Prepared For:

Western Nuclear, Inc.
Sherwood Project
Wellpinit, Washington

SHERWOOD TAILING RECLAMATION PLAN

RESPONSES TO NRC COMMENTS

VOLUME 1 OF 2
APPENDICES A THROUGH I

Prepared By:

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June 2000

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DIVISION OF RADIATION PROTECTION

June 29, 2000

SMI #03-317

Mr. Gary Robertson
Washington Department of Health
Division of Radiation Protection
P.O. Box 47827
7171 Cleanwater Lane, Building 5
Olympia, Washington 98504-7827

Subject: Responses to NRC Staff Comments (Dated May 19, 2000) on the Termination Finding of the Western Nuclear, Inc.'s Sherwood Uranium Mill License Submitted by the Washington State Department of Health

Dear Gary:

As you requested, we have reviewed the letter you received from the NRC dated May 19, 2000, (NRC, 2000) that submitted 20 questions regarding the reclamation of the Sherwood tailings impoundment. We have prepared responses to the 20 questions, as well as a subsequent verbal question you received during a meeting with the NRC. The responses that are presented below incorporate previous information that was submitted to your agency over the last 6 years. In many cases, several questions can be grouped and answered with one response. The following presents the NRC questions followed by our response:

NRC Q1: ***Please provide further information and justification to confirm that the formation of sand boils was considered, and that resulting damage could be accommodated by the design.***

NRC Q3: ***Please provide additional information and documentation to confirm that an appropriate PGA, including amplification, if necessary, was considered in the stability and liquefaction analyses.***

NRC Q4: ***Please provide additional information and documentation to support your conclusion regarding the potential for recharge of the tailings. If there is potential for ponding water to infiltrate and recharge the tailings, please provide additional information and documentation to confirm that an increased likelihood of liquefaction of a wet embankment was considered.***

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NRC Q10: ***Please provide additional information on this subject (Geologic and Seismologic Characterization) sufficient to understand the subsequent brief discussion of site stability.***

NRC Q12: ***Please provide additional information and discussion related to specific local bedrock features, especially discontinuities such as faults and fractures, for consideration in seismotectonic hazard analyses.***

NRC Q13: ***Please provide additional information and discussion of WDOH's findings related to its review of key references and the geologic map of Coulee Dam Vicinity (Waggoner report, 1990, Ref. 4) The TER points out a large discordance in the structural trends at the site. Waggoner indicates north-south; Shepherd Miller, Inc. (Reclamation Plan, 1994, Ref. 5) indicates east-west. Please provide further discussion and clarification of the significance of this discordance in WDOH's determination that all applicable standards and regulations have been met.***

NRC Q15: ***Please provide additional information and discussion of WDOH's evaluation of earthquake sources (such as capable faults) and earthquake hazards for the site. The information should include discussion of seismic design basis (maximum credible earthquake or reasonable alternative basis) for the engineering structures and WDOH's evaluation of liquefaction potential.***

The preceding 7 questions all relate to seismic stability of the reclaimed site. Western Nuclear Inc. (WNI) performed a number of analyses starting in 1994 to address the seismicity of the region and designed and constructed the reclamation facility to be stable under the seismic forces for the 1,000-year-design life of the reclaimed facility. The following chronology describes the analyses that were performed:

A comprehensive regional and local geologic evaluation was conducted. This evaluation identified key geologic structures both regionally and locally that provided a basis for understanding current site conditions as well as the expected future seismic activity. This evaluation is included in Attachment C to Appendix P of the 1994 Reclamation Plan (WNI, 1994) and is included in this report as Appendix A. This information gives the general geologic setting of the region and also includes site-specific geological features. Additionally, a separate report (prepared by R.L. Volpe & Associates, Inc., 1994) included in Attachment C of Appendix L of the 1994 Reclamation Plan (WNI, 1994) presents a geologic evaluation more closely focused on the seismicity of the region. This report is included as Appendix B to this report.

Based on the background geologic information, an evaluation of the seismic forces that should be used in the design of the reclamation system was made. This evaluation is also included in the report attached as Appendix B to this submittal.

The seismic forces that were assigned to the site were then used in the design and evaluation of the reclamation system. Specifically, the stability of the embankment outslope was evaluated under seismic loading conditions. This evaluation was included in Appendix N of the 1994 Reclamation Plan (WNI, 1994) and is included as Appendix C to this report.

The performance of the reclamation cover system under seismic loading was also evaluated. The performance of the homogenous cover system was evaluated relative to sand boils, rafting, and settlement. This evaluation used a much more conservative seismic loading condition than the one presented in Appendix B of this submittal. The seismic loading conditions used in the cover evaluation assumed a peak ground acceleration of 0.15 g (which is considerably greater than would be expected during the 1,000-year-design life as documented in Appendix B of this submittal). This seismic scenario was thought to represent a very conservative upper bound of the anticipated earthquake loading and would, therefore, give very conservative seismic stability results. The determination of the larger seismic loading conditions was included in Appendix 3 of the Revegetation Reclamation System Evaluation Report (WNI, 1995) and is included in Appendix D of this submittal. The results of the evaluation were also included in Appendix 3 of the Revegetation Reclamation System Evaluation Report (WNI, 1995) and are also included in Appendix D of this report. The results consist of the original submittal dated May 5, 1994, and a subsequent submittal dated September 13, 1995, that responded to questions from Gerald LaVassar of the Washington Department of Ecology – Dam Safety Section.

In summary, the results of the evaluations clearly show that a conservative design earthquake event was determined using all information that was available at the time of the original study, and that the performance evaluation of the reclamation system would not be adversely impacted by the design seismic event. Specifically, the embankment outslope would be stable under the anticipated maximum earthquake loading during the 1,000-year-design life. Additionally, the cover of the reclaimed impoundment would perform successfully under earthquake loading much larger than would be expected during the design life. The cover was evaluated and found to possess adequate factors of safety relative to rafting, settlement, and the formation of sand boils.

NRC Q2: ***Please provide additional information and documentation to confirm that the embankment stability under saturated conditions was considered.***

The stability of the reclaimed embankment was evaluated in Appendix N of the 1994 Reclamation Plan (WNI, 1994). This evaluation is attached as Appendix C of this submittal. As can be seen, the evaluation clearly shows that the reclaimed embankment will be stable. The evaluation assumed that there would be no phreatic surface in the embankment since the embankment is separated from the tailings by an impermeable liner, the embankment is constructed of a free draining material that would drain faster than water could seep from the tailings if the liner were to fail, and that the depth to water (or a low permeable layer) is over 150 feet that would require saturation before a phreatic surface could form.

However, the original design report for the tailings dam (D'Appolonia, 1977) assumed that a fully formed phreatic surface would exist in their evaluation of the stability of the tailings dam. Figure 7 from the D'Appolonia report is included in Appendix C and clearly shows that the tailings dam was stable under static and psuedostatic loading with saturated conditions in the embankment. Given that the embankment outslope has been flattened from 2.75:1 (h:v) to 5:1 (h:v), and the embankment is 45 feet shorter than originally designed, the embankment stability has a factor of safety much greater than required even if the embankment material were to become saturated (which, as stated above, could not occur).

NRC Q5: ***Please provide additional information and documentation to confirm whether this dam will be classified as a dam under the Federal Guidelines for Dam Safety and the National Dam Safety Program Act.***

The former tailings dam at the Sherwood Site has been reclaimed. The stability of the outslope and the reclamation cover over the tailings was evaluated under a wide range of static and seismic conditions, as described in this report. The evaluation of the reclamation system indicates that the tailings will remain isolated and contained under all scenarios for the 1,000-year-design period.

The tailings dam was operated and maintained under the Washington Department of Ecology – Dam Safety Office (WDOE-DSO) from the construction of the dam through the reclamation of the dam. In a letter dated December 15, 1997, (WDOE, 1997) the WDOE-DSO confirmed that the "provisions of the Dam Safety Section's reclamation requirements have been satisfied, and the project is hereby classified as reclaimed." In a recent letter dated June 23, 2000 (WDOE, 2000), the WDOE-DSO remained steadfast in its opinion that the engineering assessment of the reclaimed impounding structure is valid and that the reclaimed barrier represents "a practical scheme to provide a high likelihood of the structure safely impounding the process waste for the thousand-year design-life assuming little, if any, maintenance." Copies of both the December 15, 1997 and the June 23, 2000 letters are included in Appendix E of this submittal.

Jerald LaVassar of the WDOE-DSO met with representatives of the NRC, DOE, FERC, WDOH, and WNI at the site on June 21, 2000. In his letter of June 23, 2000, (WDOE, 2000) Mr. LaVassar states the WDOE-DSO views that "the reclaimed impounding barrier is a dam" and "is considered a jurisdictional dam under the provisions of Washington Administrative Code (WAC) 173-175-020." The practical consequences of such classifications are that the barrier would be inspected on a 6- to 8-year interval or in the event of an extreme storm or earthquake. There would be no cost for periodic inspections and report of findings. On the jurisdictional issue, the letter states that "The project would be removed from our jurisdiction in the event a Federal Agency assumes ownership of the project, . . ."

The DOE is the proposed long-term custodian of the site under a Long Term Surveillance Plan to be approved by the NRC. The DOE has negotiated an Access and Maintenance Agreement with the Spokane Tribe of Indians. The land, including the reclaimed barrier, remains Tribal land and the DOE has access for inspections and maintenance required by UMTRCA. Inasmuch as DOE

will have no legal rights of ownership, it seems that the WDOE-DSO will likely retain jurisdiction of the reclaimed barrier under the relevant provisions of the WAC.

The current WDOH licensee, WNI, takes the position that, without agreeing or disagreeing with the technical and legal conclusion that the barrier is a dam, the fact that the WDOE-DSO retains jurisdiction presents no impediment to achieving site closure and license termination by August 2000 as presently scheduled by the NRC. The Long Term Surveillance Plan can incorporate a provision for periodic inspection and reports by the WDOE-DSO. In the remote instance that a deficiency be found with the integrity of the barrier, the DOE's obligation for necessary repairs would be no greater than that already imposed by UMTRCA for the containment and stabilization of by-product material. The DOE would be exposed to no greater responsibility or liability than it otherwise would have. The WDOE-DSO jurisdiction just provides another layer of institutional control.

Inasmuch as it would appear unlikely that a federal agency would assume ownership of sovereign Indian property, it is unnecessary to undertake a FERC review for it to determine whether the impounding barrier is a dam. That determination has already been made by the WDOE-DSO having jurisdiction. Additional federal review would seem to be duplicitous, unnecessary, and jurisdictionally problematic.

NRC Q6: ***Please provide additional information and discussion of rock durability test results that supports WDOH's final approval of the quarry for riprap source.***

NRC Q7: ***Please provide additional information and justification of the representativeness of the 3 samples on which durability estimates were based. Based on field photos, the samples tested do not appear to be representative of the rocks used and could have led to underestimation of rock durability.***

NRC Q8: ***Please provide additional information and justification of the acceptability of the rock that has already been placed to function for the performance period of 1000 years and at least 200 years, given that some areas have degraded. The objective is to get a more realistic basis for projected performance of the rocks than can be gotten from more pristine samples from quarry walls.***

NRC Q9: ***Please provide further information and analyses that demonstrate that large areas of non-quartz monzonite rock or poor quality quartz monzonite rock have not been placed in the rock cover, particularly in the diversion channel.***

NRC Q21: ***(This question was added during a May 24, 2000, meeting in Spokane, Washington, and is paraphrased from the conversation.) Please provide information that standing water or freeze/thaw effects on weathering of rock (riprap) has been considered during WDOH review of rock durability and longevity in relation to millsite performance in meeting 10 CFR 40 Appendix A criteria.***

Questions 6 through 9 and 21 all relate to the durability of the rock used as erosion protection for the site. A brief discussion of the sampling and analyses of the rock along with references of previously submitted material is provided that demonstrates that the riprap that was used meets the requirement of the reclamation plan that were developed in accordance with NRC guidance on rock durability.

An initial evaluation of the available on and near-site rock sources was conducted in 1994 and documented in Appendix B of the 1994 Reclamation Plan (WNI, 1994). This is included as Appendix F to this submittal. This report indicated that the on-site basalt rock would be acceptable for use as riprap for any application, and the quartz monzonite material was marginal. The testing and evaluation were conducted using NRC guidance (NRC, 1990). The evaluation consisted of petrographic analyses as well as physical durability testing.

After the initial testing, another source of on-site quartz monzonite material was identified that appeared to have better durability qualities than the originally sampled locations. Subsequent petrographic and physical durability samples were obtained and tested. The results of the testing indicated that the quartz monzonite material from the new area that ultimately became the quarry would be acceptable. The results of the testing was included in the Construction Completion Report (WNI, 1997) and is attached as Appendix G to this submittal.

Included in Appendix H are the field logs of WDOH personnel that are relevant to the durability of the quartz monzonite from the quarry. These field logs were originally included in Appendix Z of the Construction Completion Report (WNI, 1997). Specifically the field log dated March 11, 1996, written by Dorothy Stoffel, WDOH geologist, documents her visual observation of the proposed quarry area. Her observations are consistent with the determination that the quartz monzonite in the quarry area is durable. Subsequent observations made by WDOH confirm that the quartz monzonite material appeared durable as the quarrying operation continued.

After the initial testing that indicated the quartz monzonite area would be acceptable for use as rock protection, durability tests were conducted on samples taken that represent every 10,000 cubic yards of rock produced. The samples were taken from the quarry after the area was blasted and before the material was crushed and processed. The rock samples were taken by AGRA Earth and Environmental technicians. The samples were taken to be representative of the area blasted, and every effort was taken to not bias the samples based on visual differences in the material (personal communications with Jay Martin, AGRA Earth and Environment, June 16, 2000). Documentation of the sample locations and results of the durability testing were submitted in the Construction Completion Report and are included in Appendix G of this submittal.

The results of the durability testing clearly indicate two key pieces of information. First, all of the rock meets or exceeds the minimum durability requirements of the NRC guidance with the exception of one sample which scored 79 instead of 80. The rock that was represented by this one test result was oversized by 1 percent over the design requirement as required by the NRC guidance. The second key point is that the durability of the quartz monzonite material was very

uniform. The material scored from 79 to 81 which indicates that the source was very uniform. That combined with the random nature of the sampling procedures clearly shows that the samples were representative of the quarry.

It should be noted that the basalt material that was also used as riprap scored much higher than the quartz monzonite (durability rating of 90 percent). This indicates that the basalt is more durable than the quartz monzonite, which is counter to the implication made in Question 9.

The guidance provided by NRC gives minimum durability ranking for rock to be used in various conditions. Specifically, rock that is located in areas that could be frequently saturated should have a score of at least 80 percent or be oversized. Since all of the durability requirements for the riprap on site were for the most restrictive conditions (i.e., areas that could be frequently saturated), all of the rock that was placed meets the guidance requirements for rock that might be in standing water and subjected to freeze/thaw events.

There is no indication that any significant amount of rock that would not meet the NRC guidance for durability is concentrated in any particular area. As stated above, the random nature of the sample selection along with the consistent values that were obtained from the durability testing clearly indicates that the rock is uniform and meets the durability requirements as outlined in NRC guidance.

The information discussed above and attached to this document clearly shows that the rock that was used for erosion protection meets the requirements of the approved reclamation plan that were developed using NRC guidance. However, it is also important to note that the conservative nature of the design would not necessarily require that rock be used for erosion protection at all. This is especially true after vegetation becomes established in areas that received riprap.

Much of the diversion channel and all of the swale outlet was excavated into quartz monzonite bedrock. This underlying material will be resistant to erosion if riprap would not have been placed in these areas. Further, analyses show that erosional velocities will not occur in the diversion channel after vegetation has become established. An evaluation was performed to determine the necessary size of the diversion channel after vegetation becomes established. This evaluation shows that the maximum velocities in the channel overbank would range from 0.3 to 1.5 ft/sec and the maximum velocity in the channel would range from 0.9 to 4.9 ft/sec, which is less than 5 ft/sec that the NRC STP on erosional stability recommends as the maximum velocity for grass lined channels. These analyses were included in the Responses to WDOH Comments on the December 1994 Tailing Reclamation Plan (WNI, 1995) dated August 1995 and included in this report as Appendix I. While similar calculations were not performed for the swale outlet and the embankment outslope, similar results would be anticipated.

In summary, the sampling, testing, and analyses of the riprap material were in accordance with NRC guidance. Further, the results of the testing indicate that the rock is acceptable for use as erosion protection for reclamation of uranium mill tailings. Finally, the conservative nature of

the reclamation design shows that rock protection is likely not necessary at the site, especially after vegetation becomes established.

NRC Q11: ***Please provide additional information and discussion of WDOH's findings on its review of the key reference materials relevant to site stability analysis.***

NRC Q14: ***Please provide additional information, technical discussion and/or summaries of operative surface processes, including but not limited to mass movements, stream erosion/deposition potential at the site that supports a finding that there are not potential processes which would lead to impoundment instability.***

These two questions relate to the overall geologic or geomorphic stability of the site. The discussion of the geological setting (Appendix A to this submittal) provides a good framework from which to understand the geologic and geomorphic conditions at the site.

The geologic setting of the area is very stable and is expected to remain so for many thousands of years. The geologic stability of the area is provided by the Loon Lake Granite Pluton. This massive geologic formation underlies the entire area and would prevent any significant geomorphic instability.

Sandy alluvial deposits overlie the granitic pluton in the area of the tailings impoundment. This sandy alluvial material varies in depths from a few feet to approximately 200 feet at the toe of the embankment outslope. This material provides an excellent base on which the tailings impoundment was founded. The unsaturated granular nature of the alluvial material precludes any settlement concerns, and the geotechnical stability of the foundation is more than sufficient to support the reclaimed impoundment.

The slopes around the reclaimed tailings impoundment are gently sloping and there is no evidence of landslides or other mass movement. There is very little evidence of surface erosion in the undisturbed surrounding areas. The lack of erosion is due to the gently sloping surfaces, the high infiltration rate of the sandy alluvial material, and the mature vegetation community. Confluences were designed and constructed to convey water from the drainage basin above the reclaimed impoundment in an erosionally stable manner. This, combined with the relatively small total water shed area, contributes to stable hydraulic conditions.

In conclusion, the geomorphic and geologic conditions at the site are conducive to the long-term stabilization of the reclaimed tailings impoundment.

NRC Q16: ***Please provide documentation demonstrating that the review and acceptance, if appropriate, of licensee submitted information pertaining to the impacts to groundwater caused by potential releases of liquids from the disposal cell, given credible failure scenarios of the engineering design components of the disposal cell. This information should not be limited to synthetic liner failure and over-***

topping from water buildup, but include any other credible scenario that could cause release of liquids.

There are no credible failure scenarios that could release water from the impoundment into the groundwater system other than overtopping. That scenario, along with the worst-case bounding scenario of liner failure was evaluated to determine the expected and the worst-case bounding scenario impact of liquids in the impoundment on groundwater. Even under the worst-case condition of complete liner failure, groundwater at the point of compliance would meet site standards. A complete description of the groundwater conditions at the site and the modeled prediction of future concentrations are included in the Groundwater Technical Integration Report (WNI, 1995) which is attached as Appendix J. As this document shows, groundwater will remain protected under the worst-case scenario and would, therefore, remain protected for any other scenario.

NRC Q17: ***Please provide discussion of results of confirmatory soil samples and radiation surveys (including highest, lowest and average values, and data comparisons between WNI and WDOH results) that indicates that the subject site has been cleaned up to the State standards (including uranium and thorium limits) for both surface and subsurface soil.***

A comprehensive radiological program was conducted at the site to determine areas with residual radioactive contamination greater than applicable standards and to verify that those areas had been remediated. The Radiological Verification Completion Report – Executive Summary (Volume 1 of 11) and Report (Volume 2 of 11) (WNI, 1996) summarize the program and are included as Appendix K.

A total of approximately 375,000 cubic yards of material was excavated from the mill area and around the tailings impoundment and placed in the impoundment. A total of 4,968 gamma surveys and 1,320 soil samples were taken to verify that the areas outside of the impoundment could be released for unrestricted use. The program included standards for radium, uranium, and thorium. A summary of the laboratory test results is presented on Figures ES-7 (radium), ES-8 (thorium), and ES-9 (uranium) in Appendix K. Tables 14, 15, and 16 from the main report, attached as Appendix K, present the comparison between WNI and WDOH laboratory test results.

As can be seen, the results of the gamma surveys and the laboratory analyses clearly show that all areas have residual radioactive contamination well below the regulatory limits with the vast majority of the areas at background levels. Additionally, the WDOH laboratory results confirm that all areas have been cleaned up to applicable limits.

NRC Q18: ***Please provide information on the cleanup criteria used for remaining structures, if any, to demonstrate compliance with the State's equivalent of 10 CFR 40.42(k)(2).***

A water tank and a pump house exist on the former millsite area. The building and the water tank were surveyed for surface contamination. All contaminated materials were removed and buried in the tailing impoundment. This information was documented in the Mill Decommissioning Completion Report (WNI, 1997) and is attached as Appendix L.

NRC Q19: ***Please provide information and discussion of the evaluation of the site's compliance with the State's equivalent of 10 CFR 40 Appendix A criteria 6 (2) and (5), concerning the overall gamma radiation level and radioactivity content of the cover material.***

After the cover was placed, radon measurements were taken in accordance with Appendix A criterion 6 (2). The results of this testing were submitted to WDOH on December 16, 1996 (attached to this report as Appendix M). The results of the testing indicated an average radon emanation rate of 0.51 pCi/m2sec which is well below 20 pCi/m2sec specified in criterion 6.

All cover material was obtained from borrow sources around the tailings impoundment. All of this material was used for cover only after the areas were determined to meet the radiological cleanup criteria as discussed above (see Appendix K). The cover material was obtained from near surface soils and had background levels of radionuclides. The background levels are approximately 1 pCi/g for radium-226 and thorium-230 and 2 pCi/g for natural uranium. Appendix K presents a complete summary of the background values for the near surface soils that were used for the cover.

NRC Q20: ***Please provide additional information to support your basis that WNI's remedial work was performed according to the approved plans and specifications.***

In August 1999, WDOH submitted 12 questions resulting from field inspections of the reclaimed site. These questions were address as part of the Request for License Termination (WNI, 1999). The applicable portions of this report are attached as Appendix N.

As documented in Appendix N, all areas identified by WDOH were addressed. Some of the issues were addressed by submittal of information that demonstrated that no additional work was necessary, and that the elements of the reclamation system were performing as designed. Remedial reclamation work was performed to address the remaining areas. Appendix N presents a discussion of each question, the design of the remedial efforts (for elements that required a design effort), the activities that were performed, and the site stability inspection that was performed by an independent third party engineer (Sheila Pachernegg) that confirmed that the remedial efforts were successfully completed. In addition to WNI's efforts, WDOH performed site inspections of the remedial efforts during the construction process. Their inspections concluded that the required remedial effort was performed as required.

We trust that these responses will assist you in responding to the NRC. Should you need any additional assistance, please let us know.

REFERENCES

D'Appolonia Consulting Engineers, Inc., 1977. Earth Dam Design Tailings Storage Facility, Western Nuclear Inc., Sherwood Project, Spokane, Washington. Project No. RM77-400. July.

State of Washington Department of Ecology (WDOE), 1997. Letter from Jerald LaVassar to Stephanie J. Baker. December 15.

State of Washington Department of Ecology (WDOE), 2000. Letter from Jerald LaVassar to John Blacklaw. June 23.

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U.S. Nuclear Regulatory Commission (NRC), 2000. Letter from Paul Lohaus to John Erickson. May 19.

Western Nuclear, Inc. (WNI), 1994. Sherwood Project Tailing Reclamation Plan. December.

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Western Nuclear, Inc. (WNI), 1997. Sherwood Project Mill Decommissioning Construction Completion Report. May.

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Sincerely,

SHEPHERD MILLER, INC.

Louis L. Miller by HMR

Louis L. Miller, P.E.
Vice President

LLM:hmr
Enclosures

SHERWOOD PROJECT
TAILING RECLAMATION PLAN

Volume 6 of 7
APPENDIX P

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December 1994

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ATTACHMENT C
GEOLOGIC INVESTIGATION

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ATTACHMENT C

INTRODUCTION

A multi-phase geologic investigation was initiated for Western Nuclear, Inc. at their Sherwood Project tailing impoundment facility to provide a physical framework for point of compliance ground water monitoring. The tailing impoundment facility area is next to the reclaimed Sherwood Project mine. The following attachments discuss the four phases of this investigation which consisted of a review of the geologic literature, a geologic field mapping study, a borehole geophysical study, and a seismic study. References for all Attachment C are included at the end of Attachment C.4.

ATTACHMENT C.1
REVIEW OF GEOLOGIC LITERATURE

REVIEW OF GEOLOGIC LITERATURE

Existing geologic literature was reviewed to establish a geologic framework for the ground water characterization associated with the closure of the tailing impoundment facility. References are listed at the end of this section.

Geologic Setting

The Sherwood tailing impoundment facility area is located in southern Stevens County, in northeast Washington approximately 40 miles northwest of Spokane on the Spokane Indian Reservation (Figure C.1.1). The mine is located between 47°45'N and 48°00'N, and 18°00'W and 18°15'W and is on the southern edge of the Okanogan Highlands physiographic province, just north of the Columbia River Plateau. The site is located to the north of the Spokane River arm of Franklin D. Roosevelt Lake, to the east of Blue Creek, and south of the Oyachen drainage system.

The topography of the area is relatively mature and consists of gently sloped hills and valleys with steeply sloping bluffs to the south. The area has been mapped as containing two primary topographic terrain units (Dames and Moore, 1976). The first topographic terrain unit is characterized by gentle slopes of less than 1V:5H, and a relatively smooth ground surface which drains toward the south and southwest and ranges in elevation between 1,850 feet and 2,330 feet above mean sea level. The second terrain unit is characterized by steep slopes ranging from 1V:5H to 1V:1H, is dissected by gullies which trend west to southwest, and ranges in elevation from 1,370 feet to 2,190 feet.

The tailing impoundment facility area lies between two regional structural lineaments: the Spokane River-Enterprise Valley Lineament (Becraft and Weis, 1963) to the south

and the Lewis and Clark Lineament to the north (Robbins, 1978) (see Figure C.1.1). Both structures possess right lateral strike slip offset. Rocks at the facility area range in age from late Cretaceous to Holocene and include granitic, andesitic, basaltic, and tuffaceous igneous lithologies as well as silts, sands, shales, and conglomerates (Becraft and Weis, 1963; Dames and Moore, 1976; D'Appalonia, 1977). The site is underlain by Cretaceous igneous rocks of the Loon Lake Granite, which are of granitic and syenitic composition, with Oligocene Gerome Formation sedimentary and pyroclastic rocks that are locally preserved. Quaternary glacial and alluvial sands and gravels cover most of the bedrock and control the majority of surface topography at the Sherwood Project.

Geologic History

Yates et al. (1966) locate the Sherwood Project in the tectonic regime of the Kootenay Arc subprovince within the eastern cordilleran tectonic province. The Kootenay Arc is a fold belt mapped from Canada to the Columbia River Plateau, a result of compressional tectonics in the late Jurassic or early Cretaceous period. The axes of these folds trend north to northeast and are associated with some faulting. Some of these folds to the west of the Kootenay Arc Subprovince exhibit up to 2½ miles of right lateral offset (Yates et al., 1966).

Though rocks as old as the Precambrian are present to the north of the Sherwood tailing impoundment facility (Becraft and Weis, 1963), the geologic history of the facility area begins in the mid- to late-Cretaceous with the emplacement of the granitic Colville-Loon Lake Batholith (Waters and Krauskopf 1941; Becraft and Weis 1963; Yates et al. 1966). The emplacement of the granitic rocks in the Sherwood Project area, named the Loon Lake Granite (Weaver 1920), was associated with a period of severe structural deformation, including north-south normal faulting, which continued

into the Oligocene epoch (U.S. Dept. of the Interior, 1975; Yates, 1966; Becraft and Weis, 1963).

Becraft and Weis (1963) describes five main rock types of the Loon Lake Granite mapped in the Tailing impoundment facility area which is located within the Turtle Lake Quadrangle: granodiorite, porphyritic quartz monzonite, equigranular quartz monzonite, fine-grained equigranular quartz monzonite, and fine-grained quartz monzonite rich in mafic minerals. Though their relationship in the mine area is unclear, in at least one location in the Turtle Lake Quadrangle the granodiorite is evidently older than the quartz monzonites (Becraft and Weis, 1963). The pluton is intruded by many small alaskite, aplite, and pegmatite dikes composed primarily of quartz and potassium feldspar with minor amounts of biotite, muscovite, and magnetite. The texture and composition of these dikes, however, varies considerably within a small area. The dikes tend to be more resistant to erosion and appear in outcrops as low ridges (Becraft and Weis, 1963). A period of regional uplift followed the emplacement of the pluton, unroofing the igneous rocks and creating an erosional unconformity between the Cretaceous and Tertiary lithologies.

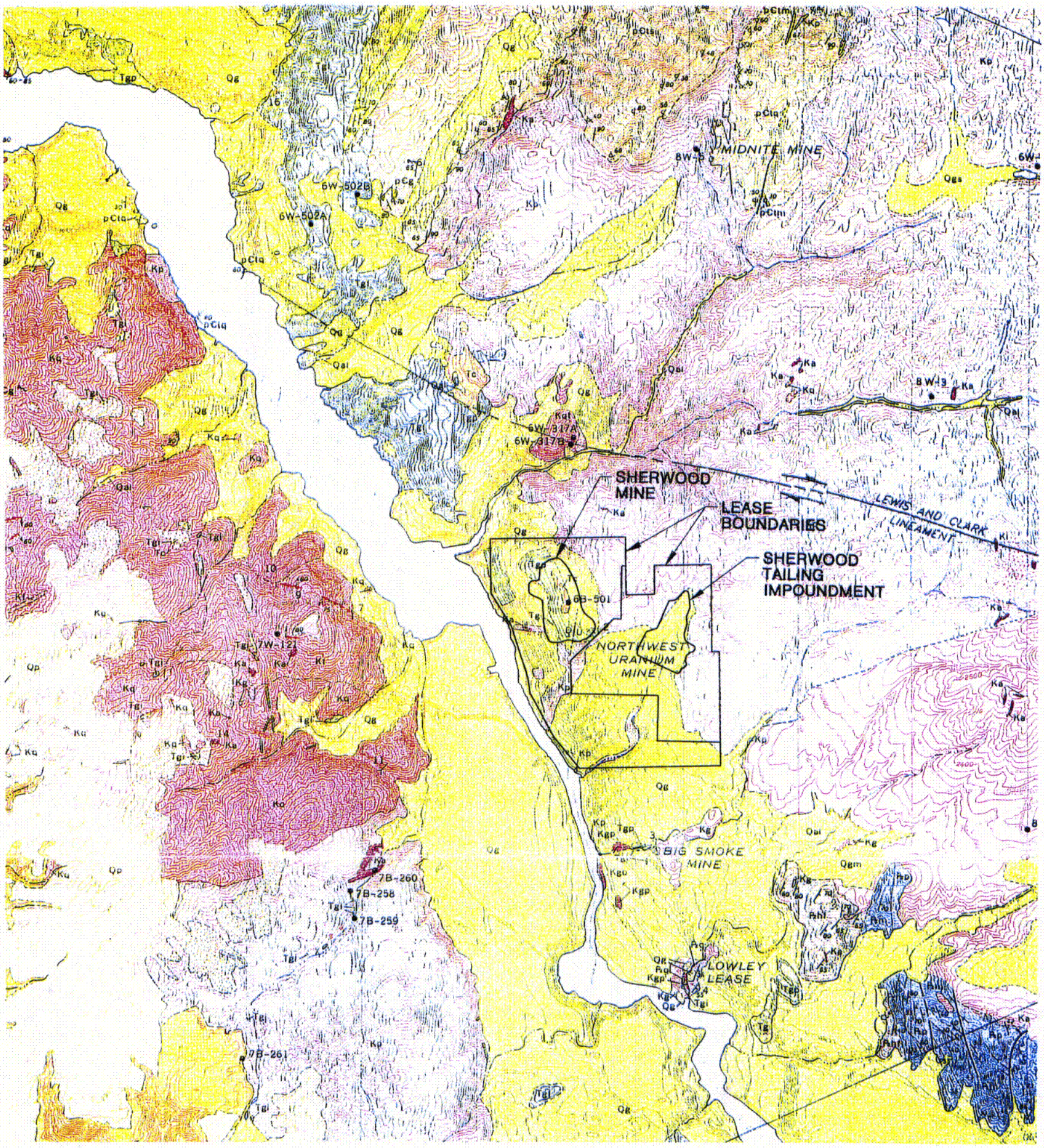
The Loon Lake Granite is unconformably overlain locally by the Gerome Formation, which is dated as Oligocene (Becraft and Weis, 1963; Yates et al., 1966). The Gerome Formation consists of a basal boulder conglomerate of pre-Tertiary rocks and sequences of arkosic sandstone, carbonaceous shale, tuffaceous sandstone, welded tuff, and andesitic lava flows (U.S. Dept. of the Interior, 1975; Becraft and Weis, 1963). In the Sherwood Project area the andesite dikes of the Gerome Formation are most commonly found near the Spokane River Valley, suggesting that they intruded along a fault zone, referred to as the Spokane River-Enterprise Valley Lineament, which controls the valley orientation (Becraft and Weis, 1963). The lineament, which has suggested right lateral displacement of over one mile, roughly parallels the Lewis

and Clark Lineament to the north, which also has an indicated right lateral displacement (Robbins, 1978; Huntting et al., 1961) as illustrated in Figure C.1.1. Becraft and Weis (1963) indicates that the association between the Gerome andesite and the Spokane River-Enterprise Valley Lineament suggests a pre-Gerome, pre-Oligocene date for these right lateral, regional structures which bound the Sherwood Project to the north and south. These regional deformations continued through the Oligocene and included north-trending normal faults that cut the Gerome Formation and preserved thick sequences of the clastics and volcanics in the down-dropped blocks. Eickelberg (1979) shows several normal faults in the pit area. These faults trend roughly north with west side down or trend north west with north east side down. Yates et al. (1966) reports that thick sequences of Oligocene continental clastic and volcaniclastic rocks are preserved in the down-thrown blocks or grabens in the pluton to the west of the facility area in the same way the orebody sediments were preserved at the Sherwood Project. Dames and Moore (1971) indicates that the main pit fault may actually be a fault zone composed of several en echelon fractures which may have decreasing dip with depth. Two dominant joint orientations are mentioned with the first paralleling the pit fault at N.10°W. and dips of 70°-80° SW., and a second orientation of approximately N.45°E. with dips of 20°-40° SE.

Following the deposition of the Oligocene clastics, the encroachment of the Columbia River Basalts from the south is the next recorded depositional event. These basalt flows formed a thick sheet as the result of several intertwining flows (Griggs, 1976). Becraft and Weis (1963) indicates that no faulting of these Miocene flows is observed in the study area.

Periods of glacial scouring and sediment deposition have dominated the last 32,000 years of geologic activity in the Sherwood Project area and have modified the post-miocene geomorphology. The ice sheet of the Bull Lake Glaciation covered the facility

area until 32,000 years ago (Richmond et al., 1965), scouring the topography and later depositing outwash sediments. The Pinedale Glaciations which followed did not cover the facility, but created several ice dams which, when breached, scoured the Columbia and Spokane River Valleys and deposited significant amounts of silts, sands, clays, and gravels (Richmond et al., 1965; Bretz, 1930). The Spokane River valley has experienced a period of overall erosion since the last glaciation that has eroded significant portions of the glacially-deposited alluvium and has left successive river terraces on both side of the valley.



EXPLANATION

- Qal — Alluvium
- Qgs — Glacial lake silt
- Qg — Glacial deposits undivided
- pCtq, pCtm, pCtq, pCts — Togo formation
- Tc — Columbia River basalt
- Tgf — Flows of Gerome andesite
- Tg — Gerome andesite undivided
- Kg — Granodiorite
- Kqf — Fine-grained quartz monzonite rich in mafic minerals
- Kq — Equigranular quartz monzonite
- Kgp — Granite porphyry dikes
- Ka — Alaskite, aplite, and pegmatite
- Kp — Porphyritic quartz monzonite

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SMI
SHEPHERD MILLER, INC.

FIGURE C.1.1
GENERAL SITE GEOLOGY
(AFTER BECRAFT & WEIS, 1963 AND ROBBINS, 1978)

Date:	DEC., 1994
Project:	317\T39\
File:	GEO-FIG

ATTACHMENT C.2

GEOLOGIC FIELD MAPPING STUDY

GEOLOGIC FIELD MAPPING STUDY

Shepherd Miller Inc. (SMI) performed geological mapping of the Sherwood tailing impoundment facility area from August 23 to September 2, 1993. The objectives of this mapping were to: 1) map existing geologic rock outcrops, 2) map joint, fracture, shear, and fault orientations, and 3) provide data for the evaluation of the influence of geologic structures on ground water flow. The scope of this field mapping included the tailing facility and surrounding area. The purpose of this mapping included the establishment of baseline geologic conditions and the selection of areas for future seismic and surface geophysical investigation. This mapping effort was intended to supplement the extensive general field reconnaissance performed by Western Nuclear, Inc. (WNI) over the past 16 years.

Geologic Mapping

Mapping was focused on the area around and adjacent to the tailing impoundment facility and included the mill and pit area, and the drainages to the southwest and southeast of the impoundment. Vegetative and alluvial cover were extensive. Outcrops were scarce,generally weathered, and poorly exposed. As a result, the mapping area was extended beyond the tailing impoundment facility area in an effort to gather as much relevant geologic data as possible.

More than 100 rock outcrops were identified; lithologic, textural, and structural features were noted and recorded at 97 different outcrops. Strikes and dips of joints, slickensided surfaces, faults, and fractures were measured and are summarized in Table C.2-1. In addition, WNI staff with detailed knowledge of the area were questioned and contributed to the location of outcrops and interpretation of the field data. Figure C.2.1 illustrates the outcrops identified during the mapping effort. Figure

C.2.2 illustrates the interpretation of the overall geology of the area, based on the review of aerial photographs and the field mapping.

Field mapping results and review of the geologic literature both indicate that the Cretaceous igneous rocks and the Tertiary Gerome Formation sediments and intrusives are fractured and faulted. Most deformation appears to result from extensional tectonic forces with faulting mostly normal, dip-slip, or strike-slip in nature. Outcrops tend to coincide with topographic highs or deeply incised gullies. The walls of the impoundment diversion channel and the pit wall provided excellent exposure and insight into the structures associated with the bedrock material.

Lithologies

The existing geologic literature identifies the igneous rocks found on and around the Sherwood Project as quartz monzonites and granodiorites of Cretaceous age. Hand sample identification and thin section analysis on selected samples indicate that the lithologies include quartz monzonite and granite, rather than granodiorite. The quartz monzonite samples were composed of approximately 15 to 20 percent quartz, 35 to 50 percent potassium feldspar, 35 to 40 percent plagioclase feldspar, and less than 5 percent biotite or muscovite. The quartz monzonite was present in many lithologic phases including equigranular, fine-grained equigranular, and porphyritic. The granite samples were composed of approximately 30 percent quartz, 30 percent potassium feldspar, 30 percent plagioclase feldspars, and up to 10 percent biotite and muscovite. The quartz-rich phases of the Colville-Loon Lake Batholith observed at the Sherwood Project are classified in this report as granite, due to the higher potassium feldspar to plagioclase feldspar ratios. The lithology identified by Becraft and Weis (1963) as granodiorite may occur in other areas. The boundary between the quartz monzonite and the granite, as mapped in the field, was several hundred feet east of

the tailing impoundment facility, approximately ¾ miles west of the contact identified by Becraft and Weis (1963).

Although field mapping identified the various igneous lithologies and textures, no correlation between lithology and mechanical behavior (i.e. fracturing, jointing) or structural control, which may influence ground water flow was observed. Therefore, the igneous lithologies are not distinguished on Figure C.2.2 of this report.

Petrographic analysis on thin sections of selected field hand samples indicates that the igneous rocks have experienced at least one and possibly several periods of melting and re-crystallization during early phases of emplacement. Evidence of deformational strain is preserved in the quartz grains as undulatory extinctions under cross-polarized light.

Structures

Four primary structural patterns were observed at the Sherwood Project: 1) steeply dipping normal faults with north-south strikes, 2) east-west trending strike-slip faults with right lateral relative offsets, 3) northeast trending shear zones, and 4) secondary northwest trending structures. Mapping in the nearby pit area yielded the best exposure of fractured bedrock; however, some fractures may have been induced from mining activities.

Three major structural orientations were observed in the pit wall. Fractures dipping from 73°-85°NW. ranged in strike from N.55°E. to N.80°E. Fractures dipping 70°S to vertical ranged in strike from N.35°E. to N.75°E. and fractures N.10°W. to N.40°W. dipped 60°SW to vertical. All fractures tended to be dipping greater than 60°. Major shear zones exhibited two primary orientations, the first trending between N.36°E. and

N.80°E., and the second ranging from N.10°W. to N.30°W. However, a single major shear zone was oriented approximately N.85°W. The northeasterly trending structures were the most abundant in the pit wall with seven shear zones identified, while only three major shear zones possessed a northwest orientation. Figure C.2.3 illustrates the locations of selected pit outcrops with field data, and the observed or inferred structures.

Two faults, labeled "A" and "B" in Figures C.2.2 and C.2.3, are inferred to bound the east orebody. These two faults, which trend northwest, are steeply-dipping normal faults and form the graben structure which hosted the east orebody.

The primary faulting which controls the main orebody, labeled "C" in Figures C.2.2 and C.2.3, appears to be the result of two, parallel steeply-dipping normal faults which trend approximately N.10°W.and dip 85° SW. These faults are visible in the north wall of the pit at mapping station 83. In addition, there appears to be a large shear zone at mapping station 83c approximately 2 feet wide which strikes N.70°E. and dips approximately 65° SE. This shear may be part of a series of parallel structures which define the northern boundary of the main orebody.

Several faults in the southern pit area, labeled "D" in Figures C.2.2 and C.2.3, were inferred from the structural relationships between the pit area and the three small knobs which are bedrock remnants left from mining the main orebody. The westernmost knob is the only knob capped by a layer of Gerome Formation gravels, indicating that a normal fault exists between the westernmost knob and the other knobs to the east. In addition, the main orebody must have been down-dropped below the level of the westernmost knob, suggesting a north side down normal fault trending roughly northwest.

Two faults were identified south of the pit area. The first fault, labeled "E" in Figure C.2.2, appears as a 150-foot high rock/cliff band along the west side of a prominent ridge of quartz monzonite, trending roughly north-south to the south of the pit. The prominent point along this outcrop ridge is often referred to as the BLAZE survey control point. This part of the fault set is a high-angle normal fault which trends approximately N.10°W. to due north and dips approximately 80° W. to vertical, with the west side down-thrown at least 200 feet. This fault may be the southern extension of the main pit fault. The second fault, labeled "F" in Figure C.2.1, is an inferred east trending fault, with an apparent right lateral offset of approximately 200 feet. This fault appears to off-set fault "E".

Based on an inspection of aerial photograph stereo pairs taken in 1974 for the Bureau of Indian Affairs (BIA), an additional east trending fault, labeled "G" in Figure C.2.2, was inferred parallel to fault "F" but located approximately 500 feet to the north. Fault "G" was also interpreted as being steeply dipping and having a right lateral offset of approximately 100 feet. Several planar structures were observed in the walls of the diversion channel that borders the tailing impoundment on the west, north, and east. These structures are typically thin joints with occasional secondary sericite or quartz precipitated in the joint planes. A few planar structures identified in the diversion channel walls to the east of the tailing impoundment exhibit abundant clayey material within the structure, for example, mapping location 9p (see Figure C.2.1 and Table C.2-1) in the diversion channel east wall. The clayey material may be gouge material from shearing or weathering products of dike material which is more easily weathered than the host rock. The orientation of this structure could not be readily determined during field mapping. The origin of these structures, whether shear zones, faults, or dikes, could not be determined from surface mapping. Similarly, the influence of these structures on the hydrogeology is uncertain; however, the abundance of fine-grained material in the structures suggests that they may not

have significant hydraulic capacity for ground water transport.

The outcrop ridge of quartz monzonite observed in the drainage below the tailing impoundment was interpreted to be bounded by a east-dipping normal fault, labeled "H" in Figure C.2.2. The eastern side of this drainage appears to be composed entirely sand with no exposed bedrock. The eastern exposure of the outcrop ridge trends approximately N.10°W. and the measured strike of outcrops along this ridge are consistent with this orientation.

Two parallel linear features trending east-west that are visible on the 1974 series BIA photographs southeast of the tailing impoundment dam were interpreted to be possible shear zones or faults. These features, labeled "I" in Figure C.2.2, form topographic lows along the north-trending ridge east of the impoundment area and may be the eastern extension of the east-west trending faults observed near the BLAZE survey control point.

Similar east trending structures, labeled "J" in Figure C.2.2, also are inferred north of the tailing impoundment facility area and parallel to faults "F", "G", and "I", to the southwest. There is no evidence of offset or displacement along these structures.

Table C.2-1 Summary of Strikes and Dip Data

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
1Pb	N20E N-S	60 E 65 W	Kqm, joint orientation. Kqm, joint orientation.
2P	N65E	70 NW	Kqm, joint orientation.
2Pb	N60W N30W N20E	50 SW 75 SW 25 E	Kqm, joint orientation. Kqm, joint orientation. Kqm, joint orientation.
3P	N40W N40W N-S N-S N70E	Vertical 45 NE 55 W 45 E 60 S	Pegmatite dike orientation. Joint orientation in pegmatite dike Kqm, Primary joint orientation. Kqm, joint orientation. Kqm, minor joint orientation.
4P	N-S N-S	45 W 40 E	Kqm, primary joint orientation. Kqm, secondary joint orientation.
5P	E-W	Vertical	Aplite dike orientation in Kqm, follows primary joint orientation.
6P	N-S N70E	60 W 20 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
7P	N10E N20E	45 W 45 E	Kqm, primary joint orientation. Kqm, secondary joint orientation.
8P	N10W N55W	25 E 55 SW	Kqm, primary joint orientation. Kqm, secondary joint orientation.
9P	NA	NA	Possible fault gouge in shear zone, no orientation visible.
10P	N5E	75 E	Kqm, primary joint orientation.
11P	N5W N25E	55 W 45 NE	Kqm, dominant joint or shear orientation. Kqm, dominant joint or shear orientation.
12P	N50W N70E	75 NE 85 S	Kqm, primary joint orientation, poor outcrop. Kqm, secondary joint orientation.

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
13P	N-S	55 W	Kqm, joint orientation, low joint density, massive.
14P	N45W N-S	70 NE 60 W	Kqm, joint orientation. Kqm, joint orientation.
1	N20E N30W	Vertical 58 S	Kqm, joint orientation. Kqm, joint orientation.
2	N8E N25E	60 W 20 SE	Kg, joint orientation. Kg, joint orientation.
3	E-W	56 N	Kg, joint orientation.
4	NA	NA	Kg, no visible strike or dips.
5	NA	NA	Kqm, highly weathered, no visible strikes or dips.
6	NA	NA	Kqm, highly weathered, no visible strikes or dips.
7	NA	NA	Kqm, highly weathered, no visible strikes or dips.
8	NA	NA	Kqm, highly broken outcrop, no dominant structural orientation.
9	NA	NA	Kqm, possible outcrop, no visible strikes or dips.
10	NA	NA	Kqm, no visible strikes or dips, abundant white bull quartz.
11	N70E Horizontal	63 S Horizontal	Kg, joint orientation, massive outcrop. Kg, joint orientation, massive outcrop.
12	N80E	NA	Andesite dike trend.
13	N80E N25W	68 S 85 NE	Kqm, primary joint orientation. Kqm, secondary joint orientation.

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
14	NA	NA	Kqm, no visible strikes or dips.
15	N77E	80 N	Kqm, joint orientation.
16	N-S N75E	54 W 82 S	Kqm, joint orientation. Kqm, joint orientation.
17	NA	NA	Kqm, no visible strikes or dips.
18	NA	NA	Kqm, no visible strikes or dips.
19	NA	NA	Kqm, no visible strikes or dips.
20	NA	NA	Kqm, no visible strikes or dips.
21	NA	NA	Kqm, no visible strikes or dips.
22	N40W N50E	73 NE Vertical	Kqm, joint orientation. Kqm, joint orientation.
23	N-S N50W N75E	35 W Vertical Vertical	Kqm, joint orientation. Kqm, joint orientation. Kqm, joint orientation.
24	N5W N70W	56 W 85 N	Kqm, joint orientation. Kqm, joint orientation.
25	N-S N55W N30W N10W N45W	83 W Vertical 70 SW 70 NE 80 NE	Strike and dip in pegmatite Dike. Strike and dip in pegmatite Dike. Kg, joint orientation. Kg, joint orientation. Kg, fracture orientation with slickensides, dip 8° SE.
25A	N45W N10W E-W	78 NE 50 SW 70 S	Kqm, joint orientation. Kqm, joint orientation. Kqm, joint orientation, silica vein along joint plane.
26	NA	NA	Kqm, no visible strikes or dips.

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
27	NA	NA	Kqm, no visible strikes or dips.
28	N60E N5E N68W	70 NW 60 W Vertical	Kg, joint orientation. Kqm, joint orientation. Kqm, joint orientation.
29	N65W	Vertical	Andesite dike orientation.
30	N15E N65W	65 W 75 NE	Kg, joint orientation. Kg, joint orientation.
31	N5E N60W	Vertical 55 NE	Kg, joint orientation. Kg, joint orientation.
32	N60W N50E	50 NE 70 NW	Kg, joint orientation. Kg, joint orientation.
39	N55E N20W N15W	73 NW 72 SW 75 W	Kqm, joints and small shears. Kqm, joints with slickensides. Kqm, face.
40	N73E N15W	75 N 76 SW	Kqm, 1 foot wide shear zone. Conjugate face, possible horizontal slickensides.
41	N30W	Vertical	Kqm, sulfide stained fracture with gouge.
42A	N70E N73E	75 SE 76 SE	Kqm, major shear zone, bearing water. (Same shear, upper bench)
42B	N60E N35E	70 SE 84 NW	Kqm, major shear zone. (Same shear, upper bench)
42C	N36E N10W N60E	76 SE 60 SW 80 SE	Kqm, major shear zone. Kqm, minor shear zone. (Same major shear, upper bench)
42D	N60E N40W N70E	77 SE 86 SW 85 SE	Kqm, major shear zone. Conjugate joint face, more moist, stained with sulfides. (Same major shear, upper bench)

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
43	N70E N70E	85 S Vertical	Kqm, Major shear zone of 2 parallel shears which converge. (Same major shear, upper bench)
44	N40E N80E N10W N55E	Vertical 77#N 60 SW NA	Kqm, major shear zone, 2 zones, for total pit wall height. Left side of shear zone. Pit face at station 44. (Same major shear, upper bench)
45	N75E N70E	Vertical Vertical	Kqm, narrow shear zone (<6 inches), moist. (Same shear, upper bench)
46	N85W	83 S	Kqm, broad zone of shear (24 inches wide). Not visible on upper bench.
47	N10W	70 SW	Kqm, very large zone of shear (20 ft. wide), slickensides dip approx. 25° to south. Not visible on upper bench.
48	N15W N65E	85 SW 85 NW	Kqm, joint orientation. Kqm, joint orientation.
49	N10W N73E N65E N8W	80 SW Vertical 65 NW 62 SW	Kqm, joint orientation. Kqm, joint orientation. Fault, slickensides dip 30° SW. Fault, slickensides dip 30° SW.
50	NA	NA	Kqm, porphyritic, as in pit, no strikes or dips.
51	NA	NA	Basalt.
52	NA	NA	Andesite dike in Kqm, no visible strikes or dips.
53	NA	NA	Kqm, no visible strikes or dips.
54	N85W	80 S	Kqm, joint orientation.
55	N-S	NA	Kqm, rough orientation of joint.

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
56	NA	NA	Kg, no visible strikes or dips.
57	NA	NA	Kg, no visible strikes or dips.
58	NA	NA	Kqm, no strikes or dips.
59	NA	NA	Kqm, no strikes or dips.
60	N17W N77E	60 SW 80 NW	Kqm, joint orientation. Kqm, joint orientation.
61	N70W N20W	75 NE 80 NE	Kqm, joint orient., gully below BLAZE, trends N20W. Kqm, joint orientation.
62	N10W N85W	40 SW 37 S	Kqm, joint orientation. Kqm, joint orientation, possible slickensides down dip.
63	N17W	Vertical	Kqm, joint orientation.
64	N25E N-S E-W	Vertical 85 W 70 S	Andestite dike attitude. Kqm, joints in quartz monzonite above dike. Kqm, joints in quartz monzonite above dike.
65	N7W N75E	65 SW 70 SE	Kqm, slickensides dip 50° S. Kqm, no slickensides.
66	N5W N53W	64 W 85 NE	Kqm, primary joint orientation. Kqm, secondary joint orientation.
67	N80W N18W N43E	75 S 85 SW 80 SE	Kqm, highly weathered. Kqm, primary joint orientation. Kqm, secondary joint orientation.
68	N10W E-W	55 SW Vertical	Kqm, primary joint orientation. Kqm, secondary joint orientation.
69	N10W E-W	70 SW 65 N	Kqm, possible in situ outcrop. (primary) Kqm, secondary joint orientation.

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
70	N72E N85W N-S	85 S 77 N 45 E	Kqm, joint orientation. Kqm, joint orientation. Kqm, joint orientation.
80	N40E N60E N35W N65E	80 SE 45 NW Vertical 80 SE	Kqm, major shear zone, 30 inches wide. Kqm, joint orientation. Kqm, joint, possible slickensides, vertical. Kqm, slickensides dip 50° NE.
81	N15W N65E	70 SE 75 SW	Kqm, joint orientation. Kqm, joint orientation.
82	N60E N70E	75 SE 65 NW	Kqm, fault with two sets of slickensides: 52°S and 37° NE. Kqm, north side of same contact, no slicks.
83A	NA	73 W	Tg, pit fault, tuff on conglomerate.
83B	NA	53 W	Kqm, pit fault, conglomerate on Kqm porphyry.
83C	N70E	65 SE	Kqm, major shear zone, upper bench.
85	N15W N-S	60 SW 75 W	Kg, joint orientation of various outcrops. Kg, joint orientation of various outcrops.
86	NA	NA	Kqm, no strikes or dips.
87	N4E E-W	60 W Vertical	Kqm, joint orientation. Kqm, joint orientation.
88	N5E E-W	85 E Vertical	Kqm, joint orientation, varies ± 10° on strike. Kqm, joint orientation.
89	N20E E-W	85 E Vertical	Kqm, joint orientation. Kqm, joint orientation.
90	N20E	80 E	Kqm, joint orientation.
91	N10W	80 SW	Kqm, joint orientation.

Mapping Station	Strike	Dip (degrees)	Remarks
1P	N-S N80E	55 W 80 S	Kqm, primary joint orientation. Kqm, secondary joint orientation.
92	N10W N75E N40W	40 SW Vertical Vertical	Kqm, possible outcrop, joint orientation. Kqm, possible outcrop, joint orientation. Kqm, possible outcrop, joint orientation.
93	N70E	Vertical	Kqm, possible outcrop, joint orientation.
94	N40W N45E N35W N80E N45W N65E N-S N65W	65 NE 25 SE 65 NE 45 N 75 SW 55 SE 70 E 70 NW	Kqm, group of outcrops, outcrop A, joint. Kqm, group of outcrops, outcrop A, joint. Kqm, group of outcrops, outcrop B, joint. Kqm, group of outcrops, outcrop C, joint. Kqm, group of outcrops, outcrop C, joint. Kqm, group of outcrops, outcrop F, joint. Kqm, group of outcrops, outcrop F, joint. Kqm, group of outcrops, outcrop F, joint.
95	N80E N10W	Vertical Vertical	Kqm, primary joint orientation. Kqm, secondary joint orientation.
96	NA	NA	Kqm, no strikes or dips.
97	N15W	55 SW	Kqm, primary joint orientation.

Notes:
Kqm = Cretaceous quartz monzonite.
Kgd = Cretaceous granite or "granodiorite".
Tg = Tertiary Gerome Formation
NA = Not Available

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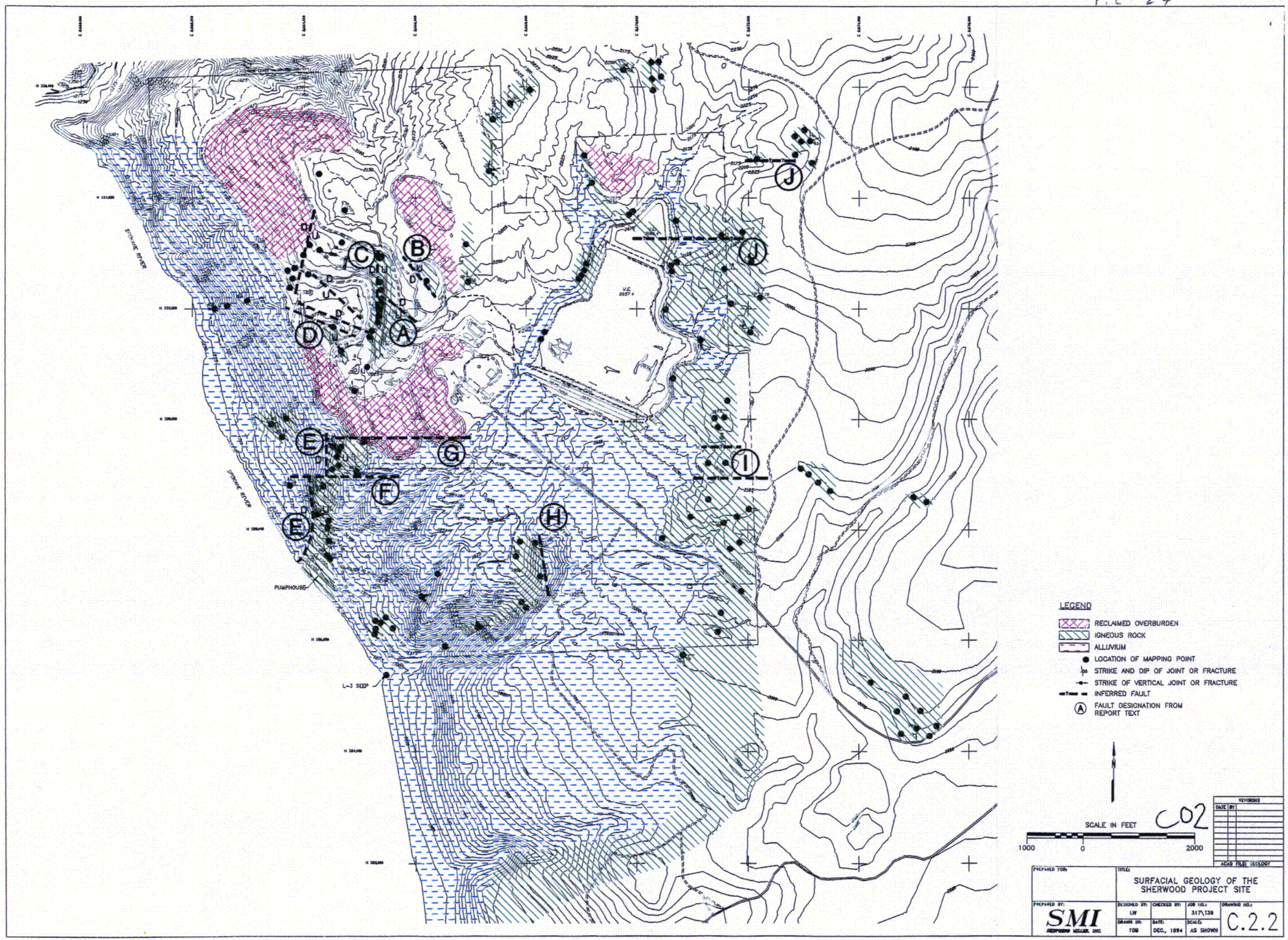
STRIKE RIVER
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PUMPHOUSE
L-3 SEEP
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IGNEOUS ROCK
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TITLE:
SURFACIAL GEOLOGY OF THE SHERWOOD PROJECT SITE
PREPARED BY:
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SHERWOOD MILLER, INC.
DESIGNED BY: LW
CHECKED BY:
JOB NO.: 317\138
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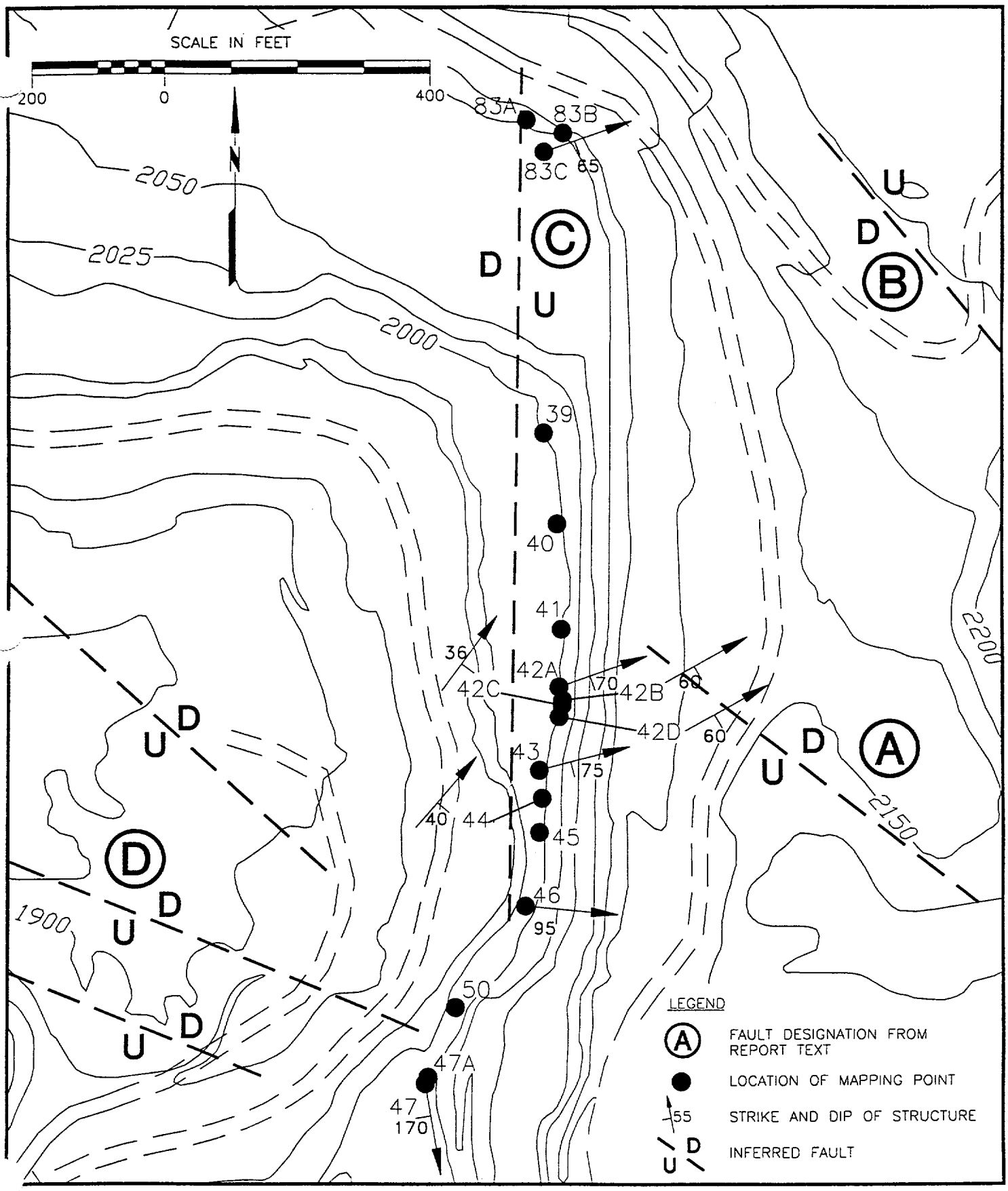

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FIGURE C.2.3
STRUCTURAL GEOLOGY OF
THE PIT AREA

Date:	DEC., 1994
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ATTACHMENT C.3
BOREHOLE GEOPHYSICAL STUDY

BOREHOLE GEOPHYSICAL STUDY

A borehole geophysical study was conducted in October 1993 on the older point of compliance (POC) wells MW-4, MW-5, and MW-6 and upgradient well MW-1A to gather information about the hydrogeologic relationships between the older wells (MW-1a, MW-4, MW-5, and MW-6) and the newer POC wells (MW-8, MW-9, and MW-10) . The objectives of this study were to: (1) observe ground water flow and fracture character in the open portion of the wells, (2) confirm the depths of alluvium, weathered bedrock and competent bedrock, and (3) confirm well construction details for the older wells. All appropriate QA/QC procedures were followed for decontamination of down hole equipment.

Down Hole Camera

To evaluate the mode of well recharge and observe in situ groundwater flow, a 3-inch diameter color video camera was lowered down wells MW-4 and MW-5 while recording the depth from top of casing. The observed parameters included: (1) the depth and length of slotted steel casing, (2) the total depth of casing, (3) the total depth of hole, and 4) the nature of bedrock fractures in the open portions of the well.

Well MW-4 was observed to have steel casing extending to a depth of 200.2 feet below top of steel casing (approximately 1789.4 feet above msl). Narrow vertical slots, apparently cut in the casing with a torch, extend from approximately 186.75 feet below the top of steel casing (1802.9 feet above msl) to the bottom of the casing. The bottom of the well bore was determined to be approximately 219.1 feet below the top of steel casing (1770.4 feet above msl). Static water elevation in the boring was constant throughout the year at approximately 215.6 feet below the top of steel casing (1773.9 feet above msl).

Discrete fractures were observed with the video camera in the open portion of the wellbore. Fracture orientation varied between individual fractures; however, no absolute orientation could be determined from the camera logs. Fractures varied from steeply-dipping (greater than 70 degrees from horizontal) to very flat (less than 10 degrees from horizontal) with no discernable preferential orientation. Apparent fracture dilation also varied from relatively large (visually estimated at approximately 0.5 centimeter) to very fine (a few tenths millimeters). Flow into Well MW-4 was not observed due to the small saturated thickness observed in the well.

Well MW-5 was observed to have steel casing extending to a depth of 160.05 feet below the top of steel casing (1828.5 feet above msl) with narrow vertical slots, apparently cut in the casing with a torch, extending from approximately 150.0 feet below the top of steel casing (1839.1 ft above msl) to the bottom of the casing. The bottom of the well bore was not determined due to the presence of an obstruction at 172.3 feet below the top of steel casing (1816.8 feet above msl). Fracture patterns, orientations, and dilations observed in MW-5 were similar to those observed in MW-4.

To observe the nature of groundwater flow in the wellbore, Well MW-5 was chosen for visual observation due to its relatively slower recharge rate. The well was bailed with a Teflon bailer to a final water level of approximately 170 feet below the top of casing (1819.1 feet above msl), leaving approximately 2.3 feet of water in the well. The camera was then lowered into the hole and well recharge was observed.

No well recharge was observed from the slotted portion of the well. Only two discrete locations in the wellbore were observed to contribute to well recharge. The uppermost point of observed well recharge was located at approximately 163.5 feet below the top of steel casing (1825.6 feet above msl). This inflow of ground water appeared to be controlled by a minor fracture without major dilation and of

indeterminate orientation. The flow was very small (visually estimated at less than 0.5 gpm) and originated from a single point.

The second location of observed groundwater recharge to the well was located approximately 165.5 feet below the top of steel casing (1823.6 feet above msl) and was also controlled by a minor fracture. The orientation of this fracture could not be determined from the video log because no frame of reference (e.g., north, south) could be determined. This flow was estimated to be slightly greater than the upper fracture but less than 0.5 gpm. Recharge to the well was not observed from the slotted portion of the casing or from the larger fractures higher in the well bore.

The static water level was approximately 0.5 foot below the bottom of steel casing (1,830 feet above msl). This indicates that recharge to the well is from fracture flow in the bed rock and not directly from the weathered bedrock or alluvial zones. In addition, recharge appeared to be controlled by the smaller fractures observed in the well bore.

Natural gamma logs

The geologic logs and well construction details of the older POC wells, installed in 1978 by CTL Thompson, Inc., are not very detailed. Subsurface geologic conditions were investigated in Wells MW-1a, MW-4, MW-5, and MW-6 with a Mount Sopris MGX 200 data logger and natural gamma radiation probe. The natural gamma probe was chosen due to its ability to collect data despite the ⅜-inch-thick steel casing in most of the wells. The data logger and probe record proportional measurements in counts per seconds (cps) of the naturally occurring radio- isotope decay series of potassium-40 (K^{40}), uranium-238 (U^{238}), and thorium 232 (Th^{232}). Though the wells are cased with 5.6-inch outer-diameter steel casing over most of the wellbore length,

the alluvium, weathered bedrock, and competent bedrock should have discernably different gamma signatures. The strip chart records of the natural gamma tests are included as Figures C.3.1 through C.3.4. The probe utilizes a cylindrical sodium iodide crystal that is ⅞- inch in diameter and 3 inches long. The probe was lowered from the surface at approximately 12 feet per minute (fpm). Correction factors for steel casing and water in the well bore were supplied by COLOG Inc. and are based on calibrated field tests in Colorado.

Inspection of the natural gamma logs for Wells MW-1a, MW-4, MW-5, and MW-6 indicated that the alluvium has an observed signature of approximately 70 to 100 cps. Using a correction factor of 1.9 for ⅜-inch thick, 5-inch diameter steel casing yields actual counts of approximately 130 to 190 cps.

Observed gamma counts for the weathered bedrock ranged from 90 to 150 cps, indicating corrected counts of approximately 170 to 285 cps. The weathered zone, as interpreted from the gamma logs, ranged in thickness from 5 feet thick in Well MW-5 to 15 feet thick in Well MW-6, indicating a variable degree of bedrock weathering within a relatively small area. Well MW-1a appears to lie entirely within the weathered zone.

The competent bedrock material had a dramatically different gamma signature than the alluvium and weatherd bedrock. Competent bedrock exhibited peak gamma counts of approximately 380 to 500 cps. These counts were observed in open well bores below the steel casing; therefore, no correction factor was used. However, a correction factor of 1.1 was used to correct for water in the well bore. Corrected gamma counts for the competent bedrock ranged from approximately 420 to 550 cps.

The weathered bedrock appeared to have a much less distinct gamma signature than

expected. This may be due to the leaching of gamma emitting elements from the weathered zone with the degradation of the crystalline structure. This oxidation and leaching of gamma emitting minerals is consistent with the elevated uranium values observed in some monitoring wells completed in the competent and weathered bedrock zones. The designation between weathered and competent bedrock as observed during field drilling is based on the mechanical aspects of the bedrock and drilling penetration and may not correspond directly to the gamma log signatures.

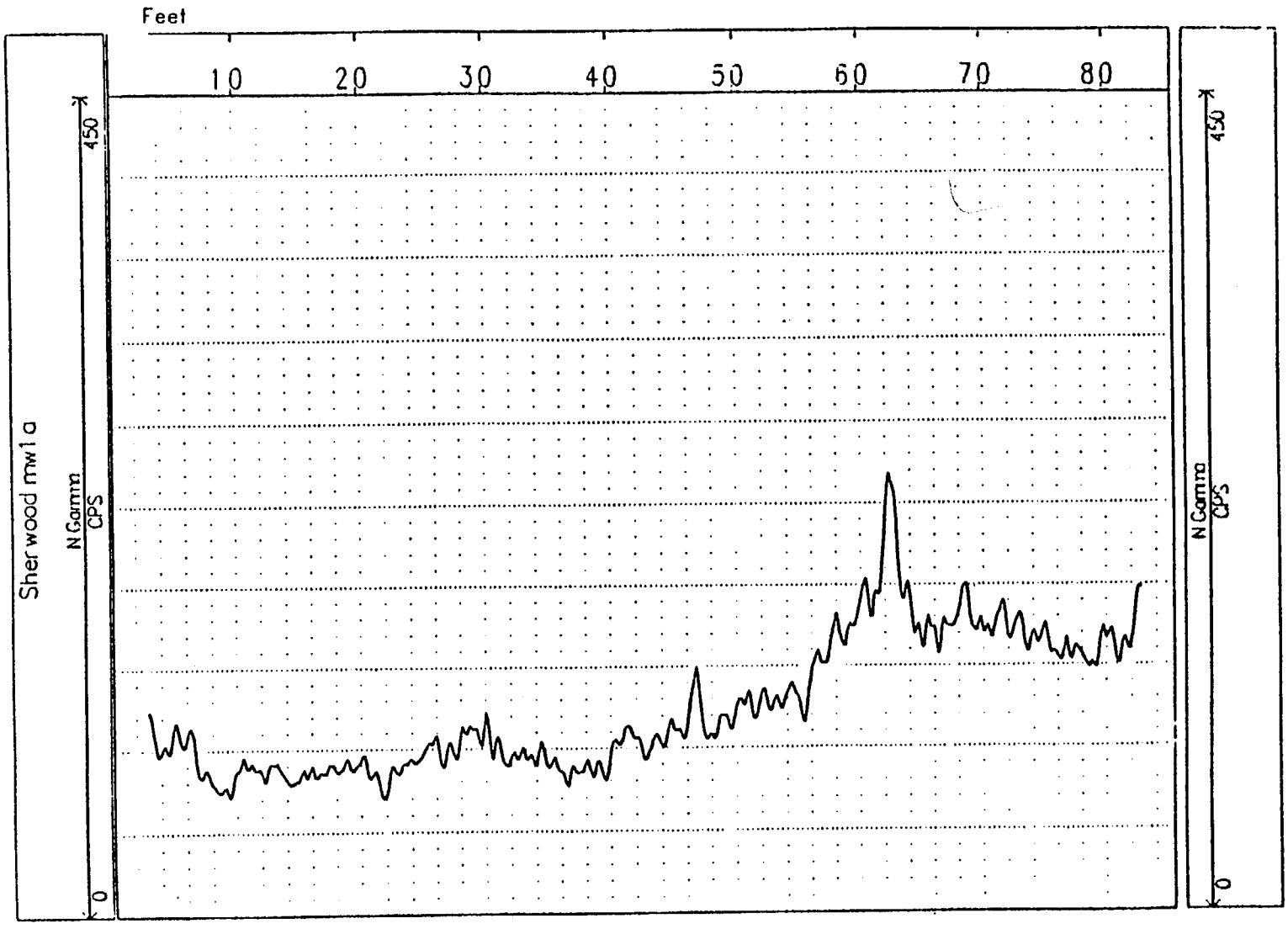

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FIGURE C.3.1
NATURAL GAMMA LOG
FOR WELL MW-1

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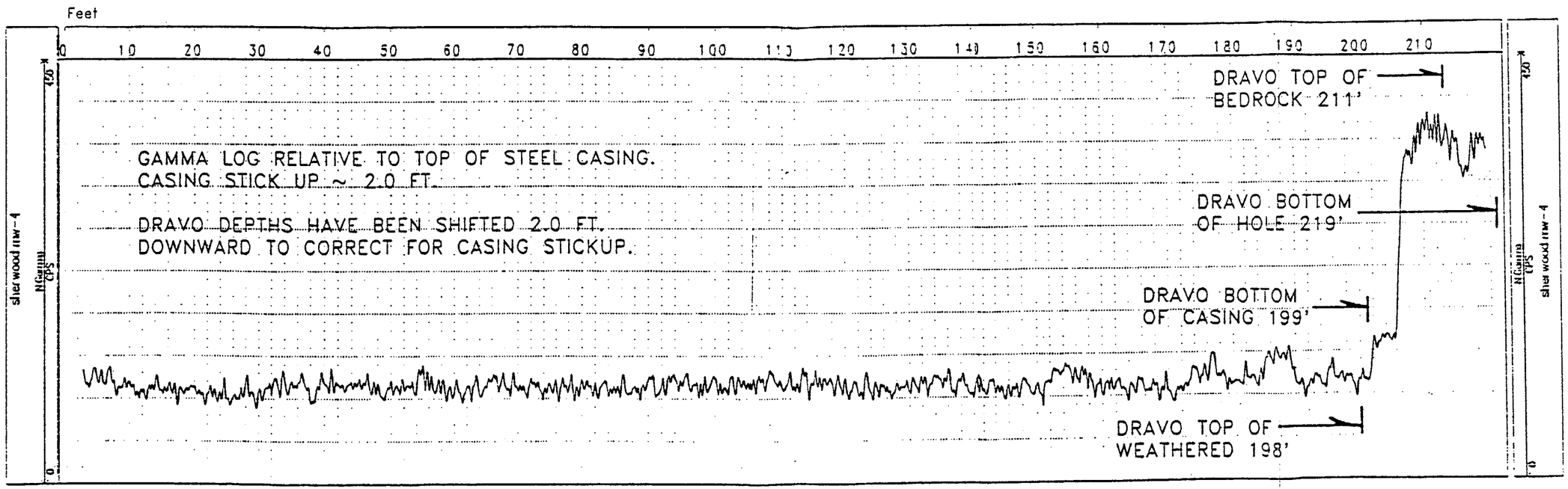

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FIGURE C.3.2
NATURAL GAMMA LOG
FOR WELL MW-4

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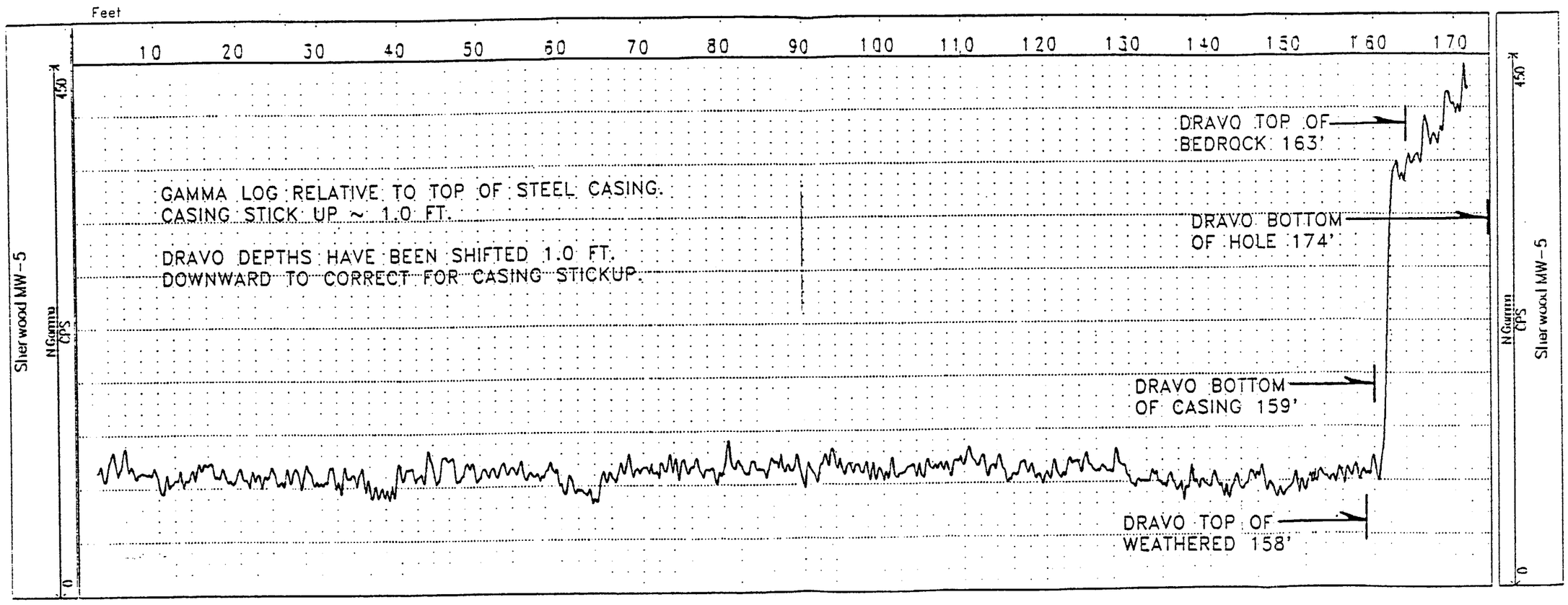

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Project: 317 TASK31
File: 11X17

FIGURE C.3.3
NATURAL GAMMA LOG
FOR WELL MW-5

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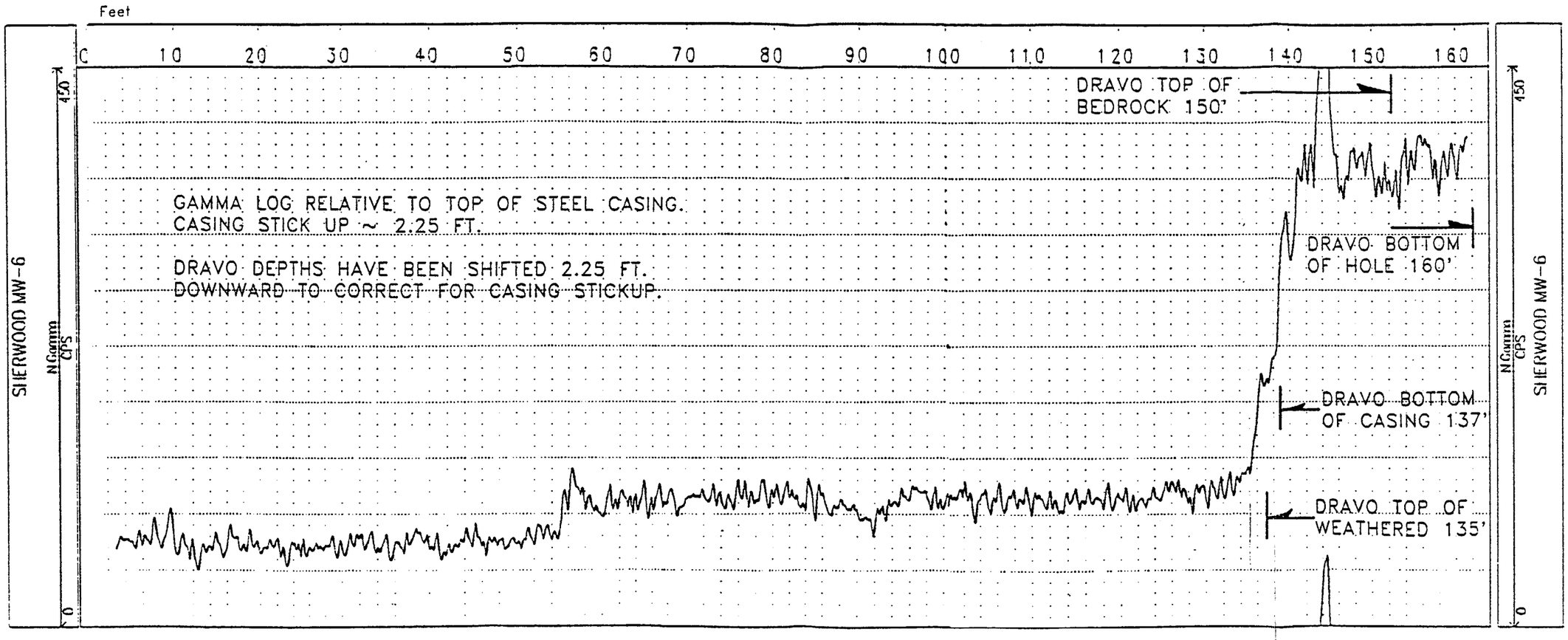

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Project: 317 TASK51
File: 11X17

FIGURE C.3.4
NATURAL GAMMA LOG
FOR WELL MW-6

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ATTACHMENT C.4
SEISMIC STUDY

SEISMIC STUDY

A seismic study was undertaken in order to more completely characterize the geologic and hydrologic controls of the ground water system next to the Sherwood Project tailing impoundment. The objectives of this survey were: (1) to identify the location of the bedrock surface and (2) to determine if potential bedrock structures, which may control bedrock surface geometry and groundwater flow, can be detected. This survey was performed in two phases. The first phase, performed in October and November 1992, consisted of two seismic reflection lines (lines A and B) set at the toe of the tailing impoundment dam parallel to the dam axis. The second phase, performed in September and October 1993, consisted of 5 refraction survey lines (lines C, D, E, H, and I) and three reflection survey lines (lines F, G, and J). All seismic line locations are illustrated in Figure C.4.1. Cooksley Geophysics, Inc. (CGI) of Redding, California performed the field data acquisition. CGI and J.R. Rezowalli Associates (JRA), of San Jose California, were jointly responsible for seismic data processing and interpretation.

Phase I

Phase I of the seismic study was performed to assist in locating additional Wells MW-8, MW-9, and MW-10. Geologic data developed from drilling logs were used to confirm seismic data interpretation.

Phase II

Phase II of the seismic study was performed to (1) locate the depth to bedrock surface adjacent to the tailing impoundment and (2) identify any potential geologic structures which may influence groundwater flow.

Seismic Study Results

The original seismic report produced by CGI/JRA is included as Attachment C.5. Data from five refraction lines (Lines C, D, E, H, and I) were collected over a total length of 17,280 feet. In addition, data were collected from five reflection lines over a total of 9,570 feet.

Refraction survey analysis identified three material types:

1) The uppermost layer which occurred between the surface and 25 feet in depth. This layer is represented by loose alluvial materials and characterized by seismic velocities in the range of 900 feet per second (fps) to 3,300 fps.

2) The middle layer which is between five and 85 feet thick. This layer is represented by alluvial materials or highly altered bedrock materials and characterized by velocities in the range of 1,700 fps to 8,000 fps. Bedrock material may be altered by weathering or structural deformation (ie: shearing, faulting) which would tend to produce lower material densities and lower seismic velocity characteristics.

3) The lower layer which is represented by competent to slightly altered bedrock. This layer is considered to be the top of bedrock, occurs from 11 feet to greater than 110 feet below the ground surface, and is characterized by seismic velocities ranging between 7,500 fps to 20,000 fps.

The middle layer is not observed at the south end of line D or at the northwest end of line C. Bedrock is known to occur at very shallow depths in these locations from site geologic mapping.

The top of the third layer was considered top of bedrock. Results from refraction lines C, D, E, and H confirm that the tailing impoundment drainage basin is surrounded to the west, north, and east by structurally high bedrock. There are no bedrock surface drainage flow paths which would allow ground water to leave the drainage basin to the west, north or east. These data support the existing ground water flow model in which ground water flows to the south through a buried bedrock valley.

Zones of low seismic velocity (less than 10,000 fps in the lower layer) were observed on refraction lines D,E,H, and I. These low velocity zones (LVZ) represent regions of altered bedrock material where seismic waves move more slowly in response to lower bedrock material densities. The alteration of the bedrock may be due to locally intense weathering or may be due to structural deformation such as fracturing and jointing. The nature of the alteration cannot be determined from these data.

No correlation of LVZ's between refraction lines could be made based on these data. No LVZ's were associated with significant bedrock features or changes in bedrock slope with the exception of the LVZ observed on line E. This LVZ coincides with a slight depression in the bedrock surface approximately 300 feet wide. However, no similar association of a bedrock surface depression and LVZ was identified on any adjacent seismic line. Therefore, the LVZ associated with the bedrock surface depression observed on line E is not interpreted to be the result of structural deformation.

Geologic conditions at the study site were not conducive to the collection of seismic

reflection data. Reflections from subsurface units were often absent or weak. Numerical models of three layered systems were performed to better understand why reflections were absent in many profile sections and to aid in interpretation of first arrival seismic wave data. It was determined from these models that interference from ground roll and wide angle refractions frequently masked reflection patterns from the bedrock and deep alluvium. Numerical model analysis indicated that first arrivals from bedrock surface reflectors may be seen in the seismic data when bedrock is less than 150 below ground surface. In addition, reflectors and in the alluvial materials should provide reflections from depths between 150 feet below ground surface and the bedrock surface.

Data from line A presented the best reflection profile and confirmed the bedrock surface below the impoundment dam as a incised valley with Monitoring Wells MW-4, MW-5, and MW-6 located on the western slope of the valley. Data from line B presented few strong reflecting surfaces. However, borehole data from Well MW-8 confirms that the deeper, weak reflectors coincide with the bedrock surface. Data from line B was used to locate Monitoring Wells MW-8, MW-9, and MW-10 in the lowest point of the bedrock drainage surface. Data from lines A and B indicate that the bottom of the bedrock valley has a northwest orientation at this location.

No reflection surfaces could be developed from lines F and G due to interference from ground roll and wide angle refractions. The first arrival refraction data was used to estimate depth to the bedrock surface from line G. The bedrock surface along line G ranges between 25 and 100 feet below the ground surface. The bedrock surface could not be identified at any location along line F from either first arrival refraction data or reflection data. The lack of first arrival data for line F indicates that bedrock is present at depths greater than 150 feet except at the ends of the line where geologic mapping has shown bedrock to exist at or near the surface.

No bedrock structures, such as faults or shear zones, were identified from the seismic reflection or refraction survey data. Low velocity zones observed in some of the refraction survey lines could not be correlated as individual structures with persistent trends. In addition, LVZ locations relative to bedrock surface morphology does not suggest an association with structural features in the bedrock.

Bedrock elevation data from seismic reflection and refraction lines and bedrock elevation data from boring logs and geologic mapping were digitized into AutoCAD to develop a computer contour map of the bedrock surface. Three-dimensional grids were generated from the interpreted contour data. Figure C.4.2 illustrates the interpreted bedrock surface as a contour map. Figure C.4.3 illustrates a grid of the existing topographic surface superimposed on a grid of the interpreted bedrock surface. These data confirm previous interpretations of bedrock surface morphology (US Dept. of the Interior, 1975; D'Appolonia, 1977). Bedrock surface drainage next to the tailing impoundment roughly parallels the surface drainage, with bedrock surface gradients sloping to the south. No bedrock structures were identified which could potentially influence ground water flow in this basin.

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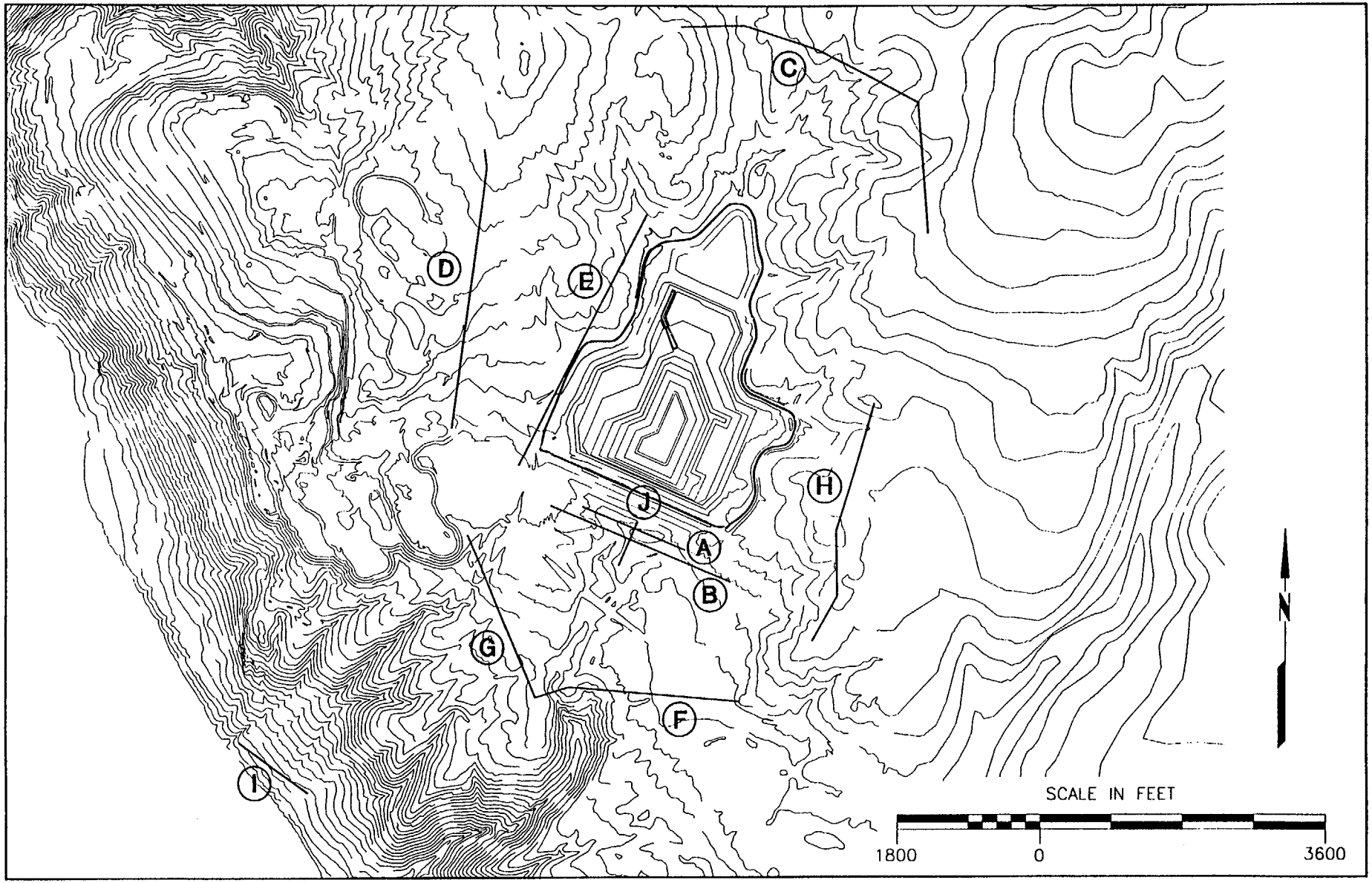

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FIGURE C.4.1
SEISMIC LINE LOCATIONS

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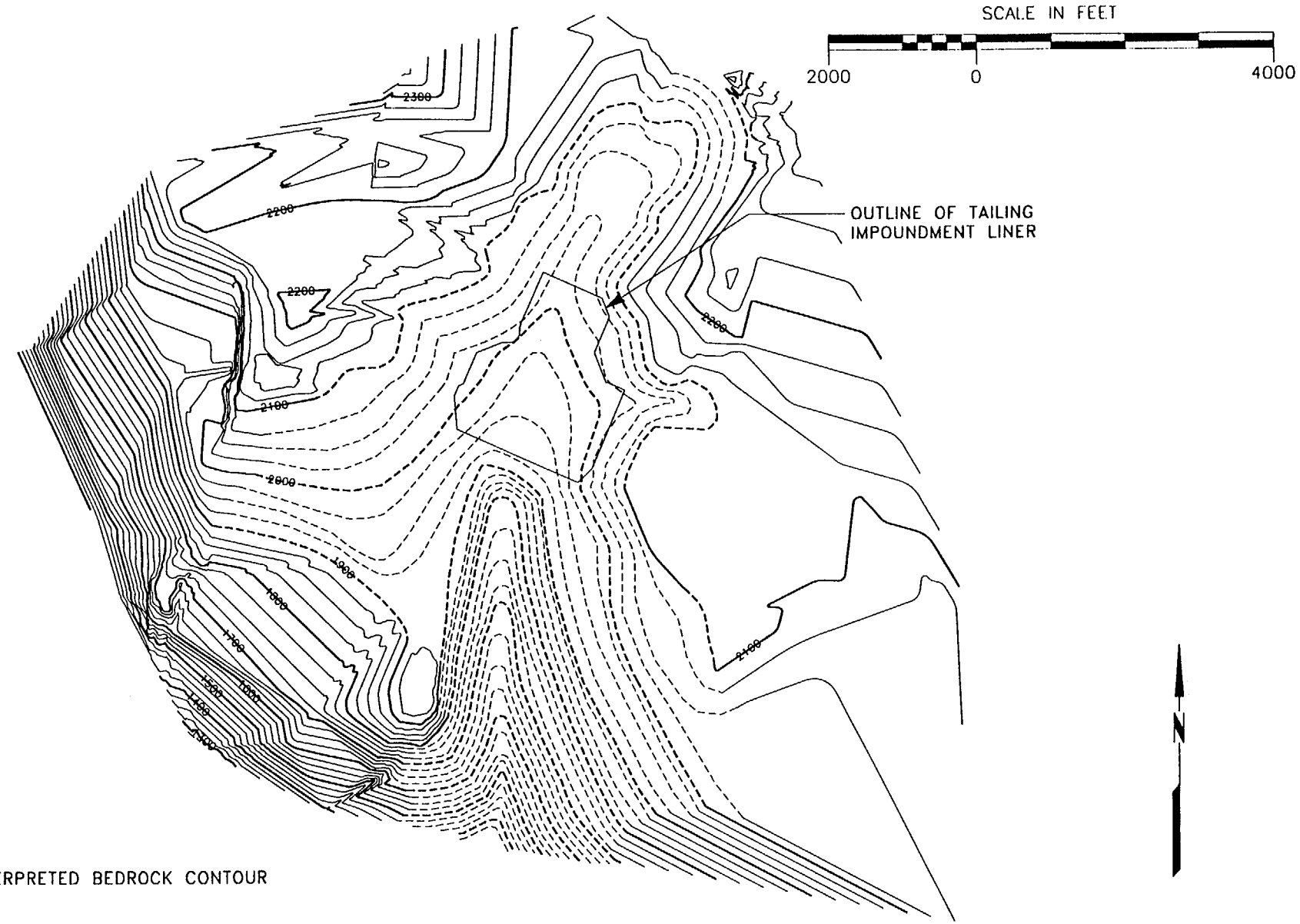

LEGEND:

------ INTERPRETED BEDROCK CONTOUR

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FIGURE C.4.2
INTERPRETED BEDROCK
SURFACE CONTOUR

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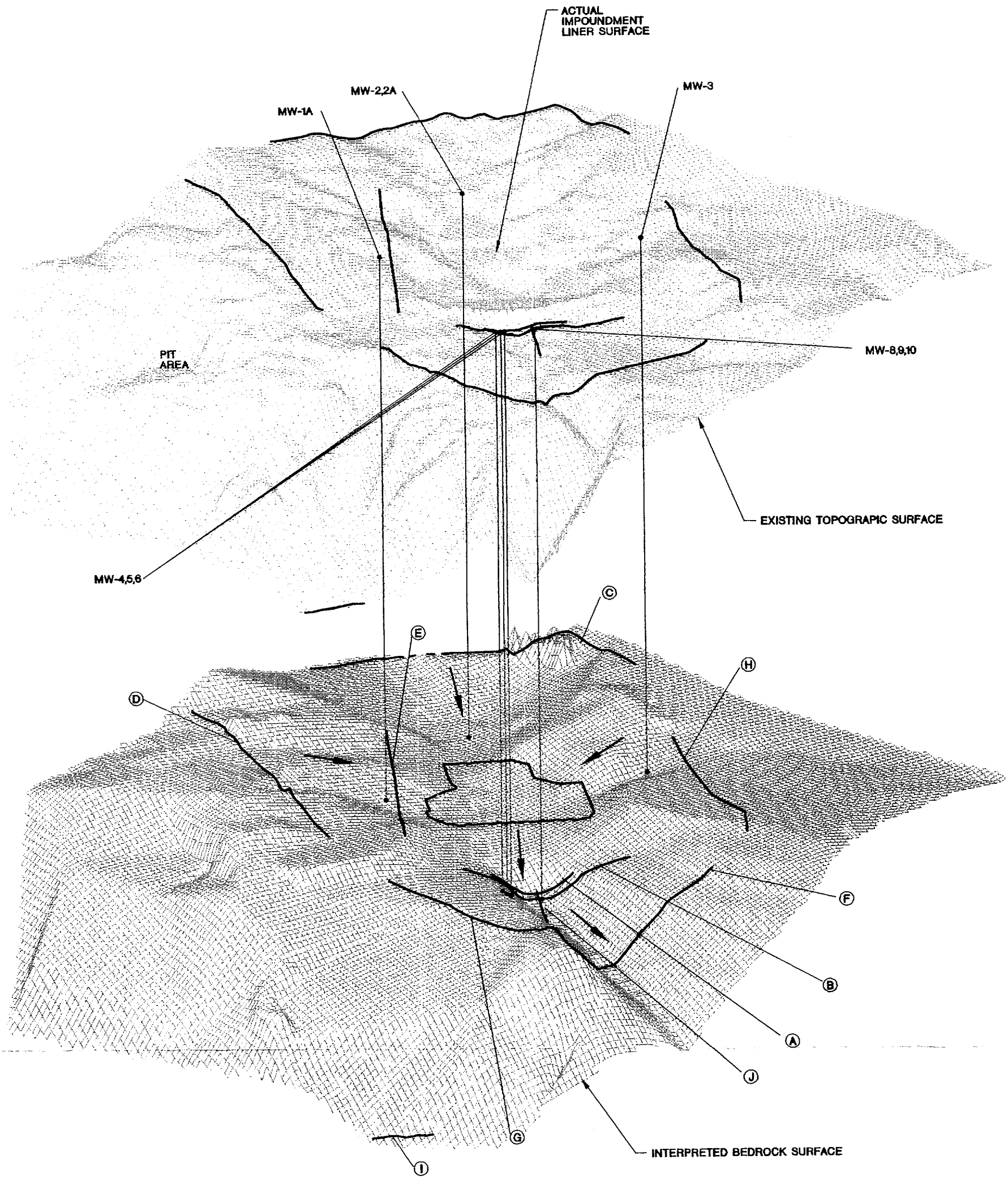

Ⓐ SEISMIC LINE LOCATIONS

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DESIGNERS ENGINEERING RECORD	BY	DATE

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SMI
SHEPHERD MILLER, INC.

PREPARED FOR
WNI
WESTERN NUCLEAR, INC.

TITLE
EXISTING SURFACE AND INTERPRETED BEDROCK SURFACE

PROJECT 317\T31\SEIS\	DATE DEC., 1994	C.4.3
SCALE 1" = 1300'	DWG FILE GRID	

SHERWOOD PROJECT
TAILING RECLAMATION PLAN

Volume 5 of 7
APPENDICES L, M, N, O

Prepared for

Western Nuclear, Inc.
Sherwood Project
Wellpinit, Washington

Prepared by

Shepherd Miller Inc.
1600 Specht Point Drive, Suite F
Fort Collins, CO 80525

December 1994

SMI
Shepherd Miller, Inc.

ATTACHMENT C
RLVA SEISMICITY REPORT

L.C-2

RLV-A

EARTHQUAKE-INDUCED SETTLEMENT SHERWOOD TAILING IMPOUNDMENT

Stevens County, Washington

Prepared for:

Shepherd Miller, Inc.
1600 Specht Point Drive
Fort Collins, Colorado 80525

Prepared by:

R. L. Volpe & Associates, Inc.
110 Atwood Court
Los Gatos, California 95032

December 13, 1994

L.C-3

RICHARD L. VOLPE
Geotechnical Consultant

R. L. Volpe & Associates, Inc.
110 Atwood Court
Los Gatos, CA 95032-4801
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December 13, 1994

Mr. Lou Miller, P.E.
Shepherd Miller, Inc.
1600 Specht Point Drive, Suite F
Fort Collins, CO 80525

Subject: Earthquake-Induced Settlement Investigation
Sherwood Tailing Impoundment

Dear Mr. Miller:

We are transmitting herewith our report entitled "Earthquake-Induced Settlement Sherwood Tailing Impoundment".

The report presents the results of our engineering assessment of the seismotectonic setting of the project site, a detailed probabilistic analysis to determine the most likely earthquake-induced ground accelerations to impact the site during a design period of 1,000 years, a study of the engineering properties affecting the liquefaction potential of the tailing materials, a detailed liquefaction analysis of the tailing materials when subjected to a conservative range of design peak horizontal ground accelerations, and a principal set of conclusions derived from the project study. Backup and supporting data associated with the engineering analysis are presented in the appendices.

We appreciate the opportunity to provide our services in this phase of the project. If any questions arise during the review of this report, please do not hesitate to contact us.

REGISTERED PROFESSIONAL ENGINEER
RICHARD L. VOLPE
No. 866
Exp. 9-30-97
GEOTECHNICAL
STATE OF CALIFORNIA

Very truly yours,

R. L. VOLPE & ASSOCIATES, Inc.

Richard L. Volpe

Richard L. Volpe, P.E.
GE No. 866, California

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EARTHQUAKE-INDUCED SETTLEMENT
SHERWOOD TAILING IMPOUNDMENT

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EARTHQUAKE-INDUCED SETTLEMENT SHERWOOD TAILING IMPOUNDMENT

Stevens County, Washington

I. INTRODUCTION

This report has been prepared by R. L. Volpe & Associates, Inc. (RLVA) of Los Gatos, California, for Shepherd Miller, Inc. (SMI) of Fort Collins, Colorado. It presents the results of a special earthquake-induced settlement assessment of the tailing impoundment located at the Sherwood uranium tailing facility in eastern Washington. SMI is providing consulting engineering services related to the Sherwood Tailing Reclamation Plan which is currently being prepared for submittal to the Washington Department of Health. The completion of the reclamation plan includes the design of a protective earthen cover over the tailing pond, and other activities related to tailing reclamation. This report first presents a summary of the seismotectonic setting of the project site, and then focuses on presenting a discussion of the methodology and detailed engineering assessment performed as a part of this study to evaluate the potential for earthquake-induced differential settlement of the tailing material and its impact on the protective cover.

The Sherwood mill complex is located in Stevens County, Washington, on the Spokane Indian reservation, about 6 miles southwest of Wellpinit. The site lies immediately east of FDR Lake on the Spokane River (see Fig. 1). The facility was opened in 1977. Ore was processed in the mill using conventional acid leach and solvent extraction technology to produce uranium oxide. Tailing leaving the mill was slurried and flowed by gravity to the adjacent Sherwood tailing pond where it was neutralized with lime prior to deposition. The Sherwood tailing impoundment was constructed in 1977 and subsequently enlarged in stages until 1982 when the mill operations ceased. During its eight years of operation (1977 to 1984), the Sherwood impoundment received an estimated total of 3 million cubic yards of tailing. Based on a review of original and current topography, and the results of a recently (1993) completed field investigation, the maximum thickness of the tailing is about 70 feet. A typical cross section through the tailing impoundment and surrounding dikes is presented in Fig. 2.

The containment dikes which support the tailing impoundment were compacted in place using site soils and a synthetic liner was used to cover the impoundment area prior to initiation of tailing deposition. A more complete description of the site facilities and local geology, along with a presentation of detailed results of a field and laboratory investigation, locations of exploratory borings, and other engineering analyses, can be found in a report detailing the Sherwood Tailing Reclamation Plan currently being prepared by SMI. Much of the previously collected data have been submitted to the Washington Department of Health in the form of appendices. The current study, which evaluates the potential for earthquake-induced settlement, has used field and laboratory data developed for the site soils and supplied to RLVA by SMI.

II. SEISMICITY

A. Introduction

The seismicity of eastern Washington surrounding the Sherwood Tailing Dam site is relatively quiescent. This fact is confirmed by the results of a recent in-depth seismotectonic study of the eastern Washington region performed for seven dams scattered throughout this area that are owned and operated by the U.S. Bureau of Reclamation. The detailed results of this study are published in a report entitled "Seismotectonic Evaluation - Walla Walla Section of the Columbia Plateau Geomorphic Province". This report was prepared by Geomatrix Consultants, Inc. of San Francisco, California, in April, 1990, and provided the majority of seismicity information presented in this report. The results of the Geomatrix study are summarized in Appendix A of this report and includes a discussion and map of the seismotectonic provinces surrounding the site, a summary of the geologic history of eastern Washington, a discussion of the current tectonic setting surrounding the site, and a discussion of the historical seismicity and significant earthquakes of magnitude 5.0 or greater that have occurred within the surrounding geomorphic provinces.

In order to assess whether future earthquake shaking had the potential for impacting the tailing pond cover, it was necessary to develop a conservative set of design earthquake assumptions. Before describing the methodology used in developing these assumptions, it is necessary to put in perspective the hazard associated with Sherwood project. When dealing with potentially catastrophic failure of any structure, such as that associated with a major dam, whose failure could pose a great hazard to those living downstream of the dam within the path of potential destruction, we normally use a deterministic approach in evaluating the earthquake hazard potential. For earthquake engineering purposes, this requires that we evaluate the impact of the Maximum Credible Earthquake (MCE) for those active faults that could impact the site. The MCE is defined as the largest possible earthquake magnitude that could occur for a specific fault given the current seismotectonic setting. Generally, the MCE is either equal to, or larger than, the largest historical earthquake to have occurred on a given fault. The MCE event is analogous to the Probable Maximum Precipitation (PMP), and corresponding Probable Maximum Flood (PMF), two hydrologic parameters that are used when designing for estimated precipitation runoff or flood potential. Like the MCE, the PMP and PMF values <u>are not</u> directly related to a historical record; rather, they represent the largest rainfall or flood event that could occur at a given project site under a given set of meteorological assumptions. If the structure, or particular component of design, is not susceptible to catastrophic failure, or does not otherwise pose a major threat to the surrounding community, then we typically use probabilistic considerations for the design. In adopting a probabilistic design approach, it is necessary for the owner, or the design engineer, to adopt a time frame by which to assess the probability of occurrence for the event in question (e.g., a 100-yr flood for design of drainage facilities). The time frame may be associated with the economic life of the structure, or it may be as set forth by codified design requirements, or as established by the

applicable governing agency. For the case of the earthquake design relationships associated with the Sherwood Uranium Tailing Impoundment cover, both deterministic and probabilistic concepts were used to assess the potential for earthquake-induced settlement of the tailing pond cover. Deterministic concepts were used to assess the earthquake hazard for earthquakes emanating from known potential earthquake sources. Probabilistic concepts were used to assess the potential earthquake hazard associated with random earthquake occurrence. For the probabilistic analyses, a "design life" for the facility of 1000 years was used to assess the hazard associated with random earthquakes. In other words, for any earthquake magnitude considered in the design, the probability calculations have assumed that such an event will occur at a frequency of only once per 1000 years. The justification for this 1000-year design period was that this was the time period ultimately adopted by the Nuclear Regulatory Commission for considering the long-term design requirements of uranium tailing facilities (Nelson, et al., 1982).

B. Earthquake Potential of Eastern Washington

As mentioned previously, two separate approaches were used to assess/characterize the earthquake potential at the project site. In the first approach, the seismic impact of the potentially active geologic structures known to exist within a 250-km radius, and thought capable of generating significant strong ground shaking, were evaluated. In the second approach, the likelihood of a random earthquake occurring within the vicinity of project was assessed based on an analysis of the historical seismicity data and using probabilistic theory. The basic input parameters and methodology of these two approaches are discussed in the following sections.

1. Seismic Potential of Active and Potentially Active Structures

Based on the available data and on the regional geological and seismological studies described in more detail in Appendix A, eight geologic features have been identified to exist within a 250-km (150± miles) radius from the site in the surrounding seismotectonic regions that are interpreted to be active or potentially active earthquake sources. These geologic features include: (1) the Status Peak segment of the Toppenish Ridge fault, which exhibits geomorphic characteristics of repeated late Quaternary displacement along a southward-dipping reverse fault: (2) the Ahtanum Creek fault, which has some structural and geomorphic characteristics similar to those along the active segment of the Toppenish Ridge fault; (3) the Central fault on Gable Mountain, which displaces late Quaternary glaciofluvial sediments, (4) a potentially active inferred fault along the Gable Butte-Gable Mountain segment of the Umtanum Ridge-Gable Mountain structural trend, which might be related to the Central fault on Gable Mountain; (5) the Smyrna Bench segment of the Saddle Mountains fault, which displays evidence for Quaternary displacement that is much less definitive than the evidence on the Smryna Bench segment; and (7 and 8) the Powder Ranch and West Canal segments of the Frenchman Hills fault, which exhibit geomorphic evidence of Quaternary fault displacement. Table 1 gives the closest distance between the project site and the various seismic sources that have been identified. The seismic potential

of each feature is evaluated with respect to the maximum credible earthquake that can reasonably be assigned to the feature. Also, it will be noted that the eight geologic features mentioned above are shown on Fig. A-1 (Appendix A) which shows the boundaries for the seismotectonic regions surrounding the site. As shown on this figure, the Sherwood Project site is located at the boundary of Columbia Plateau and Northwest Rocky Mountains Okanogan Uplands. The eight features referenced above are all located within the Yakima Fold Belt Subprovince of the Columbia Plateau, which is located southwest of the site. All other faults and other geologic features shown on Fig. A-1 are judged to be not active.

Based on the MCE magnitudes and locations of the potentially active seismic sources referenced above, peak ground accelerations expected to occur at the site were determined using attenuation relationships developed by Sadigh (1989). These data are presented on Table 2. It should be noted that none of these potential earthquakes generates substantial ground shaking much above 0.02 g at the site.

2. Random Earthquake Sources

By definition, a random earthquake does not occur on a known active fault or potential seismic feature, but rather occurs randomly throughout the seismotectonic region. Since very few active or potentially active geologic features have been identified within the seismotectonic region of the project area, the probability of occurrence of a random earthquake most likely will govern the earthquake hazard assessment of the site. In order to assess the potential hazard associated with a random earthquake, it is necessary to assess both the magnitude and distance of possible random events, given a 1000 year design window. These concepts are discussed in the remaining portions of this section.

a. Maximum Random Earthquakes - The largest historical earthquake in the adjacent seismotectonic provinces, not associated with an identified tectonic feature, was a magnitude 7+ event that occurred in 1872. As described in Appendix A, there is considerable uncertainty concerning the epicentral location, focal depth and even the size of this event. With respect to its location, there is no indication that this event occurred in the Columbia Plateau geomorphic province. It probably occurred northwest of the project site in the Northern Cascades, possibly even in British Columbia. Because it cannot be attributed to a specific structure, it is treated as a random event. Based on a comprehensive review of all the available information regarding the 1872 earthquake, conducted for the Washington Public Power Supply System, Woodward-Clyde Consultants (1977) concluded that the 1872 earthquake could have had either a magnitude of 7.1 to 7.3 and a focal depth of 40 to 60 km, or a magnitude of 6.5 and a shallow focal depth of about 20 km. Although the 6.5 magnitude - 20 km focal depth scenario is considered to be more likely, both scenarios are considered possible because of the high degree of uncertainty in the available historical data and because the prevalent interpretations of these data are so conflicting.

There have been few historical earthquakes in the Columbia Plateau province larger than magnitude 4. Given the relatively low level of seismic activity in the province and the

short historical record, it is likely that the maximum random event has not yet been recorded. The maximum random event could be at least M_L 6.0 (local magnitude). Shallow crustal earthquakes larger than M_L 6½ are generally associated with surface fault rupture (Tocher, 1958); repeated events of this magnitude would also produce features recognizable at the earth's surface. It is to be expected, therefore, that events larger than 6½ will be associated with specific fault structures and not occur randomly. Accordingly, the maximum credible earthquake for a random source in the Columbia Plateau province within which the project site is located is estimated to be in the range of M_L 6 to 6½.

Almost all of the recorded seismicity in the associated provinces is occurring at depths of less than 20 km and some small events are occurring at very shallow depths of less than 2 km. Because rupture areas associated with magnitude 6 to 6½ earthquakes will be 100 km^2 to 200 km^2 or larger, it is very unlikely that the earthquakes of this size would nucleate at depths shallower than 5 to 10 km.

b. Probability of Random Earthquakes

The mathematical relationships used to assess the probability of various earthquakes relationships, including magnitude and distance, are presented in Appendix B. Using these relationships, the correlation between earthquake magnitude, probability, and most likely epicentral distances for random earthquakes in the site vicinity were computed and the results are presented on Table 3. As shown in this table, data are presented for magnitudes ranging from 5.0 to 6.5, and for various probabilities ranging from 10^{-2} (1 event of the given magnitude per 100 years) to 10^{-5} (1 event per 10,000 years). Also shown is the 90% confidence interval of the radius of the event (in kilometers) as measured from the site. As shown in Table 3, the epicentral distances associated with any random earthquake event increases with increasing probability (i.e. shorter occurrence interval). Also, for a given probability of occurrence, the epicentral distance increases for increasing earthquake magnitude.

C. Design Basis Earthquakes

As mentioned in the introduction to this section, a time window of 1000 years was selected to determine the computed range in earthquake accelerations associated with the occurrence of random earthquakes. Peak horizontal bedrock accelerations, for both the mean and the mean-plus-one values, were computed for each earthquake magnitude ranging from 5.0 to 6.5, using the attenuation relationships by Sadigh (1989). These acceleration values are presented on Table 4 and, in graphical form, in Fig. 3. Estimates of the peak rock acceleration for earthquake magnitudes other than the values presented on this table can be interpolated from the results presented in Fig. 3.

For design purposes, a value intermediate between the mean and the mean-plus-one is recommended and that the following peak horizontal rock acceleration vs. earthquake magnitude relationship be used:

Random E/Q Magnitude	Peak Horizontal Acceleration, g
5.0	0.04
5.5	0.025
6.0	0.015
6.5	0.01

These values of peak rock acceleration (see Fig. 3) are considered to conservatively represent the earthquake exposure prevalent at the Sherwood Tailing Impoundment site.

The USGS recently completed a probabilistic analysis for the entire United States (Algermissen and others, 1990) in which they estimated the peak ground acceleration. The USGS results for the Pacific Northwest are presented in Fig. 4 in the form of contours of equal acceleration. As shown on this figure, the USGS results indicate that the estimated peak horizontal rock acceleration at the Sherwood site should not exceed a value of 0.06g (i.e., 6% gravity) in the next 250 years. Due to the fact that the seismic data base for eastern Washington is very limited (see Appendix A), we believe the USGS study is based on very conservative assumptions. We further believe that the seismotectonic study performed for the Bureau of Reclamation (Geomatrix, 1990), and used for the seismic assessment of the Sherwood site, is substantially more detailed than that used by the USGS.

D. Conclusions

The in-depth review of the seismotectonic setting performed as a part of this study leads one to conclude that the earthquake activity in the eastern portion of Washington, within which the Sherwood site is located, is relatively quiescent, especially when compared to other relatively nearby areas including western Montana, south-central Idaho, or the western (coastal) regions of Washington and Oregon. Only two historical earthquakes of magnitude 6 or greater are known to have impacted eastern Washington in recorded history. Both of these events, however, occurred prior to the implementation of earthquake instruments. Based on the estimated epicentral distance from the site for both of these events, and the assumed magnitude based on felt area, the peak ground accelerations for these two events were calculated to be less than 0.01 g. The peak bedrock accelerations expected to be generated at the site from future earthquakes on any of eight potentially active faults within 250 km from the site are shown to be between 0.01 and 0.02 g.

The major earthquake exposure to the site is that associated with the occurrence of a random earthquake. Using probabilistic theory based on recorded data, epicentral distances were computed for random earthquake magnitudes ranging from 5.0 to 6.5. Using the unique relationship for a probability of non-occurrence of 0.001 (frequency of one event per 1000 years), the peak horizontal acceleration values were computed using both mean and mean-plus-one values, and are presented on Table 4. The recommended design values for peak ground acceleration versus earthquake magnitude are presented in Fig. 3.

TABLE 1

CLOSEST APPROACHES OF POTENTIAL EARTHQUAKE SOURCES TO SHERWOOD TAILING IMPOUNDMENT

Source	Classi-fication	Maximum Credible Earthquake[1]	Closest Distance (km)
Satus Peak Segment/ Toppenish Ridge Fault	Potentially Active Fault	M_s 7	253
Ahtanum Creek Fault	Potentially Active Fault	M_s 7-7¼	247
Central Fault on Gable Mountain	Active Fault	M_s 5	177
Gable Butte-Gable Mtn. Segment	Potentially Active Inferred Fault	M_s 7-7¼	175
Smyrna Bench Segment/ Saddle Mtn. Fault	Potentially Active Fault	M_s 6¾	152
Saddle Gap Segment/ Saddle Mtn. Fault	Potentially Active Fault	M_s 7	153
Powder Ranch Segment/ Frenchman Hills Fault	Potentially Active Fault	M_s 7	173
West Canal Segment/ Frenchman Hills Fault	Potentially Active Fault	M_s 7	150

Notes:

(1) MCE values are from Geomatrix (1990). (M_s = Surface Wave Magnitude)

(2) Fault locations are shown on Figure A-1.

TABLE 2

EXPECTED PEAK ROCK ACCELERATION VALUES FOR POTENTIALLY ACTIVE FAULTS IMPACTING THE SHERWOOD TAILING IMPOUNDMENT

Source	Maximum Credible Earthquake	Closest Distance (km)	Expected Peak Rock Acceleration, g Mean	Mean+1
Satus Peak Segment/ Toppenish Ridge Fault	M_s 7	253	0.006	0.009
Ahtanum Creek Fault	M_s 7-7¼	247	0.009	0.012
Central Fault on Gable Mountain	M_s 5	177	0.002	0.003
Gable Butte-Gable Mtn. Segment	M_s 7-7¼	175	0.016	0.022
Smyrna Bench Segment/ Saddle Mtn. Fault	M_s 6¾	152	0.013	0.018
Saddle Gap Segment/ Saddle Mtn. Fault	M_s 7	153	0.016	0.022
Powder Ranch Segment/ Frenchman Hills Fault	M_s 7	173	0.013	0.018
West Canal Segment/ Frenchman Hills Fault	M_s 7	150	0.016	0.023

Notes:

1. Peak rock acceleration (PRA) values were computed using attenuation model by Sadigh (1989).

2. For those MCE values showing a range, PRA values are for the higher MCE value.

TABLE 3

MOST LIKELY EPICENTRAL DISTANCES (AND 90% CONFIDENCE LIMITS) FOR RANDOM EARTHQUAKES NEAR SHERWOOD TAILING IMPOUNDMENT

Most likely and (90%) Confidence intervals for Radius in kilometers.

Random Earthquake Magnitude	Probability of Non-occurrence				
	$1.0x10^{-5}$	$2.0x10^{-5}$	$1.0x10^{-4}$	$1.0x10^{-3}$	$1.0x10^{-2}$
5.0	5 (3-6)	6 (4-7)	12 (9-15)	35 (27-45)	101 (72-131)
5.5	7 (5-9)	10 (6-12)	20 (14-27)	61 (44-78)	170 (121-224)
6.0	12 (8-16)	16 (11-22)	35 (23-49)	104 (74-124)	>250
6.5	20 (12-29)	27 (17-41)	61 (39-84)	185 (121-232)	>250

Notes:

1. See Appendix B for probabilistic analysis used to develop the data presented hereon.

2. A probability of non-occurrence of $1x10^{-3}$, representing a frequency of 1 event per 1000 years was selected for "Design Earthquake" considerations.

TABLE 4

MOST LIKELY PEAK ROCK ACCELERATION VALUES FOR RANDOM EARTHQUAKES SHERWOOD TAILING FACILITIES

Random Earthquake Magnitude	Most Likely Radius from Site (km)	Maximum Peak Bedrock Acceleration, g	
		Mean Value	Mean +1 Value
5.0	35	0.029	0.051
5.5	61	0.019	0.030
6.0	104	0.012	0.018
6.5	185	0.007	0.010

Notes:

1. All acceleration values were computed using the attenuation relationships presented by Sadigh (1989).

2. The probability of occurrence for each random earthquake is 0.001 (a frequency of one event per 1000 years).

3. The mean value indicates that there is a 50% chance that the actual peak bedrock acceleration could be higher or lower that the value presented. The mean + 1 value indicates that there is a 13% chance that the actual peak acceleration could be higher and a 87% chance that the value would be lower than the value shown.

III. MATERIALS DISTRIBUTION

A. Construction Procedures

The exterior dam, which acts as the containment for the tailing impoundment, is a zoned earth embankment. The initial starter dam was expanded as necessary in a downstream direction. The tailing were discharged into the impoundment by perimeter spigotting. As such, the grain size distribution within the tailing pond was controlled during construction primarily by the principles of natural sedimentation. This natural material distribution occurs in any type of slurry discharge due to the sedimentation of the coarser grains closest to the point of discharge and the finer grains further away from the point of discharge. The grain size distribution, however, is not uniform because points of discharge vary during construction, the pond size increases as the perimeter dikes are raised, and distribution methods do not remain constant during construction. The potential for earthquake-induced settlement within the impoundment is directly tied to the variability of sands and silts within the impoundment. In order to assess this settlement potential, we must evaluate how these two materials will act during earthquake motions.

B. Exploration Results

SMI drilled a number of exploratory borings within the tailing pond area to assess the nature and distribution of the tailing materials. One series of borings, which we understand was located somewhat closer to the crest than the other exploratory holes, was specifically drilled to assess whether it would be practical to consider dewatering the tailing during reclamation. Samples for this series of borings were obtained at relatively close intervals (6-7 inches) as compared to the other exploratory holes. A description of the methods used, hole locations, and field results is presented in Appendix A of the Tailing Reclamation Plan, and was transmitted to RLVA in a letter from SMI dated July 13, 1993, which is included in Appendix C of this report. In general, laboratory test results indicate that the tailing material varies from a relatively clean, poorly graded, sand (SP) to a highly elastic silt (MH), although the majority of results show the tailing to vary from a silty sand (SM) to a silt of low plasticity (ML). The following discussion of field and laboratory test results focusses only on those results that have an impact in assessing the liquefaction potential of the tailing material.

1. Gradation Test Results

As mentioned above, the samples for the drill holes were taken at close vertical intervals to assess the variation in the percentage of fines within the tailing material. The test results for one of these holes (Hole 1A) are presented in Fig. 5, in the form of percentage of fines vs. depth. As shown in Fig. 5, within the upper 10 feet the results indicate that the percentage of fines varies between a low of 2% fines to a maximum of 32% fines. Between a depth of 10 and 20 feet the results indicate the percentage of fines varies

dramatically over relatively short thickness intervals. For example, at a depth of about 15 feet the percentage of fines is about 80%, whereas at a depth of 17 feet the percentage of fines has dropped to about 15%. Between a depth of about 20 feet to a depth of 48 feet, the percentage of fines ranges between 5% and 20% (average of 13%) with three relatively thin lenses of siltier horizons where the percentage of fines increases to between 37% and 42%. Below a depth of 48 feet, and down to 70 feet which represents the approximate maximum depth of tailing, the gradation results indicate interlayered silty sand and silt materials. Based on the gradation results from this hole, the pure silt horizons (i.e. more than 50% fines) do not appear to be more than about 1-2 feet in thickness, although this observation may be influenced by the sampling/testing interval. Other holes from this series of exploration showed similar variations in the percentage of fines, but not necessarily at the same depth intervals. This apparent lack of horizontal continuity in material type was confirmed when pumping tests performed on two relatively close holes (one of which was Hole 1A) showed a wide range in well capacity (less than 1 gal/min to about 5 gal/min) (see Appendix P of the Tailing Reclamation Plan). These field results suggest that, at least over the distance of the two test pump holes (60 feet), the tailing material does not appear to contain similar or contiguous thicknesses of more permeable sands.

The sandier portion of the tailing material is defined as fine to very fine grained sand. The cleaner portion of the sands classify as an SP-SM (poorly graded clean to silty sands with between 5% and 12% fines) and have a median grain size (D_{50}) of between 0.25mm and 0.35mm (between the No. 40 and No. 50 U.S. Standard Sieve). The dirtier sands classify as an SM (between 12% and 50% fines) and have a D_{50} size of about 0.15mm (No. 100 U.S. Standard Sieve).

The gradation results from 7 of the 10 other exploratory borings (T-1 through T-10) are presented in Appendix C. Although these borings were performed to gather general engineering data for the tailing pond area, and were not sampled specifically for gradation results at the same relatively close frequency discussed above for Hole 1A, the gradation results for the seven holes tested show a similar, but perhaps finer, trend of interlayers of more pervious silty sands and less pervious sandy silts to silts. The specific trend from Hole 1A that was not confirmed by the other exploratory holes was a similar range in the percentage of fines between a depth interval from about 20 and 48 feet. Six of the seven holes penetrated at least to 20 feet, and two of the Holes (T-4 and T-7) were taken to depths greater than 60 feet. Within these six holes, 33 gradation tests were performed between a depth interval of 20 and 48 feet. Only 7 of these 33 gradation tests had a percentage of fines less than 20%. More likely than not, the above results tend to confirm that major areas of the tailing impoundment, at points greater than about 200 to 300 feet from the point of tailing discharge, tend to be finer grained (siltier) than those portions on the tailing impoundment closer to the point of tailing discharge. This increase in fines content toward the interior of the impoundment is entirely consistent with other sites where perimeter discharge was used (Volpe, 1979), and with other uranium tailings materials (Vick, 1983).

2. In-Place Water Content and Dry Density

Relatively undisturbed samples were obtained using thin-wall tube samples from 5 of the 10 borings referenced above for the purpose of determining the variation of in-place water content and dry density and other engineering properties. These laboratory results are also summarized on Sheet 1 in Appendix C. As shown on this summary sheet, 26 samples were tested; 14 samples are classified as a silty sand (SM), and 12 samples were classified as sandy silt (ML). The average results are summarized below on Table 5:

Table 5

Summary of Water Content and Density Test Results

Mat. Type	No. of Samples	Total Unit Wt. (pcf) High	Low	Mean	Dry Unit Wt. (pcf) High	Low	Mean	Water Content (%) High	Low	Mean
SM	14	122.4	101.2	111.9	100.5	67.5	84.2	55.1	21.7	33.6
ML	12	113.5	92.6	102.8	76.1	44.3	60.1	107.9	46.7	72.7

Based on the field data, it appears that the current water table within the pond is at a depth of about 10 feet below ground surface. After the reclamation cover has been constructed, however, it has been assumed by SMI that the water table could migrate upward to the interface of the new cover and the current tailing surface. This assumption, as discussed below, does not adversely affect the liquefaction analyses discussed herein.

For purposes of assessing the settlement of the new reclamation cover, SMI reviewed the field and laboratory results and decided to divide the tailing impoundment into approximately 70 cells, each having its own engineering properties. Also, based on the composition of the future reclamation cover, is has been assumed that it will impose a total average overburden pressure of 898 psf over the impoundment surface. In order to assess the liquefaction potential of the tailing, it is necessary to estimate both the effective and total overburden pressures as a function of depth. For the liquefaction analysis, the following assumptions were made relative to the total and effective overburden pressures:

1) it was assumed that the reclamation cover will be in place and that the full impact of this increase in effective overburden should be taken into account;

2) the following relationships were assumed in order to compute the approximate effective and total overburden pressure as a function of depth (D);

Depth (feet)	Equations used for Overburden Pressure (psf) Effective	Total
< 10	898 + 87.9*D	898 + 87.9*D
> 10	1777 + 48.6*(D-10)	1777 + 112*(D-10)

Estimates of the effective and total future overburden stresses within the tailing impoundment, using the relationships presented above, are presented in Fig. 6. This effective and total stress distribution was used in the liquefaction analysis. As shown in graphical form in Fig. 6, it is assumed that the water table migrates upward to the interface of the new reclamation cover and the tailing surface.

3. Standard Penetration Test Results

The variation of Standard Penetration Test Results (SPT or N Value) is a measure of in-place relative density of the material and was performed in accordance with ASTM D-2056. The SPT test result represents the number of blows of a 140 pound hammer required to drive a sampler of a specified size 18 inches in the soil. The number of blows to drive the sampler is recorded for each 6-inch interval and the N Value is reported as the total number of blows to drive the sample the last 12 inches, hence the units are blows/ft. For this project, the SPT tests were performed in a hollow-stem auger drill stem. Once the free standing water surface was encountered, the hollow stem was filled with water in order to maintain essentially the same water pressure at the drill bit and prevent excessively high seepage gradients from developing at the tip of the drill bit. Plots of the measured N Value as a function of depth for Borings T-1 through T-10 are presented in Appendix D. More discussion regarding how the N values were used to assess the liquefaction potential of the tailing is presented in following section.

IV. ASSESSMENT OF LIQUEFACTION POTENTIAL

A. Introduction

The engineering studies carried out to assess the liquefaction potential of the tailing material at the Sherwood Tailing Impoundment are described in this section of the report. Before commencing this discussion, however, it should be noted that only the tailing materials are considered susceptible to potential liquefaction. The foundation glacial deposits are not considered potentially liquefiable since they are both unsaturated, and considerably more dense than the tailing materials.

B. Simplified Liquefaction Analyses

1. Introduction

The year 1966 marked the start of geotechnical earthquake engineering as currently practiced with the publication by H.B. Seed and K.L. Lee from the University of California at Berkeley on the "Liquefaction of Saturated Sands During Cyclic Loading". Since that time, various analytical procedures have been developed by a number of investigators for evaluating liquefaction potential of saturated cohesionless soil deposits. The liquefaction potential of a soil deposit is dependent on many factors other than gradation. Included in these other factors are such values as peak ground acceleration, duration of strong shaking, relative density or degree of compaction of the soil, boundary conditions, and permeability/drainage characteristics of the soil deposit. Although much of the earlier liquefaction research dealt with the development of proper laboratory testing procedures, it is now standard practice in geotechnical earthquake engineering to use carefully developed empirical methods which rely heavily on field data. These empirical methods have been developed by reviewing the results of saturated cohesionless soil deposits where liquefaction is known to have occurred, or been resisted, during actual earthquake shaking.

Standard Penetration Test (SPT) blow count measurements have been shown to provide an excellent correlation with the degree of compaction (and liquefaction potential) of cohesionless soils. As part of the current study for the Sherwood Impoundment, the number of blows required to drive an SPT sampler in the soil deposit a distance of up to 18 inches was recorded. The samplers were driven into the soil deposit using a doughnut-shaped 140-pound hammer (hammer energy ratio of 45%) falling freely through a distance of 30 inches, and using a rope-and-pulley system. These field procedures adopted by SMI are in compliance with the procedures recommended by Seed, et al. (1985). In this article, the authors evaluate the influence of SPT procedures in evaluating soil liquefaction resistance (i.e., the current ASTM Test method D-1586).

An evaluation of the liquefaction potential at the Sherwood Impoundment was completed using SPT blow-count data performed at the site, and the "Simplified Seed Method" for a horizontal soil deposit (Seed et al., 1967). This method was originally developed for evaluating the liquefaction potential of saturated clean sand and silty deposits, and subsequently has been modified to include other soil types as discussed below.

The relationship of modified penetration resistance, defined herein as the $(N_1)_{60}$ value, versus cyclic stress ratio (CSR) (τ_{avg}/σ_0'), for observed conditions of liquefaction is presented on Fig. 7 for a Magnitude 7.5 earthquake (Seed et al., 1983, 1984). The data points shown in this figure represent a comprehensive collection and assessment of site conditions where evidence of liquefaction, marginal liquefaction, or no liquefaction, is known to have occurred during past Magnitude 7.5 earthquakes. Relationships of this type have been developed for different magnitude earthquakes and for sands with different fines contents. It should be noted that the majority of liquefaction case histories shown on Fig. 7 have occurred at relatively shallow depths, generally on the order of 30 feet or less. The relationship shown in Fig. 7, together with similar relationships developed for other values of fines content, have been used in the liquefaction analysis discussed herein. Finally, it should be noted that the $(N_1)_{60}$ value is derived directly from the field measured (uncorrected) N value as discussed below.

2. Review and Interpretation of Blow-Count Data

In order to evaluate the liquefaction potential of the tailing material using the "Simplified Seed Method," the uncorrected field-measured blow count (N) data were first reviewed, interpreted and analyzed in various ways. In order to use the field-measured N value data with the "Simplified Seed Method," it is first necessary to apply various correction factors to the data to account for the overburden stress and the percentage of fines for the sample where the N value is determined. The field-measured N values were corrected to account for the following:

a. Drill Rod Stiffness - This correction is appropriate when the drill rod length is less than 10 feet; N_c = 0.75 (Seed et al., 1985).

b. Hammer Efficiency - When using a doughnut-type hammer with rope and pulley, the energy ratio is only 45% of that for a safety hammer. It has been recommended that the uncorrected N value be multiplied by N_c = 0.75 to account for this difference (Seed et al., 1985).

c. SPT Sampler Without Liner - Blow counts measured without liners are lower than those obtained when liners are used inside the SPT sampler, N_c = 1.2 (Seed et al., 1985).

d. Silty Materials - Blow counts were increased by 7.0 when fines content was greater than 35% (Seed et al., 1985). Since gradation tests were not

performed for every SPT, it was necessary to assume a gradation based on the description of materials presented on drill hole logs.

e. Overburden Effects - The relationships provided by Seed et al. (1983) were used to correct the measured blow counts. This relationship is referred to as C_n and its relationship with effective overburden is presented in Fig. 8.

The $(N_1)_{60}$ corrected blow counts were determined for all field-determined SPT values, on a hole-by-hole basis, using the above referenced five correction factors in the order presented. Spread sheets showing the detailed calculations are presented on Sheets 1 through 6 in Appendix D and the computed $(N_1)_{60}$ values are plotted as a function of depth for Borings T-1 through T-10 on Fig. D-11 through Fig. D-20, respectively. As noted in Section III, the liquefaction calculations were performed using the future effective stress that will be imposed after the protective cover has been placed over the tailing impoundment.

A statistical assessment was performed for the $(N_1)_{60}$ values computed for depths greater than 10 feet. The data from a depth less than 10 feet was not used in the statistical assessment because this section of the impoundment tends to be desiccated and the original SPT data may be higher than normal. The results of the statistical assessment for the 108 $(N_1)_{60}$ values are presented below in Table 6.

Table 6

Summary of Statistical Assessment of $(N_1)_{60}$ values

Parameter	Value
Average (numerical)	12
Median (middle value)	10
Mode (most common value)	8

According to Tokimatsu and Seed (1987), the corrected blow counts are representative of sand materials with relative densities that are loose to medium dense in consistency.

3. Correlations For Different Magnitude Earthquakes

The results presented in Fig. 7 provide a realistic basis for developing correlations between SPT values and the liquefaction characteristics of sands and silty sands for a Magnitude 7½ earthquake. These results can be extended to other magnitude events by noting that from a liquefaction point of view, the main difference between different magnitude events is in the number of cycles of stress which they produce. Statistical studies of actual earthquake accelerograms (Seed et al., 1983) show that the number of cycles representative of different magnitude earthquakes is typically as shown in Table 7, below.

Table 7

Number of Cycles Representative of Different Magnitude Earthquakes

Earthquake Magnitude	Number of Representative Cycles at 0.65 τ_{max}
8½	26
7½	15
6¾	10
6	5-6
5¼	2-3

Using this concept of a lower number of representative cycles for a lower magnitude earthquake, the data presented in Fig. 7 for a magnitude 7½ event have been modified for other earthquakes of lower magnitude. These data are presented in Fig. 9 in the form of modified penetration resistance $(N_1)_{60}$ vs. cyclic stress ratio causing liquefaction in clean sands for earthquakes ranging from M 5¼ to M 7½. The same curves are also used for silty sands and sandy silts, provided the SPT values are normalized using the correction factors previously discussed, before entering the chart shown in Fig. 9. The steps used in the liquefaction assessment for the Sherwood Tailing Impoundment are discussed below.

4. Liquefaction Assessment

The liquefaction potential of the tailing material was evaluated using the "Simplified Seed Method" previously described and shown on Fig. 9. The cyclic stress ratios (τ_{avg}/σ_0') induced by the several random earthquakes analyzed for this study were computed using a simplified procedure outlined by Seed and Idriss (1967). In this method, the cyclic stress ratio within a horizontal soil deposit may be estimated using the relationship:

$$\left(\frac{\tau_{avg}}{\sigma_o'}\right)=0.65*a_{max}\left(\frac{\sigma_o}{\sigma_o'}\right)*r_d \qquad (1)$$

where a_{max} = peak ground acceleration; σ_o = total overburden pressure at a given depth; σ_o' = effective overburden pressure; and r_d = a stress reduction factor varying from a value of 1.0 at the ground surface to an average value of about 0.9 at a depth of 30 feet. For these analyses, the value of r_d was fixed at 0.9 for depths greater than 30 feet.

Detailed liquefaction analyses were performed for all ten borings at each depth location where a penetration test was performed, and for random earthquake magnitudes of 5.0, 5.5, 6.0 and 6.5 (See Appendix E). The methodology used to assess the liquefaction potential at the site is summarized below:

1) Values of the induced CSR were computed as a function of depth through the tailing deposit using the relationship presented in Equation (1), the unique peak ground acceleration (a_{max}) associated with each random earthquake, and the overburden stress ratio assuming the reclamation cover is in place.

2) Values of the resisting CSR based on the $(N_1)_{60}$ value was developed using the relationships presented in Fig. 9.

3) The factor of safety (FS) against liquefaction was computed by dividing the resisting CSR determined in step (2) by the induced CSR determined in step (1).

4) Steps (1) through (3) were repeated for each random earthquake.

Detailed calculations for Borings T-1 through T-10 are presented in Appendix E, and are grouped according to each random earthquake magnitude. A summary of the 118 computed factors of safety against liquefaction are summarized below on Table 8. In a manner similar to that described previously for Table 6, only those values for depths greater than 10 feet have been used in the statistical assessment.

Table 8

Summary of Statistical Assessment of FS Against Liquefaction

	---------- Random Earthquake Magnitude ----------			
Parameter	M=5.0	M=5.5	M=6.0	M=6.5
Average (numerical)	5.1	7.6	11.6	16.1
Median (middle value)	4.8	7.1	10.9	15.1
Mode (most common value)	5.6	8.3	12.7	17.6
Standard Deviation	1.4	2.0	3.1	4.3

5. Sensitivity Analysis

A special study was performed to determine the sensitivity of the liquefaction assessment. Using the same methodology as presented above, the maximum peak ground acceleration was increased until the median FS value was reduced to 1.0 for the various random earthquake magnitudes referenced above. The results of this special study show that the peak accelerations required to induce liquefaction are increased from the design values used in the analysis, and presented in Fig. 3, to 0.16g, 0.17g, 0.18g, and 0.20g for magnitude 6.5, 6.0, 5.5, and 5.0 random earthquakes, respectively. The probability of such peak acceleration values being developed at the Sherwood Tailing Impoundment over a 1000 year time interval is virtually non-existent.

C. Conclusions

Only one location from boring T-5 at a depth of 28 feet had a computed factor of safety less than 1.0 for a magnitude 5.0 event. Even at this depth, the factor of safety increases to above 1.0 for a magnitude 5.5 event. Clearly, the major material property that will inhibit the development of liquefaction within the tailing is the fines content. It should also be noted that the liquefaction analyses were performed using conservative assumptions, especially with regard to the estimated peak ground acceleration and the significant improvement in density and shear strength that will occur, within the upper 30 feet or so of the tailing, after the reclamation cover has been installed. Based on the results presented above, it is concluded that the tailing materials are not susceptible to earthquake-induced liquefaction, even under the most conservative set of earthquake related assumptions.

V. LIMITATIONS

The recommendations and opinions stated in this report reflect RLVA's current understanding of the project requirements and the current state-of-practice for geotechnical engineering, engineering geology, and seismic geology. Our understanding is based on the investigation and evaluation methods performed by others and described in this report, and on the assumptions implicit in those methods. In the performance of our professional services, RLVA, its employees, and its agents comply with the standard of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. No warranty, either express or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services or by the furnishing of oral or written reports or findings. We are responsible for the conclusions and recommendations contained in this report, which are based on data relating only to the specific project and location discussed herein.

Future changes in the understanding of the seismotectonic setting of central and eastern Washington could impact the findings presented in this report, although it should be noted that rather conservative assumptions have been used to assess the potential earthquake related accelerations impacting the site. As noted in Section IV of this report, the peak acceleration required to develop potential liquefaction within the tailing is between 0.16 g and 0.20 g, depending on the magnitude of the earthquake. These values are between about 4.0 and 5.0 times larger than the 0.04 g peak acceleration value used for the liquefaction analysis in assessing the impact of a magnitude 5.0 random earthquake event. In conclusion, therefore, if any future study or findings suggest that a peak ground acceleration in excess of 0.20 g for a magnitude 5.0 or greater earthquake can impact the site, then the conclusions regarding the liquefaction stability of the tailings should be reviewed.

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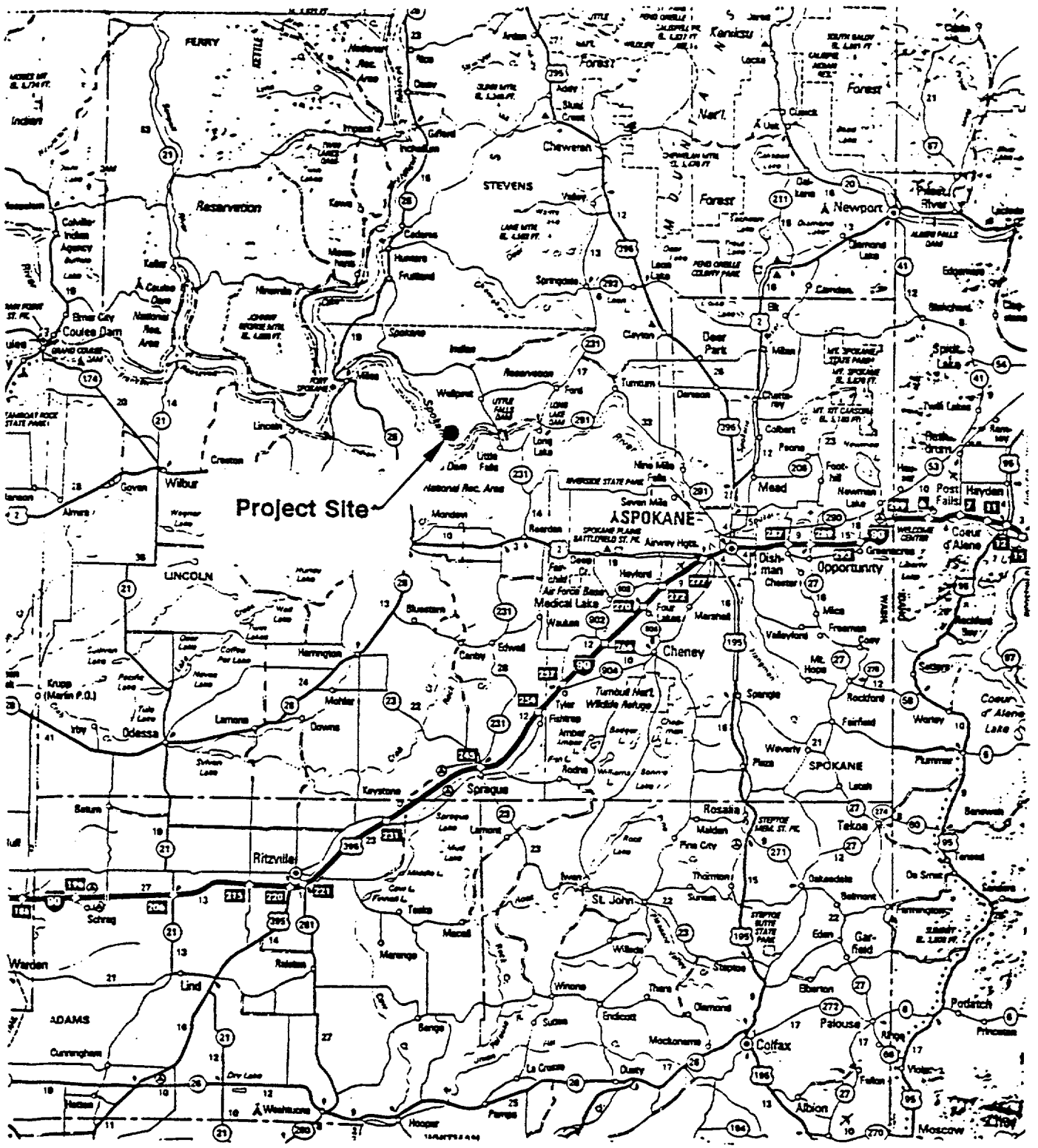
L.C-35



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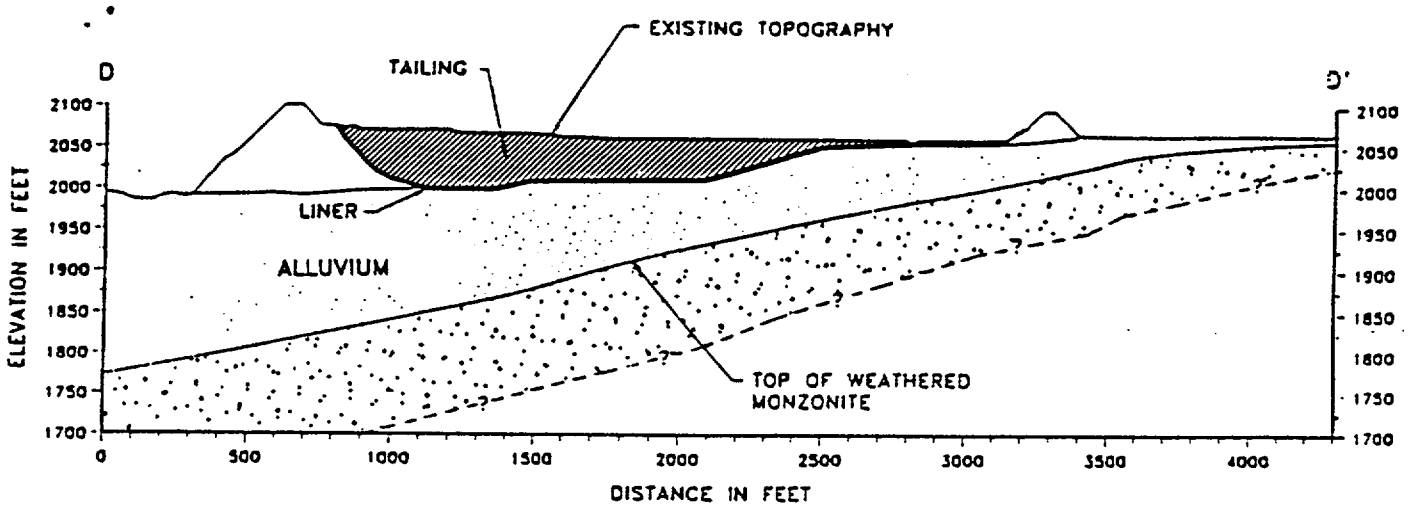
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

LOCATION MAP

Checked by Jw	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100.1	1

L.C-36

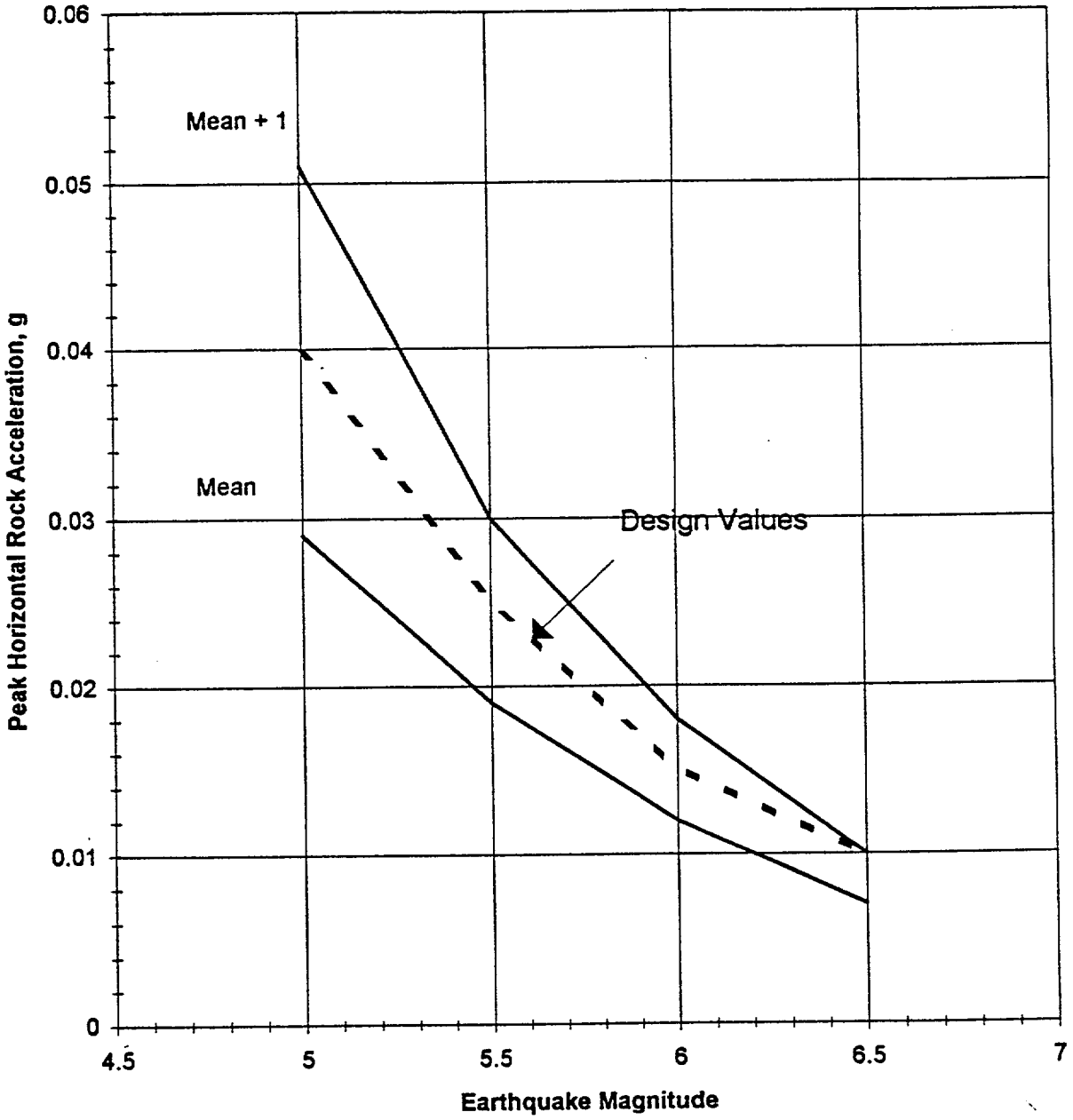


SCALE: V:H = 3:1

Cross section D-D' showing typical geologic cross section of tailing impoundment and foundation conditions. Note that the ratio of the vertical and horizontal scale is 3:1.

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT TYPICAL CROSS SECTION			
Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100	2

L.C-37



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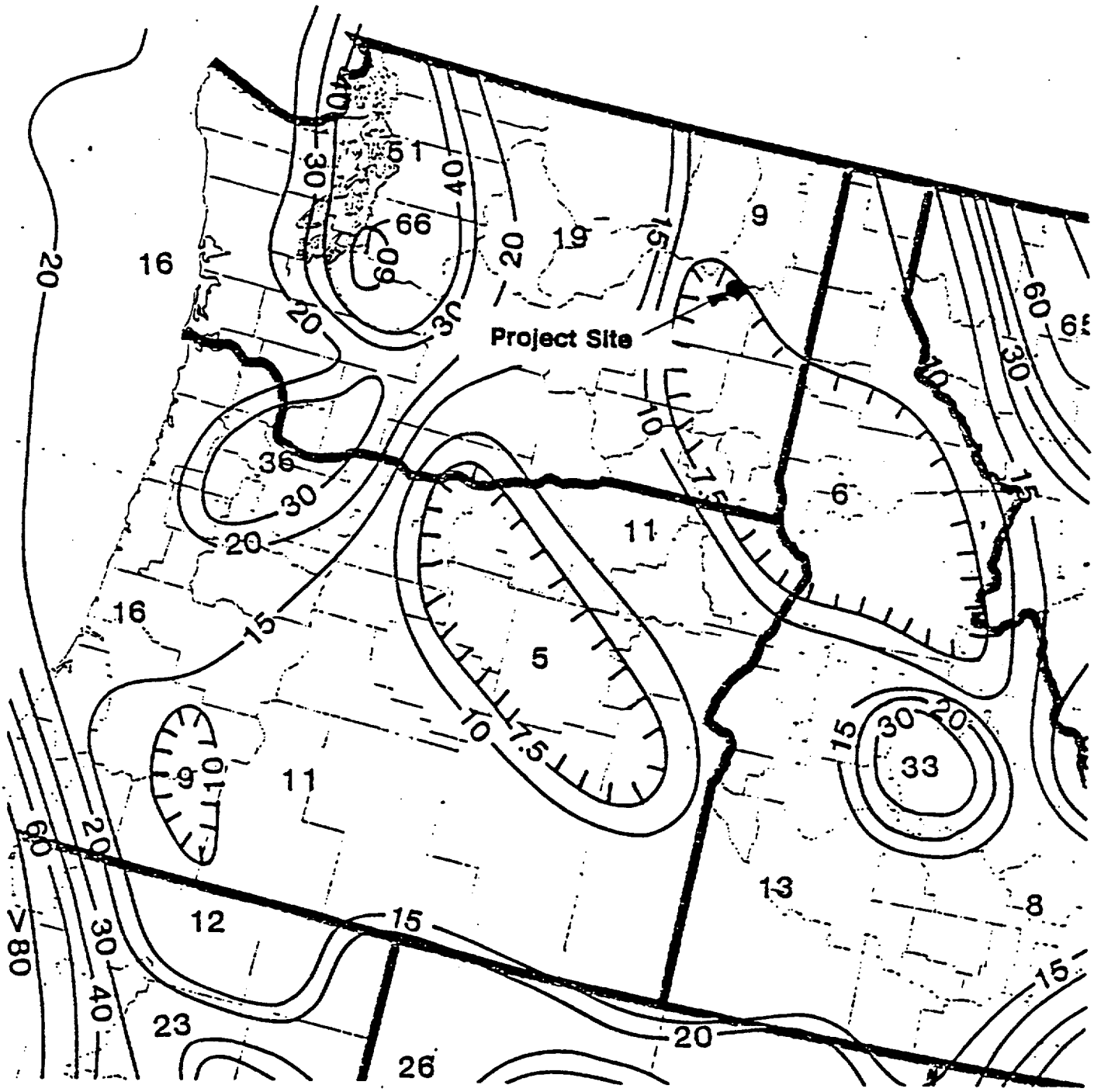
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

DESIGN PEAK ACCELERATION VALUES

Checked by _____ Date _____	Project No.	Figure No.
Approved by _____ Date _____	SMI-100	3

L. C-38



Map showing contours of equal ground acceleration in percent of gravity based on probabilistic analysis. The probability of the acceleration shown occurring in the next 250 years is 90%.

(From Algermissen et al., 1990)

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT			
CONTOURS OF EQUAL ACCELERATION			
Checked by [illegible] Date 5-4-94	Project No.	Figure No.	
Approved by RLV Date 5-5-94	SMI-100	4	

WELL COMPLETION LOG

MONITORING WELL NO: 3A

LOGGED BY: JGC

DEPTH (FT)	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
0	2075.84	MIX
5	SAND	
6	SLIMY-SAND	
7	SAND	
	SLIMY-SAND	
8	SAND	
9	SLIMY-SAND	SAND
	SAND	MIX
10	SLIMY-SAND	MIX
	SANDY-SLIMES	
11	SLIMY-SAND	
	SLIMES	
12	SLIMY-SAND	SLIMES
	SAND	
13	SLIMES	
14	SLIMY-SAND	MIX
15		SLIMES
16	YELLOW SLIMES	
	SLIMES (SOME YELLOW)	
	SANDY-SLIMES	
17	SLIMES	MIX
	SANDY-SLIMES	
18	SLIMES	
	SLIMY-SAND	
19	SLIMES	
	SLIMY-SAND	
20	SLIMY-SAND	MIX
21	SLIMES	
22	SAND	SLIMES
	SANDY-SLIMES	
	SLIMES	
23	SLIMY-SAND	SAND
	SLIMES	
	SLIMY-SAND	
24	SAND	MIX
	SLIMY-SAND	
25	SANDY-SLIME	
26	SLIMY-SAND	
27	SAND	SAND
28	SLIMY-SAND	
29	SAND	MIX
	SLIMY-SAND	

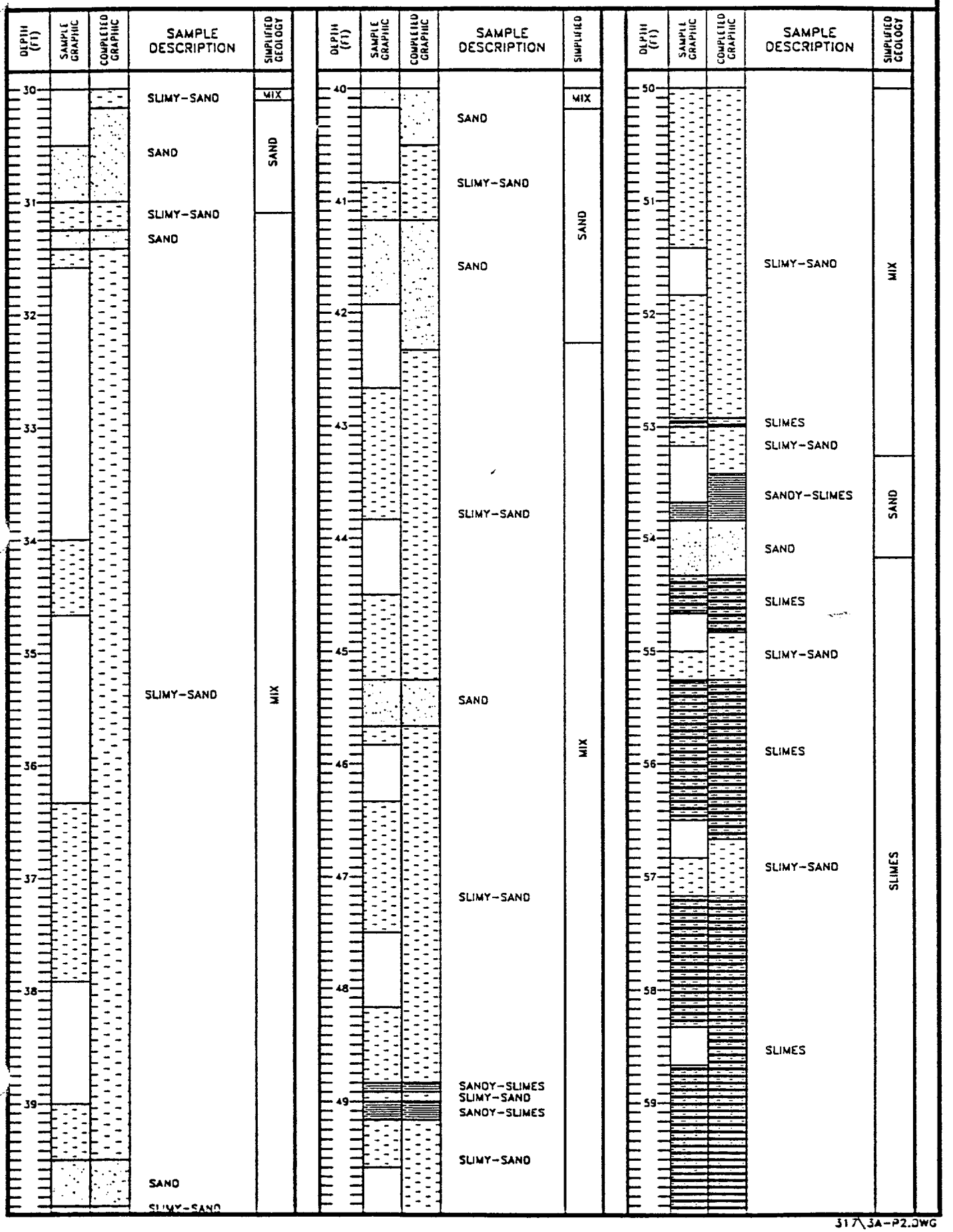
317\3A-P1.DWG

L.C-21

WELL COMPLETION LOG

MONITORING WELL NO: 3A

LOGGED BY: JGC



317\3A-P2.DWG

L.C-82

WELL COMPLETION LOG

MONITORING WELL NO: 3A

LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60				SLIMES
			SLIMES	MIX
61				
			SLIMY-SAND	
62				SLIMES
63			SLIMES	
64				
65			SANDY-SLIMES	
			SLIMES	MIX
66			SANDY-SLIMES	
67				
68				SLIMES
69				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				SLIMES
71				
72				
73				
74				
75				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

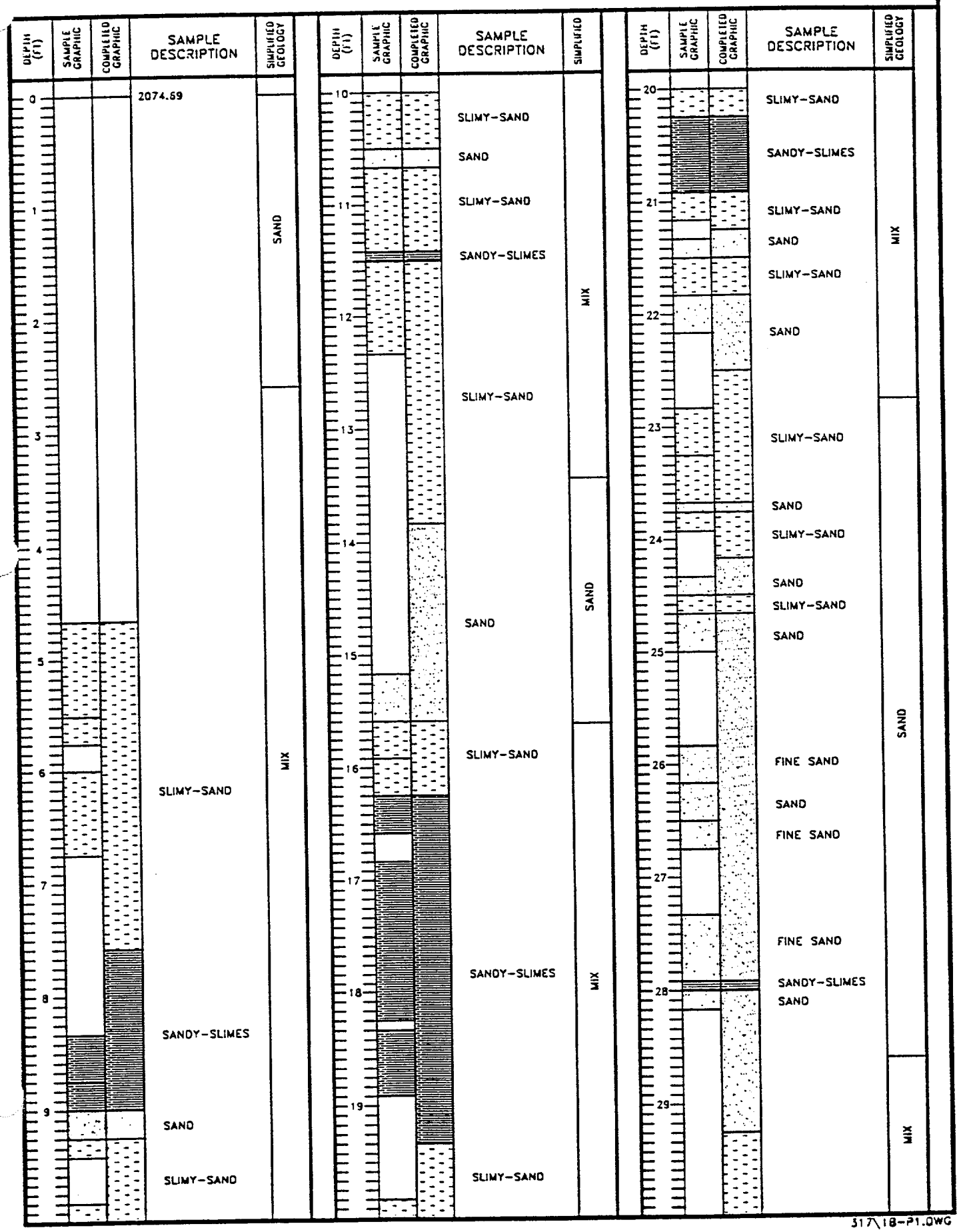
317\3A-P3.DWG

L.C-23

WELL COMPLETION LOG

MONITORING WELL NO: 1B

LOGGED BY: JGC



317\18-P1.DWG

L.2-24

WELL COMPLETION LOG

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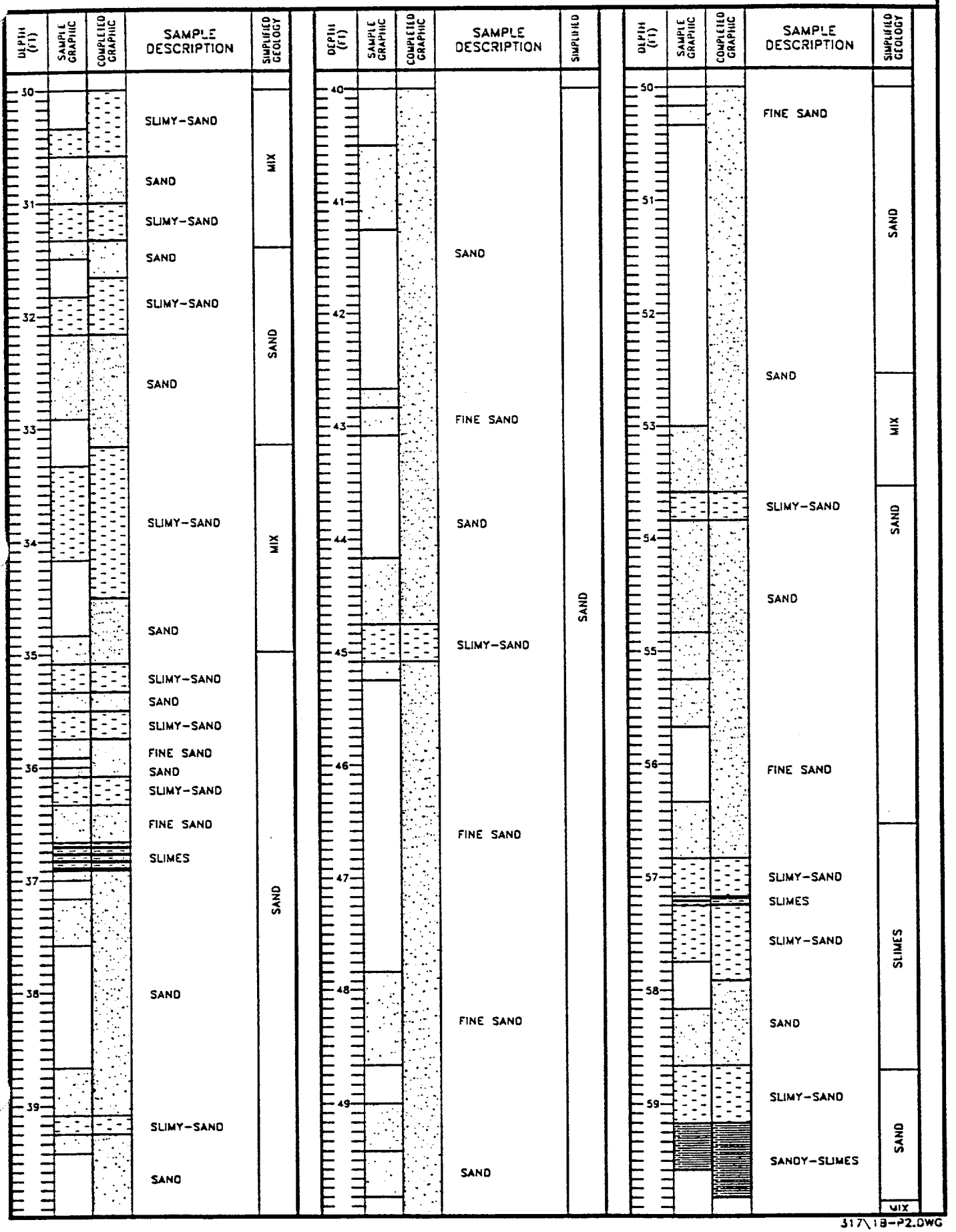
LOGGED BY: JGC



317\1B-P2.DWG

L.C-25

WELL COMPLETION LOG

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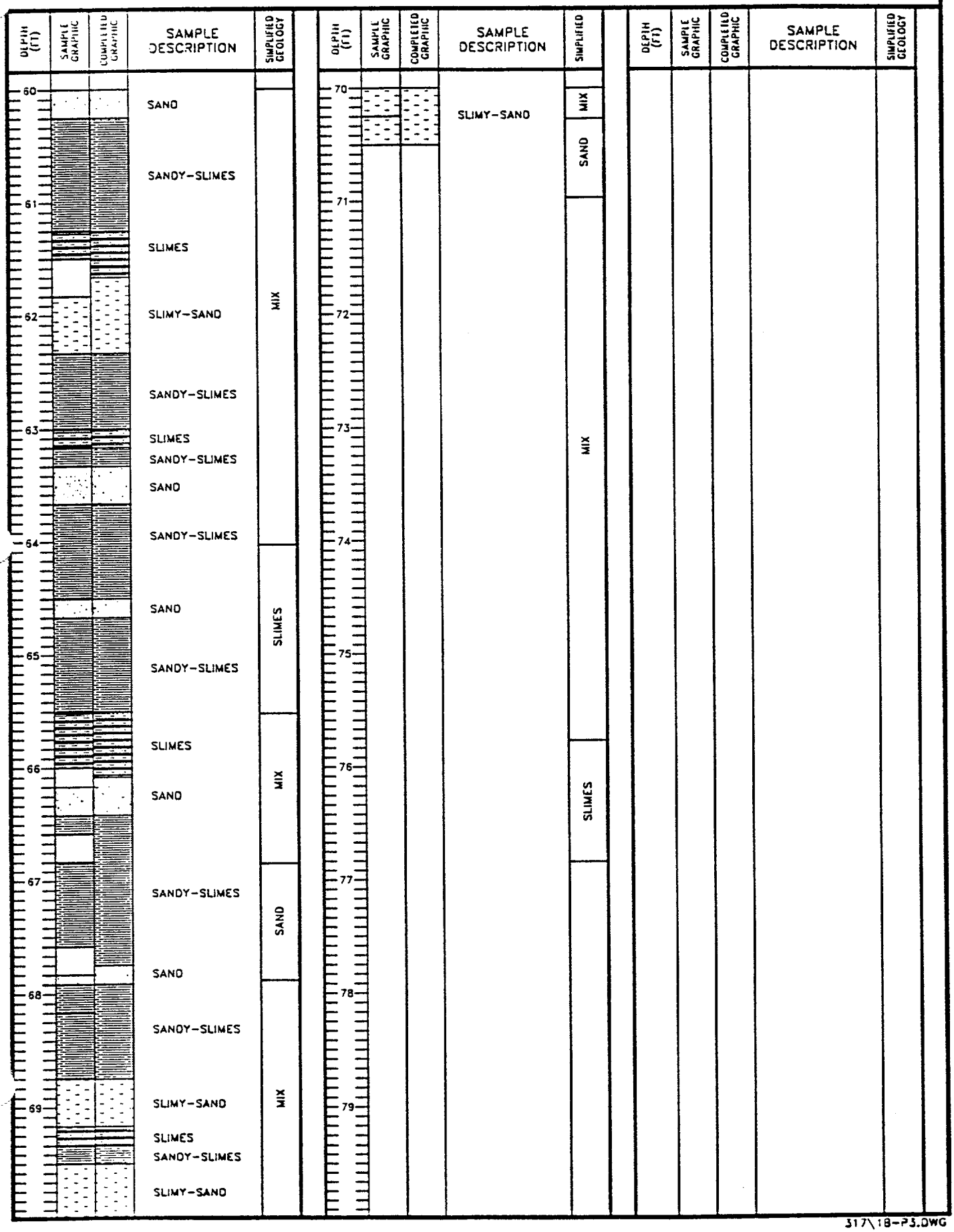
LOGGED BY: JGC



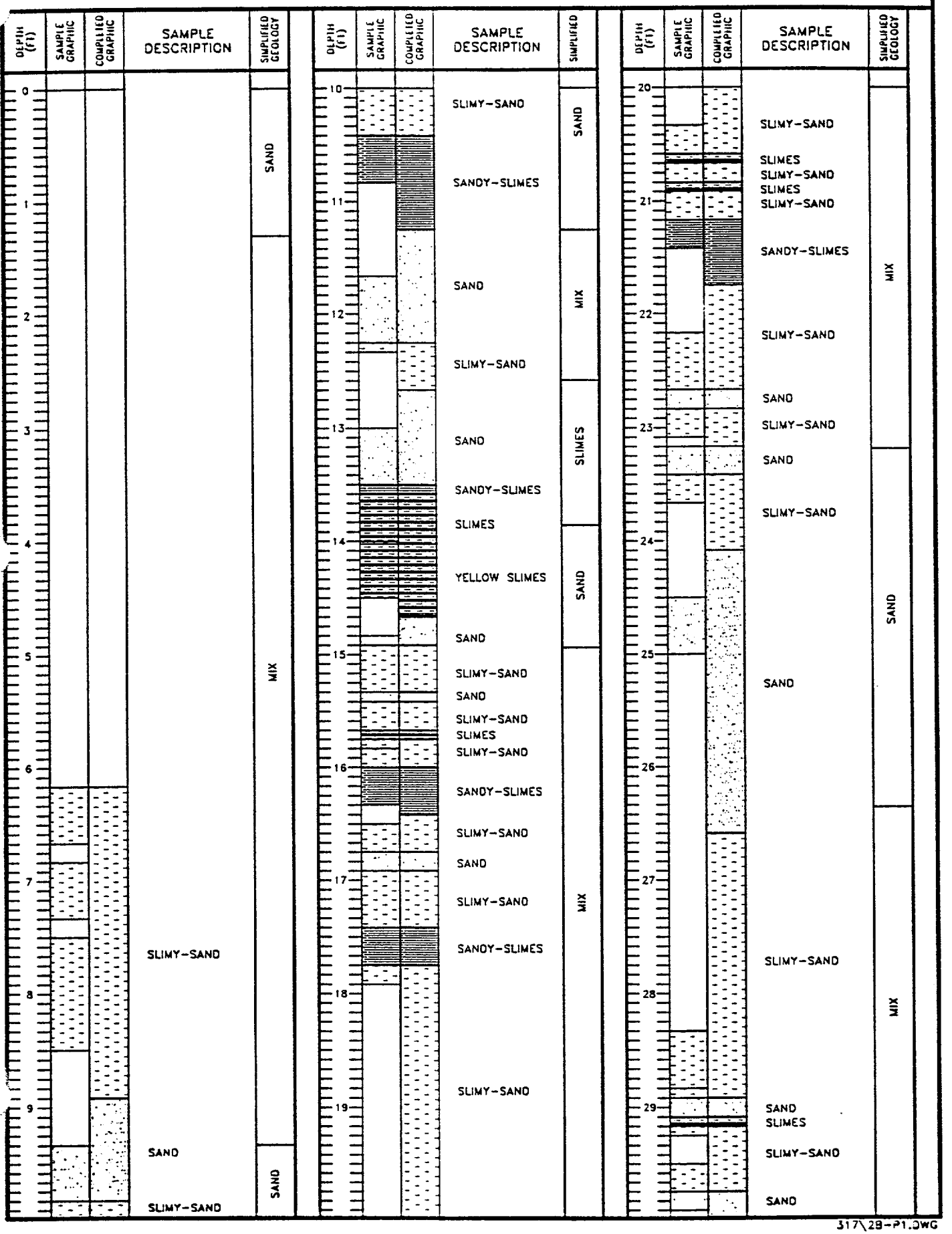
317\18-P3.DWG

L.C-86

WELL COMPLETION LOG

MONITORING WELL NO: 2B

LOGGED BY: JGC



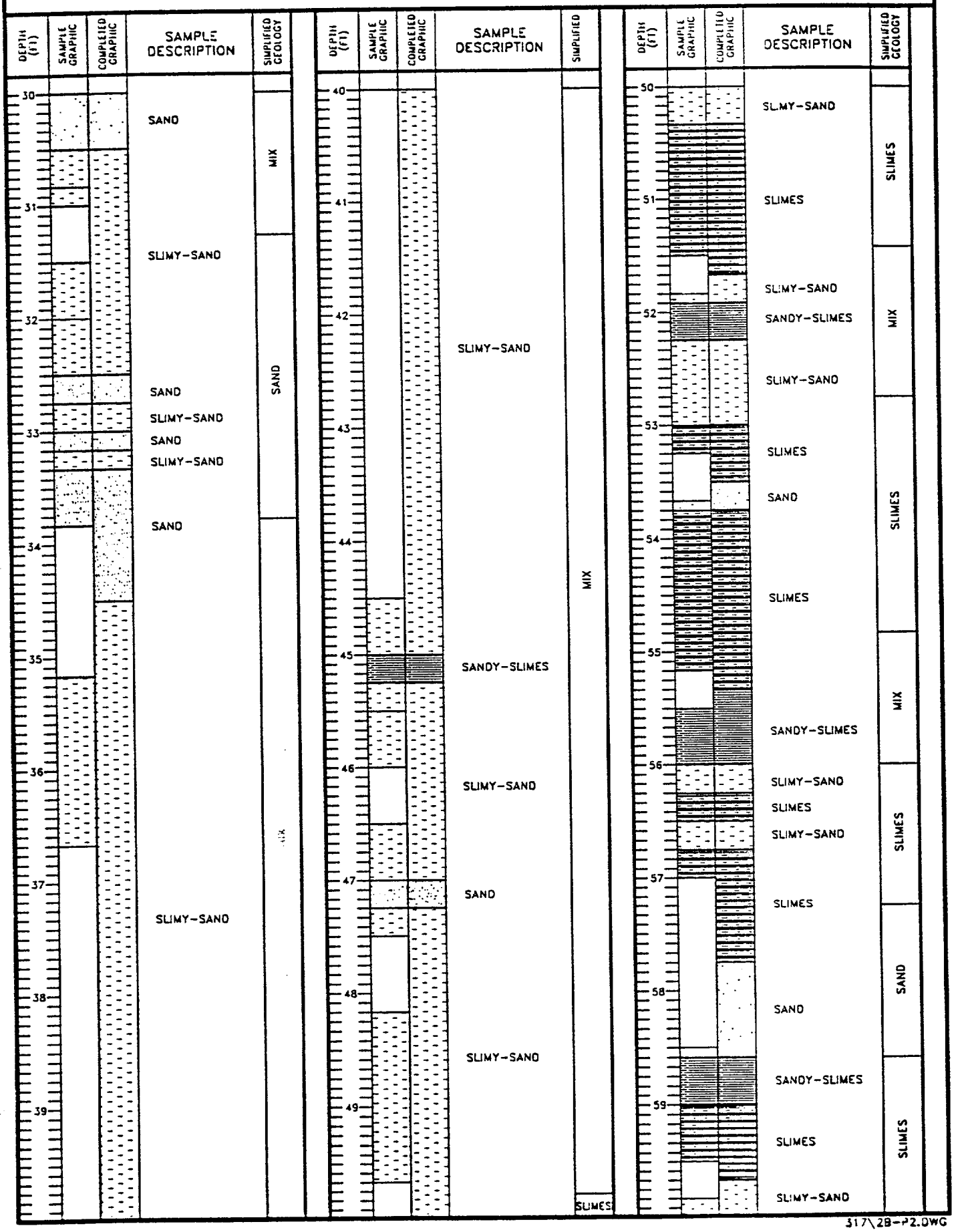
317\28-P1.DWG

L.C-87

WELL COMPLETION LOG

MONITORING WELL NO: 2B

LOGGED BY: JGC



317\2B-P2.DWG

L.C-88

WELL COMPLETION LOG

MONITORING WELL NO: 2B

LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60				SLIMES
			SLIMES	
			SLIMY-SAND	
61				
62				MIX
			SANDY-SLIMES	
63				
64				
65				SLIMES
			SLIMES	
66				
			SANDY-SLIMES SLIMY-SAND	
67			SANDY-SLIMES	MIX
68			SLIMY-SAND	
69			SLIMES	SLIMES
				SAND

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				
71				MIX
72				
73				SLIMES
74				
75				
76				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

317\2B-P3.DWG

L.C-89

WELL COMPLETION LOG

MONITORING WELL NO: 3B

LOGGED BY: JGC

DEPTH (FT)	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
0		SAND
1		MIX
2		
3		
4		
5		
6		
7	SLIMY-SAND	
8		
9		
10	SLIMY-SAND	MIX
	SANDY-SLIMES	
11	SLIMES	
	SLIMY-SAND	
12	SLIMES	SLIMES
	SLIMY-SAND	
13	SLIMES	
	SANDY-SLIMES	MIX
	SLIMY-SAND	
14	YELLOW SLIMES	
	SLIMY-SAND	
15	SAND	
	SANDY-SLIMES	
	SLIMY-SAND	
16	SANDY-SLIMES	
	SLIMY-SAND	SLIMES
17		
	SLIMES	MIX
18	SLIMY-SAND	
	SAND	
	SLIMY-SAND	
19	SLIMES	
	SLIMY-SAND	
	SLIMES	
	SLIMY-SAND	
20	SLIMY-SAND	SAND
21	SAND	
22		MIX
23	SLIMY-SAND	
24		
25	SAND	
26	SLIMY-SAND	
	SAND	
	SLIMY-SAND	SAND
27		
	SAND	
28		MIX
29	SLIMY-SAND	

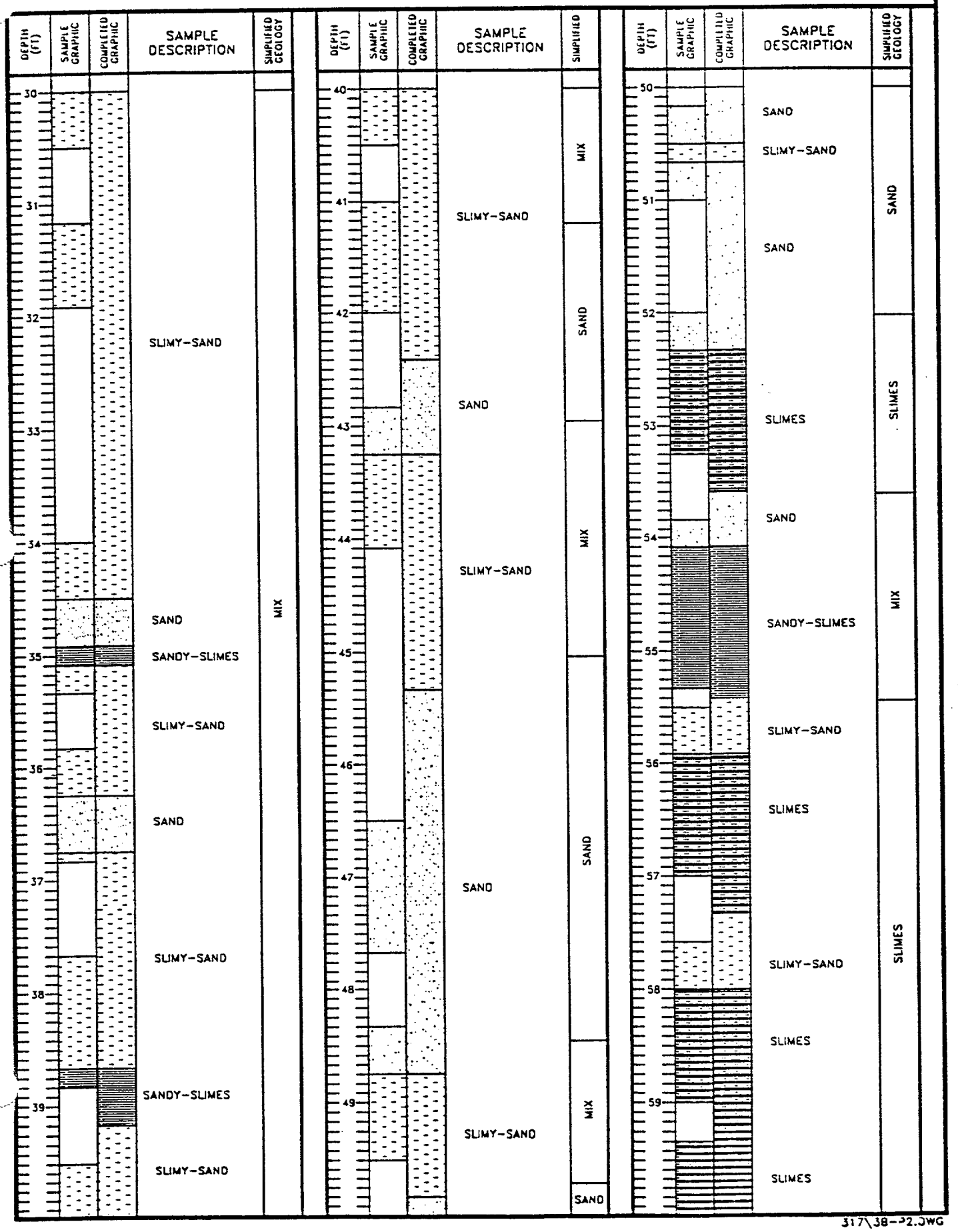
317\38-P1.DWG

L.C-90

WELL COMPLETION LOG

MONITORING WELL NO: 3B

LOGGED BY: JGC



317\38-P2.DWG

L.C-91

WELL COMPLETION LOG

MONITORING WELL NO: 3B

LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60				
				SLIMES
			SLIMES	SAND
61				
			SAND	
62				
			SANDY-SLIMES	MIX
63				
			SLIMES	
			SLIMY-SAND	
64				
				SLIMES
			SLIMES	
65				
			SAND	
66				
			SANDY-SLIMES	MIX
67				
			SLIMY-SAND	
68				
				SAND
69				
				MIX

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				
				MIX
71				
72				
73				SLIMES
74				
75				
76				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

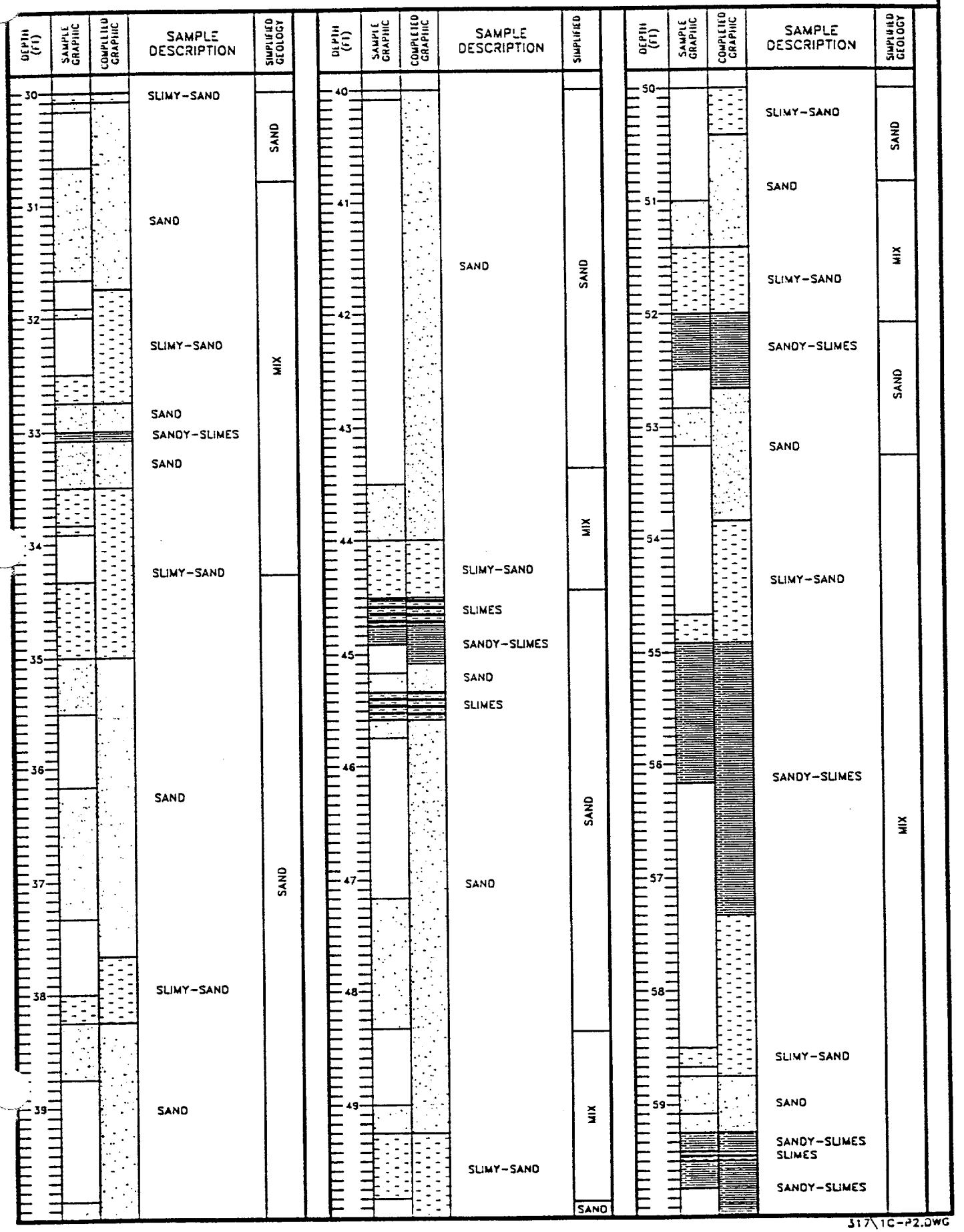
317\3B-P3.DWG

L.C-92

WELL COMPLETION LOG

MONITORING WELL NO: 1C

LOGGED BY: JGC



317\1C-P2.DWG

L.C-93

WELL COMPLETION LOG

MONITORING WELL NO: 1C

LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60				MIX
			SLIMY-SAND	SLIMES
61			SANDY-SLIMES	
			SLIMES	MIX
62				
			SLIMY-SAND	
63				
			SLIMES	
64				
			SLIMY-SAND	
			SLIMES	
65				
			SLIMY-SAND	SLIMES
66				
				SAND
			SLIMES	MIX
67				
			SANDY-SLIMES	
68				
			SLIMES	
69			SANDY-SLIMES	

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				MIX
71				
72				
73				
74				SLIMES
75				
76				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

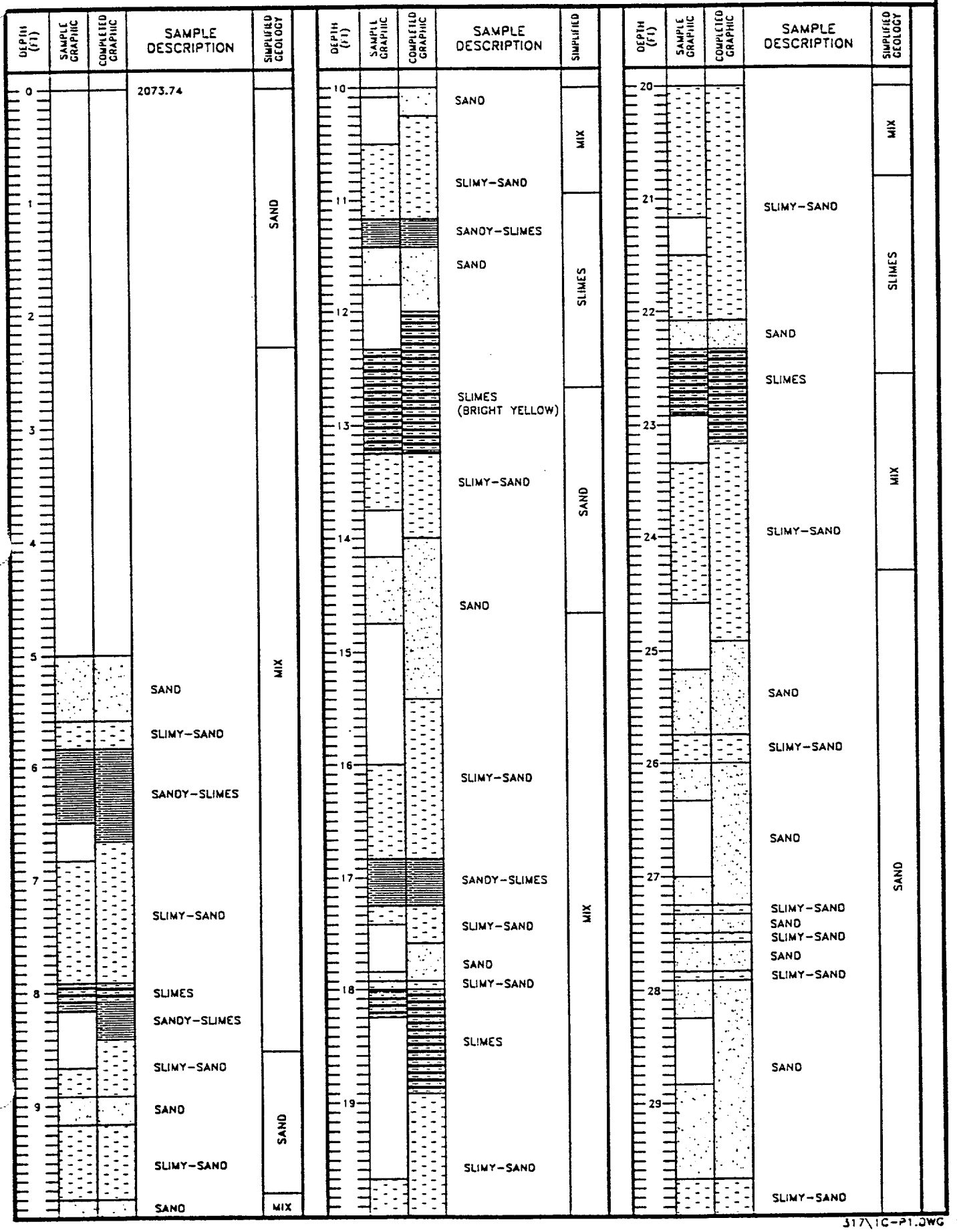
317\1C-P3.DWG

L.C-94

WELL COMPLETION LOG

MONITORING WELL NO: 1C

LOGGED BY: JGC

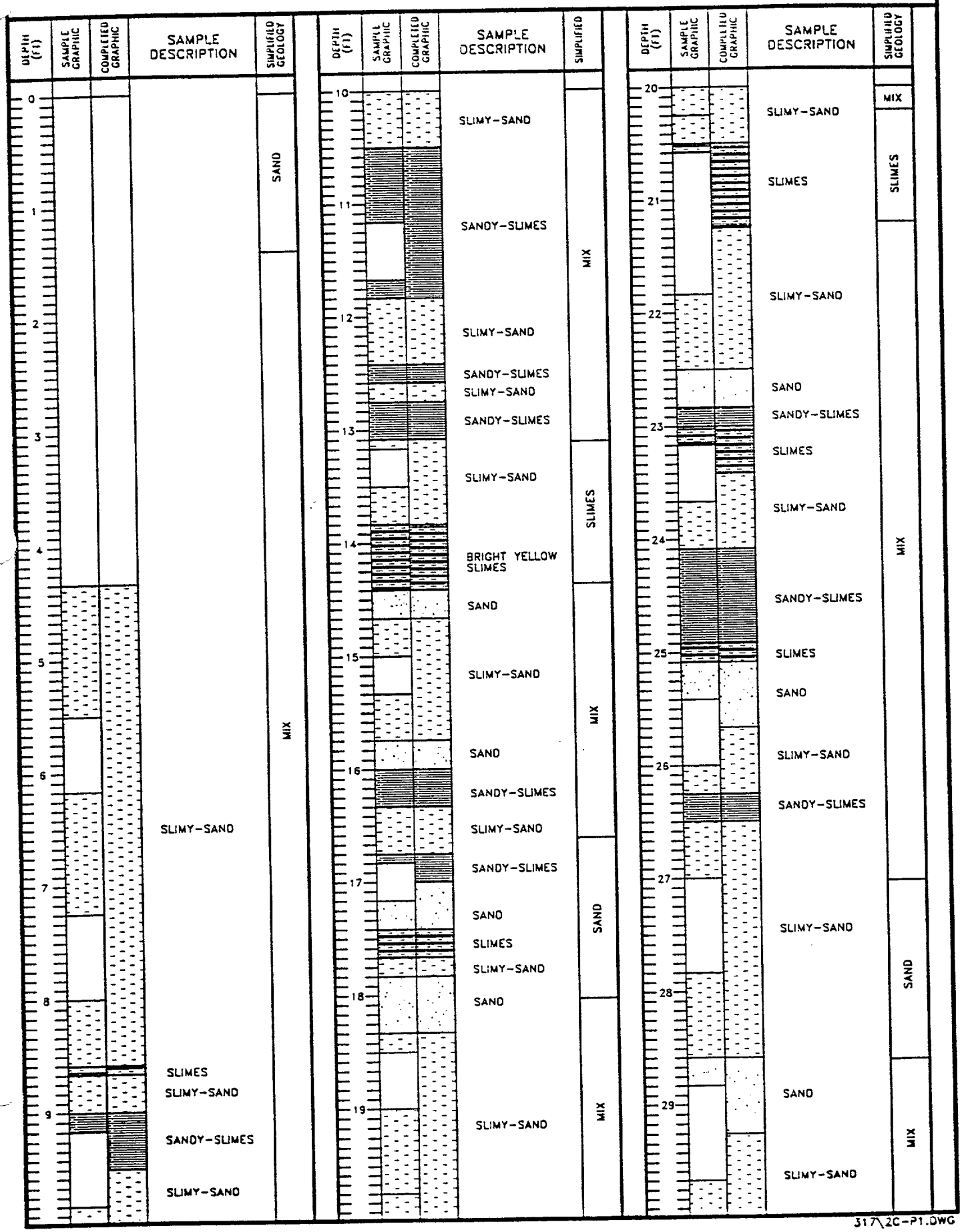


L.C-95

WELL COMPLETION LOG

MONITORING WELL NO: 2C

LOGGED BY: JGC



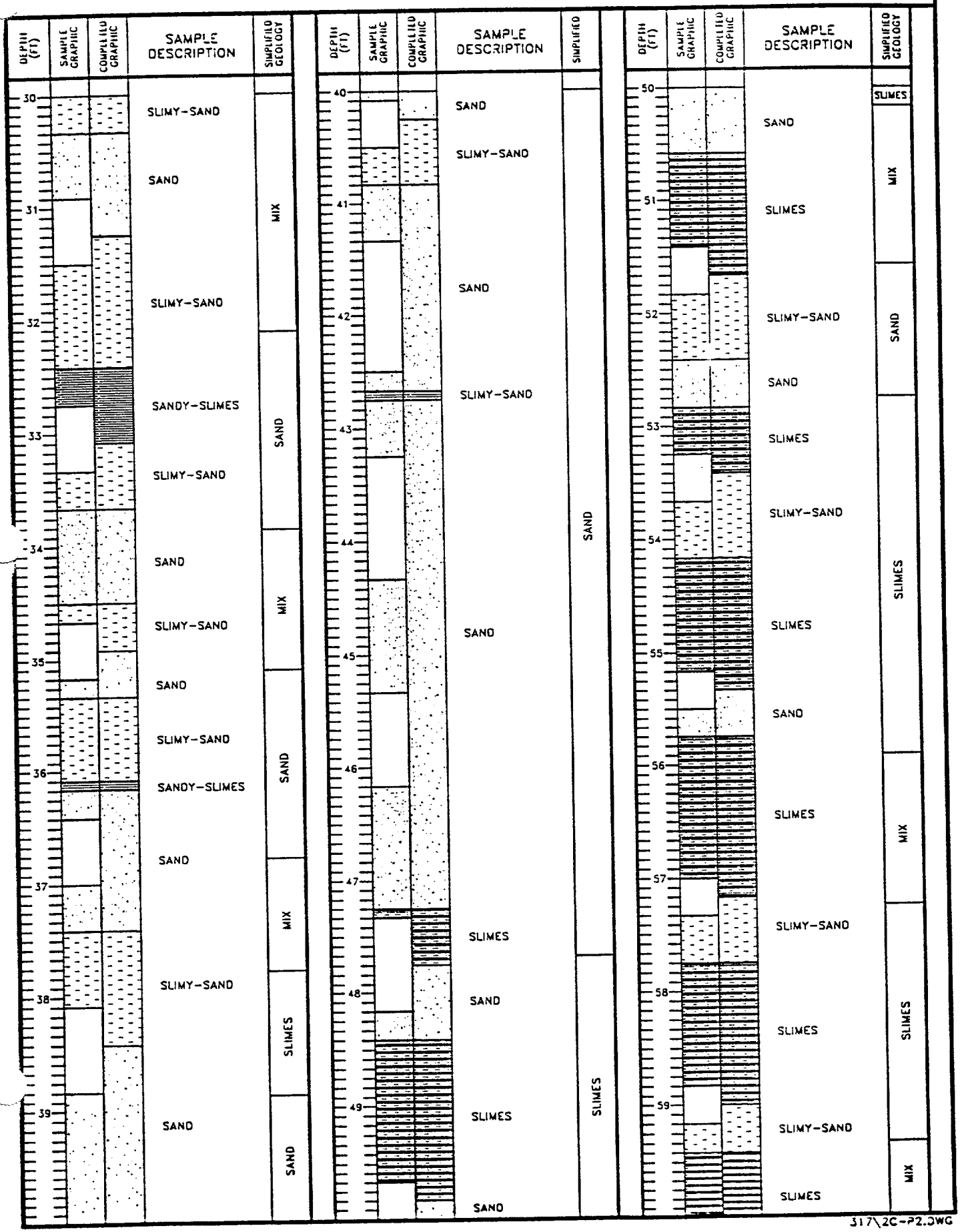
317\2C-P1.DWG

L.C-96

WELL COMPLETION LOG

MONITORING WELL NO: 2C

LOGGED BY: JGC



317\2C-P2.DWG

L.C-97

WELL COMPLETION LOG

MONITORING WELL NO: 2C

LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60			SLIMES	MIX
			SANDY-SLIMES	
61				
			SLIMES	SLIMES
62				
			SANDY-SLIMES	MIX
63				
			SLIMES	
64				
			SANDY-SLIMES	SAND
			SLIMY-SAND	
65				
			SAND	SLIMES
			SLIMES	
66				
			SAND	
67				MIX
			SLIMES	
			SANDY-SLIMES	SAND
68			SLIMES	
			SANDY-SLIMES	MIX
69				
			SLIMES	

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				MIX
71				SLIMES
72				
73				
74				
75				
76				
77				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

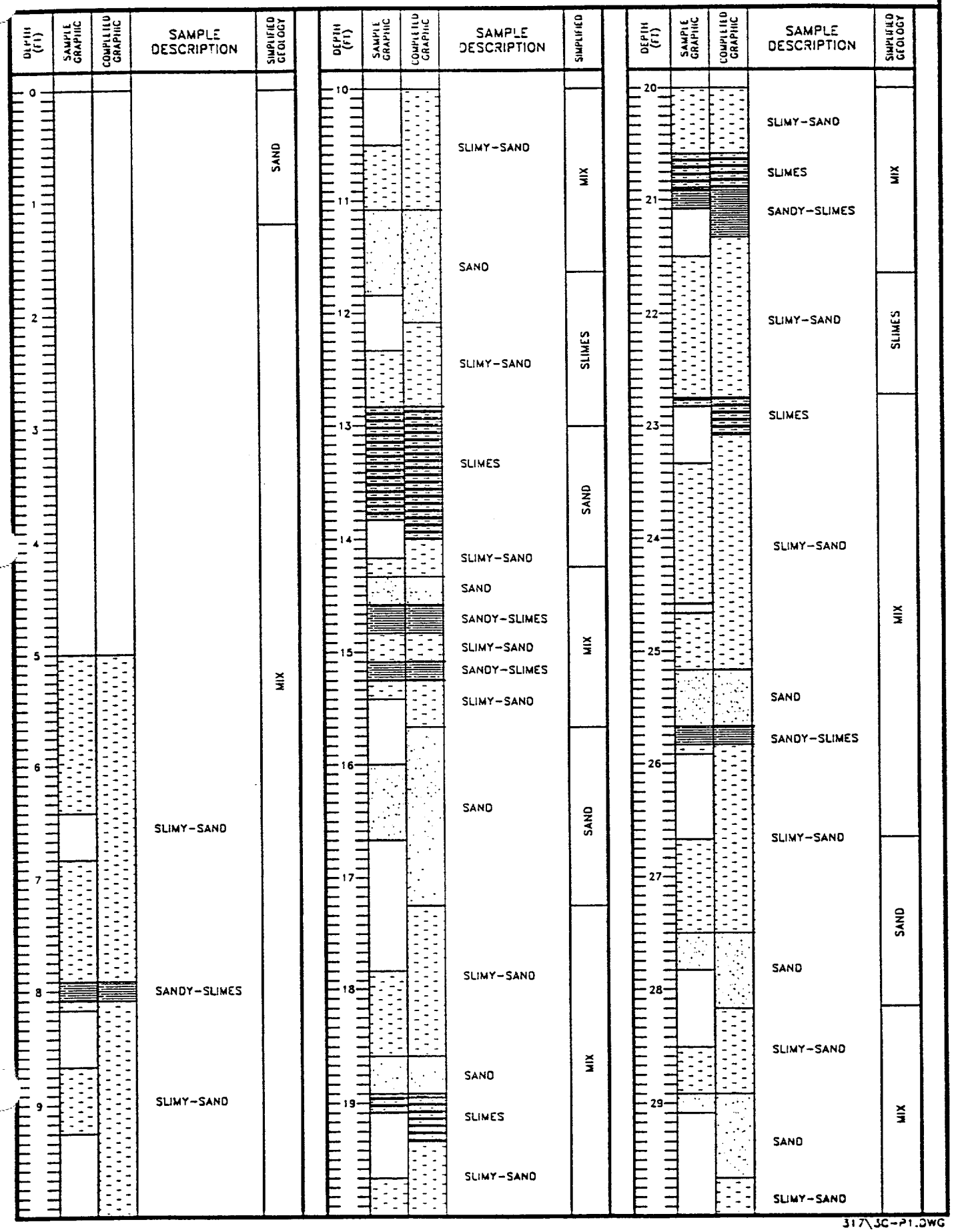
317\2C-P3.DWG

L.C-98

WELL COMPLETION LOG

MONITORING WELL NO: 3C

LOGGED BY: JGC



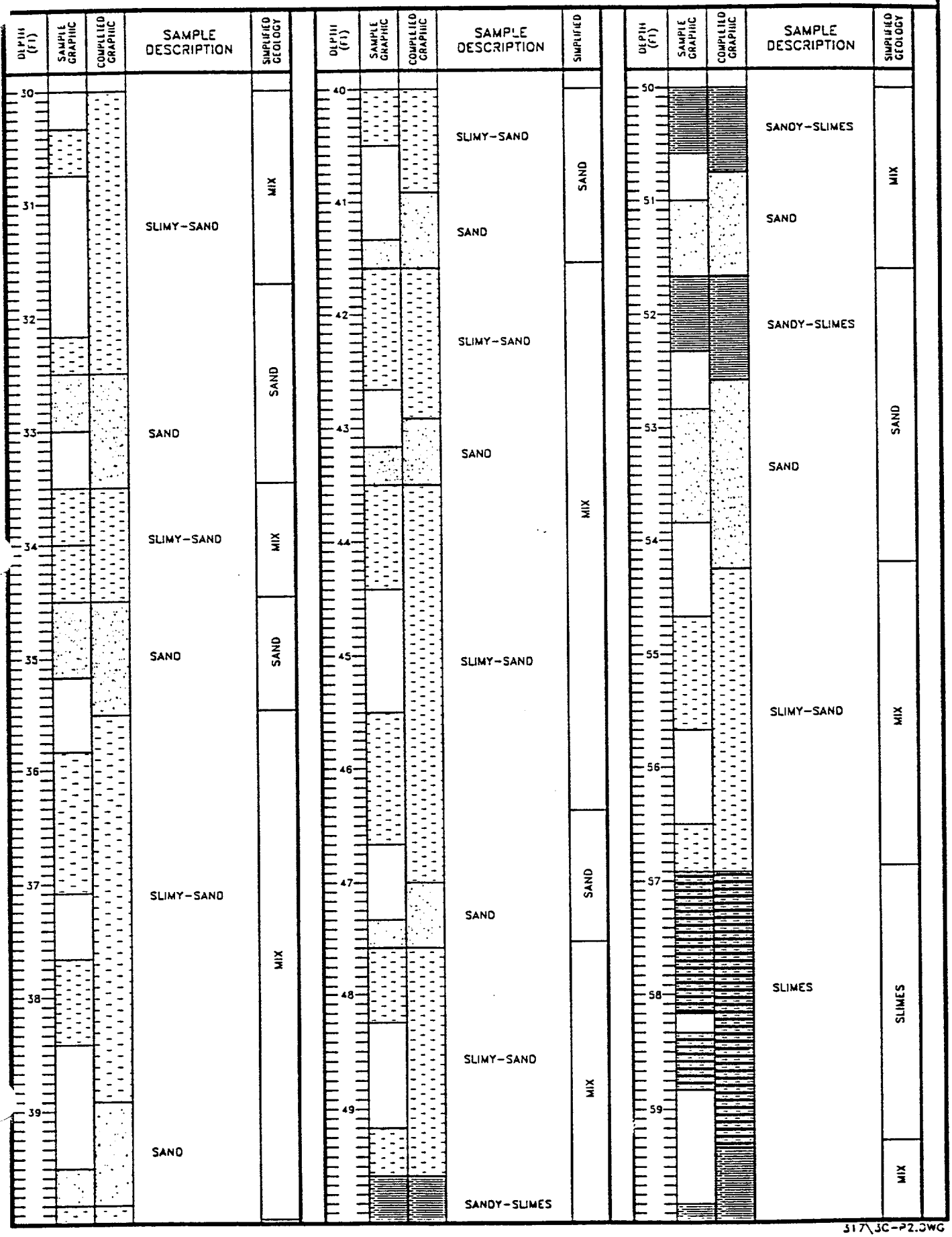
317\3C-P1.DWG

L.C-99

WELL COMPLETION LOG

MONITORING WELL NO: 3C

LOGGED BY: JGC



317\3C-P2.DWG

L.C-100

WELL COMPLETION LOG

MONITORING WELL NO: 3C

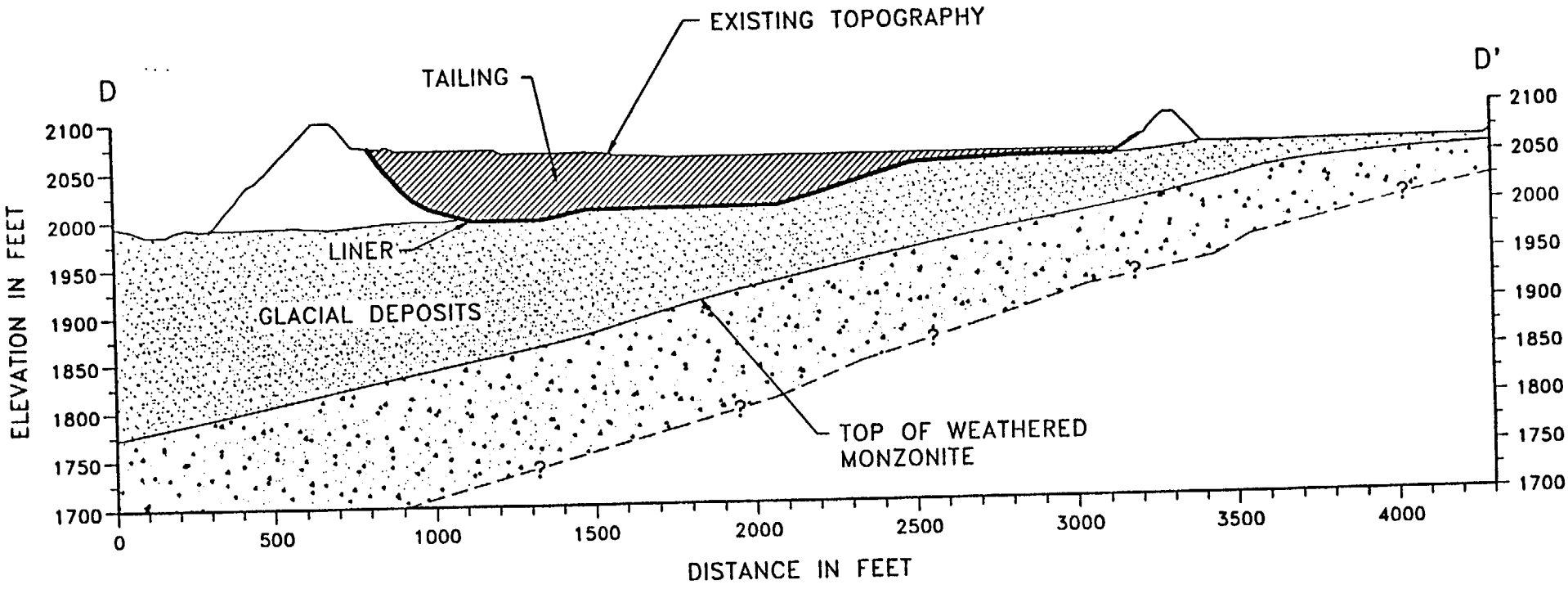
LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60				
61			SANDY-SLIMES	MIX
62				
63			SLIMES	
			SLIMY-SAND	
64			SLIMES	SLIMES
65				
			SANDY-SLIMES	
66				
67				MIX
			SLIMES	
68				
			SANDY-SLIMES	
69				
			SLIMES	

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				
71				MIX
72				
73				SLIMES
74				
75				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

317\3C-P3.DWG



SCALE: V:H = 3:1

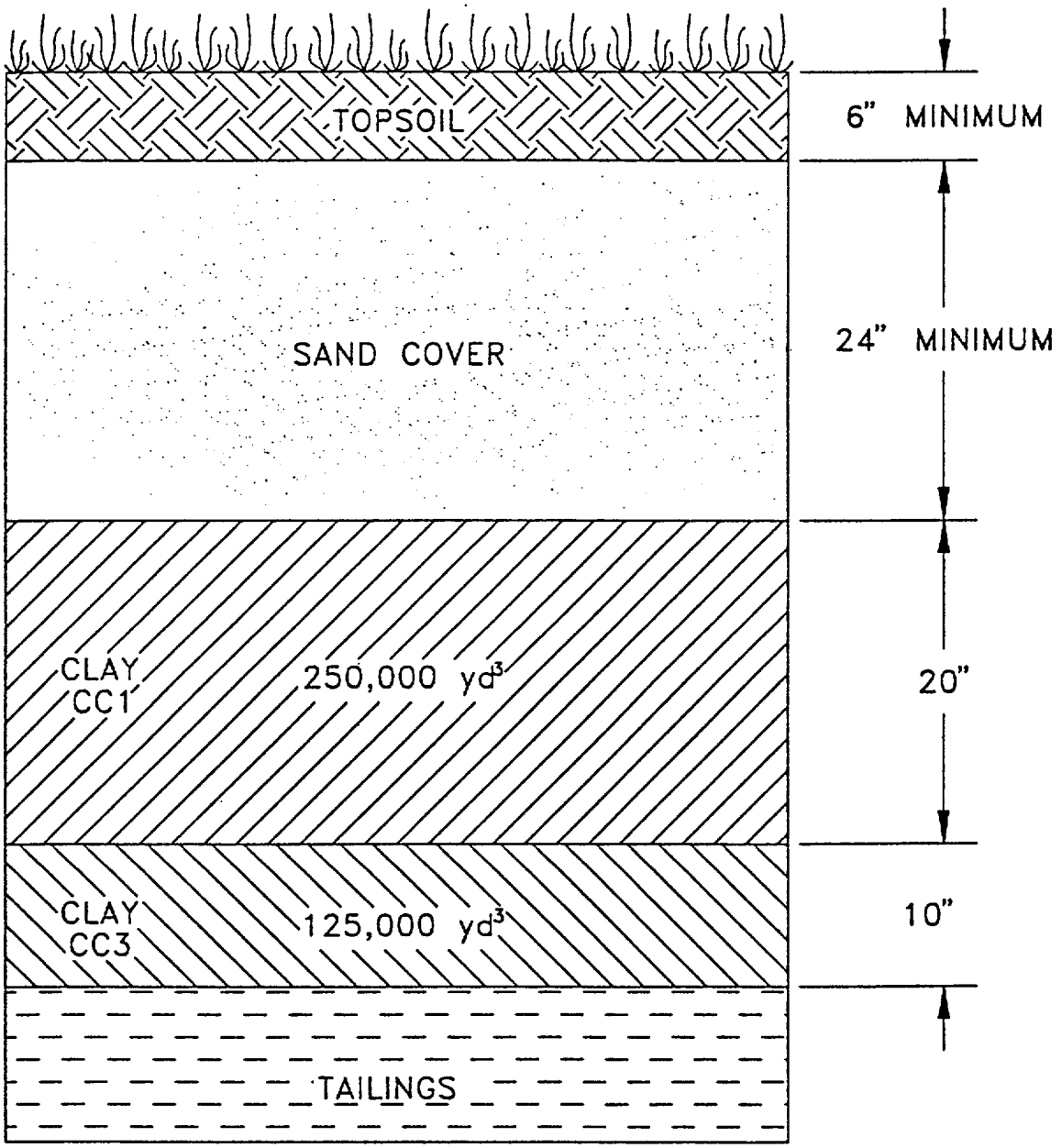
L.C-101

SMI
SHEPHERD MILLER, INC.

CROSS-SECTION D-D'

Date:	JAN., 1993
Project:	317
File:	XSECT-D

L.C-102



FILE : 317SLD

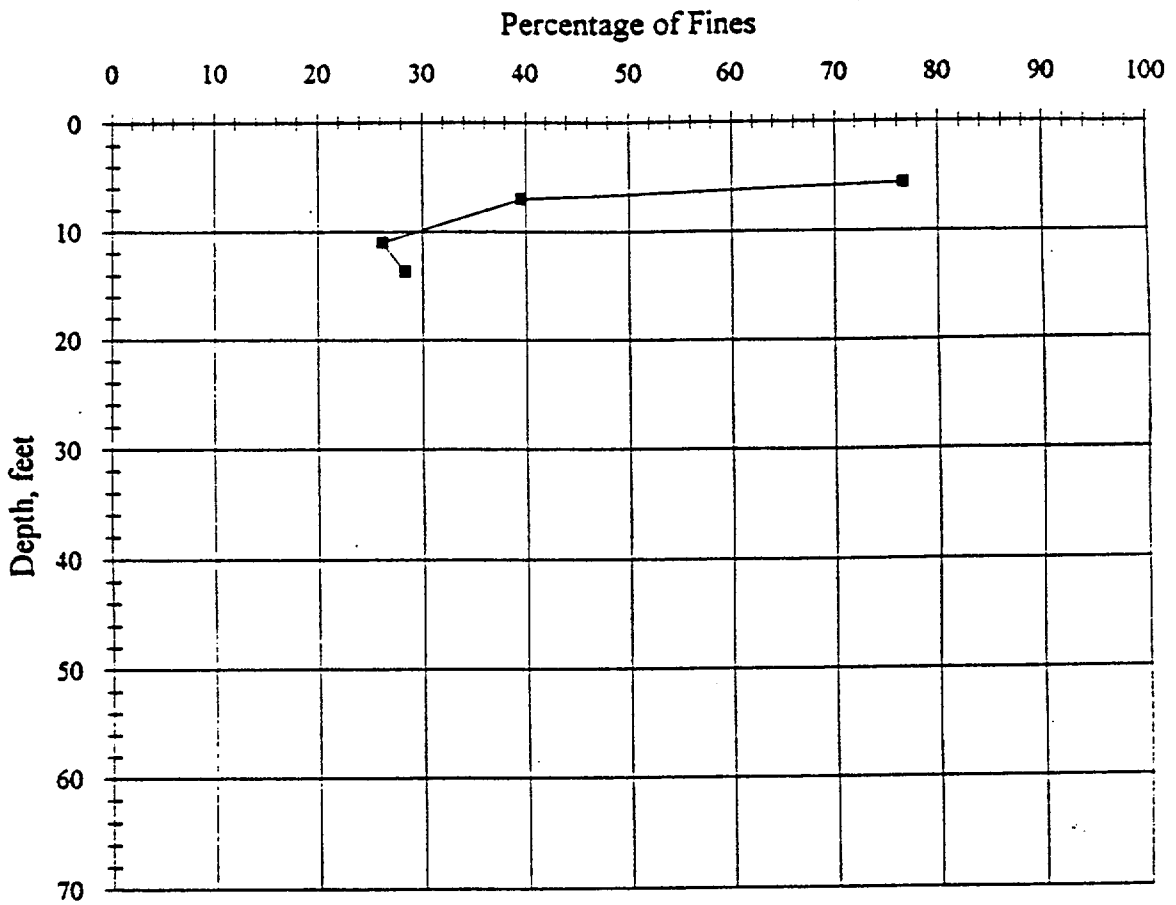
L.C-103

Summary of Laboratory Test Results From Thin-Wall Tube Samples

A	B	C	D	E	F	G	H	I
Boring	Depth, ft	Total Unit Weight, Pcf	Dry Unit Weight, pcf	Natural Moisture, %	Percent Pass No. 200	Void Ratio	Saturation, %	Soil Classification
T-2	18.0-18.5	116.1	88.9	30.6	34.9	0.86	0.94	SM
	18.5-19.0	122.4	100.5	21.7	14.8	0.65	0.89	SM
	21.5-22.0	104.7	67.5	55.1	45.6	1.45	1.00	SM
	22.0-22.5	112.7	85.6	31.6	31.9	0.93	0.90	SM
	22.5-23.0	109.2	76.6	42.6	40.5	1.16	0.97	SM
	23.0-23.5	112.3	77.5	44.9	42.8	1.13	1.00	SM
T-4	45.0-45.5	94.6	59.7	58.4	74.8	1.77	0.87	ML
	45.5-46.0	100.5	57	76.2		1.90	1.00	ML
	46.0-46.5	105	67.7	55.1	70.2	1.44	1.00	ML
	46.5-47.0	107.3	72.8	47.4	61.2	1.27	0.99	ML
	58.0-58.5	113.5	74.5	52.4	73.4	1.22	1.00	ML
	58.5-59.0	111.6	76.1	46.7	70	1.17	1.00	ML
	59.0-59.5	110.6	82.9	33.4	23.9	0.99	0.89	SM
	59.5-60.0	113.8	85.7	32.8	13	0.93	0.94	SM
T-5	20.0-20.5	106.7	85.8	24.3	15.7	0.93	0.69	SM
	20.5-21.0	109.6	86.3	27	19.6	0.92	0.78	SM
	21.0-21.5	116.3	85.9	35.4	39	0.93	1.00	SM
	21.5-22.0	114.1	83.6	36.5	22.5	0.98	0.99	SM
T-7A	40.0-40.5	97.4	49.5	96.8	55.6	2.34	1.00	ML
	40.5-41.0	101.2	79.1	27.9	17.3	1.09	0.68	SM
	41.0-41.5	97.3	50.9	91.3	99.2	2.25	1.00	ML
	41.5-42.0	103.7	57	81.8		1.90	1.00	ML
T-8	26.0-26.5	99.9	56.3	77.6	98.1	1.94	1.00	ML
	26.5-27.0	99.6	55	81.2	69.6	2.01	1.00	ML
	27.0-27.5	92.2	44.3	107.9	91.4	2.73	1.00	ML
	27.5-28.0	117.1	92.3	117.1	13	0.79	1.00	SM

L.C-104

Boring T-1



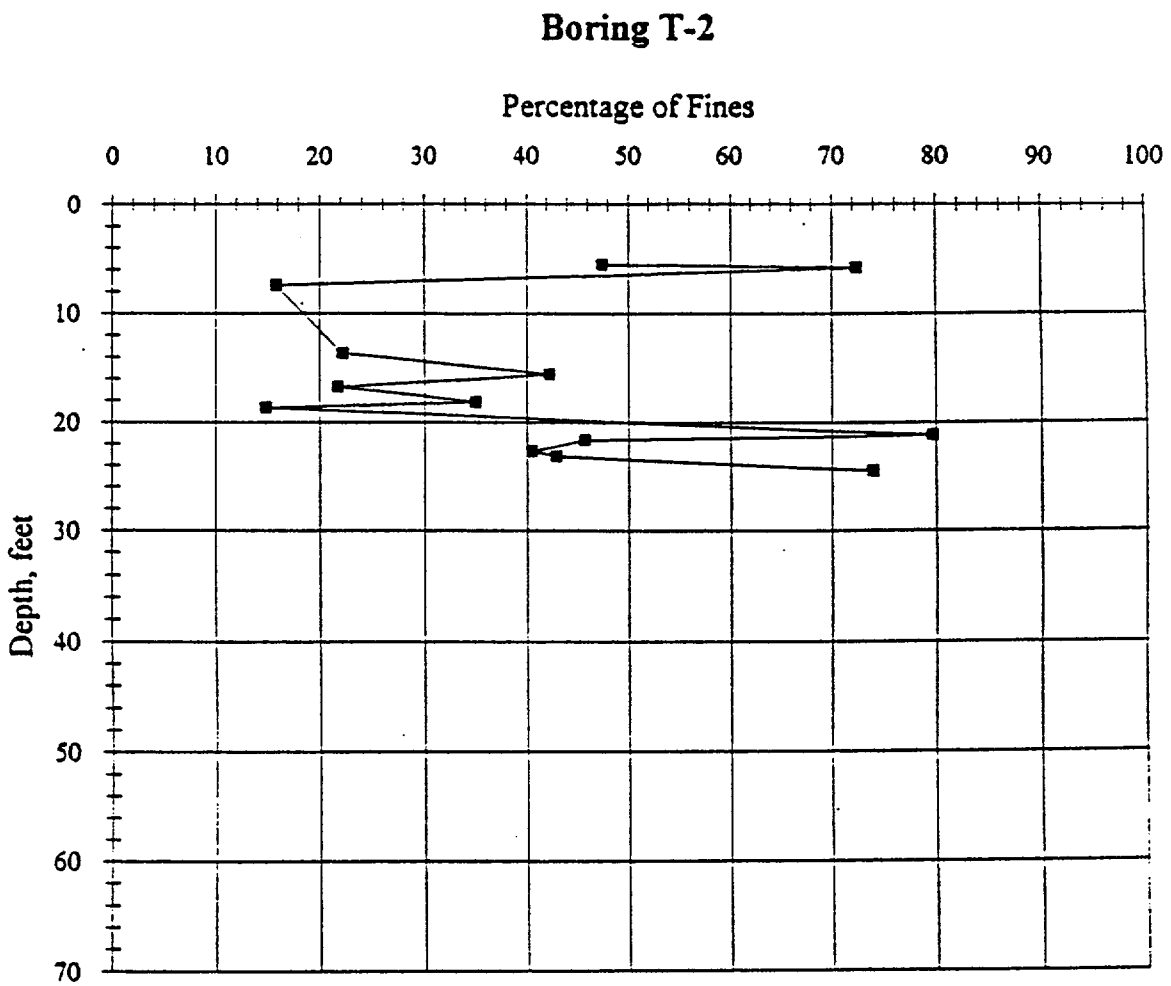
R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

FINES CONTENT BORING T-1

Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-1001	C-1

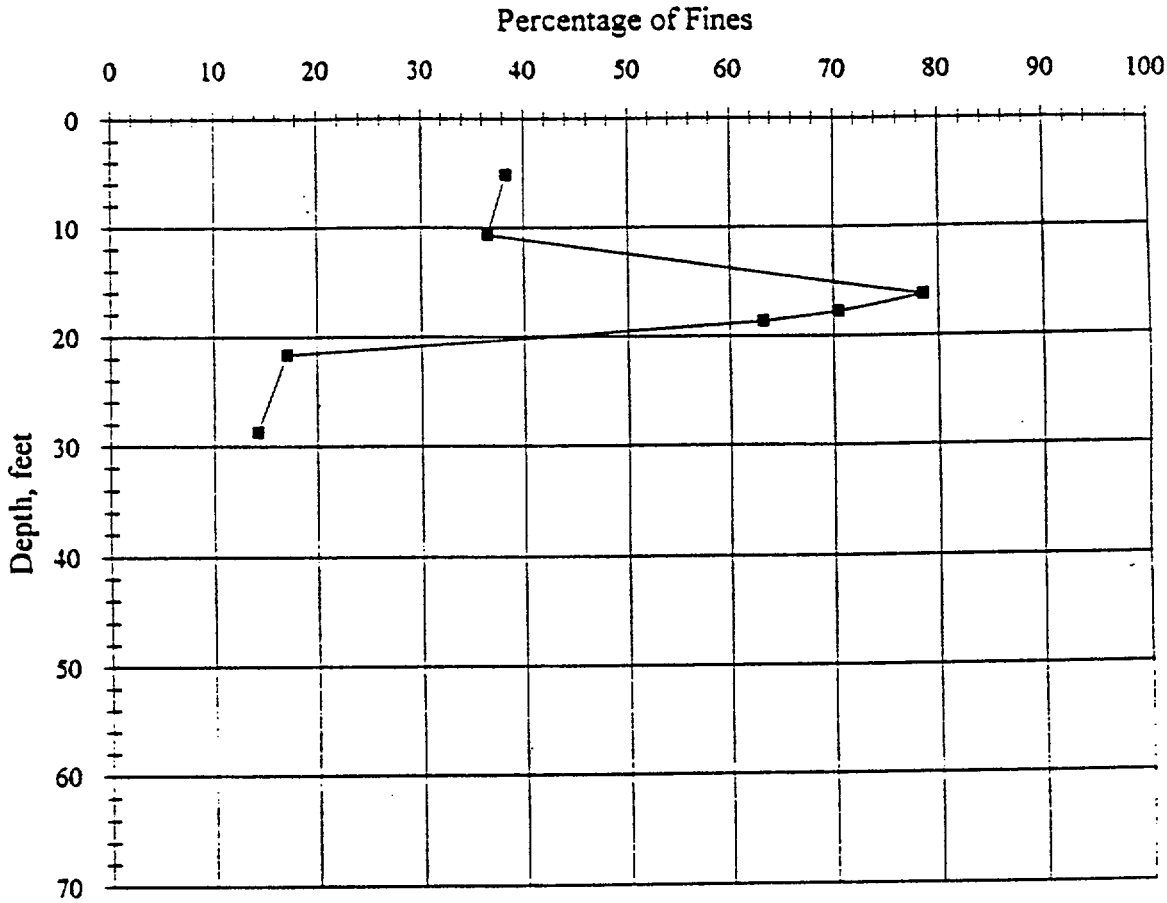
L.C-105



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT FINES CONTENT BORING T-2			
Checked by [illegible] Date 5-4-96 Approved by RLU Date 5-5-96	Project No. SMI-100	Figure No. C-2	

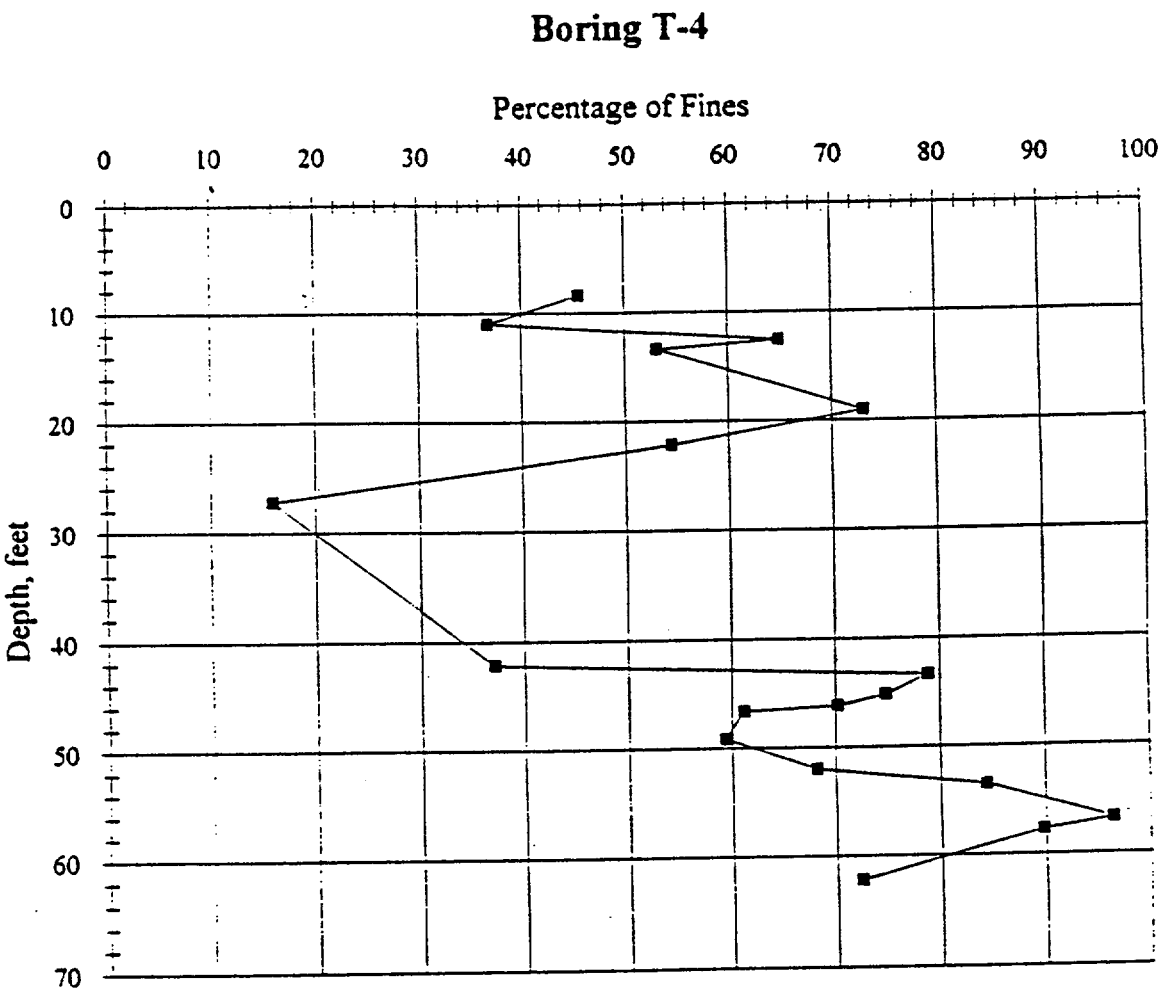
L.C-106

Boring T-3



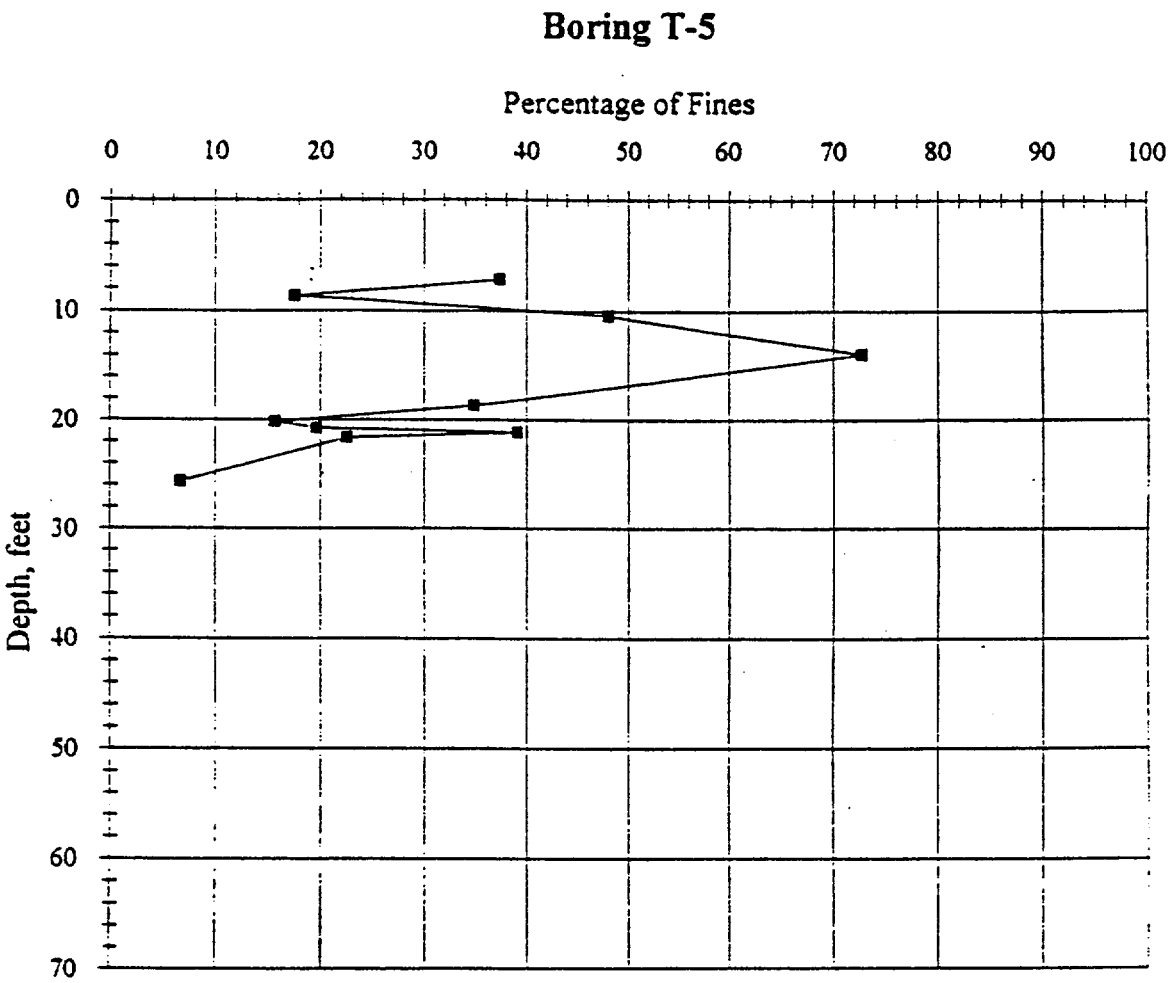
R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT **FINES CONTENT BORING T-3**			
Checked by [illegible] Date 5-5-94 Approved by RW Date 5-5-94	Project No. SMI-100	Figure No. C-3	

L.C-107



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT FINES CONTENT BORING T-4			
Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100	C-4

L.C-108



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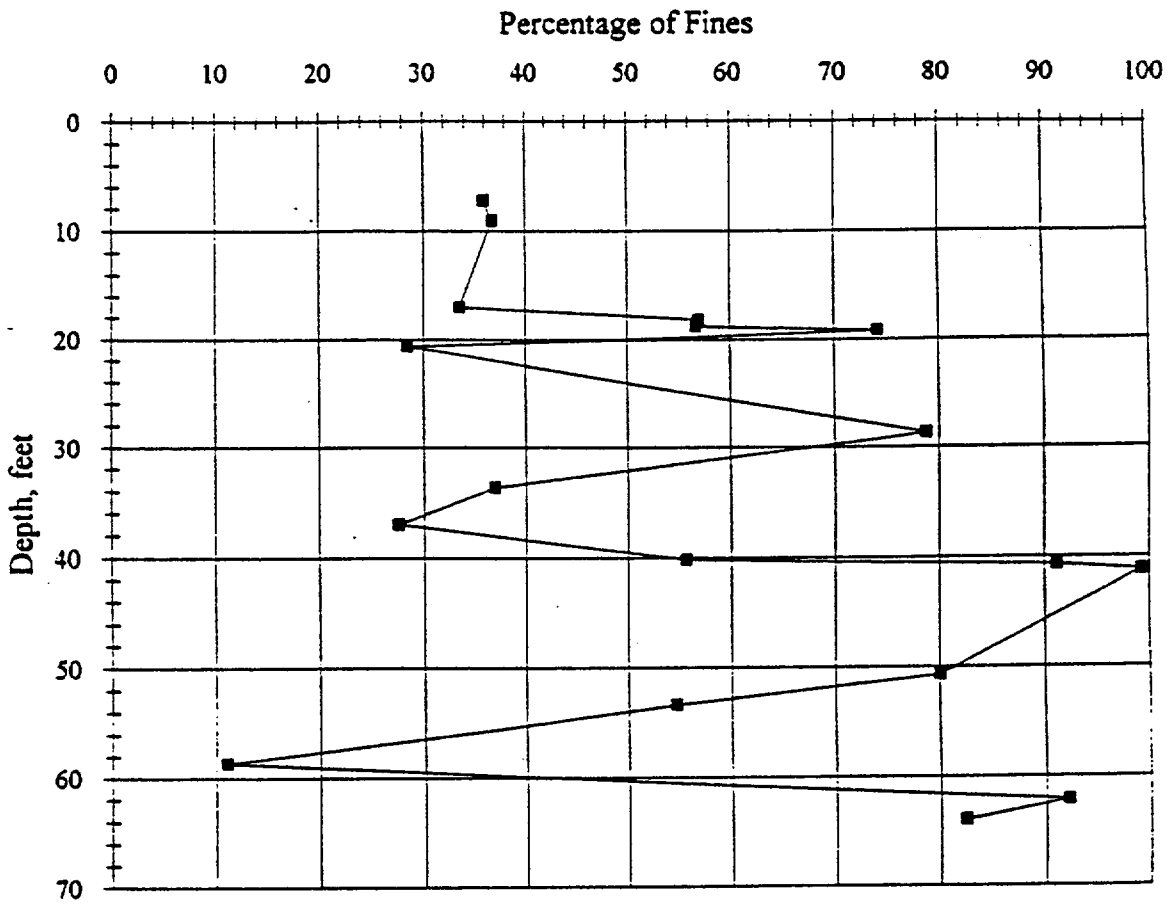
SHERWOOD TAILING IMPOUNDMENT

FINES CONTENT BORING T-5

Checked by [illegible] Date 5-4-94	Project No.	Figure No.
Approved by RW Date 5-5-94	SMI-100	C-5

L.C-109

Boring T-7



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Los Gatos, California

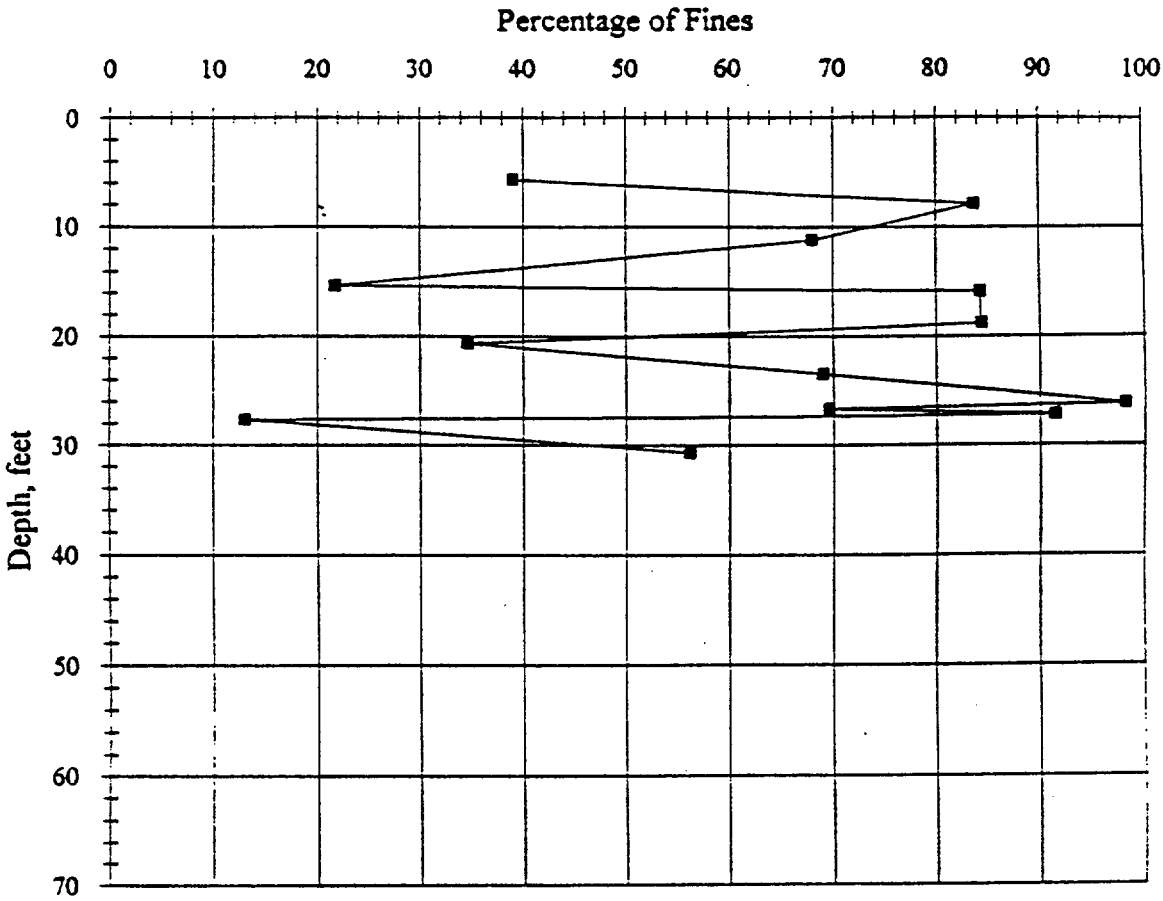
SHERWOOD TAILING IMPOUNDMENT

FINES CONTENT BORING T-7

Checked by [illegible] Date 5-4-94	Project No.	Figure No.
Approved by RLV Date 5-5-94	SMI-1001	C-6

L.C -110

Boring T-8



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SHERWOOD TAILING IMPOUNDMENT

FINES CONTENT BORING T-8

Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-1001	C-7

L.C-111

RLV-A

APPENDIX D

STANDARD PENETRATION TEST RESULTS AND $(N_1)_{60}$ VALUES vs. DEPTH

APPENDIX D

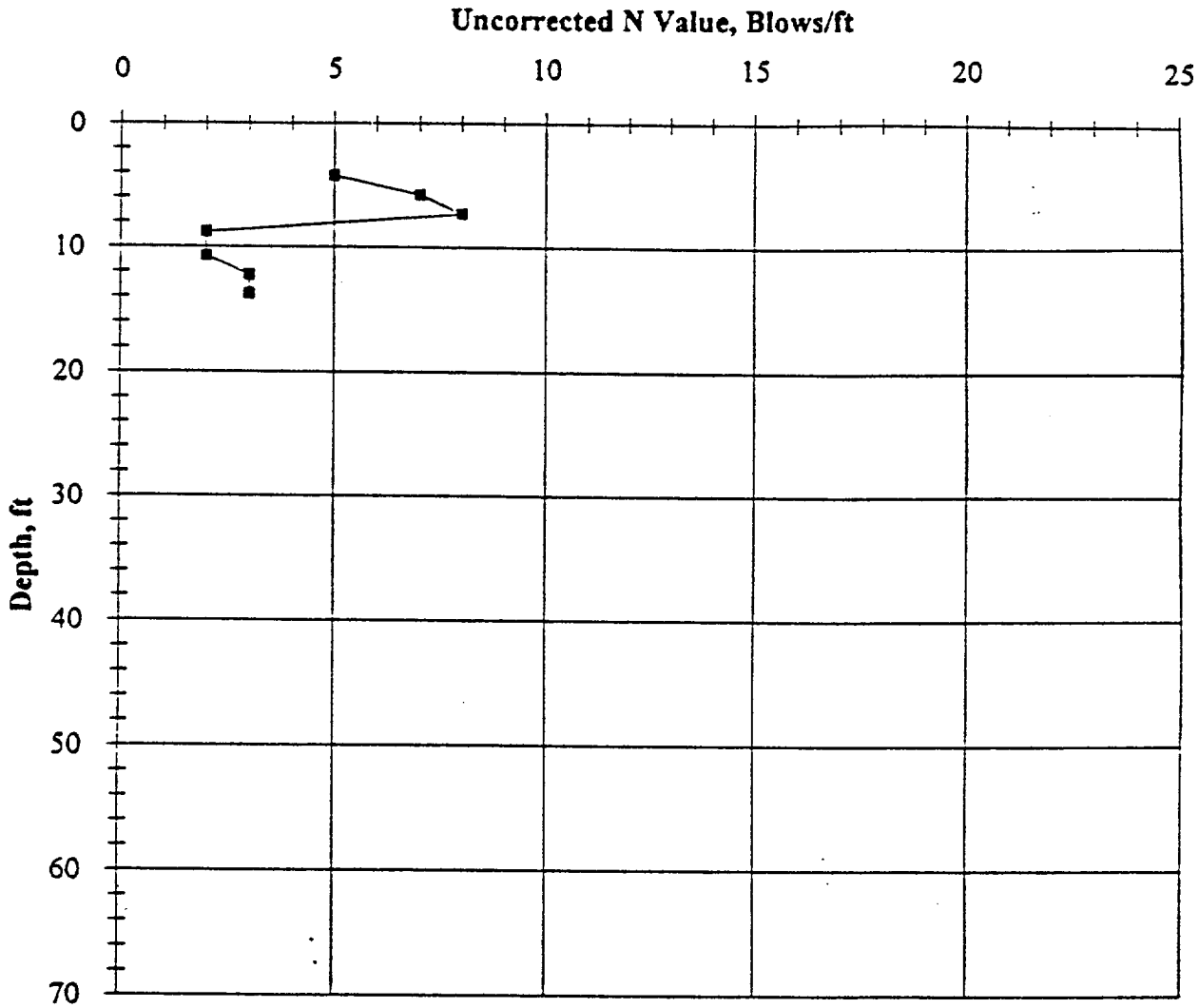
STANDARD PENETRATION TEST RESULTS AND $(N_1)_{60}$ VALUES vs. DEPTH

This appendix presents the results of Standard Penetration Test (SPT) results and $(N_1)_{60}$ values vs. depth for borings performed at the Sherwood Tailing Impoundment. The SPT results were obtained by SMI during their original field investigation of September 1991. These logs are not reproduced herein but can be found in Appendix A of the Tailing Reclamation Plan. The SPT results for Borings T-1 through T-10 are presented in Figs. D-1 through D-10, respectively. Following Figure D-10, there are six sheets which present the results of the engineering analyses used to compute the $(N_1)_{60}$ values as a function of depth. The analyses are direct copies of the computer spread sheets showing the correction factors applied to the original SPT values as discussed in Section IV of this report. Several of the sheets contain results from more than one boring. It should be noted that the effective stress acting at any depth represents the future effective stress that will be acting following construction of the protective cover over the tailing impoundment, as further discussed in Section III, and presented in Fig. 6.

The computed $(N_1)_{60}$ values vs. depth for Borings T-1 through T-10 are presented in Figs. D-11 through D-20, respectively. These values were then used to assess the liquefaction potential of the tailing impoundment. It should be noted that the plot of a particular $(N_1)_{60}$ value may be fractional (e.g. 12.4 blows/ft) since it is the result of a calculation. Although the data on the data for $(N_1)_{60}$ shown on the spread sheets has been rounded to the nearest whole number for presentation, the fractional data were used for plotting purposes. Detailed results of the liquefaction assessment are presented in Appendix E.

L.C-113

Boring T-1



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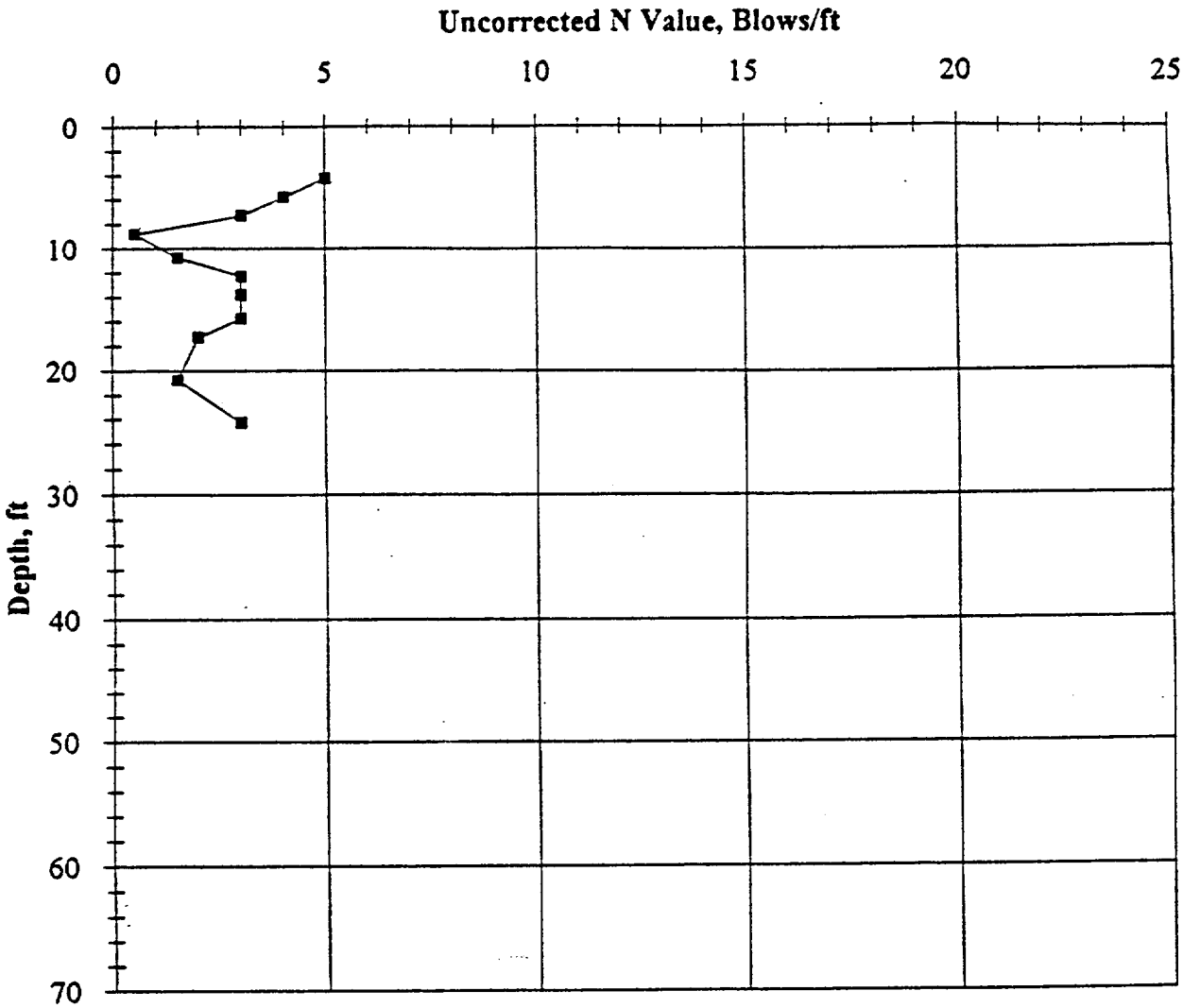
SHERWOOD TAILING IMPOUNDMENT

SPT RESULTS BORING T-1

Checked by [illegible] Date 5-4-94	Project No.	Figure No.
Approved by RLV Date 5-5-94	SMI-100	D-1

L.C-114

Boring T-2



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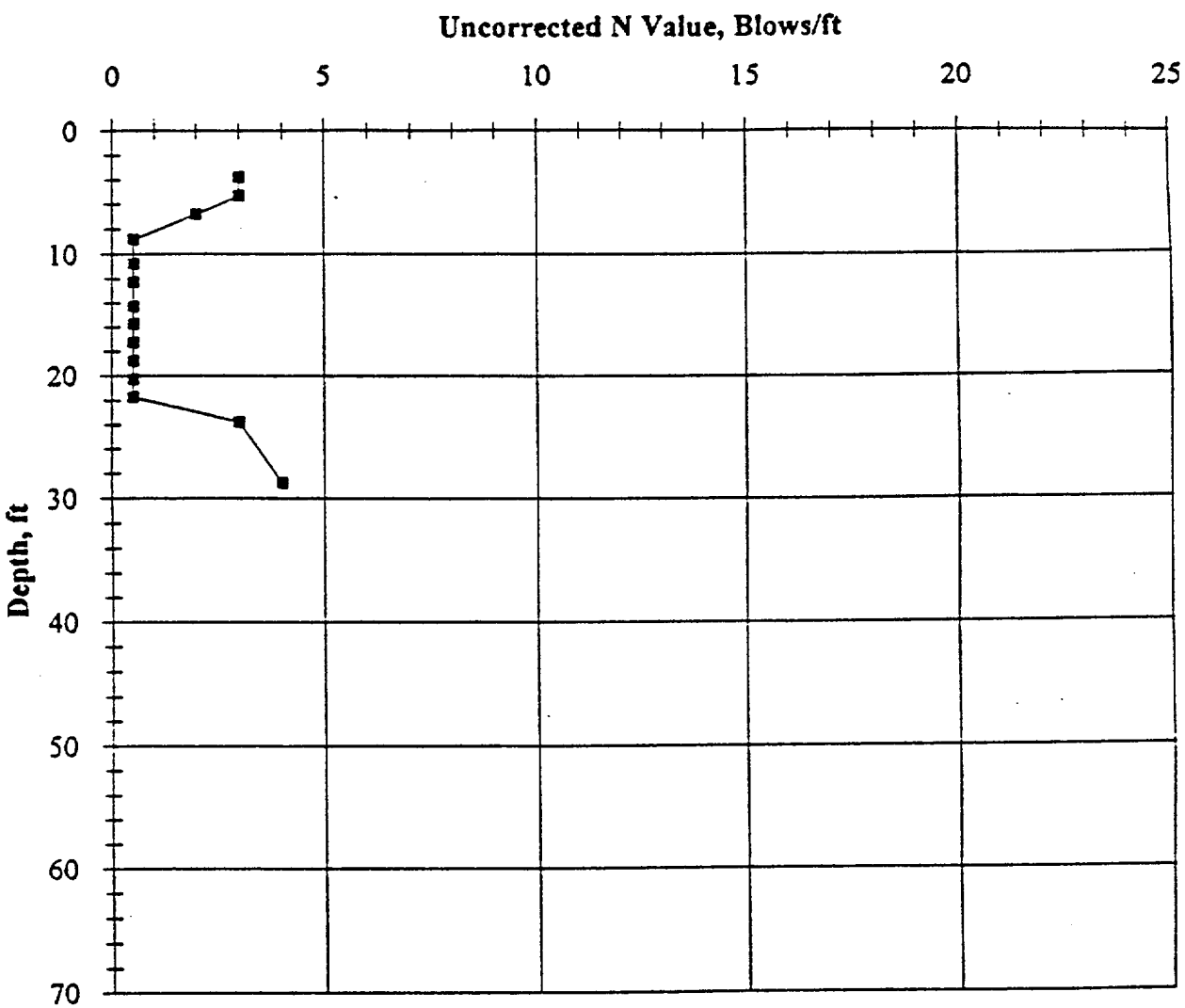
SHERWOOD TAILING IMPOUNDMENT

SPT RESULTS BORING T-2

Checked by [illegible] Date 5-4-94	Project No.	Figure No.
Approved by RLV Date 5-5-94	SMI-100	D-2

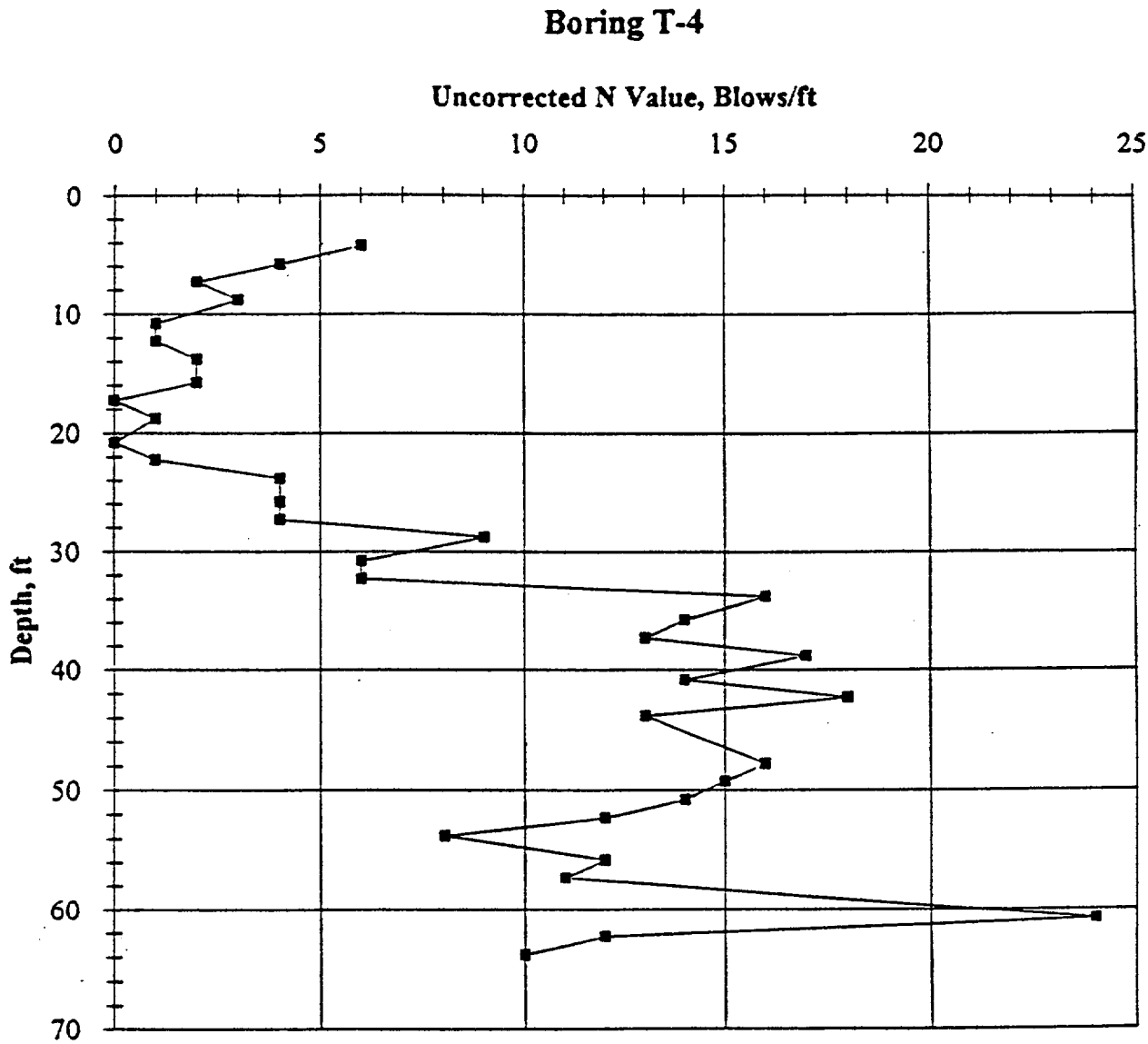
L.C-115

Boring T-3



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT SPT RESULTS BORING T-3			
Checked by lw Approved by RLV	Date 5-4-94 Date 5-5-94	Project No. SMI-100	Figure No. D-3

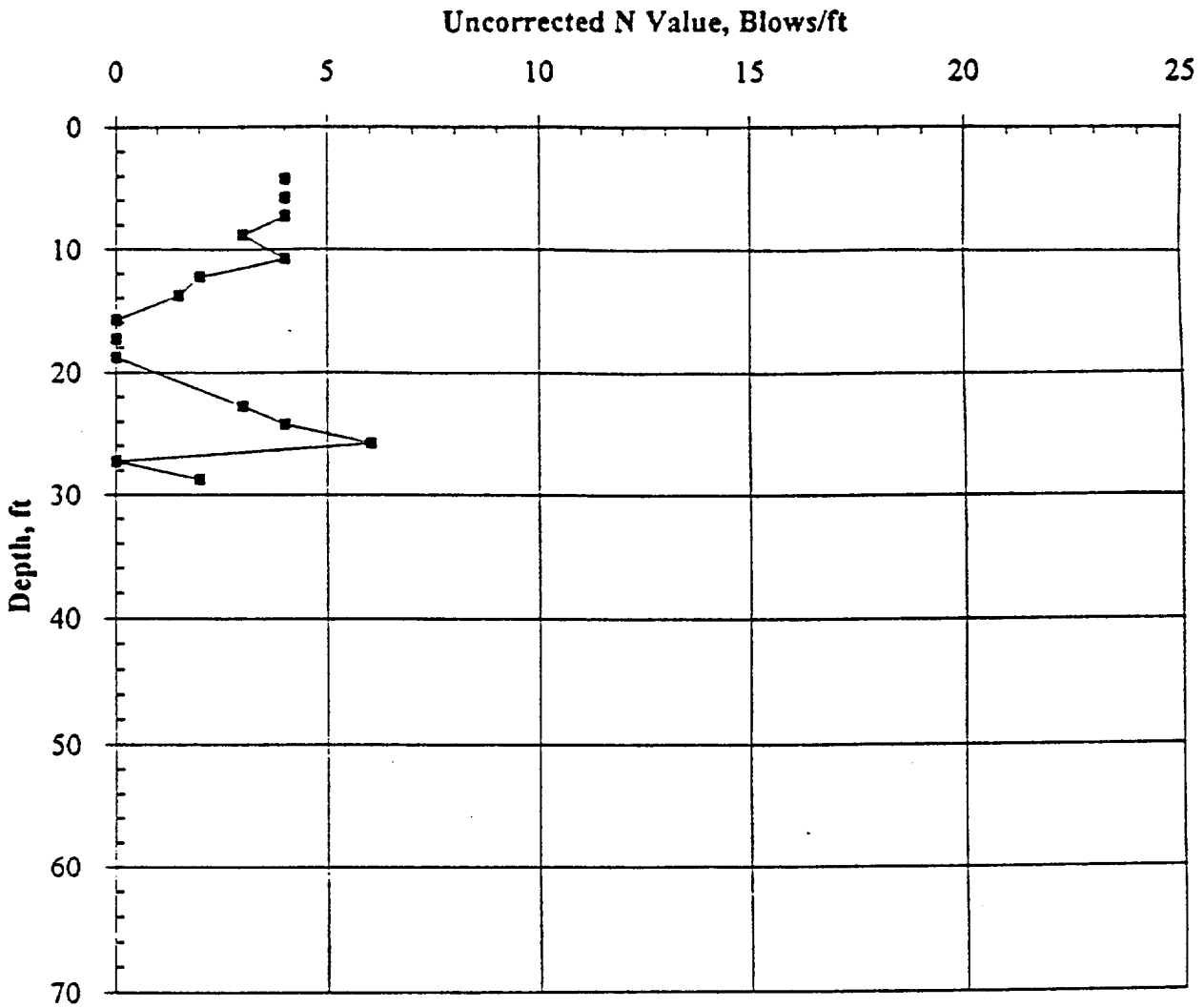
L.C-116



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT SPT RESULTS BORING T-4			
Checked by [illegible] Date 5-4-94 Approved by RLV Date 5-5-94	Project No. SMI-100	Figure No. D-4	

L.C-117

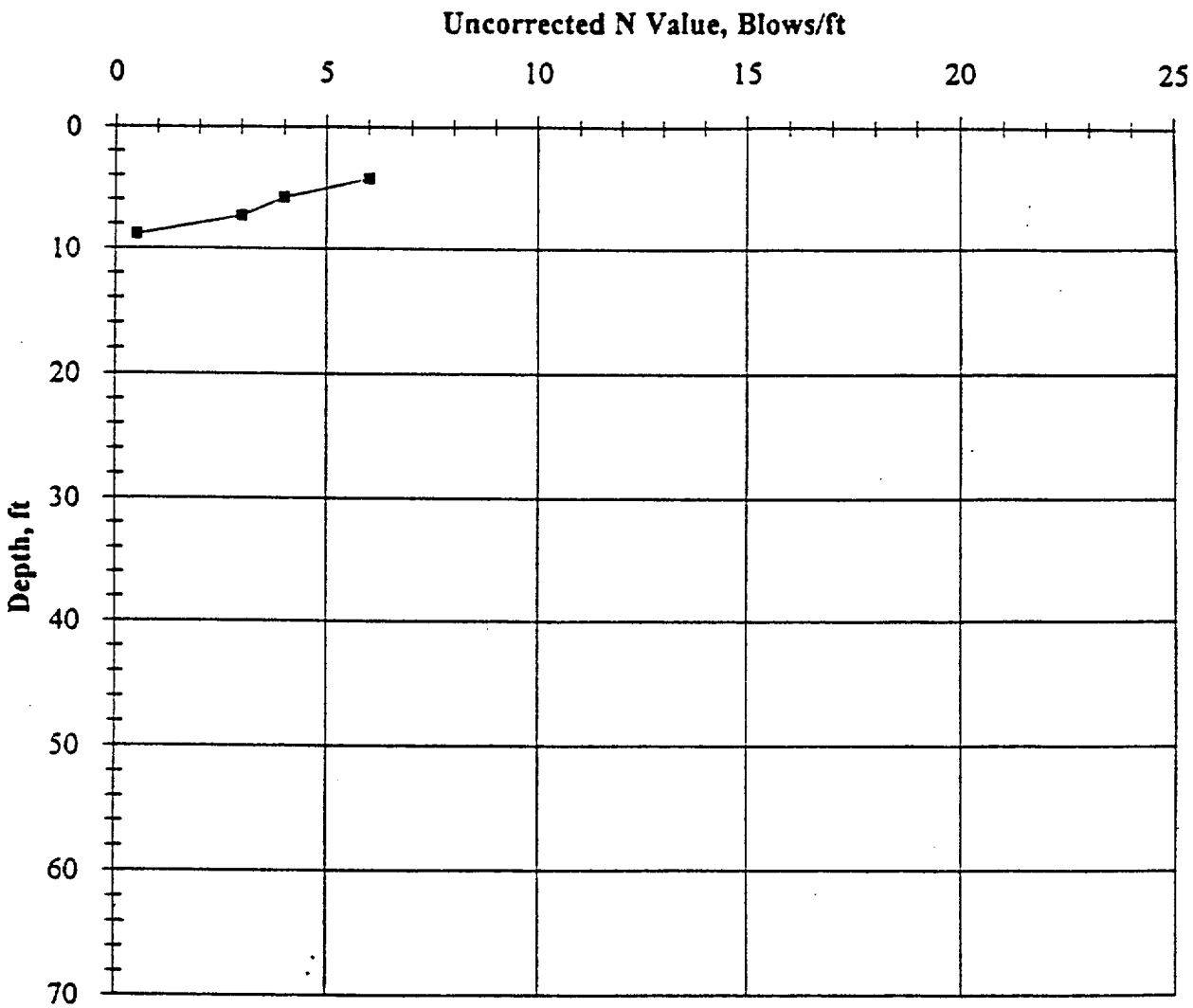
Boring T-5



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT SPT RESULTS BORING T-5			
Checked by lw	Date 5-4-94	Project No.	Figure No.
Approved by RW	Date 5-5-94	SMI-100	D-5

L.C-118

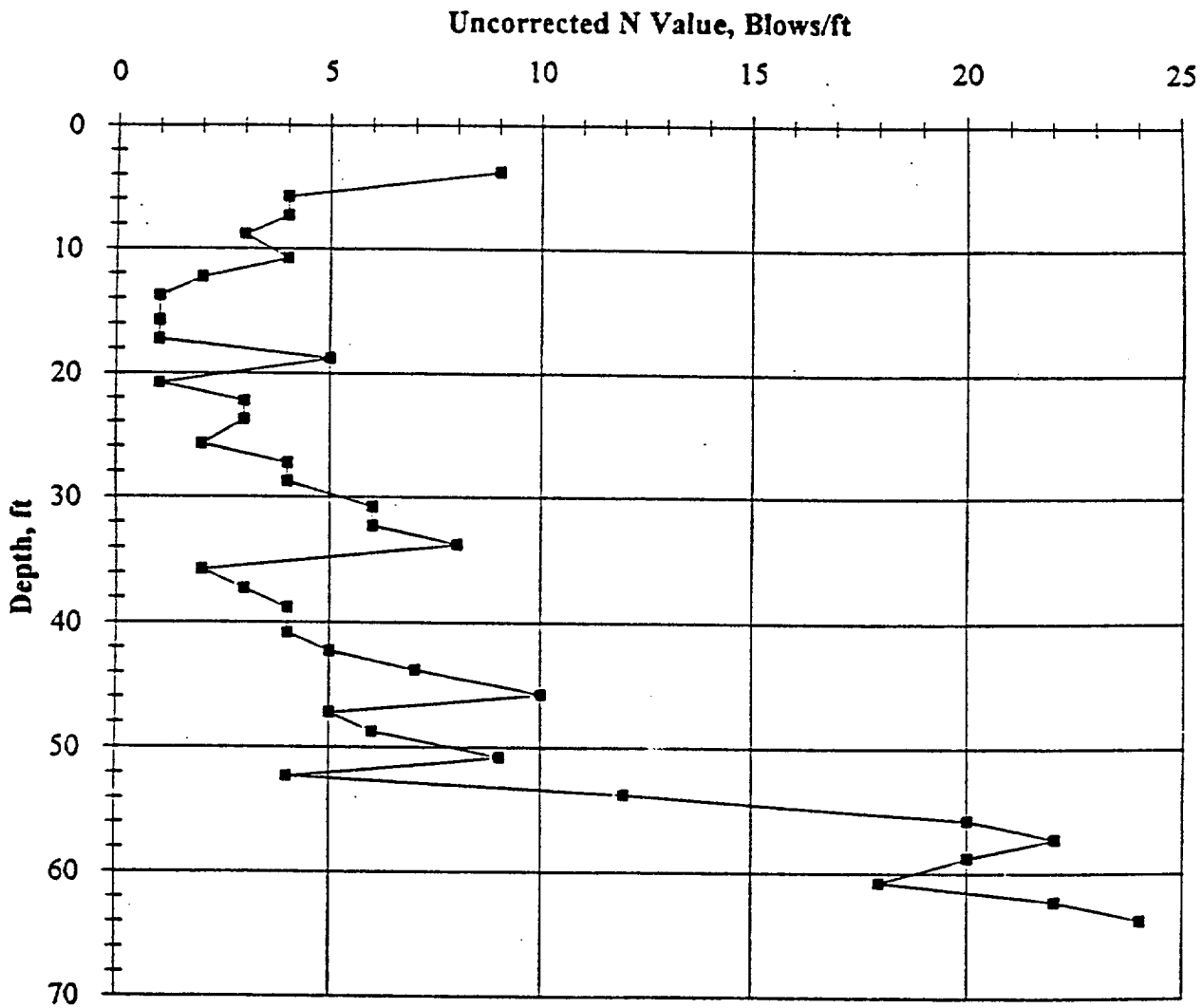
Boring T-6



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT SPT RESULTS BORING T-6			
Checked by Clv Approved by RLV	Date 5-4-94 Date 5-5-94	Project No. SMI-100	Figure No. D-6

L.C-119

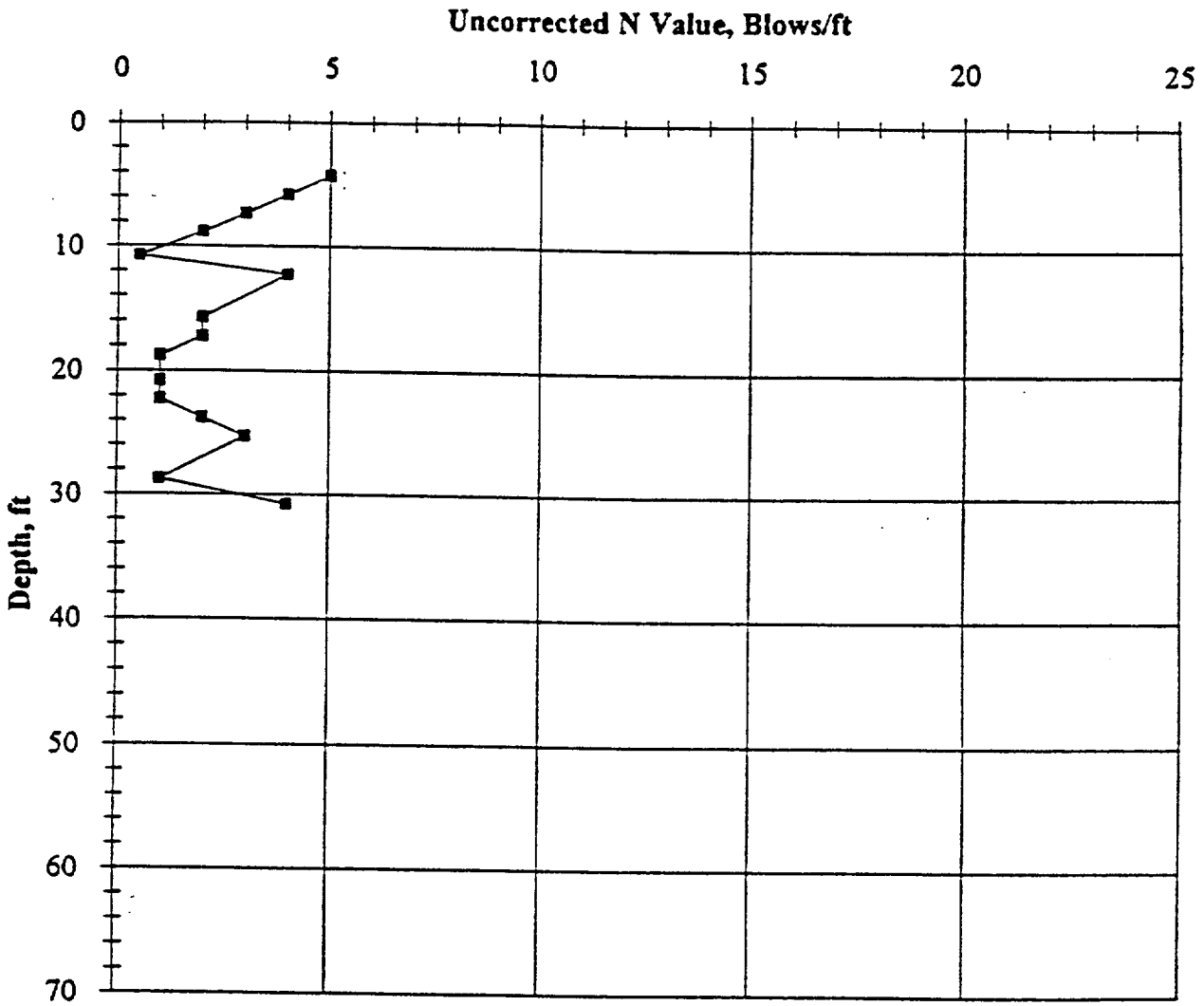
Boring T-7



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT SPT RESULTS BORING T-7			
Checked by _____ Date 5-4-94 Approved by RLV Date 5-5-94	Project No. SMI-100	Figure No. D-7	

L.C-120

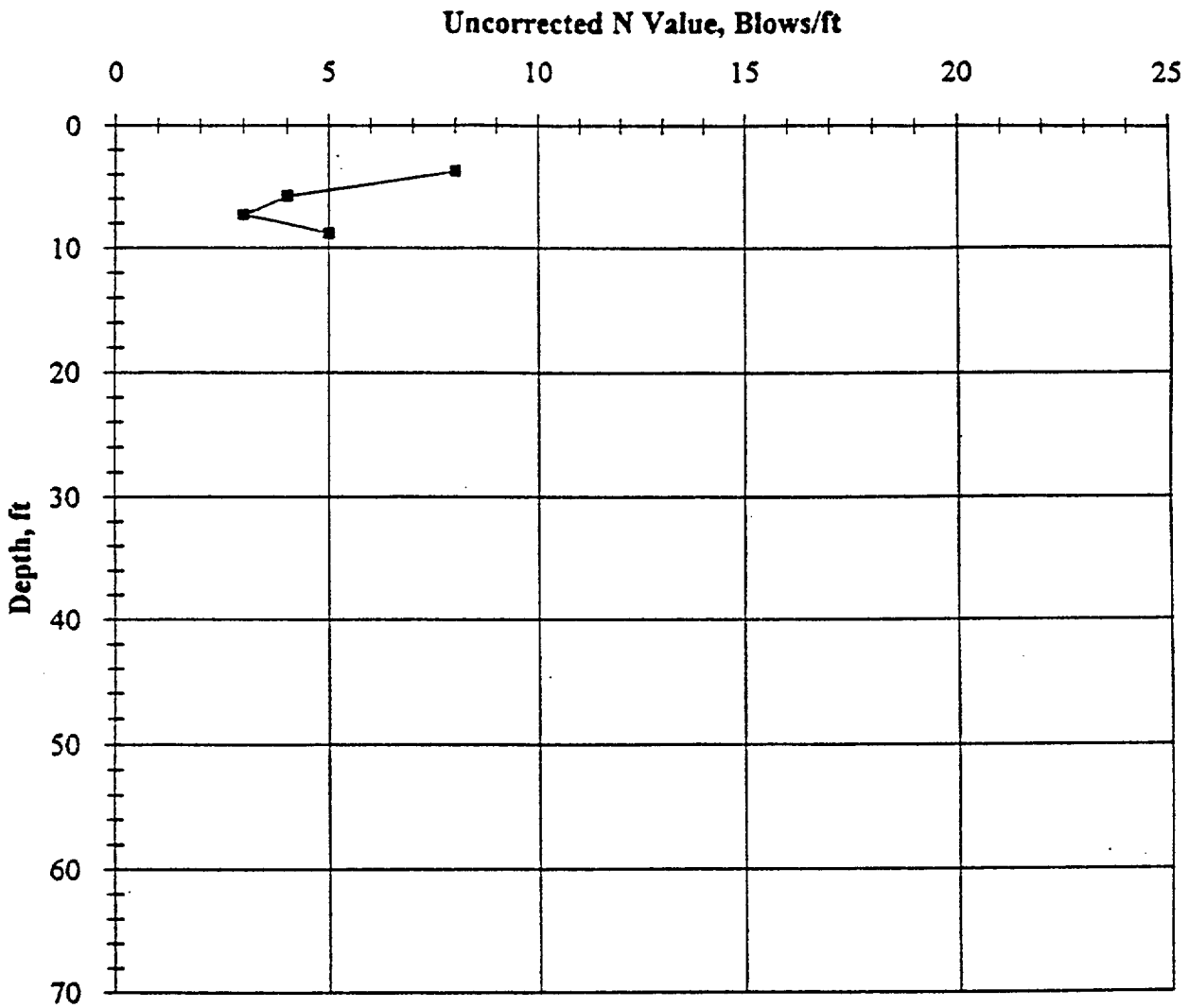
Boring T-8



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT SPT RESULTS BORING T-8			
Checked by [illegible] Date 5-4-94 Approved by RLV Date 5-5-94	Project No. SMI-100	Figure No. D-8	

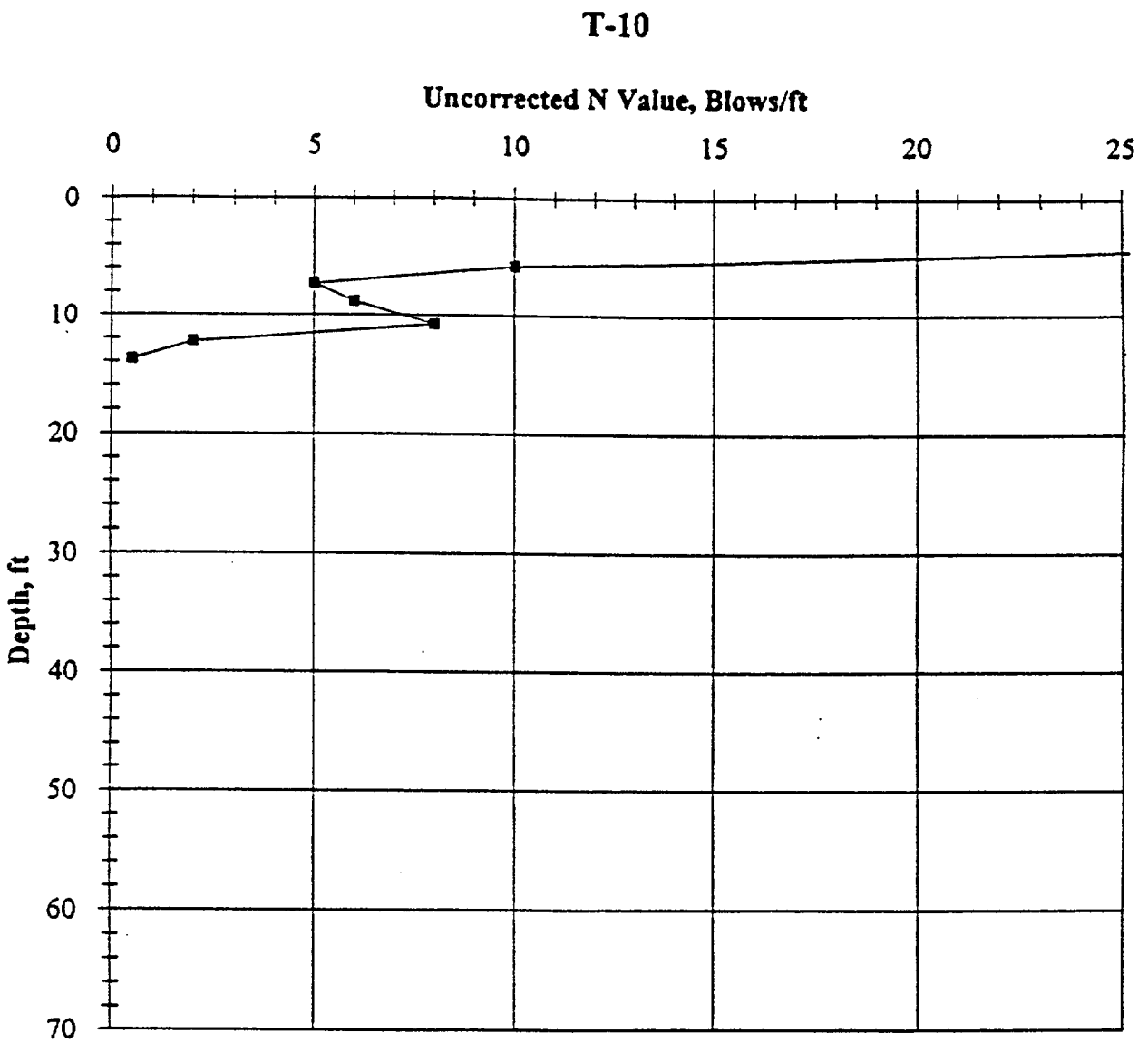
L.C-130

Boring T-9



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT **SPT RESULTS BORING T-9**			
Checked by [illegible] Date 5-4-94 Approved by RLV Date 5-5-94	Project No. SMI-100	Figure No. D-9	

L.C-131



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT **SPT RESULTS BORING T-10**			
Checked by [illegible] Date 5-4-94 Approved by RLV Date 5-5-94	Project No. SMI-100	Figure No. D-10	

Corrections to Obtain SPT-Equivalent Blow Counts

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (*.55)	CORRECTION FACTORS DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60
T-1	4.5	SPT	5	SM	N	5	4	3	3	10	10	1294	1.60	17
	6	SPT	7	SM	N	7	5	4	5	12	12	1425	1.60	19
	7.5	SPT	8	ML	N	8	6	5	5	12	12	1557	1.12	14
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1.08	9
	11	SPT	3	SM	N	3	3	2	3	10	10	1826	1.05	10
	12.5	SPT	2	SM	N	2	2	2	2	9	9	1899	1.03	9
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8
T-2	4.5	SPT	5	SM	N	5	4	3	3	10	10	1294	1.60	17
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10
	9	SPT	0.5	SM	N	1	0	0	0	7	7	1689	1.08	8
	11	SPT	1	SM	N	1	1	1	1	8	8	1826	1.05	8
	12.5	SPT	3	SM	N	3	3	2	3	10	10	1899	1.03	10
	14	SPT	3	SM	N	3	3	2	3	10	10	1971	1.01	10
	16	SPT	3	SM	N	3	3	2	3	10	10	2069	0.99	10
	17.5	SPT	2	SM	N	2	2	2	2	9	9	2142	0.98	9
	21	SPT	1.5	ML	N	2	2	1	1	8	8	2312	0.95	8
	24.5	SPT	3	ML	N	3	3	2	3	10	10	2482	0.91	9

L.C-132

Corrections to Obtain SPT-Equivalent Blow Counts

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS MOD. CAL. (*.55)	DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60
T-3	4	SPT	3	SM	N	3	2	2	2	9	9	1250	1.60	14
	5.5	SPT	3	SM	N	3	2	2	2	9	9	1381	1.60	14
	7	SPT	2	SM	N	2	2	1	1	8	8	1513	1.60	13
	9	SPT	0.5	SM	N	1	0	0	0	7	7	1689	1.08	8
	11	SPT	0.5	SM	N	1	1	0	0	7	7	1826	1.05	8
	13	SPT	0.5	SM	N	1	1	0	0	7	7	1923	1.03	8
	14.5	SPT	0.5	ML	N	1	1	0	0	7	7	1996	1.01	8
	16	SPT	0.5	ML	N	1	1	0	0	7	7	2069	0.99	7
	17.5	SPT	0.5	ML	N	1	1	0	0	7	7	2142	0.98	7
	19	SPT	0.5	ML	N	1	1	0	0	7	7	2214	0.96	7
	20.5	SPT	0.5	SM	N	1	1	0	0	7	7	2287	0.95	7
	22	SPT	0.5	SM	N	1	1	0	0	7	7	2360	0.94	7
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0.92	9
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0.88	9

L.C-133

Corrections to Obtain SPT-Equivalent Blow Counts

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (*.55)	CORRECTION FACTORS DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60
T-4	4.5	SPT	6	SM	N	6	5	3	4	11	11	1294	1.60	18
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	2	SM	N	2	2	1	1	8	8	1557	1.12	9
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1.08	10
	11	SPT	1	ML	N	1	1	1	1	8	8	1826	1.05	8
	12.5	SPT	1	ML	N	1	1	1	1	8	8	1899	1.03	8
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0.99	9
	17.5	SPT	0	SM	N	0	0	0	0	7	7	2142	0.98	7
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0.96	8
	21	SPT	0	SM	N	0	0	0	0	7	7	2312	0.95	7
	22.5	SPT	1	SM	N	1	1	1	1	8	8	2385	0.93	7
	24	SPT	4	SM	N	4	4	3	4	11	11	2457	0.92	10
	26	SPT	4	SM	N	4	4	3	4	11	11	2555	0.90	10
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0.88	9
	31	SPT	6	SM	N	6	6	5	5	12	12	2798	0.86	11
	32.5	SPT	6	SM	N	6	6	5	5	12	12	2871	0.85	11
	34	SPT	16	SM	N	16	16	12	14	21	21	2943	0.84	18
	36	SPT	14	SM	N	14	14	11	13	20	20	3041	0.83	16
	37.5	SPT	13	SM	N	13	13	10	12	19	19	3114	0.82	15
	39	SPT	17	SM	N	17	17	13	15	22	22	3186	0.81	18
	41	SPT	14	SM	N	14	14	11	13	20	20	3284	0.79	16
	42.5	SPT	18	SM	N	18	18	14	16	23	23	3357	0.78	18
	44	SPT	13	ML	N	13	13	10	12	19	19	3429	0.77	14
	48	SPT	16	ML	N	16	16	12	14	21	21	3624	0.75	18
	49.5	SPT	15	ML	N	15	15	11	14	21	21	3697	0.74	15
	51	SPT	14	ML	N	14	14	11	13	20	20	3770	0.73	14
	52.5	SPT	12	ML	N	12	12	9	11	18	18	3843	0.73	13
	54	SPT	8	ML	N	8	8	6	7	14	14	3915	0.72	10
	56	SPT	12	ML	N	12	12	9	11	18	18	4013	0.71	13
	57.5	SPT	11	ML	N	11	11	8	10	17	17	4086	0.70	12
	61	SPT	24	ML	N	24	24	18	22	29	29	4256	0.68	19
	62.5	SPT	12	ML	N	12	12	9	11	18	18	4329	0.67	12
	64	SPT	10	SM	N	10	10	8	9	16	16	4401	0.67	11

L.C-134

Corrections to Obtain SPT-Equivalent Blow Counts

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS MOD. CAL. (*.55)	DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60
T-5	4.5	SPT	4	SM	N	4	3	2	3	10	10	1294	1.60	16
	6	SPT	4	ML	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	4	ML	N	4	3	2	3	10	10	1557	1.12	11
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1.08	10
	11	SPT	4	SM	N	4	4	3	4	11	11	1826	1.05	11
	12.5	SPT	2	ML	N	2	2	2	2	9	9	1899	1.03	9
	14	SPT	1.5	ML	N	2	2	1	1	8	8	1971	1.01	8
	16	SPT	0	ML	N	0	0	0	0	7	7	2069	0.99	7
	17.5	SPT	0	ML	N	0	0	0	0	7	7	2142	0.98	7
	19	SPT	0	ML	N	0	0	0	0	7	7	2214	0.96	7
	23	SPT	3	SM	N	3	3	2	3	10	10	2409	0.93	9
	24.5	SPT	4	SM	N	4	4	3	4	11	11	2482	0.91	10
	26	SPT	6	SM	N	6	6	5	5	12	12	2555	0.90	11
	27.5	SPT	0	SM	N	0	0	0	0	7	7	2628	0.89	6
	29	SPT	2	SP-SM	N	2	2	2	2	2	2	2700	0.88	2
T-6	4.5	SPT	6	SM	N	6	5	3	4	11	11	1294	1.60	18
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10
	9	SPT	0.5	ML	N	1	0	0	0	7	7	1689	1.08	8

L.C-135

Corrections to Obtain SPT-Equivalent Blow Counts

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS MOD. CAL. (*.55)	DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60
T-7	4	SPT	9	SM	N	9	7	5	6	13	13	1250	1.60	21
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	4	SM	N	4	3	2	3	10	10	1557	1.12	11
	9	SPT	3	ML	N	3	2	2	2	9	9	1689	1.08	10
	11	SPT	4	ML	N	4	4	3	4	11	11	1826	1.05	11
	12.5	SPT	2	ML	N	2	2	2	2	9	9	1899	1.03	9
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8
	16	SPT	1	SM	N	1	1	1	1	8	8	2069	0.99	8
	17.5	SPT	1	ML	N	1	1	1	1	8	8	2142	0.98	8
	19	SPT	5	ML	N	5	5	4	5	12	12	2214	0.96	11
	21	SPT	1	SM	N	1	1	1	1	8	8	2312	0.95	7
	22.5	SPT	3	SM	N	3	3	2	3	10	10	2385	0.93	9
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0.92	9
	26	SPT	2	ML	N	2	2	2	2	9	9	2555	0.90	8
	27.5	SPT	4	SM	N	4	4	3	4	11	11	2628	0.89	9
	29	SPT	4	ML	N	4	4	3	4	11	11	2700	0.88	9
	31	SPT	6	ML	N	6	6	5	5	12	12	2798	0.86	11
	32.5	SPT	6	SM	N	6	6	5	5	12	12	2871	0.85	11
	34	SPT	8	SM	N	8	8	6	7	14	14	2943	0.84	12
	36	SPT	2	SM	N	2	2	2	2	9	9	3041	0.83	7
	37.5	SPT	3	SM	N	3	3	2	3	10	10	3114	0.82	8
	39	SPT	4	ML	N	4	4	3	4	11	11	3188	0.81	9
	41	SPT	4	ML	N	4	4	3	4	11	11	3284	0.79	8
	42.5	SPT	5	ML	N	5	5	4	5	12	12	3357	0.78	9
	44	SPT	7	ML	N	7	7	5	6	13	13	3429	0.77	10
	46	SPT	10	ML	N	10	10	8	9	16	16	3527	0.76	12
	47.5	SPT	5	ML	N	5	5	4	5	12	12	3600	0.75	9
	49	SPT	6	ML	N	6	6	5	5	12	12	3672	0.75	9
	51	SPT	9	ML	N	9	9	7	8	15	15	3770	0.73	11
	52.5	SPT	4	ML	N	4	4	3	4	11	11	3843	0.73	8
	54	SPT	12	ML	N	12	12	9	11	18	18	3915	0.72	13
	56	SPT	20	SM	N	20	20	15	18	25	25	4013	0.71	18
	57.5	SPT	22	SM	N	22	22	17	20	27	27	4086	0.70	19
	59	SPT	20	SM	N	20	20	15	18	25	25	4158	0.69	17
	61	SPT	18	ML	N	18	18	14	16	23	23	4256	0.68	16
	62.5	SPT	22	ML	N	22	22	17	20	27	27	4329	0.67	18
	64	SPT	24	ML	N	24	24	18	22	29	29	4401	0.67	19

L.C-136

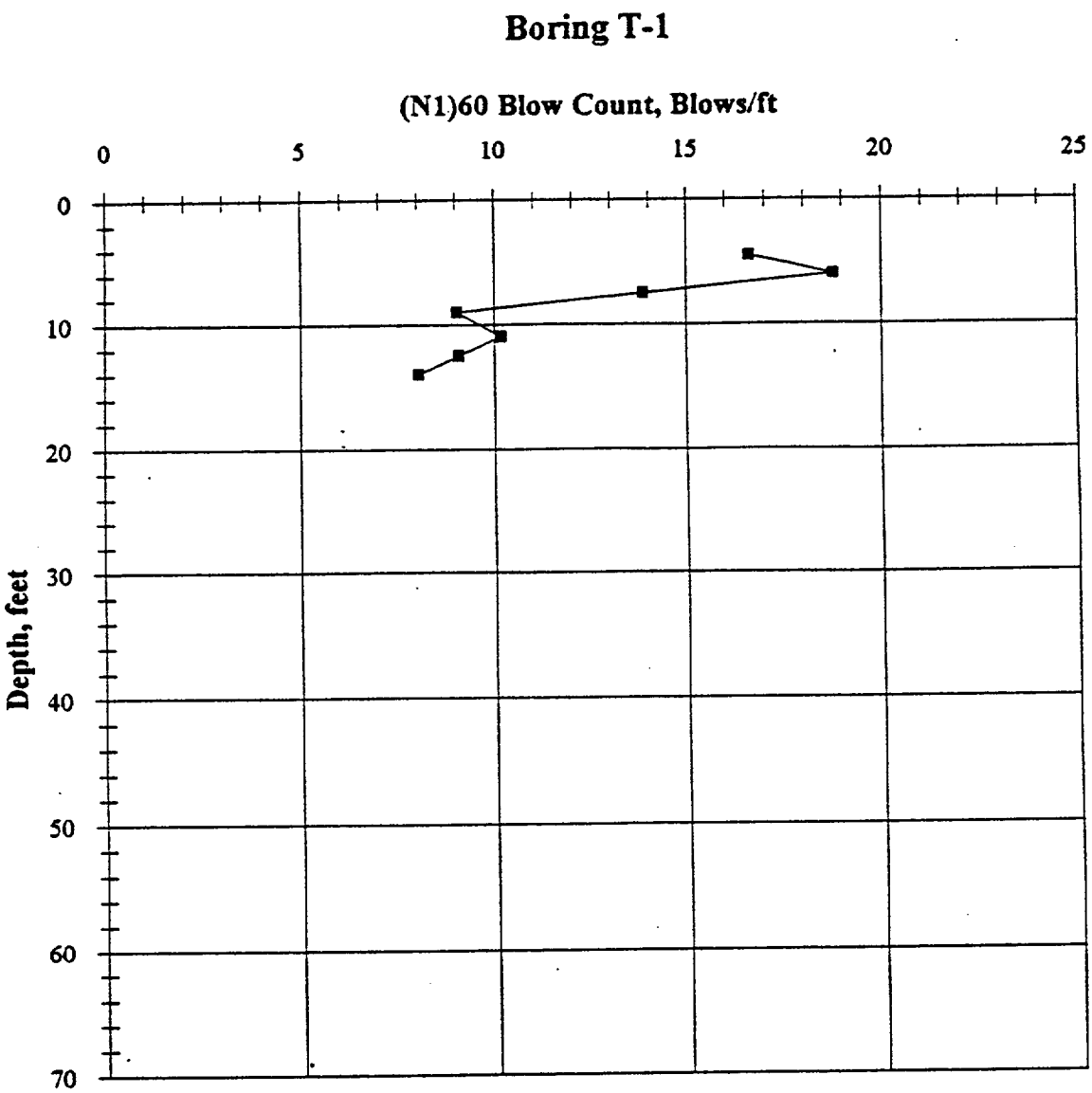
Corrections to Obtain SPT-Equivalent Blow Counts

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (*.55)	CORRECTION FACTORS DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60
T-8	4.5	SPT	5	SM	N	5	4	3	3	10	10	1294	1.60	17
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	3	ML	N	3	2	2	2	9	9	1557	1.12	10
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1.08	9
	11	SPT	0.5	ML	N	1	1	0	0	7	7	1826	1.05	8
	12.5	SPT	4	ML	N	4	4	3	4	11	11	1899	1.03	11
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0.99	9
	17.5	SPT	2	SM	N	2	2	2	2	9	9	2142	0.98	9
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0.96	8
	21	SPT	1	ML	N	1	1	1	1	8	8	2312	0.95	7
	22.5	SPT	1	SM	N	1	1	1	1	8	8	2385	0.93	7
	24	SPT	2	ML	N	2	2	2	2	9	9	2457	0.92	8
	25.5	SPT	3	ML	N	3	3	2	3	10	10	2530	0.91	9
	29	SPT	1	ML	N	1	1	1	1	8	8	2700	0.88	7
	31	SPT	4	ML	N	4	4	3	4	11	11	2798	0.86	9
T-9	4	SPT	8	SM	N	8	6	5	5	12	12	1250	1.60	20
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16
	7.5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10
	9	SPT	5	SM	N	5	4	3	3	10	10	1689	1.08	11
T-10	4.5	SPT	28	SM	N	28	21	16	19	26	26	1294	1.60	41
	6	SPT	10	SM	N	10	8	6	7	14	14	1425	1.60	22
	7.5	SPT	5	SM	N	5	4	3	3	10	10	1557	1.12	12
	9	SPT	6	ML	N	6	5	3	4	11	11	1689	1.08	12
	11	SPT	8	ML	N	8	8	6	7	14	14	1826	1.05	15
	12.5	SPT	2	SM	N	2	2	2	2	9	9	1899	1.03	9
	14	SPT	1	SM	N	1	1	1	1	8	8	1971	1.01	8

L.C-137

L.C-138



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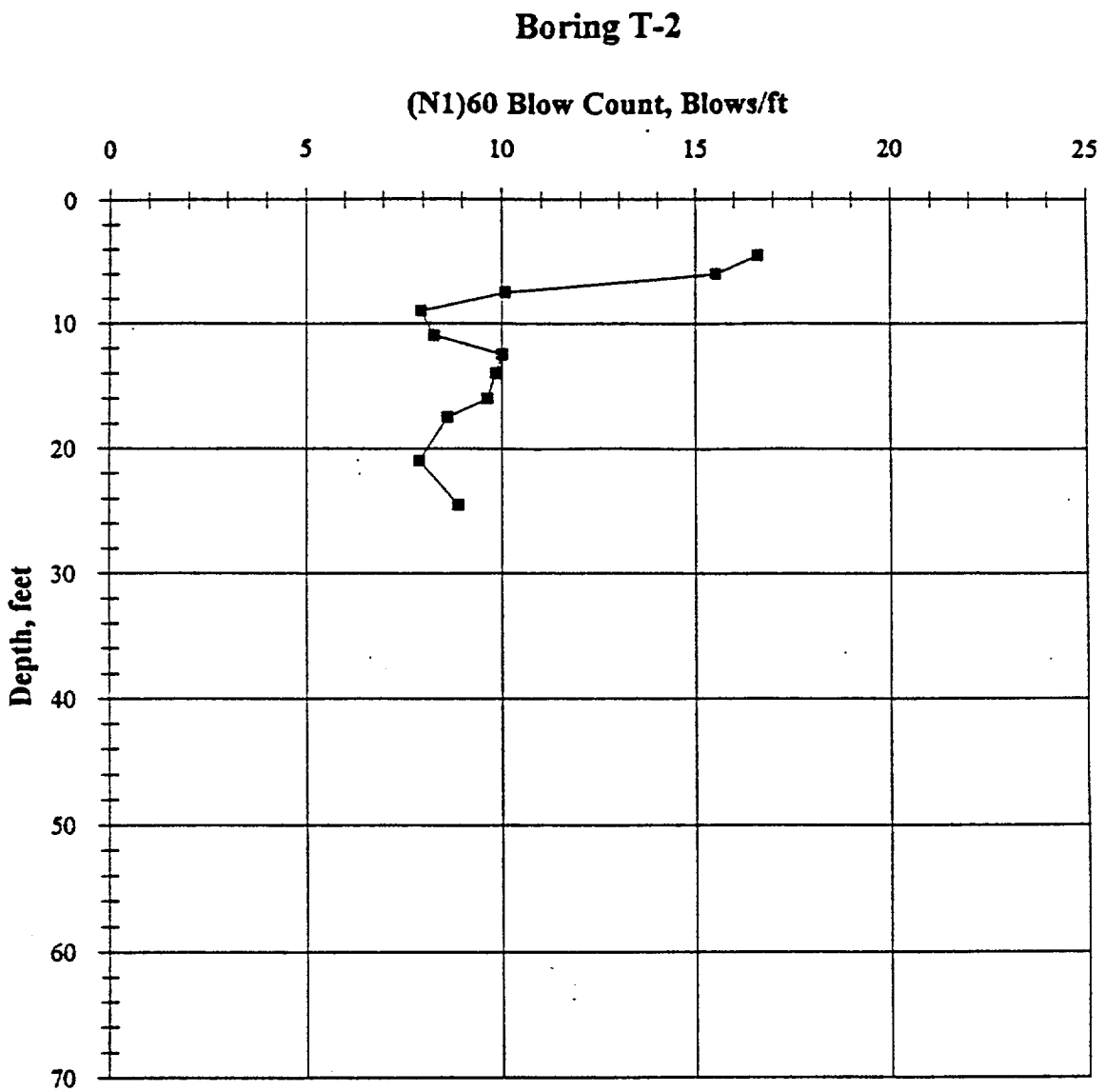
SHERWOOD TAILING IMPOUNDMENT

$(N_1)_{60}$ BLOW COUNT vs. DEPTH
BORING T-1

Checked by _____ Date _____		Project No.	Figure No.
Approved by _____ Date _____		SM-100	D-11

L.C-139



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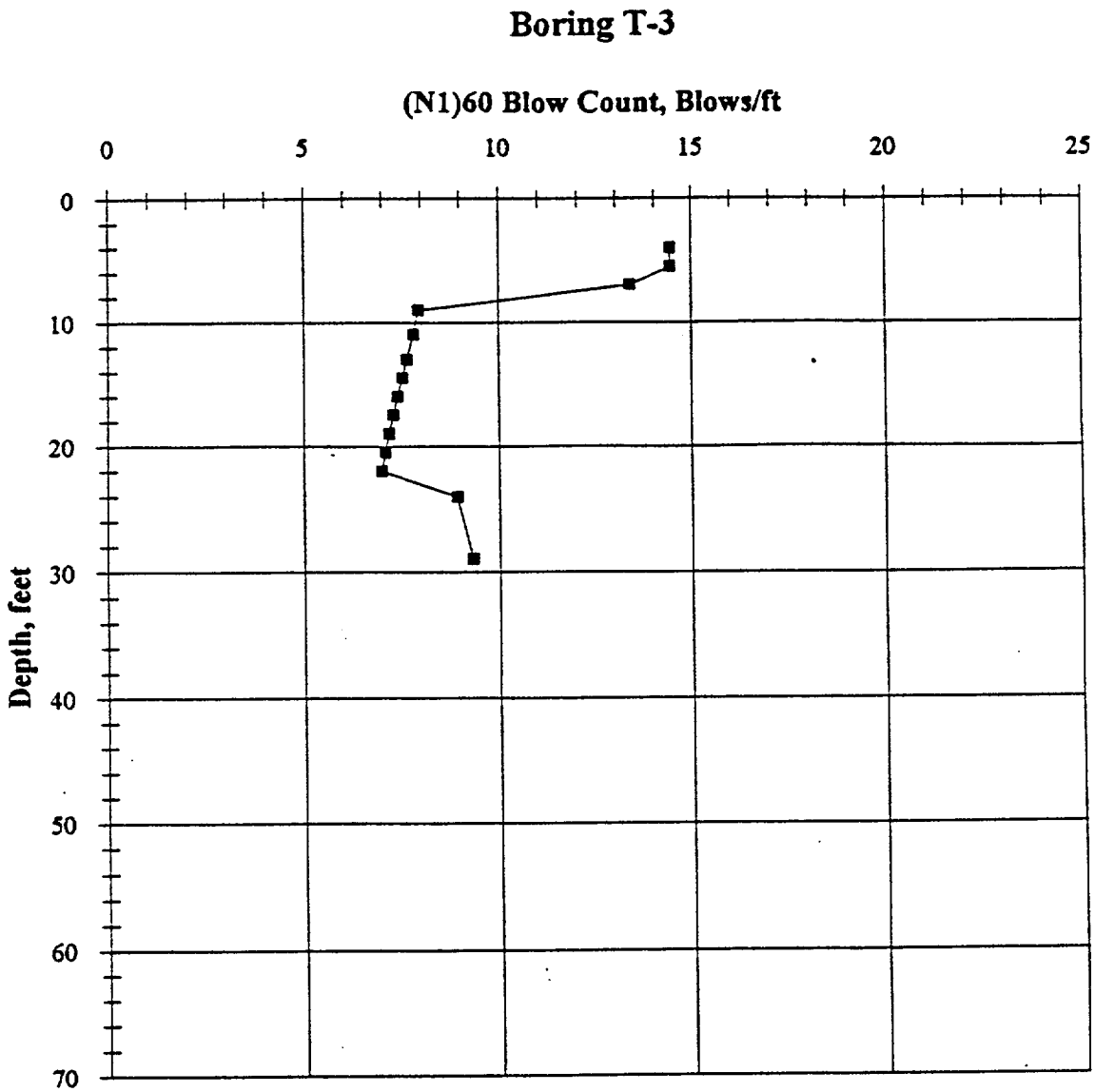
SHERWOOD TAILING IMPOUNDMENT

$(N_1)_{60}$ BLOW COUNT vs. DEPTH
BORING T-2

Checked by _____	Date _____	Project No.	Figure No.
Approved by _____	Date _____	SM-100	D-12

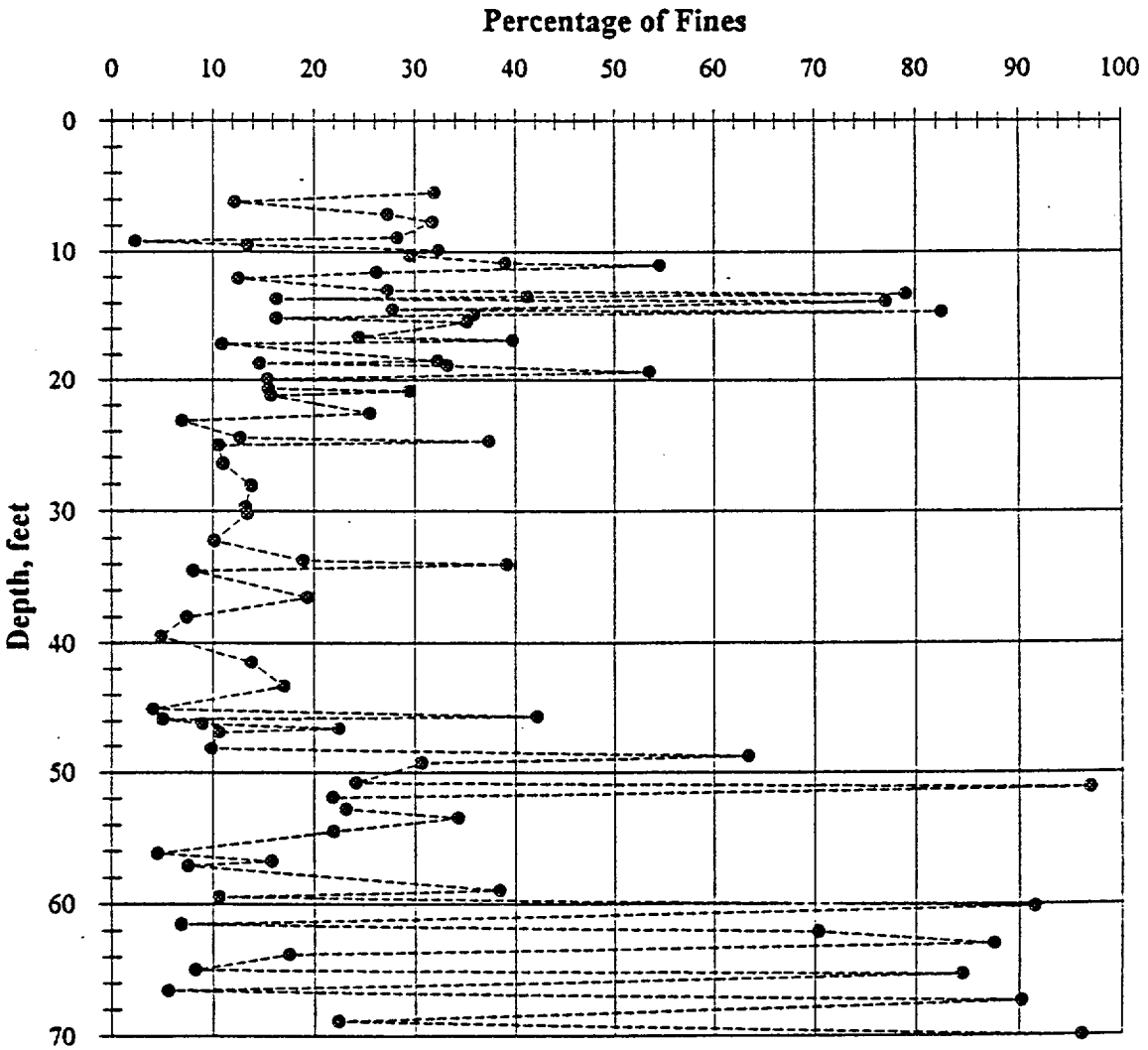
L.C-140



R.L. VOLPE & ASSOCIATES Los Gatos, California		
SHERWOOD TAILING IMPOUNDMENT $(N_1)_{60}$ BLOW COUNT vs. DEPTH BORING T-3		
Checked by _____ Date _____ Approved by _____ Date _____	Project No. SM-100	Figure No. D-13

L.C-39

Percentage Fines vs. Depth (Hole 1A)

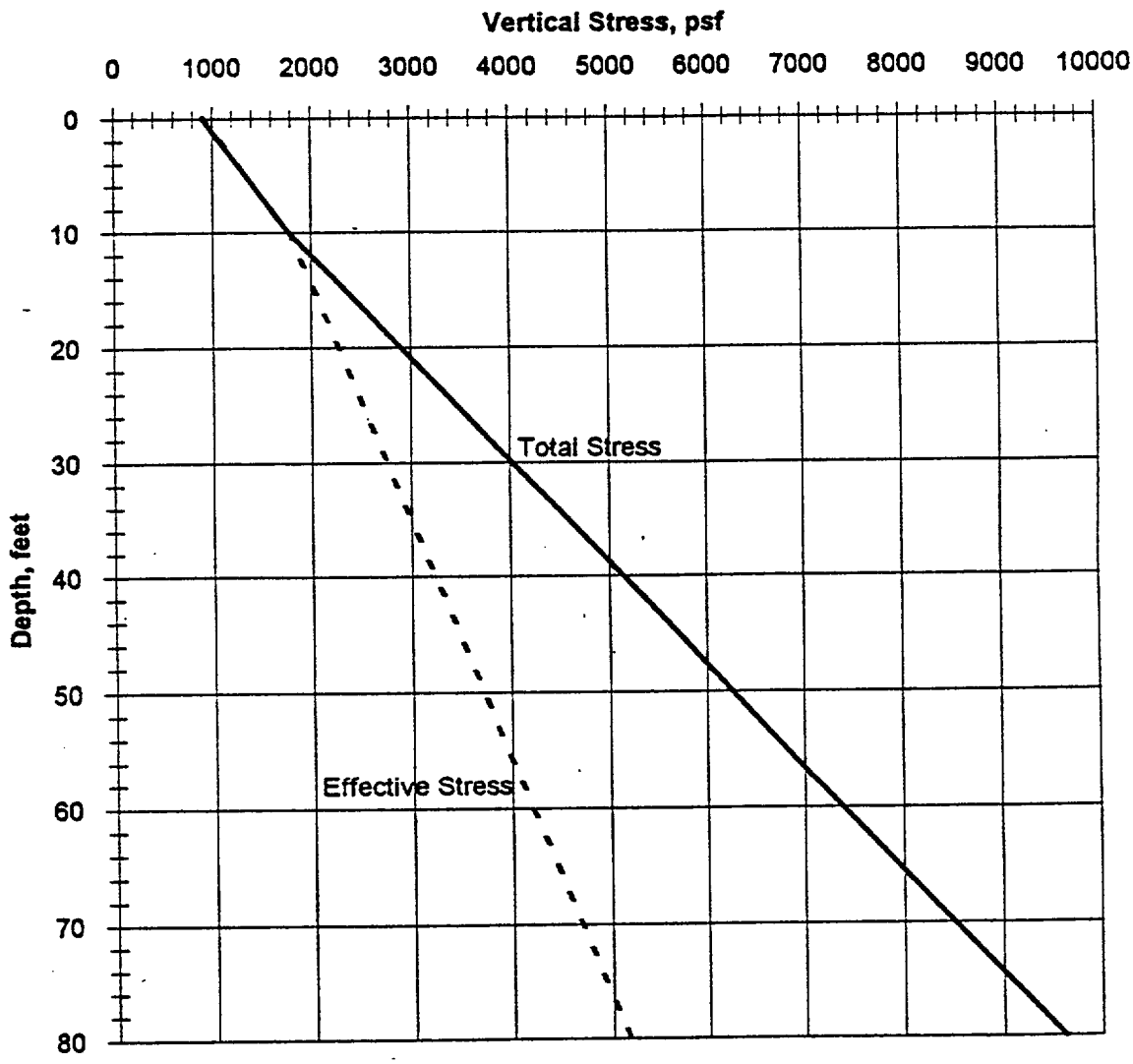


Note: The percentage of fines data are based on laboratory tests performed by SMI.

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT **PERCENTAGE OF FINES vs. DEPTH (HOLE 1A)**			
Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100	5

L.C-40

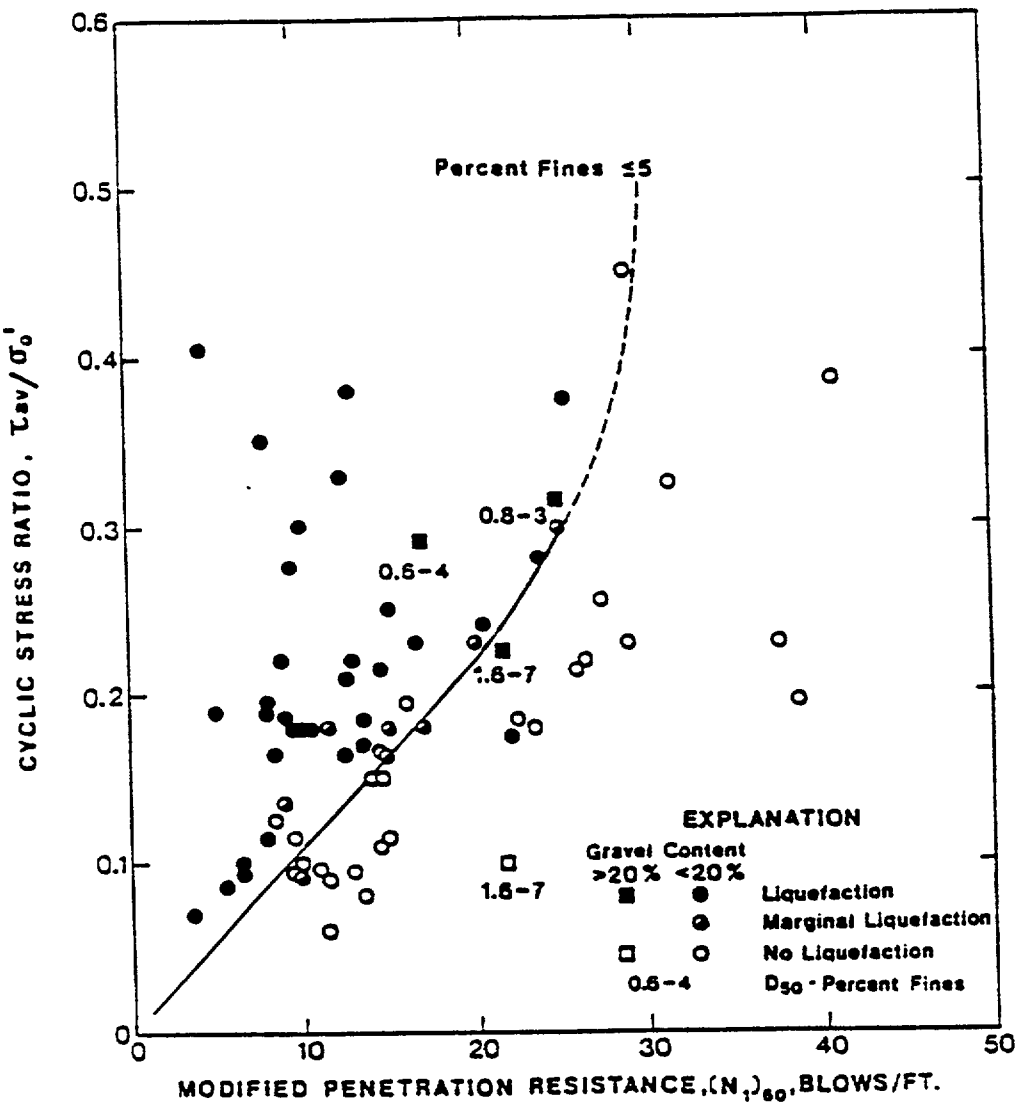
Total and Effective Overburden Pressure



Note: The stress distributions shown are based on average conditions expected to occur within the tailing pond after the reclamation cover has been installed.

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT OVERBURDEN STRESS IN POND			
Checked by _____ Date _____ Approved by RW Date 12/12/94	Project No. SMI-100	Figure No. 6	

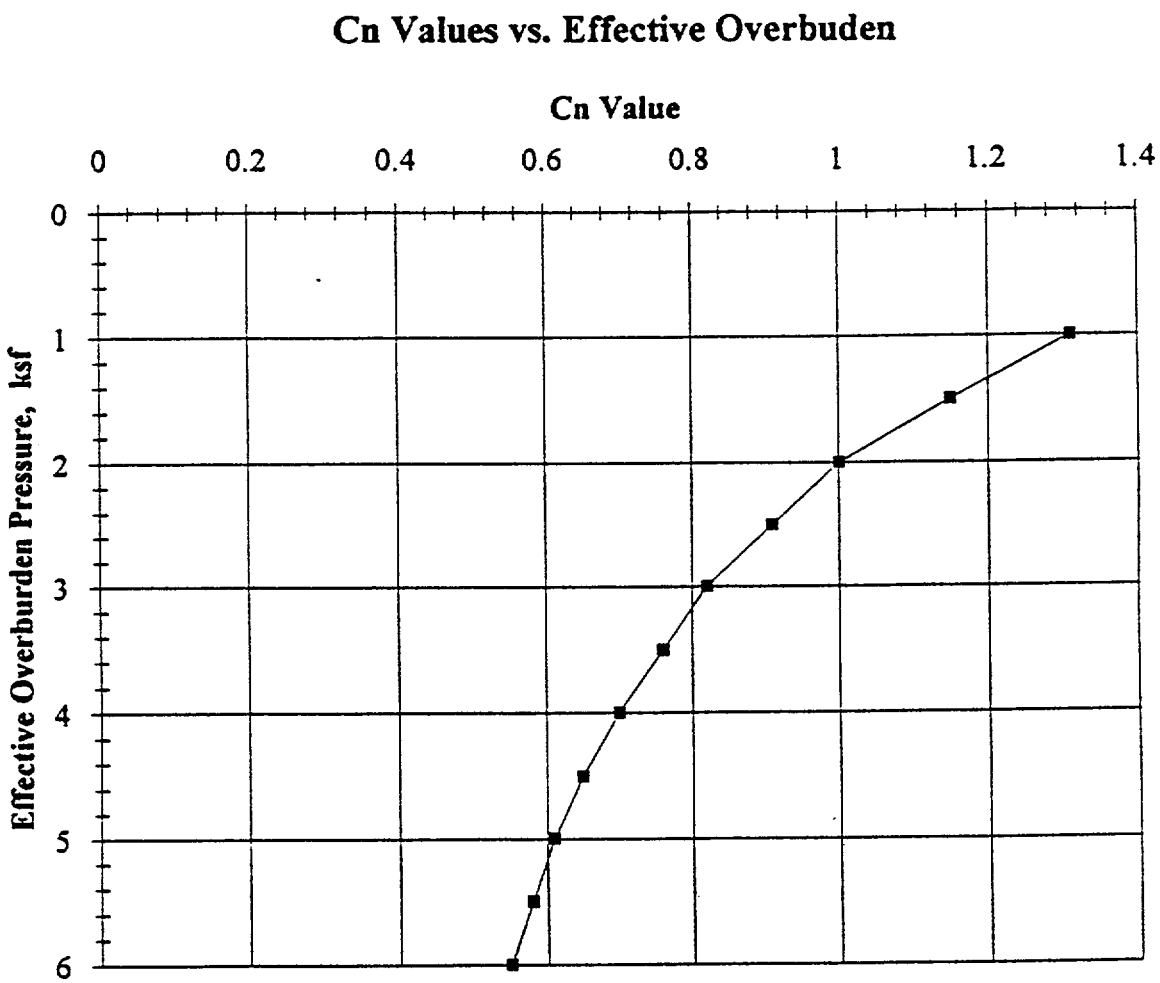
L.C-41



Liquefaction susceptibility chart with data prepared by Seed and others (1984; 1985) for clean sands (percentage of fines ≤ 5%) and a 7.5 magnitude earthquake.

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT			
LIQUEFACTION SUSCEPTIBILITY CHART 7.5 MAGNITUDE EARTHQUAKE			
Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100	7

L.C-42

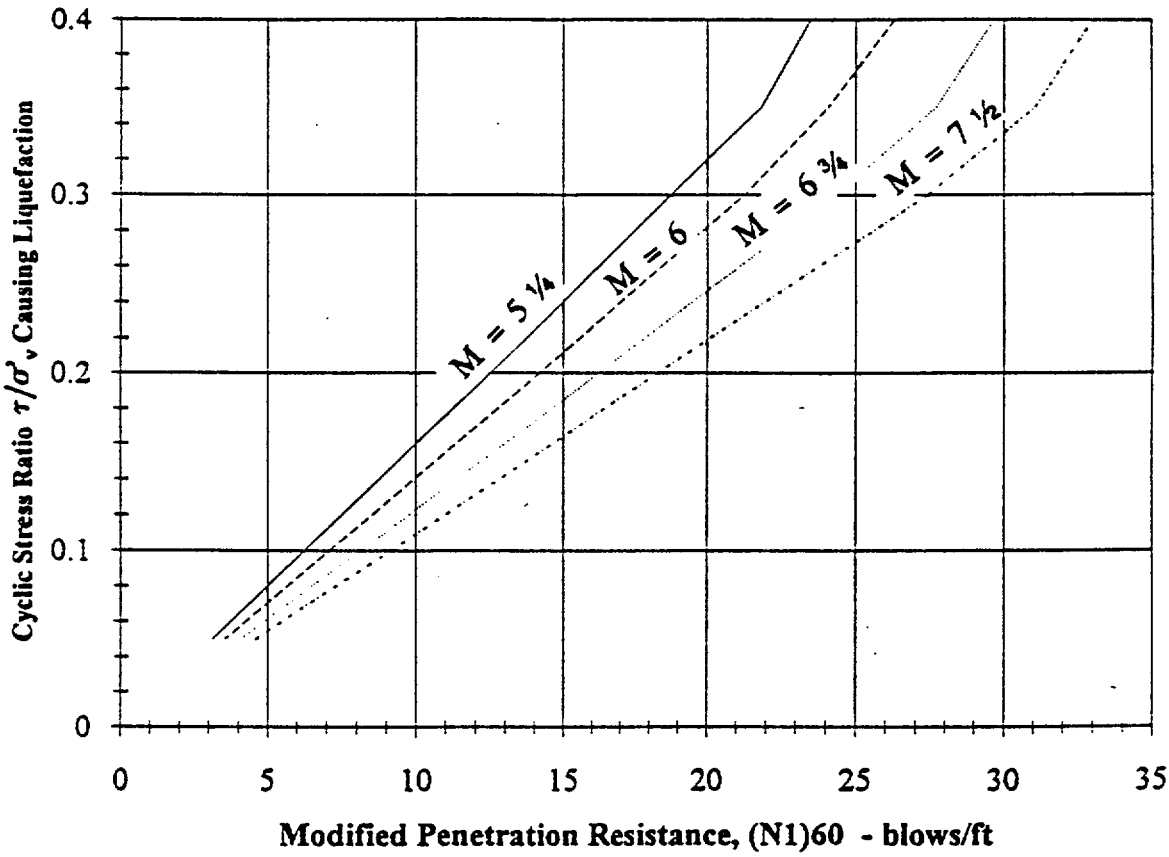


Note: The C_n relationship shown are taken from Seed et al., 1983

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT C_n VALUES vs. EFFECTIVE OVERBURDEN PRESSURE			
Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100	8

L. C-43

Chart for Evaluation of Liquefaction Potential for Different Magnitude Earthquakes



Note: The $(N_1)_{60}$ relationship shown is taken from Seed et al., 1983.

R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT LIQUEFACTION SUSCEPTIBILITY CHART DIFFERENT MAGNITUDE EARTHQUAKES			
Checked by	Date 5-4-94	Project No.	Figure No.
Approved by RLV	Date 5-5-94	SMI-100	9

L.C-44

APPENDIX A

SEISMOTECTONIC SETTING OF EASTERN WASHINGTON

APPENDIX A

Seismotectonic Setting of Eastern Washington

This appendix presents a discussion of the seismotectonic setting for portions of central and eastern Washington as it relates to the Sherwood Tailing Impoundment. The seismicity of this area is relatively quiescent, especially when compared to other areas of the northwest, such as the area adjacent to the coastal subduction zone where the Pacific Plate is being forced under the North America Plate. This relatively low level of seismicity is confirmed by the results of a recent in-depth study performed for the U.S. Bureau of Reclamation, entitled "Seismotectonic Evaluation - Walla Walla Section of the Columbia Plateau Geomorphic Province". The referenced report was prepared by Geomatrix Consultants, Inc. of San Francisco, California, in April, 1990, and provided the majority of information presented in this appendix.

A. Introduction

A map showing the geomorphic provinces within eastern Washington is presented in Fig. A-1. As shown in this figure, the Sherwood Tailing Dam site is located on the southern boundary of the Omineca Crystalline Belt Subprovince of the Northwestern Rocky Mountains-Okanogan Uplands province, and the northern boundary of the Palouse Subprovince of the Columbia Plateau. The Columbia Plateau geomorphic province is a region of relatively flat surfaces interrupted by east-trending rolling hills and valleys. Major drainages flow towards the west and south into the Columbia River and the Snake River.

The study region is thought to be undergoing north-south compression, with a regional uparching of the Northern Cascades and downwarping of the Columbia Plateau, and possible release of stress following glacial unloading. The tectonics of the region appear to be influenced primarily by the ongoing oblique northeast subduction of the Juan de Fuca plate beneath the North America plate in eastern Washington. The direction of plate convergence is between N39°E and N57°E, and convergence is occurring at a rate of between 3 and 4 cm per year (Heaton and Kanamori, 1984). The oblique direction of convergence is possible enough to impose right-lateral shear component of stress in the overlying North American Plate (Davis, 1981).

Major uparching of the Northern Cascades occurred during the late Pliocene to early Pleistocene along a north-south axis (Misch, 1977; Hammond and others, 1977). This uparching reaches a maximum near the Canadian border and steadily diminishes to the south.

The Columbia Plateau has undergone extensive downwarping since the initial deposition of flood basalts approximately 17 million years ago (Ma). The oldest basalts flowed onto Oligocene terrestrial sediments and are now approximately 3100 meters (m) below mean sea level (MSL). Geophysical data suggest that thicker basalts extend 5000 to 6000 m below MSL, and then become thinner along the margin of the plateau. This suggests that the Columbia Plateau is a large saucer-shaped basin. It is not known if the tendency for arching in the Northern Cascades and sagging in the Columbia Plateau is continuing at the present.

B. Seismotectonic Provinces

The project site is located on the boundary of two seismotectonic provinces: the Northwestern Rocky Mountains-Okanogan Uplands to the north, and the Columbian Plateau to the south. A seismotectonic province is defined as an area or region that exhibits similar geologic characteristics, structural features, and tectonic history such that it can be reasonably expected to exhibit relatively uniform potential for seismic activity.

In some areas the boundaries between seismotectonic provinces are structurally controlled, such as along a fault or zone of faulting. In other areas, the boundaries are not structurally controlled, and may consist of broad transitional zones. The boundary between the Northwestern Rocky Mountains-Okanogan Uplands and the Columbian Plateau provinces is defined by the limit of the onlap of the Columbia River basalts, and is not structurally controlled. The boundary between the Northern Cascades and the Northwestern Rocky Mountains-Okanogan Uplands, located about 75 miles west of the project site, is defined and structurally controlled by the Pasayten fault of the Methow graben.

1. Northwestern Rocky Mountains-Okanogan Uplands - The Northwestern Rocky Mountains-Okanogan Uplands province borders the Columbia Plateau to the south and the Northern Cascades to the west. The province consists of two distinct subprovinces: the Intermontane Belt on the west, and the Omineca Crystalline Belt on the east (see Fig. A-1). Only the later subprovince, which lies immediately north of the site, is discussed below.

Omineca Subprovince - This subprovince is bounded on the west by the Okanogan shear zone and on the south by the basalt flows within the Columbia Plateau province. The subprovince is characterized by medium- to high-grade metamorphic rocks of Pennsylvanian to middle Jurassic ages, and gneiss and granitic plutons of Mesozoic and early Cenozoic ages.

The Okanogan gneiss dome (Fig. A-1) is the largest and most westerly of the gneiss domes in the Okanogan Uplands. It is composed largely of Precambrian to Mesozoic orthogneiss (Cheney, 1980) and Cenozoic granite plutons (Holder and Holder, 1988). Bordering on the west side of the gneiss dome is a mylonitic zone that separates the gneiss from granitic and metasedimentary rocks. This low-angle fault, called the Okanogan shear zone (Templeton-Kluit and Parkinson, 1986), has been interpreted as a detachment zone along which there has been crustal extension, separating metamorphosed ductily deformed mid-crustal rocks (Potter and others, 1986) from overlying upper-crustal brittely deformed rocks. The southern end of the Okanogan shear zone is located about 80 miles northwest of the project site.

The Lincoln and Kettle gneiss domes are two similar but smaller gneiss domes located to the east of the Okanogan gneiss dome (Fig. A-1). These domes are bordered on their eastern sides by low-angle faults. The three gneiss domes in the subprovince are highly asymmetric; they may represent a single dome, bisected by the Republic and Keller graben (Atwater and Reinhart, 1984).

The Republic graben, the eastern edge of which is located about 45 miles northwest of the project site, is the most extensive structure within the subprovince. The graben extends from the vicinity of Nespelem north-northeast to slightly beyond the U.S.-Canada border. The west side of the graben is bound by the Bacon Creek-Scatter Creek fault zone and the east side is bound by the

Sherman fault zone. The southern ends of these faults die out approximately 16 km north of the Columbia River. The graben is filled by up to 6,000 m of Eocene volcanic rocks (Muessig, 1962) and is believed to be downfolded and partly downfaulted remnant of a formerly more extensive blanket of andesitic volcanics, remnants of which are also preserved southwest and east of the Republic graben (Atwater and Reinhart, 1984).

2. Columbia Plateau - The Columbia Plateau comprises the area east of the Cascade Range and south of the Northwestern Rocky Mountains-Okanogan Uplands, that is underlain by the Columbian River basalts. The plateau is a large intermontane basin filled with tholeitic basalt and minor interbedded sedimentary layers. Fill thicknesses range from over 4600 m in the central part near Pasco (15 miles north of the Oregon border) to less than 30 m at the margins.

Deposition of the Columbia River basalts occurred episodically between 17 Ma and 6 Ma. More than 99 percent of the total volume of basalt had erupted by 13.5 Ma (Tolan and others, 1987). Surface drainage from elevated lands surrounding much of the depositional basin probably coalesced to flow westward approximately along the course of the modern Columbia River (Kienle, 1971). Tongues of Columbia River basalts also followed this route westward and are present near the mouth of the Columbia River.

The Columbia Plateau is divided into three areas having different late Cenozoic strain histories: 1) an eastern area located due south of the site, the Palouse subprovince is characterized by east-northeast extension (Rockwell Hanford, 1979); 2) a western area, the Yakima Fold Belt subprovince, is characterized by north-south compression; and, 3) a southeastern area, the Blue Mountains, is characterized by an anticlinal arch (Davis, 1981). Only the Palouse subprovince is discussed below.

Palouse Subprovince - The Palouse subprovince of the Columbia Plateau generally consists of horizontal basalt flows. The area is characterized by gentle folding; it is distinguished from the Yakima Fold Belt and the Blue Mountains subprovince by a relative lack of major deformation. Unlike the other subprovinces within the Columbian Plateau, the topography of the Palouse subprovince is primarily a result of erosional and depositional processes, rather than structural processes. Much of the area is covered by Palouse loess, a layer of silty sediments up to one meter or more in thickness. The loess was brought into the area by wind, but sedimentary structures indicate that it was deposited by streams and lakes. Fossil remains show that the Palouse deposits are of Pleistocene age (Campbell, 1962).

The most significant structures in the Palouse subprovince are the Badger Mountain anticline, Coulee monocline, and Baker Canyon monocline (Fig. A-1). These features are all located between 45 and 80 miles west of the project site. Known faulting within the Palouse subprovince consists of a zone of thrust faulting on the southeast extension of the Badger Mountain anticline and a minor west-trending fault north of the fold. This faulting, located about 80 miles west of the project site, is most likely of Pliocene and possibly earliest Pleistocene age (Shannon and Wilson, 1977a).

3. Summary of Geologic History

The geologic history of the Columbia Plateau region is not completely understood because the Columbia Basin basalts conceal the underlying basement rocks. Information on the pre-basalt (pre-Miocene) history of the region can come only from rocks exposed around the basalt margins, very sparse well data, and inferences from geophysical surveys. The post-Miocene history of the Columbia Plateau has been reconstructed in considerable detail.

A study of the pre-middle Cretaceous rocks of the surrounding terrane combined with reconstruction of the plate positions (Engebretson, 1982) suggests that the Pacific Northwest region was the site of prolonged oblique subduction, as the westward-moving North American plate overrode the northeastward-moving Farallon plate and associated plates. The suture between the North American continental crust and the accumulating exotic terranes to the west may have extended north through western Washington until the late Early Cretaceous. The accreted terrains, rated in from the west and south, are represented today by the metasedimentary-ophiolitic rocks of the Cascade Mountains, Okanogan Highlands, Blue Mountains and possibly the pre-Aptian sediments of the Methow graben.

A complete understanding of the origins of the Columbia Plateau superimposed across the accreted terranes, remains and enigma. One possible explanation is that an expanding wedge of sea floor spreading propagated northeastward into the edge of the continental crust from the impinging Kula-Farallon rift that may have lain just to the west during the middle Cretaceous (Engebretson, 1982). The rift may have persisted as late as the Eocene, centered under what is now the Columbia Plateau (also see Davis and others, 1978; Ewing, 1980).

Following suspension of sea floor spreading in the "Columbia rift" in the early Eocene, cooling and thermal contraction and subsidence of the new oceanic crust occurred. This resulted in accumulation in the rift of a thick sequence of early Tertiary non-marine strata, and created a drainage way for the ancestral Columbia River to the Pacific Ocean along the thermally subsiding rift axis. During the Miocene, accumulation of the Columbia River basalts in the basin resulted in additional subsidence. These basaltic magmas rose through north-northwest extensional rifts that may have been related to backarc spreading east of the Cascade volcanic arc (Carlson, 1987).

The Columbia Plateau was apparently being deformed during the eruption of basalt that commenced in the middle Miocene. Synchronous with the extension that allowed voluminous extrusions of magma, the southeast part of the area rose and gently north-south compression took place. This resulted in northeast-trending sinistral strike-slip faulting and in east-trending folds and reverse faults. Based on studies of the variations in thickness and the lateral extent of flows of the Columbia River basalts and the interbedded and suprabasalt formations, the Yakima folds were actively developing throughout much of the Miocene (as cited in U.S. Department of Energy (DOE), 1988: Reidel and others, 1980, 1982; Reidel and Fecht, 1981; Reidel, 1984). For the Saddle Mountains area, the average rate of uplift is postulated to have been greatest during the middle Miocene (Grand Ronde time), when it averaged approximately 250 m/m.y. By the late Miocene, the average uplift rate had decreased to approximately 40 to 80 m/m.y.

4. Current Tectonic Setting

The geologic region within which the project site is located is in a transitional zone characterized by low seismicity that lies between the tectonically and seismically active Pacific Borderland provinces to the west and the more stable continental interior to the east. In general, the region is considered to be undergoing north-south compression (Laubscher, 1981; Davis 1977, 1981) with regional up-arching of the Northern Cascades to the west and downwarping of the Columbia Plateau (Ludwin and others, 1991). To the west, northeast oblique subduction appears to be occurring beneath western Washington, with a convergence rate on the order of 3 to 4 cm/year between the Juan de Fuca and North American Plates (Heaton and Kanamori, 1984).

Within the Yakima Fold Belt Subprovince of the Columbia Plateau, which abuts the Palouse Subprovince and lies, at its closest point, about 75 km southwest of the project site, the east-trending folds and faults have developed as a result of north-south compression. Focal mechanism solutions for shallow and deep earthquakes in the area are consistent with the geologic data, and have an approximately horizontal, north-south axis of maximum compression and nearly vertical axis of least compression. Information on vertical and lateral crustal movement in the Yakima Fold Belt, derived from interpretation of geologic data and geodetic survey results, are summarized in the Site Characterization Plan for the Hanford Site (DOE, 1988). The history of deformation during the last 3 m.y. is difficult to reconstruct because of extensive erosion; however, the rate of uplift is estimated to be about 40 m/m.y. Extrapolation of the 40 m/m.y. rate is supported by models of microearthquake swarms that indicate a rate of seismic deformation that is in reasonable agreement with the uplift rate interpreted from geologic data (DOE, 1988).

C. Historical Seismicity

The Walla Walla section of the Columbia Plateau geomorphic province is characterized by a moderate to low level of historical seismicity. Within eastern Washington, seismicity is diffuse and generally shallow. Concentrations of seismicity are observed in the vicinity of Lake Chelan and in the region of the Saddle Mountains and Frenchman Hills anticlines. Epicenter plots of historical seismicity occurring within the geomorphic provinces near the project site are shown on Fig. A-1. The remaining portion of this section presents a discussion of historical earthquakes activity and recurrence.

1. Historical and Instrumental Earthquakes

Only four earthquakes of magnitude 5.0 or greater are known to have occurred or been recorded in eastern Washington in the modern era as verified by the establishment of regularly published newspapers in the 1850's; operation of the Victoria, British Columbia seismograph station in 1898; and establishment of the University of Washington seismograph network in 1969. A brief discussion of the significant earthquakes to have occurred in the region is presented below.

2. Significant Earthquakes

Three earthquakes of magnitude 5.0 or greater have been recorded in the Columbia Plateau geomorphic province. These events include the 1918 Corfu earthquake, the 1959 Lake Chelan earthquake, and the 1973 Royal Slope earthquake that occurred in the vicinity of the 1918 event.

A magnitude 7.4 event occurred on December 15, 1872 and was probably located in the adjoining Norther Cascades geomorphic province. However, because of the effects of the this earthquake were felt widely in the eastern Washington region, it is also described below. The significant events are presented below on Table A-1:

Table A-1

Historic Events Occurring Within Geomorphic Provinces
Surrounding the Project Site

Year	Magnitude	Location	Distance From Site (km)
1872	7.4 (I_0= 9)	(see below)	
1918	5.0 M_l, 4.4 M_S	46.7°N 119.5°W	155
1959	5.0 M_l, 4.4 M_L	47.8°N 119.9°W	125
1973	5.0 M_l, 4.4 M_L	46.9°N 119.4°W	134

Note:

I_0 - Magnitude based on epicentral intensity.
M_L - Magnitude based on local intensity; $M_L = \frac{2}{3} I_o + 1$ (Gutenburg and Richter, 1956)

a. The Earthquake of December 15, 1872 (magnitude 7.4)

The December 1872 earthquake was one of the strongest historical earthquake to occur in the Pacific Northwest. It was probably centered in the Northern Cascade geomorphic province, but no geologic structure has been identified as the probable source of this event. The epicentral location, focal depth, and magnitude of the earthquake have been extensively studied, but these studies have often had inconclusive results and/or contradicted other studies. The differences in the conclusions of reports are principally a result of varying interpretations of maximum earthquake intensity. The discrepancies are due to the different weighting given to the earthquake reports are evaluations of how ground disturbances such as landslides and groundwater effects should be incorporated into the intensity interpretations.

The 1872 earthquake was felt throughout the present state of Washington and into British Columbia, Oregon, Idaho, and Montana. Algermissen and others (1969) estimated a felt area of 365,000 km². An aftershock sequence following the mainshock was reported from locations throughout the felt area. At the time of the earthquake, population density in the Northern Cascades region was very low. For a 35,000 km² area in north-central Washington and southern British Columbia, no eye witness accounts of the earthquake are available. The low degree of confidence in historical reports about the earthquake does not preclude the possibility that two or

more moderate sized earthquakes occurred in widely separated regions (Coombs and others, 1977). Three possible epicentral areas for the 1872 earthquake have been proposed: 1) near the Canadian border, 2) near Lake Chelan, and 3) within a zone that overlaps the first two locations (Malone and Bor, 1979, Shannon and Wilson, 1977b, and Woodward-Clyde Consultants, 1977a). Figure A-2 is a map showing the three possible epicentral areas proposed for the 1872 earthquake.

b. The Corfu Earthquake of November 1, 1918 and Royal Slope Earthquake of December 20, 1973

These two earthquake sequences are the largest instrumentally recorded historical earthquakes in the Central Columbian Plateau. The events occurred at similar locations and shallow depths in a region of concentrated activity north of Saddle Mountains. Neither event is associated with a known tectonic structure. The instrumental magnitude for the 1918 event is based on a moment calculation from the single seismogram recorded at Spokane.

Little is known about the 1918 event because of the lack of adequate instrumental data. However, the sequence of activity associated with the 1973 event was studied extensively by Malone and others (1975) using a small portable network. The 1973 Royal Slope sequence was unusual for eastern Washington in that it contained a main shock followed by an aftershock sequence, as opposed to the more typical swarm sequence. Rupture occurred at depths of less than 2 km on more than one surface, all generally consistent with thrusting along east/west trending planes, as is typical for earthquakes in the Columbian Plateau. Malone and others (1975) conclude that the presence of shallow through-going structures is unlikely for the region of the 1973 event.

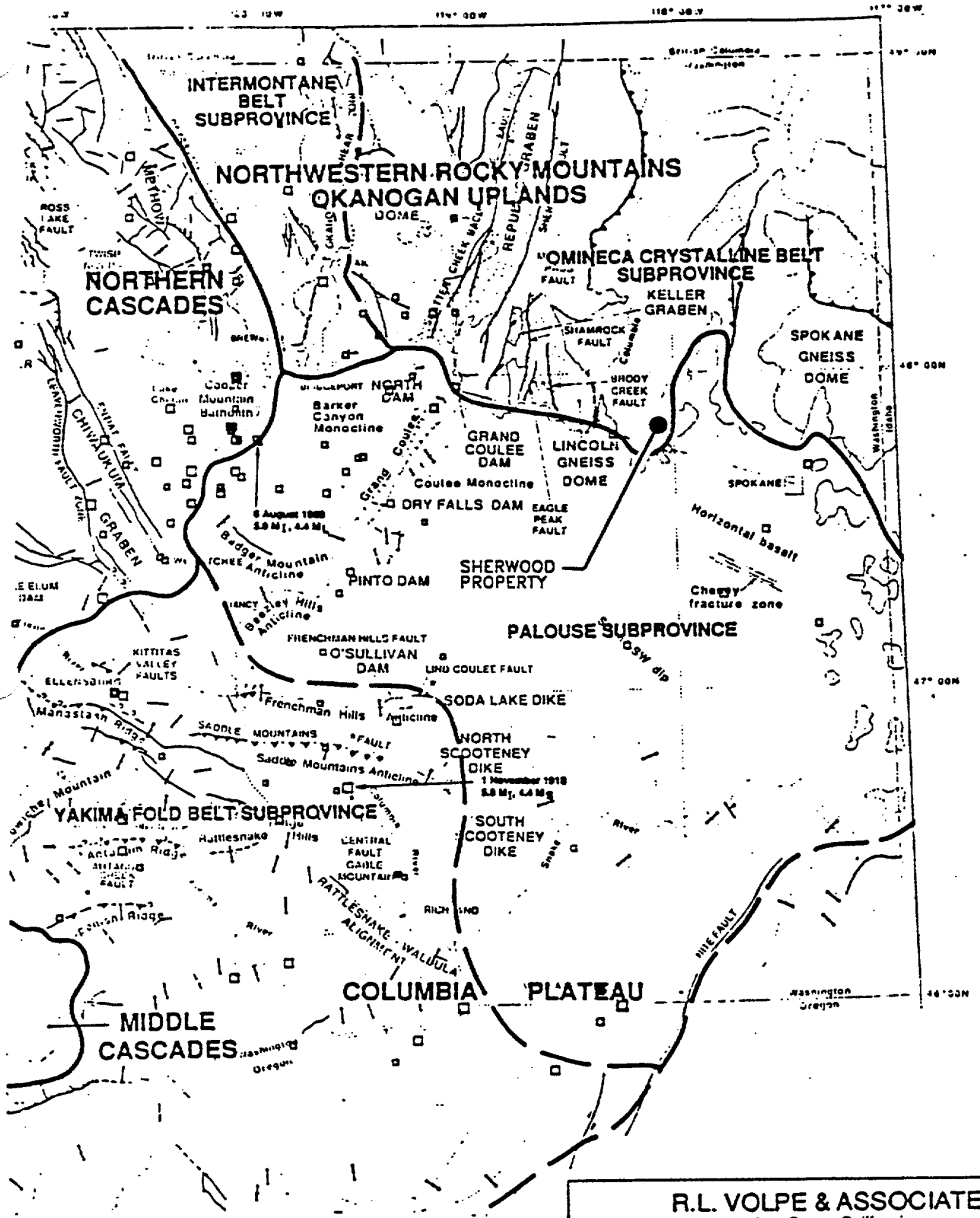
c. Lake Chelan Earthquake of August 6, 1959

A magnitude 5.0 earthquake occurred in the Lake Chelan area (47.8°N, 119.9°W) on August 5, 1959. This event was located within the microseismicity ellipse described in the previous section. However, the depth of the earthquake was 35 km (Noson and others, 1988), well below the average depth of the regional microseismicity. While the earthquake has an instrumental magnitude of 5.0, it's large felt area of 64,000 km^2 would suggest a magnitude of 5.5.

Microseismicity near Lake Chelan - A cluster of microearthquakes is located approximately 16 km south of Lake Chelan. Additional microearthquakes that may be associated with the main cluster are located to the east, and the two areas together form an east-west oriented ellipse. The reported focal depths of these earthquakes range from 0 to 11 km, and are centered around a depth of 6 km; focal depths of less than 4 km are typical of microseismicity located to the south near Hanford. Focal mechanisms from a few of the events near Lake Chelan indicate reverse faulting that could be a response to the regional north-south compressive stresses (Wooward-Clyde Consultants, 1977b).

The microearthquake activity located south of Lake Chelan is not considered to be significant to the facilities discussed in this report because the epicenters are not related to a specific geologic structure, and the regional random earthquake is judged to have equal or higher magnitude and is located at a closer distance.

L.1-32



From Geomatrix, 1990

SCALE
1" = APPROXIMATELY 30 MILES

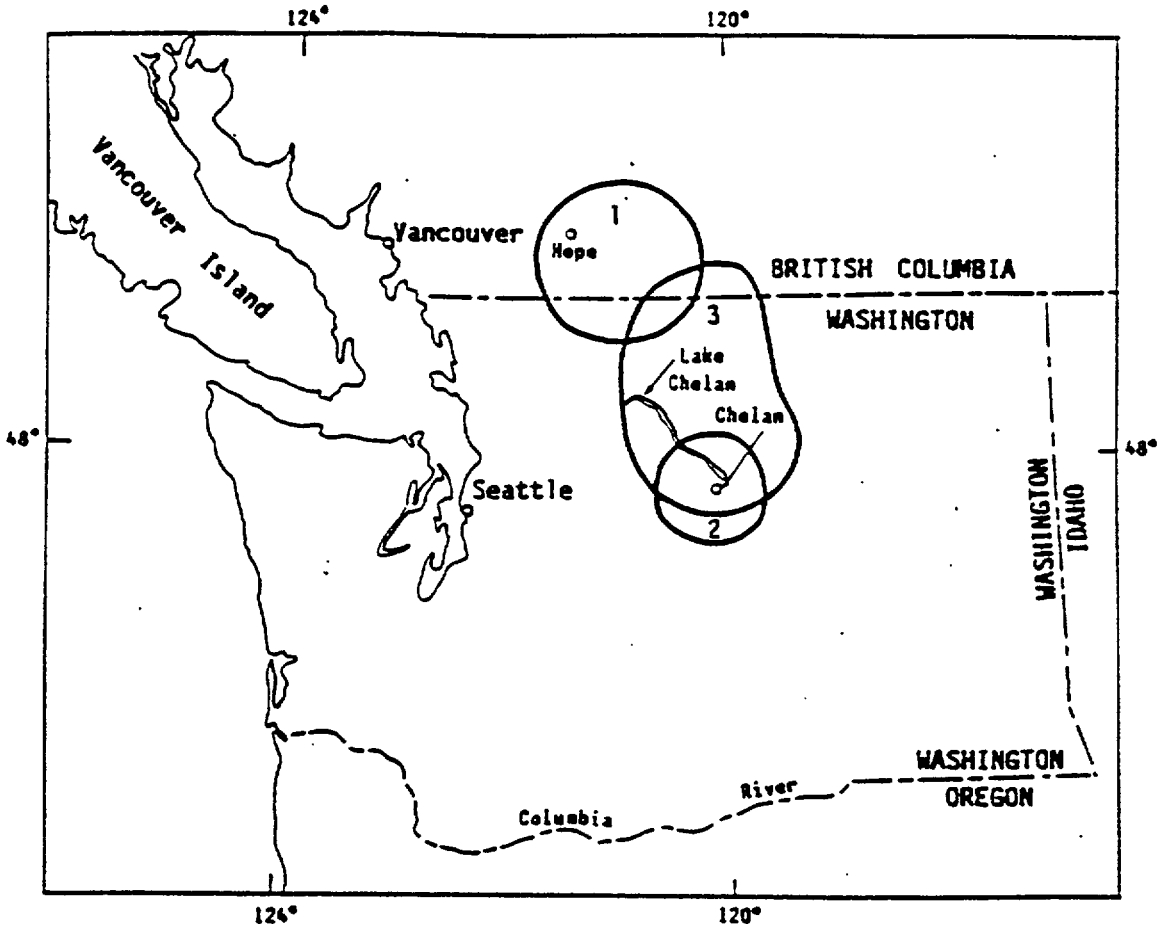
R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

MAP OF GEOMORPHIC PROVINCES

Checked by	Date	Project No.	Figure No.
Approved by RLV	Date 12/12/94	3M1-100	A-1

L.C-53



EXPLANATION

Sources of proposed epicentral areas:

1. Milne (1956)
2. Puget Sound Power and Light (1975, 1976, 1977)
3. Coombs and others (1976)

N

0 50 100 Miles

0 50 100 Kilometers

From Geomatrix, 1990

R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

MAP SHOWING EPICENTRAL AREAS PROPOSED FOR THE 1872 EARTHQUAKE

Checked by	Date	Project No.	Figure No.
Approved by RCV	Date 12/12/94	SMC-100	A-2

L.C-54

RLV-A

APPENDIX B

PROBABILITY OF RANDOM EARTHQUAKES

APPENDIX B

Probability of Random Earthquakes

The occurrence of a random earthquake appears to be the dominant seismic event to potentially impact the Sherwood Tailing Impoundment. This appendix presents the basic considerations that impact the probability calculations for random events.

Assuming that the occurrence of independent earthquakes is a Poisson process, then the probability of no events of magnitude m ± Δm in time interval T is given by:

$$P=e^{-\lambda(m)T} \tag{1}$$

where λ(m) is the annual frequency of events of magnitude m ± Δm. Assuming that the spatial distribution of earthquakes in the region around the site is uniform, the annual frequency of events within a circle of radius R about the site is obtained from a truncated exponential earthquake recurrence relationship by the expression:

$$\lambda(m)=\pi R^2 N(m^0)\frac{10^{-b(m-\Delta m-m^0)}-10^{-b(m+\Delta m-m^0)}}{1-10^{-b(m^u-m^0)}} \tag{2}$$

where $N(m^0)$ is the cumulative annual frequency per unit area of events having magnitudes greater than a minimum magnitude m^o, b, is the b-value of the recurrence relationship, and m^u is the maximum event size in the region. Equations (1) and (2) can be used to obtain the radius of a circle about site within which a magnitude m event has a specified probability of <u>not</u> occurring in time T.

Equation (2) is based on the assumption of a uniform spatial distribution of seismicity around the site. However, as indicated in Chapter II, the seismicity in the site region is characterized by several seismic zones with different rates of activity. Accordingly, Equation (2) was modified to include the contributions of multiple zones. The modified relationship is defined as follows:

$$\lambda(m|R=r)=\sum N_i(m^0)A_iP_i(R\le r)\frac{10^{-b_i(m-\Delta m-m^0)}-10^{-b_i(m+\Delta m-m^0)}}{1-10^{-b_i(m^u{}_i-m^0)}} \tag{3}$$

where λ(m | R=r) is the annual frequency of events of magnitude m ± Δm within a radius of R=r around the site and the subscripts i refer to the recurrence parameters for the ith zone. A_i is the total area of zone i and $P_i(R\le r)$ is the cumulative probability distribution for distance from the site to a random event in zone i.

Equation (3) was used to compute appropriate radii for the specified probabilities of nonoccurrence using the recurrence parameters for the various seismic zones impacting the site (see Table B-1). Three sets of parameters are listed in Table B-1, corresponding to the three criteria used for identifying independent earthquakes. These criteria include both time and distance windows and are graphically shown in Fig. B-1.

Earthquake recurrence relationships were developed for the seismotectonic provinces surrounding the site. The recurrence relationships are based on the maximum likelihood method developed by Weichert (1980). Fig. B-2 shows the recurrence relationships developed for the Palouse Subprovince and the Okanogan Uplands. The figure shows the cumulative annual earthquake frequencies obtained using all events and the cumulative frequency of independent events as determined by the three criteria for identifying dependent events shown in Fig. B-1. The relationship was fit to data for magnitude 2.0 and larger since it appeared that smaller magnitudes are not reported completely in many of the zones. The fitted recurrence relationships are presented on Table B-1.

Epicentral distances were computed using each set of recurrence parameters and the three distances averaged to obtain a single estimate. The computations were made using Δm = ¼ magnitude units and m^u was taken to be $m_{max} + \Delta m$, with m_{max} the assessed maximum magnitude for the seismic zone. Table 3 in Section II of the main body of the report lists the resulting average epicentral distances for random events of magnitude 5.0 to 6.5. The 90% confidence intervals reported in the table reflect the uncertainty in selecting aftershock identification criteria and the uncertainty in the recurrence parameter estimates listed in Table B-1.

The principal results of the site specific probabilistic analysis of random earthquake occurrence at the Sherwood site are presented in two different formats in Figs. B-3 and B-4. Fig. B-3 presents the results in terms of the most likely epicentral radii vs. annual probability of non-occurrence for earthquake magnitudes ranging from 5.0 to 6.5. Fig. B-4, on the other hand, shows the annual frequency of occurrence of random events versus distance from the site.

It should be noted that random earthquakes having magnitudes less than the maximum credible random earthquake (M 6½) have smaller epicentral radii (Table 3) than the random MCE. There will be less attenuation of the earthquake ground motions due to the shorter source-to-site distance to these smaller events. Consequently, random events smaller that the random MCE can produce peak ground accelerations greater than the random MCE. However, smaller events will tend to produce higher frequency ground motions, and the duration of intense shaking will tend to be shorter. Due to the relatively long natural period for earthen structures, even though the peak ground acceleration for these smaller random events could be higher than the maximum random events, it does not appear that they will pose a greater hazard.

Table B-1

Earthquake Recurrence Parameters

Zone [1]	Parameter [2]	All Events	Aftershock Criteria [3] 1	2	3
Yakima Fold Belt Subprovince	N(m°=2)	6.99(±0.49)	5.44(±0.42)	5.20(±0.42)	4.20(±0.37)
	b	1.048(±0.056)	0.995(±0.058)	0.991(±0.059)	0.945(±0.061)
	a	2.94	2.73	2.70	2.51
Palouse Subprovince	N(m°=2)	3.05(±0.32)	2.87(±0.31)	2.76(±0.31)	2.39(±0.28)
	b	0.999(±0.079)	0.999(±0.079)	0.999(±0.079)	0.999(±0.079)
	a	2.48	2.45	2.42	2.28
Okanogan Uplands	N(m°=2)	0.86(±0.17)	0.86(±0.17)	0.82(±0.16)	0.82(±0.16)
	b	1.043(±0.16)	1.043(±0.16)	1.025(±0.159)	1.025(±0.159)
	a	2.02	2.02	1.96	1.96
Northern Cascades	N(m°=2)	9.3(±0.6)	7.4(±0.5)	6.5(±0.5)	4.5(±0.4)
	b	1.148(±0.05)	1.094(±0.053)	0.921(±0.075)	0.918(±0.053)
	a	3.26	3.06	3.72	2.49

Notes:

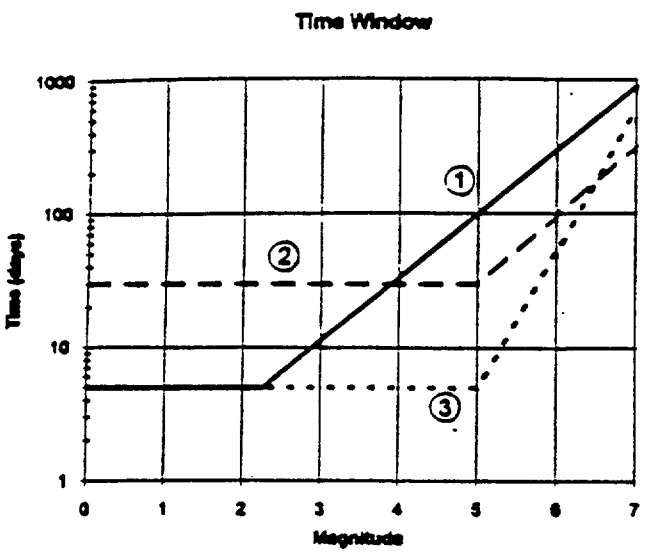
(1) Data from Northern Cascades Zone from Geomatrix Consultants (1988). All other data from Geomatrix Consultants (1990).

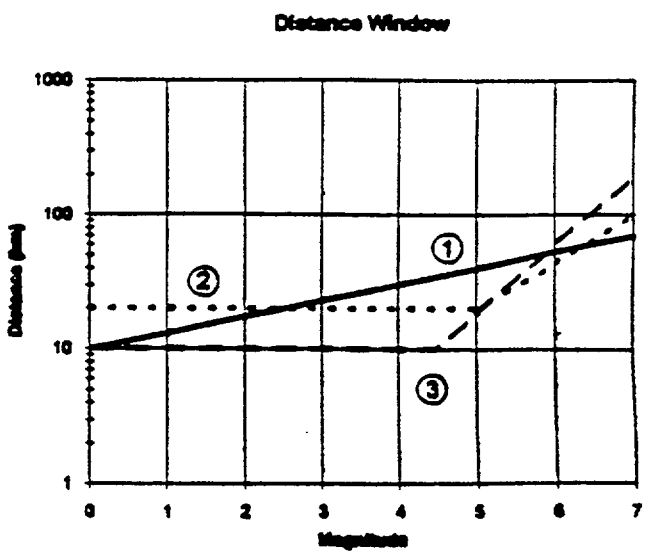
(2) Parameter definitions are as follows:

N(m°=2) = annual number of events of magnitude 2 and greater per 10,000 km^2
b = slope of magnitude vs. log frequency plot
a= Log_{10} N(m°=0) in events per year per 10,000 km^2.

(3) See Fig. B-1 for graphical representation of various aftershock criteria used to identify dependent earthquake events.

L.C-52





Criteria		Time Window		Distance Window
①	=	Arabasz & Robinson (1976)	+	Wyss (1979)
②	=	Gardner & Knopff (1974)	+	Gardner & Knopff (1974)
③	=	Uhrhammer (1986)	+	Uhrhammer (1986)

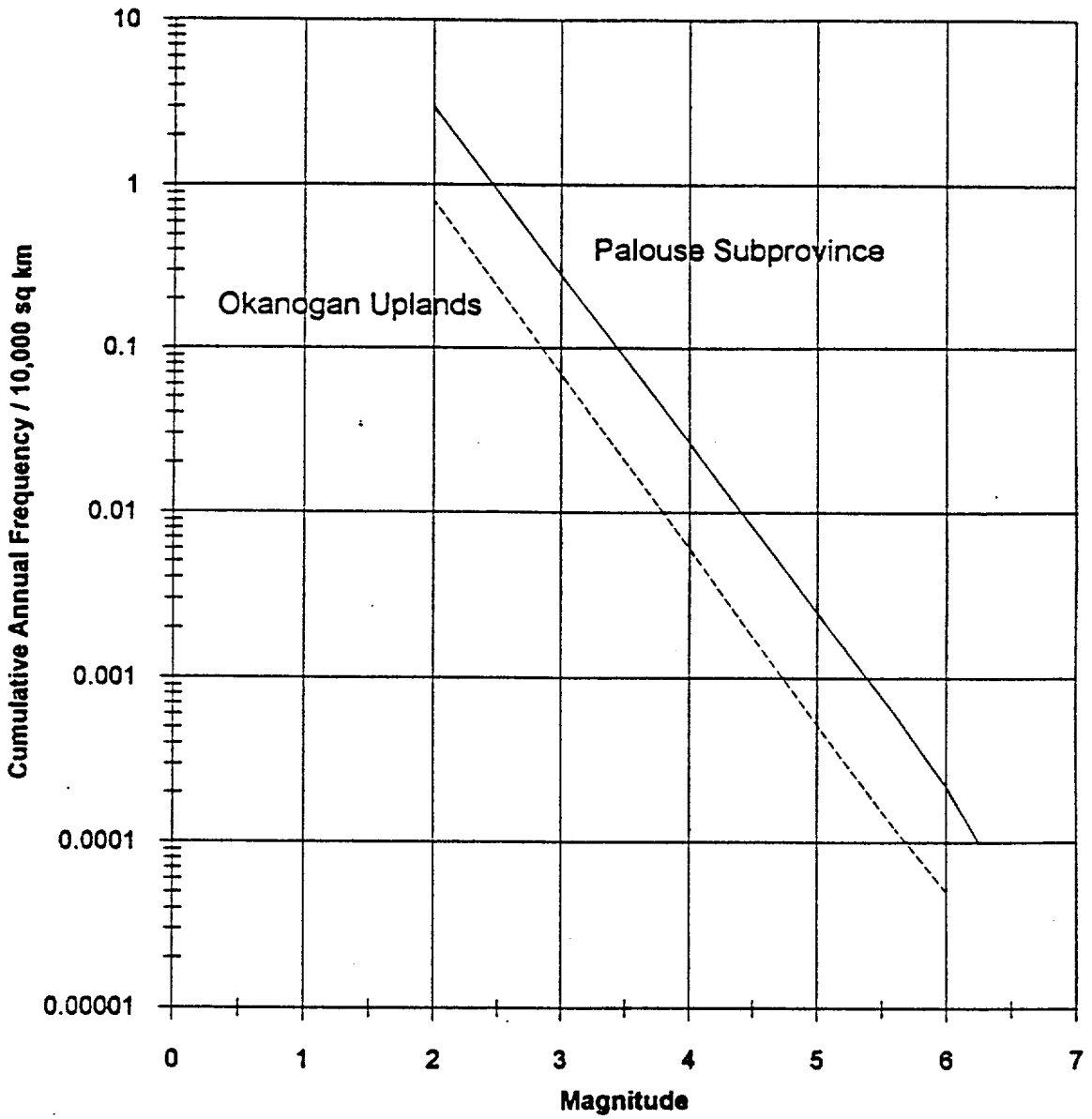
From Geomatrix, 1990

R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

CRITERIA USED TO IDENTIFY DEPENDENT EARTHQUAKE EVENTS

Checked by	Date	Project No.	Figure No.
Approved by RCU	Date 12/8/94	SM1-600	B-1



From Geomatrix, 1990

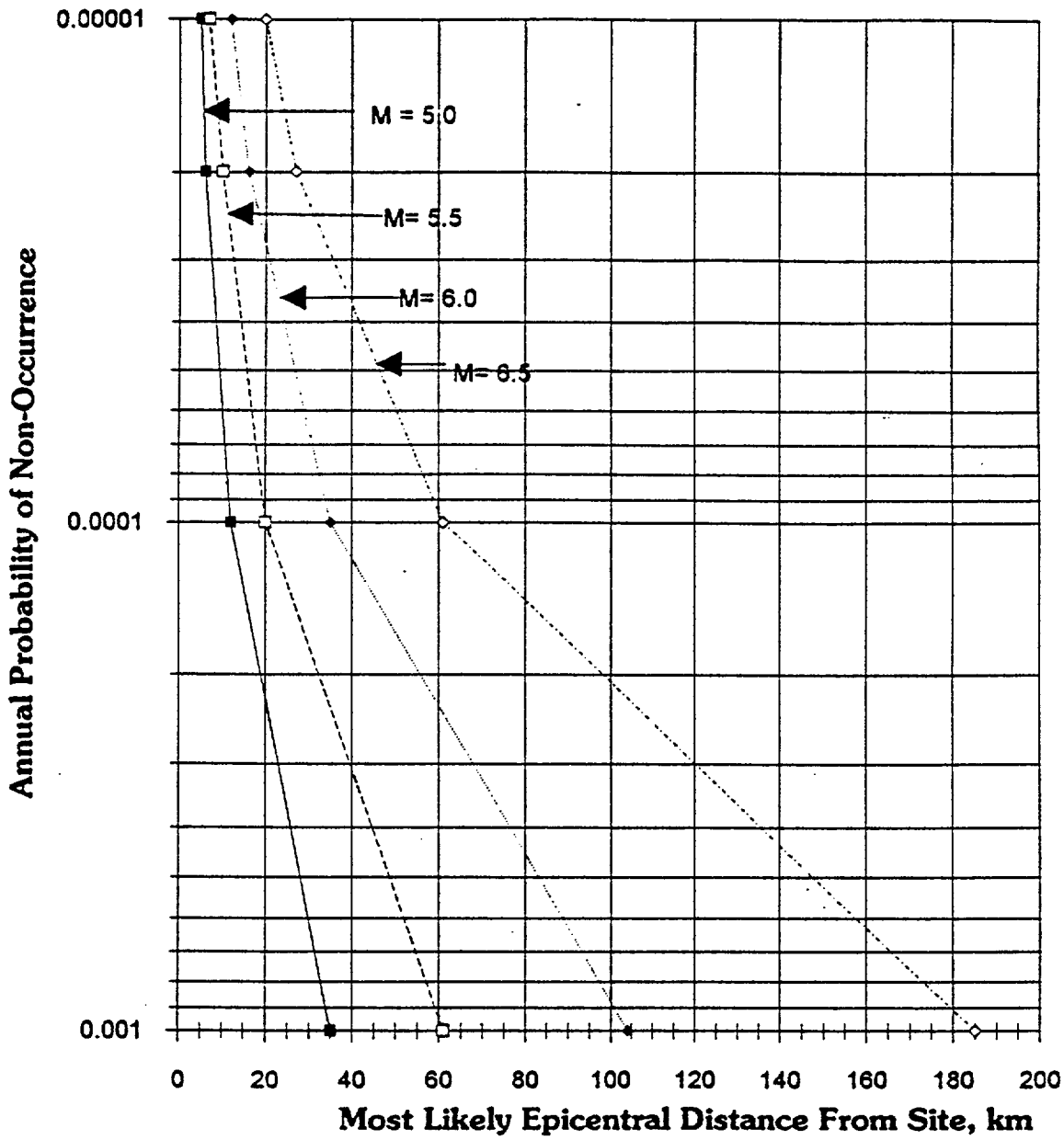
R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

CUMULATIVE ANNUAL FREQUENCY vs. MAGNITUDE

Checked by _____	Date _____	Project No.	Figure No.
Approved by RLV	Date 12/12/94	SM1-100	B-2

L.C-60



From Geomatrix, 1990

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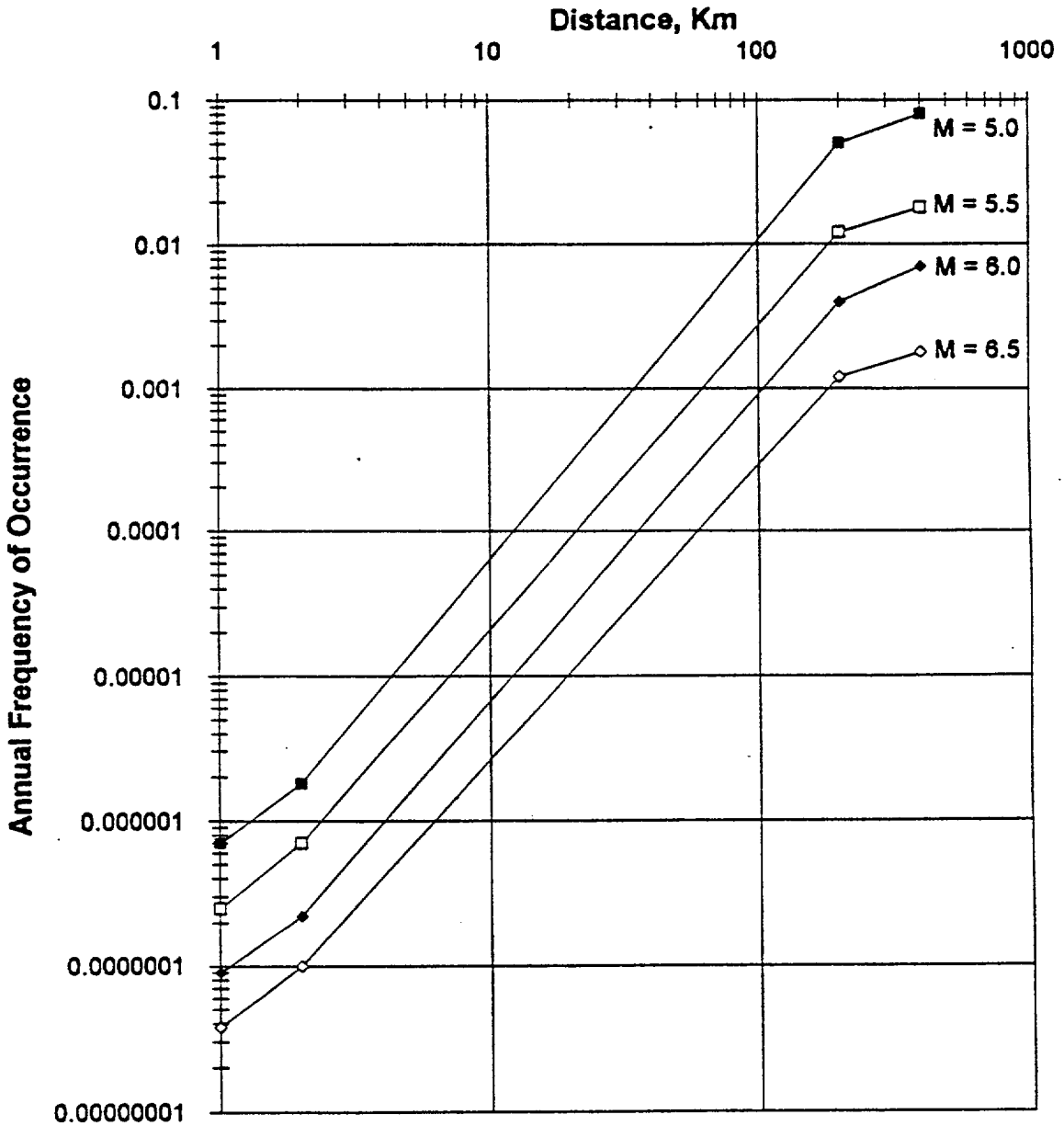
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

PROBABILITY OF NON-OCCURRENCE vs. MOST LIKE EPICENTRAL DISTANCE

Checked by	Date	Project No.	Figure No.
Approved by RLV	Date 12/12/94	SML-100	B-3

L.C-61



From Geomatrix, 1990

R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

ANNUAL FREQUENCY OF OCCURRENCE vs. DISTANCE

Checked by	Date	Project No.	Figure No.
Approved by RLV	Date 12/12/94	SML-100	B-4

APPENDIX C

SUMMARY OF LABORATORY TEST RESULTS USED FOR LIQUEFACTION ASSESSMENT

This appendix presents the results of laboratory test performed on thin wall tube samples from borings at the Sherwood Tailing Impoundment. The laboratory tests were performed by SMI during their original field investigation of September 1991. The first sheet is a convenient summary of laboratory tests and other computed engineering parameters computed from the basic data. Following the first sheet are plots (Figs. C-1 through C-7) of percentage of fines vs. depth for those holes in which tests were performed. Percentage of fines vs. depth relationships are presented since this value controls the classification of the sample. Full gradation test results are presented in Appendix A of the Tailing Reclamation Plan. Two items should be noted about the referenced figures: 1) most of the gradation results are not summarized on the first data summary sheet since the samples tested for gradation were obtained from Standard Penetration Test samples and no other engineering tests were performed due to sample disturbance; 2) although the figures show a solid line connecting consecutive individual data points, this is not meant to infer that we assume the percentage of fines, at any depth not sampled, would be equal to that value shown by the solid line.

L.C-63

SHEPHERD MILLER
INCORPORATED

July 13, 1993

Mr. Richard L. Volpe
R L Volpe and Associates
110 Atwood Court
Los Gatos, CA 95032

Dear Dick:

Enclosed is the information we discussed for the Sherwood uranium tailing reclamation project. Specifically, you will find the following:

1. A site map.

2. Logs of the holes drilled in the tailings.

3. Sketches showing the proposed cover configuration and a generalized cross section through the tailing impoundment.

I am looking forward to working with you again. Please call if you have any questions.

Sincerely,

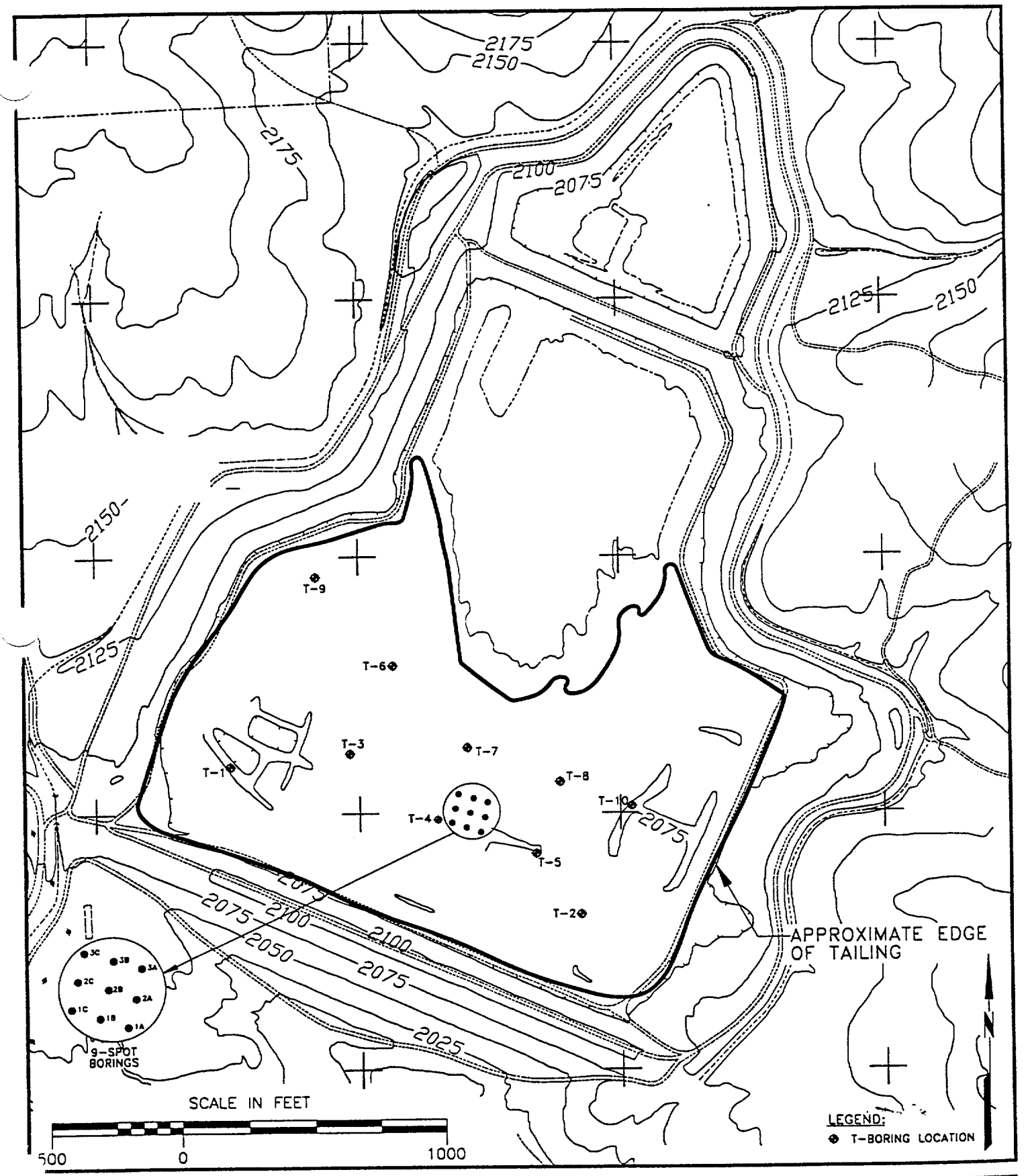
SHEPHERD MILLER, INC.

Louis Miller
Principal

enclosures

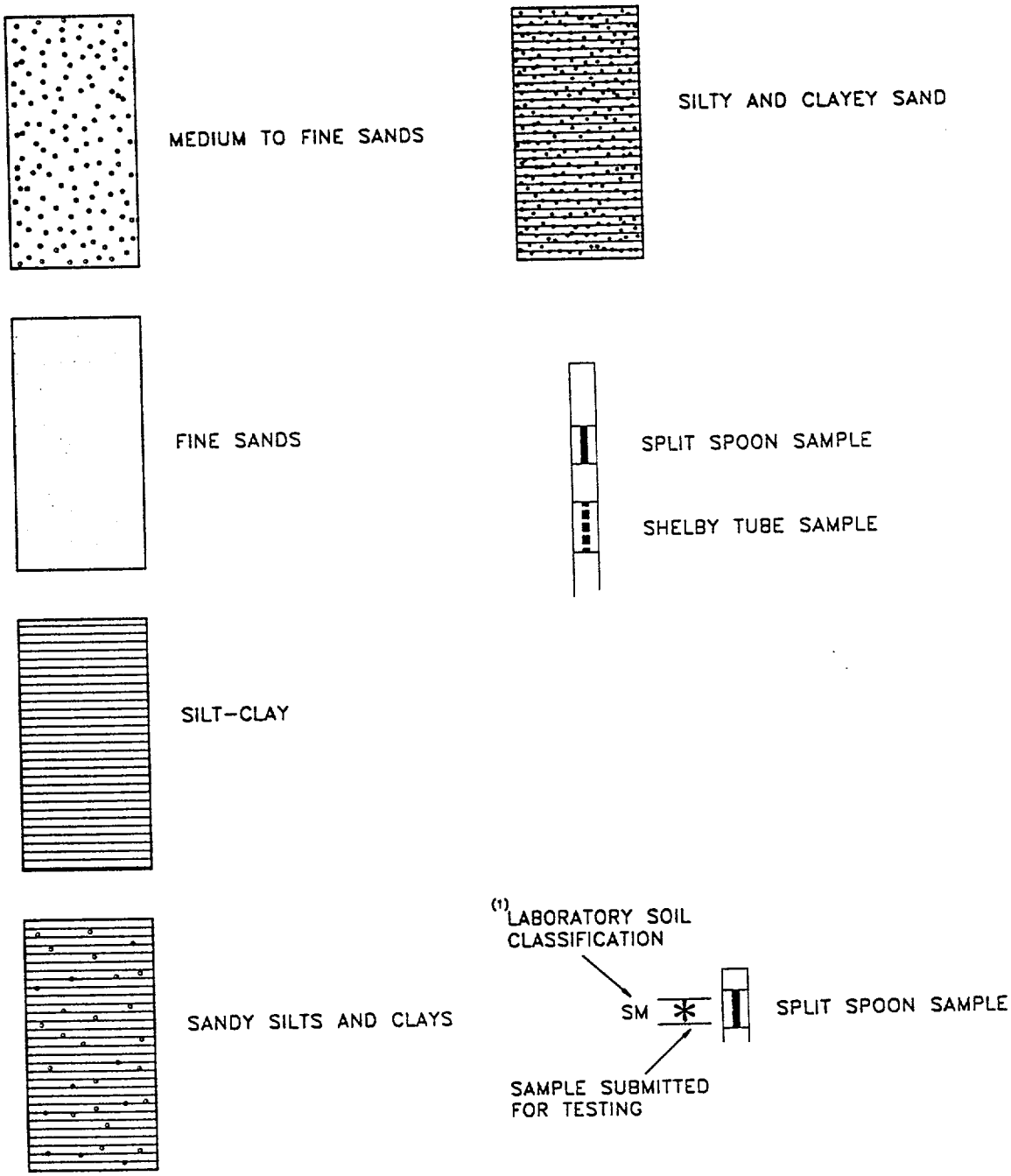
Consulting Environmental & Geotechnical Engineers

1600 Specht Point Dr., Suite F
Fort Collins, CO 80525
Phone (303) 484-1414
Fax (303) 484-7540



SMI SHEPHERD MILLER, INC.	BROING LOCATIONS	Date: JUNE 1993 Project: 317 File: LOCBOR

L.C-65



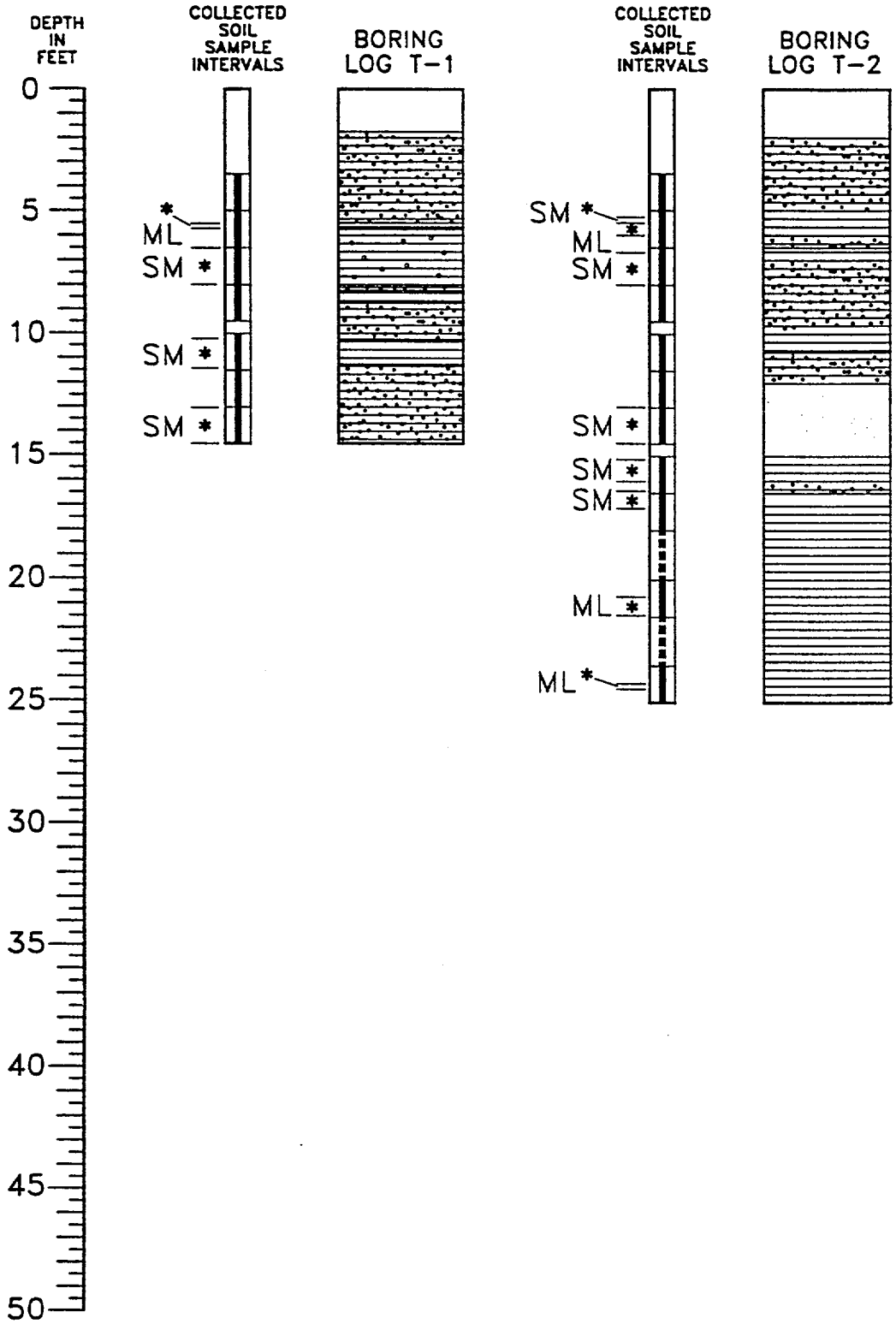
(1) SOME LABORATORY CLASSIFICATIONS VARY FROM FIELD IDENTIFICATIONS SHOWN ON BORING LOGS.

SMI
SHEPHERD MILLER, INC.

FIGURE 4.2
KEY TO TAILING LOGS

Date:	JUNE 1993
Project:	307
File:	KEY-T

L.C-66

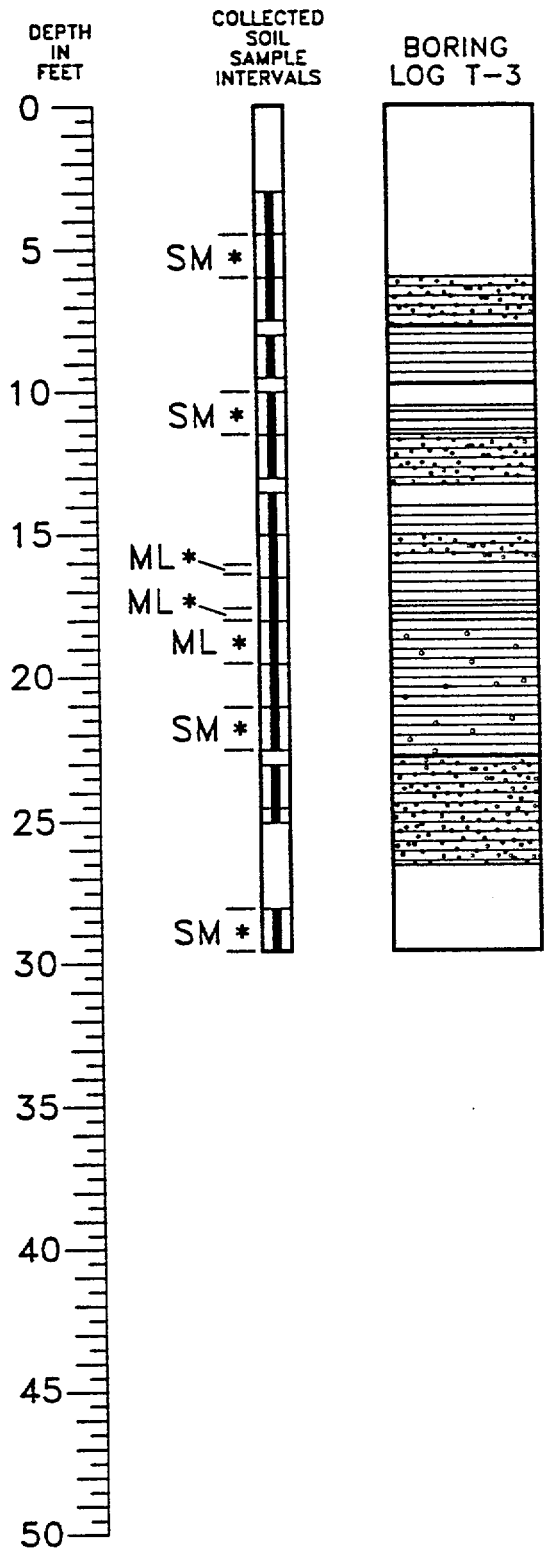


* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.3
TAILING BORING LOGS
T-1 AND T-2

Date:	JUNE 1993
Project:	307
File:	BLOG-T12



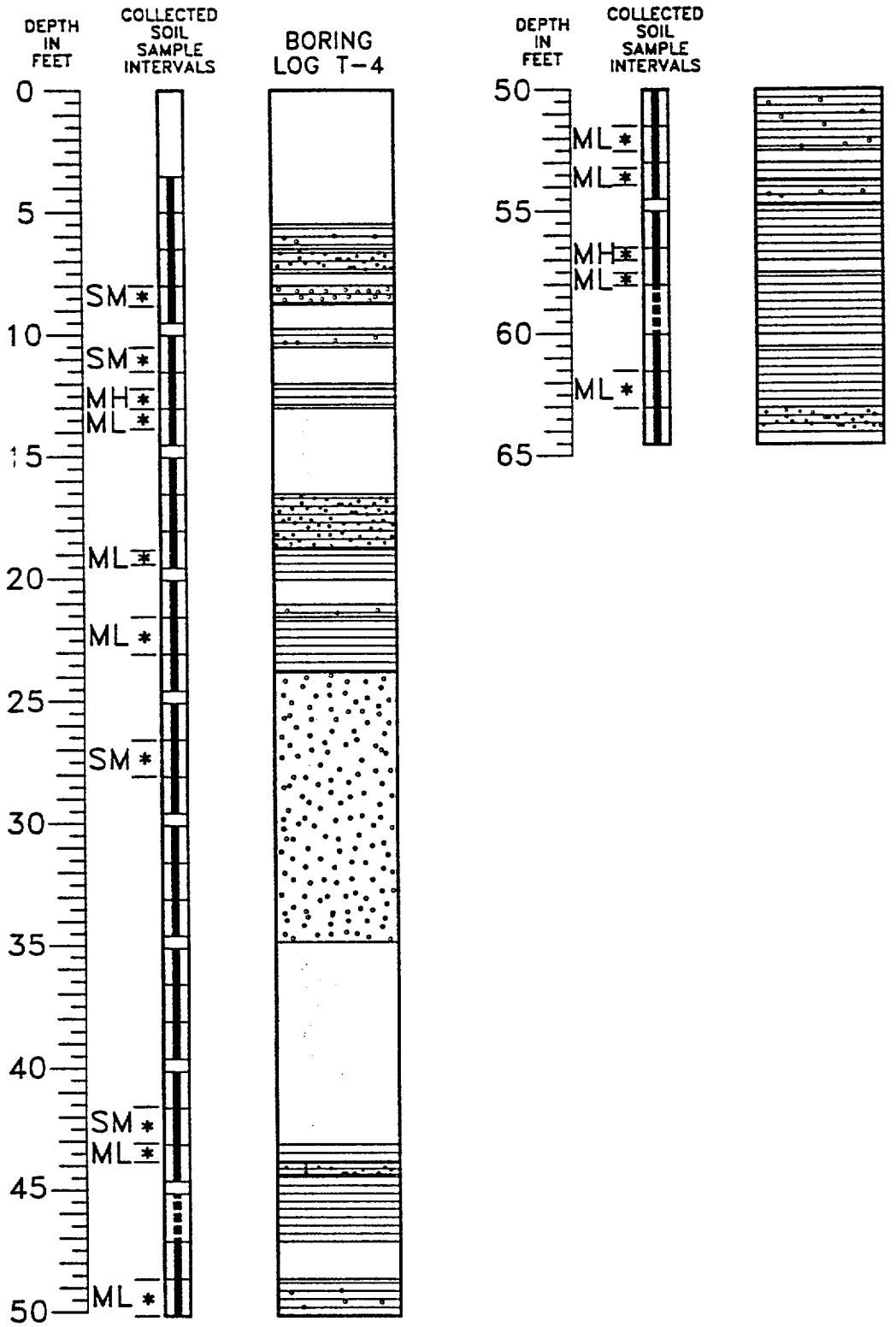
* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.4
TAILING BORING LOG
T-3

Date:	JUNE 1993
Project:	307
File:	BLOG-T3

L.C-62



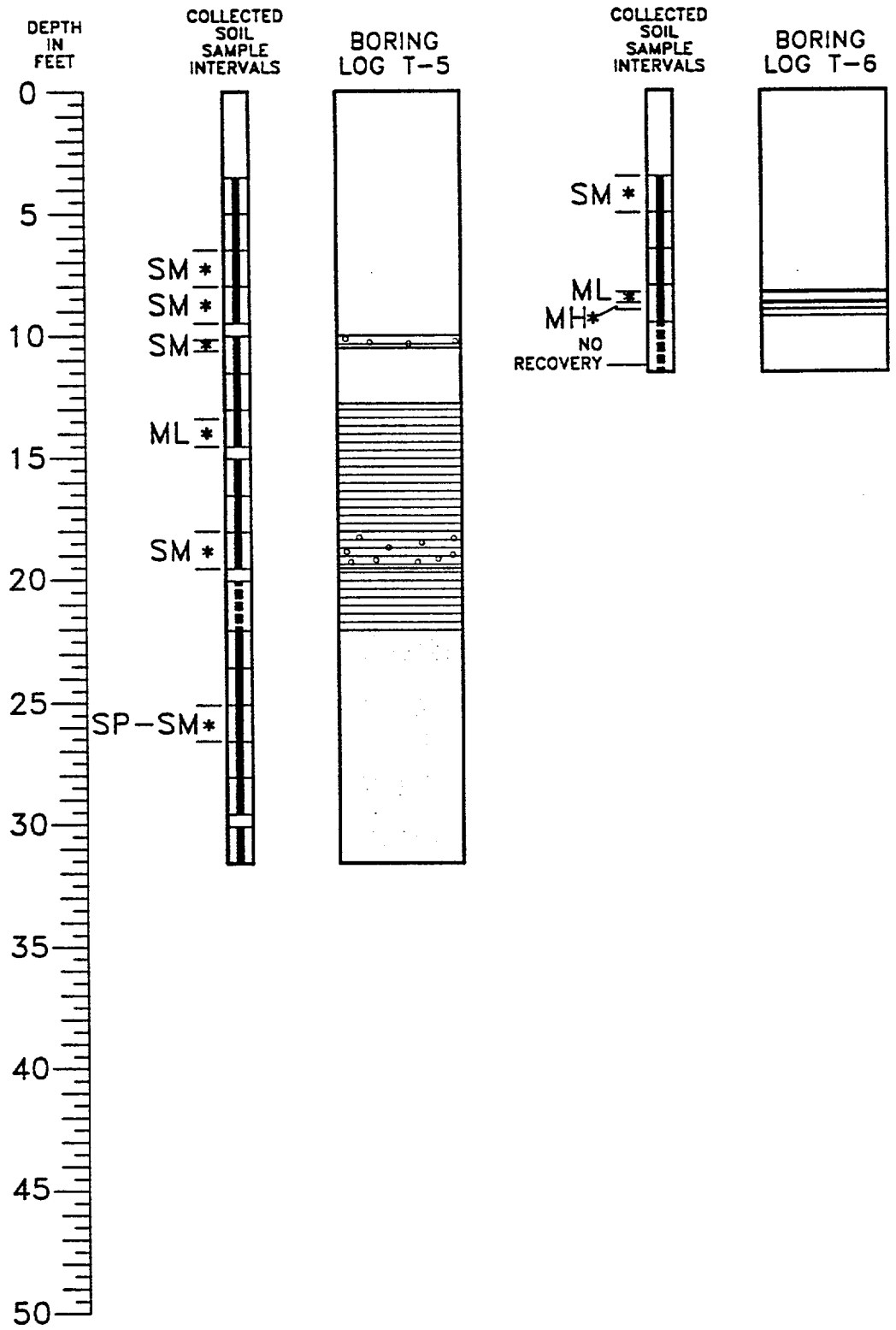
* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.5
TAILING BORING LOG
T-4

Date:	JUNE 1993
Project:	307
File:	BLOG-T4

L.C-51

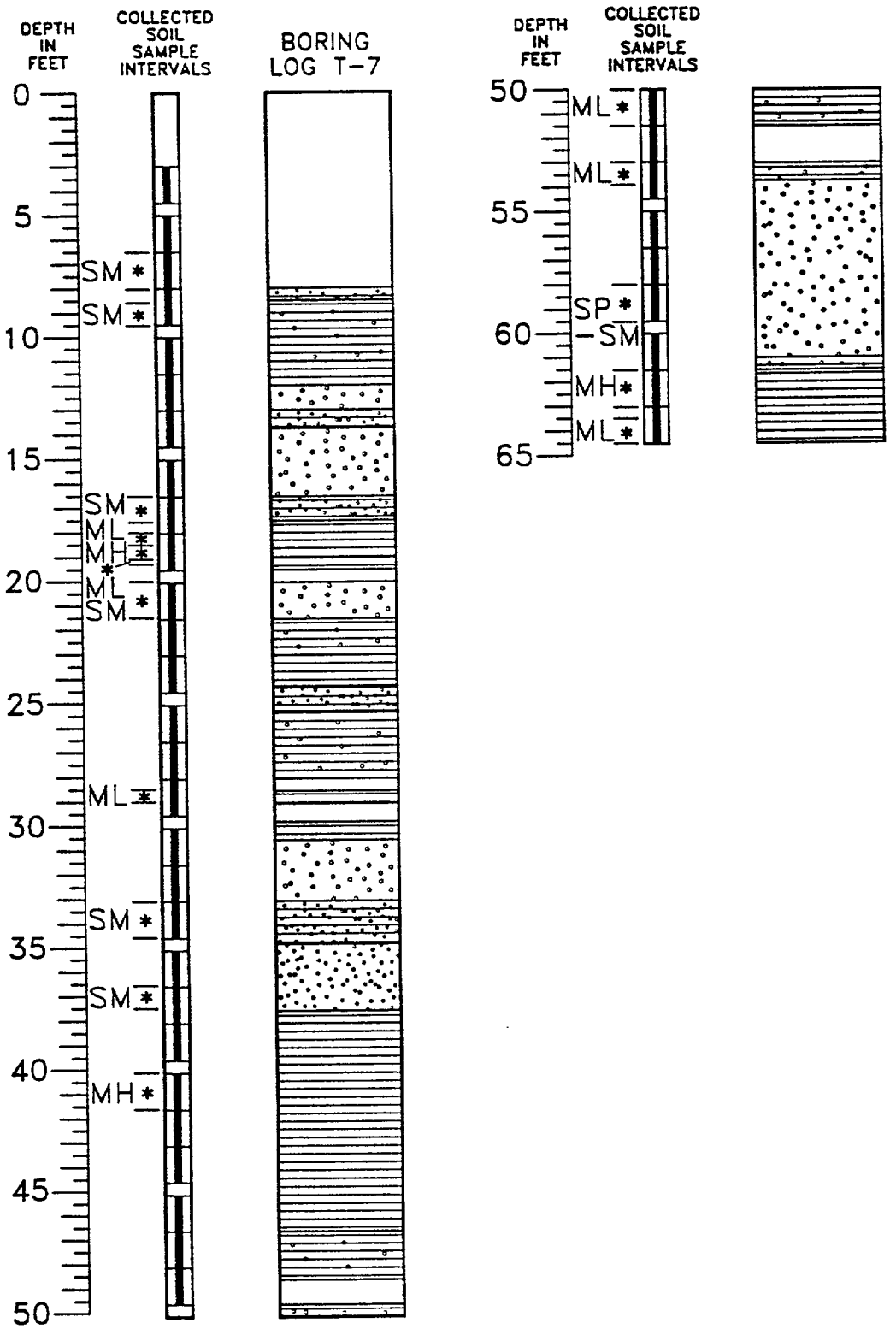


* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.6
TAILING BORING LOGS
T-5 AND T-6

Date:	JUNE 1993
Project:	307
File:	BLOG-T56

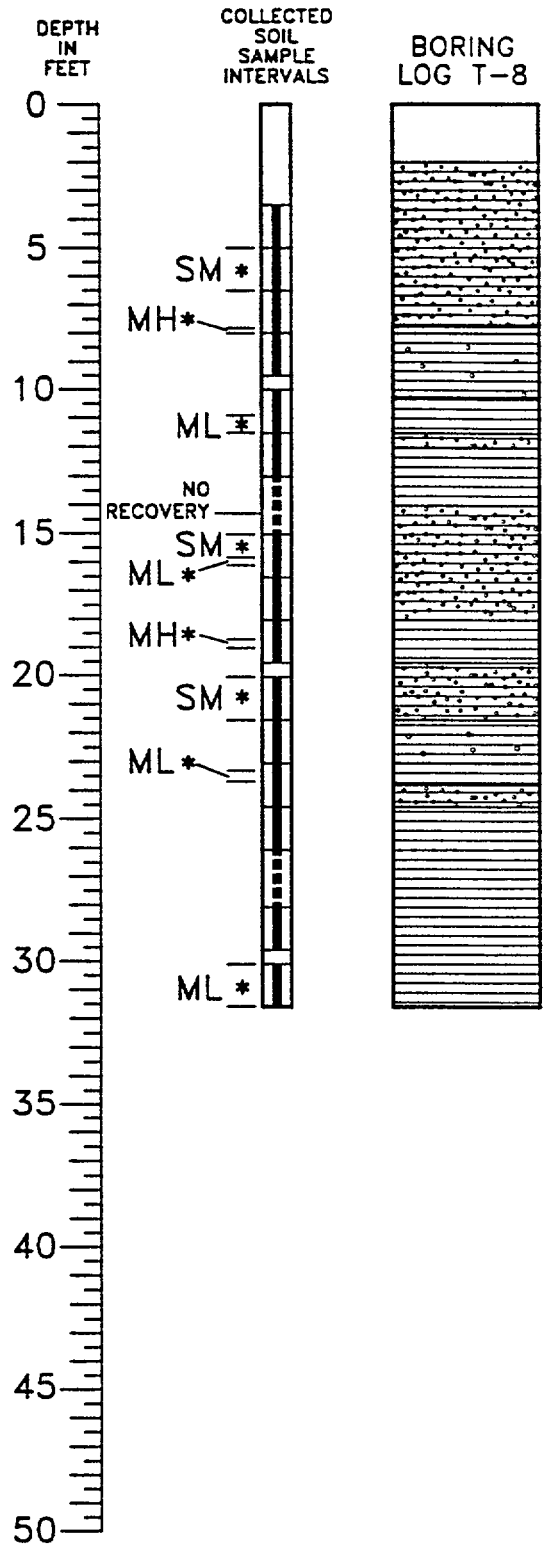


* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.7
TAILING BORING LOG
T-7

Date:	JUNE 1993
Project:	307
File:	BLOG-T7

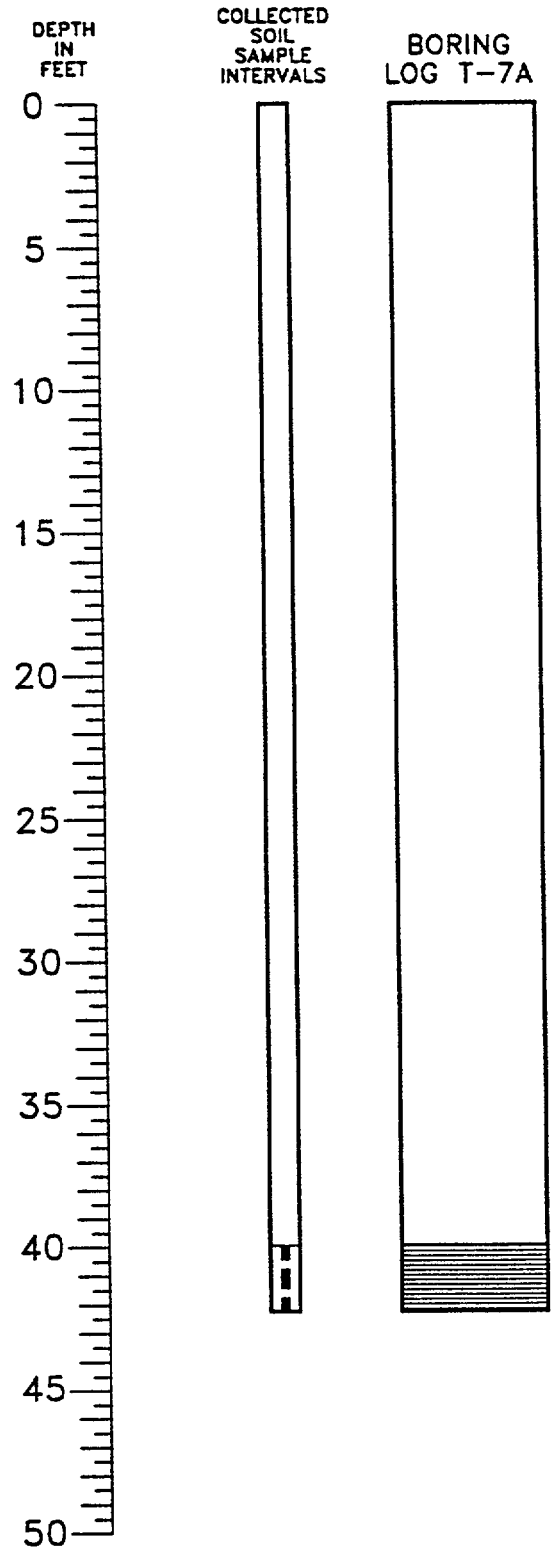


* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.9
TAILING BORING LOG
T-8

Date:	JUNE 1993
Project:	307
File:	BLOG-T8

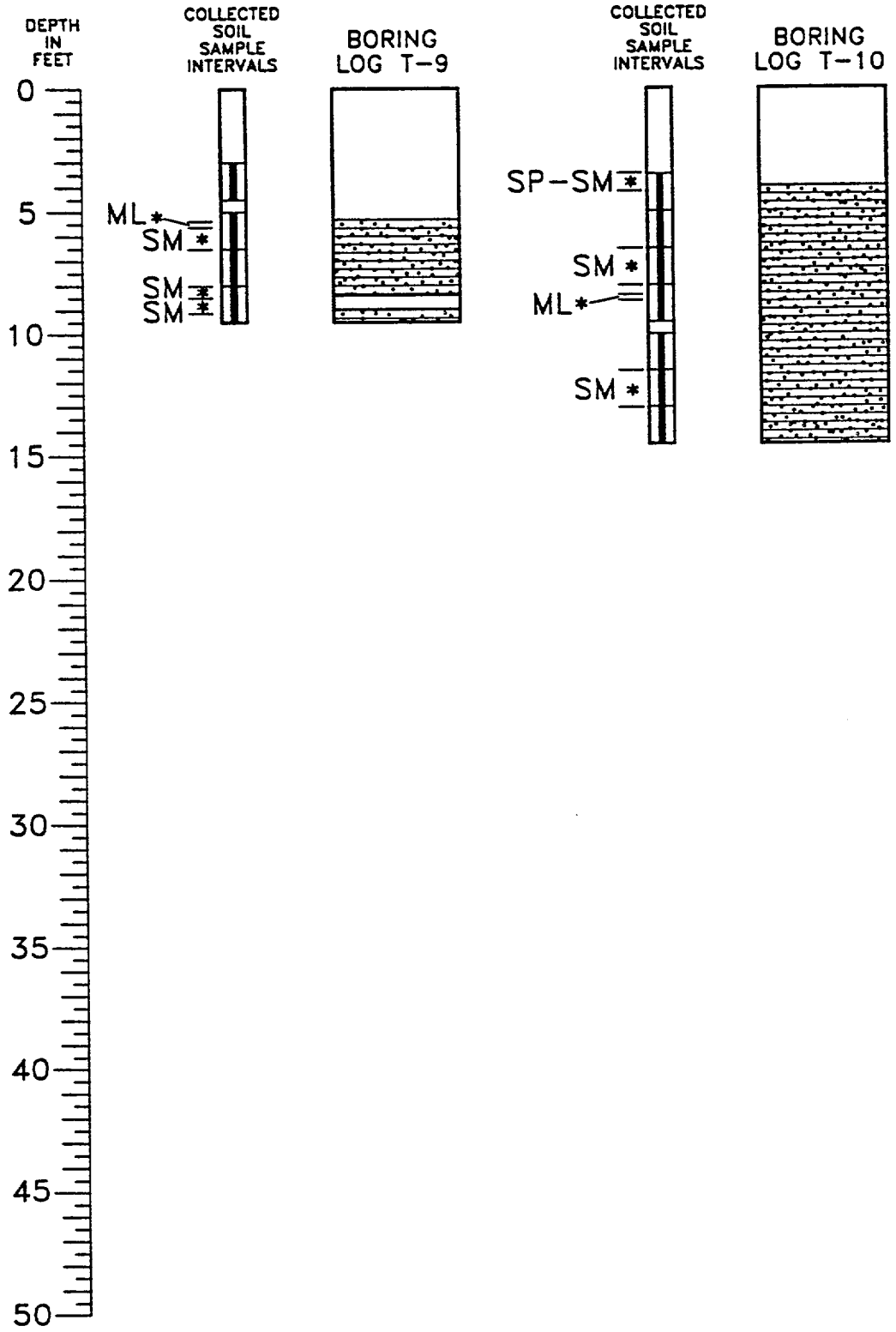


* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

FIGURE 4.8
TAILING BORING LOG
T-7A

Date:	JUNE 1993
Project:	307
File:	BLOG-T7A



* SAMPLE SUBMITTED FOR TESTING

SMI
SHEPHERD MILLER, INC.

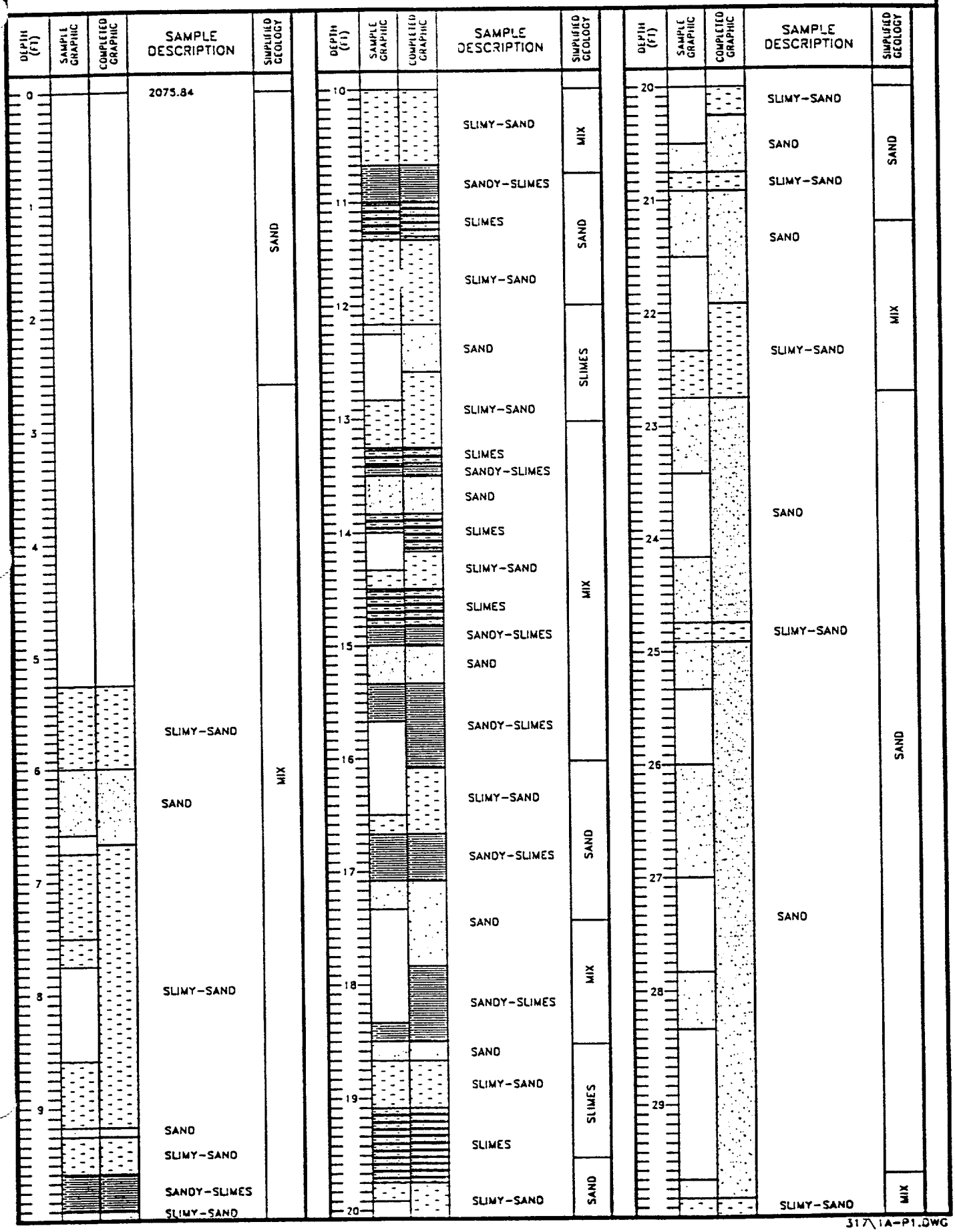
FIGURE 4.10
TAILING BORING LOGS
T-9 AND T-10

Date:	JUNE 1993
Project:	307
File:	BLOG-T90

WELL COMPLETION LOG

MONITORING WELL NO: 1A

LOGGED BY: JGC



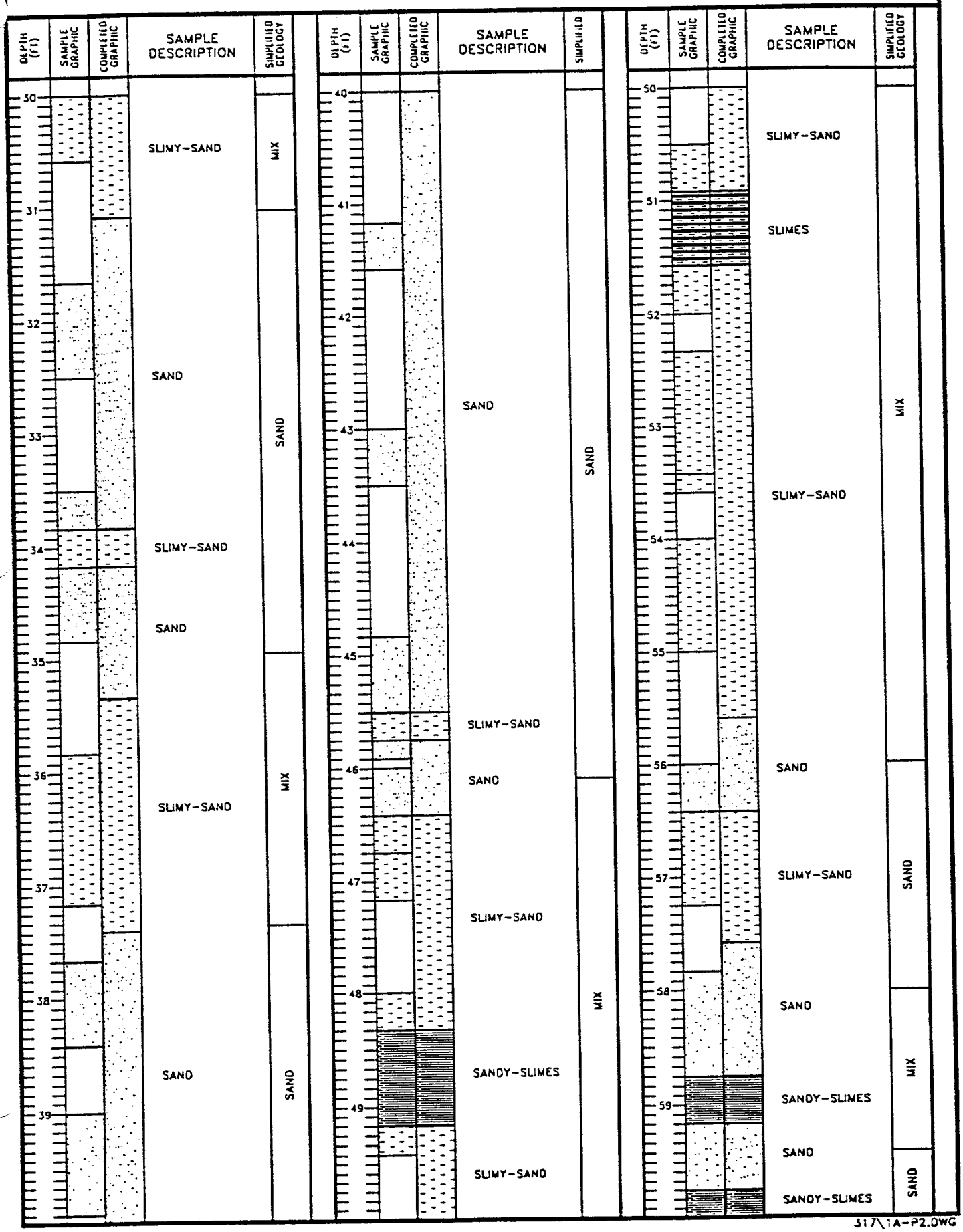
317\1A-P1.DWG

LC-7E

WELL COMPLETION LOG

MONITORING WELL NO: 1A

LOGGED BY: JGC



317\1A-P2.DWG

L.C-74

WELL COMPLETION LOG

MONITORING WELL NO: 1A

LOGGED BY: JGC

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60			SANDY-SLIMES	SAND
61			SAND	MIX
62			SANDY-SLIMES	MIX
63			SLIMY-SAND	MIX
64			SAND	MIX
65			SANDY-SLIMES	SAND
66			SAND	MIX
67			SANDY-SLIMES	MIX
68			SLIMY-SAND	MIX
69			SANDY-SLIMES	SLIMES

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70			SANDY-SLIMES	SLIMES
71			SLIMES	SLIMES
72				MIX
73				MIX
74				MIX
75				MIX
76				SLIMES
77				SAND
78				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

317\1A-P3.DWG

WELL COMPLETION LOG

MONITORING WELL NO: 2A

LOGGED BY: JGC

DEPTH (FT)	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
0	2073.2	MIX
5	SLIMY-SAND	SAND
6	SAND	MIX
7	SLIMY-SAND	
9	SAND	SAND
10	SLIMY-SAND	MIX
11	SLIMES	
12	SLIMY-SAND	
13	SLIMES SLIMY-SAND SLIMES	SLIMES
14	SLIMY-SAND	MIX
15	SLIMES SLIMY-SAND SLIMES SLIMY-SAND SLIMES	
16	SLIMY-SAND	
17	SANDY-SLIMES SLIMY-SAND SANDY-SLIMES SLIMY-SAND SLIMES	SLIMES
18	SLIMY-SAND	MIX
19	SLIMES SLIMY-SAND	
20	SLIMY-SAND	MIX
21	SLIMES SANDY-SLIMES	
22	SLIMY-SAND	
23	SAND	
24	SLIMY-SAND	SAND
25	BLACK SAND	MIX
26	SLIMY-SAND	
27		
28	SAND	
29	SLIMY-SAND SAND SLIMY-SAND SAND SANDY-SLIMES	SAND MIX

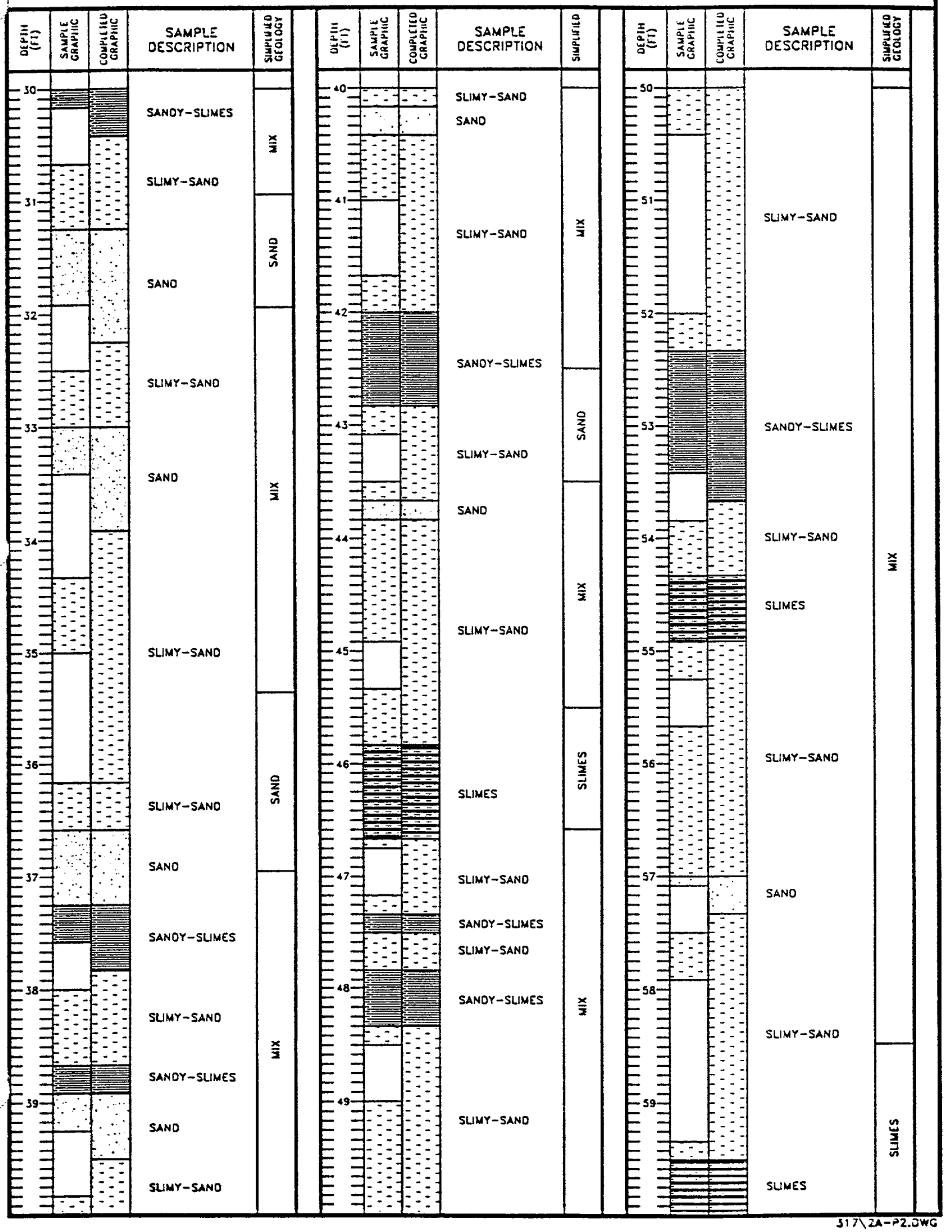
317\2A-P1.DWG

L.C-72

WELL COMPLETION LOG

MONITORING WELL NO: 2A

LOGGED BY: JGC



317\2A-P2.DWG

L.C-79

WELL COMPLETION LOG

MONITORING WELL NO: 2A

LOGGED BY: JGC

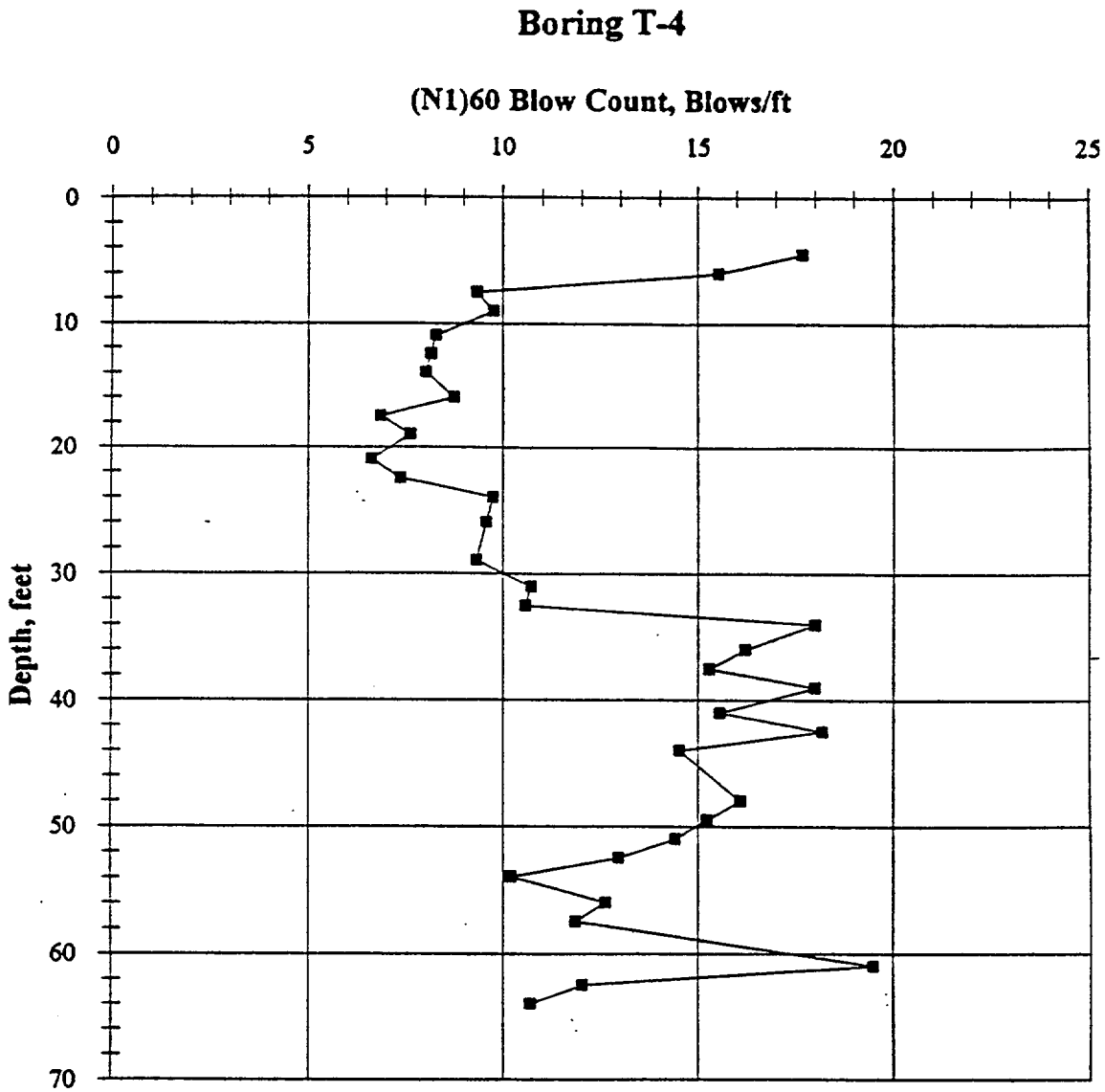
DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY
60				
			SLIMES	SLIME
61			SLIMY-SAND	
62			SLIMES	SAND
63			SAND	SLIMES
64				
			SLIMES	
65				
			SAND	
66				
67			SLIMES	
68				SAND
69				MIX

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED
70				
				MIX
71				
72				SLIMES
73				
74				
75				
66				

DEPTH (FT)	SAMPLE GRAPHIC	COMPLETED GRAPHIC	SAMPLE DESCRIPTION	SIMPLIFIED GEOLOGY

317\2A-P3.DWG

L.C-141



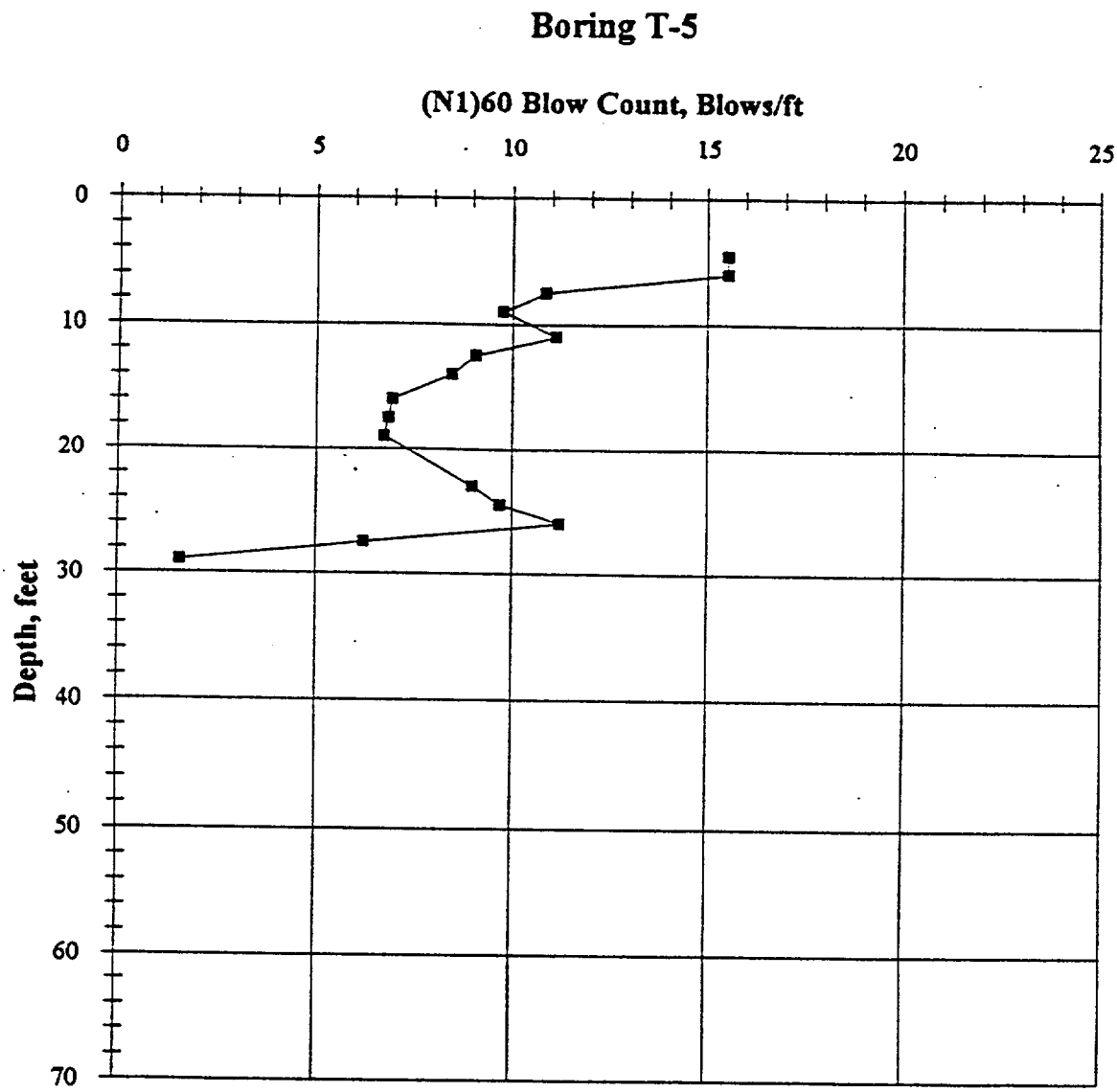
R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

$(N_1)_{60}$ BLOW COUNT vs. DEPTH
BORING T-4

Checked by _____	Date _____	Project No.	Figure No.
Approved by _____	Date _____	SM-100	D-14

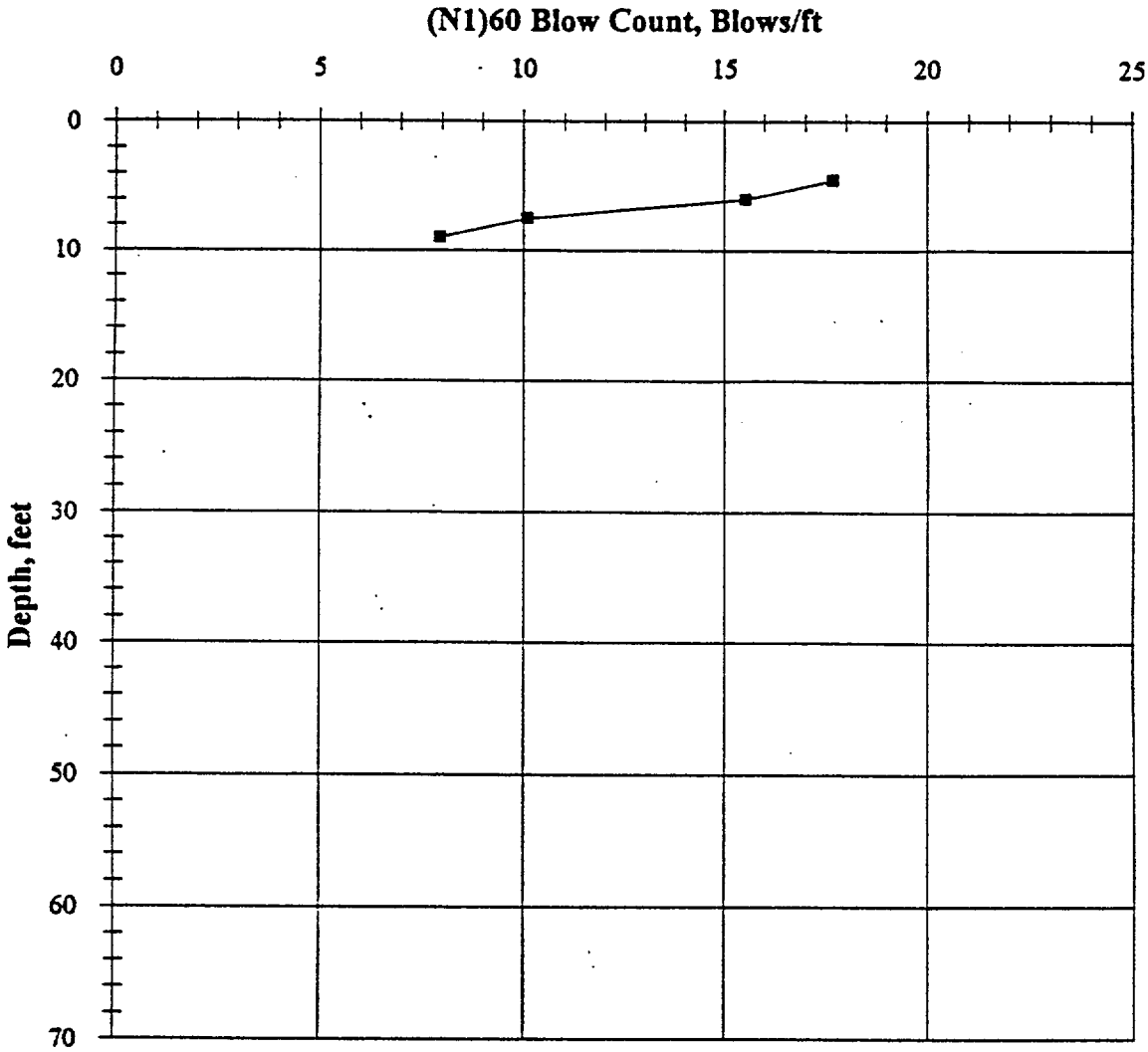
L.C-142



R.L. VOLPE & ASSOCIATES Los Gatos, California		
SHERWOOD TAILING IMPOUNDMENT $(N_1)_{60}$ BLOW COUNT vs. DEPTH BORING T-5		
Checked by _____ Date _____ Approved by _____ Date _____	Project No. SM-100	Figure No. D-15

L.C-143

Boring T-6



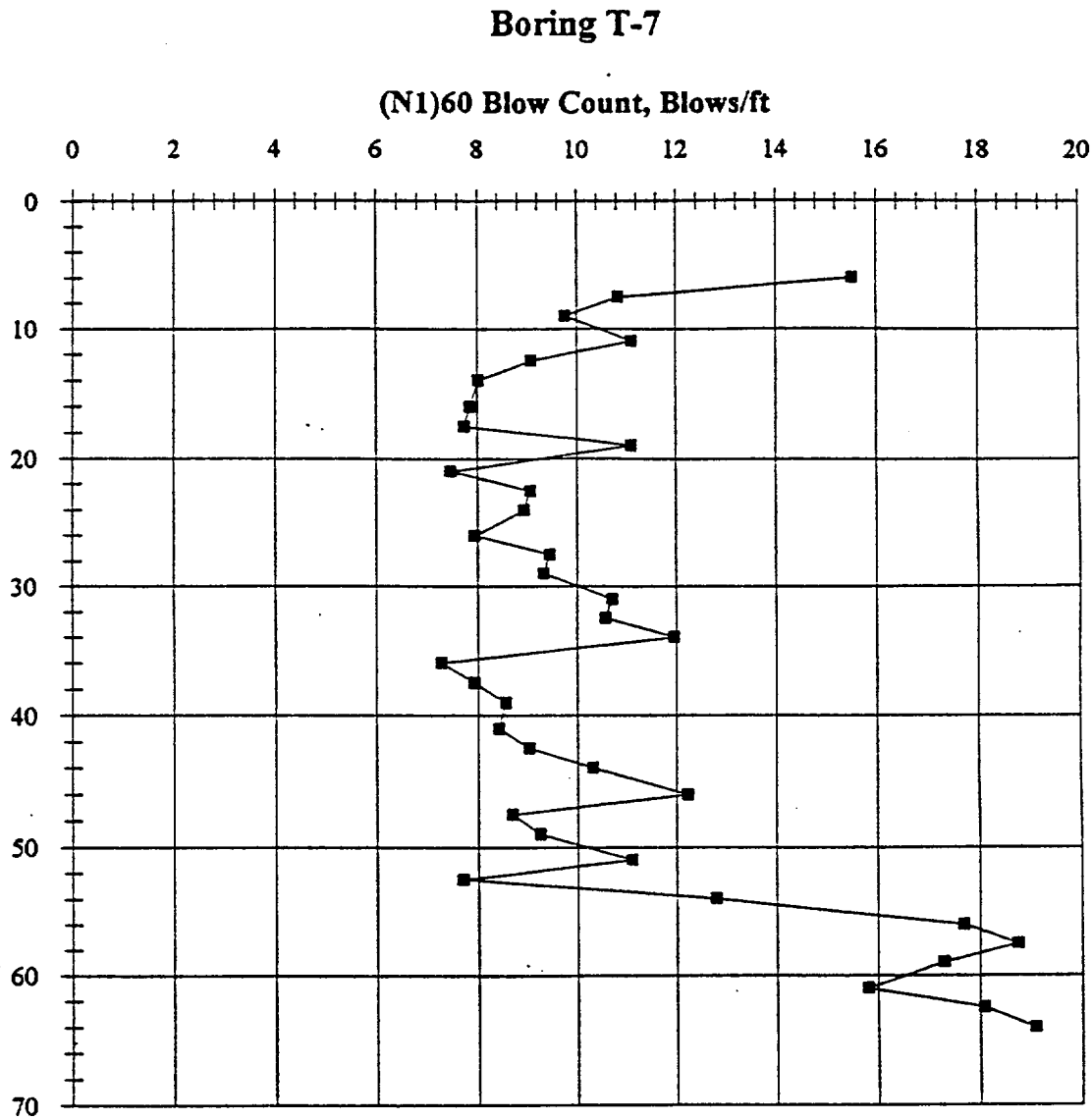
R.L. VOLPE & ASSOCIATES
Los Gatos, California

SHERWOOD TAILING IMPOUNDMENT

$(N_1)_{60}$ BLOW COUNT vs. DEPTH
BORING T-6

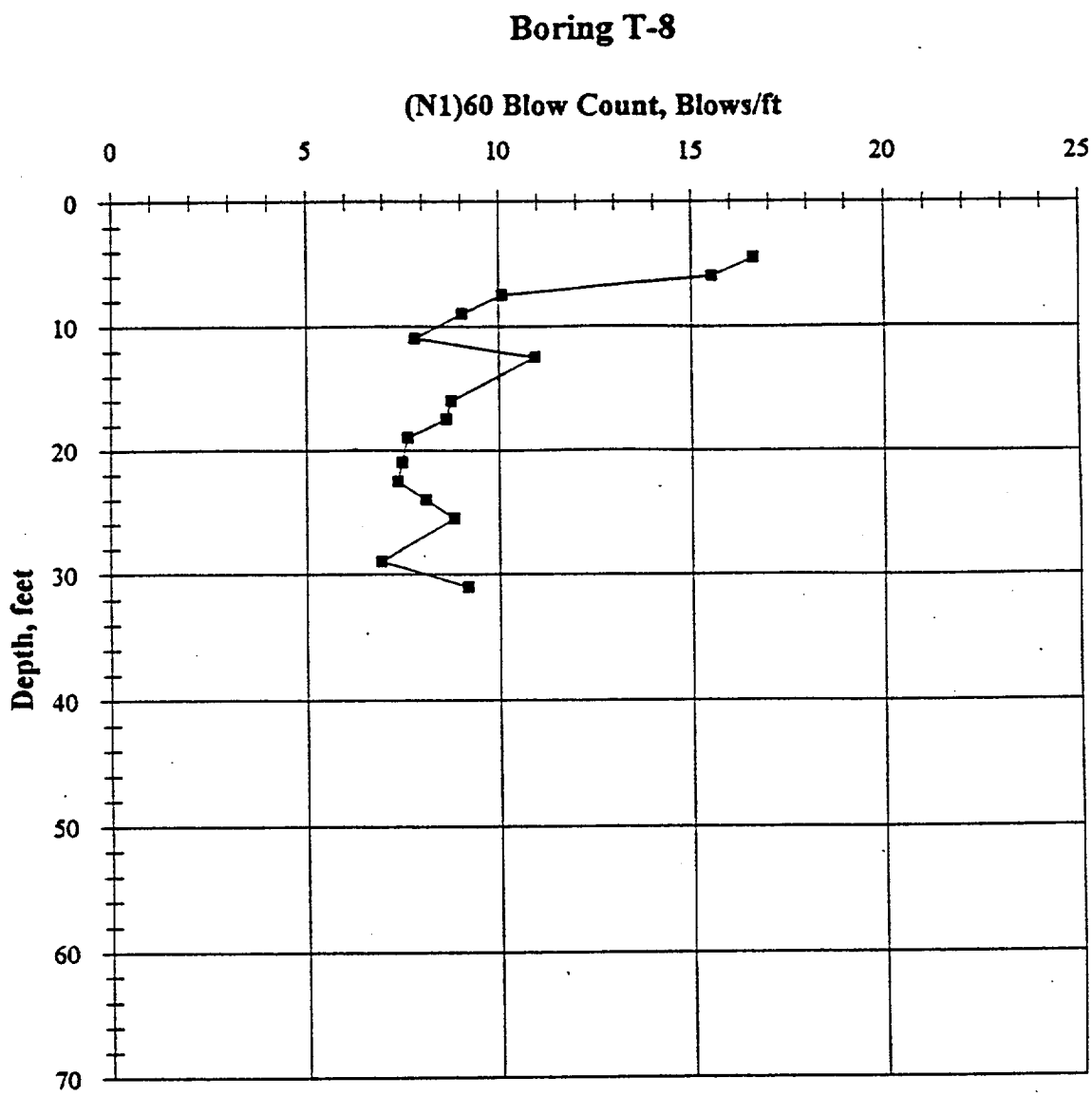
Checked by _____	Date _____	Project No.	Figure No.
Approved by _____	Date _____	SM-100	D-16

L.C-144



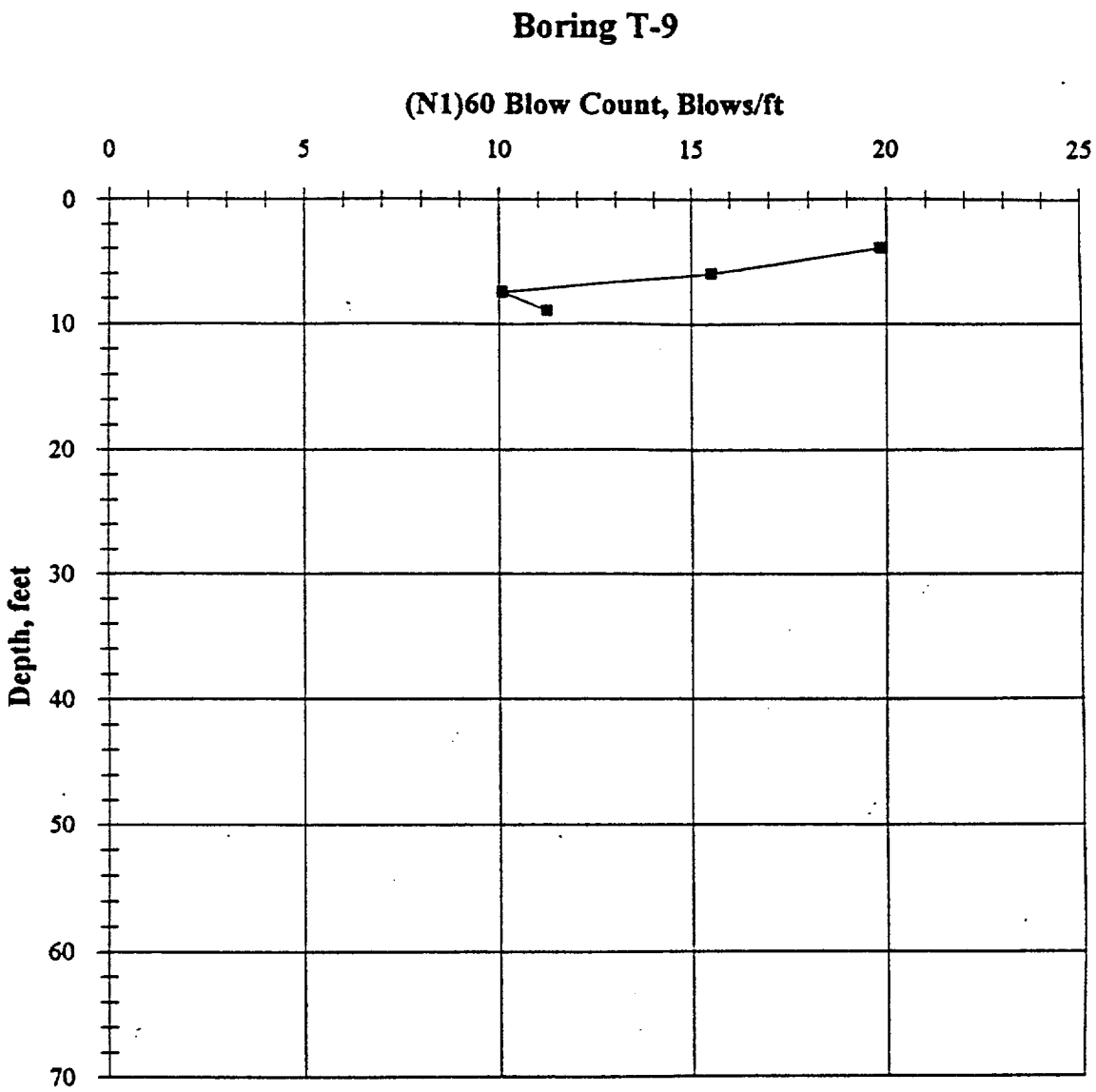
R.L. VOLPE & ASSOCIATES Los Gatos, California		
SHERWOOD TAILING IMPOUNDMENT $(N_1)_{60}$ BLOW COUNT vs. DEPTH BORING T-7		
Checked by _____ Date _____ Approved by _____ Date _____	Project No. SM-100	Figure No. D-17

L.C-145



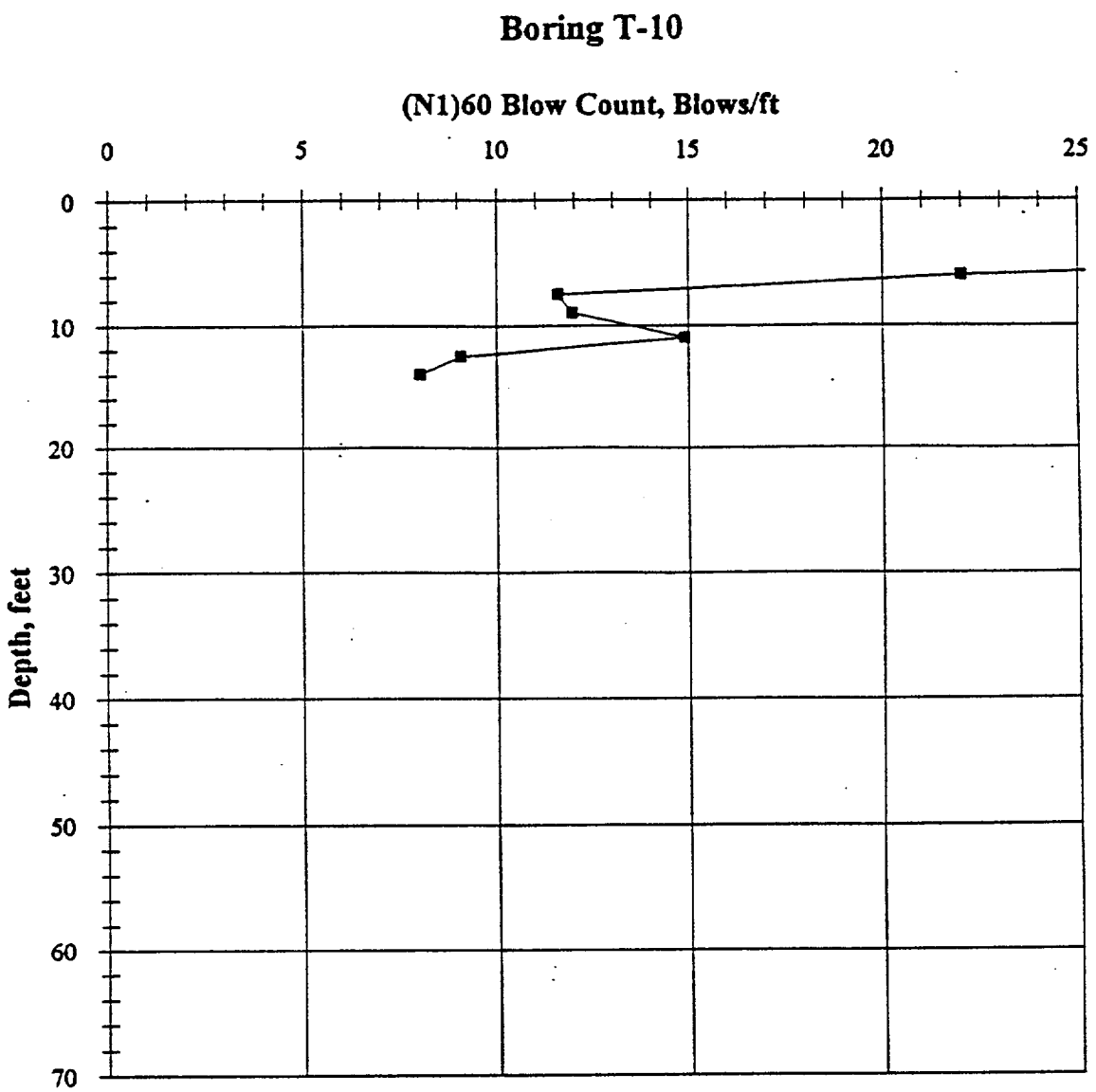
R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT $(N_1)_{60}$ BLOW COUNT vs. DEPTH BORING T-8			
Checked by _____ Date _____ Approved by _____ Date _____		Project No. SM-100	Figure No. D-18

L.C-146



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT $(N_1)_{60}$ BLOW COUNT vs. DEPTH BORING T-9			
Checked by _____ Date _____ Approved by _____ Date _____		Project No. SMI-100	Figure No. D-19

L.C-147



R.L. VOLPE & ASSOCIATES Los Gatos, California			
SHERWOOD TAILING IMPOUNDMENT $(N_1)_{60}$ BLOW COUNT vs. DEPTH BORING T-10			
Checked by _____	Date _____	Project No.	Figure No.
Approved by _____	Date _____	SMI-100	D-20

L.C-148

APPENDIX E

CALCULATIONS TO DETERMINE LIQUEFACTION POTENTIAL

L.C-149

Magnitude 5.0 Random Earthquake

The following spread sheets present the calculations to assess the liquefaction potential within the tailing materials in the event of a random earthquake of Magnitude 5.0 occurring approximately 35 km from the site and producing a peak horizontal acceleration of 0.04 g at the site. Due to the variable depth of each hole, the calculations are presented on six separate sheets which are divided as follows:

Sheet No.	Borehole No.
1	T-1 and T-2
2	T-3
3	T-4
4	T-5 and T-6
5	T-7
6	T-8, T-9 and T-10

Assessment of Liquefaction Potential - Magnitude 5.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL (* 55)	CORRECTION FACTORS: DRILL ROD (* 75@<10')	AMMER EFF (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-1	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 0258139	0 27556	10 76
	6	SPT	7	SM	N	7	5	4	5	12	12	1425	1 60	19	1425	1 00	0 98	0 0254852	0 311416	12 22
	7 5	SPT	8	ML	N	8	6	5	5	12	12	1557	1 12	14	1557	1 00	0 98	0 0253565	0 22080963	9 06
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1 08	9	1689	1 00	0 97	0 0252278	0 14987828	5 94
	11	SPT	3	SM	N	3	3	2	3	10	10	1826	1 05	10	1888	1 03	0 96	0 02591263	0 16869328	6 51
	12 5	SPT	2	SM	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 96	0 0269758	0 15058635	5 58
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8	2221	1 13	0 95	0 02793859	0 1330294	4 76
T-2	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 0258139	0 27556	10 76
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 0254852	0 257832	10 11
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1 12	10	1557	1 00	0 98	0 0253565	0 16726064	6 60
	9	SPT	0 5	SM	N	1	0	0	0	7	7	1689	1 08	8	1689	1 00	0 97	0 0252278	0 13170266	5 22
	11	SPT	1	SM	N	1	1	1	1	8	8	1826	1.05	8	1888	1 03	0 96	0 02591263	0 13738938	5 30
	12 5	SPT	3	SM	N	3	3	2	3	10	10	1899	1 03	10	2055	1 08	0 96	0 0269758	0 16596518	6 15
	14	SPT	3	SM	N	3	3	2	3	10	10	1971	1 01	10	2221	1.13	0 95	0 02793859	0 16333989	5 85
	16	SPT	3	SM	N	3	3	2	3	10	10	2089	0 99	10	2443	1 18	0 95	0 02908453	0 15088665	5 50
	17 5	SPT	2	SM	N	2	2	2	2	9	9	2142	0 98	9	2610	1 22	0 94	0 02965236	0 14295332	4 79
	21	SPT	1 5	ML	N	2	2	1	1	8	8	2312	0 95	8	2998	1.30	0 93	0 03138355	0 131059	4 18
	24 5	SPT	3	ML	N	3	3	2	3	10	10	2482	0 91	9	3387	1 36	0 92	0 03261081	0 1473011	4 52

T.C-150

Assessment of Liquefacti[illegible]tential - Magnitude 5.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV 10/27/94

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS: MOD. CAL (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-3	4	SPT	3	SM	N	3	2	2	2	9	9	1250	1 60	14	1250	1 00	0 99	0 0256568	0 239704	9 34
	5 5	SPT	3	SM	N	3	2	2	2	9	9	1381	1 60	14	1381	1 00	0 98	0 0255281	0 239704	9 39
	7	SPT	2	SM	N	2	2	1	1	8	8	1513	1 60	13	1513	1 00	0 98	0 0253994	0 221776	8 73
	9	SPT	0 5	SM	N	1	0	0	0	7	7	1689	1 08	8	1689	1 00	0 97	0 0252278	0 13170266	5 22
	11	SPT	0 5	SM	N	1	1	0	0	7	7	1826	1 05	8	1888	1 03	0 96	0 02591263	0 12956338	5 00
	13	SPT	0 5	SM	N	1	1	0	0	7	7	1923	1 03	8	2110	1 10	0 96	0 02730732	0 12678751	4 64
	14 5	SPT	0 5	ML	N	1	1	0	0	7	7	1996	1 01	8	2277	1 14	0 96	0 02823912	0 1247962	4 42
	16	SPT	0 5	ML	N	1	1	0	0	7	7	2069	0 99	7	2443	1 18	0 95	0 02908453	0 12287634	4 22
	17 5	SPT	0 5	ML	N	1	1	0	0	7	7	2142	0 98	7	2610	1 22	0 94	0 02985236	0 12102298	4 05
	19	SPT	0 5	ML	N	1	1	0	0	7	7	2214	0 96	7	2776	1 25	0 94	0 03066029	0 11923167	3 90
	20 5	SPT	0 5	SM	N	1	1	0	0	7	7	2287	0 95	7	2943	1 29	0 93	0 03118498	0 11749638	3 77
	22	SPT	0 5	SM	N	1	1	0	0	7	7	2360	0 94	7	3109	1 32	0 93	0 03176233	0 11581948	3 65
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0 92	9	3331	1 36	0 92	0 0324517	0 14798668	4 56
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0 88	9	3886	1 44	0 90	0 03383456	0 1545379	4 57

L.C-151

Assessment of Liquefaction Potential - Magnitude 5.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculation to Determine Liquefaction Potential					
						CORRECTION FACTORS														
BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD CAL (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reducting Factor r sub d	Mag 5 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-4	4 5	SPT	6	SM	N	6	5	3	4	11	11	1294	1 60	18	1294	1 00	0 99	0 0256139	0 293488	11 46
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 0254852	0 257632	10 11
	7 5	SPT	2	SM	N	2	2	1	1	8	8	1557	1 12	9	1557	1 00	0 98	0 0253565	0 15475084	6 10
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1 08	10	1689	1 00	0 97	0 0252278	0 18199203	6 42
	11	SPT	1	ML	N	1	1	1	1	8	8	1826	1 05	8	1888	1 03	0 96	0 02591263	0 13738936	5 30
	12 5	SPT	1	ML	N	1	1	1	1	8	8	1899	1 03	8	2055	1 08	0 96	0 0260758	0 13516751	5 01
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 02793859	0 1330294	4 76
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0 99	9	2443	1 18	0 95	0 02808453	0 14514253	4 99
	17 5	SPT	0	SM	N	0	0	0	0	7	7	2142	0 98	7	2610	1 22	0 94	0 02985236	0 11371287	3 81
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0 96	8	2776	1 25	0 94	0 03055029	0 12643358	4 14
	21	SPT	0	SM	N	0	0	0	0	7	7	2312	0 95	7	2998	1 30	0 93	0 03138356	0 10986982	3 50
	22 5	SPT	1	SM	N	1	1	1	1	8	8	2385	0 93	7	3165	1 33	0 93	0 03194294	0 12223406	3 83
	24	SPT	4	SM	N	4	4	3	4	11	11	2457	0 92	10	3331	1 36	0 92	0 0324517	0 1617174	4 98
	26	SPT	4	SM	N	4	4	3	4	11	11	2555	0 90	10	3553	1 39	0 91	0 03306878	0 15876388	4 80
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0 88	9	3886	1 44	0 90	0 03383456	0 1545379	4 57
	31	SPT	6	SM	N	6	6	5	5	12	12	2798	0 86	11	4108	1 47	0 90	0 03436069	0 1776306	5 17
	32 5	SPT	6	SM	N	6	6	5	5	12	12	2871	0 85	11	4275	1 49	0 90	0 03484525	0 17533941	5 03
	34	SPT	16	SM	N	16	16	12	14	21	21	2943	0 84	18	4441	1 51	0 90	0 0353059	0 29874692	8 46
	36	SPT	14	SM	N	14	14	11	13	20	20	3041	0 83	16	4663	1 53	0 90	0 03588576	0 26904469	7 50
	37 5	SPT	13	SM	N	13	13	10	12	19	19	3114	0 82	15	4830	1 55	0 90	0 03629687	0 25350824	6 98
	39	SPT	17	SM	N	17	17	13	15	22	22	3186	0 81	18	4996	1 57	0 90	0 03668918	0 29860479	8 14
	41	SPT	14	SM	N	14	14	11	13	20	20	3284	0 79	16	5218	1 59	0 90	0 03718516	0 2582209	6 94
	42 5	SPT	18	SM	N	18	18	14	16	23	23	3357	0 78	18	5385	1 60	0 90	0 0375383	0 30198958	8 04
	44	SPT	13	ML	N	13	13	10	12	19	19	3429	0 77	14	5551	1 62	0 90	0 03787642	0 24052797	6 35
	46	SPT	16	ML	N	16	16	12	14	21	21	3624	0 75	16	5995	1 65	0 90	0 03871157	0 26678123	6 89
	49 5	SPT	15	ML	N	15	15	11	14	21	21	3697	0 74	15	6162	1 67	0 90	0 03900211	0 25262868	6 48
	51	SPT	14	ML	N	14	14	11	13	20	20	3770	0 73	14	6328	1 68	0 90	0 03928141	0 23878839	6 08
	52 5	SPT	12	ML	N	12	12	9	11	18	18	3843	0 73	13	6495	1 69	0 90	0 03955011	0 21440969	5 42
	54	SPT	8	ML	N	8	8	6	7	14	14	3915	0 72	10	6661	1 70	0 90	0 03980881	0 16912915	4 25
	56	SPT	12	ML	N	12	12	9	11	18	18	4013	0 71	13	6883	1 72	0 90	0 04013911	0 20887173	5 20
	57 5	SPT	11	ML	N	11	11	8	10	17	17	4086	0 70	12	7060	1 73	0 90	0 04037653	0 19612521	4 86
	61	SPT	24	ML	N	24	24	18	22	29	29	4256	0 68	19	7438	1 75	0 90	0 04089866	0 32352444	7 91
	62 5	SPT	12	ML	N	12	12	9	11	18	18	4329	0 67	12	7605	1 76	0 90	0 04111015	0 19918273	4 85
	64	SPT	10	SM	N	10	10	8	9	16	16	4401	0 67	11	7771	1 77	0 90	0 04131445	0 17712122	4 29

L.C-152

Assessment of Liquefaction (ıtial - Magnitude 5.0 Event

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
JOB NAME	SMI - Sherwood Tailings Impoundments																			
JOB NO	SMI-1																			
BY	RLV																			
BOREHOLE	DEPTH (ft)	SPT or MOD. CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (* 55)	CORRECTION FACTORS DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor	Mag 5 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-5	4 5	SPT	4	SM	N	4	3	2	3	10	10	1294	1 60	16	1294	1 00	0 99	0 0256139	0 257632	10 06
	6	SPT	4	ML	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 0254852	0 257632	10 11
	7 5	SPT	4	ML	N	4	3	2	3	10	10	1557	1 12	11	1557	1 00	0 98	0 0253565	0 17977044	7 09
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1 08	10	1689	1 00	0 97	0 0252278	0 16199203	6 42
	11	SPT	4	SM	N	4	4	3	4	11	11	1826	1 05	11	1888	1 03	0 96	0 02591263	0 18434572	7 11
	12 5	SPT	2	ML	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 96	0 0269758	0 15066635	5 58
	14	SPT	1 5	ML	N	2	2	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 02793859	0 14060702	5 03
	16	SPT	0	ML	N	0	0	0	0	7	7	2069	0 99	7	2443	1 18	0 95	0 02908453	0 11545428	3 97
	17 5	SPT	0	ML	N	0	0	0	0	7	7	2142	0 98	7	2610	1 22	0 94	0 02985238	0 11371287	3 81
	19	SPT	0	ML	N	0	0	0	0	7	7	2214	0 96	7	2778	1 25	0 94	0 03055029	0 11202978	3 67
	23	SPT	3	SM	N	3	3	2	3	10	10	2409	0 93	9	3220	1 34	0 92	0 03211792	0 14937842	4 65
	24 5	SPT	4	SM	N	4	4	3	4	11	11	2482	0 91	10	3387	1 36	0 92	0 03261081	0 16096821	4 94
	26	SPT	6	SM	N	6	6	5	5	12	12	2555	0 90	11	3553	1 39	0 91	0 03306878	0 18572378	5 62
	27 5	SPT	0	SM	N	0	0	0	0	7	7	2628	0 89	6	3720	1 42	0 91	0 03346559	0 10342934	3 09
	29	SPT	2	SP-SM	N	2	2	2	2	2	2	2700	0 88	2	3886	1 44	0 90	0 03383456	0 02624229	0 78
T-6	4 5	SPT	6	SM	N	6	5	3	4	11	11	1294	1 60	18	1294	1 00	0 99	0 0256139	0 293488	11 46
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 0254852	0 257632	10 11
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1 00	0 98	0 0253565	0 16726064	6 60
	9	SPT	0 5	ML	N	1	0	0	0	7	7	1689	1.08	8	1689	1 00	0 97	0 0252278	0 13170268	5 22

i.C-153

Assessment of Liquefaction P 'ential - Magnitude 5.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts				CORRECTION FACTORS								Calculations to Determine Liquefaction Potential					
T-7	4	SPT	9	SM	N	9	7	5	6	13	13	1250	1.60	21	1250	1 00	0 9868	0 0256568	0 347272	13 54
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1 00	0 9802	0 0264852	0 257632	10 11
	7 5	SPT	4	SM	N	4	3	2	3	10	10	1557	1 12	11	1557	1 00	0 97525	0 0253666	0 17977044	7 09
	9	SPT	3	ML	N	3	2	2	2	9	9	1689	1 08	10	1689	1 00	0 9703	0 0252278	0 16199203	6 42
	11	SPT	4	ML	N	4	4	3	4	11	11	1826	1.05	11	1888	1 03	0 9637	0 02591263	0 18434522	7 11
	12 5	SPT	2	ML	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 95875	0 0269758	0 16056635	5 58
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8	2221	1 13	0 9538	0 02793859	0 1330294	4 76
	16	SPT	1	SM	N	1	1	1	1	8	8	2069	0 99	8	2443	1 18	0 9472	0 02908453	0 1302984	4 48
	17 5	SPT	1	ML	N	1	1	1	1	8	8	2142	0 98	8	2610	1 22	0 94225	0 02985236	0 1283331	4 30
	19	SPT	5	ML	N	5	5	4	5	12	12	2214	0 96	11	2776	1 25	0 9373	0 03066029	0 18404889	6 02
	21	SPT	1	SM	N	1	1	1	1	8	8	2312	0.95	7	2998	1 30	0 9307	0 03138355	0 12399564	3 95
	22 5	SPT	3	SM	N	3	3	2	3	10	10	2385	0 93	9	3165	1 33	0 92575	0 03194294	0 15008485	4 70
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0 92	9	3331	1 36	0 9208	0 0324517	0 14796668	4 56
	26	SPT	2	ML	N	2	2	2	2	9	9	2555	0.90	8	3553	1 39	0 9142	0 03305878	0 13180397	3 99
	27 5	SPT	4	SM	N	4	4	3	4	11	11	2628	0 89	9	3720	1 42	0 90925	0 03346559	0 15662157	4 68
	29	SPT	4	ML	N	4	4	3	4	11	11	2700	0 88	9	3886	1 44	0 9043	0 03383456	0 1545379	4 57
	31	SPT	6	ML	N	6	6	5	5	12	12	2798	0 86	11	4108	1 47	0 9	0 03436069	0 1776306	5 17
	32 5	SPT	6	SM	N	6	6	5	5	12	12	2871	0 85	11	4275	1 49	0 9	0 03484525	0 17533941	5 03
	34	SPT	8	SM	N	8	8	6	7	14	14	2943	0 84	12	4441	1 51	0 9	0 0353059	0 19823393	5 61
	36	SPT	2	SM	N	2	2	2	2	9	9	3041	0 83	7	4663	1 53	0 9	0 03588576	0 12079558	3 37
	37 5	SPT	3	SM	N	3	3	2	3	10	10	3114	0 82	8	4830	1 55	0 9	0 03629687	0 13149892	3 62
	39	SPT	4	ML	N	4	4	3	4	11	11	3186	0 81	9	4996	1 57	0 9	0 03668918	0 1419377	3 87
	41	SPT	4	ML	N	4	4	3	4	11	11	3284	0 79	8	5218	1 59	0 9	0 03718516	0 13964966	3 76
	42 5	SPT	5	ML	N	5	5	4	5	12	12	3357	0 78	9	5385	1 60	0 9	0 0375383	0 14969311	3 99
	44	SPT	7	ML	N	7	7	5	6	13	13	3429	0 77	10	5551	1 62	0 9	0 03787842	0 17107069	4 52
	46	SPT	10	ML	N	10	10	8	9	16	16	3527	0.76	12	5773	1 64	0 9	0 03830561	0 2025873	5 29
	47 5	SPT	5	ML	N	5	5	4	5	12	12	3600	0 75	9	5940	1 65	0 9	0 03861211	0 14391962	3 73
	49	SPT	6	ML	N	6	6	5	5	12	12	3672	0 75	9	6106	1 66	0 9	0 03890655	0 15339695	3 94
	51	SPT	9	ML	N	9	9	7	8	15	15	3770	0 73	11	6328	1 68	0 9	0 03928141	0 18396453	4 68
	52 5	SPT	4	ML	N	4	4	3	4	11	11	3843	0 73	8	6495	1 69	0 9	0 03955011	0 12768229	3 23
	54	SPT	12	ML	N	12	12	9	11	18	18	3915	0.72	13	6661	1 70	0 9	0 03980881	0 21200696	5 33
	56	SPT	20	SM	N	20	20	15	18	25	25	4013	0 71	18	6883	1 72	0 9	0 04013911	0 29335917	7 31
	57 5	SPT	22	SM	N	22	22	17	20	27	27	4066	0 70	19	7050	1 73	0 9	0 04037653	0 31101513	7 70
	59	SPT	20	SM	N	20	20	15	18	25	25	4158	0 69	17	7216	1 74	0 9	0 04060562	0 28695012	7 07
	61	SPT	18	ML	N	18	18	14	16	23	23	4256	0 68	16	7438	1 75	0 9	0 04089886	0 2624394	6 42
	62 5	SPT	22	ML	N	22	22	17	20	27	27	4329	0 67	18	7605	1 76	0 9	0 04111015	0 29949309	7 29
	64	SPT	24	ML	N	24	24	18	22	29	29	4401	0 67	19	7771	1 77	0 9	0 04131445	0 31660418	7 66

L.C-154

Assessment of Liquefactio. ential - Magnitude 5.0 Event

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
JOB NAME	SMI - Sherwood Tailings Impoundments																			
JOB NO	SMI-1																			
BY	RLV																			
BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS MOD. CAL. (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub u	Mag 5 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-8	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 0256139	0 27556	10 76
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 0254852	0 257632	10 11
	7 5	SPT	3	ML	N	3	2	2	2	9	9	1557	1 12	10	1557	1 00	0 98	0 0253565	0 16726064	6 60
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1.08	9	1689	1 00	0 97	0 0252278	0 14987628	5 94
	11	SPT	0.5	ML	N	1	1	0	0	7	7	1826	1 05	8	1888	1 03	0 96	0 02591263	0 12956338	5 00
	12 5	SPT	4	ML	N	4	4	3	4	11	11	1899	1 03	11	2055	1 08	0 96	0 0260758	0 18136401	6 72
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0 99	9	2443	1 18	0 95	0 02906453	0 14514253	4 99
	17 5	SPT	2	SM	N	2	2	2	2	9	9	2142	0 98	9	2610	1 22	0 94	0 02985236	0 14295332	4 79
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0 96	8	2776	1 25	0 94	0 03066029	0 12643358	4 14
	21	SPT	1	ML	N	1	1	1	1	8	8	2312	0.95	7	2998	1.30	0 93	0 03138356	0 12399594	3 95
	22 5	SPT	1	SM	N	1	1	1	1	8	8	2385	0 93	7	3165	1 33	0 93	0 03194294	0 12223406	3 83
	24	SPT	2	ML	N	2	2	2	2	9	9	2457	0 92	8	3331	1 36	0 92	0 0324517	0 13425596	4 14
	25 5	SPT	3	ML	N	3	3	2	3	10	10	2530	0 91	9	3498	1 38	0 92	0 03291421	0 14594985	4 43
	29	SPT	1	ML	N	1	1	1	1	8	8	2700	0 88	7	3886	1 44	0 90	0 03383456	0 11517447	3 40
	31	SPT	4	ML	N	4	4	3	4	11	11	2798	0 86	9	4108	1 47	0 90	0 03436059	0 15184551	4 42
T-9	4	SPT	8	SM	N	8	6	5	6	12	12	1250	1.60	20	1250	1 00	0 99	0 0256568	0 329344	12 84
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 0254852	0 257632	10 11
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1 00	0 98	0 0253565	0 16726064	6 60
	9	SPT	5	SM	N	5	4	3	3	10	10	1689	1.08	11	1689	1 00	0 97	0 0252278	0 18672353	7 38
T-10	4 5	SPT	28	SM	N	28	21	16	19	26	26	1294	1 60	41	1294	1 00	0 99	0 0256139	0 687904	26 86
	6	SPT	10	SM	N	10	8	6	7	14	14	1425	1.60	22	1425	1 00	0 98	0 0254852	0 3652	14 33
	7 5	SPT	5	SM	N	5	4	3	3	10	10	1557	1.12	12	1557	1 00	0 98	0 0253565	0 19228024	7 58
	9	SPT	6	ML	N	6	5	3	4	11	11	1689	1 08	12	1689	1 00	0 97	0 0252278	0 19833927	7 86
	11	SPT	8	ML	N	8	8	6	7	14	14	1826	1 05	15	1888	1 03	0 96	0 02591263	0 24695303	9 53
	12 5	SPT	2	SM	N	2	2	2	2	9	9	1899	1.03	9	2055	1 08	0 96	0 0260758	0 15066635	5 68
	14	SPT	1	SM	N	1	1	1	1	8	8	1971	1.01	8	2221	1 13	0 95	0 02793859	0 1330294	4 76

L.C.-15

L.C-156

Magnitude 5.5 Random Earthquake

The following spread sheets present the calculations to assess the liquefaction potential within the tailing materials in the event of a random earthquake of Magnitude 5.5 occurring approximately 61 km from the site and producing a peak horizontal acceleration of 0.025 g at the site. Due to the variable depth of each hole, the calculations are presented on six separate sheets which are divided as follows:

Sheet No.	Borehole No.
1	T-1 and T-2
2	T-3
3	T-4
4	T-5 and T-6
5	T-7
6	T-8, T-9 and T-10

Assessment of Liquefaction P[illegible]tial - Magnitude 5.5 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (* 55)	DRILL ROD (*.75@<10')	AMMER EFF (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5 5 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts				CORRECTION FACTORS								Calculations to Determine Liquefaction Potential					
T-1	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 01600669	0 25564	15 97
	6	SPT	7	SM	N	7	5	4	5	12	12	1425	1 60	19	1425	1 00	0 98	0 01592825	0 288904	18 14
	7 5	SPT	8	ML	N	8	6	5	5	12	12	1557	1 12	14	1557	1 00	0 98	0 01584781	0 21319669	13 45
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1 06	9	1689	1 00	0 97	0 01576736	0 13904185	8 82
	11	SPT	3	SM	N	3	3	2	3	10	10	1826	1 05	10	1888	1 03	0 96	0 0161954	0 15649657	9 66
	12 5	SPT	2	SM	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 96	0 01685967	0 13968203	8 28
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 01746162	0 12341282	7 07
T-2	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 01600669	0 25564	15 97
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 01592825	0 239008	15 01
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1 00	0 98	0 01584781	0 15510951	9 79
	9	SPT	0 5	SM	N	1	0	0	0	7	7	1689	1 06	8	1689	1 00	0 97	0 01576736	0 12218199	7 75
	11	SPT	1	SM	N	1	1	1	1	8	8	1826	1 05	8	1888	1 03	0 96	0 0161954	0 1274576	7 87
	12 5	SPT	3	SM	N	3	3	2	3	10	10	1899	1 03	10	2055	1 08	0 96	0 01685967	0 15398769	9 13
	14	SPT	3	SM	N	3	3	2	3	10	10	1971	1 01	10	2221	1 13	0 95	0 01746162	0 15153219	8 68
	16	SPT	3	SM	N	3	3	2	3	10	10	2089	0 99	10	2443	1.16	0 95	0 01817783	0 14842135	8 16
	17 5	SPT	2	SM	N	2	2	2	2	9	9	2142	0 98	9	2610	1 22	0 94	0 01865773	0 13261935	7 11
	21	SPT	1 5	ML	N	2	2	1	1	8	8	2312	0 95	8	2998	1 30	0 93	0 01961472	0 12156485	6 20
	24 5	SPT	3	ML	N	3	3	2	3	10	10	2482	0 91	9	3387	1 36	0 92	0 02038175	0 13665263	6 70

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Assessment of Liquefactio. ential - Magnitude 5.5 Event

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
JOB NAME	SMI - Sherwood Tailings Impoundments																			
JOB NO	SMI-1																			
BY	RLV	10/27/84																		
BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS MOD CAL (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5 5 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-3	4	SPT	3	SM	N	3	2	2	2	9	9	1250	1 60	14	1250	1 00	0 99	0 0160355	0 222376	13 87
	5 5	SPT	3	SM	N	3	2	2	2	9	9	1381	1 60	14	1381	1 00	0 98	0 01595606	0 222376	13 94
	7	SPT	2	SM	N	2	2	1	1	8	8	1513	1 60	13	1513	1 00	0 98	0 01587463	0 205744	12 96
	9	SPT	0 5	SM	N	1	0	0	0	7	7	1689	1 08	8	1689	1 00	0 97	0 01576738	0 12218199	7 75
	11	SPT	0 5	SM	N	1	1	0	0	7	7	1826	1 05	8	1888	1 03	0 96	0 0161964	0 12019736	7 42
	13	SPT	0 5	SM	N	1	1	0	0	7	7	1923	1 03	8	2110	1 10	0 96	0 01706707	0 11762215	6 89
	14 5	SPT	0 5	ML	N	1	1	0	0	7	7	1996	1 01	8	2277	1 14	0 95	0 01764945	0 11577479	6 56
	16	SPT	0 5	ML	N	1	1	0	0	7	7	2069	0 99	7	2443	1 18	0 95	0 01817783	0 11399372	6 27
	17 5	SPT	0 5	ML	N	1	1	0	0	7	7	2142	0 98	7	2610	1 22	0 94	0 01865773	0 11227433	6 02
	19	SPT	0 5	ML	N	1	1	0	0	7	7	2214	0 96	7	2778	1 25	0 94	0 01909393	0 11061251	5 79
	20 5	SPT	0 5	SM	N	1	1	0	0	7	7	2287	0 95	7	2943	1 29	0 93	0 01949062	0 10900453	5 59
	22	SPT	0 5	SM	N	1	1	0	0	7	7	2360	0 94	7	3109	1 32	0 93	0 01985148	0 10744699	5 41
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0 92	9	3331	1 36	0 92	0 02028231	0 13728885	6 77
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0 88	9	3886	1 44	0 90	0 0211466	0 14336649	6 78

L.C-158

Assessment of Liquefactio ntial - Magnitude 5.5 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD CAL (*.55)	DRILL ROD (*.75@<10')	HAMMER EFF (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5.5 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts				CORRECTION FACTORS								Calculation to Determine Liquefaction Potential					
T-4	4.5	SPT	6	SM	N	6	5	3	4	11	11	1294	1.60	18	1294	1.00	0.99	0.01600869	0.272272	17.01
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1.00	0.98	0.01692825	0.239008	15.01
	7.5	SPT	2	SM	N	2	2	1	1	8	8	1557	1.12	9	1557	1.00	0.98	0.01584781	0.14356403	9.06
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1.06	10	1689	1.00	0.97	0.01576738	0.15028176	9.53
	11	SPT	1	ML	N	1	1	1	1	8	8	1826	1.05	8	1888	1.03	0.96	0.0161964	0.1274576	7.87
	12.5	SPT	1	ML	N	1	1	1	1	8	8	1899	1.03	8	2055	1.08	0.96	0.01685967	0.12539637	7.44
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8	2221	1.13	0.95	0.01746182	0.12341282	7.07
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0.99	9	2443	1.18	0.95	0.01817783	0.1346503	7.41
	17.5	SPT	0	SM	N	0	0	0	0	7	7	2142	0.98	7	2610	1.22	0.94	0.01866773	0.10549266	5.65
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0.96	8	2776	1.25	0.94	0.01909393	0.11729381	6.14
	21	SPT	0	SM	N	0	0	0	0	7	7	2312	0.95	7	2998	1.30	0.93	0.01981472	0.10192742	5.20
	22.5	SPT	1	SM	N	1	1	1	1	8	8	2385	0.93	7	3165	1.33	0.93	0.01996434	0.11339786	5.68
	24	SPT	4	SM	N	4	4	3	4	11	11	2457	0.92	10	3331	1.36	0.92	0.02028231	0.15002699	7.40
	26	SPT	4	SM	N	4	4	3	4	11	11	2555	0.90	10	3553	1.39	0.91	0.02066174	0.14728697	7.13
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0.88	9	3888	1.44	0.90	0.0211466	0.14336649	6.78
	31	SPT	6	SM	N	6	6	5	5	12	12	2798	0.86	11	4108	1.47	0.90	0.02147537	0.16478943	7.67
	32.5	SPT	6	SM	N	6	6	5	5	12	12	2871	0.85	11	4275	1.49	0.90	0.02177828	0.16266427	7.47
	34	SPT	16	SM	N	16	16	12	14	21	21	2943	0.84	18	4441	1.51	0.90	0.02206619	0.27715075	12.56
	36	SPT	14	SM	N	14	14	11	13	20	20	3041	0.83	16	4663	1.53	0.90	0.02242859	0.24959568	11.13
	37.5	SPT	13	SM	N	13	13	10	12	19	19	3114	0.82	15	4830	1.55	0.90	0.02268554	0.23518234	10.37
	39	SPT	17	SM	N	17	17	13	15	22	22	3186	0.81	18	4996	1.57	0.90	0.02293074	0.2770189	12.08
	41	SPT	14	SM	N	14	14	11	13	20	20	3284	0.79	16	5218	1.59	0.90	0.02324073	0.23956396	10.31
	42.5	SPT	18	SM	N	18	18	14	16	23	23	3357	0.78	18	5385	1.60	0.90	0.02346144	0.28015901	11.94
	44	SPT	13	ML	N	13	13	10	12	19	19	3429	0.77	14	5551	1.62	0.90	0.02367276	0.2231404	9.43
	46	SPT	16	ML	N	16	16	12	14	21	21	3624	0.75	16	5995	1.65	0.90	0.02419473	0.24749584	10.23
	49.5	SPT	15	ML	N	15	15	11	14	21	21	3697	0.74	15	6162	1.67	0.90	0.02437632	0.23436636	9.61
	51	SPT	14	ML	N	14	14	11	13	20	20	3770	0.73	14	6328	1.68	0.90	0.02455068	0.22152658	9.02
	52.5	SPT	12	ML	N	12	12	9	11	18	18	3843	0.73	13	6495	1.69	0.90	0.02471882	0.19891038	8.05
	54	SPT	8	ML	N	8	8	6	7	14	14	3915	0.72	10	6661	1.70	0.90	0.0248805	0.15690294	6.31
	56	SPT	12	ML	N	12	12	9	11	18	18	4013	0.71	13	6883	1.72	0.90	0.02506694	0.19377257	7.72
	57.5	SPT	11	ML	N	11	11	8	10	17	17	4086	0.70	12	7050	1.73	0.90	0.02523533	0.18194749	7.21
	61	SPT	24	ML	N	24	24	18	22	29	29	4256	0.68	19	7438	1.75	0.90	0.02556179	0.30013713	11.74
	62.5	SPT	12	ML	N	12	12	9	11	18	18	4329	0.67	12	7605	1.76	0.90	0.02569385	0.18478398	7.19
	64	SPT	10	SM	N	10	10	8	9	16	16	4401	0.67	11	7771	1.77	0.90	0.02582153	0.16431728	6.36

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Assessment of Liquefaction (ıtial - Magnitude 5.5 Event

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
JOB NAME	SMI - Sherwood Tailings Impoundments																			
JOB NO	SMI-1																			
BY	RLV																			
BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD CAL (* 55)	CORRECTION FACTORS DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor	Mag 5 5 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-5	4 5	SPT	4	SM	N	4	3	2	3	10	10	1294	1 60	16	1294	1 00	0 99	0 01600669	0 239008	14 93
	6	SPT	4	ML	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 01592825	0 239008	15 01
	7 5	SPT	4	ML	N	4	3	2	3	10	10	1557	1 12	11	1557	1 00	0 98	0 01584781	0 16677499	10 62
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1 08	10	1689	1 00	0 97	0 01576738	0 15028178	9 53
	11	SPT	4	SM	N	4	4	3	4	11	11	1826	1.05	11	1888	1 03	0 96	0 0161954	0 17101906	10 56
	12 5	SPT	2	ML	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 96	0 01685987	0 13964203	8 28
	14	SPT	1 5	ML	N	2	2	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 01746162	0 13044266	7 47
	16	SPT	0	ML	N	0	0	0	0	7	7	2069	0 99	7	2443	1 18	0 95	0 01817783	0 10710819	5 89
	17 5	SPT	0	ML	N	0	0	0	0	7	7	2142	0 98	7	2610	1 22	0 94	0 01865773	0 10549266	5 65
	19	SPT	0	ML	N	0	0	0	0	7	7	2214	0 96	7	2776	1 25	0 94	0 01908393	0 10393122	5 44
	23	SPT	3	SM	N	3	3	2	3	10	10	2409	0 93	9	3220	1 34	0 92	0 0200737	0 13657998	6 90
	24 5	SPT	4	SM	N	4	4	3	4	11	11	2482	0 91	10	3387	1 36	0 92	0 02038175	0 14933195	7 33
	26	SPT	6	SM	N	6	6	5	5	12	12	2555	0 90	11	3553	1 39	0 91	0 02066174	0 17229797	8 34
	27 5	SPT	0	SM	N	0	0	0	0	7	7	2628	0 89	6	3720	1 42	0 91	0 02091599	0 09595252	4 59
	29	SPT	2	SP-SM	N	2	2	2	2	2	2	2700	0.88	2	3886	1 44	0 90	0 0211466	0 02434525	1 15
T-6	4 5	SPT	6	SM	N	6	5	3	4	11	11	1294	1 60	18	1294	1 00	0 99	0 01600669	0 272272	17 01
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 01592825	0 239008	15 01
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1 00	0 98	0 01584781	0 15516951	9 79
	9	SPT	0 5	ML	N	1	0	0	0	7	7	1689	1 08	8	1689	1 00	0 97	0 01576738	0 12218199	7 75

L.C-160

Assessment of Liquefactio 'ential - Magnitude 5.5 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 5 5 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts				CORRECTION FACTORS								Calculations to Determine Liquefaction Potential					
T-7	4	SPT	9	SM	N	9	7	5	6	13	13	1250	1 60	21	1250	1 00	0 9868	0 0180355	0 322168	20 09
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 9802	0 01592825	0 239008	15 01
	7 5	SPT	4	SM	N	4	3	2	3	10	10	1557	1 12	11	1557	1 00	0 97525	0 01584781	0 16677499	10 52
	9	SPT	3	ML	N	3	2	2	2	9	9	1689	1 08	10	1689	1 00	0 9703	0 01576734	0 15028176	9 53
	11	SPT	4	ML	N	4	4	3	4	11	11	1826	1 05	11	1888	1 03	0 9637	0 0181954	0 17101906	10 56
	12 5	SPT	2	ML	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 95875	0 01685987	0 13968203	8 28
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1 01	8	2221	1 13	0 9538	0 01746182	0 12341282	7 07
	16	SPT	1	SM	N	1	1	1	1	8	8	2069	0 99	8	2443	1 18	0 9472	0 01817783	0 12087924	6 65
	17 5	SPT	1	ML	N	1	1	1	1	8	8	2142	0 98	8	2610	1 22	0 94225	0 01865773	0 11906601	6 38
	19	SPT	5	ML	N	5	5	4	5	12	12	2214	0 96	11	2776	1 25	0 9373	0 01909393	0 17074415	8 94
	21	SPT	1	SM	N	1	1	1	1	8	8	2312	0 95	7	2998	1 30	0 9307	0 01961472	0 11503238	5 86
	22 5	SPT	3	SM	N	3	3	2	3	10	10	2385	0 93	9	3165	1 33	0 92575	0 01996434	0 13923534	6 97
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0 92	9	3331	1 36	0 9208	0 02028231	0 13728665	6 77
	26	SPT	2	ML	N	2	2	2	2	9	9	2555	0 90	8	3553	1 39	0 9142	0 02066174	0 12227598	5 92
	27 5	SPT	4	SM	N	4	4	3	4	11	11	2628	0 89	9	3720	1 42	0 90925	0 02091599	0 14529953	6 95
	29	SPT	4	ML	N	4	4	3	4	11	11	2700	0 88	9	3886	1 44	0 9043	0 0211466	0 14336649	6 78
	31	SPT	6	ML	N	6	6	5	6	12	12	2798	0 86	11	4108	1 47	0 9	0 02147537	0 16478983	7 67
	32 5	SPT	6	SM	N	6	6	5	6	12	12	2871	0 85	11	4275	1 49	0 9	0 02177828	0 16266427	7 47
	34	SPT	8	SM	N	8	8	6	7	14	14	2943	0 84	12	4441	1 51	0 9	0 02206619	0 18390377	8 33
	36	SPT	2	SM	N	2	2	2	2	9	9	3041	0 83	7	4663	1 53	0 9	0 02242859	0 11206337	5 00
	37 5	SPT	3	SM	N	3	3	2	3	10	10	3114	0 82	8	4830	1 55	0 9	0 02268554	0 12199298	5 38
	39	SPT	4	ML	N	4	4	3	4	11	11	3186	0 81	9	4996	1 57	0 9	0 02293074	0 13167714	5 74
	41	SPT	4	ML	N	4	4	3	4	11	11	3284	0 79	8	5218	1 59	0 9	0 02324073	0 12956469	5 57
	42 5	SPT	5	ML	N	5	5	4	5	12	12	3357	0 78	9	5385	1 60	0 9	0 02346144	0 13887192	5 92
	44	SPT	7	ML	N	7	7	5	6	13	13	3429	0 77	10	5551	1 62	0 9	0 02367276	0 15870414	6 70
	46	SPT	10	ML	N	10	10	8	9	16	16	3527	0 76	12	5773	1 64	0 9	0 02394094	0 18794243	7 85
	47 5	SPT	5	ML	N	5	5	4	5	12	12	3600	0 75	9	5940	1 65	0 9	0 02413257	0 13351569	5 53
	49	SPT	6	ML	N	6	6	5	5	12	12	3672	0 75	9	6106	1 66	0 9	0 02431659	0 14230801	5 85
	51	SPT	9	ML	N	9	9	7	8	15	15	3770	0 73	11	6328	1 68	0 9	0 02455088	0 17066589	6 95
	52 5	SPT	4	ML	N	4	4	3	4	11	11	3843	0 73	8	6496	1 69	0 9	0 02471882	0 11845225	4 79
	54	SPT	12	ML	N	12	12	9	11	18	18	3915	0 72	13	6661	1.70	0 9	0 0248805	0 19668115	7 91
	56	SPT	20	SM	N	20	20	15	18	25	25	4013	0 71	18	6883	1 72	0 9	0 02508694	0 27215248	10 85
	57 5	SPT	22	SM	N	22	22	17	20	27	27	4086	0 70	19	7050	1 73	0 9	0 02523533	0 28653211	11 43
	59	SPT	20	SM	N	20	20	15	18	25	25	4158	0 69	17	7216	1 74	0 9	0 02537851	0 26620674	10 49
	61	SPT	18	ML	N	18	18	14	16	23	23	4256	0 68	16	7438	1 75	0 9	0 02556179	0 24346788	9 52
	62 5	SPT	22	ML	N	22	22	17	20	27	27	4329	0 67	18	7605	1 76	0 9	0 02569385	0 27821408	10 83
	64	SPT	24	ML	N	24	24	18	22	29	29	4401	0 67	19	7771	1 77	0 9	0 02582153	0 29371713	11 37

Assessment of Liquefactio. (ntial - Magnitude 5.5 Event

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
JOB NAME	SMI - Sherwood Tailings Impoundments																			
JOB NO	SMI-1																			
BY	RLV																			
BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (* 55)	CORRECTION FACTORS DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor (sub u	Mag 5 5 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-8	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 01600869	0 25564	15 97
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 01592825	0 239008	15 01
	7 5	SPT	3	ML	N	3	2	2	2	9	9	1557	1 12	10	1557	1 00	0 98	0 01584781	0 15516951	9 79
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1.08	9	1689	1 00	0 97	0 01576736	0 13904185	8 82
	11	SPT	0 5	ML	N	1	1	0	0	7	7	1826	1 05	8	1888	1 03	0 96	0 0161954	0 12019736	7 42
	12 5	SPT	4	ML	N	4	4	3	4	11	11	1899	1 03	11	2055	1 08	0 96	0 01685987	0 16825336	9 98
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0 99	9	2443	1 18	0 95	0 01817783	0 1346503	7 41
	17 5	SPT	2	SM	N	2	2	2	2	9	9	2142	0 98	9	2610	1 22	0 94	0 01865773	0 13261935	7 11
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0.96	8	2776	1 25	0 94	0 01909393	0 11729381	6 14
	21	SPT	1	ML	N	1	1	1	1	8	8	2312	0 95	7	2998	1 30	0 93	0 01961472	0 11503234	5 86
	22 5	SPT	1	SM	N	1	1	1	1	8	8	2385	0 93	7	3165	1 33	0 93	0 01996434	0 11339786	5 68
	24	SPT	2	ML	N	2	2	2	2	9	9	2457	0 92	8	3331	1 36	0 92	0 02028231	0 12455071	6 14
	25 5	SPT	3	ML	N	3	3	2	3	10	10	2530	0 91	9	3498	1 38	0 92	0 02057138	0 13539926	6 56
	29	SPT	1	ML	N	1	1	1	1	8	8	2700	0 88	7	3886	1 44	0 90	0 0211468	0 10684861	5 05
	31	SPT	4	ML	N	4	4	3	4	11	11	2798	0 86	9	4108	1 47	0 90	0 02147537	0 14066873	6 56
T-9	4	SPT	8	SM	N	8	6	5	5	12	12	1250	1 60	20	1250	1 00	0 99	0 0160355	0 305536	19 06
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1 00	0 98	0 01592825	0 239008	15 01
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1 12	10	1557	1 00	0 98	0 01584781	0 15516951	9 79
	9	SPT	6	SM	N	5	4	3	3	10	10	1689	1 08	11	1689	1 00	0 97	0 01576736	0 17276169	10 96
T-10	4 5	SPT	28	SM	N	28	21	16	19	26	26	1294	1 60	41	1294	1 00	0 99	0 01600869	0 638176	39 86
	6	SPT	10	SM	N	10	8	6	7	14	14	1425	1 60	22	1425	1 00	0 98	0 01592825	0 3388	21 27
	7 5	SPT	5	SM	N	5	4	3	3	10	10	1557	1 12	12	1557	1 00	0 98	0 01584781	0 17838046	11 26
	9	SPT	6	ML	N	6	5	3	4	11	11	1689	1 08	12	1689	1 00	0 97	0 01576736	0 1840015	11 67
	11	SPT	8	ML	N	8	8	6	7	14	14	1826	1 05	15	1888	1 03	0 96	0 0161954	0 229101	14 15
	12 5	SPT	2	SM	N	2	2	2	2	9	9	1899	1 03	9	2055	1 08	0 96	0 01685987	0 13968203	8 28
	14	SPT	1	SM	N	1	1	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 01746162	0 12341282	7 07

L.C-162

L.C-163

Magnitude 6.0 Random Earthquake

The following spread sheets present the calculations to assess the liquefaction potential within the tailing materials in the event of a random earthquake of Magnitude 6.0 occurring approximately 104 km from the site and producing a peak horizontal acceleration of 0.015 g at the site. Due to the variable depth of each hole, the calculations are presented on six separate sheets which are divided as follows:

Sheet No.	Borehole No.
1	T-1 and T-2
2	T-3
3	T-4
4	T-5 and T-6
5	T-7
6	T-8, T-9 and T-10

Assessment of Liquefaction Potential - Magnitude 6.0 Event

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
JOB NAME	SMI - Sherwood Tailings Impoundments																			
JOB NO	SMI-1																			
BY	RLV																			
BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD CAL (* 55)	CORRECTION FACTORS DRILL ROD (* 75@<10')	AMMER EFF (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 6 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-1	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1 60	17	1294	1 00	0 99	0 00960521	0 23572	24 54
	6	SPT	7	SM	N	7	5	4	5	12	12	1425	1 60	19	1425	1 00	0 98	0 00955095	0 266392	27 87
	7 5	SPT	8	ML	N	8	6	5	5	12	12	1557	1.12	14	1557	1 00	0 98	0 00950869	0 19058414	20 07
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1 06	9	1689	1 00	0 97	0 00946043	0 12820742	13 55
	11	SPT	3	SM	N	3	3	2	3	10	10	1826	1 05	10	1888	1 03	0 96	0 00971724	0 14430368	14 85
	12 5	SPT	2	SM	N	2	2	2	2	9	9	1899	1 03	9	2055	1 06	0 96	0 01011592	0 12679772	12 73
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 01047697	0 11379623	10 86
T-2	4 5	SPT	5	SM	N	5	4	3	3	10	10	1294	1.60	17	1294	1 00	0 99	0 00960521	0 23572	24 54
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 00955095	0 220384	23 06
	7 5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1 00	0 98	0 00950869	0 14307838	15 06
	9	SPT	0 5	SM	N	1	0	0	0	7	7	1689	1 06	8	1689	1 00	0 97	0 00946043	0 11266131	11 91
	11	SPT	1	SM	N	1	1	1	1	8	8	1826	1 05	8	1888	1 03	0 96	0 00971724	0 11752584	12 09
	12 5	SPT	3	SM	N	3	3	2	3	10	10	1899	1 03	10	2055	1 06	0 96	0 01011592	0 14197021	14 03
	14	SPT	3	SM	N	3	3	2	3	10	10	1971	1 01	10	2221	1.13	0 95	0 01047697	0 13972449	13 34
	16	SPT	3	SM	N	3	3	2	3	10	10	2089	0 99	10	2443	1 18	0 95	0 0109087	0 13685605	12 55
	17 5	SPT	2	SM	N	2	2	2	2	9	9	2142	0 98	9	2610	1 22	0 94	0 01119464	0 12228537	10 92
	21	SPT	1.5	ML	N	2	2	1	1	8	8	2312	0 95	8	2998	1 30	0 93	0 01176883	0 11211071	9 53
	24 5	SPT	3	ML	N	3	3	2	3	10	10	2482	0 91	9	3387	1 38	0 92	0 01222905	0 12600455	10 30

L.C-164

Assessment of Liquefaction Potential - Magnitude 6.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV 10/27/94

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
						CORRECTION FACTORS														
BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (*.55)	DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 6.0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-3	4	SPT	3	SM	N	3	2	2	2	9	9	1250	1.60	14	1250	1.00	0.99	0.0096213	0.205048	21.31
	5.5	SPT	3	SM	N	3	2	2	2	9	9	1381	1.60	14	1381	1.00	0.98	0.00957304	0.205048	21.42
	7	SPT	2	SM	N	2	2	1	1	8	8	1513	1.60	13	1513	1.00	0.96	0.00952478	0.189712	19.92
	9	SPT	0.5	SM	N	1	0	0	0	7	7	1669	1.08	8	1669	1.00	0.97	0.00946043	0.11266131	11.91
	11	SPT	0.5	SM	N	1	1	0	0	7	7	1826	1.05	8	1888	1.03	0.96	0.00971724	0.11083133	11.41
	13	SPT	0.5	SM	N	1	1	0	0	7	7	1923	1.03	8	2110	1.10	0.96	0.01024024	0.10845679	10.59
	14.5	SPT	0.5	ML	N	1	1	0	0	7	7	1996	1.01	8	2277	1.14	0.95	0.01058967	0.10675338	10.08
	16	SPT	0.5	ML	N	1	1	0	0	7	7	2069	0.99	7	2443	1.18	0.95	0.0109067	0.10511109	9.64
	17.5	SPT	0.5	ML	N	1	1	0	0	7	7	2142	0.98	7	2610	1.22	0.94	0.01119484	0.10352568	9.25
	19	SPT	0.5	ML	N	1	1	0	0	7	7	2214	0.96	7	2776	1.25	0.94	0.01145636	0.10199336	8.90
	20.5	SPT	0.5	SM	N	1	1	0	0	7	7	2287	0.95	7	2943	1.29	0.93	0.01169437	0.10051067	8.59
	22	SPT	0.5	SM	N	1	1	0	0	7	7	2360	0.94	7	3109	1.32	0.93	0.01191087	0.0990745	8.32
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0.92	9	3331	1.36	0.92	0.01216939	0.12659101	10.40
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0.88	9	3886	1.44	0.90	0.01268796	0.13219507	10.42

L.C-165

Assessment of Liquefaction ..(..tial - Magnitude 6.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD CAL (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF (* 75)	SPT w/o Liner (*1 2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 6 0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts				CORRECTION FACTORS								Calculation to Determine Liquefaction Potential					
T-4	4 5	SPT	6	SM	N	6	5	3	4	11	11	1294	1 60	18	1294	1 00	0 99	0 00960021	0 251056	26 14
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1 60	16	1425	1 00	0 98	0 00955896	0 220384	23 06
	7 5	SPT	2	SM	N	2	2	1	1	8	8	1557	1 12	9	1557	1 00	0 98	0 00950889	0 13237723	13 92
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1 08	10	1689	1 00	0 97	0 00946043	0 1385715	14 65
	11	SPT	1	ML	N	1	1	1	1	8	8	1826	1 05	8	1888	1 03	0 96	0 00971724	0 11752584	12 09
	12 5	SPT	1	ML	N	1	1	1	1	8	8	1899	1 03	8	2055	1 08	0 96	0 01011592	0 11562522	11 43
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1 01	8	2221	1 13	0 95	0 01047607	0 11379623	10 86
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0 99	9	2443	1 18	0 95	0 0109057	0 12415806	11 38
	17 5	SPT	0	SM	N	0	0	0	0	7	7	2142	0 98	7	2610	1 22	0 94	0 01119484	0 09727246	8 69
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0 96	8	2776	1 25	0 94	0 01145636	0 10815403	9 44
	21	SPT	0	SM	N	0	0	0	0	7	7	2312	0 95	7	2998	1 30	0 93	0 01176683	0 09398503	7 99
	22 5	SPT	1	SM	N	1	1	1	1	8	8	2385	0 93	7	3165	1 33	0 93	0 0119786	0 10456166	8 73
	24	SPT	4	SM	N	4	4	3	4	11	11	2457	0 92	10	3331	1 36	0 92	0 01216939	0 13833657	11 37
	26	SPT	4	SM	N	4	4	3	4	11	11	2555	0 90	10	3553	1 39	0 91	0 01239704	0 13581006	10 96
	29	SPT	4	SM	N	4	4	3	4	11	11	2700	0.88	9	3886	1 44	0 90	0 01268796	0 13219507	10 42
	31	SPT	6	SM	N	6	6	5	5	12	12	2798	0 86	11	4108	1 47	0 90	0 01288522	0 15194907	11 79
	32 5	SPT	6	SM	N	6	6	5	5	12	12	2871	0 85	11	4275	1 49	0 90	0 01306697	0 14998914	11 48
	34	SPT	16	SM	N	16	16	12	14	21	21	2943	0 84	18	4441	1.51	0 90	0 01323971	0 25555459	19 30
	36	SPT	14	SM	N	14	14	11	13	20	20	3041	0 83	16	4663	1 53	0 90	0 01345715	0 23014667	17 10
	37 5	SPT	13	SM	N	13	13	10	12	19	19	3114	0 82	15	4830	1 55	0 90	0 01361133	0 21685644	15 93
	39	SPT	17	SM	N	17	17	13	15	22	22	3186	0 81	18	4996	1 57	0 90	0 01375844	0 25543301	18 57
	41	SPT	14	SM	N	14	14	11	13	20	20	3284	0 79	16	5218	1 59	0 90	0 01394444	0 22088741	15 84
	42 5	SPT	18	SM	N	18	18	14	16	23	23	3357	0 78	18	5385	1 60	0 90	0 01407686	0 25832843	18 35
	44	SPT	13	ML	N	13	13	10	12	19	19	3429	0.77	14	5551	1 62	0 90	0 01420366	0 20576284	14 49
	46	SPT	16	ML	N	16	16	12	14	21	21	3624	0.75	16	5995	1 65	0 90	0 01451684	0 22821045	15 72
	49 5	SPT	15	ML	N	15	15	11	14	21	21	3697	0.74	15	6162	1 67	0 90	0 01462579	0 21610406	14 78
	51	SPT	14	ML	N	14	14	11	13	20	20	3770	0.73	14	6328	1 68	0 90	0 01473063	0 20426477	13 87
	52 5	SPT	12	ML	N	12	12	9	11	18	18	3843	0.73	13	6495	1 69	0 90	0 01483129	0 18341067	12 37
	54	SPT	8	ML	N	8	8	6	7	14	14	3916	0.72	10	6661	1.70	0 90	0 0149283	0 14467674	9 69
	56	SPT	12	ML	N	12	12	9	11	18	18	4013	0 71	13	6883	1 72	0 90	0 01506217	0 17667341	11 87
	57 5	SPT	11	ML	N	11	11	8	10	17	17	4066	0.70	12	7050	1 73	0 90	0 0151412	0 16776976	11 06
	61	SPT	24	ML	N	24	24	18	22	29	29	4256	0 68	19	7438	1 75	0 90	0 01533707	0 27674982	18 04
	62 5	SPT	12	ML	N	12	12	9	11	18	18	4329	0 67	12	7806	1 76	0 90	0 01541631	0 17038522	11 05
	64	SPT	10	SM	N	10	10	8	9	16	16	4401	0 67	11	7771	1.77	0 90	0 01549292	0 15151333	9 78

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Assessment of Liquefaction Potential - Magnitude 6.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO.: SMI-1
BY: RLV

			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
BOREHOLE	DEPTH (ft)	SPT or MOD. CAL.	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	CORRECTION FACTORS MOD. CAL. (* 55)	DRILL ROD (* 75@<10')	HAMMER EFF. (* 75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor	Mag 6.0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
T-5	4.5	SPT	4	SM	N	4	3	2	3	10	10	1294	1.60	16	1294	1.00	0.99	0.00960521	0.220384	22.94
	6	SPT	4	ML	N	4	3	2	3	10	10	1425	1.60	16	1425	1.00	0.98	0.00955695	0.220384	23.06
	7.5	SPT	4	ML	N	4	3	2	3	10	10	1557	1.12	11	1557	1.00	0.98	0.00950869	0.15377953	16.17
	9	SPT	3	SM	N	3	2	2	2	9	9	1689	1.08	10	1689	1.00	0.97	0.00946043	0.1365715	14.65
	11	SPT	4	SM	N	4	4	3	4	11	11	1826	1.05	11	1868	1.03	0.96	0.00971724	0.1676929	16.23
	12.5	SPT	2	ML	N	2	2	2	2	9	9	1899	1.03	9	2055	1.08	0.96	0.01011592	0.12879772	12.73
	14	SPT	1.5	ML	N	2	2	1	1	8	8	1971	1.01	8	2221	1.13	0.95	0.01047897	0.1202763	11.48
	16	SPT	0	ML	N	0	0	0	0	7	7	2069	0.99	7	2443	1.18	0.95	0.0109067	0.0987621	9.06
	17.5	SPT	0	ML	N	0	0	0	0	7	7	2142	0.98	7	2610	1.22	0.94	0.01119464	0.09727246	8.69
	19	SPT	0	ML	N	0	0	0	0	7	7	2214	0.96	7	2776	1.25	0.94	0.01145636	0.09583268	8.37
	23	SPT	3	SM	N	3	3	2	3	10	10	2409	0.93	9	3220	1.34	0.92	0.01204422	0.12778154	10.61
	24.5	SPT	4	SM	N	4	4	3	4	11	11	2482	0.91	10	3387	1.36	0.92	0.01222905	0.1376967	11.26
	26	SPT	6	SM	N	6	6	5	5	12	12	2555	0.90	11	3553	1.39	0.91	0.01239704	0.15887216	12.82
	27.5	SPT	0	SM	N	0	0	0	0	7	7	2628	0.89	6	3720	1.42	0.91	0.0125496	0.0884757	7.05
	29	SPT	2	SP-SM	N	2	2	2	2	2	2	2700	0.88	2	3886	1.44	0.90	0.01268796	0.02244822	1.77
T-6	4.5	SPT	6	SM	N	6	5	3	4	11	11	1294	1.60	18	1294	1.00	0.99	0.00960521	0.251056	26.14
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1.00	0.98	0.00955695	0.220384	23.06
	7.5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1.00	0.98	0.00950869	0.14307838	15.05
	9	SPT	0.5	ML	N	1	0	0	0	7	7	1689	1.08	8	1689	1.00	0.97	0.00946043	0.11266131	11.91

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Assessment of Liquefaction P[illegible]al - Magnitude 6.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD. CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL (*.55)	CORRECTION FACTORS: DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub d	Mag 6.0 Induced Stress Ratio *	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
T-7	4	SPT	9	SM	N	9	7	5	6	13	13	1250	1.60	21	1250	1.00	0.9668	0.0096213	0.297064	30.88
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1.00	0.9602	0.00955895	0.220384	23.06
	7.5	SPT	4	SM	N	4	3	2	3	10	10	1557	1.12	11	1557	1.00	0.97525	0.00950689	0.16377953	16.17
	9	SPT	3	ML	N	3	2	2	2	9	9	1689	1.06	10	1689	1.00	0.9703	0.00946043	0.1365715	14.65
	11	SPT	4	ML	N	4	4	3	4	11	11	1826	1.05	11	1886	1.03	0.9637	0.00971724	0.1576929	16.23
	12.5	SPT	2	ML	N	2	2	2	2	9	9	1890	1.03	9	2055	1.08	0.95875	0.01011582	0.12879772	12.73
	14	SPT	1	ML	N	1	1	1	1	8	8	1971	1.01	8	2221	1.13	0.9538	0.01047897	0.11379623	10.86
	16	SPT	1	SM	N	1	1	1	1	8	8	2069	0.99	8	2443	1.18	0.9472	0.0109087	0.11148008	10.22
	17.5	SPT	1	ML	N	1	1	1	1	8	8	2142	0.98	8	2610	1.22	0.94225	0.01119484	0.10977891	9.81
	19	SPT	5	ML	N	5	5	4	6	12	12	2214	0.96	11	2776	1.25	0.9373	0.01145636	0.15743941	13.74
	21	SPT	1	SM	N	1	1	1	1	8	8	2312	0.95	7	2998	1.30	0.9307	0.01176883	0.10806881	9.01
	22.5	SPT	3	SM	N	3	3	2	3	10	10	2385	0.93	9	3165	1.33	0.92575	0.0119786	0.12836584	10.72
	24	SPT	3	SM	N	3	3	2	3	10	10	2457	0.92	9	3331	1.36	0.9208	0.01218939	0.12669101	10.40
	26	SPT	2	ML	N	2	2	2	2	9	9	2555	0.90	8	3553	1.39	0.9142	0.01239704	0.11274786	9.09
	27.5	SPT	4	SM	N	4	4	3	4	11	11	2628	0.89	9	3720	1.42	0.90925	0.0126496	0.13397749	10.58
	29	SPT	4	ML	N	4	4	3	4	11	11	2700	0.88	9	3886	1.44	0.9043	0.01268798	0.13219507	10.42
	31	SPT	6	ML	N	6	6	5	5	12	12	2798	0.86	11	4108	1.47	0.9	0.01288522	0.15194907	11.79
	32.5	SPT	6	SM	N	6	6	5	5	12	12	2871	0.85	11	4275	1.49	0.9	0.01306697	0.14996914	11.48
	34	SPT	8	SM	N	8	8	6	7	14	14	2943	0.84	12	4441	1.51	0.9	0.01323971	0.16067361	12.81
	36	SPT	2	SM	N	2	2	2	2	9	9	3041	0.83	7	4663	1.53	0.9	0.01345715	0.10333116	7.68
	37.5	SPT	3	SM	N	3	3	2	3	10	10	3114	0.82	8	4830	1.55	0.9	0.01361133	0.11248703	8.26
	39	SPT	4	ML	N	4	4	3	4	11	11	3186	0.81	9	4996	1.57	0.9	0.01375844	0.12141659	8.82
	41	SPT	4	ML	N	4	4	3	4	11	11	3284	0.79	8	5218	1.59	0.9	0.01394444	0.11945962	8.57
	42.5	SPT	6	ML	N	5	5	4	5	12	12	3357	0.78	9	5385	1.60	0.9	0.01407686	0.12806073	9.10
	44	SPT	7	ML	N	7	7	5	6	13	13	3429	0.77	10	5551	1.62	0.9	0.01420366	0.14633758	10.30
	46	SPT	10	ML	N	10	10	8	9	16	16	3527	0.76	12	5773	1.64	0.9	0.01436457	0.17329757	12.06
	47.5	SPT	5	ML	N	5	5	4	5	12	12	3600	0.75	9	5940	1.65	0.9	0.01447954	0.12311187	8.50
	49	SPT	6	ML	N	6	6	5	5	12	12	3672	0.75	9	6106	1.66	0.9	0.01458995	0.13121908	8.99
	51	SPT	9	ML	N	9	9	7	8	15	15	3770	0.73	11	6328	1.68	0.9	0.01473053	0.15736725	10.68
	52.5	SPT	4	ML	N	4	4	3	4	11	11	3843	0.73	8	6495	1.69	0.9	0.01483129	0.1092222	7.36
	54	SPT	12	ML	N	12	12	9	11	18	18	3915	0.72	13	6661	1.70	0.9	0.0149283	0.18135635	12.15
	56	SPT	20	SM	N	20	20	15	18	25	25	4013	0.71	18	6883	1.72	0.9	0.01506217	0.2509458	16.67
	57.5	SPT	22	SM	N	22	22	17	20	27	27	4086	0.70	19	7050	1.73	0.9	0.0151412	0.26604909	17.57
	59	SPT	20	SM	N	20	20	15	18	25	25	4158	0.69	17	7216	1.74	0.9	0.01522711	0.24546336	16.12
	61	SPT	18	ML	N	18	18	14	16	23	23	4256	0.68	16	7438	1.75	0.9	0.01533707	0.22449636	14.64
	62.5	SPT	22	ML	N	22	22	17	20	27	27	4329	0.67	18	7605	1.76	0.9	0.01541631	0.25653506	16.64
	64	SPT	24	ML	N	24	24	18	22	29	29	4401	0.67	19	7771	1.77	0.9	0.01549292	0.27043008	17.46

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Assessment of Liquefaction (ıtial - Magnitude 6.0 Event

JOB NAME: SMI - Sherwood Tailings Impoundments
JOB NO: SMI-1
BY: RLV

BOREHOLE	DEPTH (ft)	SPT or MOD CAL	UNCORRECTED BLOW COUNT N	SOIL TYPE	GRAVEL? Y=yes N=no	MOD. CAL. (*.55)	DRILL ROD (*.75@<10')	HAMMER EFF. (*.75)	SPT w/o Liner (*1.2)	SILTS (add 7)	FINAL CORRECTED N	Effective Overburden (psf)	Cn	(N1)60	Total Overburden (psf)	Overburden Stress Ratio	Stress Reduction Factor r sub u	Mag 6.0 Induced Stress Ratio	Available Stress Ratio Based on (N1)60	Factor of Safety Against Liquefaction
			Corrections to Obtain SPT-Equivalent Blow Counts												Calculations to Determine Liquefaction Potential					
T-8	4.5	SPT	5	SM	N	5	4	3	3	10	10	1294	1.60	17	1294	1.00	0.99	0.00960521	0.27656	28.69
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1.00	0.98	0.00955695	0.257632	26.96
	7.5	SPT	3	ML	N	3	2	2	2	9	9	1557	1.12	10	1557	1.00	0.98	0.00950869	0.16726064	17.59
	9	SPT	2	SM	N	2	2	1	1	8	8	1689	1.08	9	1689	1.00	0.97	0.00946043	0.14987628	15.84
	11	SPT	0.5	ML	N	1	1	0	0	7	7	1826	1.05	8	1858	1.03	0.96	0.00971724	0.12956338	13.33
	12.5	SPT	4	ML	N	4	4	3	4	11	11	1899	1.03	11	2055	1.08	0.96	0.01011592	0.18136401	17.93
	16	SPT	2	SM	N	2	2	2	2	9	9	2069	0.99	9	2443	1.18	0.95	0.0109067	0.14514253	13.31
	17.5	SPT	2	SM	N	2	2	2	2	9	9	2142	0.98	9	2610	1.22	0.94	0.01119464	0.14295332	12.77
	19	SPT	1	ML	N	1	1	1	1	8	8	2214	0.96	8	2776	1.25	0.94	0.01145638	0.12643356	11.04
	21	SPT	1	ML	N	1	1	1	1	8	8	2312	0.95	7	2998	1.30	0.93	0.01178863	0.12399594	10.54
	22.5	SPT	1	SM	N	1	1	1	1	8	8	2385	0.93	7	3165	1.33	0.93	0.0119786	0.12223406	10.20
	24	SPT	2	ML	N	2	2	2	2	9	9	2457	0.92	8	3331	1.36	0.92	0.01216939	0.13425696	11.03
	26.5	SPT	3	ML	N	3	3	2	3	10	10	2530	0.91	9	3498	1.38	0.92	0.01234283	0.14594985	11.82
	29	SPT	1	ML	N	1	1	1	1	8	8	2700	0.88	7	3886	1.44	0.90	0.01268796	0.11817447	9.08
	31	SPT	4	ML	N	4	4	3	4	11	11	2798	0.86	9	4108	1.47	0.90	0.01288622	0.15184561	11.78
T-9	4	SPT	8	SM	N	8	6	5	5	12	12	1250	1.60	20	1250	1.00	0.99	0.0096213	0.329344	34.23
	6	SPT	4	SM	N	4	3	2	3	10	10	1425	1.60	16	1425	1.00	0.98	0.00955695	0.257632	26.96
	7.5	SPT	3	SM	N	3	2	2	2	9	9	1557	1.12	10	1557	1.00	0.98	0.00950869	0.16726064	17.59
	9	SPT	5	SM	N	5	4	3	3	10	10	1689	1.08	11	1689	1.00	0.97	0.00946043	0.18622353	19.68
T-10	4.5	SPT	26	SM	N	26	21	16	19	26	26	1294	1.60	41	1294	1.00	0.99	0.00960521	0.687904	71.62
	6	SPT	10	SM	N	10	8	6	7	14	14	1425	1.60	22	1425	1.00	0.98	0.00955695	0.3652	38.21
	7.5	SPT	6	SM	N	5	4	3	3	10	10	1557	1.12	12	1557	1.00	0.98	0.00950869	0.19228024	20.22
	9	SPT	6	ML	N	6	5	3	4	11	11	1689	1.08	12	1689	1.00	0.97	0.00946043	0.19833927	20.97
	11	SPT	8	ML	N	8	8	6	7	14	14	1826	1.05	15	1858	1.03	0.96	0.00971724	0.24696303	25.41
	12.5	SPT	2	SM	N	2	2	2	2	9	9	1899	1.03	9	2055	1.08	0.96	0.01011592	0.15066635	14.88
	14	SPT	1	SM	N	1	1	1	1	8	8	1971	1.01	8	2221	1.13	0.95	0.01047697	0.1330294	12.70

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SHERWOOD PROJECT
TAILING RECLAMATION PLAN

Volume 5 of 7
APPENDICES L, M, N, O

Prepared for

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December 1994

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Shepherd Miller, Inc.

APPENDIX N
MAIN EMBANKMENT SLOPE STABILITY

TABLE OF CONTENTS

APPENDIX N

SECTION N.1
CALCULATION BRIEF

CALCULATION BRIEF FOR MAIN EMBANKMENT SLOPE STABILITY

PURPOSE:

In accordance with current regulations, the out-slope of the main Tailing Embankment will be reduced to 5H:1V. The purpose of these calculations is to evaluate the long-term structural stability of the modified main embankment.

METHOD:

The PC Stable 5M computer program (Verduin and Thomas, 1987) was used to evaluate the structural stability of the main embankment. These results were compared to the analyses of the existing embankment that were performed by D'Appolonia (1977).

ASSUMPTIONS:

The Stable 5M program was run at the location of the longest slope length of the main embankment, shown in Figure 1, and therefore represents the most critical case for slope stability. Program output is presented in Attachment A. Material properties used in the model are presented in Table 1, and the embankment cross-section is shown in Figure 2.

Other assumptions are as follows:

No phreatic surface is present. The embankment is separated from the tailing by an impermeable liner. It was assumed that any water which might seep from the impoundment would drain through the embankment without forming a phreatic surface because of the free draining nature of the underlying foundation material.

Furthermore, in order for a phreatic surface to exist in the embankment, the entire 150 to 200-feet of sandy material between the base of the embankment and the water table would have to become saturated. Given the 150 to 200-foot thickness of currently unsaturated foundation between the embankment and the water table, and the free draining

nature of the embankment and foundation it is not feasible for a phreatic surface to exist in the embankment.

The densities and internal friction angles for the existing embankment and the foundation soils were obtained from information in the Engineers Report "Earth Dam Design Tailings Storage Facility," prepared by D'Appolonia Consulting Engineers, Inc. for Western Nuclear, Inc. July 1977.

Total densities for the clay layers were calculated using information presented in Appendix A, Section A.2. The total density of each clay layer was calculated using the following equation:

$$\rho_t = \rho_d \ (1+w)$$

Where:

ρ_t = total density
ρ_d = dry density (assumed 95% standard Proctor density)
w = water content (assumed 15-bar moisture content)

Internal friction angles and the cohesion intercept for the clay layers of the cover were based on typical values for clay obtained from the "Geotechnical Engineering Investigation Manual," (HUNT, 1984). The lowest table value for the cohesion intercept (0.2 kg/cm^2 $\approx$ 410 psf) was chosen along with a typical internal friction angle (30^0) for loess and glacial soils.

The properties of the sand borrow material that will be used to create the new embankment outslope and the sand layer of the reclamation cover are presented in Appendix A, Section A.3, representing sand material "SB." The total density of the material in the new embankment outslope was conservatively assumed to be 80% of the standard Proctor density of 117.8 pcf for this material. This density will be achieved by placing the material in lifts without any active compaction are field quality control. The internal friction angle of 30^0 was assumed based on the lower end of the range of values (28^0 to 42^0) given by Holtz and Kovacs (1981) for loose sands.

Trees and other vegetation that are anticipated to become established on the reclaimed embankment will provide increased resistance to slope failure through propagation of root structures into the embankment material. Buried root structures will continue to provide anchorage if mature growth dies (i.e., fire, blight) until successional vegetation populations become re-established. In addition, rooting in the embankment material will not decrease fill material densities below the conservatively low values assumed for these analyses.

Tailing density was obtained from information presented in Appendix I, Radon Barrier Design. Total density was calculated assuming saturated conditions using the following formula:

$$\rho_t = \rho_d + \eta(\rho_w)$$

Where:

ρ_t = total density (pcf)
ρ_d = dry density (82.05 pcf, see Appendix I)
η = porosity (0.516, see Appendix I)
ρ_w = density of water (62.4 pcf)

The tailing internal friction angle of 30^0 was estimated assuming a loose sand. It was conservatively assumed that the tailings have no cohesion.

Pertinent information from the D'Appolonia report and tables from Hunt and Holtz and Kovacs are presented in Attachment A.1.

A seismic coefficient of 0.05 was used in the pseudo-static analysis to determine stability under earthquake loading. This value is based on the results of a seismicity and earthquake-induced liquefaction and settlement evaluation of the Sherwood Project Tailing Impoundment area presented in Attachment D to Appendix L of the TRP. Liquefaction of the embankment will not occur since the materials will be unsaturated.

TABLE 1 - MATERIAL PROPERTIES USED IN STABLE 5M PROGRAM

SOIL DESIGNATION	MATERIAL LOCATION	MATERIAL	TOTAL UNIT WEIGHT (pcf)	INTERNAL FRICTION ANGLE	COHESION INTERCEPT (psf)
SOIL 1	FOUNDATION SOIL	---	115.0	39.0	0
SOIL 2	OUTSLOPE EXTENSION	SB	94.2	30.0	0
SOIL 2	IMPOUNDMENT TOP LAYER	SB	94.2	30.0	0
SOIL 3	EXISTING EMBANKMENT	---	110.0	37.0	0
SOIL 4	IMPOUNDMENT UPPER CLAY LAYER	CA	120.1	30.0	410
SOIL 5	IMPOUNDMENT LOWER CLAY LAYER	CC	113.7	30.0	410
SOIL 6	TAILING	TAILINGS	114.3	30.0	0

SUMMARY:

The Stable 5M program found the main embankment to be stable with a minimum factor of safety of 2.9 for the static condition and 2.3 for the dynamic condition. The failure surfaces for the two cases are shown in Figures 3 and 4. These factors of safety are much greater than the standard acceptable values of 1.5 for static and 1.1 for dynamic conditions. These results were expected since flattening the out-slope of the embankment will effectively create a buttress for the existing embankment and therefore will lead to higher factors of safety than reported by D'Appolonia (1977) for the original embankment. D'Appolonia (1977) reported factors of safety of 1.9 to 2.0 for static conditions and 1.6 to 1.1 for seismic conditions (Attachment A.1).

Liquefaction occurs in saturated loose sands when these materials are subjected to dynamic loading. D'Appolonia (1977) assessed the potential for liquefaction failure of the existing tailings dam and calculated that liquefaction is not of concern. Buttressing the embankment by flattening the outslope will cause the potential of liquefaction to be even more remote. Therefore, since the existing embankment is not subject to liquefaction, the final reclaimed embankment will not be subject to liquefaction.

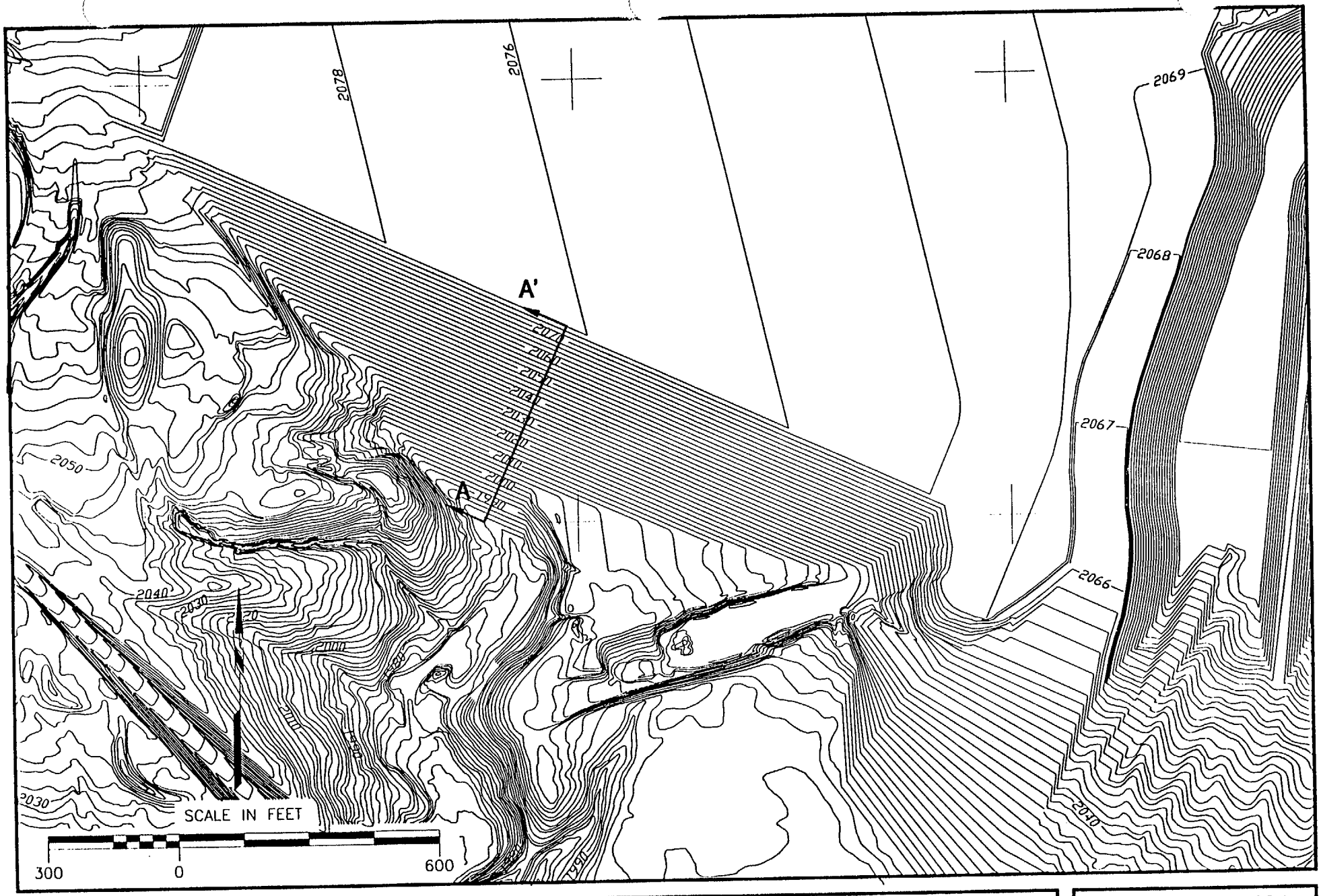
REFERENCES:

D'Appolonia Consulting Engineers, Inc. (1977). "Engineer's Report, Earth Dam Design, Tailings Storage Facility, Western Nuclear Inc., Sherwood Project, Spokane Washington," Project No. RM77-400, July.

Holtz, R.D., and W. D. Kovacs (1981). "An introduction to Geotechnical Engineering," Prentice-Hall, Englewood Cliffs, N.J., p.516.

Hunt, R.E. (1984). "Geotechnical Engineering Investigation Manual," McGraw Hill, N.Y., pp.127-243.

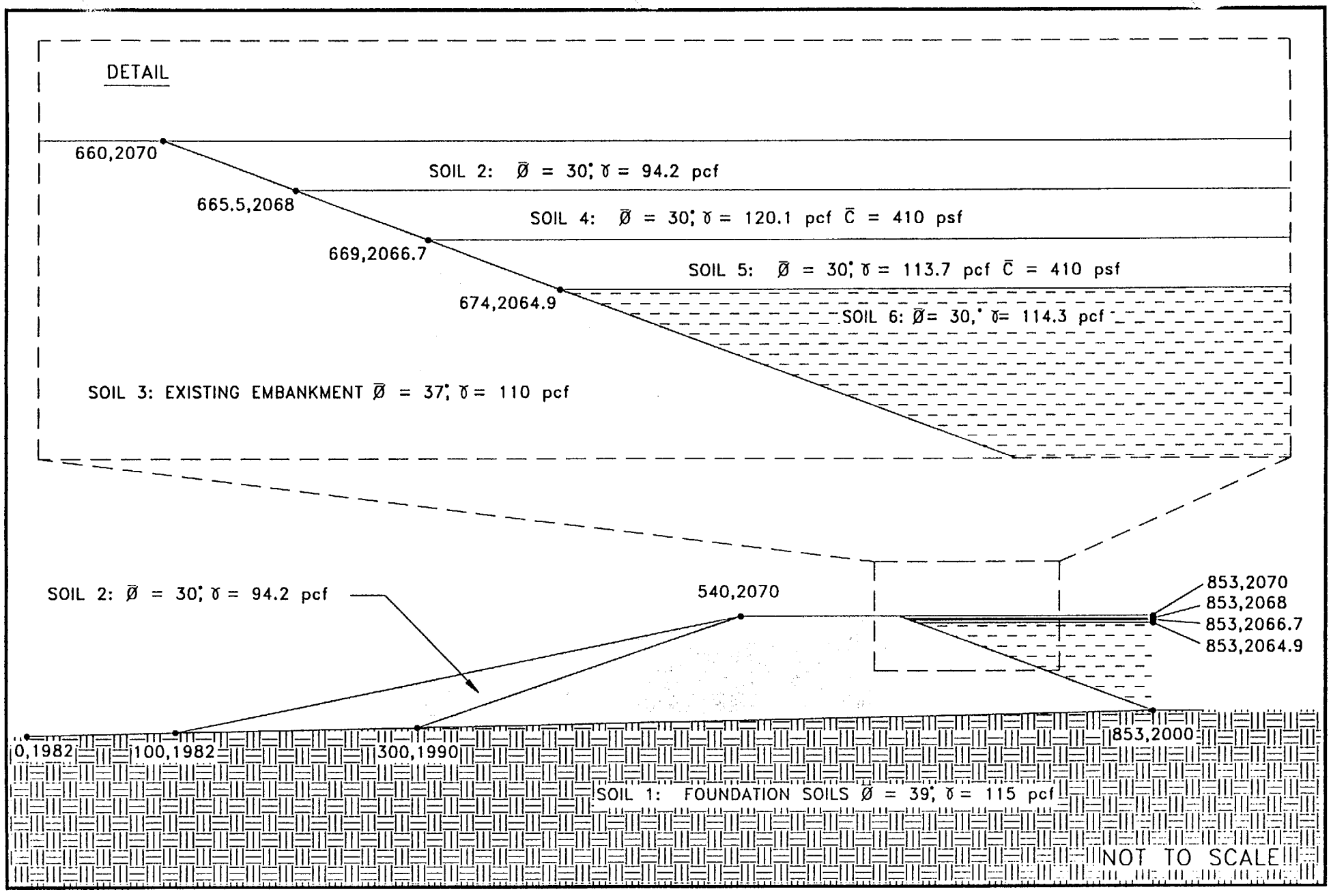
Verduin, J.R., and J.E. Thomas (1987). "Computerized Slope Stability Analysis for Indiana Highways," PC STABL5M, Developed for the Indiana Department of Transportation, Purdue University.



SMI
SHEPHERD MILLER, INC.

FIGURE 1
EMBANKMENT STABLILITY ANALYSIS LOCATION

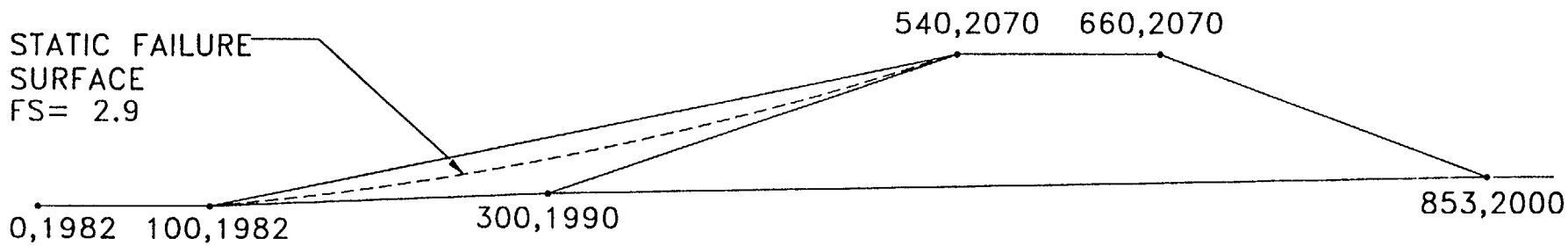
Date:	DEC., 1994
Project:	317
File:	EMBX



SMI
SHEPHERD MILLER, INC.

FIRGURE 2
CROSS SECTION A–A'
PC STBL5M PROFILE

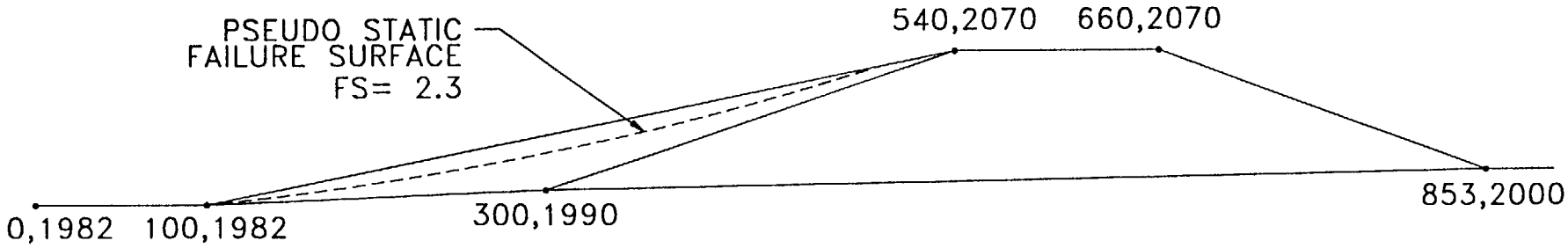
Date:	DEC., 1994
Project:	317
File:	XSECTAA



SMI
SHEPHERD MILLER, INC.

FIRGURE 3
STATIC
FAILURE SURFACE

Date:	DEC., 1994
Project:	317
File:	STATIC



SMI
SHEPHERD MILLER, INC.

FIRGURE 4
PSEUDO STATIC
FAILURE SURFACE

Date:	DEC., 1994
Project:	317
File:	PSEUDO

ATTACHMENT A
SLOPE STABILITY OF MAIN EMBANKMENT

```
STABLE 5M PROGRAM INPUT - MAIN.ST5
PROFIL
JOB#317 - MAIN EMBANKMENT SLOPE STABILITY
14 4
0. 1982. 100. 1982. 1
100. 1982. 540. 2070. 2
540. 2070. 660. 2070. 3
660. 2070. 710. 2070. 2
100. 1982. 300. 1990. 1
300. 1990. 540. 2070. 3
660. 2070. 665.5 2068. 3
665.5 2068. 715.5 2068. 4
665.5 2068. 669. 2066.7 3
669. 2066.7 719. 2066.7 5
669. 2066.7 674. 2064.9 3
674. 2064.9 724. 2064.9 6
674. 2064.9 853. 2000. 3
300. 1990. 853. 2000. 1
SOIL
6
115. 0. 0. 39. 0. 0. 0
94.2 0. 0. 30. 0. 0. 0
110. 0. 0. 37. 0. 0. 0
120.1 0. 410. 30. 0. 0. 0
113.7 0. 410. 30. 0. 0. 0
114.3 0. 0. 30. 0. 0. 0
CIRCL2
20 20
50. 100. 490. 540.
150. 20. 0. 0.
EQUAKE
0.05 0. 0.
CIRCL2
20 20
50. 100. 490. 540.
150. 20. 0. 0.
```

** PCSTABL5M **
by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 11/94
Time of Run:
Run By: JGC
Input Data Filename: MAINC.ST5
Output Filename: MAINC.OUT

PROBLEM DESCRIPTION JOB#317 - MAIN EMBANKMENT SLOPE STABILITY

BOUNDARY COORDINATES

4 Top Boundaries
14 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	1982.00	100.00	1982.00	1
2	100.00	1982.00	540.00	2070.00	2
3	540.00	2070.00	660.00	2070.00	3
4	660.00	2070.00	710.00	2070.00	2
5	100.00	1982.00	300.00	1990.00	1
6	300.00	1990.00	540.00	2070.00	3
7	660.00	2070.00	665.50	2068.00	3
8	665.50	2068.00	715.50	2068.00	4
9	665.50	2068.00	669.00	2066.70	3
10	669.00	2066.70	719.00	2066.70	5
11	669.00	2066.70	674.00	2064.90	3
12	674.00	2064.90	724.00	2064.90	6
13	674.00	2064.90	853.00	2000.00	3
14	300.00	1990.00	853.00	2000.00	1

ISOTROPIC SOIL PARAMETERS

6 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	115.0	.0	.0	39.0	.00	.0	0
2	94.2	.0	.0	30.0	.00	.0	0
3	110.0	.0	.0	37.0	.00	.0	0
4	120.1	.0	410.0	30.0	.00	.0	0
5	113.7	.0	410.0	30.0	.00	.0	0
6	114.3	.0	.0	30.0	.00	.0	0

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between X = 50.00 ft.
and X = 100.00 ft.

Each Surface Terminates Between X = 490.00 ft.
and X = 540.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y =150.00 ft.

20.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	1982.00
2	119.91	1983.85
3	139.81	1985.90
4	159.68	1988.14
5	179.53	1990.58
6	199.36	1993.21
7	219.16	1996.04
8	238.93	1999.06
9	258.67	2002.28
10	278.37	2005.70
11	298.05	2009.31
12	317.68	2013.11
13	337.28	2017.11
14	356.83	2021.30
15	376.35	2025.68
16	395.82	2030.25
17	415.24	2035.02
18	434.62	2039.98
19	453.94	2045.13
20	473.22	2050.47
21	492.44	2055.99
22	511.60	2061.71
23	530.71	2067.62
24	535.06	2069.01

Circle Center At X = -77.3 ; Y = 3999.5 and Radius, 2025.3

*** 2.901 ***

Individual data on the 23 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	19.9	2001.0	.0	.0	.0	.0	.0	.0	.0

2	19.9	5809.0	.0	.0	.0	.0	.0	.0	.0
3	19.9	9233.8	.0	.0	.0	.0	.0	.0	.0
4	19.9	12275.4	.0	.0	.0	.0	.0	.0	.0
5	19.8	14933.2	.0	.0	.0	.0	.0	.0	.0
6	19.8	17208.0	.0	.0	.0	.0	.0	.0	.0
7	19.8	19100.4	.0	.0	.0	.0	.0	.0	.0
8	19.7	20611.7	.0	.0	.0	.0	.0	.0	.0
9	19.7	21742.0	.0	.0	.0	.0	.0	.0	.0
10	19.7	22493.7	.0	.0	.0	.0	.0	.0	.0
11	19.6	22868.2	.0	.0	.0	.0	.0	.0	.0
12	19.6	22867.3	.0	.0	.0	.0	.0	.0	.0
13	19.6	22493.0	.0	.0	.0	.0	.0	.0	.0
14	19.5	21748.1	.0	.0	.0	.0	.0	.0	.0
15	19.5	20635.2	.0	.0	.0	.0	.0	.0	.0
16	19.4	19157.1	.0	.0	.0	.0	.0	.0	.0
17	19.4	17316.6	.0	.0	.0	.0	.0	.0	.0
18	19.3	15118.4	.0	.0	.0	.0	.0	.0	.0
19	19.3	12564.6	.0	.0	.0	.0	.0	.0	.0
20	19.2	9660.6	.0	.0	.0	.0	.0	.0	.0
21	19.2	6409.1	.0	.0	.0	.0	.0	.0	.0
22	19.1	2815.9	.0	.0	.0	.0	.0	.0	.0
23	4.3	106.7	.0	.0	.0	.0	.0	.0	.0

A Horizontal Earthquake Loading Coefficient
Of .050 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = .0 psf

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.

400 Trial Surfaces Have Been Generated.

20 Surfaces Initiate From Each Of 20 Points Equally Spaced
Along The Ground Surface Between X = 50.00 ft.
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First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	1982.00
2	119.90	1984.04
3	139.77	1986.28
4	159.62	1988.71
5	179.45	1991.33
6	199.25	1994.16
7	219.02	1997.17
8	238.76	2000.39
9	258.47	2003.79
10	278.14	2007.39
11	297.78	2011.19
12	317.38	2015.18
13	336.93	2019.36
14	356.45	2023.73
15	375.92	2028.30
16	395.35	2033.06
17	414.73	2038.01
18	434.05	2043.15
19	453.33	2048.48
20	472.55	2053.99
21	491.72	2059.70
22	497.62	2061.52

Circle Center At X = -96.6 ; Y = 3999.1 and Radius, 2026.6

*** 2.296 ***

Individual data on the 21 slices

Slice No.	Width Ft(m)	Weight Lbs(kg)	Water Force Top Lbs(kg)	Water Force Bot Lbs(kg)	Tie Force Norm Lbs(kg)	Tie Force Tan Lbs(kg)	Earthquake Force Hor Lbs(kg)	Earthquake Force Ver Lbs(kg)	Surcharge Load Lbs(kg)
1	19.9	1817.2	.0	.0	.0	.0	90.9	.0	.0
2	19.9	5258.7	.0	.0	.0	.0	262.9	.0	.0
3	19.9	8317.4	.0	.0	.0	.0	415.9	.0	.0
4	19.8	10993.3	.0	.0	.0	.0	549.7	.0	.0
5	19.8	13287.3	.0	.0	.0	.0	664.4	.0	.0
6	19.8	15199.3	.0	.0	.0	.0	760.0	.0	.0
7	19.7	16730.6	.0	.0	.0	.0	836.5	.0	.0
8	19.7	17882.1	.0	.0	.0	.0	894.1	.0	.0
9	19.7	18655.4	.0	.0	.0	.0	932.8	.0	.0
10	19.6	19052.1	.0	.0	.0	.0	952.6	.0	.0
11	19.6	19074.2	.0	.0	.0	.0	953.7	.0	.0
12	19.6	18723.5	.0	.0	.0	.0	936.2	.0	.0
13	19.5	18002.9	.0	.0	.0	.0	900.1	.0	.0
14	19.5	16914.6	.0	.0	.0	.0	845.7	.0	.0
15	19.4	15461.8	.0	.0	.0	.0	773.1	.0	.0
16	19.4	13647.8	.0	.0	.0	.0	682.4	.0	.0
17	19.3	11475.8	.0	.0	.0	.0	573.8	.0	.0
18	19.3	8949.4	.0	.0	.0	.0	447.5	.0	.0
19	19.2	6072.6	.0	.0	.0	.0	303.6	.0	.0
20	19.2	2849.5	.0	.0	.0	.0	142.5	.0	.0
21	5.9	177.9	.0	.0	.0	.0	8.9	.0	.0

ATTACHMENT A.1
REPRODUCTION OF TABLES FROM HUNT, LABORATORY DATA, AND INFORMATION FROM THE D'APPOLONIA REPORT

An Introduction to Geotechnical Engineering

ROBERT D. HOLTZ, PH.D., P.E.
Purdue University
West Lafayette, IN

WILLIAM D. KOVACS, PH.D., P.E.
National Bureau of Standards
Washington, DC

TABLE 11-2 Angle of Internal Friction of Cohesionless Soils*

No.	General Description	Grain Shape	D_{10} (mm)	C_u	Loose e	Loose ϕ(deg)	Dense e	Dense ϕ(deg)
1	Ottawa standard sand	Well rounded	0.56	1.2	0.70	28	0.53	35
2	Sand from St. Peter sandstone	Rounded	0.16	1.7	0.69	31	0.47	37†
3	Beach sand from Plymouth, MA	Rounded	0.18	1.5	0.89	29	—	—
4	Silty sand from Franklin Falls Dam site, NH	Subrounded	0.03	2.1	0.85	33	0.65	37
5	Silty sand from vicinity of John Martin Dam, CO	Subangular to subrounded	0.04	4.1	0.65	36	0.45	40
6	Slightly silty sand from the shoulders of Ft. Peck Dam, MT	Subangular to subrounded	0.13	1.8	0.84	34	0.54	42
7	Screened glacial sand, Manchester, NH	Subangular	0.22	1.4	0.85	33	0.60	43
8‡	Sand from beach of hydraulic fill dam, Quabbin Project, MA	Subangular	0.07	2.7	0.81	35	0.54	46
9	Artificial, well-graded mixture of gravel with sands No. 7 and No. 3	Subrounded to subangular	0.16	68	0.41	42	0.12	57
10	Sand for Great Salt Lake fill (dust gritty)	Angular	0.07	4.5	0.82	38	0.53	47
11	Well-graded, compacted crushed rock	Angular	—	—	—	—	0.18	60

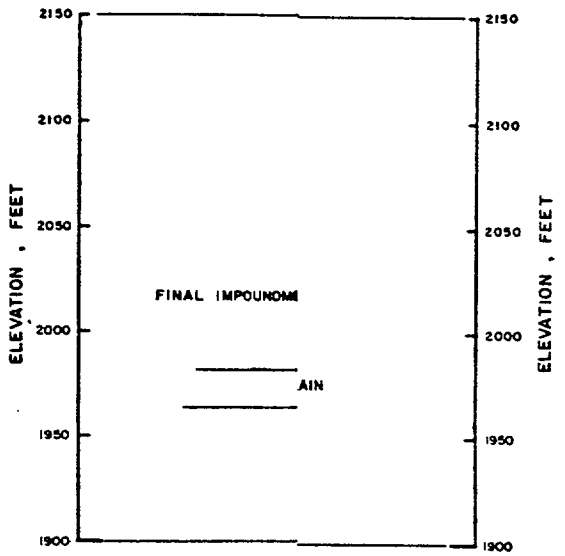
*By A. Casagrande.

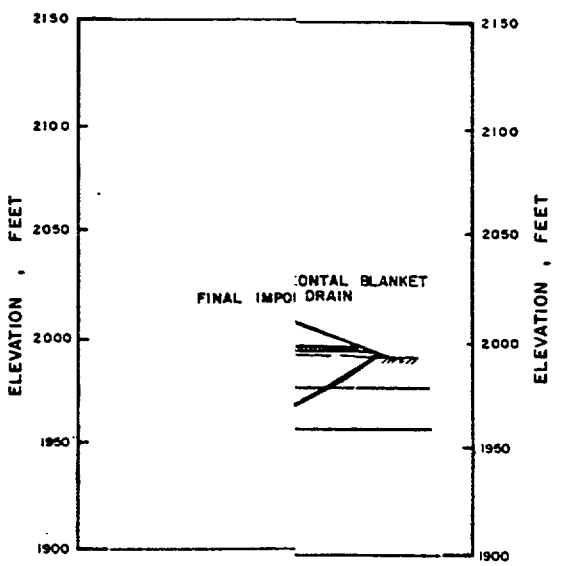
†The angle of internal friction of the undisturbed St. Peter sandstone is larger than 60° and its cohesion so small that slight finger pressure or rubbing, or even stiff blowing at a specimen by mouth, will destroy it.

‡Angle of internal friction measured by direct shear test for No. 8, by triaxial tests for all others.

DRAWN BY	TEJ	CHECKED BY	TAL	DRAWING NUMBER	RM77-400-E3
	4-13-77	APPROVED BY			

SOIL STRENGTH PARAMETERS			
SOIL	γ (p.c.f.)	$\bar{\phi}$ (deg.)	$\bar{c}$ (p.s.f.)
EMBANKMENT SOILS (FINE TO MED. SAND)	110	37	0
FOUNDATION SOILS (FINE TO MED. SAND)	115	39	0





FAILURE SURFACE	STATIC * SAFETY FACTOR	SEI FC
①	1.93	0.0 0.1 0.2 0.2
②	2.01	0.0 0.1 0.2 0.2

* CASE II

** CASE III

FIGURE 7

STABILITY ANALYSES

PREPARED FOR

DRAVO CORPORATION
DENVER, COLORADO

D'APPOLONIA

GEOTECHNICAL ENGINEERING INVESTIGATION MANUAL

ROY E. HUNT
Consulting Engineer

McGraw-Hill Book Company
New York St. Louis San Francisco Auckland
Bogotá Hamburg Johannesburg London Madrid
Mexico Montreal New Delhi Panama Paris
São Paulo Singapore Sydney Tokyo Toronto

TABLE 3.38
TYPICAL PROPERTIES OF FORMATIONS OF COHESIVE MATERIALS

Material	Type*	Location	γ_d, g/cm³	w_n, %	LL, %	PI, %	s_u, kg/cm²	$\bar{c}$, kg/cm²	$\bar{\phi}$	Remarks
CLAY SHALES (WEATHERED)										
Carlisle (Cret.)	CH	Nebraska	1.48	18				0.5	45	ϕ extremely variable
Bearpaw (Cret.)	CH	Montana	1.44	32	130	90		0.35	15	
Pierre (Cret.)		South Dakota	1.47	28				0.9	12	
Cucaracha (Cret.)	CH	Panama Canal		12	80	45				ϕ_r = 10°
Pepper (Cret.)	CH	Waco, Texas		17	80	58		0.4	17	ϕ_r = 7°
Bear Paw (Cret.)	CH	Saskatchewan		32	115	92		0.4	20	ϕ_r = 8°
Modelo (Tert.)	CH	Los Angeles	1.44	29	66	31		1.6	22	Intact specimen
Modelo (Tert.)	CH	Los Angeles	1.44	29	66	31		0.32	27	Shear zone
Martinez (Tert.)	CH	Los Angeles	1.66	22	62	38		0.25	26	Shear zone
(Eocene)	CH	Menlo Park, Calif.	1.65	30	60	50		Free swell 100%: P = 10 kg/cm²		
RESIDUAL SOILS										
Gneiss	CL	Brazil; buried	1.29	38	40	16		0	40	e_0 = 1.23
Gneiss	ML	Brazil; slopes	1.34	22	40	8		0.39	19	c, ϕ— unsoaked
Gneiss	ML	Brazil; slopes	1.34		40	8		0.28	21	
COLLUVIUM										
From shales	CL	West Virginia		28	48	25		0.28	28	ϕ_r = 16°
From gneiss	CL	Brazil	1.10	26	40	16		0.2	31	ϕ_r = 12°
ALLUVIUM										
Back swamp	OH	Louisiana	0.57	140	120	85	0.15			
Back swamp	OH	Louisiana	1.0	60	85	50	0.1			
Back swamp	MH	Georgia	0.96	54	61	22	0.3			e_0 = 1.7
Lacustrine	CL	Great Salt Lake	0.78	50	45	20	0.34			
Lacustrine	CL	Canada	1.11	62	33	15	0.25			
Lacustrine (volcanic)	CH	Mexico City	0.29	300	410	260	0.4			e_0 = 7, S_t = 13
Estuarine	CH	Thames River	0.78	90	115	85	0.15			
[illegible]arine	CH	Lake Maricaibo		65	73	50	0.25			
[illegible]arine	CH	Bangkok		130	118	75	0.05			
Estuarine	MH	Maine		80	60	30	0.2			
MARINE SOILS (OTHER THAN ESTUARINE)										
Offshore	MH	Santa Barbara, Calif.	0.83	80	83	44	0.15			e_0 = 2.28
Offshore	CH	New Jersey		65	95	60	0.65			
Offshore	CH	San Diego	0.58	125	111	64	0.1			Depth = 2 m
Offshore	CH	Gulf of Maine	0.58	163	124	78	0.05			
Coastal Plain	CH	Texas (Beaumont)	1.39	29	81	55	1.0	0.2	16	ϕ_r = 14, e_0 = 0.8
Coastal Plain	CH	London	1.60	25	80	55	2.0			
LOESS										
Silty	ML	Nebraska-Kansas	1.23	9	30	8		0.6	32	Natural w%
Silty	ML	Nebraska Kansas	1.23	(35)	30	8		0	23	Prewetted
Clayey	CL	Nebraska-Kansas	1.25	9	37	17		2.0	30	Natural w%
GLACIAL SOILS										
Till	CL	Chicago	2.12	23	37	21	3.5			
Lacustrine (varved)	CL	Chicago	1.69	22	30	15	1.0			e_0 = 0.6 (OC)
Lacustrine (varved)	CL	Chicago		24	30	13	0.1			e_0 = 1.2 (NC)
Lacustrine (varved)	CH	Chicago	1.18	50	54	30	0.1			
Lacustrine (varved)	CH	Ohio	0.96	46	58	31	0.6			S_t = 4
Lacustrine (varved)	CH	Detroit	1.20	46	55	30	0.8			e_0 = 1.3 (clay)
Lacustrine (varved)	CH	New York City		46	62	34	1.0			e_0 = 1.25 (clay)
Lacustrine (varved)	CL	Boston	1.35	38	50	26	0.8			S_t = 3
Lacustrine (varved)	CH	Seattle		30	55	22			30	ϕ_r = 13°
Marine†	CH	Canada-Leda clay	0.89	80	60	32	0.5			S_t = 128
Marine†	CL	Norway	1.34	40	38	15	0.13			S_t = 7
Marine†	CL	Norway	1.29	43	28	15	0.05			S_t = 75

*See Figure 3.12.

†Marine clays strongly leached.