
280	392	30	5.12%	255.58	5.05	50.61	3.64	0
290	405		5.30%	264.06	5.06	52.19	3.76	
300	419	29.5	5.48%	273.19	5.07	53.89	3.88	-0.036
310	433	29.5	5.67%	282.32	5.08	55.58	4.00	-0.036
320	443	29	5.85%	288.84	5.09	56.75	4.09	-0.072
330	459		6.03%	299.27	5.10	58.69	4.23	
340	472	28.5	6.22%	307.74	5.11	60.23	4.34	-0.108
350	484		6.40%	315.57	5.12	61.64	4.44	
360	493	28	6.58%	321.44	5.13	62.67	4.51	-0.144
370	510		6.76%	332.52	5.14	64.70	4.66	
380	522	28	6.95%	340.34	5.15	66.09	4.76	-0.144
390	533	27.5	7.13%	347.52	5.16	67.35	4.85	-0.18
400	547	27	7.31%	356.64	5.17	68.99	4.97	-0.216
410	558	27	7.50%	363.82	5.18	70.24	5.06	-0.216

Fig. B-19

A-494 Wrangell Dam

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
420	568	26.5	7.68%	370.34	5.19	71.35	5.14	-0.252
430	580	26.5	7.86%	378.16	5.20	72.72	5.24	-0.252
440	592	26	8.04%	385.98	5.21	74.07	5.33	-0.288
450	604	26	8.23%	393.81	5.22	75.43	5.43	-0.288
460	617	25.5	8.41%	402.28	5.23	76.90	5.54	-0.324
470	628	25.5	8.59%	409.46	5.24	78.11	5.62	-0.324
480	639	25.5	8.78%	416.63	5.25	79.32	5.71	-0.324
490	651	25	8.96%	424.45	5.26	80.65	5.81	-0.36
500	661	24.5	9.14%	430.97	5.27	81.72	5.88	-0.396
510	671	24.5	9.32%	437.49	5.28	82.79	5.96	-0.396
530	692	24	9.69%	451.18	5.31	85.04	6.12	-0.432
540	701	24	9.87%	457.05	5.32	85.97	6.19	-0.432
550	711	24	10.05%	463.57	5.33	87.02	6.27	-0.432
560	717	23	10.24%	467.48	5.34	87.57	6.31	-0.504
570	725	23	10.42%	472.70	5.35	88.37	6.36	-0.504
580	732	23	10.60%	477.26	5.36	89.04	6.41	-0.504
590	740	22.5	10.79%	482.48	5.37	89.83	6.47	-0.54
600	739	22.5	10.97%	481.83	5.38	89.53	6.45	-0.54
610	749	22.5	11.15%	488.35	5.39	90.55	6.52	-0.54
620	760	22	11.33%	495.52	5.40	91.69	6.60	-0.576

Fig. B-19



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample BC, S2-4

May 1993 A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-19

Specimen Data

Height, in.	5.64	
Diameter, in.	2.38	
Hgt./Dia. Ratio	2.37	
Sample Weight	752.17	798.40
Bulk Dens., pcf	114.20	
Pan No.	**He12**	**T36**
Wet + Tare	177.60	189.29
Dry + Tare	162.50	163.70
Tare	11.41	11.29
% Water	9.99%	16.79%

Boring:	**BC 2nd Run**
Sample:	**S2-4**
Depth:	

Equipment Constants

Axial Def. Dial:	0.001		In/Div	Proving Ring: 0.652 Lb/Div
Confining Pressure	**psi., tsf**	20.00	1.44	

Resulting Data

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
0	0	20	0.00%	0.00	4.45	0.00	0.00	0
2.5	3	20	0.04%	1.96	4.45	0.44	0.03	0
4	11	20	0.07%	7.17	4.45	1.61	0.12	0
5	20	20	0.09%	13.04	4.45	2.93	0.21	0
6.5	32	20.5	0.12%	20.86	4.45	4.68	0.34	0.036
8	41	20.5	0.14%	26.73	4.46	6.00	0.43	0.036
10	51	21	0.18%	33.25	4.46	7.46	0.54	0.072
12.5	57	21.5	0.22%	37.16	4.46	8.34	0.60	0.108
15	64	21.5	0.27%	41.73	4.46	9.35	0.67	0.108
17.5	69	22	0.31%	44.99	4.46	10.08	0.73	0.144
20	77.5	22	0.35%	50.53	4.46	11.32	0.81	0.144
22	81	22	0.39%	52.81	4.47	11.82	0.85	0.144
24	88	28.5	0.43%	57.38	4.47	12.84	0.92	0.612

Fig. B-20

Axial Deformation Dial Reading (Div.)	Proving Ring Dial Reading	Pore Pressure Reading (psi)	Axial Strain	Load lbs	Area inches^2	Deviator Stress (psi)	Deviator Stress (tsf)	Pore Pressure (tsf)
26.5	90	23	0.47%	58.68	4.47	13.13	0.95	0.216
29	96	23.5	0.51%	62.59	4.47	14.00	1.01	0.252
32	99	24	0.57%	64.55	4.47	14.43	1.04	0.288
35	104	24	0.62%	67.81	4.48	15.15	1.09	0.288
38	105	24.5	0.67%	68.46	4.48	15.28	1.10	0.324
40.5	111	25	0.72%	72.37	4.48	16.15	1.16	0.36
43.5	114	25	0.77%	74.33	4.48	16.58	1.19	0.36
46	117	25.5	0.82%	76.28	4.49	17.01	1.22	0.396
48.5	119	25.5	0.86%	77.59	4.49	17.29	1.24	0.396
63	133.5	27	1.12%	87.04	4.50	19.35	1.39	0.504
78.5	141	28	1.39%	91.93	4.51	20.38	1.47	0.576
92	152	29	1.63%	99.10	4.52	21.91	1.58	0.648
106	156	30	1.88%	101.71	4.53	22.43	1.62	0.72
122	163	30.5	2.16%	106.28	4.55	23.37	1.68	0.756
136	165	31.5	2.41%	107.58	4.56	23.60	1.70	0.828
152	169.5	32	2.70%	110.51	4.57	24.17	1.74	0.864
167	171	32.5	2.96%	111.49	4.58	24.32	1.75	0.900
198	175.5	33	3.51%	114.43	4.61	24.82	1.79	0.936
228	178.5	33.5	4.04%	116.38	4.64	25.10	1.81	0.972
259	181	34	4.59%	118.01	4.66	25.31	1.82	1.008
304	186.5	34.5	5.39%	121.60	4.70	25.86	1.86	1.044
349	191	35	6.19%	124.53	4.74	26.26	1.89	1.08
396	197	34.5	7.02%	128.44	4.78	26.84	1.93	1.044
444	199	35.5	7.87%	129.75	4.83	26.87	1.93	1.116
532	209	35.5	9.43%	136.27	4.91	27.74	2.00	1.116
624	221	35.5	11.06%	144.09	5.00	28.81	2.07	1.116
716	230	35.5	12.70%	149.96	5.10	29.43	2.12	1.116
807	240	35	14.31%	156.48	5.19	30.14	2.17	1.08
895	251	35	15.87%	163.65	5.29	30.95	2.23	1.08
990	258	35	17.55%	168.22	5.40	31.17	2.24	1.08
1074	273	35	19.04%	178.00	5.50	32.39	2.33	1.08
1125	277	35	19.95%	180.60	5.56	32.50	2.34	1.08

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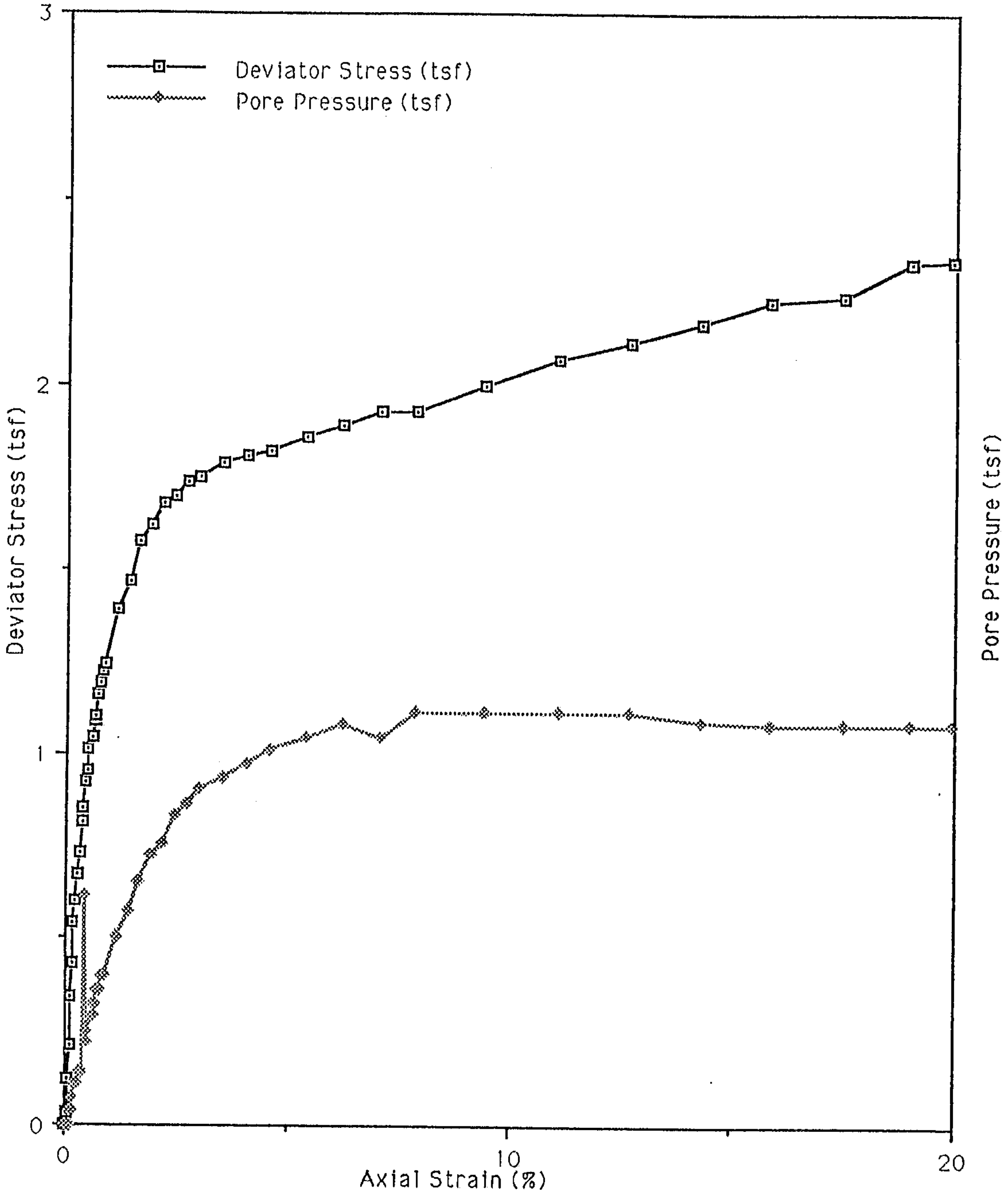
Sample BC 2nd Run S2-4 Page 2 of 2

Fig. B-20



Upper & Lower Dams Evaluation
Wrangell, Alaska

Triaxial Test of
Sample BC, S2-4, 2nd Run

May 1993 A-494

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. B-20