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Operating Specification	[]	Interface Control Drawing	[]	Spares Multiple Unit Listing	[]
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Conceptual Design Report	[]	Installation Procedure	[]	Component Index	[]
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Cog. Eng. V Roscha	11-18-94	PE	
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QA JR McGee	11-18-94	Safety	
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Document Title:	Definitive Design Report - Design Report Project W-025, Radioactive Mixed Waste (RMW) Land Disposal Facility NON-DRAG-OFF
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7. Abstract

WHC-SD-W025-FDR-001 provides final design calculations for the Radioactive Mixed Waste Disposal Facility.

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DESIGN REPORT
PROJECT W-025, RADIOACTIVE MIXED WASTE (RMW)
LAND DISPOSAL FACILITY
NON-DRAG-OFF

REVISION 1

VOLUME I

Prepared for the
U.S. Department of Energy-Richland Operations Office
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by
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VOLUME II

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NOTE: Dashed lines in the margins indicate the latest revisions to this document.

1. INTRODUCTION

1.1 Purpose

The purpose of this report is to describe the definitive design of the Radioactive Mixed Waste (RMW) Non-Drag-Off disposal facility, Project W-025. This report presents a description of the major landfill design features and a discussion of how each of the regulatory criteria is addressed in the design. The appendices include laboratory test results, design drawings, and individual analyses that were conducted in support of the design.

Revision 1 of this document incorporates design changes resulting from an increase in the required operating life of the W-025 landfill from 2 to 20 years. The rationale for these design changes is described in Golder Associates Inc. 1991a. These changes include (1) adding a 1.5-foot-thick layer of compacted admix directly under the primary FML on the floor of the landfill to mitigate the effects of possible stress cracking in the primary flexible membrane liner (FML), and (2) increasing the operations layer thickness from two to three feet over the entire landfill area, to provide additional protection for the secondary admix layer against mechanical damage and the effects of freezing and desiccation.

The design of the W-025 Landfill has also been modified in response to the results of the EPA Method 9090 chemical compatibility testing program (Golder Associates Inc. 1991b and 1991c), which was completed after the original design was prepared. This program consisted of testing geosynthetic materials and soil/bentonite admix with synthetic leachate having the composition expected during the life of the W-025 Landfill. The results of this program indicated that the polyester geotextile originally specified for the landfill might be susceptible to deterioration. On this basis, polypropylene geotextiles were substituted as a more chemically-resistant alternative. In addition, the percentage of bentonite in the admix was increased to provide sufficiently low permeability to the expected leachate.

Finally, revision 1 of this document includes a number of minor changes and clarifications resulting from comments by the Washington State Department of Ecology (Ecology) on the previous version.

1.2 Scope

The scope of work for this project included preparing a definitive design for the RMW Non-Drag-Off landfill. The facility must meet all Federal and State regulatory requirements. These include the requirements of the EPA, 40 CFR 260-265; DOE, Order 6430.1A; and Ecology, WAC 173-303.

A limited field investigation was conducted as part of this study to characterize the near surface deposits at the Project W-025 site. In addition, a subsurface exploration program was performed to investigate three borrow areas to supply admix for the soil liner material. The scope of work did not include any requirements for installation of groundwater monitoring wells.

The design services included preparing design drawings, technical specifications, a Construction Quality Assurance (CQA) plan, an acceptance test procedure for the pump system, a construction cost estimate, and a construction schedule. Reduced-size copies of the design drawings are included in Appendix B of this report. The latest revisions of the full size drawings, technical specifications (Golder Associates Inc. 1994a), CQA plan (Golder Associates Inc. 1993), and construction cost estimate and schedule (Golder Associates Inc. 1992) are separate documents that have been submitted previously. Details of landfill construction and as-built conditions are presented in the CQA Report (Golder Associates Inc. 1994b).

This report, as well as the analyses that were conducted as part of the design, was prepared in accordance with the Quality Assurance Program Plan (QAPP) for this project (Golder Associates Inc. 1990).

1.3 Authorization

This work was carried out by Golder Associates Inc. (GAI) under contract to DOE-RL (Contact No. DE-AC06-89RL11615). The Westinghouse Hanford Company (WHC) has acted as the Contracting Officer's Technical Representative (COTR).

2. DESIGN REQUIREMENTS

2.1 Functional Design Criteria

In 1988, WHC issued Functional Design Criteria (FDC) for the Project W-025 disposal facility (Carlson 1988). These FDC specified minimum design criteria for the following items:

- Capacity: The project must provide for disposal of a minimum of 66,000 cubic feet of waste which is assumed to be packaged in 55 gallon drums and miscellaneous burial boxes.

- Range of Operating Conditions: The landfill must allow year-round operations, including access by loaded forklifts during inclement weather conditions. The liner system must withstand anticipated loads from operating equipment, waste, and cover.

- Degree of Reliability: The facility should function safely and effectively throughout its operating and post-closure periods with minimal maintenance.

- Intended Life: The facility should be designed for a 2-year operating life and a 30-year post-closure period. The operating life was subsequently increased to 20 years (Golder Associates Inc. 1991a).

In addition, the FDC indicate that the design should meet all Federal (EPA) and State (Ecology) regulatory requirements.

2.2 Conceptual Design

A Conceptual Design Report (CDR) for the Project W-025 Disposal Facility was prepared in mid-1988 by Kaiser Engineers Hanford Company (KEH 1988). The CDR presented one concept which satisfies the requirements listed in the FDC. However, GAI was asked to present alternatives to the existing CDR if they provided an improvement to the design. During development of the work plan for definitive design, GAI evaluated alternatives to the existing conceptual design which increased the ultimate storage capacity, facilitated construction, and improved operational flexibility. Specific modifications included the following elements:

- The general layout would consist of a rectangular landfill with 3H:1V (horizontal:vertical) side slopes. The maximum depth of the facility would be approximately 35 feet, and the floor dimensions would be approximately 100 feet by 250 feet. The size of the facility was dictated primarily by constraints on the maximum side slope angle, the maximum access ramp slope, and the minimum dimensions on the floor for equipment to operate.

- Access to the facility would be by a 28 foot wide, 8 percent access ramp from the crest to the floor of the landfill. This would eliminate the wide paved access ramp that was shown in the CDR.

- The floor and side slopes of the landfill would be covered with a two-foot-thick operations layer of general fill, rather than the one-foot-thick layer of sand proposed in the CDR. This operations layer would protect the primary liner components from damage during filling and would minimize the potential for desiccation cracking of the secondary admix liner. (Note that the operations layer thickness was subsequently increased to 3 feet to provide additional protection [Golder Associates Inc. 1991a]).

- The leachate collection sump would be located at one end of the facility and would consist of a depressed section of the floor. The floor would slope at a minimum of 2 percent towards the sump.

- The leachate collection facilities would be located at the crest of the side slope directly above the sump. Leachate collection pipes would extend up the side slopes, thus eliminating any protrusions through the liner system within the landfill footprint or deep excavations as shown in the CDR.

These and other operational considerations were presented and discussed in the work plan meeting. After this meeting, WHC approved these modifications, and they were incorporated into the definitive design.

The liner design presented in the CDR was used for the definitive design and is considered satisfactory with respect to all regulatory criteria. The liner section consists of the following (from top to bottom):

- operations layer, to protect the underlying liner system components
- primary leachate collection system, to remove leachate from the landfill
- primary flexible membrane liner, to provide a moisture barrier
- secondary leak detection and collection system, to detect and remove leakage (if any) through the primary liner system
- secondary flexible membrane liner, to provide a moisture barrier
- secondary soil/bentonite admix liner, to provide a moisture barrier.

In the revision to the original design, a primary soil/bentonite admix liner has been added directly beneath the primary FML.

The specific materials that were chosen for the individual components are discussed in Section 6.

3. GEOLOGIC AND HYDROLOGIC SITE CONDITIONS

3.1 Regional Geologic Conditions

The Hanford site occupies approximately one-third of the land area within the Pasco Basin. The dominant regional geologic characteristics of the Pasco Basin have resulted from flood basalt volcanism and regional deformation.

The surface topography of the Hanford site has been modified by Pleistocene cataclysmic flooding and Holocene Eolian activity. The cataclysmic flood deposits were formed when ice dams in western Montana and northern Idaho breached, creating the channeled scablands of eastern Washington and the resulting channels and flood bar deposits on the Hanford site. The 200 Areas are located along one of these deposits known as the Cold Creek flood bar. The Holocene Eolian deposits have resulted from winds reworking the flood sediments, creating sand dunes and loess deposits within the Pasco Basin including the Project W-025 land disposal facility project area (Last et al. 1989).

3.2 Site Geology

The proposed location of the Project W-025 waste disposal facility is in the 200 West Area, within the existing Low Level Burial Ground 218-W-5, between Trenches 39 and 47 (see design drawings, Appendix B). Approximately 85 percent of the proposed Project W-025 landfill volume will be excavated in Recent Eolian sands. These sands extend from ground surface to a depth of about 17 feet. Underlying the Eolian sands is the coarse-grained sequence of the Hanford Formation which consists of an interstratified deposit of coarse sand, gravelly sand, and/or sandy gravel. This coarse-grained deposit is part of the Cold Creek flood bar. A fine-grained sequence consisting of mostly silty sand to sandy silt is present at the base of the coarse-grained sequence and is often referred to as the basal slack water sequence of the Hanford Formation. Based on previous borings (Last et al. 1989), the Hanford Formation is about 100 feet thick at the landfill site. Units underlying the Hanford Formation include the early Palouse soil, the Plio-Pleistocene unit, the middle Ringold unit, and the Elephant Mountain Member (basalt).

3.3 Regional Hydrologic Conditions

The region surrounding the project area is relatively arid, while the climate is typical of inland continental with hot summers and cold winters. The average annual precipitation recorded at the Hanford Town Site for the period 1912-1980 is 6.30 inches (Stone et al. 1983). The major hydrologic and climatological data of interest to this project is precipitation, solar radiation, and temperature. The regional hydrology is described in substantial detail in Last et al. 1989. Additional information is provided in Gephart et al. 1979.

The FDC require that project facilities be capable of withstanding the runoff resulting from a 25 year, 24 hour precipitation event without damage. Extreme value precipitation data for the project site are presented in Table 1. The 25 year, 24 hour precipitation depth is 1.56 inches.

Regionally, the groundwater table is relatively deep, lying in the Ringold formation above the basalt. At the project site, the water table is about 270 feet below the ground surface, as determined by borehole information presented in Last et al. 1989. Recorded ground water levels show little fluctuation. Since the ground water level is unlikely to rise to an elevation which could influence the disposal facilities, no further groundwater investigations or studies were conducted. Numerous groundwater monitoring wells surround the Project W-025 landfill site. Review of groundwater monitoring data from these wells was beyond the scope of this study.

4. SITE INVESTIGATION AND LABORATORY TEST PROGRAM

4.1 Field Exploration

A limited site investigation program was carried out to provide engineering data required for the design of the Project W-025 land disposal facility. The field study included an evaluation of borrow soils at the landfill site; in Area 1, about 1 mile to the west of the landfill site; and at the McGee Ranch, about 4 miles west of the site. Exploration at the landfill site included 12 test pits excavated to maximum depths of 17 feet and one borehole drilled to a depth of 53 feet. Samples were obtained for geotechnical laboratory testing and chemical analyses. The results of these investigations indicated that the engineering and chemical properties of the local soils at the landfill site were suitable for landfill construction and use as liner material. The site investigation and initial testing results are presented in Golder Associates Inc. 1989.

4.2 Laboratory Testing

As part of site investigation, samples were collected for more detailed laboratory testing during definitive design. This phase of testing had two major objectives. The first objective was to test various mixtures of on-site Eolian sand with bentonite clay to develop a suitable admix for the secondary soil liner. The second objective was to evaluate the shear strength parameters of various geosynthetic interfaces within the liner system for use in slope stability analyses. The results of this admix and interface strength testing are presented in the following sections.

4.2.1 Admix Testing Program

The admix used for the soil liner is required by regulations to have a permeability of 1×10^{-7} cm/sec or less. Two bentonite clay sources were used in the test trials: a calcium bentonite clay referred to as "Ephrata bentonite" and a commercially processed sodium bentonite clay. In total, nine flexible wall permeability tests (EPA Method 9100 Section 2.8) were conducted on samples with bentonite concentrations ranging from 6 to 20 percent by weight. A moisture-density-permeability relationship was then developed by comparing the admix permeability and dry density at various moisture contents. Laboratory tests were also performed to characterize the admix; these included standard Proctor, Atterberg Limits, sieve analyses, a consolidation test, and a consolidated undrained triaxial test. All laboratory testing was conducted in compliance with applicable ASTM or EPA procedures.

4.2.2 Geosynthetics Interface Testing

Geosynthetic interface tests were performed to provide shear strength parameters for side slope and access ramp stability analyses. Test parameters were selected to simulate post-construction conditions. Three combinations of geosynthetic interfaces were tested: geocomposite drainage material against operations layer soil, geocomposite drainage material against textured high density polyethylene (HDPE) FML, and textured HDPE FML against compacted admix. Westinghouse Environmental and Geotechnical Services, Inc. of Cincinnati, Ohio, provided laboratory services.

Samples of the textured HDPE geomembrane were supplied by Gundle Lining Systems Inc. of Houston, Texas. The geocomposite material was supplied by Fluid Systems, Inc. of Cincinnati, Ohio. If materials from other suppliers are used to construct the W-025 Landfill, additional interface shear strength testing will be required as part of the conformance test program to confirm strength parameters used in this design.

4.3 Laboratory Test Results

4.3.1 Admix Permeability Tests

Permeability test data are shown on Figure 1. These results indicate that an eight to ten percent (by weight) commercial sodium bentonite admix will meet the permeability requirement with a margin of safety of about one order of magnitude. This margin of safety is recommended since the mixing and compaction process used for small-scale laboratory test samples may produce more homogeneous material than field operations during construction. The Ephrata bentonite did not provide sufficiently low permeability and was not tested further.

The moisture-density-permeability relationship for the eight percent bentonite admix is shown on Figure 2. Other laboratory test results are summarized in Table 2. Detailed laboratory test data are presented in Appendix A.

Additional permeability testing with simulated leachate was performed on several admix compositions. This testing indicated that the percentage of bentonite should be increased from 8% to 12% by dry weight to provide sufficiently low permeability to the expected leachate (Golder Associates Inc. 1991c).

4.3.2 Geosynthetics Interface Strength Tests

The interface test results are presented in terms of the shear strength parameters, i.e., adhesion (psf) and friction angle (degrees). The laboratory plots are presented in Appendix A and the strength parameters are summarized in Table 3.

5. ENGINEERING ANALYSIS

In preparing the W-025 Landfill design, a number of design criteria as presented in the EPA draft guidance documents (EPA 1985) for double lined landfills and various other EPA resource documents have been considered. In addition, criteria specified in DOE Order 6430.1A have been evaluated. The specific criteria evaluated for the design include:

- stability
- settlement and uplift
- admix liner
- geosynthetic liner
- drainage layer
- collection system
- surface water
- leachate production

The details of the design analyses conducted for each of these criteria are discussed individually in the following sections.

5.1 Stability

5.1.1 Subgrade Side Slope Stability

The stability of the 3H:1V side slopes excavated into the subgrade was evaluated. In granular, cohesionless, and drained soils such as those underlying the Project W-025 facility, the stability of the slope is primarily related to the maximum slope angle. Therefore, an infinite slope stability analysis was used to determine both static and dynamic side slope stability. The dynamic analysis was conducted with a horizontal acceleration of 0.12 g as required in Standard Architectural-Civil Design Criteria, Design Loads for Facilities, SDC-4.1, Revision No. 11.

Generally, a factor of safety of 1.5 for static conditions and 1.1 for dynamic conditions are considered adequate for a facility of this type and were established as the minimum acceptable values for the design analyses. A friction angle of 36 degrees was assumed for the Eolian sands, as determined by laboratory testing of admix material (Appendix A). This value is slightly lower than the 38 degrees measured as part of the original site investigation (Golder Associates Inc. 1989) and is therefore slightly conservative. Coarse sand and gravel deposits typically have higher friction angles than finer grained soils, so the Eolian sand was analyzed as the critical case. The results of the stability analysis indicated a static factor of safety of 2.4 and a dynamic factor of safety of 1.7, both substantially higher than the minimum required values. Supporting calculations for the subgrade side slope stability analysis are presented in Appendix C.

5.1.2 Liner Side Slope Stability

Static and dynamic stability analyses were also performed for each geosynthetic interface on the landfill side slope, using the infinite slope method and residual strength parameters (friction angle and adhesion) as presented in Section 4.3. The horizontal acceleration for dynamic analysis and the acceptable factors of safety were the same as those used for the subgrade stability evaluation.

The operations layer/geotextile interface proved to be the critical interface with static and dynamic factors of safety of 1.8 and 1.3, respectively. Supporting calculations for the liner side slope stability analysis are presented in Appendix C.

5.1.3 Ramp Stability Analysis

Both static and dynamic stability analyses were performed on each liner interface assuming a maximum vehicle weight of 100 tons. In the dynamic analysis, a vehicle braking force equal to 30 percent of the static vehicle weight was incorporated into the stability equation. The minimum calculated factors of safety for static and dynamic analyses were 7.2 and 5.3, respectively. Both of these safety factors are well above the EPA-recommended values for ramp areas of 3 for static loading and 2 for dynamic loading. Supporting calculations for the ramp stability analysis are presented in Appendix C.

5.1.4 Waste Stability

Depending on the strengths of the liner component interfaces, waste placement can be a critical element in design of a waste disposal facility. If waste is placed too high on the sideslopes without sufficient material on the floor to support and restrain it, sliding and damage can occur. This will be prevented at the Project W-025 Landfill by placing the waste in horizontal lifts extending entirely across the landfill.

A conceptual fill plan is presented on the design drawings in Appendix B. Each lift will consist of one layer of drums and 1 to 2 feet of cover soil. Each lift will be completed across the entire facility prior to the start of the following lift. Filling in this manner will not result in any unbalanced loading which could cause failure along one of the liner interfaces.

In order to shield operating personnel from remote handled waste and to minimize air space loss that would result from construction of earth berms, the filling plan includes small concrete block barrier walls. The walls will surround the remote handled waste packages, and allow the area to be backfilled immediately after waste placement. A stability analysis of the concrete block barrier wall is presented in Section 5.1.5.

5.1.5 Concrete Barrier Block Wall Stability

The remote handled waste will be covered with approximately eight feet of soil which will act as a radiation shield. A detail showing the concrete block wall and backfill is presented on the design drawings. The backfill will exert a lateral force against the wall. Both overturning and lateral sliding stability analyses were performed to determine the stability of the wall under this load. The results indicated that the wall was stable against overturning up to its design height of 12.5 feet. Sliding stability was evaluated along the critical interface in the primary liner. To achieve acceptable stability, a 2.5-foot-thick by 10-foot-wide soil berm is required along the exterior base of the barrier wall. This berm is required only for barrier walls constructed immediately on the operations layer as part of the first lift of waste. The berm will not be required for walls constructed in subsequent lifts of waste.

Supporting calculations for the stability checks on the barrier block wall are presented in Appendix C.

5.1.6 Bearing Capacity of Subgrade Soil

The bearing capacity of the subgrade soil is required for the design of the leachate collection tank and control building foundations. It is assumed that all structural fill will be moisture conditioned and compacted to at least 95 percent of the maximum modified Proctor dry density (ASTM D1557) and that the native soils will be compacted prior to fill placement. The maximum allowable bearing capacities are summarized as follows:

- Compacted structural fill of Eolian sand: 4000 lb/ft^2
- Dense native sands and gravels: 3000 lb/ft^2
- Proof-rolled silty sands (Eolian sand): 2500 lb/ft^2

Additional details regarding settlement estimates and general earthwork recommendations are presented in the field investigation report (Golder Associates Inc. 1989).

5.2 Settlement and Uplift Analyses

5.2.1 Elastic Settlement of Foundation Soils

The subgrade settlement produced by waste loading will be essentially elastic in behavior due to the coarse grained, non-cohesive, and drained nature of the soil. The subgrade will rebound during the excavation phase of construction and will settle as the facility is filled. A Schmertmann elastic settlement analysis was performed to determine the magnitude of the total and differential settlement. This is considered to be a conservative method of

estimating settlement for this type of loading. The maximum subgrade settlement after the landfill is filled will be about 2 inches, with a maximum differential settlement between the toe and crest of the side slopes of approximately 0.5 inch. Supporting calculations for the elastic settlement of the foundation soil are presented in Appendix C.

5.2.2 Consolidation Settlement of Compacted Admix

The secondary admix liner will be placed and compacted to nearly 100 percent saturation. When the admix liner is loaded by overlying liner components and waste, the pore water pressure will increase. The pressure gradient which exists between the admix and the underlying porous soil will drive pore water out of the admix into the soil. As the pore water leaves, the admix will consolidate. The consolidation settlement analysis was based on a laboratory consolidation test of the compacted admix. The maximum consolidation settlement after the landfill is filled will be about 1.5 inch, with a maximum differential settlement between the toe and crest of the side slopes of about 0.5 inch. Supporting calculations for the consolidation settlement of the secondary admix are presented in Appendix C.

5.2.3 Settlement Summary

The total settlement will be a combination of the subgrade elastic and the admix consolidation settlements. The maximum total settlement is estimated to be about 4 inches, and the differential settlement between the toe and crest of the side slopes is estimated to be approximately 1 inch. Supporting calculations for the settlement summary are presented in Appendix C. Total and differential settlements of this magnitude are not expected to cause a problem with the function of the liner system or operation of the landfill. Since the design of the closure cover for the landfill was not within the scope of work for this project, settlement estimates for the waste placed in the landfill were not performed.

5.2.4 Subsidence and Sinkhole Potential

Subsidence and sinkhole development is not expected to occur, since the soils underlying the landfill are coarse-grained sands and gravels and the groundwater level is very deep. In addition, the geotechnical boring and test pits demonstrated that the soils are well graded and relatively dense. Supporting discussion for the low subsidence and sinkhole potential is presented in Appendix C.

5.2.5 Uplift Potential

The potential for uplift of the composite liner system is very low. The groundwater level is very deep, and perched groundwater is unlikely to develop due to the absence of aquitards in the coarse-grained sediments. Gas pressures are unlikely to develop due to the lack of gas generating sediments and the dissipating nature of the coarse-grained sediments. Additional supporting discussion for the low uplift potential is presented in Appendix C.

5.3 Admix Liner

5.3.1 Bearing Capacity of Admix Liner

The most severe loading condition for the admix liner will result from the concrete barrier block wall used to isolate the non-contact handled waste. An analysis of the admix liner bearing capacity under these loading conditions indicated a factor of safety of about 11. This factor of safety is substantially higher than the value of 3 typically considered acceptable for this mode of failure. Supporting calculations for the bearing capacity of the liner soils are presented in Appendix C.

5.3.2 Admix Liner Specifications

Based on the results of the testing described in Section 4, a bentonite content of 8 to 10 percent by weight was originally proposed for the admix liner. However, based on the results of permeability testing using simulated leachate (Golder Associates Inc. 1991c), the bentonite content was increased to 11 to 14 percent (nominal 12 percent) by weight. This mix is similar to that successfully used for the recently-constructed Project W-105 Interim Retention Basins. Based on the experience of Project W-105, the admix will be prepared using a pugmill to provide a uniform, homogeneous liner material. The admix will be allowed to set at least 24 hours prior to placement to ensure adequate hydration.

The objective of the admix placement specifications is to produce a material that will exhibit the required permeability of 1×10^{-7} cm/sec or less when placed at the specified compaction and moisture content. The admix liner placement specifications are based on the admix moisture-density relationship established in the laboratory test program described in Section 4. The admix should be placed at a moisture content of 1 to 5 percent over the optimum moisture content and compacted to at least 90 percent of maximum dry density as determined by the standard Proctor test (ASTM D698). The material should be placed in maximum 6-inch compacted lifts to ensure adequate compaction throughout the lift.

Additional laboratory and field testing will be conducted during production of the admix, construction of the test fill, and placement of admix in the landfill to confirm that the admix

meets the required minimum permeability. The proposed specifications can be changed if so indicated by this testing.

5.3.3 Desiccation Cracks

Previous research and experience has shown that desiccation cracks can occur under geomembrane liners when either (1) the liner is not in tight contact with the admix and moisture leaves the admix to collect in the adjacent air space, or (2) the liner is subjected to wide temperature fluctuations. The Project W-025 landfill design addresses both of these possibilities by placing three feet of operations layer over the entire landfill, including the slopes, during construction. The operations layer will act as a weight to keep the geomembrane in contact with the admix. In addition, the operations layer will perform as an insulation layer. Thermal loading from the radioactive constituents of the waste is expected to be negligible, less than 1°C (Golder Associates Inc. 1991c).

In order to allow for some desiccation cracking of the secondary admix liner during construction and still maintain a minimum 3.0 foot thickness, the design requires that the liner be placed at thickness of 3.1 feet. If desiccation cracking during construction exceeds 0.1 feet in depth or 0.25 inches in width, it will be excavated and repaired. However, desiccation cracking prior to placement of the FML will be minimized by installing a temporary plastic cover or keeping the admix surface wet.

5.4 Geomembrane Liner

5.4.1 Liner Tension Caused By Thermal Contraction

An HDPE liner will expand and contract as liner temperatures increase and decrease. Thermally induced stresses can develop in the liner if deployment and anchoring occur prior to a significant decrease in the liner temperature. Under worst case conditions, the strain in the liner will be approximately one percent, creating a tensile stress of approximately 1000 pounds per square inch. These values are well below the yield strain and stress of HDPE liner, and are not expected to cause problems. Supporting calculations for the liner tension caused by thermal contraction are presented in Appendix C.

5.4.2 Anchor Trench Pullout Resistance

During construction, the liner could experience pullout forces caused by thermal contraction or wind uplift. However, the slight displacements resulting from thermal contraction are not expected to be significant, and wind uplift will be controlled by use of sandbags or a similar approach. After construction, pullout forces on the geomembrane are expected to be non-existent, because the liner interface friction angles will be greater than the slope angle,

as required by the specifications. Supporting calculations for the theoretical maximum anchor trench pullout resistance are presented in Appendix C.

5.4.3 Puncture Resistance

The geomembrane will be protected against damage from overlying granular soil materials by other geosynthetic components of the liner system, as shown on the design drawings. A geotextile and geonet layer will be installed between the drainage gravel and the primary geomembrane on the floor of the landfill. On the side slopes, a geocomposite layer will be placed immediately above the geomembrane liner to perform both as a drainage layer and as a cushion from the overlying operations layer material. A puncture resistance analysis of the geomembrane liner, assuming no geotextile cushioning, was performed. The results indicate that even with this extremely conservative assumption, the geomembranes have adequate puncture resistance. This analysis is presented in Appendix C.

5.4.4 Chemical and Radiation Resistance

The HDPE liner material was tested for chemical resistance in accordance with EPA Method 9090 (Golder Associates Inc. 1991b). Some of the samples were subjected to the full radiation dose expected over the life of the facility. In both the irradiated and non-irradiated cases, the HDPE showed no statistically significant change in properties and consequently is considered satisfactory for use in the landfill. Another concern was thermal loading from the radioactive constituents of the waste which could cause the plastic FML to deteriorate. However, the temperature increase from the waste is expected to be negligible, less than 1°C (Golder Associates Inc. 1991c), and is therefore not a problem.

5.5 Drainage Layer

5.5.1 Geotextile Analyses

Geotextiles are required to perform several functions in the Project W-025 Landfill, including separation, filtration, and cushioning. Analyses were performed to evaluate the following parameters:

- Filtering capacity
- Permeability
- Clogging potential
- Burst resistance
- Tensile strength
- Puncture/tear resistance
- Cushioning capacity

Supporting calculations for the geotextile analyses are presented in Appendix C.

5.5.2 Geonet Selection

A geonet layer will be incorporated in the geocomposite drainage layer on the side slopes of the landfill. A geonet layer, not incorporated into a geocomposite, will provide a redundant drainage system, in combination with the drainage gravel, on the floor of the landfill. The primary selection criteria for the geonet was the required transmissivity of 7.2 gals/min/ft. Supporting calculations for selection of the geonet are presented in Appendix C.

5.5.3 Drainage Gravel Selection Analysis

Drainage gravel permeabilities were estimated using Hazen's approximation for clean drainage material. The analysis was done for both sump and drainage gravels. Estimated permeabilities of both gravels exceeded the minimum required permeability of 1×10^{-2} cm/sec. Supporting calculations are presented in Appendix C.

5.6 Leachate Collection System

5.6.1 Downdrag Analysis for the Vertical Riser Pipe

The downdrag potential caused by waste settlement around the vertical riser pipe (see design drawings) was analyzed to determine the structural support requirements for the vertical riser base. A smooth HDPE geomembrane will be wrapped around the vertical riser to reduce the skin friction, and was considered in the analysis. The results indicate that the downdrag force plus the self-weight of the vertical riser pipe will not exceed the stress caused by the waste fill, i.e., the liner under the vertical riser base will not experience additional stress due to the downdrag force. Supporting calculations for the downdrag analysis are presented in Appendix C.

5.6.2 Earth Loading - Slope Riser Pipes

The side slope riser pipes will be HDPE to provide a chemically-resistant secondary containment system for the sump pump discharge lines, as well as access to the sumps. The maximum anticipated earth load on the primary side slope riser pipe was calculated to determine the pipe wall thickness required to limit pipe deflection to less than 5 percent of the pipe diameter, as recommended by the manufacturer. The primary riser pipe was considered to be represent worst-case conditions, and a secondary riser pipe of the same dimensions would therefore perform satisfactorily. Based on earth loading considerations, the primary and secondary slope riser pipes will have a diameter of 8 inches and a Standard

Dimension Ratio (SDR) of 11 (0.78-inch wall thickness). Supporting calculations for the slope riser pipe earth loading are presented in Appendix C.

5.6.3 Earth Loading - Primary Drainage Pipe

The primary drainage pipes will be located at the toes of the landfill side slopes, along the centerline of the landfill, and along the upslope side of the access ramp. These pipes will be comprised of HDPE because of its high chemical resistance. The maximum anticipated earth load on the primary drainage pipe was calculated to determine the wall thickness required to limit pipe deflection to less than 5 percent of the pipe diameter, as recommended by the manufacturer. Based on these analyses, the primary drainage pipe will have a diameter of 4 inches and a SDR of 11 (0.41-inch wall thickness). Supporting calculations for the primary drainage pipe earth loading are presented in Appendix C.

5.6.4 Earth Loading - Leachate Collection Well

The 30-inch-diameter HDPE leachate collection well will be installed directly under the vertical riser base. The collection well will be surrounded by primary sump gravel. An earth loading analysis was performed to determine the required wall thickness to prevent collapse of the well under horizontal earth pressures. The results indicate that a 30-inch-diameter HDPE pipe with an SDR of 13.5 (2.2-inch wall thickness) will perform adequately. Supporting calculations for the leachate collection well earth loading are presented in Appendix C.

5.7 Surface Water

The Project W-025 RMW land disposal facility must be protected from uncontrolled surface water flows from surrounding areas (run-on) which might damage the facilities or add to the quantity of leachate to be treated. Following closure, the run-off from the cover must also be controlled; however, closure cover design was not included in the scope of this work.

The FDC and the EPA regulations require that run-on control facilities should be sized to handle the 25 yr, 24 hr precipitation event. The Hanford Site precipitation depth and intensity data are presented in Table 1. As shown in the table, the 25 yr, 24 hr event has a depth of 1.56 inches.

For the previous version of this design report, it was assumed that the majority of the run-on would originate in the area west of the landfill, which was characterized by higher topography than the immediate site. The following paragraphs of this section are based on this assumption. However, during 1991, the area immediately west of the landfill site was used as a borrow source for soil liner for Project W-105, producing a large depression which

is expected to intercept essentially all drainage from areas still farther west. As a consequence, the following analysis becomes extremely conservative. However, the previous ditch size has been retained for practicality of construction.

The greatest part of the drainage basin (Figure 3) which is potentially tributary to the Project W-025 land disposal facility lies to the west of the project site (west of 23rd Street). At the present time, overland runoff from this area, which constitutes about 11 acres, is intercepted by the drainage ditch which forms the western edge of 23rd Street. The runoff is collected and discharged to the south. Local runoff from areas immediately surrounding the landfill tends to flow away from the facility. Slopes in the area are generally very flat. Because these grades are nearly flat, future construction work (not part of this project) might disturb the existing drainage patterns, causing a concentration of flow toward the landfill. Therefore, for purposes of analysis and simplicity of construction, a tributary area of 17 acres was used to estimate a peak flow; this area is the entire project site and includes the 11 acres west of 23rd Street. The resulting peak flow was then used to design all drainage facilities. This is believed to be a very conservative assumption.

The runoff potential for the area surrounding the project site was evaluated using three different methods of analysis including (1) the rational method, which is applicable for areas less than 200 acres, (2) HEC-1, the Corps of Engineers unit hydrograph and flood simulation program, and (3) estimating techniques based on regression equations developed from regional stream flow measurements (Cummans et al. 1975). In general, the preferred method of analysis is one which uses actual streamflow values in the basin of interest, or a similar basin located nearby for which adjustments can be made. The regression techniques are an approximation to this approach; however, the basins of interest for the Project W-025 project are outside the parametric range permitted by the equations and the original data. The unit hydrograph procedures available in HEC-1 are suitable, but the results are very sensitive to assumptions made regarding the soil conditions. Similarly, the rational method, which is most commonly used for small basins such as are involved at the Project W-025 facility, produces results which are very sensitive to the assumed soil conditions. Without measured basin flows to allow calibration of the method, there is significant uncertainty in the results. Because none of the methods are clearly preferred over the others, all the methods were used to estimate flows and the results were examined comparatively.

The results of the three peak flow estimating techniques are presented in Table 4. The peak flow estimate using HEC-1 with a curve number of 65, which is a reasonable estimate of soil conditions, is very small; very little runoff will occur. When the curve number is increased to 85, simulating a nearly impervious, saturated, or frozen soil, the runoff increases to a peak of 9 cfs. Clearly the quantity of runoff is highly dependent on the soil runoff potential for this method. The rational formula provides an estimate of 5 cfs based on an assumed C of 0.2, suitable for lawns in poor condition. The regression equation approach supports an estimate less than 5 cfs; however, this requires extrapolating from the minimum basin size of 1.8 mi^2. Based on these estimating techniques, a design flow of 5 cfs was selected for sizing of storm runoff conveyance facilities. The resulting drainage ditches are triangular in cross

section with 3H:1V side slopes and a depth of 2 feet. Details of the surface water runoff analyses are presented in Appendix C.

5.8 Leachate Production

The quantity of leachate produced by the landfill is of interest because the leachate removal pumps, piping, and storage tank must be sized to handle the expected production rates.

5.8.1 Average Annual Leachate Production

The quantity of leachate which can be expected to arrive in the primary sump will vary over the life of the facility, depending on the amount of waste, precipitation, solar radiation, and temperature. The maximum rate of leachate generation will probably occur during the early years of operation when little waste and intermediate soil cover is present in the landfill.

The quantity of leachate expected during the early years of operation was estimated using the EPA-recommended HELP (Hydrologic Evaluation of Landfill Performance) model. For the purposes of this model only, the facility was assumed to remain open for the 10 year modelling period (1979-1988), with no waste on the operations layer. The resulting average annual leachate production was estimated to be about 155,000 gallons. Nearly all of the leachate is expected to be produced during the winter months. The HELP model is described in more detail in Section 5.8.2 below.

5.8.2 Peak Daily Leachate Production

Peak daily leachate production was estimated using the HELP model and other hydrologic methods of analysis. Although the simulations were done for periods of up to ten years, using data beginning in 1979 and continuing through 1988, the data of interest in estimating the peak daily production is limited to about 60 days.

The wettest interval during the period of study was the month of December 1983, although the wettest single day was in October 1982. The 25-year precipitation event was used as the design event and was inserted into the historical record to simulate its occurrence. Because the existing soil moisture conditions are very significant in determining the daily leachate production rate, the 25-year event was inserted in late December 1983, following the wet period which had produced relatively moist soil conditions. This approach is considered to be conservative. The pattern of precipitation during the period and the resulting daily leachate production are presented in Figure 4. The maximum daily production rate is about 110,000 gallons on the day following the 25-year precipitation event. This will require storage of the leachate in the sump for a few days as the sump is emptied; this approach is consistent with EPA's minimum technology guidance documents (EPA 1985).

6. FACILITY DESIGN AND CONSTRUCTION

6.1 Facility Layout

6.1.1 Location

The Project W-025 Landfill will be located in the northwest corner of the 200 West Area in the 218-W-5 Burial Ground between future burial trenches 39 and 47. At the ground surface, the landfill measures approximately 450 feet by 300 feet with the long axis trending east-west. The excavated depth to subgrade will range from approximately 34 feet at the west end of the floor to about 37 feet at the east end. The excavation stockpile will be located north of the landfill and covered with a layer of coarse soil to minimize wind erosion. An admix preparation area will be located south of the landfill in proximity to the access ramp during construction.

6.1.2 Access Roads and Ramps

The entrance road to the facility will tee off of Dayton Avenue and lead to a truck staging and unloading area. The truck staging area will serve both as a parking and turn-around area. The truck unloading area will be approximately 75 by 85 feet in dimension. It will be the location where waste packages are removed from trucks by forklifts or other equipment and transported into the landfill. To provide containment during these activities, the unloading area will be graded to drain into the landfill and will incorporate a geomembrane liner, a drainage pipe, and an asphalt concrete surface. The truck unloading area is designed to facilitate cleanup operations in the event of accidental spills.

Access into the landfill will be from the west end of the truck unloading area and will be provided by a 250-foot-long, 28-foot-wide, eight-percent-slope ramp. The access ramp slope was chosen as the maximum acceptable for year-round access into and out of the landfill, especially in the winter months.

A 30-foot-wide gravel road will be constructed around the perimeter of the landfill for general site access and facility maintenance.

6.1.3 Survey Grids

Survey points, using the Hanford Plant Coordinate System, were established as part of the design to assist both the Construction Contractor and the CQA Engineer during construction of the landfill. These survey points were established on a 25 foot grid over the entire landfill and include additional points to define grade breaks, such as at the crest and toe of slopes. Subgrade control points were also defined around the perimeter, down the

access ramp, and down the centerline of the landfill. The control points and the connecting control lines serve as relational reference points for landfill components such as the leachate collection system piping.

6.2 Landfill Geometry

6.2.1 Volume

The total finished airspace volume from the base of the landfill to the ground surface is estimated to be approximately 69,000 cubic yards. However, due to the required 16-foot-thick final soil cover over the waste, the estimated airspace of the landfill is approximately 28,000 cubic yards. The amount of waste that can actually be stored in this airspace depends on the size and shape of the waste containers and the amount of soil backfill placed around the containers. For example, if the waste is contained in 55-gallon drums, and 2 feet of intermediate soil cover is placed between each lift of drums, then approximately 37,000 drums can be disposed of in the landfill. The capacity can be increased by using alternative backfill materials and allowing the final cover to project above grade. These alternatives are described in a separate study (Golder Associates Inc. 1991d). The design for the final closure cover will be prepared at a later time.

6.2.2 Depth and Length

The minimum depth of the landfill was determined by the need to accommodate three layers of waste drums and operational cover soil, as well as the final cover thickness of 16 feet. It was assumed that the top of the final cover would be approximately even with the existing ground surface. The resulting finished depth is about 30 feet. The areal dimensions of the landfill were determined by the need to provide a minimum amount of space in each direction for equipment operating on the floor of the landfill. Dimensions were also determined by the overall constraints of the site and accepted engineering practice for laying out landfills in an efficient and constructable way. On this basis, the landfill floor is approximately 250 feet long and 100 feet wide.

6.3 Geosynthetic Liner Materials

6.3.1 Geomembranes

HDPE has been selected as the liner material because it has the highest chemical resistance of commercially-available liner materials, has been widely used at similar facilities, and has a high level of regulatory acceptance. Current EPA guidance documents require that a minimum of a 30 mil liner be installed in waste disposal facilities. However, current practice

in the industry is to use a minimum thickness of 60 mil. This thicker material is easier to handle and seam and has increased resistance to damage during installation.

Three types of HDPE geomembrane liners will be incorporated into the Project W-025 Landfill, as shown on the design drawings. The primary and secondary side slope liners will utilize textured (roughened) 60 mil HDPE geomembrane (Type I geomembrane), which, in conjunction with the geocomposite drainage material, will provide interface shear strengths that are sufficiently high to support the overlying operations layer. The primary and secondary floor liners, where high shear strength is not required, will incorporate smooth 60 mil HDPE geomembranes (Type II geomembrane). Under the truck unloading area, a smooth 80 mil geomembrane (Type III geomembrane) will be used to provide a higher level of mechanical resistance.

Details of required geomembrane properties are contained in the technical specifications (Golder Associates Inc. 1994a).

6.3.2 Geonet

Geonet will be used in both the primary and secondary leachate collection and recovery systems. The geonet will be comprised of HDPE to provide high chemical resistance. Details of required geonet properties are contained in the technical specifications (Golder Associates Inc. 1994a).

6.3.3 Geotextile

Two weights of geotextiles will be used in the Project W-025 Landfill. This first (Type A) is 7 oz/yd^2 and was selected to act primarily as a filter between the operations layer and the primary drainage gravel on the landfill floor. Type A geotextile will also be incorporated into the geocomposite drainage layers on the landfill side slopes. The second geotextile (Type B) is heavier at 16 oz/yd^2 and was chosen to perform primarily as a cushion for geomembranes on the landfill floor and truck unloading areas. Both types of geotextiles will be non-woven needlepunched polypropylene. This material was selected because of its anticipated high resistance to the expected leachate (Golder Associates Inc. 1991b).

Details of required geotextile properties are contained in the technical specifications (Golder Associates Inc. 1994a).

6.3.4 Geocomposite Drainage Layer

The geocomposite drainage layer will consist of a layer of Type A geotextile thermally bonded to the top and bottom of the geonet. The thermal bond prevents sliding across this interface, which otherwise would have a relatively low shear strength. The resulting

geocomposite will be used on the landfill side slopes, where the geotextile component will interlock with the texturing on the geomembrane and prevent sliding.

6.4 Leachate Collection System

6.4.1 Leachate Removal Pumps

Regulatory requirements state that the leachate collection systems shall be capable of collecting and removing leachate so that a hydraulic head of no more than 12 inches exists on the liner. Pumps were selected as the most effective method to remove the leachate and transfer it to the temporary storage tank. This section discusses the selection and installation of the pumps.

6.4.1.1 Primary System

For most of the life of the waste disposal facility, the flow of leachate is expected to be relatively low, a few tens of gallons per day on average. This leachate flow must be removed and the level maintained as low as possible. On the other hand, when precipitation and soil moisture conditions are both unfavorable, the peak daily leachate production could theoretically reach 110,000 gallons. This represents the 25 year, 24 hour precipitation event, as discussed in Section 5.8.2. Temporary storage of leachate in the sump is permitted for a few days during such extreme events. Since these two design conditions are so different (average conditions vs. 25 year event), and because redundancy is a desirable design feature, two pumps were selected for use in evacuating the primary sump. The first will be a low capacity pump for everyday use, located near the bottom of the sump. The second will be a high capacity pump which can evacuate leachate during extreme events while serving as a backup to the low capacity pump.

6.4.1.1.1 Low Capacity Submersible Pump

A submersible pump was selected as the low capacity pump because:

1. It can be obtained in low capacity sizes.

2. It is available with all-stainless steel construction which is required in order to resist the potentially corrosive effects of the leachate.

3. It is small and therefore can be inserted into a curved riser pipe, which will allow nearly all of the leachate in the sump to be evacuated.

4. Even with stainless steel construction, the pump has a relatively low cost and can be replaced as necessary.

5. Maintenance can be done from the ground surface by pulling the pump out through the side slope riser pipe. The pump is small and light and can be handled without power equipment.

A Grundfos Model 7S03-8 was selected, although "or equal" is permitted in the specifications. This pump has all stainless steel wetted parts and a nominal 10 gpm capacity against a 50 foot TDH. The discharge piping will be HDPE, with type 316 stainless steel fittings. The motor will be about ½ horsepower. The foot valve on this pump will be removed so that the leachate in the discharge pipe can drain back into the sump and will not be subject to freezing.

6.4.1.1.2 High Capacity Pump

The high capacity pump is expected to operate only following significant precipitation events. A pump capacity of 150 gpm was selected because the 10,000 gallon leachate temporary storage tank can be filled in about an hour. This is comparable to the expected evacuation rate of the storage tank by highway tanker trucks. This capacity will also allow the volume of leachate from the 25-year precipitation event to be removed from the sump within 24 hours.

A Gorman-Rupp Model 92B9B self-priming horizontal pump with a 3-horsepower motor was selected because it has the required capacity and can be installed above the expected maximum level of leachate in the vertical riser. This location eliminates many of the problems associated with long term corrosion. Furthermore, by using a lifting cable, it can be removed without requiring personnel to enter the vertical riser pipe.

The discharge piping will be HDPE. It will extend vertically from the pump to the edge of the vertical riser pipe where it will turn and follow the outside of the vertical riser down into the operations layer. From the base of the vertical riser, it will follow the inclined riser pipes up the slope to the storage tank. The low points of the discharge line inside and outside of the vertical riser will include drain down valves to remove leachate during freezing conditions, if necessary. The discharge pipe will be lengthened as the vertical riser is extended upward to accommodate the increasing thickness of waste in the landfill.

6.4.1.2 Secondary System Pump

The secondary leachate collection system is very similar to the primary system except that the expected leachate flow will be small. Since the primary system low capacity pump is the smallest of that type available, an identical pump was selected for use in the secondary sump. This will also provide duplication; operators and maintenance personnel will need to be familiar with only one submersible pump type. The foot valve on the secondary pump will also be removed to prevent freezing of the leachate in the side slope riser pipe.

6.4.1.3 Transfer Pump

The leachate temporary storage tank will be evacuated using a permanently installed transfer pump connected to the discharge from the base of the tank. This pump will be mounted on a concrete pad adjacent to the tank and within the containment sump. The transfer pump will discharge to a hose supplied by the highway tanker truck. The capacity of the transfer pump should be sufficient to fill the tanker in short period of time. A capacity of 500 gpm was selected, allowing filling of a 7500 gallon tanker in 15 minutes. The pump will be a standard horizontal centrifugal pump with epoxy coated wetted parts and a 10 horsepower, TEFC (totally-enclosed, fan cooled) motor. The cost of using 316 stainless steel in this pump is not justified because of the ease and low cost of replacement. The pump has been located so that the discharge is directed back into the tank containment sump in the event that the pump is inadvertently started without a tanker connected to the discharge.

6.4.2 Leachate Level Sensors

The leachate level in both the primary and secondary collection systems will be monitored to demonstrate that the hydrostatic head on the flexible membrane liners is below the regulatory limit of one foot. Level sensing is also required in order to allow proper control (on/off) of the pumps. The system selected for use in both the primary and secondary submersible sumps consists of an electrical pressure transducer installed in a small diameter pipe adjacent to the submersible pumps. The transducer will transmit a 4-20 milliamp signal, proportional to water level, to a current-sensitive relay in the control panel. The current-sensitive relays will be used to switch the pumps on and off. The transducer output signal will also be displayed as "inches of water" on the front of the panel.

This system was selected for several reasons. The transducers can be inserted adjacent to the secondary pump without penetrating the primary membrane. Both transducers can be removed for calibration and maintenance on the surface, and can be re-inserted to their original location with a minimum of effort. The transducers, fabricated from type 316 stainless steel, are not expected to experience current-induced corrosion, as would a conductive type level control.

The transducers will be attached to 1-inch-diameter PVC pipe to facilitate installation and removal. PVC pipe was chosen because it is generally more available than HDPE in this diameter. Since these pipes are removable and replaceable, PVC compatibility with the leachate is not considered a design constraint.

Like most electrical sensors, the level transducers are expected to show some long term drift in their readings and will require recalibration at 12 month intervals or more frequently if recommended by the manufacturer.

The high capacity primary system pump will be controlled by float switches located inside the vertical riser pipe. These provide a redundant backup to the transducer level sensing system.

At the time of the original W-025 design, the EPA proposed that the secondary sump be sensitive to a leak rate of about 5 gallons per acre per day. (This EPA proposal was ultimately not implemented, but the W-025 design reflects the earlier guidance.) Since the area of the lined facility is about 3 acres, the transducer must be capable of sensing the addition of 15 gallons of leachate to the sump. The specified transducers are accurate to a 0.5 percent of their full-scale range. In this case the range is 6 psi, so the sensitivity is 0.07 feet. Using the volume-elevation curves presented on Figure 5, the sensitivity can be shown to be about 11 gallons (see calculation in Appendix C).

6.4.3 Vertical Riser Pipe

The vertical riser pipe will consist of prefabricated 48-inch-diameter reinforced concrete manhole sections stacked vertically. The riser pipe will be supported by a 6-foot-square, 6-inch-thick reinforced concrete foundation slab. The foundations slab will be prefabricated and set in place. The riser pipe and foundation slab will be coated as described in Section 6.4.4.3 below for protection from attack by leachate.

The foundation slab will have a circular penetration for a 30-inch-diameter HDPE leachate collection well. This well will extend about 4.3 feet below the surface of the vertical riser base. The primary sump will drain into this well. In addition, leachate collection system piping will drain directly into this well. The leachate collection well was designed to facilitate construction in the sump area and to permit the load bearing vertical riser base to be placed well above the primary geomembrane liner.

A 3-inch HDPE discharge line from the high capacity sump pump will be mounted on the outside of the vertical riser pipe sections. It is recommended that a concrete block barrier be placed in front of this discharge line to protect it from equipment damage. The barrier block(s) will need to be moved after each lift is placed.

6.4.4 Leachate Temporary Storage Tank

6.4.4.1 Tank Volume

The selected tank volume of 10,000 gallons is based on practical trucking considerations and the need to remove leachate within a reasonable time following a major storm event. A standard highway tanker truck has a capacity of about 7,500 gallons (weight controlled). Assuming a peak daily leachate production of 110,000 gallons (see Section 5.8), one highway tanker will make about 15 trips to remove the leachate. If each round trip requires one hour, the leachate can be removed within 24 hours, including some contingency time.

Therefore, the capacity of the leachate storage tank can be similar to that of the tanker truck. The slightly larger volume of 10,000 gallons facilitates efficient operations.

6.4.4.2 Tank Design

A standard mild-steel pressure tank 8-feet in diameter and 25-feet-long with hemispherical heads was selected for use as the leachate temporary storage tank. This tank can be shop fabricated and delivered to the job site on a flat bed truck without special travel restrictions. Consideration was given to a fiberglass reinforced plastic tank; however, a tank of this type was not selected because of cost and the difficulty in making future changes.

Support saddles will elevate the tank above the supporting slab. A discharge pipe will be installed on the bottom of the tank at one end and will lead to the transfer pump. The influent fitting will be located on the top of the tank at the other end. A penetration will be provided for level controls (high level switch) on the top of the tank. The tank will also be equipped with a float actuated mechanical level indicator on one end. Two man-holes will be provided to allow access for inspections and painting. The tank and associated piping will be protected from freezing by insulation and heat tape.

To provide for ease of quality control, the tank is specified as a 25 psi ASME code stamped pressure tank. The ASME code tank specification was recommended by GAI at the 30 percent design review. The tank design was finalized based on concurrence from WHC. This tank design is intended to satisfy the requirements of WAC 173-303-640, and the tank will be inspected upon installation as described in the Construction Quality Assurance Plan (Golder Associates Inc. 1993).

6.4.4.3 Tank Coating

The tank will contain leachate of undetermined quality and must resist attack from that leachate. The coating was selected by comparing a list of chemicals which may be placed in the landfill (Carlson 1988) with a list of chemicals with which the coating is compatible. The list of compounds which might be placed in the landfill is long and includes many chemicals which could attack a coating. Those compounds which are expected to be present in large quantities were most carefully evaluated.

The selected coating consists of air-dried amine cured epoxy. Details are presented in the technical specifications (Golder Associates Inc. 1994a). This coating is expected to withstand the inorganic salts that might be present in the leachate in relatively high concentrations. There are some compounds, such as sodium hypochlorite, other strong oxidizers, and paint remover, which might be placed in the landfill and which could attack this type of coating. However, waste placed in the landfill will comply with Ecology's land disposal restrictions, so that these potentially damaging materials will either be stabilized (solidified) or present in very low concentrations that are not expected to present a problem for the coating.

6.4.4.4 Tank Leak Containment System

The tank leak containment system consists of a concrete sump sized to contain the tank volume plus the depth of average annual rainfall. The interior of the concrete sump will be lined with Type III (80-mil) HDPE geomembrane as described in Section 6.3 to satisfy the requirements of WAC 173-303-640(4)(e)(i)(C). Waterstops will be incorporated into all construction joints.

6.4.5 Pump Controls and Instrumentation

6.4.5.1 Control Building

The pump control panel will be a weatherproof (NEMA 4X) panel, suitable for outdoor installation. However, because of frequent high winds in the area, the need for keeping records on site, and the possibility of future changes, a control building to house the panel and associated equipment was included in the design. The control building will be a pre-manufactured metal enclosure on a concrete slab. The entrance will be equipped with a lockable metal door and weather seals to prevent ingress of dust. No heating will be required. Interior lighting will be provided. The building will house:

1. The electrical service load center.
2. The control panel.
3. Electrical outlets for auxiliary 115 V service.
4. Motor starters.
5. Submersible pump control boxes.
6. 480 V / 230 3 phase transformer panel.

6.4.5.2 Control, Operation, and Monitoring

Under normal operating conditions, pump control will be automatic. The low capacity primary pump will be switched on when a high leachate level is sensed by the transducer. It will be switched off when a low level is sensed. The secondary pump control is similar. These on/off levels will be set at the control panel through adjustment of the current sensitive relay. The high capacity primary pump is controlled by high and low level mercury float switches located in the vertical riser pipe. These are adjusted by physically moving their positions. The control system for the high capacity pump is independent of that for the low capacity pump. All sump pumps will be equipped with hand-off-auto switches; in the auto position, all pumps will be shut off by either a high leachate storage

tank level or a low sump level. The transfer pump will have an ON/OFF switch and motor starter located at the pump itself, without automatic shutoff (except thermal).

Each of the transducers will have an LCD indicator on the front of the panel indicating the depth of leachate, in inches. Sump pumps will have panel mounted failure and run lights. The control building will be provided with a rotating beacon on the roof, to be activated by either a pump failure or a high tank level. Leachate flow will be monitored by an accumulating flow meter on the discharge line.

6.4.6 Piping and Supports

The leachate removal piping between the sump pumps and the leachate temporary storage tank is comprised entirely of HDPE pipe. From the tank to the discharge fitting, the pipe is stainless steel. The piping is small diameter (3-inch-diameter or less), low pressure (25 psi or less), and much of it is located in the side slope riser pipes. All above-ground piping lies over the secondary tank containment system or has been provided with secondary containment. Leachate will be pumped to the storage tank, which will be maintained at atmospheric pressure. No in-line valves, except check valves, in the piping are planned.

No formal static and dynamic pipe stress analysis was performed on this system. The justification for this decision is summarized as follows:

- Internal pressures are low, producing low hoop stresses in the pipe.
- Diameters are small, producing small thrust forces with pipe restraint, or small longitudinal pipe stresses without pipe restraint.
- Dead loads are very small because of the small diameters.
- Pipe walls are conservatively thick.
- HDPE is an extremely ductile and energy absorbent material. HDPE pipe, through its inherent physical properties, can withstand significant deflection without failure, i.e., the pipe can be flattened without causing a fracture of the pipe wall. It is a nearly ideal material for use in resisting seismic forces.
- Seismic risk and seismic loadings at Hanford are relatively low.
- Any leaks in the piping system will be contained by concrete basins lined with HDPE membrane.
- The submittal process for pipe supports will allow a final review of pipe stresses.

6.5 Storm Water Management

All stormwater can be conveyed away from the project facilities using drainage ditches constructed as part of the access road system.

6.6 Facility Filling Plan

It is assumed that the majority of the waste stream disposed of in the Project W-025 landfill will be packaged in 55 gallon drums or irregularly shaped burial boxes. In order to maximize the landfill capacity, minimize the potential for instability of the waste fill slopes, and allow for flexibility in placement of remote handled material, the facility should be filled in horizontal lifts. Each lift will be approximately 5 feet thick. In general, it is expected that this will consist of one lift of drums and 1 to 2 feet of cover soil. Each lift will be constructed across the entire landfill floor before starting the subsequent lift.

A small percentage of the waste will be remote handled, and operating personnel will need to be shielded during and after placement. Once such waste has been placed in the landfill, shielding will be provided by concrete barrier block walls. These will be constructed from precast concrete blocks which will be stacked to approximately 12.5 feet high. The resulting enclosure will be backfilled with soil to provide adequate radiation shielding.

The FDC specified that the facility should be backfilled to grade at a minimum. Up to 2 feet above grade is allowable prior to cover construction. Furthermore, a total of 16 feet of backfill material (the closure cover may be included as part of this thickness) is required over the top of the uppermost waste package. Based on these criteria, contours for the top of waste have been prepared as shown on the design drawings (Appendix B). In addition, sections are presented which show the lifts of waste and the depth and extent of the final backfill material. Design of the final cover is beyond the scope of this study. However, it is expected that the landfill design and filling plan presented here will accommodate placement of a RCRA-compliant cover at a future date.

6.7 Electrical Service and Lighting

Power for the W-025 Landfill facilities will be obtained from the main Hanford grid system. No backup system has been included because of the high reliability of the Hanford system. Electrical power is available near the site via an existing 13.8 Kv line running along the west side of Dayton Avenue. A tap will be installed and three 10 KVA transformers will be mounted on the existing pole nearest to the control building. An above-ground line will be brought to a new pole immediately adjacent to the control building. Service will be 480 V, 3 phase, with a step down transformer in the control building to provide 230 V and 115 V service for controls and lights.

Area lighting will be provided on the new electrical service pole and on three additional poles along the entrance road from Dayton Avenue to the control building. This will provide sufficient light for the leachate temporary storage tank area and control building. Lighting for the bottom of the waste disposal facility is not provided.

7. OPERATING PROVISIONS

7.1 Period of Operations

The FDC required that the Project W-025 facility be operational for approximately 2 years. Subsequently, the operational life was extended to 20 years. The effects of this increase were evaluated in a separate study (Golder Associates Inc. 1991a), and the results have been incorporated into this revision of the landfill design. The post-closure period during which regular maintenance and monitoring will be required has not yet been finalized. However, the length of the post-closure period does not have a significant effect on the landfill design.

7.2 Monitoring and Maintenance

Routine maintenance and monitoring of the Project W-025 facility will occur during its operational phase and will continue into the post-closure period. The details of these activities will be established in an operations and maintenance plan to be prepared by others. Elements of that plan should include, but not be limited to, the following items:

- A requirement that operational cover soil, placed over each lift of waste, and any other soil materials be placed on slopes from the bottom up. Soil materials should never be placed on slopes from the top down.
- The operations layer on the slopes should be monitored periodically for wind or water erosion. If the thickness of material drops below 2 feet, additional material should be pushed up the slopes and graded to the design elevations.
- Surface water drainage ditches surrounding the facility should be cleaned out and graded in the fall of each year prior to the start of the rainy season.
- Periodic (daily) readings of the leachate levels in the primary and secondary sumps should be recorded.
- Periodic (daily) reading of the leachate flow meter and the leachate level in the storage tank should be recorded.
- Periodic testing of all four leachate removal pumps should be performed.
- Periodic (12 months) removal and re-calibration of the level transducers should be performed.

- Periodic (12 months) testing of all control panel relays and switches should be performed.

- Periodic (12 months) exercising of all valves should be performed.

Additional monitoring and maintenance of the equipment should be performed in accordance with the manufacturers' recommendations.

8. REFERENCES

Carlson, R.A., 1988, Functional Design Criteria, Interim RMW Disposal Facility, SD-W025-FDC-001, Westinghouse Hanford Company, Richland, Washington.

Cummans, J.E., M.R. Collings, and E.G. Nassar, 1975, Magnitude and Frequency of Floods in Washington, U.S. Geological Survey Open File Report 74-336, U.S. Department of the Interior, Washington, D.C.

EBASCO Services Inc. and ICF Inc., 1988, Interim Radioactive Mixed Waste Disposal Facility Leachate Control System Engineering Study, prepared for Westinghouse Hanford Company, Richland, Washington.

EPA, 1985, Draft Minimum Technology Guidance on Double Liner Systems for Landfills and Surface Impoundments - Design, Construction, and Operation, EPA/530-SW-85/014, Office of Solid Waste and Emergency Response, Washington, D.C.

EPA, 1989, Requirements for Hazardous Waste Landfill, Design, Construction, and Closure, EPA/625/4-89/022, Office of Research and Development, Cincinnati, Ohio.

Gephart, R.E., R.C. Arnett, R.G. Baca, L.S. Leonhart, and F.A. Spane, Jr., 1979, Hydrologic Studies within the Columbia Plateau, Washington: An Integration of Current Knowledge, RHO-BWI-ST-5, Rockwell Hanford Operations, Richland, Washington.

Golder Associates Inc., 1989, Site Investigation Report, Non-Drag-Off Landfill Site, Low Level Burial Area No. 5, 200 West Area, WHC-SD-W025-SE-001, Rev. 0, prepared for U.S. Department of Energy, Richland, Washington.

Golder Associates Inc., 1990, Quality Assurance Program Plan for Project W-025, Radioactive Mixed Waste (RMW) Land Disposal Facility - Non-Drag-Off, prepared for U.S. Department of Energy, Richland, Washington.

Golder Associates Inc., 1991a, Liner Performance Operating Life Study, Project W-025 Landfill, Hanford Site, Washington, WHC-SD-W025-PD-001, prepared for Westinghouse Hanford Company, Richland, Washington.

Golder Associates Inc., 1991b, Geosynthetic Liner / Leachate Compatibility Testing in Support of Project W-025 Radioactive Mixed Waste Disposal Facility, WHC-SD-W025-TRP-001, prepared for Westinghouse Hanford Company, Richland, Washington.

Golder Associates Inc., 1991c, Soil Liner / Leachate Compatibility Testing in Support of Project W-025 Radioactive Mixed Waste Disposal Facility, WHC-SD-W025-TRP-002, prepared for Westinghouse Hanford Company, Richland, Washington.

Golder Associates Inc., 1991d, Alternative Operational Backfill Material Study for W-025 Landfill, Hanford, Washington, WHC-SD-W025-ER-001, prepared for Westinghouse Hanford Company, Richland, Washington.

Golder Associates Inc., 1992, Construction Schedule and Cost Estimate for Project W-025 Radioactive Mixed Waste (RMW) Land Disposal Facility Non-Drag-Off, Revision 0, U.S. Department of Energy, Richland, Washington.

Golder Associates Inc., 1993, Construction Quality Assurance Plan for Project W-025, Radioactive Mixed Waste (RMW) Land Disposal Facility - Non-Drag-Off, WHC-SD-W025-PLN-001, Revision 3, U.S. Department of Energy, Richland, Washington.

Golder Associates Inc., 1994a, Material Specifications and Construction Requirements for the Radioactive Mixed Waste Land Disposal Facility, Non-Drag-Off, Project W-025, WHC-S-045, Revision 3, U.S. Department of Energy, Richland, Washington.

Golder Associates Inc., 1994b, Construction Quality Assurance Report, Radioactive Mixed Waste Land Disposal Facility, Non-Drag-Off, Project No. W-025, Hanford Site, Richland, Washington, WHC-SD-W025-RPT-001, Rev. 0, U.S. Department of Energy, Richland, Washington.

KEH, 1988, Conceptual Design Report, Radioactive Mixed Waste Disposal Facilities, Project W-025, Project W-031, SD-W025-CDR-001, Revision 0, Westinghouse Hanford Company, Richland, Washington.

Last, G.V., B.N. Bjornstad, M.P. Bergeron, D.W. Wallace, D.R. Newcomer, J.A. Schramke, M.A. Chamness, C.S. Cline, S.P. Airhart, and J.S. Wilbur, 1989, Hydrology of the 200 Areas Low-level Burial Grounds - An Interim Report, PNL-6820, Pacific Northwest Laboratory, Battelle Memorial Institute, Richland, Washington.

Stone, W.A., J.M. Thorp, O.P. Gifford, and D.J. Hoitink, 1983, Climatological Summary for the Hanford Area, PNL-4622, Pacific Northwest Laboratory, Battelle Memorial Institute, Richland, Washington.

FIGURES

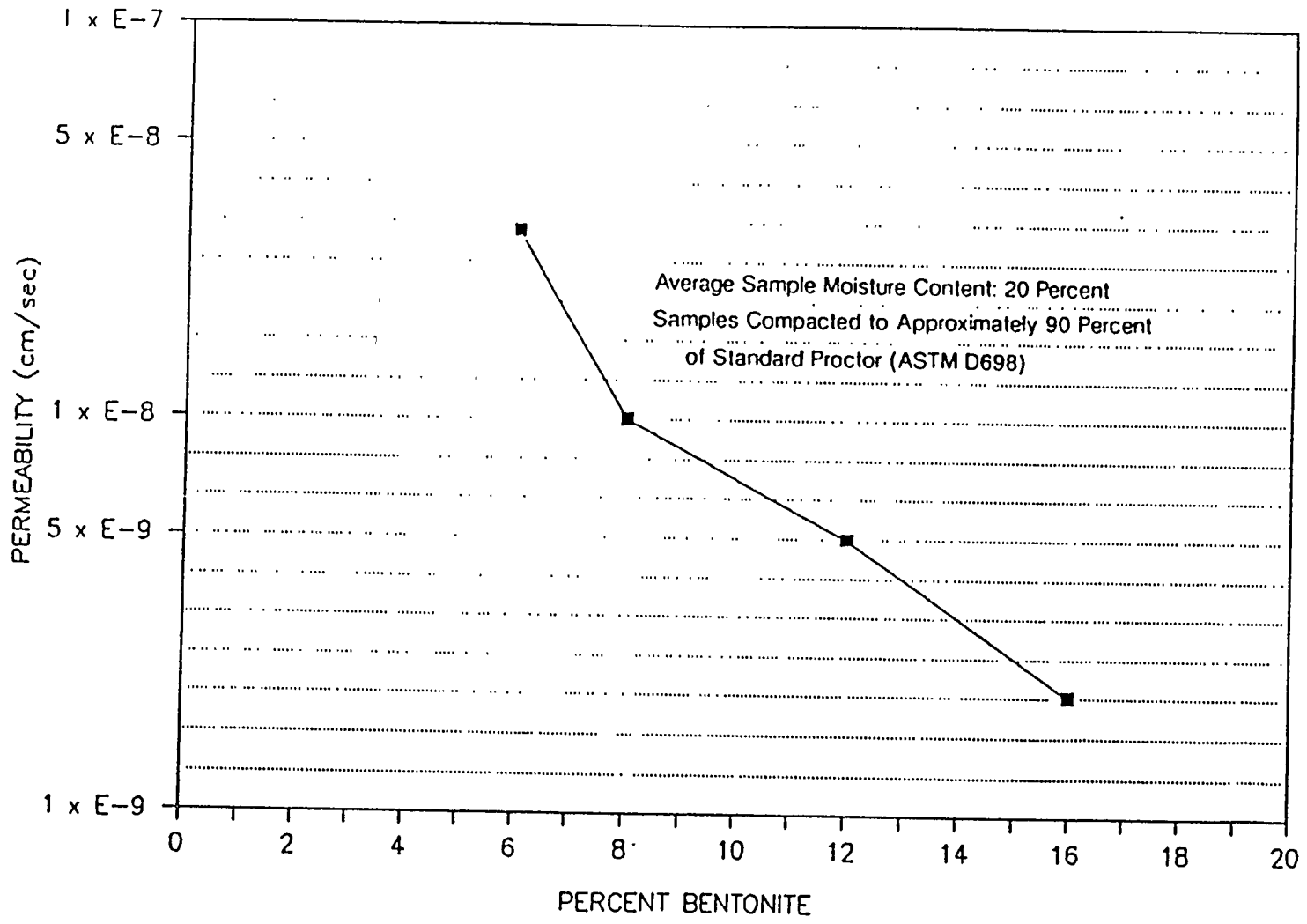


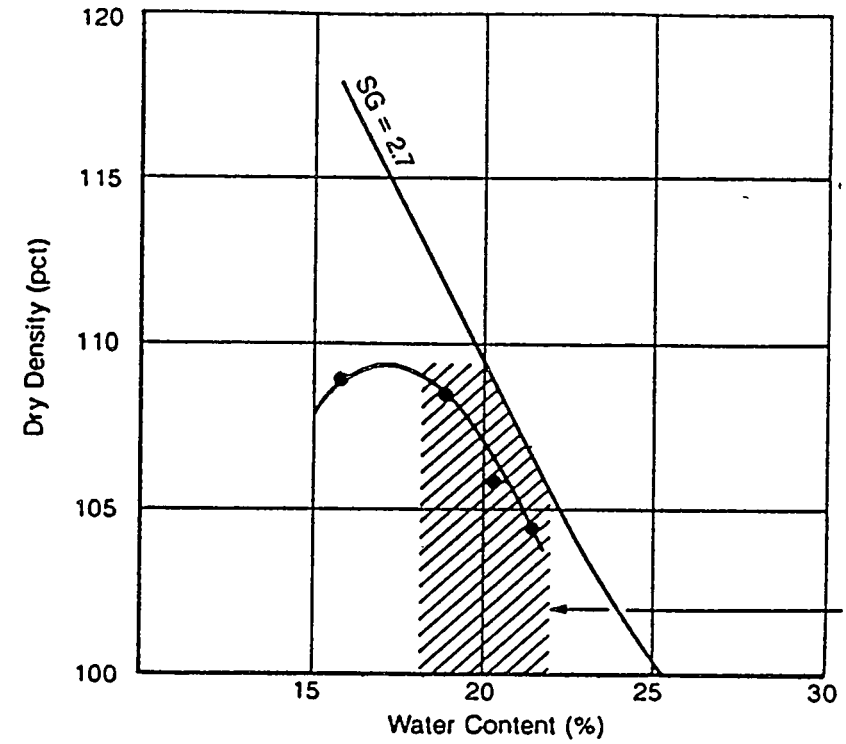
FIGURE 1

PERMEABILITY OF ADMIX VERSUS PERCENT BENTONITE

DOE-RL/NON-DRAG-OFF/WA

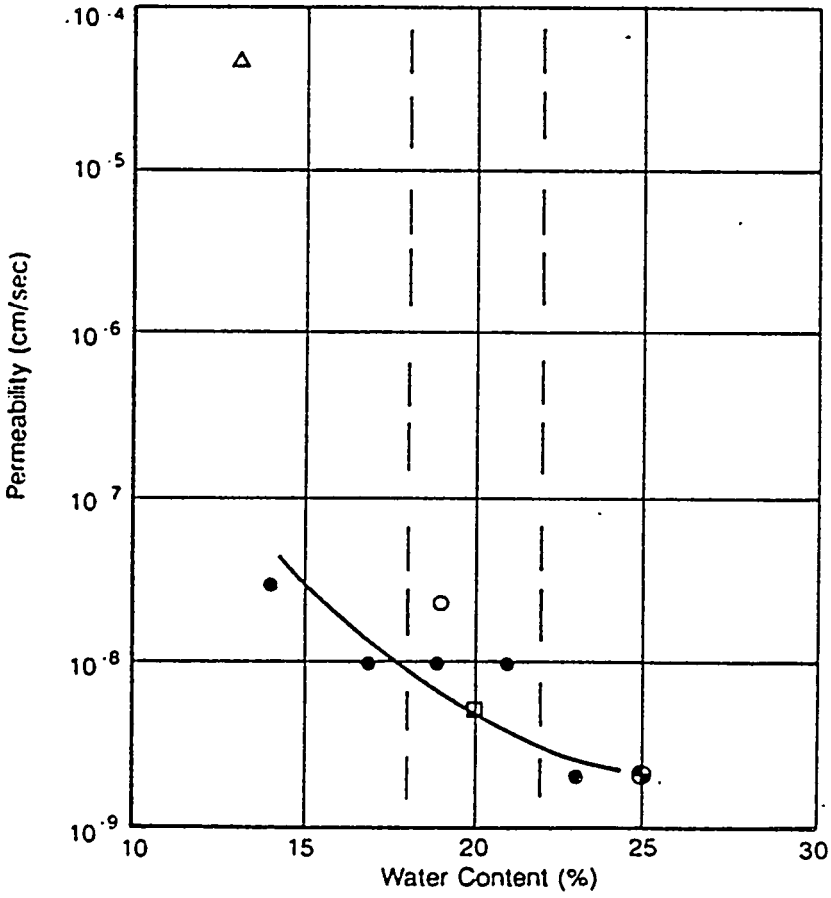
PROJECT NO 893-1186.034 DWG NO 21354 DATE 2/14/90 DRAWN RR APPROVED

Golder Associates



8% Bentonite Admix
Max. δ_d = 109.5 pcf
Wopt = 17%
ASTM D698

Range of Moisture Content and Dry Density for Placement



Admix
- ○ 6% Bentonite
- ● 8% Bentonite
- △ 10% Ephrata Bentonite
- □ 12% Bentonite
- ⊕ 16% Bentonite

Samples: Mixed, Moisture Conditioned and Recompacted in the Laboratory to Approximately 90 Percent of Standard Proctor (ASTM D698)

FIGURE 2

MOISTURE-DENSITY-PERMEABILITY RELATIONSHIP OF ADMIX

DOE-RL/NON-DRAG-OFF/WA

893-1186 034 21357 2/14/90 RR Golder Associates

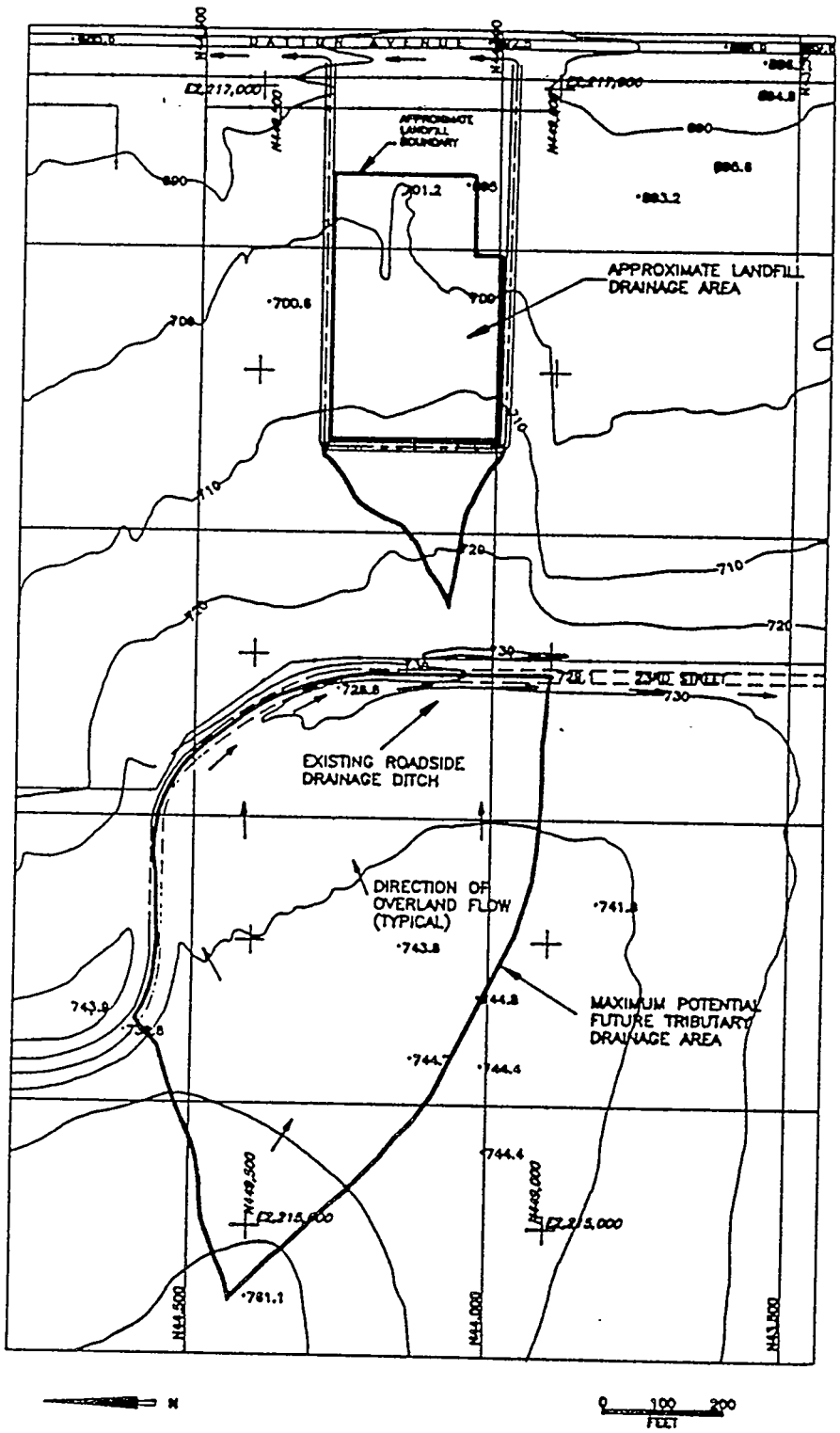
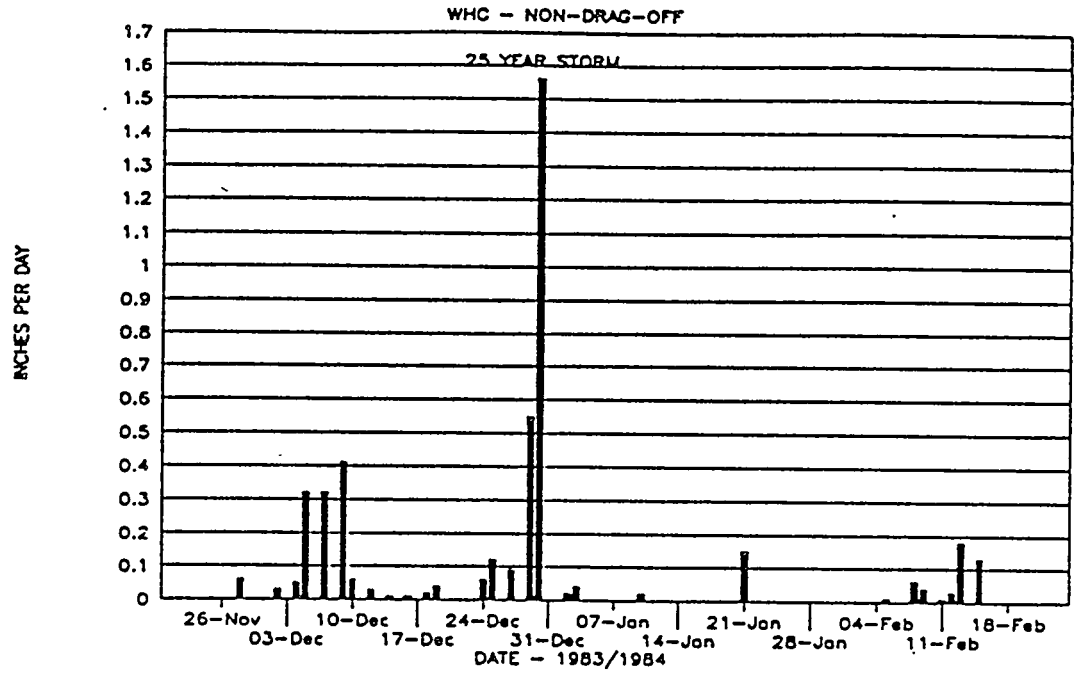


FIGURE 3
DRAINAGE BASIN
DOE/NON-DRAG-OFF/RICHLAND

PROJECT NO 893-1186.035 DWG NO 33339 DATE 2/14/90 DRAWN JSS APPROVED

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DAILY PRECIPITATION WITH 25 YEAR EVENT



LEACHATE REACHING PRIMARY COLLECTION SUMP

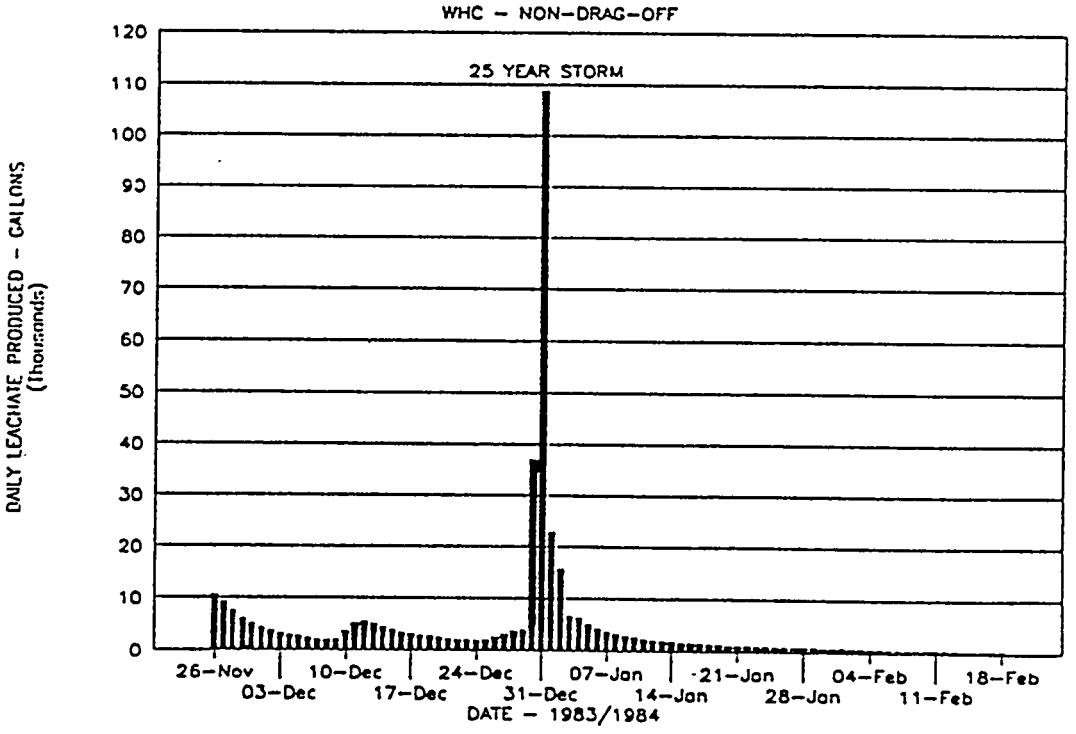


FIGURE 4

DAILY PRECIPITATION AND RESULTING DAILY LEACHATE PRODUCTION - 25 YEAR STORM

DOE-RL/NON-DRAG-OFF/WA

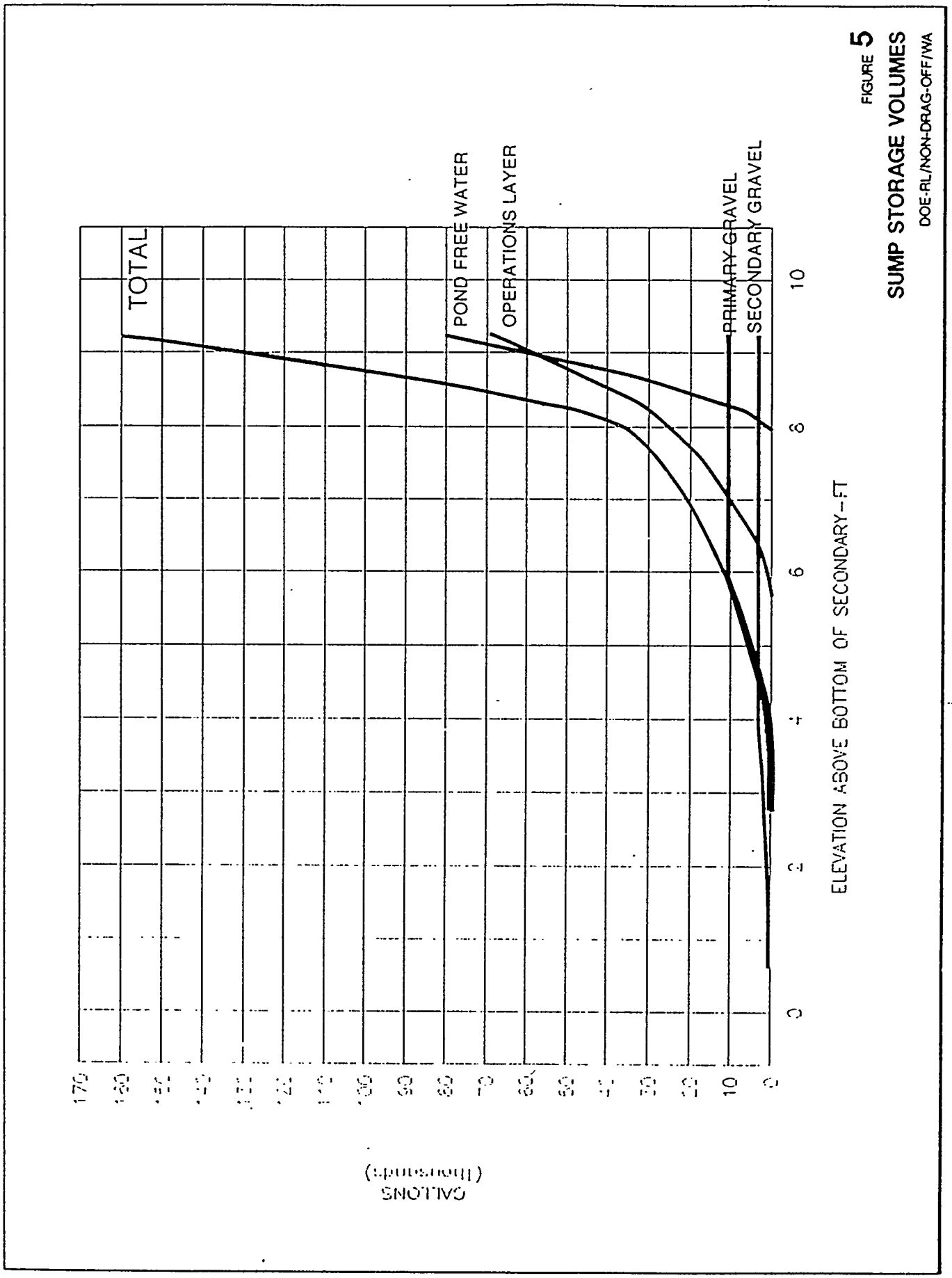


FIGURE 5
SUMP STORAGE VOLUMES
DOE-RL/NON-DRAG-OFF/WA

Golder Associates

PROJECT NO 893-1186.034 DWG NO 21353 DATE 2/14/90 DRAWN BY APPROVED

TABLES

Table 1. Hanford 25 Year Extreme Value Precipitation Depth and Intensity.

	DURATION						
	20 min	60 min	2 hrs	3 hrs	6 hrs	12 hrs	24 hrs
DEPTH (in)	0.47	0.62	0.74	0.83	1.21	1.45	1.56
INTENSITY (in/hr)	1.4	0.62	0.37	0.28	0.20	0.121	0.065

Table 2. Laboratory Test Summary.

PERMEABILITY TEST RESULTS				
PERCENT BENTONITE	PERCENT MOISTURE	PERMEABILITY (cm/sec)	UNCONFINED COMPRESSIVE STRENGTH - POCKET PEN. (TSF)	DRY DENSITY (pcf)
6	19	3E-08	0.8	102.1
8	14	3E-08	1.1	105.3
8	17	1E-08	1.9	106.7
8	19	1E-08	1.7	103.7
8	21	1E-08	1.0	98.1
8	23	2E-09	3.3	102.4
10	13	7E-05	--- (Ephrata bentonite)	104.6
12	20	5E-09	0.9	99.0
16	25	2E-09	1.1	95.6

ATTERBERG LIMITS			
PERCENT BENTONITE	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
8	54	24	30
12	70	20	50
16	100	19	81

Table 2. Laboratory Test Summary. (Cont.)

SIEVE ANALYSIS			
8 PERCENT BENTONITE		10 PERCENT BENTONITE	
SIEVE NUMBER	PERCENT PASSING	SIEVE NUMBER	PERCENT PASSING
4	100.0	4	100.0
10	99.8	10	99.9
20	95.8	20	95.9
40	84.2	40	84.7
60	74.5	60	75.7
100	56.2	100	58.9
200	22.7	200	27.5

Table 3. Geosynthetic Interface Testing Summary.

INTERFACE	FRICTION ANGLE peak (residual) (degrees)	ADHESION peak (residual) (pounds/sq. ft.)
Operations Layer vs. Texnet Geocomposite	30 (30)	0 (0)
Compacted Admix vs. Textured Geomembrane	40 (36)	425 (320)
Textured Geomembrane vs. Texnet Geocomposite	28 (28)	180 (180)

Table 4. Comparison of Estimates of Peak Runoff Using Different Methods of Analysis.

25 Year Flood Peak Flow Analysis Method	Assumptions	Peak (cfs)
HEC-1 HEC-1	CN=65 CN=85	0 9
Rational Method	C=0.2	5
Regression Equations		2.5
* All methods use a drainage area of 17 acres and a time of concentration of 15 minutes.		

APPENDIX A

LABORATORY TEST RESULTS

APPENDIX A.1

ADMIX LABORATORY TEST RESULTS

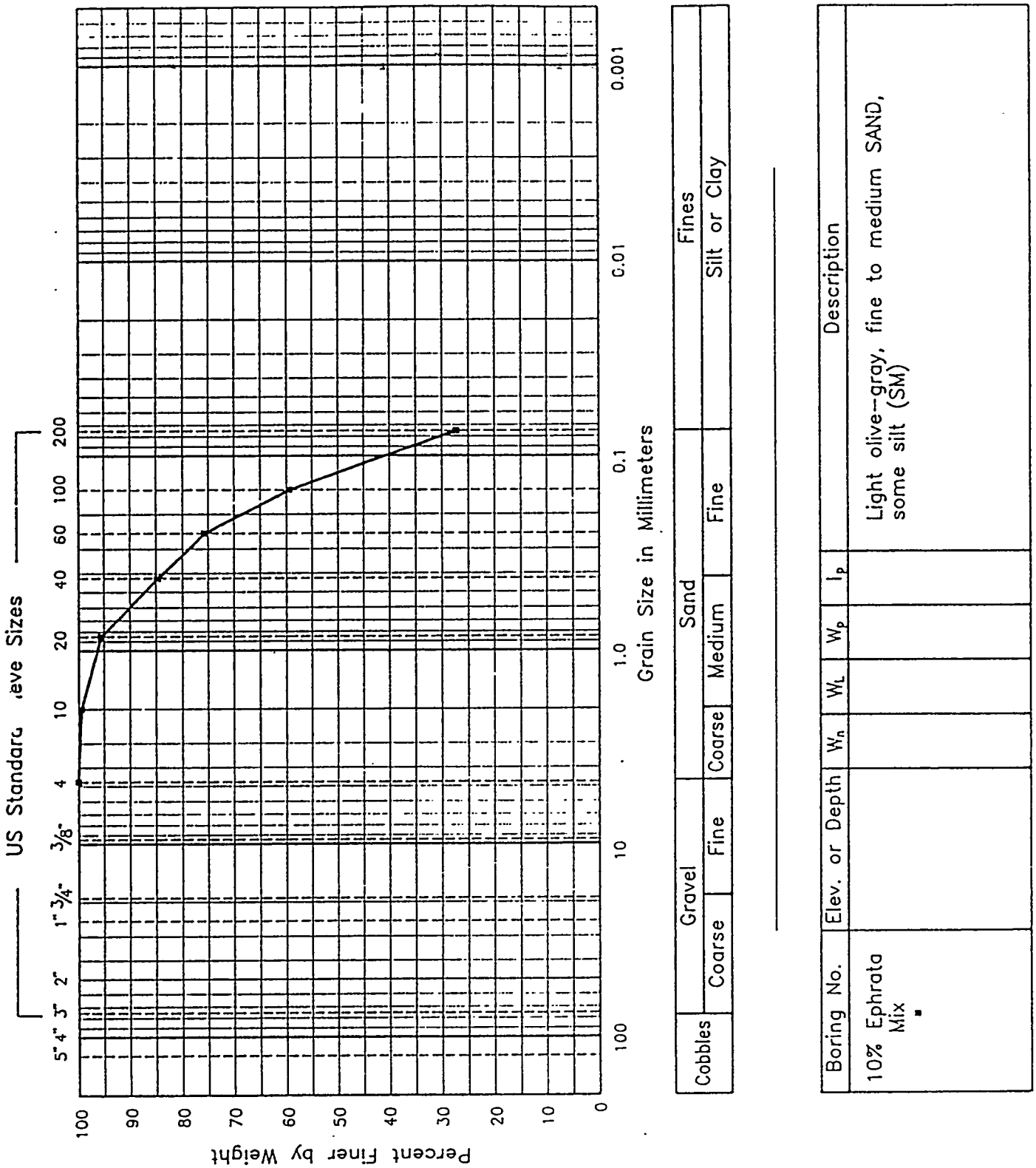


FIGURE A-1
GRAIN SIZE DISTRIBUTION

Project DOE/Non-Drag-Off/Richland
Project No. 893-1186.035 Date 2/14/90 Tested By BFW Approved By KRB
Golder Associates

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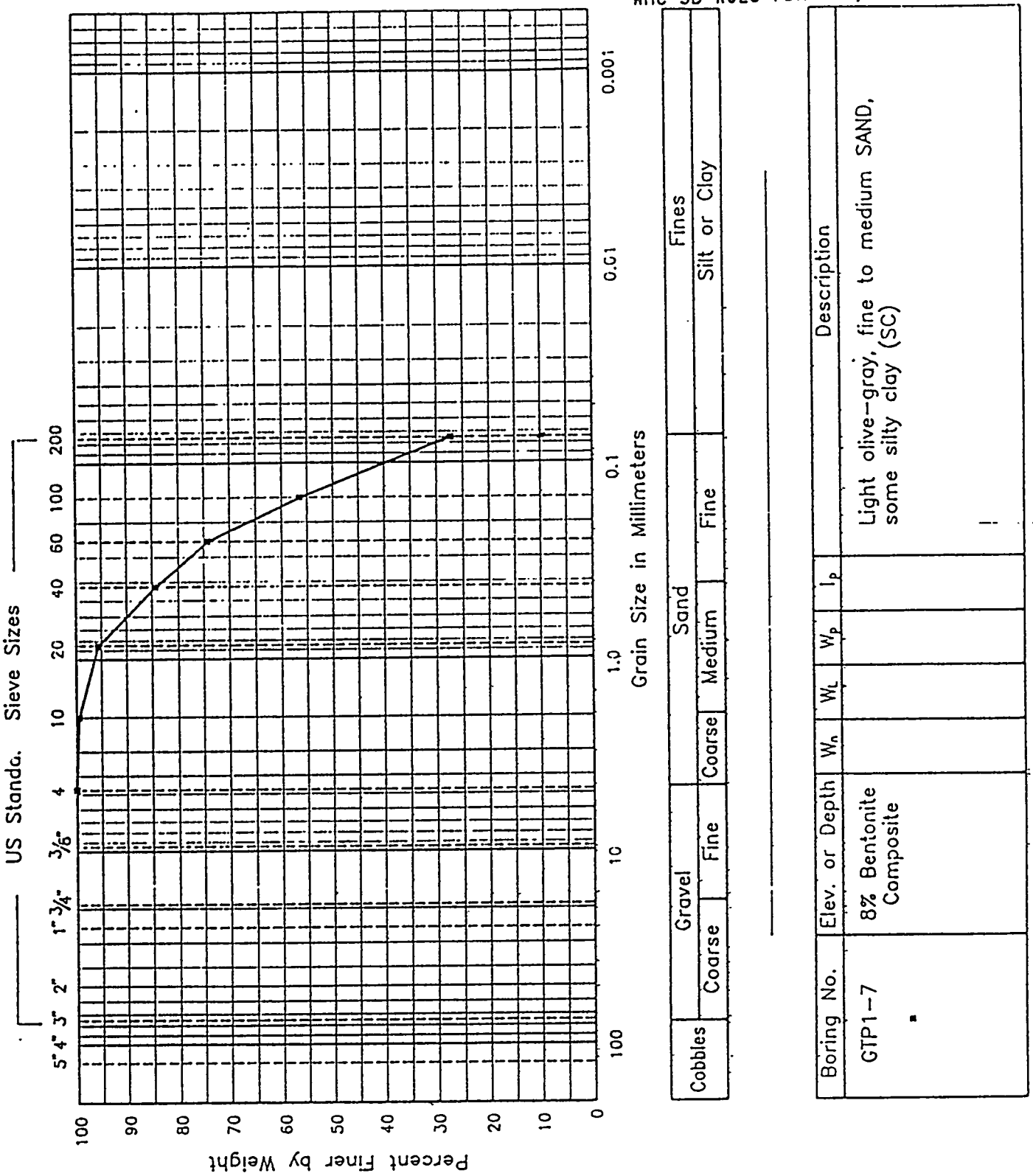


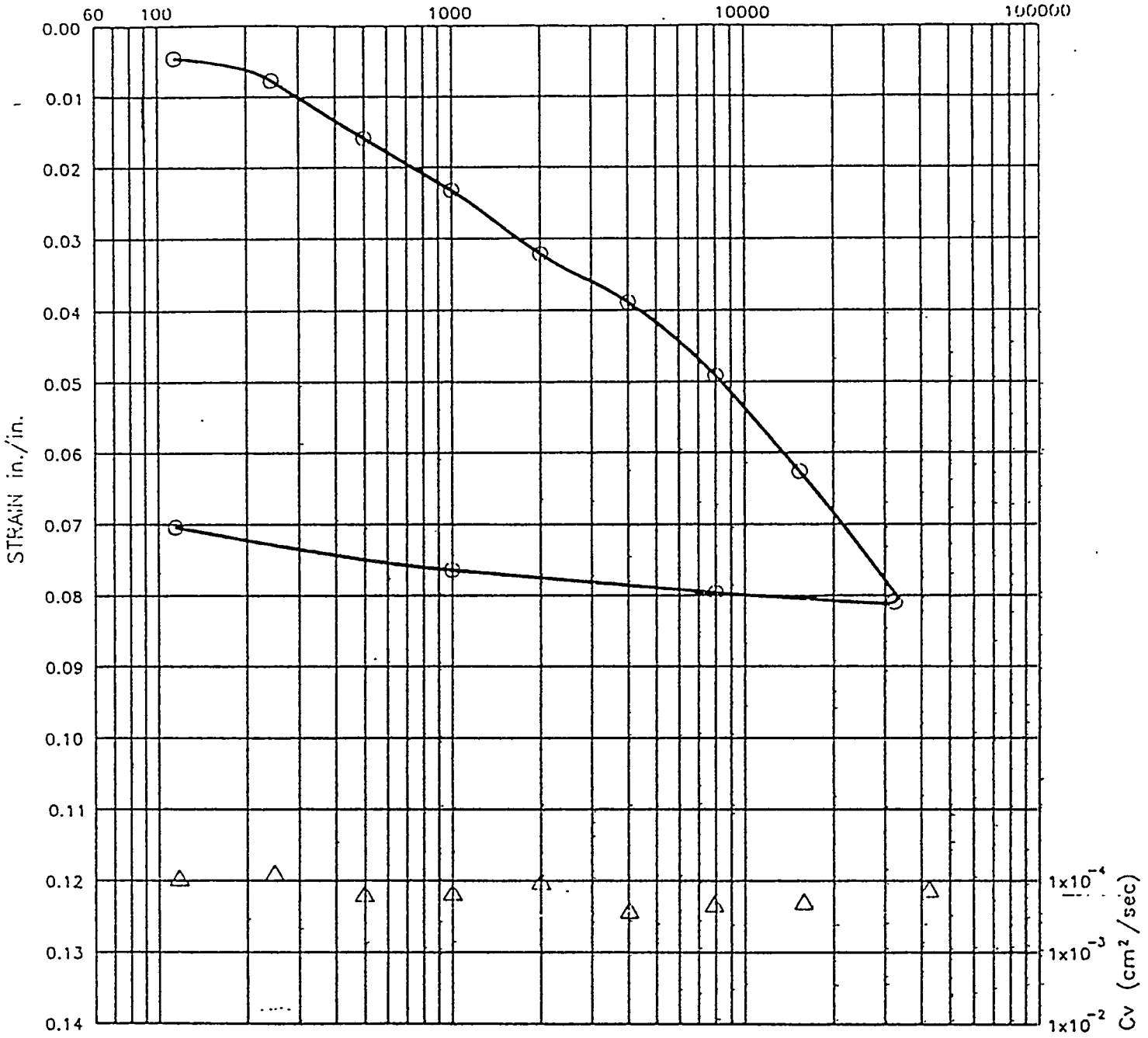
FIGURE **A-2**

GRAIN SIZE DISTRIBUTION

Project DOE/Non-Drag-Off/Richland
Project No. 893-1186.035 Date 2/14/90 Tested By BFW Approved By KRB

Golder Associates

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Boring	Boring	Soil Classification	Moisture Content Before	Moisture Content After	Dry Density in. lbs./cu. ft.
GTP1-7	8% Benonite	Light olive-gray, fine to medium SAND, some silty clay (SC)	19.6	21.7	104.2

Figure A-3
CONSOLIDATION TEST

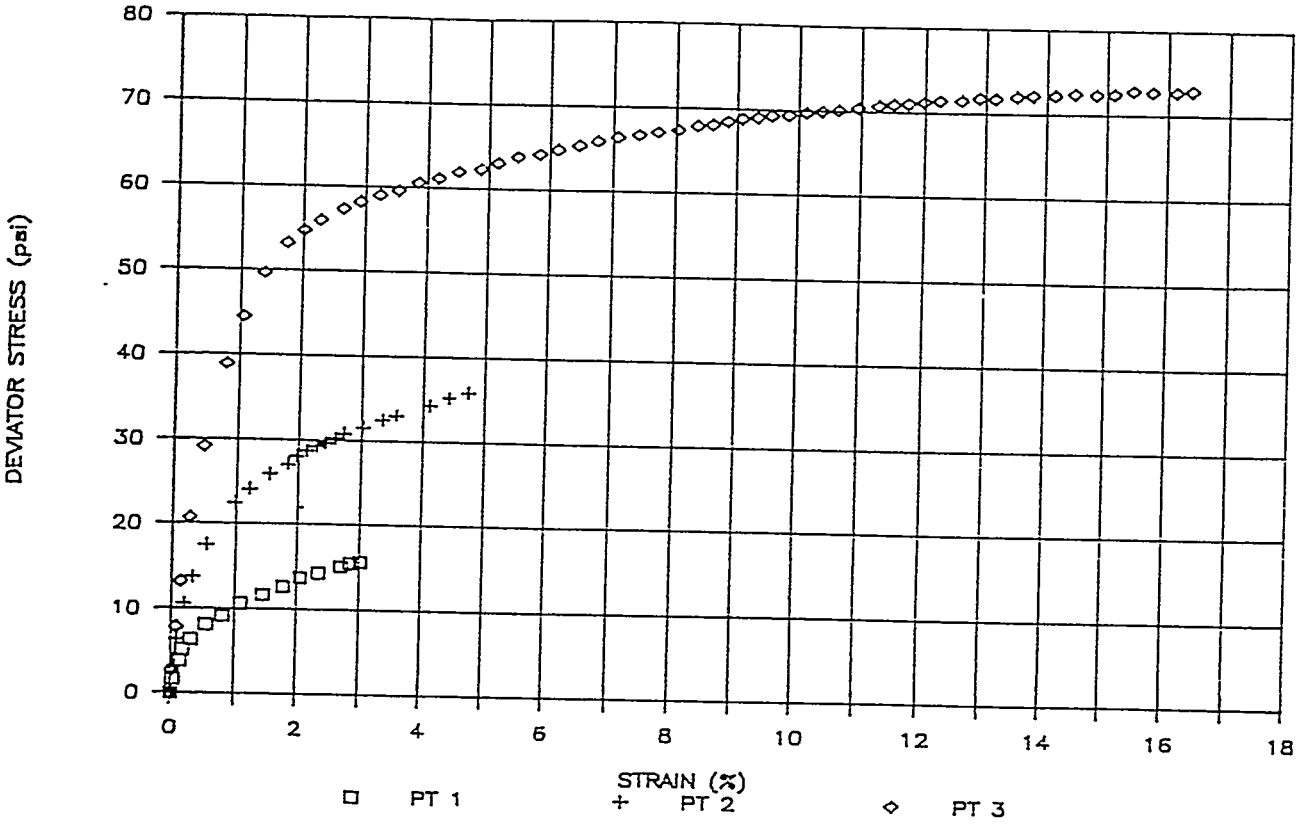
Project DOE/Non-Drag-Off/Richland

Project No. 893-1186.035 Date 12/14/89 Tested By DN Approved By KRB

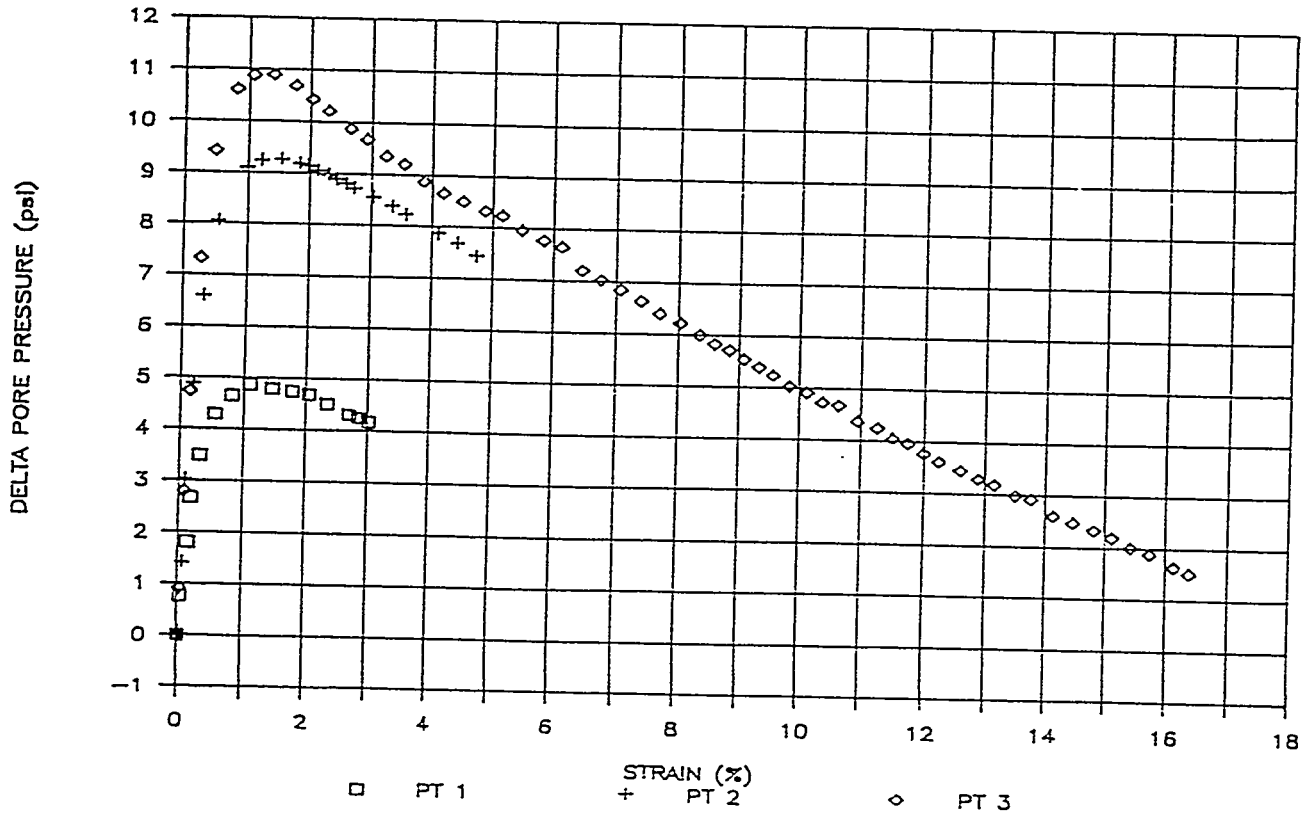
Golder Associates

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STRESS–STRAIN CURVES

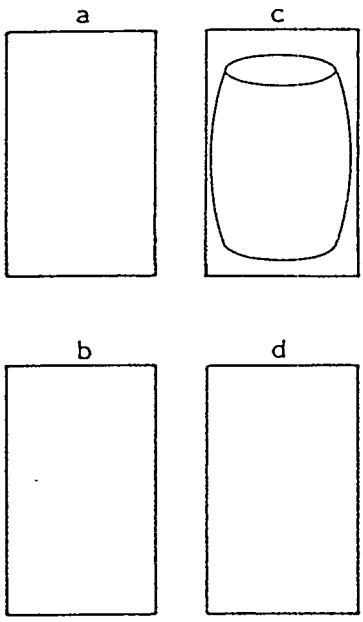


PORE PRESSURE CHANGE VS. STRAIN

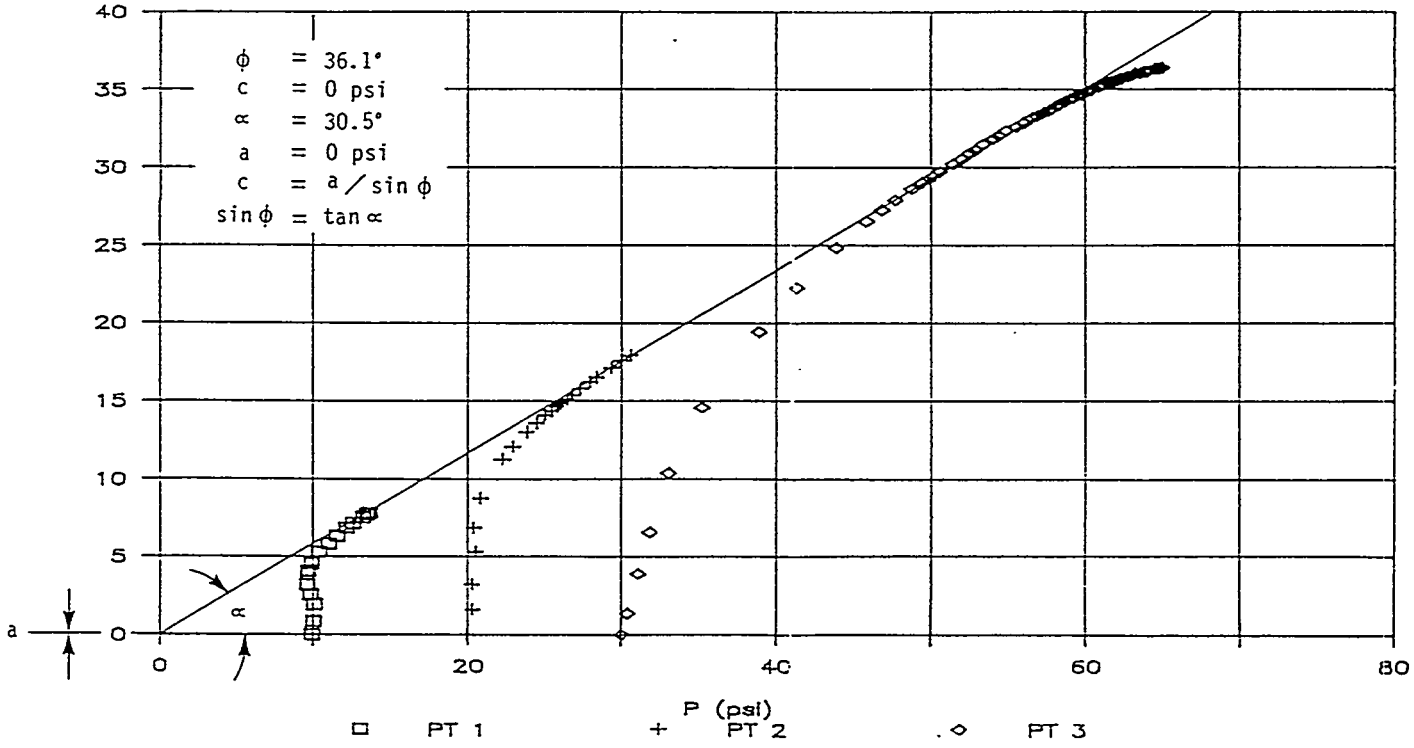


	a	b	c	d
Borehole No.	GTP			
Sample No.	1-7:8% Bentonite			
Sample Depth, ft.	Composite			
Specimen Diameter, inches	2.856	2.874	2.916	
Specimen Height, inches	6.000	5.864	5.646	
Cell Pressure σ_3, psi	80.0	90.1	100.0	
Back Pressure, psi	70.0	70.0	70.0	
Effective Confining Pressure, σ_3', psi	10.0	20.1	30.0	
Water Content before test, %	19.1	25.1	24.5	
Water Content after test, %	25.1	24.5	24.0	
Unit Weight, γ_D, lb./cu. ft.	100.5	101.1	102.2	
Average rate of Strain, in./min.	0.0014	0.0014	0.0014	

Failure Sketch



P—Q PLOT



CONSOLIDATED UNDRAINED STAGED TRIAXIAL COMPRESSION TEST

Figure A-4

Project DOE RL/NON-DRAG-OFF/SEATTLE

Project No. 893-1186.033 Date 12/19/89 Tested by DN Approved by KRB

Golder Associates

GOLDER ASSOCIATES
PERMEABILITY TEST SPREADSHEET

Project Number: 893-1186.033
Project Description: DOE RL/HOH-DRAG-OFF/SEATTLE

Sample Number: GTP1-7 6% BENTONITE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.214	7.246
Diameter:	7.272	7.323
Area:	41.53	42.12
Volume:	299.6	305.2

Sample Properties :	Pre Test	Post Test
Wet Weight (g):	583.24	611.02
Moisture (%):	19.1	24.8
Wet Density (pcf):	121.5	125.0
Dry Density (pcf):	102.1	100.2
Dry Weight (g):	489.70	489.70
Specific Grav.:	2.67	2.67
Void Ratio:	0.63	0.66
Saturation (%):	80	100

Target Test Cond. (psi):	Saturation	Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom stone:	92.5	95.0
B - coefficient:	0.953	N.A.
Gradient:	0	49

TEST RESULTS

K = 3E-8 cm/second

Tested by: BHC
Reviewed by: KLB

SPREADSHEET UPDATED 6/13/89, K. BROWN

Board number: L0014

Cell number: L0005

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)

yr	mo	da	hr	min	sec	Elapsed (min.)	CELL VOLUMES (cc) Reading	Delta	Total	INFLOW VOLUMES (cc) Reading	Refill	Delta	Total	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Bottom	Top		Head (cm)
	5	8	21	0	0	0.00	2.6	0.0	0.0	0.4	0.0	0.0	0.0	24.2	0.0	0.0	0.0	0	100.2	92.2	89.9	185.6	
	5	9	27	0	0	66.00	2.2	-0.4	-0.4	5.3	0.0	4.9	4.9	19.4	0.0	4.8	4.8	7.3E-07	100.0	95.1	89.8	393.9	
	5	10	35	0	0	134.00	2.2	0.0	-0.4	9.3	0.0	4.0	8.9	15.4	0.0	4.0	8.8	4.3E-07	99.9	95.1	89.7	385.9	
89	12	5	11	20	0	179.00	2.2	0.0	-0.4	11.6	0.0	2.3	11.2	13.1	0.0	2.3	11.1	3.8E-07	99.8	95.1	89.7	381.3	
89	12	5	12	55	0	274.00	2.2	0.0	-0.4	15.2	0.0	3.6	14.8	9.5	0.0	3.6	14.7	2.9E-07	100.1	95.1	89.9	380.1	
89	12	5	14	0	0	339.00	2.2	0.0	-0.4	17.1	0.0	1.9	16.7	7.5	0.0	2.0	16.7	2.4E-07	100.0	95.2	89.8	383.2	
89	12	5	15	8	0	407.00	2.2	0.0	-0.4	18.9	0.0	1.8	18.5	5.8	0.0	1.7	18.4	2.1E-07	99.9	95.0	89.8	382.7	
89	12	5	16	3	0	462.00	2.2	0.0	-0.4	20.2	0.0	1.3	19.8	4.6	0.0	1.2	19.6	1.9E-07	99.8	95.0	89.9	383.1	
89	12	5	17	2	0	521.00	2.2	0.0	-0.4	21.4	0.0	1.2	21.0	3.5	0.0	1.1	20.7	1.6E-07	99.8	94.9	89.8	380.8	
89	12	6	9	19	0	0.00	16.8	0.0	0.0	0.1	0.0	0.0	0.0	23.9	0.0	0.0	0.0	8.8E+00	100.0	94.9	89.9	375.5	
89	12	6	10	21	0	62.00	16.9	0.1	0.1	0.3	0.0	0.2	0.2	23.6	0.0	0.3	0.3	3.1E-08	100.0	94.9	89.9	375.0	
89	12	6	11	32	0	133.00	17.0	0.1	0.2	0.6	0.0	0.3	0.5	23.3	0.0	0.3	0.6	3.3E-08	99.9	94.8	89.9	367.4	
89	12	6	14	33	0	314.00	17.1	0.1	0.3	1.6	0.0	1.0	1.5	22.4	0.0	0.9	1.5	4.8E-08	100.1	95.1	89.8	379.5	
89	12	6	17	7	0	468.00	17.2	0.1	0.4	2.3	0.0	0.7	2.2	21.7	0.0	0.7	2.2	3.5E-08	100.0	94.9	89.9	371.1	
89	12	7	8	30	0	1391.00	17.6	0.4	0.8	6.4	0.0	4.1	6.3	17.8	0.0	3.9	6.1	3.4E-08	100.0	95.0	89.9	370.1	
89	12	7	10	58	0	1539.00	17.6	0.0	0.8	7.0	0.0	0.6	6.9	17.2	0.0	0.6	6.7	3.2E-08	100.0	94.9	89.9	361.9	
89	12	7	11	45	0	1586.00	17.6	0.0	0.8	7.2	0.0	0.2	7.1	17.0	-0.0	-0.2	6.9	3.3E-08	100.0	95.0	89.9	368.5	

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE

TEST RESULTS: K=3E-8 cm/second
****

Sample Number: GTP1-7 8%BENTONITE/14%MOISTURE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.199	7.319
Diameter:	7.239	7.298
Area:	41.16	41.83
Volume:	296.3	306.2

Board number: L0091
Cell number: L0088

Sample Properties :	Initial	Final
Wet Weight (g):	569.71	617.56
Moisture (%):	14.0	23.6
Wet Density (pcf):	120.1	125.9
Dry Density (pcf):	105.3	101.9
Dry Weight (g):	499.69	499.69
Specific Grav.:	2.65	2.65
Void Ratio:	0.57	0.62
Saturation (%):	65	100

Tested by: DN
Reviewed by: KMB

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.98	N.A.
Gradient:	0	49

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/28/89

yr mo da hr minsec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89 12 3 14 46 0 *	0.00	0.0	25.0	0.0	0.0	0.0	0.0E+00	100.0	95.0	90.0	376.7
89 12 4 9 0 0 *	1094.00	3.6	20.3	0.0	4.7	4.7	3.5E-08	101.0	95.5	90.9	340.3
89 12 4 16 54 0 *	1568.00	4.1	17.6	0.0	2.7	7.4	4.6E-08	100.7	95.2	90.0	379.3
89 12 5 7 34 0 *	2448.00	5.6	13.8	0.0	3.8	11.2	3.3E-08	100.8	95.2	90.0	374.0
89 12 5 17 8 0 *	3022.00	6.6	11.5	0.0	2.3	13.5	3.1E-08	100.7	95.2	89.9	377.7
89 12 6 7 55 0 *	0.00	8.7	14.2	0.0	0.0	0.0	0.0E+00	101.5	95.3	92.2	223.6
89 12 6 13 25 0 *	330.00	9.1	12.9	0.0	1.3	1.3	3.0E-08	100.7	95.2	87.7	531.4
89 12 6 17 9 0 *	554.00	9.4	12.1	0.0	0.8	2.1	2.3E-08	100.7	95.2	89.9	375.5
89 12 7 8 36 0 *	1481.00	10.9	9.0	0.0	3.1	5.2	2.6E-08	100.8	95.2	89.9	370.9
89 12 7 13 49 0 *	1794.00	11.4	8.0	0.0	1.0	6.2	2.5E-08	100.8	95.2	89.9	369.4
89 12 8 8 47 0 *	2932.00	2.2	4.5	0.0	3.5	9.7	2.6E-08	100.0	94.8	90.1	332.9

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE

TEST RESULTS: K=1E-8 cm/second
****

Sample Number: GTP1-7 8%BENTONITE/17%MOISTURE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.016	7.065
Diameter:	7.286	7.292
Area:	41.69	41.76
Volume:	292.5	295.0

Board number: L0091
Cell number: L0008

Sample Properties :	Initial	Final
Wet Weight (g):	584.96	612.00
Moisture (%):	17.0	22.4
Wet Density (pcf):	124.9	129.5
Dry Density (pcf):	106.7	105.8
Dry Weight (g):	500.06	500.06
Specific Grav.:	2.73	2.73
Void Ratio:	0.60	0.61
Saturation (%):	78	100

Tested by: DN
Reviewed by: KRB

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.99	N.A.
Gradient:	0	50

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/28/89

yr mo da hr minsec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89 12 3 14 46 0 *	0.00	0.0	25.0	0.0	0.0	0.0	0.0E+00	100.0	95.0	90.0	376.7
89 12 4 9 0 0 *	1094.00	3.6	23.3	0.0	1.7	1.7	1.1E-08	101.0	95.5	90.1	399.5
89 12 4 16 54 0 *	1568.00	4.1	22.4	0.0	0.9	2.6	1.3E-08	100.7	95.2	89.8	398.1
89 12 5 7 34 0 *	2448.00	5.6	21.0	0.0	1.4	4.0	1.1E-08	100.8	95.2	89.6	409.3
89 12 5 17 59 0 *	3073.00	6.6	20.2	0.0	0.8	4.8	9.0E-09	100.7	95.2	89.8	393.4
89 12 6 7 56 0 *	0.00	8.7	19.1	0.0	0.0	0.0	0.0E+00	101.5	95.3	90.1	376.2
89 12 6 13 25 0 *	329.00	9.1	18.2	0.0	0.9	0.9	2.0E-08	100.7	95.2	89.7	396.0
89 12 6 17 10 0 *	554.00	9.4	17.9	0.0	0.3	1.2	9.4E-09	100.7	95.2	89.6	402.4
89 12 7 8 36 0 *	1480.00	10.9	16.2	0.0	1.7	2.9	1.3E-08	100.7	95.2	89.6	399.2
89 12 7 13 50 0 *	1794.00	11.4	15.7	0.0	0.5	3.4	1.1E-08	100.7	95.2	89.6	398.2
89 12 8 7 48 0 *	2872.00	2.2	13.8	0.0	1.9	5.3	1.3E-08	100.0	94.8	89.7	370.3

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE

TEST RESULTS: K=1E-8 cm/second
****

Sample Number: GTP1-7 8%BENTONITE/19%MOISTURE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.202	7.213
Diameter:	7.294	7.273
Area:	41.79	41.54
Volume:	300.9	299.7

Board number: L0091
Cell number: L0007

Sample Properties :	Initial	Final
Wet Weight (g):	594.30	615.87
Moisture (%):	18.9	23.3
Wet Density (pcf):	123.3	128.3
Dry Density (pcf):	103.7	104.1
Dry Weight (g):	499.63	499.63
Specific Grav.:	2.73	2.73
Void Ratio:	0.64	0.64
Saturation (%):	80	100

Tested by: DN
Reviewed by: KRB

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.99	N.A.
Gradient:	0	49

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/28/89

yr mo da hr minsec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89 12 3 14 46 0 *	0.00	0.0	25.0	0.0	0.0	0.0	0.0E+00	100.0	95.0	90.0	376.7
89 12 4 9 1 0 *	1095.00	3.6	23.8	0.0	1.2	1.2	8.2E-09	101.0	95.5	90.2	393.0
89 12 4 16 54 0 *	1568.00	4.1	22.6	0.0	1.2	2.4	1.9E-08	100.7	95.2	90.1	377.2
89 12 5 7 34 0 *	2448.00	5.6	21.8	0.0	0.8	3.2	7.1E-09	100.8	95.2	90.2	367.9
89 12 5 17 9 0 *	3023.00	6.6	21.3	0.0	0.5	3.7	6.8E-09	100.7	95.2	90.1	373.4
89 12 6 7 56 0 *	0.00	8.7	20.6	0.0	0.0	0.0	0.0E+00	101.5	95.5	91.2	314.4
89 12 6 13 23 0 *	327.00	9.1	19.6	0.0	1.0	1.0	2.6E-08	100.7	95.2	90.1	369.2
89 12 6 17 11 0 *	555.00	9.4	19.3	0.0	0.3	1.3	1.1E-08	100.7	95.2	90.3	354.6
89 12 7 8 36 0 *	1480.00	10.9	18.0	0.0	1.3	2.6	1.2E-08	100.8	95.2	90.3	351.8
89 12 7 13 51 0 *	1795.00	11.4	17.6	0.0	0.4	3.0	1.0E-08	100.8	95.2	90.3	350.9
89 12 8 7 49 0 *	2873.00	2.2	16.2	0.0	1.4	4.4	1.1E-08	100.0	94.8	90.6	309.4

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE

TEST RESULTS: K=1E-8 cm/second
****

Sample Number: GTP1-7 8% BENTONITE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.172	7.172
Diameter:	7.279	7.279
Area:	41.61	41.61
Volume:	298.5	298.5

Board number: L0091
Cell number: L0089

Sample Properties :	Initial	Final
Wet Weight (g):	566.97	590.40
Moisture (%):	20.9	25.9
Wet Density (pcf):	118.6	123.5
Dry Density (pcf):	98.1	98.1
Dry Weight (g):	468.96	468.96
Specific Grav.:	2.65	2.65
Void Ratio:	0.69	0.69
Saturation (%):	81	100

Tested by: DH
Reviewed by: KRB

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.99	N.A.
Gradient:	0	49

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/30/89

yr	mo	da	hr	min	sec		Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89	12	2	12	17	0	*	0.00	17.9	22.9	0.0	0.0	0.0	0.0E+00	100.0	95.1	90.2	349.7
89	12	3	14	49	0	*	1592.00	24.2	16.6	0.0	6.3	6.3	3.3E-08	99.8	94.2	89.4	330.0
89	12	4	9	47	0	*	2730.00	6.8	10.2	0.0	6.4	12.7	4.6E-08	99.6	95.1	89.9	369.2
89	12	4	16	50	0	*	3153.00	7.1	8.4	16.6	1.8	14.5	3.2E-08	99.5	95.0	89.5	388.2
89	12	5	7	36	0	*	4039.00	7.8	22.2	0.0	2.8	17.3	2.4E-08	99.8	95.2	90.1	373.1
89	12	5	17	6	0	*	4609.00	8.7	20.6	0.0	1.6	18.9	2.3E-08	98.9	95.1	90.4	342.5
89	12	6	7	53	0	*	0.00	9.7	18.5	0.0	0.0	0.0	0.0E+00	99.9	94.9	89.8	367.5
89	12	6	13	20	0	*	327.00	9.8	16.4	0.0	2.1	2.1	5.0E-08	99.9	94.9	89.8	365.3
89	12	6	17	7	0	*	554.00	9.5	15.9	0.0	0.5	2.6	2.0E-08	95.9	93.3	89.7	259.6
89	12	7	8	34	0	*	1481.00	9.9	14.5	0.0	1.4	4.0	1.4E-08	100.0	94.8	89.8	356.3
89	12	8	7	39	0	*	2866.00	10.5	12.4	0.0	2.1	6.1	1.3E-08	99.4	94.3	89.7	325.5
89	12	8	13	26	0	*	3213.00	10.6	11.8	0.0	0.6	6.7	1.4E-08	100.8	95.6	90.4	367.0

Project Number: 893-1186.033
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE

TEST RESULTS: K=2E-9 cm/second
****

Sample Number: GTP1-7 8%BENTONITE/23%MOISTURE
Sample Description: Lt. olive gray, med. to fine SAND,

some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.369	7.256
Diameter:	7.259	7.211
Area:	41.39	40.84
Volume:	305.0	296.3

Board number: L0091
Cell number: L0089

Sample Properties :	Initial	Final
Wet Weight (g):	614.13	614.34
Moisture (%):	22.8	22.9
Wet Density (pcf):	125.7	129.4
Dry Density (pcf):	102.4	105.4
Dry Weight (g):	500.01	500.01
Specific Grav.:	2.75	2.75
Void Ratio:	0.68	0.63
Saturation (%):	93	100

Tested by: DM
Reviewed by: KMB

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.99	N.A.
Gradient:	0	48

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/28/89

yr mo da hr minsec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89 12 3 14 46 0 *	0.00	0.0	25.0	0.0	0.0	0.0	0.0E+00	100.0	95.0	90.0	376.7
89 12 6 7 58 0 *	0.00	8.7	23.5	0.0	0.0	0.0	0.0E+00	101.5	95.5	91.2	317.3
89 12 6 13 23 0 *	325.00	9.1	22.9	0.0	0.6	0.6	1.6E-08	100.7	95.2	90.1	372.5
89 12 7 8 36 0 *	1478.00	10.9	22.2	0.0	0.7	1.3	4.9E-09	100.7	95.2	90.3	356.0
89 12 8 7 47 0 *	2869.00	2.2	21.8	0.0	0.4	1.7	2.4E-09	100.0	94.8	90.1	350.2
89 12 8 14 25 0 *	3267.00	1.0	21.7	0.0	0.1	1.8	2.0E-09	100.1	95.0	89.9	379.4
89 12 10 12 58 0 *	0.00	8.9	17.1	0.0	0.0	0.0	0.0E+00	101.4	96.0	90.4	402.1
89 12 11 9 5 0 *	1207.00	8.4	16.1	0.0	1.0	1.0	6.1E-09	101.4	96.2	90.5	408.6
89 12 11 14 48 0 *	1550.00	8.4	16.0	0.0	0.1	1.1	2.3E-09	100.4	94.8	90.1	338.2
89 12 12 16 27 0 *	3089.00	8.6	15.6	0.0	0.4	1.6	2.5E-09	99.9	94.5	89.6	351.6
89 12 13 7 39 0 *	4001.00	9.2	15.3	0.0	0.3	1.8	2.3E-09	100.1	94.7	89.9	343.7

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE

TEST RESULTS: K=7E-5 cm/second
****

Sample Number: GTP1-7 10%EPHRATA BENTONITE
Sample Description: Lt. olive gray, med.-fine SAND, some silty clay (SM)

Sample Dimensions (cgs):	Initial	Final
Height:	7.187	7.255
Diameter:	7.345	7.255
Area:	42.37	41.34
Volume:	304.5	299.9

Board number: L0015
Cell number: L0202

Sample Properties :	Initial	Final
Wet Weight (g):	578.10	616.11
Moisture (%):	13.4	20.8
Wet Density (pcf):	118.5	128.3
Dry Density (pcf):	104.6	106.2
Dry Weight (g):	509.92	509.92
Specific Grav.:	2.63	2.63
Void Ratio:	0.57	0.55
Saturation (%):	62	100

Tested by: DN
Reviewed by: KHS

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	_____	N.A.
Gradient:	0	49

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/28/89

yr mo da hr minsec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	I (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89 11 29 9 5 0 *	0.00	1.0	22.5	0.0	0.0	0.0	0.0E+00	100.0	93.0	89.9	239.6
89 11 29 9 6 0 *	1.00	7.2	17.0	0.0	5.5	5.5	5.1E-05	100.0	95.3	89.9	391.0
89 11 29 9 7 0 *	2.00	15.5	8.9	0.0	8.1	13.6	5.9E-05	100.0	95.8	89.9	408.4
89 11 29 9 8 0 *	3.00	24.2	0.5	24.5	8.4	22.0	6.1E-05	100.0	95.8	89.9	391.3
89 11 29 9 9 0 *	4.00	10.1	15.1	0.0	9.9	31.9	7.1E-05	100.0	95.8	89.9	420.0
89 11 29 9 10 0 *	5.00	19.4	5.5	19.1	9.6	41.5	6.8E-05	100.0	95.8	89.9	401.1
89 11 29 9 11 0 *	6.00	10.3	14.7	0.0	9.9	51.4	7.1E-05	100.0	95.8	89.9	419.4
89 11 29 9 12 0 *	7.00	19.9	5.2	19.8	9.5	60.9	6.8E-05	100.0	95.8	89.9	400.3
89 11 29 9 13 0 *	8.00	9.9	15.3	0.0	9.7	70.6	6.9E-05	100.0	95.8	89.9	420.4
89 11 29 9 14 0 *	9.00	18.8	5.5	0.0	9.8	80.4	7.0E-05	100.0	95.8	89.9	401.7

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/HON-DRAG-OFF/SEATTLE

TEST RESULTS: K=5E-9 cm/second
xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

Sample Number: GTP1-7 12%BENTONITE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.204	7.174
Diameter:	7.247	7.286
Area:	41.25	41.69
Volume:	297.2	299.1

Board number: L0091
Cell number: L0008

Sample Properties :	Initial	Final
Wet Weight (g):	563.81	595.35
Moisture (%):	19.7	26.3
Wet Density (pcf):	118.5	124.3
Dry Density (pcf):	99.0	98.4
Dry Weight (g):	471.21	471.21
Specific Grav.:	2.72	2.72
Void Ratio:	0.72	0.73
Saturation (%):	75	99

Tested by: DH
Reviewed by:

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.99	N.A.
Gradient:	0	49

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/30/89

yr	mo	da	hr	min	sec		Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Bottom	Top	Head (cm H20)
89	12	2	12	16	0	*	0.00	17.9	23.8	0.0	0.0	0.0	0.0E+00	100.0	95.2	90.2	357.6
89	12	3	14	49	0	*	1593.00	24.2	22.6	0.0	1.2	1.2	6.2E-09	99.8	94.2	89.4	336.0
89	12	4	9	47	0	*	2731.00	6.8	22.3	0.0	0.3	1.5	2.1E-09	99.6	95.1	89.9	381.3
89	12	4	16	50	0	*	3154.00	7.1	21.9	0.0	0.4	1.9	6.9E-09	99.5	95.0	89.5	401.7
89	12	5	7	36	0	*	4040.00	7.8	21.4	0.0	0.5	2.4	4.3E-09	99.8	95.1	90.2	358.3
89	12	5	17	7	0	*	4611.00	8.7	21.1	0.0	0.3	2.7	4.0E-09	98.9	95.1	89.7	392.2
89	12	6	10	20	0	*	0.00	9.7	20.6	0.0	0.0	0.0	0.0E+00	99.9	94.9	89.8	369.6
89	12	6	17	4	0	*	404.00	9.5	20.4	0.0	0.2	0.2	4.5E-09	95.9	93.3	89.7	264.1
89	12	7	8	34	0	*	1334.00	9.9	20.0	0.0	0.4	0.6	3.9E-09	100.0	94.8	89.8	361.8
89	12	8	7	45	0	*	2725.00	10.5	19.2	0.0	0.8	1.4	4.6E-09	99.4	94.3	89.4	353.4
89	12	8	13	27	0	*	3067.00	10.6	19.0	0.0	0.2	1.6	4.6E-09	100.8	95.6	90.4	374.2

Golder Associates
Permeability Test Results

Project Number: 893-1186.033
Project Description: DOE RL/HOM-DRAG-OFF/SEATTLE

TEST RESULTS: K= 2E-9 cm/second
****

Sample Number: GTP1-7 16%BENTONITE
Sample Description: Lt. olive gray, med. to fine SAND, some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final
Height:	7.177	7.126
Diameter:	7.254	7.258
Area:	41.33	41.37
Volume:	296.6	294.8

Board number: L0091
Cell number: L0007

Sample Properties :	Initial	Final
Wet Weight (g):	567.16	584.42
Moisture (%):	24.9	28.7
Wet Density (pcf):	119.4	123.8
Dry Density (pcf):	95.6	96.2
Dry Weight (g):	454.03	454.03
Specific Grav.:	2.77	2.77
Void Ratio:	0.81	0.80
Saturation (%):	85	100

Tested by: DN
Reviewed by: KRB

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Botton stone:	92.5	95.0
B - coefficient:	0.99	N.A.
Gradient:	0	49

Test Method: Flexible wall permeameter with back pressure saturation (const. head computations)
Date and time test begins: 11/30/89

yr mo da hr minsec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc) Reading	Drain	Delta	Total	K (cm/s) Delta	Transducer Reading (psi) Cell	Botton	Top	Head (cm H20)
89 12 6 7 52 0 *	0.00	9.7	22.2	0.0	0.0	0.0	0.0E+00	99.9	94.9	89.8	371.2
89 12 6 13 23 0 *	331.00	9.8	22.0	0.0	0.2	0.2	4.7E-09	99.9	94.9	89.8	370.9
89 12 6 17 5 0 *	553.00	9.5	21.9	0.0	0.1	0.3	4.1E-09	95.9	93.3	89.7	265.6
89 12 7 8 34 0 *	1482.00	9.9	21.7	0.0	0.2	0.5	2.0E-09	100.0	94.8	89.8	363.5
89 12 8 7 46 0 *	2874.00	10.5	21.5	0.0	0.2	0.7	1.2E-09	99.4	94.3	89.7	334.6
89 12 8 14 25 0 *	3273.00	1.0	21.4	0.0	0.1	0.8	2.0E-09	100.1	95.0	89.9	379.1
89 12 10 12 58 0 *	0.00	8.9	21.5	0.0	0.0	0.0	0.0E+00	101.4	96.0	91.0	364.3
89 12 11 9 6 0 *	1208.00	8.4	20.9	0.0	0.6	0.6	3.8E-09	101.4	96.2	90.8	392.3
89 12 11 14 49 0 *	1551:00	8.4	20.8	0.0	0.1	0.7	2.2E-09	100.4	94.8	89.9	357.1
89 12 12 8 0 0 *	2582.00	8.6	20.6	0.0	0.2	0.9	1.5E-09	100.6	95.0	89.9	370.7
89 12 12 16 28 0 *	3090.00	8.9	20.5	0.0	0.1	1.0	1.6E-09	99.9	94.5	89.7	349.2
89 12 13 7 39 0 *	4001.00	9.2	20.4	0.0	0.1	1.1	1.5E-09	100.1	94.7	90.6	299.5

GOLDER ASSOCIATES INC., REDMOND, WA

SIEVE ANALYSIS

PROJECT	DOE RL/RON-DRAG OFF
PROJECT NUMBER	893-1186.033
ENGINEER	T MARTIN
DATE	11-22-89
TECHNICIAN	BFW
REVIEWER	KRB

SAMPLE NUMBER	GTP1-7	GTP1-7
LOCATION	8% BENT	10%EPHRATA
DEPTH	COMPOSITE	COMPOSITE
TARE NUMBER	100	34
TARE WT (g)	78.35	73.4
WET WT + TARE (g)	415.46	316.83
DRY WT + TARE (g)	408.35	309.51
MOISTURE (%)	2.2%	3.1%

	CUMULATIVE WEIGHT (g)	PERCENT FINER		CUMULATIVE WEIGHT (g)	PERCENT FINER
TARE (g)	325.05			272.95	
3"	325.05	100.0%	3"	272.95	100.0%
2"	325.05	100.0%	2"	272.95	100.0%
1"	325.05	100.0%	1"	272.95	100.0%
3/4"	325.05	100.0%	3/4"	272.95	100.0%
3/8"	325.05	100.0%	3/8"	272.95	100.0%
#4	325.05	100.0%	#4	272.95	100.0%
#10	325.84	99.8%	#10	273.24	99.9%
#20	338.75	95.8%	#20	282.52	95.9%
#40	377.04	84.2%	#40	308.96	84.7%
#60	409.04	74.5%	#60	330.28	75.7%
#100	469.64	56.2%	#100	370.02	58.9%
#200	579.99	22.7%	#200	444.06	27.5%

APPENDIX A.2

INTERFACE SHEAR STRENGTH TESTING

Westinghouse Environmental
and Geotechnical Services, Inc.

11785 Highway Drive
Suite 100
Cincinnati, Ohio 45241
(513) 733-9374
FAX (513) 733-8213

January 22, 1990

Golder Associates, Inc.
4101-148th Avenue, N.E.
Redmond, Washington 98052

Attention: Mr. Tim Martin

Regarding: Geosynthetics Testing Services
Direct Shear Friction Testing
Golder Project Reference Number: 893-1186.033
Project No. 4147-90-104

Gentlemen:

Westinghouse Environmental and Geotechnical Services, Inc., (Westinghouse) is pleased to present the results of our testing program for the above-referenced project. Our work was authorized by Golder Purchase Order No. 10847.

The program consisted of Direct Shear Friction testing of an admix soil (sandy soil with approximately 8 percent bentonite) and 80 mil textured HDPE geomembrane (HDT) interface. The soil sample and the HDT sample were provided by Golder.

Testing was performed in accordance with the latest draft of the proposed ASTM test method using a 12-inch by 12-inch direct shear device. The upper box contained the soil specimen. Golder requested the admix to be tested at 20 to 22 percent moisture content and a dry density between 98.5 to 104 pounds per cubic foot. The dry density and moisture contents were as follows:

Normal Stress (psf)	Dry Density (pcf)	Moisture Content (%)
100	100.5	21.6
200	101.9	20.4
500	101.7	20.7

The HDT specimen was fixed to the lower shear box in order that sliding failure be constrained to the soil/HDT interface. Prior to shearing, the specimen was preloaded at the test normal compressive stress for a period of 10 minutes. This allows for more uniformity and better interface contact during shearing. The shearing rate was 0.04 in/min.

During direct shear testing, the shear force was recorded as a function of horizontal displacement of the travelling box. The maximum deflection for these tests was 2.0 inches.

The testing program consisted of tests run at normal compressive stresses of 100, 200, and 500 psf.

Plots of the applied shear stress versus horizontal displacement are presented on the attached Figure 1. The tabular data used to generate these plots are presented on Table 1. From these plots, the angle of interface friction, or the friction angle as well as the apparent adhesion were determined. For this testing program, the direct shear results are as follows and are shown graphically on Figure 2:

Peak Angle of Interface Friction: 40°
Adhesion: 425 psf

Residual Angle of Interface Friction: 36°
Adhesion: 320 psf

The direct shear data provided for this test are a function of both the HDT and soil specimen used in testing. The soil parameters and normal compressive stresses selected for the test dictated the interface shear behavior for this test. For this reason, caution is advised in applying the direct shear friction angle determined in this test to other soils or other test conditions.

We trust that this is the information you require. If you have any questions or wish to discuss the contents of this report, please contact us.

Respectfully submitted,

WESTINGHOUSE ENVIRONMENTAL AND
GEOTECHNICAL SERVICES, INC.

Paula J. Shaffer, E.I.T.
Staff Engineer

Reviewed by:

John A. Bove, P.E.
Geosynthetics Department Manager

JAB:gp

Attachments

Golder Direct Shear Friction Test

Admix vs. HDT

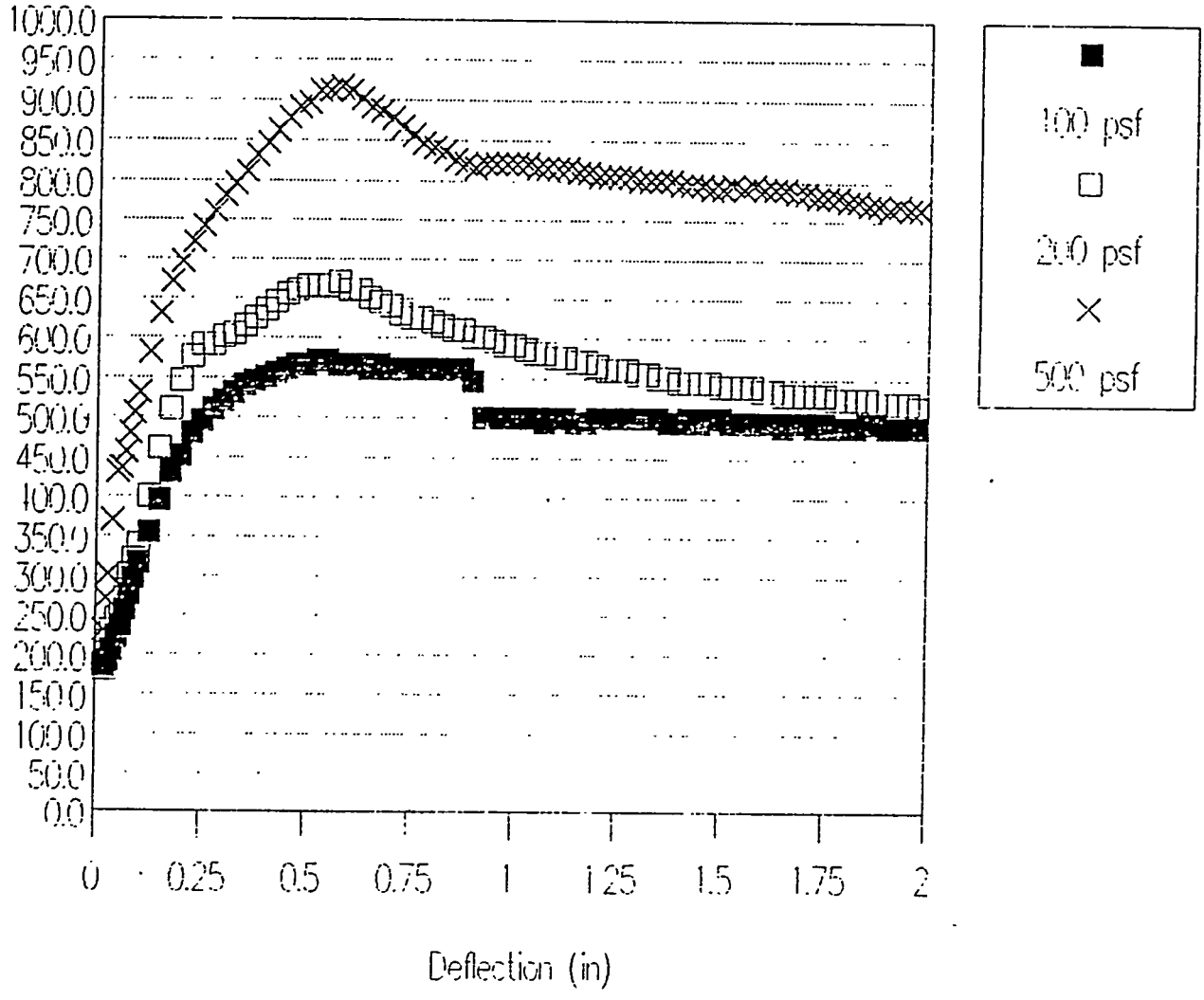
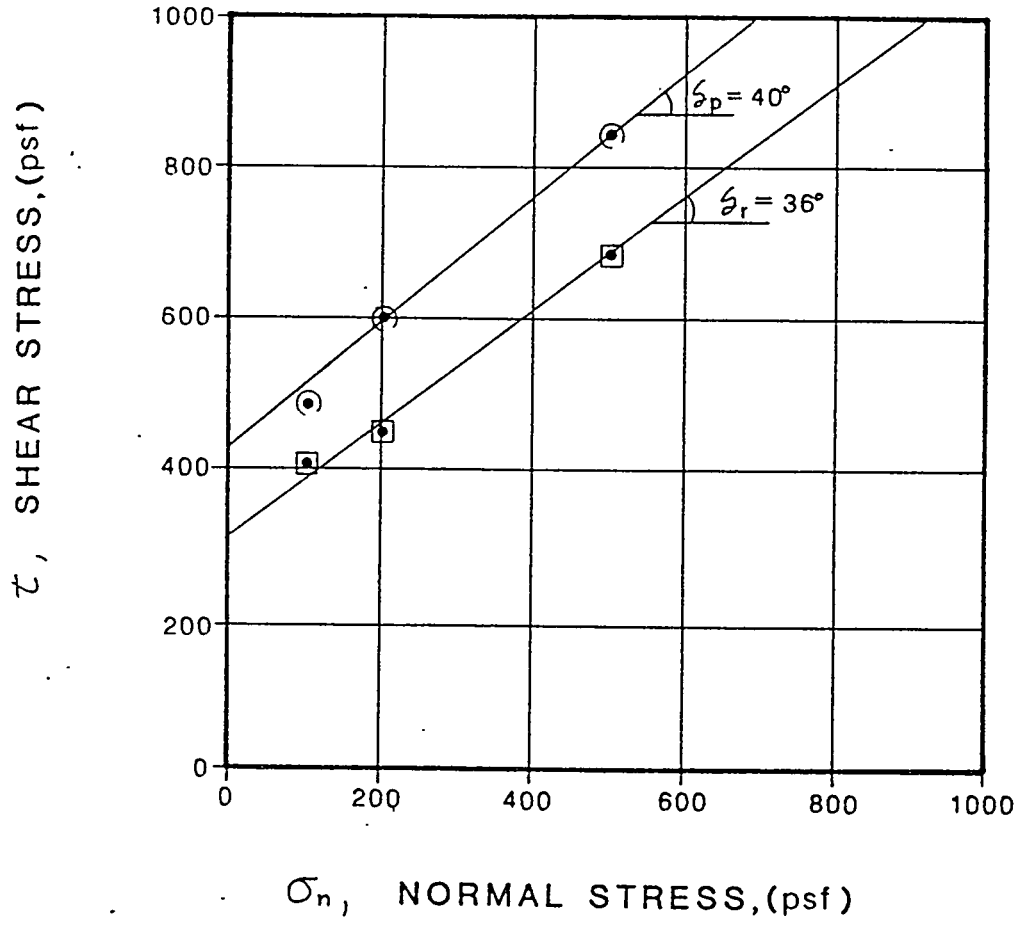


FIGURE: 1

ADMIX vs HDT GEOMEMBRANE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – ADMIX
LOWER SPECIMEN – HDT GEOMEMBRANE
SUBSTRATUM – PLATES

GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

A.2-4

Westinghouse

PROJ. NO. 4147-90-104 FIG NO. 2

W47-03

Westinghouse Environmental and Geotechnical Services
Geosynthetics Laboratory
Direct Shear Friction Test Result Summary

Cincinnati Branch
Cincinnati, Ohio 45241
(513) 733-9374
Fax No. (513) 733-8213

Date Tested: 19 DEC 89

Date of Summary: 04 JAN 90

-------- Project Identification ----------

Client: Golder Associates, Inc.
Westinghouse Project Number: 4147-90-104
Test Number: 1
Description: Admix vs. HDT
Lab Technician: FCE

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.000	0.02	10.01	0.03	15.02	0.03	15.02
0.005	0.17	85.12	0.24	120.17	0.24	120.17
0.010	0.23	115.17	0.28	140.20	0.33	165.24
0.015	0.27	135.19	0.31	155.22	0.40	200.29
0.020	0.37	185.27	0.36	180.26	0.46	230.33
0.025	0.38	190.27	0.40	200.29	0.54	270.39
0.030	0.38	190.27	0.43	215.31	0.60	300.43
0.040	0.41	205.29	0.46	230.33	0.74	370.53
0.050	0.44	220.32	0.49	245.35	0.86	430.62
0.060	0.47	235.34	0.52	260.37	0.87	435.62
0.070	0.51	255.37	0.56	280.40	0.91	455.65
0.080	0.55	275.39	0.60	300.43	0.96	480.69
0.090	0.59	295.42	0.64	320.46	1.01	505.73
0.100	0.63	315.45	0.68	340.49	1.06	530.76
0.125	0.71	355.51	0.80	400.57	1.16	580.83
0.150	0.79	395.57	0.92	460.66	1.26	630.90
0.175	0.86	430.62	1.02	510.73	1.34	670.96
0.200	0.90	450.65	1.09	545.78	1.39	696.00
0.225	0.96	480.69	1.15	575.83	1.44	721.03
0.250	0.99	495.71	1.18	590.85	1.48	741.06
0.275	1.02	510.73	1.18	590.85	1.52	761.09
0.300	1.04	520.75	1.20	600.86	1.56	781.12
0.325	1.06	530.76	1.21	605.87	1.59	796.14
0.350	1.08	540.78	1.23	615.88	1.62	811.16
0.375	1.09	545.78	1.25	625.90	1.66	831.19
0.400	1.10	550.79	1.27	635.91	1.69	846.21
0.425	1.11	555.80	1.29	645.93	1.72	861.23
0.450	1.12	560.80	1.31	655.94	1.75	876.26
0.475	1.13	565.81	1.32	660.95	1.78	891.28
0.500	1.13	565.81	1.33	665.95	1.79	896.29

TABLE 1

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.525	1.14	570.82	1.33	665.95	1.82	911.31
0.550	1.14	570.82	1.33	665.95	1.82	911.31
0.575	1.13	565.81	1.34	670.96	1.83	916.31
0.600	1.13	565.81	1.32	660.95	1.82	911.31
0.625	1.13	565.81	1.32	660.95	1.80	901.29
0.650	1.12	560.80	1.30	650.93	1.78	891.28
0.675	1.13	565.81	1.29	645.93	1.77	886.27
0.700	1.12	560.80	1.28	640.92	1.75	876.26
0.725	1.12	560.80	1.26	630.90	1.73	866.24
0.750	1.12	560.80	1.25	625.90	1.71	856.23
0.775	1.12	560.80	1.25	625.90	1.69	846.21
0.800	1.12	560.80	1.24	620.89	1.68	841.21
0.825	1.12	560.80	1.23	615.88	1.67	836.20
0.850	1.12	560.80	1.22	610.88	1.65	826.18
0.875	1.12	560.80	1.22	610.88	1.64	821.18
0.900	1.09	545.78	1.20	600.86	1.63	816.17
0.925	1.00	500.72	1.20	600.86	1.64	821.18
0.950	1.00	500.72	1.19	595.85	1.64	821.18
0.975	1.00	500.72	1.18	590.85	1.64	821.18
1.000	1.00	500.72	1.18	590.85	1.64	821.18
1.025	1.00	500.72	1.17	585.84	1.64	821.18
1.050	1.00	500.72	1.16	580.83	1.63	816.17
1.075	0.99	495.71	1.16	580.83	1.63	816.17
1.100	1.00	500.72	1.15	575.83	1.63	816.17
1.125	1.00	500.72	1.15	575.83	1.63	816.17
1.150	0.99	495.71	1.14	570.82	1.62	811.16
1.175	0.99	495.71	1.14	570.82	1.62	811.16
1.200	1.00	500.72	1.13	565.81	1.61	806.16
1.225	1.00	500.72	1.12	560.80	1.61	806.16
1.250	1.00	500.72	1.12	560.80	1.61	806.16
1.275	1.00	500.72	1.12	560.80	1.61	806.16
1.300	1.00	500.72	1.11	555.80	1.60	801.15
1.325	1.00	500.72	1.11	555.80	1.60	801.15
1.350	1.00	500.72	1.10	550.79	1.60	801.15
1.375	0.99	495.71	1.10	550.79	1.60	801.15
1.400	0.99	495.71	1.09	545.78	1.59	796.14
1.425	1.00	500.72	1.09	545.78	1.59	796.14
1.450	0.99	495.71	1.09	545.78	1.59	796.14
1.475	1.00	500.72	1.09	545.78	1.58	791.13
1.500	1.00	500.72	1.08	540.78	1.58	791.13
1.525	0.99	495.71	1.08	540.78	1.59	796.14
1.550	0.99	495.71	1.08	540.78	1.58	791.13
1.575	0.99	495.71	1.08	540.78	1.59	796.14
1.600	0.99	495.71	1.07	535.77	1.58	791.13
1.625	0.99	495.71	1.07	535.77	1.58	791.13
1.650	0.98	490.70	1.06	530.76	1.58	791.13
1.675	0.99	495.71	1.06	530.76	1.57	786.13
1.700	0.99	495.71	1.06	530.76	1.57	786.13

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
1.725	0.98	490.70	1.06	530.76	1.56	781.12
1.750	0.99	495.71	1.06	530.76	1.56	781.12
1.775	0.98	490.70	1.05	525.75	1.56	781.12
1.800	0.98	490.70	1.05	525.75	1.55	776.11
1.825	0.98	490.70	1.05	525.75	1.55	776.11
1.850	0.99	495.71	1.05	525.75	1.54	771.11
1.875	0.99	495.71	1.04	520.75	1.54	771.11
1.900	0.98	490.70	1.04	520.75	1.53	766.10
1.925	0.98	490.70	1.04	520.75	1.54	771.11
1.950	0.98	490.70	1.04	520.75	1.53	766.10
1.975	0.98	490.70	1.03	515.74	1.53	766.10
2.000	0.99	495.71	1.04	520.75	1.53	766.10

Filename: DS90104A
Source: Q21

Westinghouse Environmental
and Geotechnical Services, Inc.

11785 Highway Drive
Suite 100
Cincinnati, Ohio 45241
(513) 733-9374
FAX (513) 733-8213

January 22, 1990

Golder Associates, Inc.
4104 - 148th Avenue NE
Redmond, WA 98052

Attention: Mr. Tim Martin

Regarding: Geosynthetics Testing Services
Direct Shear Friction Testing
Golder Project Reference Number: 893-1186.033
Project No. 4147-90-104

Gentlemen:

Westinghouse Environmental and Geotechnical Services, Inc., (Westinghouse) is pleased to present the results of our testing program for the above-referenced project. Our work was authorized by Golder Purchase Order No. 10877.

The program consisted of Direct Shear Friction testing of TEXNET and 80 mil textured HDPE geomembrane (HDT) interface. The TEXNET and HDT sample were provided by Golder.

Testing was performed in accordance with the latest draft of the proposed ASTM test method using a 12-inch by 12-inch direct shear device.

The HDT specimen was fixed to the lower shear box and the TEXNET was fixed to the upper shear box in order that sliding failure be constrained to the TEXNET/HDT interface. Prior to shearing, the TEXNET was wet uniformly across the specimen for the wet interface tests.

During direct shear testing, the shear force was recorded as a function of horizontal displacement of the travelling box. The maximum deflection for these tests was 2.0 inches. The shearing rate was 0.04 in./min. The tests were run at normal compressive stresses of 100, 200 and 500 psf.

Plots of the applied shear stress versus horizontal displacement are presented on the attached Figures 1 and 2 for the wet and dry interface tests respectively. The tabular data used to generate these plots are presented on Tables 1 and 2. From these plots, the angle of interface friction, or the friction angle as well as the apparent adhesion were determined. For this testing program, the direct shear results are as follows and are shown graphically on Figure 3:

Wet Interface: Angle of Interface Friction: 22^{o}
Adhesion: 188 psf

Dry Interface: Angle of Interface Friction: 28^{o}
Adhesion: 180 psf

A Westinghouse Electric Corporation subsidiary.

The plots presented on Figures 1 and 2 exhibit no distinct peak and no classical residual behavior. This shape of curve is indicative of a specimen with a low tensile modulus material (such as the geotextile bonded to the TEXNET). The fairly good contact between the HDT and geotextile surfaces caused the geotextile to experience shear strains during shearing. The bond between the geotextile and the geonet was good, therefore the geocomposite did not significantly elongate during testing. After each test was completed, the specimens were carefully examined. In each case, geotextile fibers were examined within the textured HDT surface, and the geotextile specimens were frayed, but not torn. This supports the Shear Stress vs. Deflection behavior presented in Figures 1 and 2.

The direct shear data provided for this project are a function of both the HDT and TEXNET specimens used in testing. The specimen parameters and normal compressive stresses selected for the test dictated the interface shear behavior for this test. For this reason, caution is advised in applying the direct shear friction angle determined in this test to other test conditions.

We trust that this is the information you require. If you have any questions or wish to discuss the contents of this report, please contact us.

Respectfully submitted,

WESTINGHOUSE ENVIRONMENTAL AND
GEOTECHNICAL SERVICES, INC.

Paula J. Shaffer

Paula J. Shaffer, E.I.T.
Staff Engineer

Reviewed by:

John A. Bove/DM

John A. Bove, P.E.
Geosynthetics Department Manager

JAB:rp

Attachments

Golder Direct Shear Friction Test

Texnet (Wet) vs. HDT

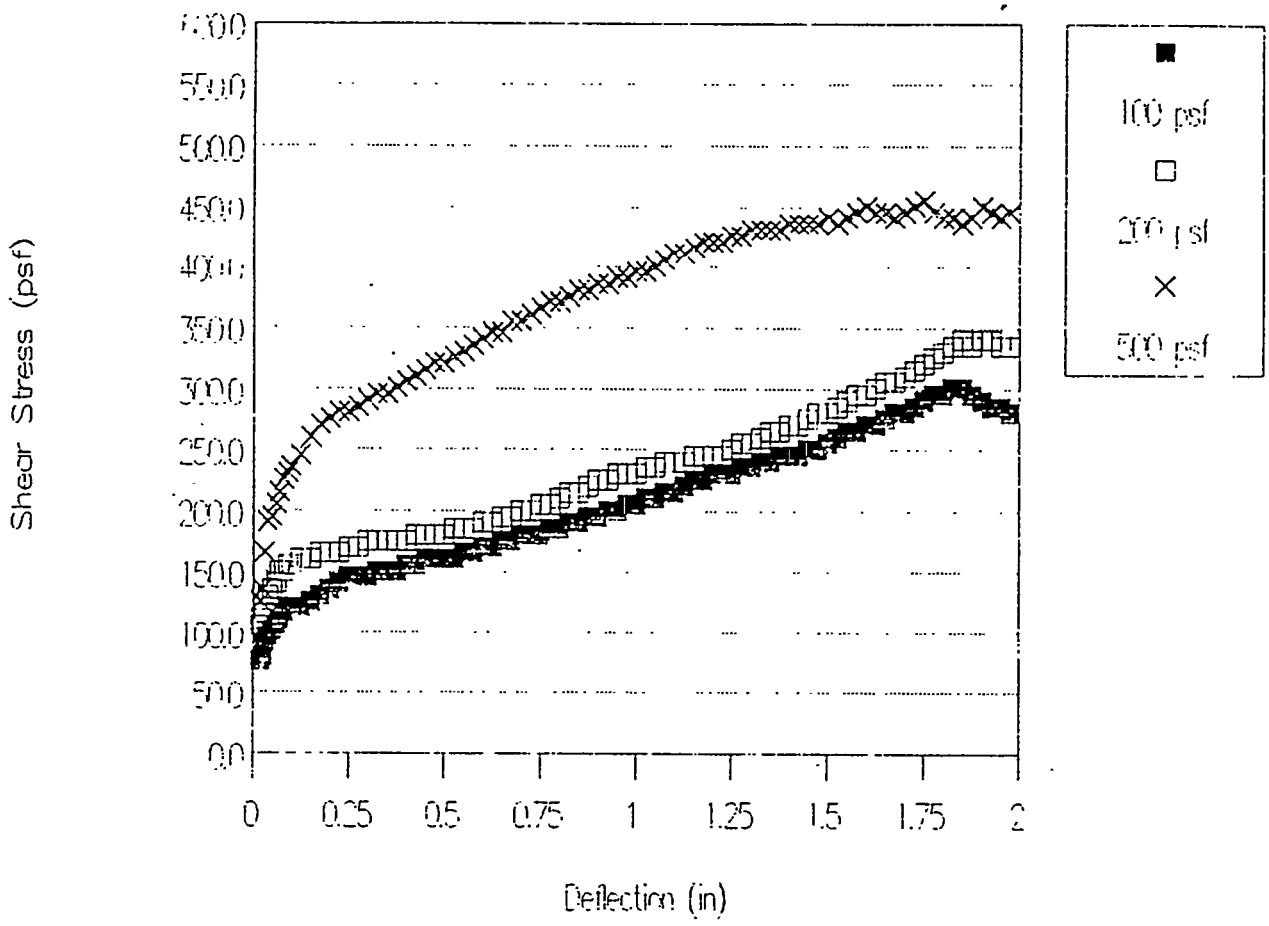


Figure 1

Westinghouse Environmental and Geotechnical Services
Geosynthetics Laboratory
Direct Shear Friction Test Result Summary

Cincinnati Branch
Cincinnati, Ohio 45241
(513) 733-9374
Fax No. (513) 733-8213

Date Tested: 15 JAN 90

Date of Summary: 15 JAN 90

Project Identification

Client: Golder Associates, Inc.
Westinghouse Project Number: 4147-90-104
Test Number: 3
Description: HDT vs. Texnet
Lab Technician: FCE

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.000	0.06	30.04	0.07	35.05	0.06	30.04
0.005	0.12	60.09	0.16	80.11	0.16	80.11
0.010	0.13	65.09	0.18	90.13	0.20	100.14
0.015	0.14	70.10	0.19	95.14	0.22	110.16
0.020	0.15	75.11	0.21	105.15	0.25	125.18
0.025	0.16	80.11	0.22	110.16	0.27	135.19
0.030	0.18	90.13	0.23	115.17	0.33	165.24
0.040	0.19	95.14	0.26	130.19	0.38	190.27
0.050	0.20	100.14	0.27	135.19	0.39	195.28
0.060	0.21	105.15	0.28	140.20	0.41	205.29
0.070	0.22	110.16	0.30	150.22	0.43	215.31
0.080	0.23	115.17	0.30	150.22	0.45	225.32
0.090	0.24	120.17	0.31	155.22	0.46	230.33
0.100	0.24	120.17	0.31	155.22	0.47	235.34
0.125	0.24	120.17	0.32	160.23	0.49	245.35
0.150	0.25	125.18	0.32	160.23	0.52	260.37
0.175	0.26	130.19	0.33	165.24	0.54	270.39
0.200	0.27	135.19	0.33	165.24	0.55	275.39
0.225	0.28	140.20	0.33	165.24	0.56	280.40
0.250	0.29	145.21	0.34	170.24	0.56	280.40
0.275	0.29	145.21	0.34	170.24	0.57	285.41
0.300	0.29	145.21	0.35	175.25	0.58	290.42
0.325	0.30	150.22	0.35	175.25	0.59	295.42
0.350	0.30	150.22	0.35	175.25	0.59	295.42
0.375	0.30	150.22	0.35	175.25	0.60	300.43
0.400	0.31	155.22	0.35	175.25	0.61	305.44
0.425	0.31	155.22	0.36	180.26	0.62	310.45
0.450	0.32	160.23	0.36	180.26	0.63	315.45
0.475	0.32	160.23	0.36	180.26	0.64	320.46
0.500	0.32	160.23	0.36	180.26	0.64	320.46

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.525	0.32	160.23	0.37	185.27	0.65	325.47
0.550	0.33	165.24	0.37	185.27	0.66	330.47
0.575	0.33	165.24	0.37	185.27	0.67	335.48
0.600	0.34	170.24	0.38	190.27	0.68	340.49
0.625	0.34	170.24	0.38	190.27	0.69	345.50
0.650	0.35	175.25	0.39	195.28	0.69	345.50
0.675	0.35	175.25	0.39	195.28	0.71	355.51
0.700	0.36	180.26	0.40	200.29	0.71	355.51
0.725	0.36	180.26	0.40	200.29	0.72	360.52
0.750	0.36	180.26	0.41	205.29	0.73	365.52
0.775	0.37	185.27	0.41	205.29	0.74	370.53
0.800	0.37	185.27	0.42	210.30	0.74	370.53
0.825	0.38	190.27	0.43	215.31	0.75	375.54
0.850	0.38	190.27	0.43	215.31	0.76	380.55
0.875	0.39	195.28	0.44	220.32	0.76	380.55
0.900	0.39	195.28	0.45	225.32	0.77	385.55
0.925	0.40	200.29	0.45	225.32	0.77	385.55
0.950	0.40	200.29	0.46	230.33	0.78	390.56
0.975	0.41	205.29	0.46	230.33	0.78	390.56
1.000	0.41	205.29	0.46	230.33	0.79	395.[illegible]
1.025	0.42	210.30	0.47	235.34	0.79	395.[illegible]
1.050	0.42	210.30	0.47	235.34	0.80	400.57
1.075	0.43	215.31	0.48	240.34	0.81	405.58
1.100	0.43	215.31	0.48	240.34	0.82	410.59
1.125	0.44	220.32	0.48	240.34	0.82	410.59
1.150	0.45	225.32	0.49	245.35	0.83	415.60
1.175	0.45	225.32	0.49	245.35	0.84	420.60
1.200	0.46	230.33	0.49	245.35	0.84	420.60
1.225	0.46	230.33	0.49	245.35	0.84	420.60
1.250	0.46	230.33	0.50	250.36	0.85	425.61
1.275	0.47	235.34	0.51	255.37	0.85	425.61
1.300	0.47	235.34	0.51	255.37	0.86	430.62
1.325	0.48	240.34	0.52	260.37	0.86	430.62
1.350	0.48	240.34	0.53	265.38	0.86	430.62
1.375	0.49	245.35	0.53	265.38	0.86	430.62
1.400	0.49	245.35	0.54	270.39	0.87	435.62
1.425	0.49	245.35	0.54	270.39	0.87	435.62
1.450	0.50	250.36	0.55	275.39	0.87	435.62
1.475	0.50	250.36	0.56	280.40	0.87	435.62
1.500	0.51	255.37	0.56	280.40	0.88	440.63
1.525	0.52	260.37	0.57	285.41	0.87	435.62
1.550	0.53	265.38	0.58	290.42	0.88	440.63
1.575	0.53	265.38	0.59	295.42	0.89	445.64
1.600	0.54	270.39	0.59	295.42	0.90	450.65
1.625	0.54	270.39	0.60	300.43	0.89	445.64
1.650	0.55	275.39	0.61	305.44	0.89	445.6[illegible]
1.675	0.56	280.40	0.61	305.44	0.88	440.6[illegible]
1.700	0.56	280.40	0.62	310.45	0.89	445.64

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
1.725	0.57	285.41	0.63	315.45	0.90	450.65
1.750	0.58	290.42	0.64	320.46	0.91	455.65
1.775	0.59	295.42	0.65	325.47	0.89	445.64
1.800	0.59	295.42	0.66	330.47	0.88	440.63
1.825	0.60	300.43	0.67	335.48	0.88	440.63
1.850	0.60	300.43	0.68	340.49	0.87	435.62
1.875	0.59	295.42	0.67	335.48	0.88	440.63
1.900	0.58	290.42	0.68	340.49	0.90	450.65
1.925	0.57	285.41	0.68	340.49	0.89	445.64
1.950	0.57	285.41	0.67	335.48	0.88	440.63
1.975	0.56	280.40	0.67	335.48	0.89	445.64
2.000	0.57	285.41	0.67	335.48	0.88	440.63

Filename: DS90104A
Source: Q22

Golder Direct Shear Friction Test

Texnet vs. HDT

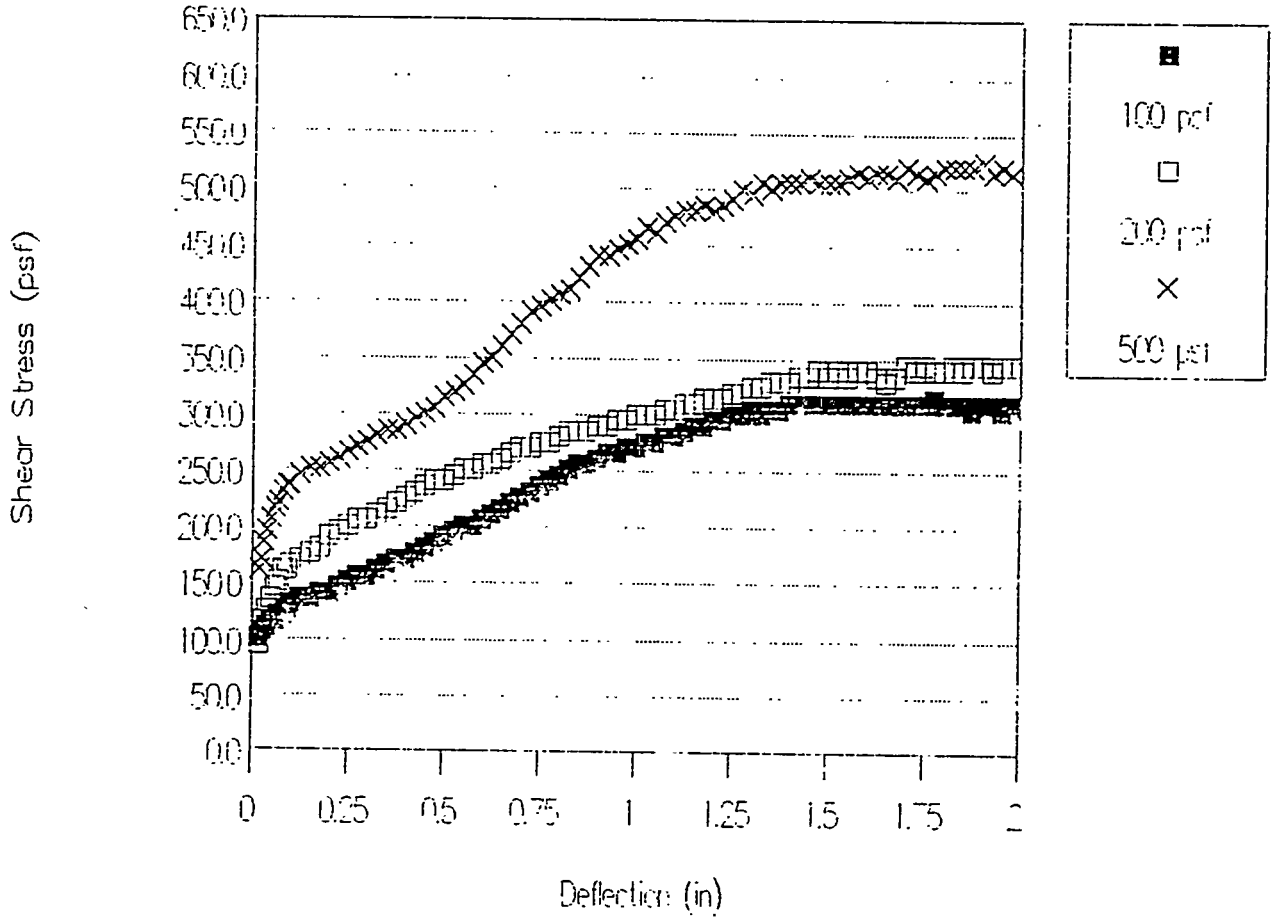


FIGURE 2

Westinghouse Environmental and Geotechnical Services
Geosynthetics Laboratory
Direct Shear Friction Test Result Summary

Cincinnati Branch
Cincinnati, Ohio 45241
(513) 733-9374
Fax No. (513) 733-8213

Date Tested: 11 JAN 90

Date of Summary: 12 JAN 90

Project Identification
Client: Golder Associates, Inc.
Westinghouse Project Number: 4147-90-104
Test Number: 3
Description: HDT vs. Texnet
Lab Technician: FCE

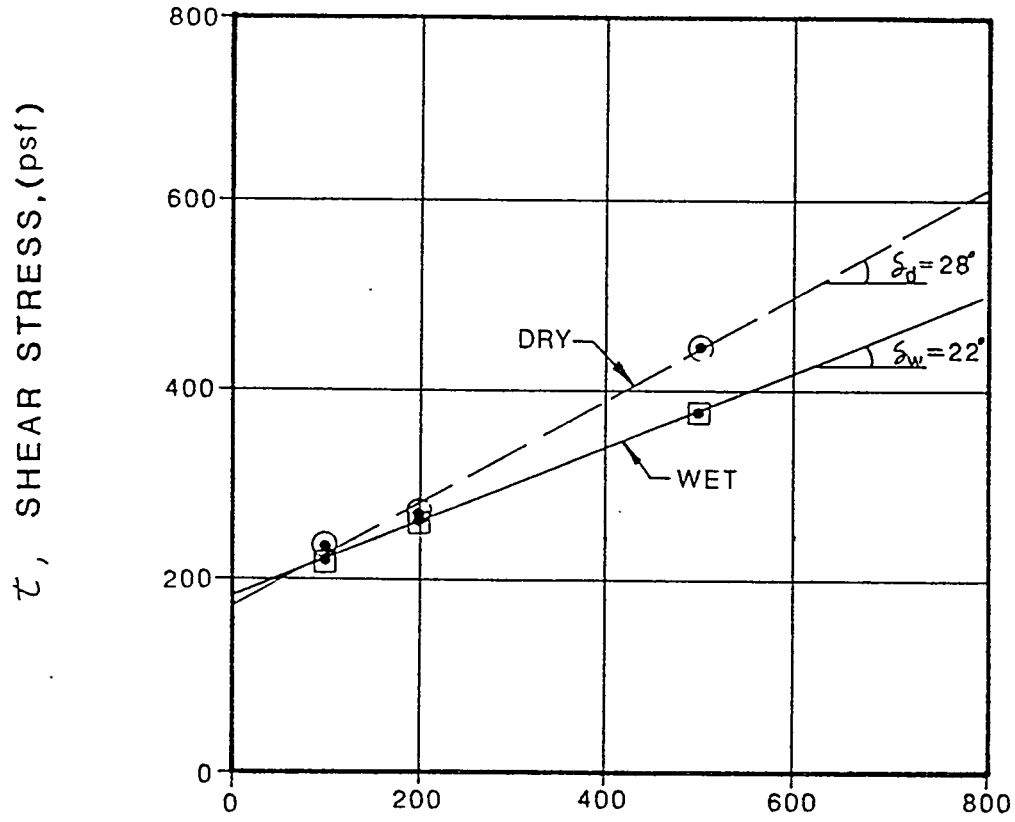
Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.000	0.06	30.04	0.06	30.04	0.06	30.04
0.005	0.16	80.11	0.14	70.10	0.20	100.14
0.010	0.18	90.13	0.15	75.11	0.25	125.18
0.015	0.19	95.14	0.18	90.13	0.29	145.21
0.020	0.20	100.14	0.19	95.14	0.32	160.23
0.025	0.21	105.15	0.21	105.15	0.34	170.24
0.030	0.21	105.15	0.23	115.17	0.37	185.27
0.040	0.22	110.16	0.25	125.18	0.39	195.28
0.050	0.23	115.17	0.27	135.19	0.41	205.29
0.060	0.23	115.17	0.29	145.21	0.43	215.31
0.070	0.24	120.17	0.30	150.22	0.44	220.32
0.080	0.24	120.17	0.32	160.23	0.46	230.33
0.090	0.25	125.18	0.32	160.23	0.47	235.34
0.100	0.26	130.19	0.33	165.24	0.47	235.34
0.125	0.27	135.19	0.34	170.24	0.49	245.35
0.150	0.27	135.19	0.35	175.25	0.50	250.36
0.175	0.28	140.20	0.36	180.26	0.50	250.36
0.200	0.28	140.20	0.38	190.27	0.51	255.37
0.225	0.29	145.21	0.39	195.28	0.52	260.37
0.250	0.30	150.22	0.40	200.29	0.53	265.38
0.275	0.31	155.22	0.41	205.29	0.54	270.39
0.300	0.31	155.22	0.41	205.29	0.55	275.39
0.325	0.32	160.23	0.42	210.30	0.56	280.40
0.350	0.33	165.24	0.43	215.31	0.57	285.41
0.375	0.34	170.24	0.44	220.32	0.57	285.41
0.400	0.34	170.24	0.45	225.32	0.58	290.42
0.425	0.35	175.25	0.46	230.33	0.59	295.42
0.450	0.36	180.26	0.47	235.34	0.60	300.43
0.475	0.37	185.27	0.48	240.34	0.61	305.44
0.500	0.38	190.27	0.48	240.34	0.63	315.45

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.525	0.39	195.28	0.49	245.35	0.64	320.46
0.550	0.40	200.29	0.50	250.36	0.65	325.47
0.575	0.40	200.29	0.50	250.36	0.67	335.48
0.600	0.41	205.29	0.51	255.37	0.69	345.50
0.625	0.42	210.30	0.51	255.37	0.70	350.50
0.650	0.43	215.31	0.52	260.37	0.72	360.52
0.675	0.44	220.32	0.53	265.38	0.74	370.53
0.700	0.45	225.32	0.54	270.39	0.76	380.55
0.725	0.46	230.33	0.54	270.39	0.78	390.56
0.750	0.47	235.34	0.55	275.39	0.79	395.57
0.775	0.48	240.34	0.55	275.39	0.80	400.57
0.800	0.49	245.35	0.56	280.40	0.81	405.58
0.825	0.50	250.36	0.57	285.41	0.82	410.59
0.850	0.51	255.37	0.57	285.41	0.84	420.60
0.875	0.51	255.37	0.57	285.41	0.86	430.62
0.900	0.52	260.37	0.58	290.42	0.88	440.63
0.925	0.53	265.38	0.58	290.42	0.88	440.63
0.950	0.53	265.38	0.59	295.42	0.89	445.64
0.975	0.54	270.39	0.59	295.42	0.90	450.65
1.000	0.54	270.39	0.60	300.43	0.91	455.65
1.025	0.55	275.39	0.60	300.43	0.93	465.6
1.050	0.55	275.39	0.60	300.43	0.92	460.66
1.075	0.56	280.40	0.61	305.44	0.94	470.67
1.100	0.56	280.40	0.61	305.44	0.95	475.68
1.125	0.57	285.41	0.62	310.45	0.96	480.69
1.150	0.58	290.42	0.62	310.45	0.96	480.69
1.175	0.58	290.42	0.63	315.45	0.97	485.70
1.200	0.59	295.42	0.63	315.45	0.96	480.69
1.225	0.59	295.42	0.63	315.45	0.97	485.70
1.250	0.60	300.43	0.64	320.46	0.98	490.70
1.275	0.60	300.43	0.64	320.46	1.00	500.72
1.300	0.61	305.44	0.65	325.47	0.99	495.71
1.325	0.60	300.43	0.65	325.47	1.01	505.73
1.350	0.61	305.44	0.66	330.47	1.00	500.72
1.375	0.61	305.44	0.66	330.47	1.01	505.73
1.400	0.62	310.45	0.66	330.47	1.01	505.73
1.425	0.62	310.45	0.67	335.48	1.01	505.73
1.450	0.62	310.45	0.67	335.48	1.02	510.73
1.475	0.62	310.45	0.68	340.49	1.01	505.73
1.500	0.62	310.45	0.67	335.48	1.01	505.73
1.525	0.62	310.45	0.68	340.49	1.01	505.73
1.550	0.62	310.45	0.67	335.48	1.02	510.73
1.575	0.62	310.45	0.68	340.49	1.03	515.74
1.600	0.62	310.45	0.68	340.49	1.02	510.73
1.625	0.62	310.45	0.67	335.48	1.03	515.74
1.650	0.62	310.45	0.66	330.47	1.03	515.74
1.675	0.62	310.45	0.67	335.48	1.02	510.73
1.700	0.62	310.45	0.69	345.50	1.04	520.75

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
1.725	0.62	310.45	0.68	340.49	1.03	515.74
1.750	0.62	310.45	0.69	345.50	1.02	510.73
1.775	0.63	315.45	0.68	340.49	1.03	515.74
1.800	0.62	310.45	0.68	340.49	1.04	520.75
1.825	0.62	310.45	0.69	345.50	1.04	520.75
1.850	0.62	310.45	0.68	340.49	1.04	520.75
1.875	0.61	305.44	0.69	345.50	1.04	520.75
1.900	0.62	310.45	0.69	345.50	1.05	525.75
1.925	0.62	310.45	0.68	340.49	1.03	515.74
1.950	0.61	305.44	0.69	345.50	1.04	520.75
1.975	0.62	310.45	0.69	345.50	1.03	515.74
2.000	0.62	310.45	0.68	340.49	1.04	520.75

Filename: DS90104A
Source: Q22

TEXNET GEOCOMPOSITE vs. HDT GEOMEMBRANE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – TEXNET GEOCOMPOSITE
LOWER SPECIMEN – HDT GEOMEMBRANE
SUBSTRATUM – PLATES

GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

A.2-18

Westinghouse

PROJ NO 4147-90-104 FIG NO. 3

W47-03

Westinghouse Environmental
and Geotechnical Services, Inc.

11785 Highway Drive
Suite 100
Cincinnati, Ohio 45241
(513) 733-9374
FAX (513) 733-8213

January 22, 1990

Golder Associates, Inc.
4101-148th Avenue, N.E.
Redmond, Washington 98052

Attention: Mr. Tim Martin

Regarding: Geosynthetics Testing Services
Direct Shear Friction Testing
Golder Project Reference Number: 893-1186.033
Project No. 4147-90-104

Gentlemen:

Westinghouse Environmental and Geotechnical Services, Inc., (Westinghouse) is pleased to present the results of our testing program for the above-referenced project. Our work was authorized by Golder Purchase Order No. 10847.

The program consisted of Direct Shear Friction testing of a sandy soil and TEXNET (a geotextile bonded to a geonet manufactured by National Seal Company) interface. The soil sample and the TEXNET sample were provided by Golder.

Testing was performed in accordance with the latest draft of the proposed ASTM test method using a 12-inch by 12-inch direct shear device. The upper box contained the soil specimen. Golder requested the sandy soil to be tested at 15 to 20 percent moisture content and a dry density between 87 to 92 pounds per cubic foot. The dry densities and moisture contents for the testing were as follows:

Normal Stress (psf)	Dry Density (pcf)	Moisture Content (%)
100	98.4	17.8
200	99.5	17.8
500	98.5	19.0

The TEXNET specimen was fixed to the lower shear box with the side labeled "TOP" by Golder facing against the soil in order that sliding failure be constrained to the soil/TEXNET interface. The TEXNET was tested parallel to the roll direction in the bonded zone. Prior to shearing, the specimen was preloaded at the test normal compressive stress for a period of 10 minutes. This allows for more uniformity and better interface contact during shearing. The shearing rate was 0.04 in/min.

During direct shear testing, the shear force was recorded as a function of horizontal displacement of the travelling box. The maximum deflection for these tests was 2.0 inches.

A Westinghouse Electric Corporation subsidiary.

The testing program consisted of tests run at normal compressive stresses of 100, 200, and 500 psf.

Plots of the applied shear stress versus horizontal displacement are presented on the attached Figure 1. The tabular data used to generate these plots is presented on Table 1. From these plots, the angle of interface friction, or the friction angle as well as the apparent adhesion were determined. For this testing program, the direct shear results are as follows and are shown graphically on Figure 2:

Peak and Residual Angle of Interface Friction: 30°

The friction angle was determined by the solid, straight line. The dashed line is the curve produced by the normal stresses selected. It appears that the adhesion between the TEXNET/soil interface is approaching zero at the lower normal stresses.

The direct shear data provided for this test are a function of both the TEXNET and soil specimen used in testing. The soil parameters and normal compressive stresses selected for the test dictated the interface shear behavior for this test. For this reason, caution is advised in applying the direct shear friction angle determined in this test to other soils or test conditions.

We trust that this is the information you require. If you have any questions or wish to discuss the contents of this report, please contact us.

Respectfully submitted,

WESTINGHOUSE ENVIRONMENTAL AND
GEOTECHNICAL SERVICES, INC.

Paula J. Shaffer, E.I.T.
Staff Engineer

Reviewed by:

John A. Bove, P.E.
Geosynthetics Department Manager

JAB:gp

Attachments

Golder Direct Shear Friction Test

Sand vs. Texnet

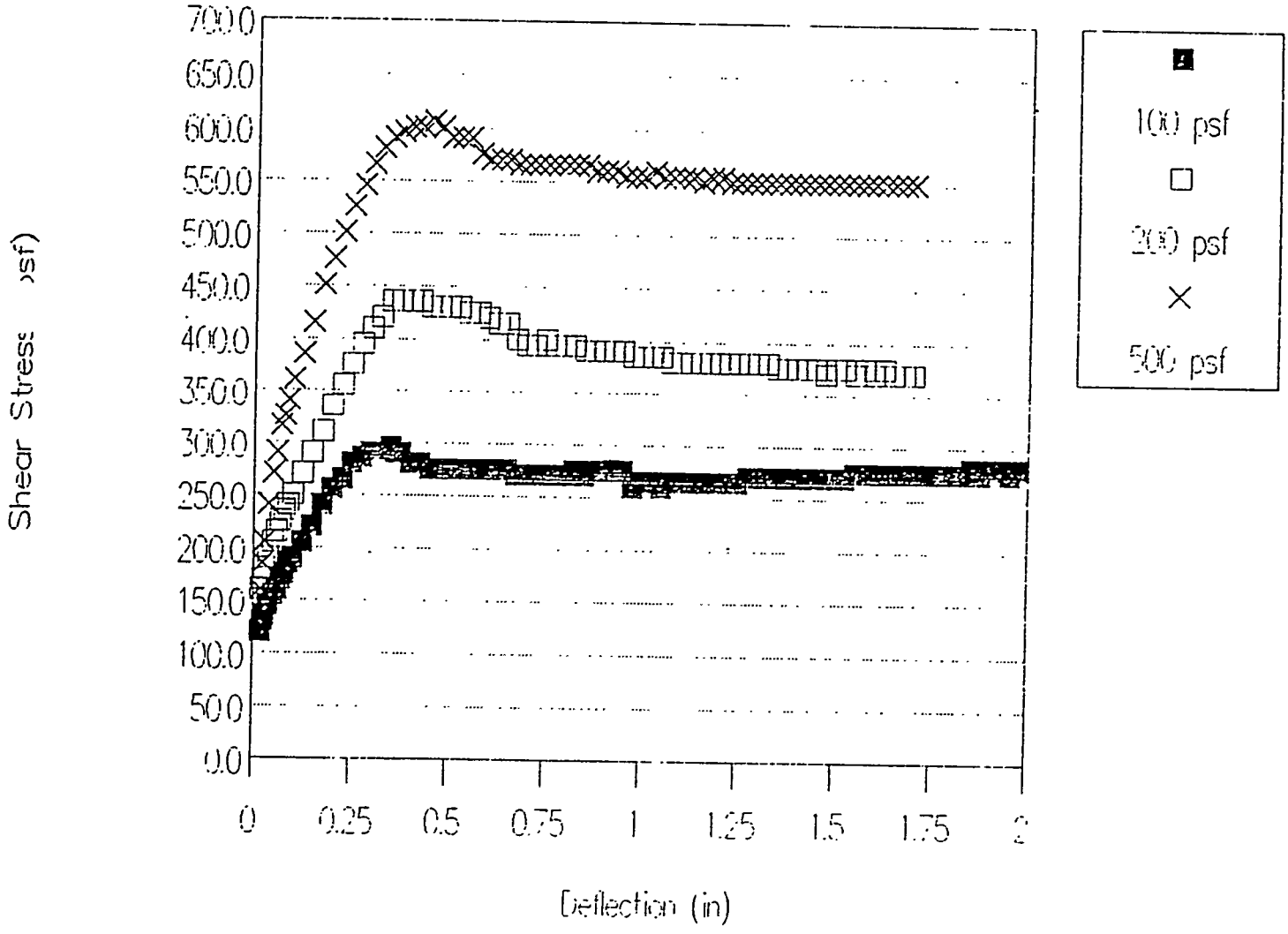
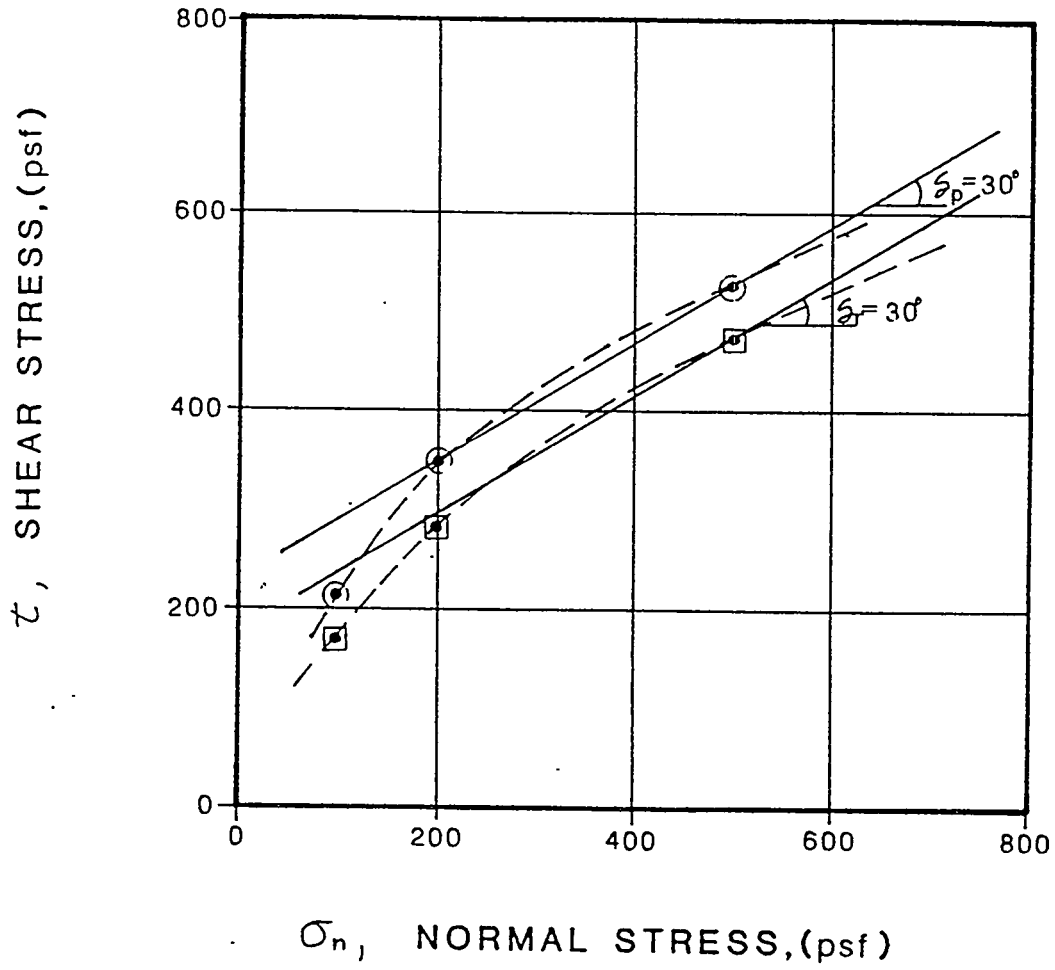


FIGURE: 1

SANDY SOIL vs TEXNET GEOCOMPOSITE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – SANDY SOIL
LOWER SPECIMEN – TEXNET GEOCOMPOSITE
SUBSTRATUM – PLATES

GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

A.2-22

Westinghouse

PROJ. NO. 4147-90-104 FIG. NO 2

W67-003

Westinghouse Environmental and Geotechnical Services
Geosynthetics Laboratory
Direct Shear Friction Test Result Summary

Cincinnati Branch
Cincinnati, Ohio 45241
(513) 733-9374
Fax No. (513) 733-8213

Date Tested: 21 DEC 89

Date of Summary: 04 JAN 90

-------- Project Identification ----------

Client: Golder Associates, Inc.
Westinghouse Project Number: 4147-90-104
Test Number: 2
Description: Sand vs. Texnet
Lab Technician: FCE

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.000	0.07	35.05	0.07	35.05	0.06	30.04
0.005	0.18	90.13	0.20	100.14	0.19	95.14
0.010	0.20	100.14	0.25	125.18	0.24	120.17
0.015	0.23	115.17	0.29	145.21	0.29	145.21
0.020	0.24	120.17	0.32	160.23	0.32	160.23
0.025	0.26	130.19	0.34	170.24	0.37	185.27
0.030	0.27	135.19	0.36	180.26	0.41	205.29
0.040	0.29	145.21	0.38	190.27	0.48	240.34
0.050	0.31	155.22	0.41	205.29	0.54	270.39
0.060	0.32	160.23	0.43	215.31	0.58	290.42
0.070	0.34	170.24	0.45	225.32	0.63	315.45
0.080	0.35	175.25	0.47	235.34	0.65	325.47
0.090	0.37	185.27	0.48	240.34	0.68	340.49
0.100	0.38	190.27	0.50	250.36	0.72	360.52
0.125	0.41	205.29	0.54	270.39	0.77	385.55
0.150	0.44	220.32	0.58	290.42	0.83	415.60
0.175	0.48	240.34	0.62	310.45	0.90	450.65
0.200	0.51	255.37	0.67	335.48	0.95	475.68
0.225	0.53	265.38	0.71	355.51	1.00	500.72
0.250	0.56	280.40	0.75	375.54	1.05	525.75
0.275	0.57	285.41	0.79	395.57	1.09	545.78
0.300	0.58	290.42	0.82	410.59	1.13	565.81
0.325	0.58	290.42	0.84	420.60	1.16	580.83
0.350	0.59	295.42	0.87	435.62	1.18	590.85
0.375	0.58	290.42	0.87	435.62	1.19	595.85
0.400	0.56	280.40	0.87	435.62	1.20	600.86
0.425	0.56	280.40	0.87	435.62	1.20	600.86
0.450	0.55	275.39	0.86	430.62	1.21	605.87
0.475	0.55	275.39	0.86	430.62	1.20	600.86
0.500	0.55	275.39	0.86	430.62	1.18	590.85

Table 1

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
0.525	0.55	275.39	0.86	430.62	1.18	590.85
0.550	0.55	275.39	0.85	425.61	1.18	590.85
0.575	0.55	275.39	0.85	425.61	1.15	575.83
0.600	0.55	275.39	0.84	420.60	1.14	570.82
0.625	0.55	275.39	0.83	415.60	1.14	570.82
0.650	0.55	275.39	0.83	415.60	1.14	570.82
0.675	0.54	270.39	0.80	400.57	1.13	565.81
0.700	0.54	270.39	0.79	395.57	1.13	565.81
0.725	0.54	270.39	0.79	395.57	1.13	565.81
0.750	0.54	270.39	0.80	400.57	1.13	565.81
0.775	0.54	270.39	0.79	395.57	1.13	565.81
0.800	0.54	270.39	0.79	395.57	1.13	565.81
0.825	0.55	275.39	0.79	395.57	1.13	565.81
0.850	0.54	270.39	0.78	390.56	1.13	565.81
0.875	0.55	275.39	0.78	390.56	1.12	560.80
0.900	0.55	275.39	0.78	390.56	1.12	560.80
0.925	0.55	275.39	0.78	390.56	1.12	560.80
0.950	0.55	275.39	0.78	390.56	1.11	555.80
0.975	0.52	260.37	0.77	385.55	1.11	555.80
1.000	0.53	265.38	0.77	385.55	1.11	555.8
1.025	0.53	265.38	0.77	385.55	1.12	560.80
1.050	0.52	260.37	0.77	385.55	1.11	555.80
1.075	0.53	265.38	0.76	380.55	1.11	555.80
1.100	0.53	265.38	0.76	380.55	1.11	555.80
1.125	0.53	265.38	0.76	380.55	1.11	555.80
1.150	0.53	265.38	0.76	380.55	1.10	550.79
1.175	0.53	265.38	0.76	380.55	1.11	555.80
1.200	0.53	265.38	0.76	380.55	1.11	555.80
1.225	0.53	265.38	0.76	380.55	1.10	550.79
1.250	0.53	265.38	0.76	380.55	1.10	550.79
1.275	0.54	270.39	0.76	380.55	1.10	550.79
1.300	0.54	270.39	0.76	380.55	1.10	550.79
1.325	0.54	270.39	0.76	380.55	1.10	550.79
1.350	0.54	270.39	0.75	375.54	1.10	550.79
1.375	0.54	270.39	0.75	375.54	1.10	550.79
1.400	0.54	270.39	0.75	375.54	1.10	550.79
1.425	0.54	270.39	0.75	375.54	1.10	550.79
1.450	0.54	270.39	0.75	375.54	1.10	550.79
1.475	0.54	270.39	0.74	370.53	1.10	550.79
1.500	0.54	270.39	0.75	375.54	1.10	550.79
1.525	0.54	270.39	0.75	375.54	1.10	550.79
1.550	0.55	275.39	0.74	370.53	1.10	550.79
1.575	0.55	275.39	0.75	375.54	1.10	550.79
1.600	0.55	275.39	0.74	370.53	1.10	550.79
1.625	0.55	275.39	0.75	375.54	1.10	550.79
1.650	0.55	275.39	0.74	370.53	1.10	550.79
1.675	0.55	275.39	0.74	370.53	1.10	550.79
1.700	0.55	275.39	0.74	370.53	1.10	550.79

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
1.725	0.55	275.39				
1.750	0.55	275.39				
1.775	0.55	275.39				
1.800	0.55	275.39				
1.825	0.55	275.39				
1.850	0.56	280.40				
1.875	0.55	275.39				
1.900	0.56	280.40				
1.925	0.56	280.40				
1.950	0.55	275.39				
1.975	0.56	280.40				
2.000	0.56	280.40				

Filename: DS90104B
Source: Q21

Normal Stress -->	100 psf		200 psf		500 psf	
Def'l. (in)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)	Volts	Shear Stress (psf)
1.725	0.55	275.39				
1.750	0.55	275.39				
1.775	0.55	275.39				
1.800	0.55	275.39				
1.825	0.55	275.39				
1.850	0.56	280.40				
1.875	0.55	275.39				
1.900	0.56	280.40				
1.925	0.56	280.40				
1.950	0.55	275.39				
1.975	0.56	280.40				
2.000	0.56	280.40				

Filename: DS90104B
Source: Q21

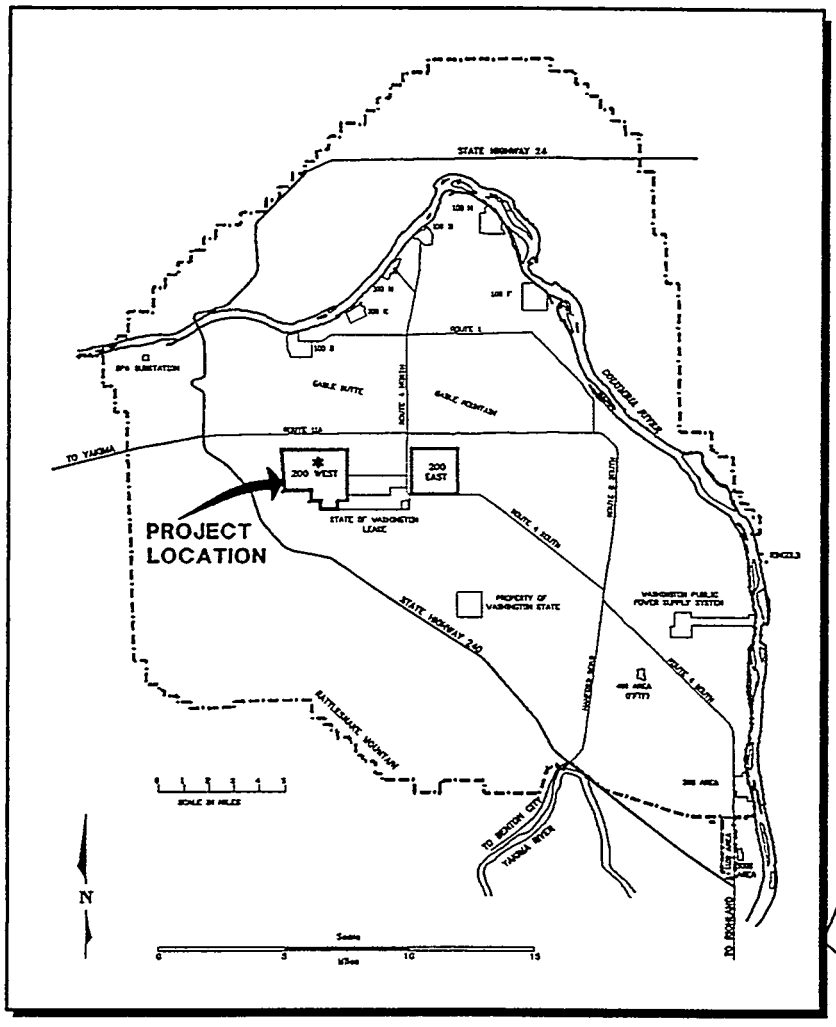
APPENDIX B

REDUCED DESIGN DRAWINGS

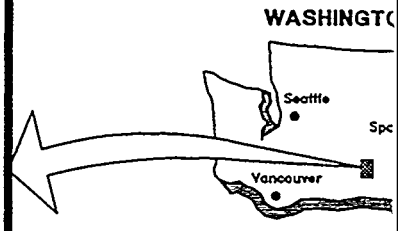
RADIOACTIVE
LAND DISP
NON-D

PROJ

PREPARED BY GOLDER ASSOCIA



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MIXED WASTE (RMW)
OSAL FACILITY
RAG-OFF

CT W-025

ES INC., REDMOND (SEATTLE), WASHINGTON

LIST OF DRAWINGS

31573	COVER SHEET	H-2-131584	TRUCK STAGING AREA AND ACCESS RAMP
31574	SITE PLAN AND EXPLORATIONS	H-2-131585	LEACHATE COLLECTION TANK AND PIPING (2 SHEETS)
31575	GRADING PLAN	H-2-131586	SUMP PUMP DETAILS
31576	SECONDARY ADMIX CONTOURS	H-2-131587	ELECTRICAL (4 SHEETS)
31577	OPERATIONS LAYER CONTOURS	H-2-131588	CONCEPTUAL FILL PLAN
31578	CROSS-SECTIONS	H-2-131589	SUBGRADE GRID POINTS
31579	LINER SYSTEM DETAILS	H-2-131590	SECONDARY ADMIX GRID POINTS
31580	SUMP LAYOUT	H-2-131591	SECONDARY DRAINAGE GRAVEL GRID POINTS
31581	SUMP CROSS-SECTIONS	H-2-131592	PRIMARY DRAINAGE GRAVEL GRID POINTS
31582	SUMP LEACHATE COLLECTION PIPES	H-2-131593	OPERATIONS LAYER GRID POINTS
31583	SIDE SLOPE AND VERTICAL RISER PIPES	H-2-131594	PRIMARY ADMIX GRID POINTS

FRANK S. SHURI — PROFESSIONAL ENGINEER

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT PER RFW	SGH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	EDITORIAL	KGH	F. SHURI	5/1/92

REVISIONS

CAD FILE B131573A | CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED	T. MARTIN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / FOR / DATE

IMPACT LEVEL | CAT 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

COVER SHEET
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

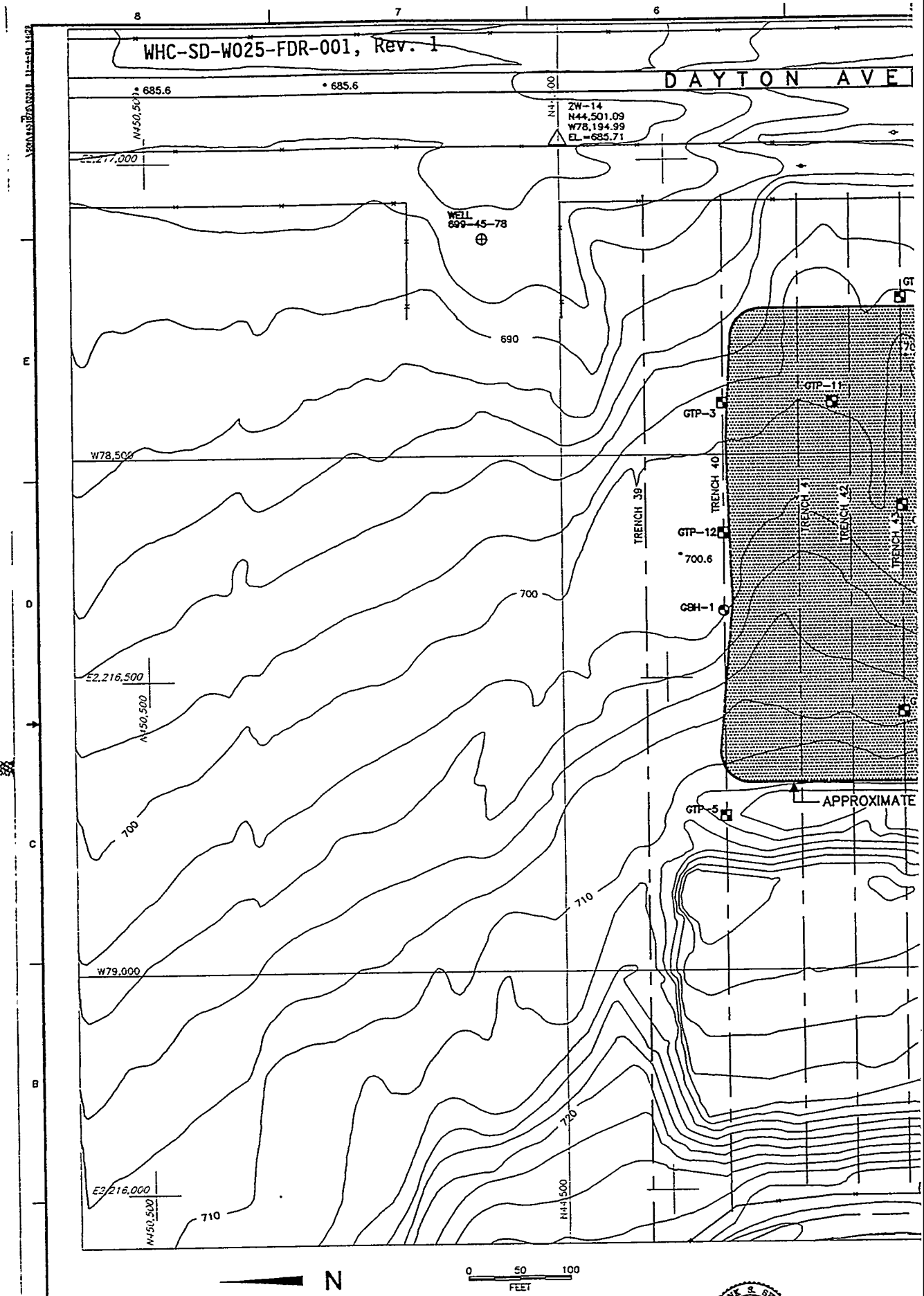
PROJECT W-025

SIZE	BLDG. NO.	INDEX NO.	DWG. NO.	REV NO
F	2W-W-6	0000	H-2-131573	3

SCALE AS SHOWN | JOB NO. 903-1186.036 | SHEET 1 OF 1

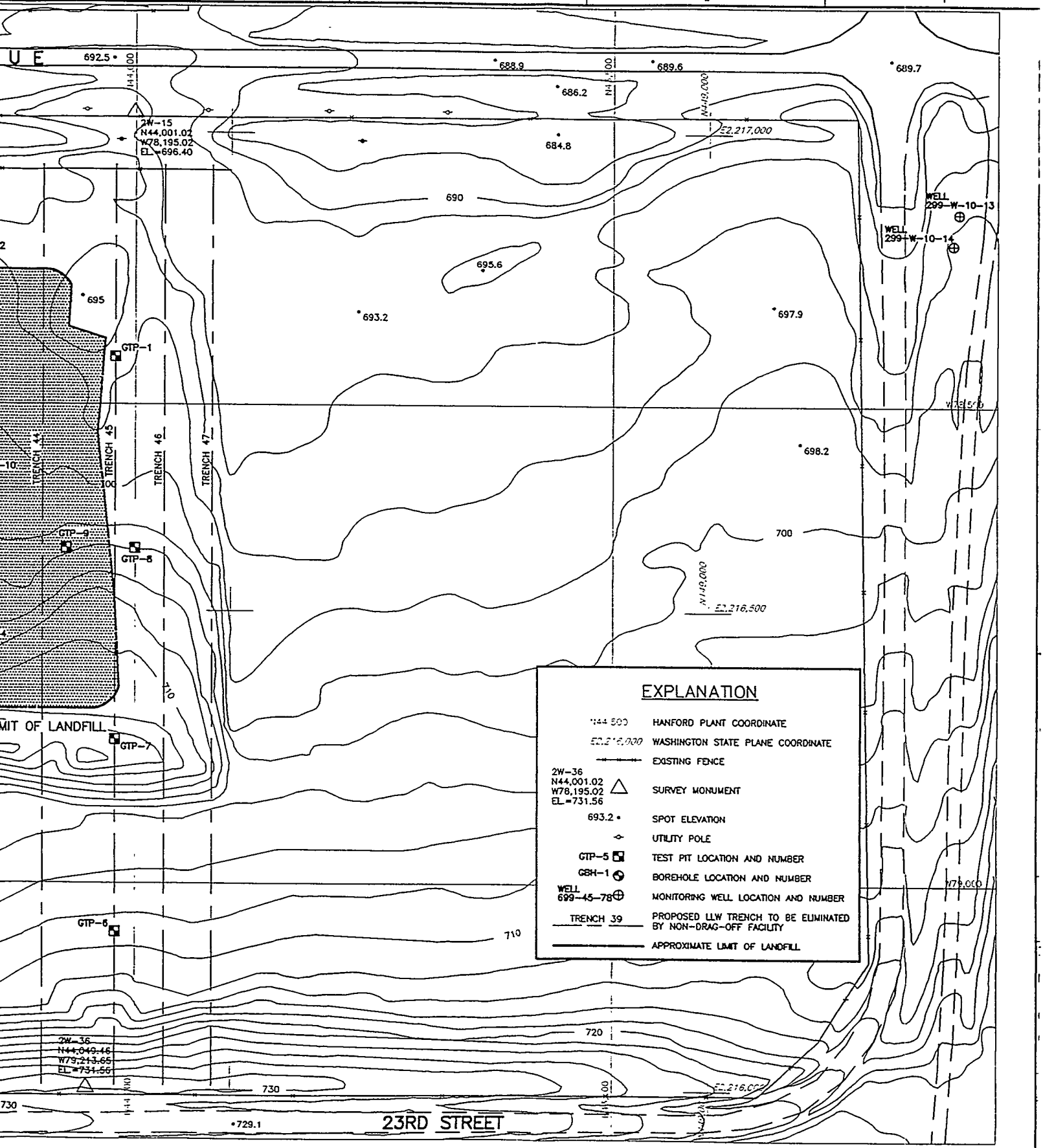
NUMBER	TITLE
REFERENCES	
NEXT USED ON	

DRY WASTE BURIAL GROUND 218-W-6
TITLE
TRACEABILITY LIST



NOTES:

1. For test pit and borehole logs see Site Investigation Report, Non-Drag-Off Landfill Site Low Level Burial Area No. 5, 200 West Area, Dated December 4, 1989.
2. Topographic maps prepared by Degrosa Aerial Mapping based on July 18, 1988, aerial photography. Topography west of landfill revised January 21, 1992 per ECN 119451.



DESIGNED	T. MARTIN	4/2/90	
DRAWN	J.S. SNOW	4/2/90	
CHECKED	F. SHURI	4/2/90	
REVIEWED	P. CORSER	4/2/90	
QA	M.J. OTTERSON	4/2/90	

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SITE PLAN AND EXPLORATION LOCATIONS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025

SIZE	BLDG. NO.	MACH. NO.	DWG. NO.	REV. NO.
F		0501	H-2-131574	3

SCALE AS SHOWN | JOB NO. 803-1166.036 | SHEET 1 OF 1

IMPACT LEVEL | EDT 151914

NO.	DESCRIPTION	DATE	APPROVALS
3	AS-BUILT PER EDN EDN	SDH F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP F. SHURI	5/21/93
1	REVISE TOPOGRAPHIC CONTOURS WEST OF LANDFILL	WA F. SHURI	5/1/92

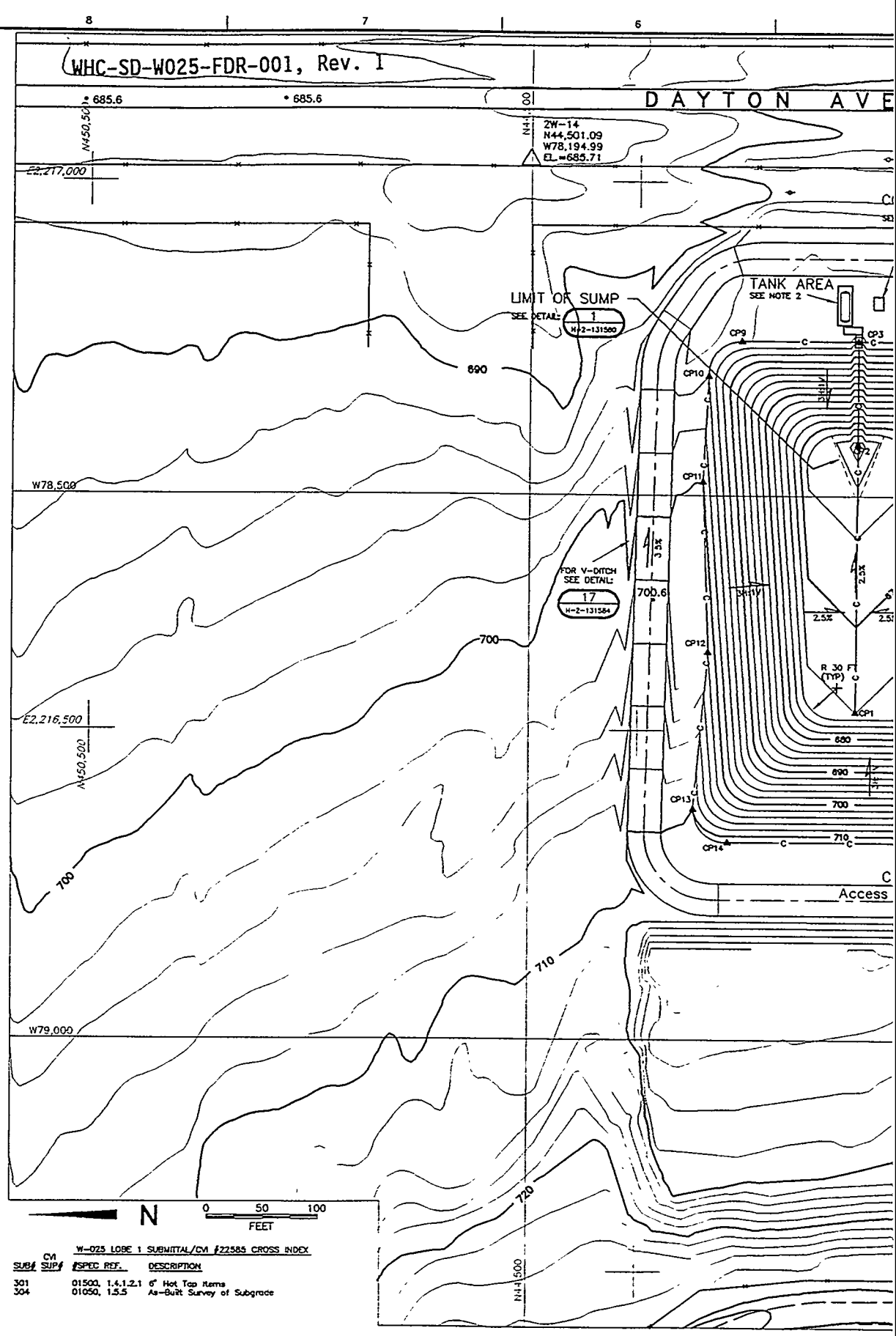
APPROVED FOR RELEASE BY FOR DATE

REFERENCES

NUMBER	TITLE	NEXT USED ON
DRY WASTE BURIAL GROUND 218-W-5		
GRAPHIC MAP LOW LEVEL BURIAL GROUND (SHEETS 2,3, AND 7)		

TRACEABILITY LIST

CAD FILE B131574A CAD CODE 2W:IBM:ACD2:10:SS



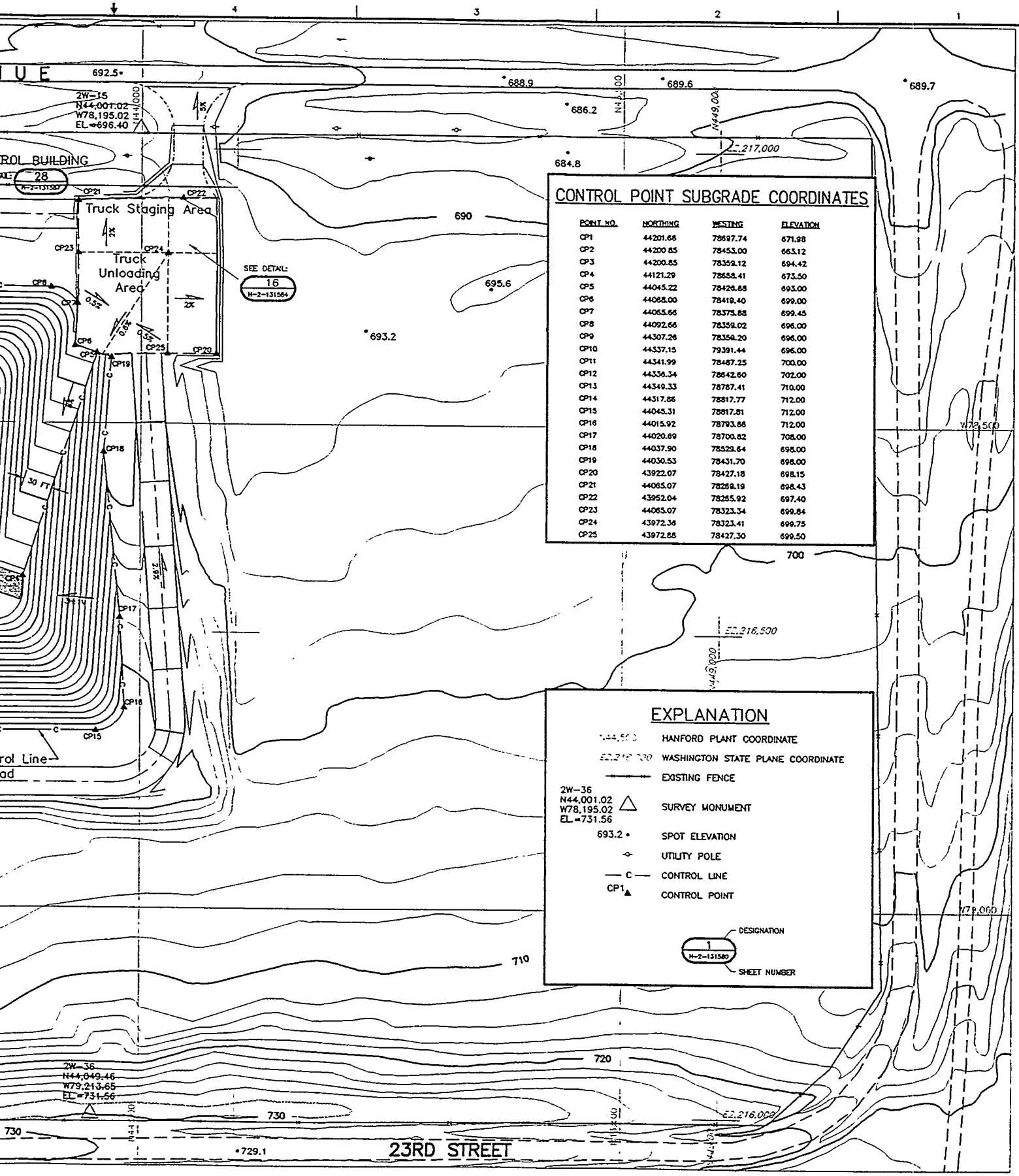
W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
301		01500, 1.4.1.2.1	6" Hot Tap Items
304		01050, 1.5.5	As-Built Survey of Subgrade

NOTES:

1. Excavated surface is shown in section on Drawing H-2-131578.
2. Details of the leachate collection tank and piping shown on Drawing H-2-131585.
3. Topographic maps prepared by Degrossa Aerial Mapping based on July 18, 1988, aerial photography. Topography west of landfill revised January 21, 1992 per ECN 119451.
4. Location of excavation stockpile and borrow preparation areas to be approved by Construction Manager.
5. For as-built conditions, see "Subgrade As-Built" drawing in the CQA report, WHC-SD-W025-RPT-001.

FRANK S. SEURI PROFESSIONAL ENGINEER



CONTROL POINT SUBGRADE COORDINATES

POINT NO.	NORTHING	WESTING	ELEVATION
CP1	44201.68	78697.74	671.98
CP2	44200.85	78453.00	663.12
CP3	44200.85	78359.12	694.42
CP4	44121.29	78658.41	673.50
CP5	44045.22	78428.88	693.00
CP6	44068.00	78419.40	699.00
CP7	44065.66	78375.88	699.45
CP8	44092.66	78359.02	696.00
CP9	44307.26	78359.20	696.00
CP10	44337.15	79391.44	696.00
CP11	44341.99	78487.25	700.00
CP12	44336.34	78642.60	702.00
CP13	44349.33	78787.41	710.00
CP14	44317.86	78817.77	712.00
CP15	44045.31	78817.81	712.00
CP16	44015.92	78793.86	712.00
CP17	44020.69	78700.82	708.00
CP18	44037.90	78529.64	698.00
CP19	44030.53	78431.70	698.00
CP20	43922.07	78427.18	698.15
CP21	44065.07	78258.19	698.43
CP22	43952.04	78255.92	697.40
CP23	44065.07	78323.34	699.84
CP24	43972.38	78323.41	699.75
CP25	43972.88	78427.30	699.50

EXPLANATION

- N44,500 — HANFORD PLANT COORDINATE
- E217,500 — WASHINGTON STATE PLANE COORDINATE
- ——— EXISTING FENCE
- 2W-36 N44,001.02 W78,195.02 EL.=731.56 △ — SURVEY MONUMENT
- 693.2 • — SPOT ELEVATION
- ◇ — UTILITY POLE
- — C — CONTROL LINE
- CP1 ▲ — CONTROL POINT
- 1 / H-2-131580 — DESIGNATION / SHEET NUMBER

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	ADDED CONTROL POINTS PER ECN 9-025-003 AND CHANGED BLOCK AND NOTE 5 PER ECN 9-025-028 ADDED TRUCK STAGING AREA CONTROL POINTS PER ECN 9-025-021 MADE ACCESS ROAD 10 FT NORTH PER ECN 9-025-012	SGH	F. SHURI	8/26/94
2	REVISED SPECIFIC AREAS FOR EXCAVATION PER ECN 9-025-004	LPP	F. SHURI	5/21/93
1	REVISE TOPOGRAPHIC CONTOURS WEST OF LANDFILL, MODIFY LANDFILL CEMETERY AND CONTROL POINTS	KGH	F. SHURI	5/1/92

REVISIONS

APPROVED FOR IMPLEMENTATION BY

DESIGNED	T. MARTIN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90

IMPACT LEVEL — ECN 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

GRADING PLAN
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025

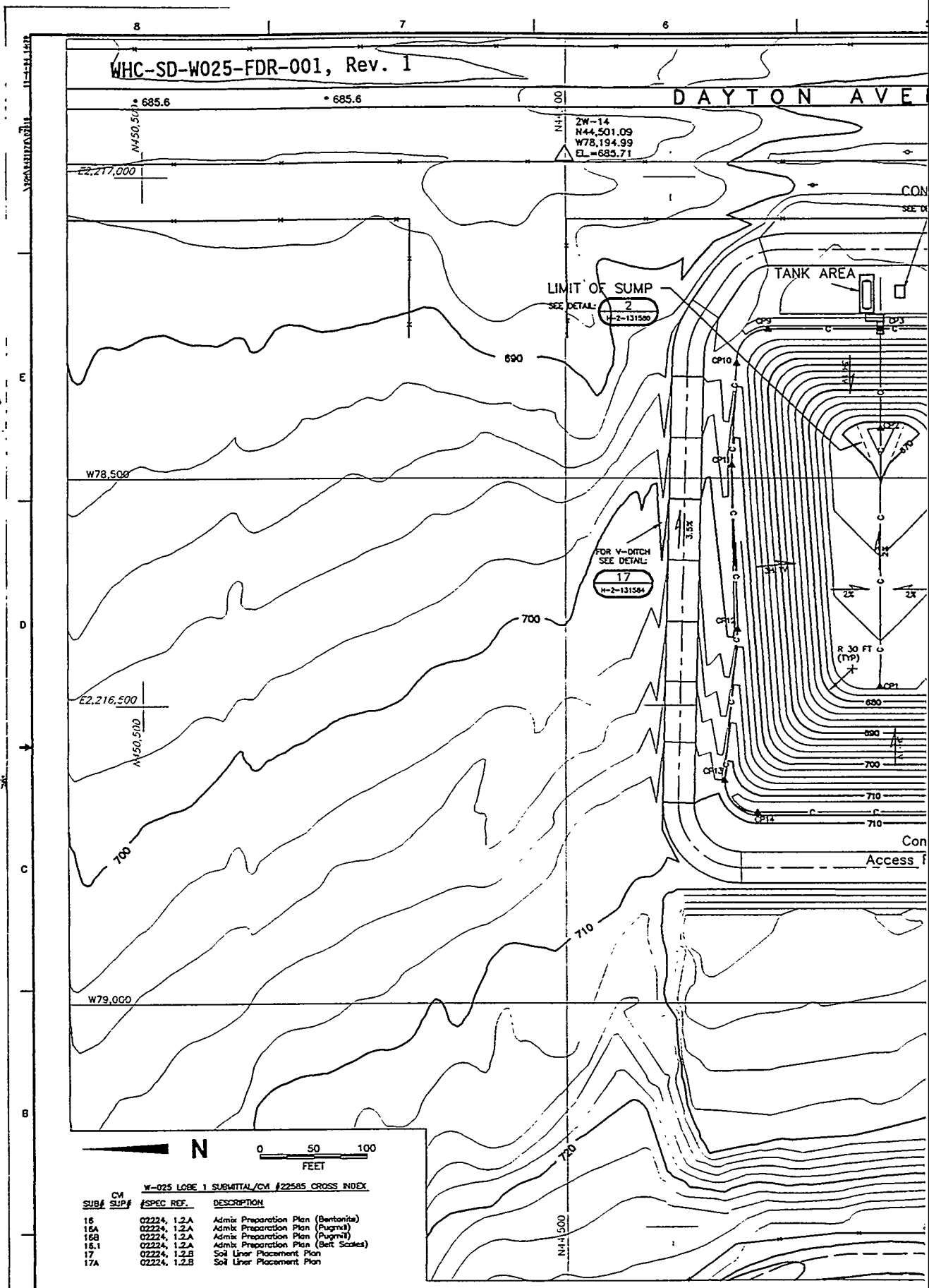
SIZE	BLDG. NO.	INDEX NO.	DWG. NO.	REV NO
F	240-9-6	0401	H-2-131575	3

NUMBER	TITLE
REFERENCES	
NOT USED ON	

TRACEABILITY LIST

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ACAD FILE R131575A — CAD CODE 2W-RW-ACD2-10-SS

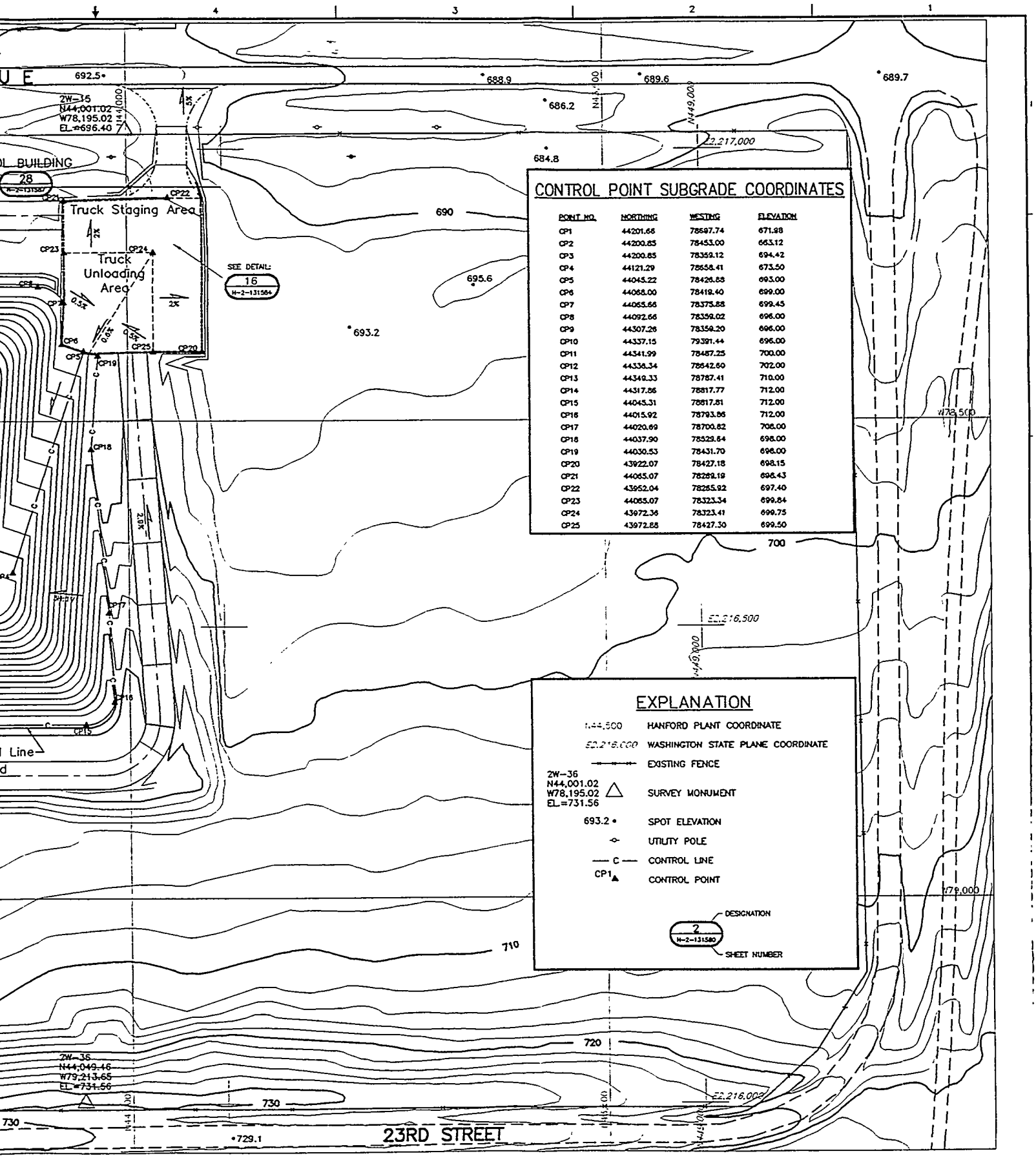


W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUB#	SPEC REF.	DESCRIPTION
16		02224, 1.2.A	Admix Preparation Plan (Bentonite)
16A		02224, 1.2.A	Admix Preparation Plan (Pugmill)
16B		02224, 1.2.A	Admix Preparation Plan (Pugmill)
16.1		02224, 1.2.A	Admix Preparation Plan (Belt Scales)
17		02224, 1.2.B	Soil Liner Placement Plan
17A		02224, 1.2.B	Soil Liner Placement Plan

NOTES:

1. Access ramp shown in profile on Drawing H-2-131584.
2. Secondary side slope riser pipe trench not shown on this sheet. Side slope riser pipe trenches shown on section on Drawing H-2-131581.
3. Topographic maps prepared by Degrass Aerial Mapping based on July 18, 1988, aerial photography. Topography west of landfill revised January 21, 1992 per ECN 119451.
4. Location of excavation stockpile and admix preparation areas to be approved by Construction Manager.
5. For as-built conditions, see "Top of Secondary Admix As-Built" drawing in the CQA Report, WHC-SD-W025-RPT-001.



CONTROL POINT SUBGRADE COORDINATES

POINT NO.	NORTHING	WESTING	ELEVATION
CP1	44201.66	78697.74	671.98
CP2	44200.85	78453.00	663.12
CP3	44200.85	78359.12	694.42
CP4	44121.29	78658.41	673.50
CP5	44045.22	78426.85	693.00
CP6	44068.00	78419.40	699.00
CP7	44065.66	78375.88	699.45
CP8	44092.66	78359.02	696.00
CP9	44307.26	78359.20	696.00
CP10	44337.15	79391.44	696.00
CP11	44341.99	78487.25	700.00
CP12	44338.34	78642.60	702.00
CP13	44349.33	78787.41	710.00
CP14	44317.86	78817.77	712.00
CP15	44045.31	78817.81	712.00
CP16	44015.92	78793.86	712.00
CP17	44020.69	78700.82	708.00
CP18	44037.90	78529.84	698.00
CP19	44030.53	78431.70	698.00
CP20	43922.07	78427.18	698.15
CP21	44065.07	78289.19	698.43
CP22	43952.04	78265.92	697.40
CP23	44065.07	78323.34	699.84
CP24	43972.36	78323.41	699.75
CP25	43972.88	78427.30	699.50

EXPLANATION

- N44,500 — HANFORD PLANT COORDINATE
- E2,216,000 — WASHINGTON STATE PLANE COORDINATE
- EXISTING FENCE
- 2W-36 / N44,001.02 / W78,195.02 / EL.=731.56 — SURVEY MONUMENT
- 693.2 • — SPOT ELEVATION
- UTILITY POLE
- — C — CONTROL LINE
- CP1 ▲ — CONTROL POINT
- 2 / H-2-131580 — DESIGNATION / SHEET NUMBER

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	ADD CP'S, CHANGE CONTROL POINTS PER ECN 9-025-033, ADD CH CROSS SLOPE AND NOTE 2 PER ECN 9-025-035, ADJUST TRUCK STAGING AREA CONTROL POINTS PER ECN 9-025-025, MOVE ACCESS ROAD 10 FT NORTH PER ECN 9-025-012, EDITORIAL PER ECN 9-025-036, ECN 9-025-038	SGH	F. SHURI	8/26/94
2	REVISE SPECIFIC AREAS FOR STOCKPILES PER ECN 9-025-001	JPP	F. SHURI	5/21/93
1	REVISE ELEVATION CONTOURS IN LANDFILL	KGH	F. SHURI	5/1/92

REVISIONS

DESIGNED	T. MARTIN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90

APPROVED FOR IMPLEMENTATION — BY — TITLE — DATE

IMPACT LEVEL — CF 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SECONDARY ADMIX CONTOURS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

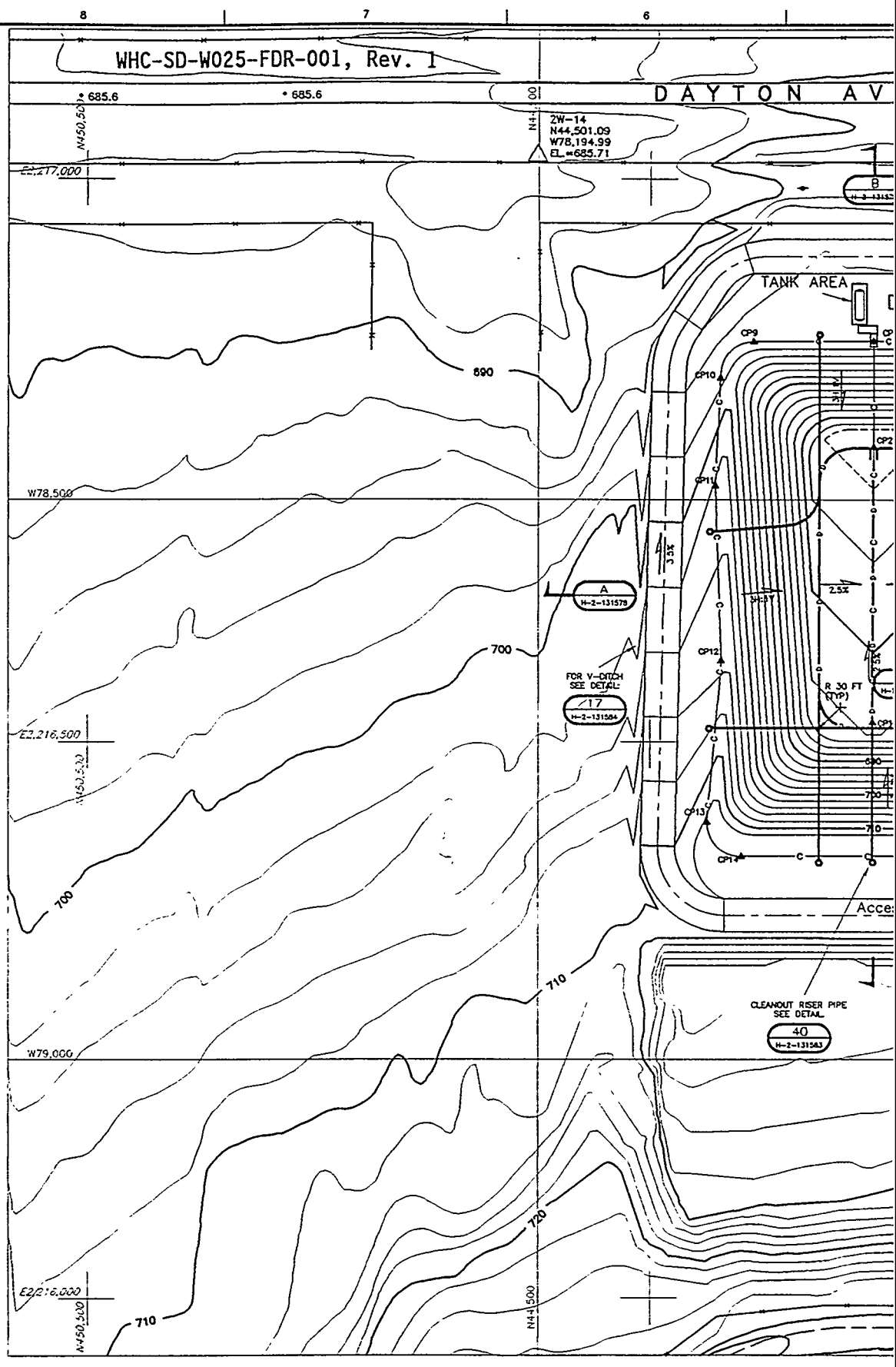
PROJECT W-025

SIZE	BLDG. NO.	INDEX NO.	DWG. NO.	REV. NO.
F	220-0-6	0401	H-2-131576	3

SCALE AS SHOWN — JOB NO. 893-1186.0.035 — SHEET 1 OF 1

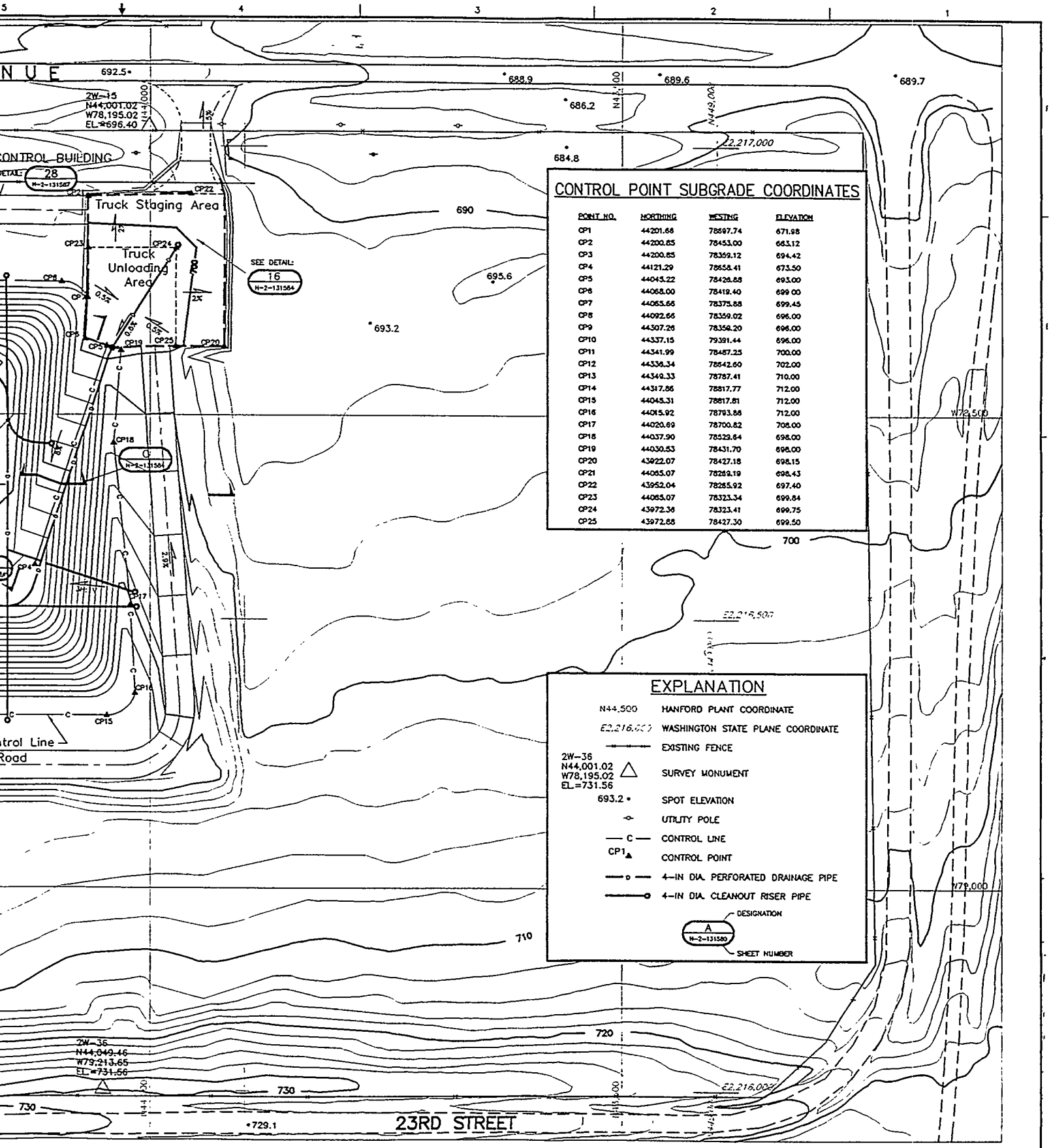
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REFERENCES
NOT USED ON
TRACEABILITY LIST

CAD FILE B131576A — CAD CODE 2W:IBM:ACD2:10:SS



NOTES:

1. Vertical riser pipe not shown.
2. Topographic maps prepared by Degrosa Aerial Mapping based on July 18, 1988, aerial photography.
3. Topography west of landfill revised January 21, 1992 per ECN 119451.
4. Location of excavation stockpile and admix preparation areas to be approved by Construction Manager.
5. For as-built conditions, see "Top of Operations Layer As-Built" drawing in the COA report, WHC-SD-W025-RPT-001.



CONTROL POINT SUBGRADE COORDINATES

POINT NO.	NORTHING	WESTING	ELEVATION
CP1	44201.68	78697.74	671.98
CP2	44200.85	78453.00	683.12
CP3	44200.85	78359.12	694.42
CP4	44121.29	78658.41	673.50
CP5	44045.22	78426.88	693.00
CP6	44068.00	78419.40	699.00
CP7	44065.66	78375.88	699.45
CP8	44092.66	78359.02	696.00
CP9	44307.26	78356.20	696.00
CP10	44337.15	79391.44	696.00
CP11	44341.99	78487.25	700.00
CP12	44336.34	78642.60	702.00
CP13	44349.33	78787.41	710.00
CP14	44317.86	78817.77	712.00
CP15	44045.31	78817.81	712.00
CP16	44015.92	78793.86	712.00
CP17	44020.69	78700.82	706.00
CP18	44037.90	78529.64	698.00
CP19	44030.53	78431.70	698.00
CP20	43922.07	78427.18	698.15
CP21	44065.07	78269.19	698.43
CP22	43952.04	78265.92	697.40
CP23	44065.07	78323.34	699.84
CP24	43972.36	78323.41	699.75
CP25	43972.88	78427.30	699.50

EXPLANATION

- N44,500 — HANFORD PLANT COORDINATE
- E2,216,000 — WASHINGTON STATE PLANE COORDINATE
- EXISTING FENCE
- 2W-36 N44,001.02 W78,195.02 EL=731.56 — SURVEY MONUMENT
- 693.2 • — SPOT ELEVATION
- UTILITY POLE
- — C — CONTROL LINE
- CP1 — CONTROL POINT
- — D — 4-IN DIA. PERFORATED DRAINAGE PIPE
- — O — 4-IN DIA. CLEANOUT RISER PIPE
- A / H-2-131580 — DESIGNATION / SHEET NUMBER

NO	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT FOR EDH; CHANGE CONTROL POINTS PER EDH H-028-033; ADD CALLOUT FOR DETAIL 16; ADD NOTE 5 PER EDH H-028-028; ADD 5' TRUCK STAGING AREA CONTROL POINTS PER EDH H-028-031; REMOVE ACCESS ROAD 16 FT NORTH PER EDH H-028-032; REVISED SPECIFIC AREAS FOR STOCKPILES PER EDH H-028-031	SH	F. SHURI	10/6/94
2	REVISED PER EDH H-028-031	JPP	F. SHURI	5/17/93
1	REVISE TOPOGRAPHIC CONTOURS WEST OF LANDFILL; ADDED CONTROL POINT LOCATIONS	KGH	F. SHURI	5/1/92

REVISIONS

CAD FILE B131577A — CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED	T. MARTIN	4/2/90
DRAWN	J.S. SNOW	4/2/90
CHECKED	F. SHURI	4/2/90
REVIEWED	P. CORSER	4/2/90
QA	M.J. OTTERSON	4/2/90

APPROVED FOR IMPLEMENTATION — BY — FOR — DATE

IMPACT LEVEL — EDF 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

OPERATIONS LAYER CONTOURS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025

SIZE	BLDG. NO.	HOOD NO.	DWG. NO.	REV NO.
F	220-B-6	0401	H-2-131577	3

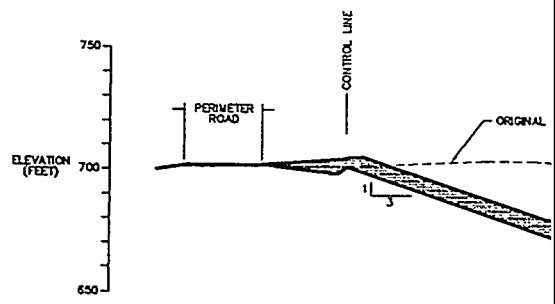
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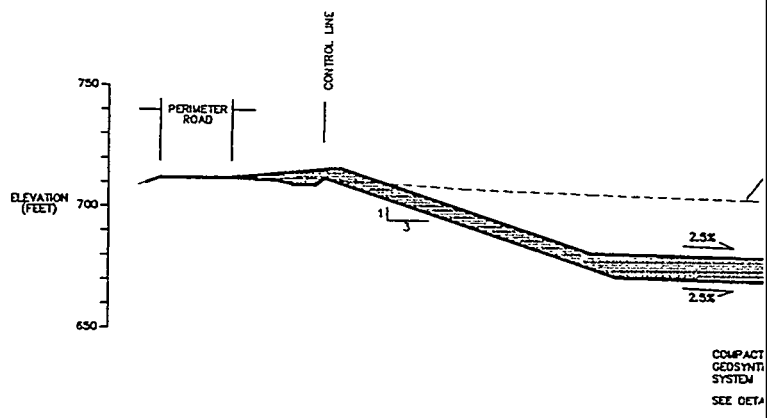
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REFERENCES	
NEXT USED ON	

TOPOGRAPHIC MAP LOW LOG BURN, CCSSW (SHEETS 2,3, AND 7)

TITLE

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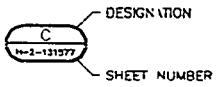


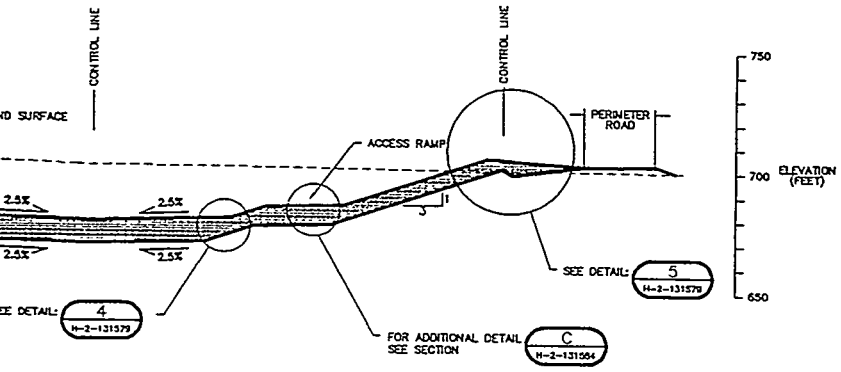


COMPACT
GEOSYNTH
SYSTEM

SEE DETA

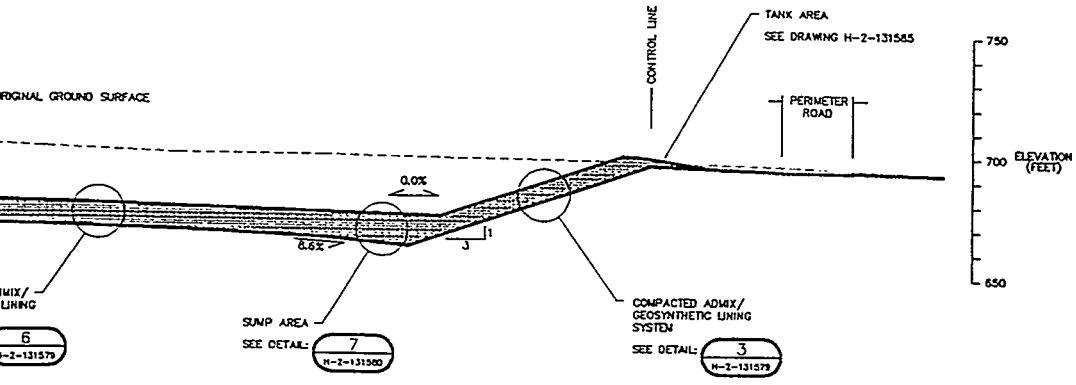
KEY:





SECTION A H-2-131577

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SCALE IN FEET



SECTION B H-2-131577

0 30 60
SCALE IN FEET

FRANK S. SHURI — STATE OF WASHINGTON — PROFESSIONAL ENGINEER

3	AS-BUILT PER ECN [illegible] EDITORIAL PER ECN [illegible]	SCH	F. SHURI 8/26/94
2	MINOR EDITORIAL	APP	F. SHURI 5/27/93
1	INCREASE THICKNESS OF LINER SYSTEM	KCH	F. SHURI 5/1/92
REV NO	DESCRIPTION	DATE	APPROVALS

REVISIONS

CAD FILE B131578A CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED T. MARTIN 04/02/90
DRAWN J. SNOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
QA M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION
BY
DATE

APPROVED
TITLE
REFERENCES
DRAWING USED ON
TRACEABILITY LIST

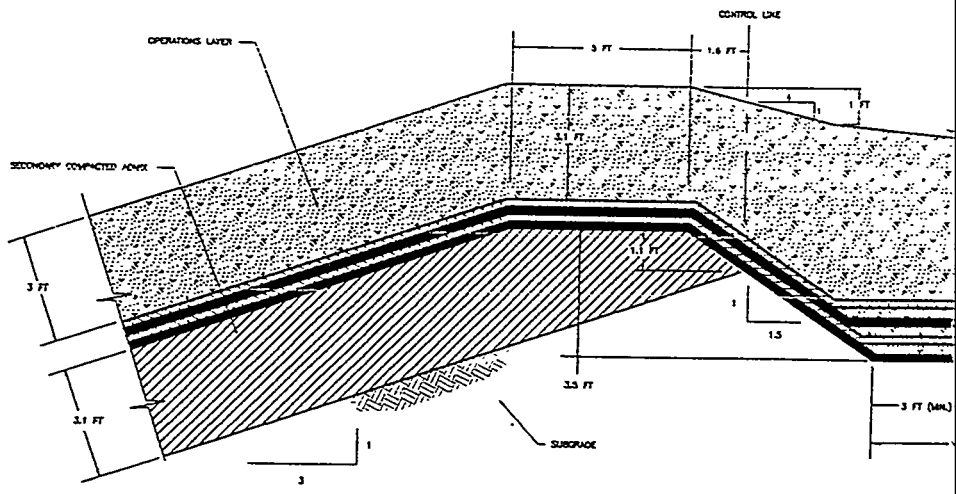
SURFACE LEVEL | EDT 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

CROSS-SECTIONS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | SUBS. NO. [illegible] | ACCN NO. 0401 | DWG. NO. H-2-131578 | REV NO. 3
SCALE AS SHOWN | JOB NO. 293-1166.035 | SHEET 1 OF 1

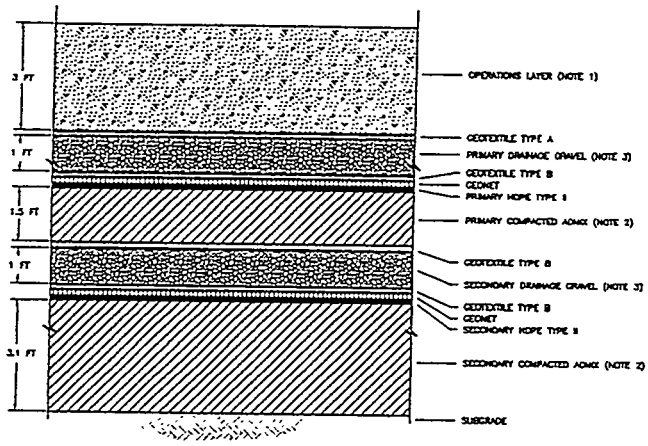
H-2-131578



ANCHOR TRENCH DETAIL

5
H-2-131579

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SCALE IN FEET



BASE LINER DETAIL

6
H-2-131579

0 2 4
SCALE IN FEET

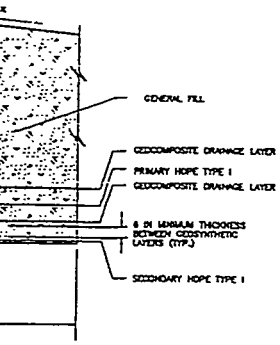
W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

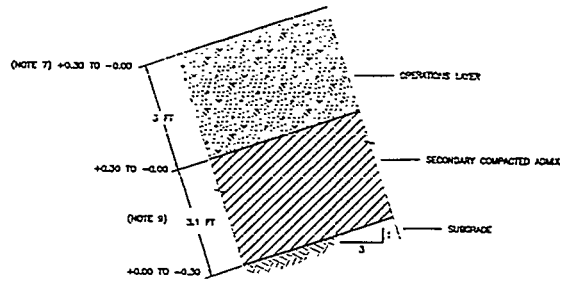
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25		02275, 1.7.A	Geomembrane Installer's QA Manual
25A		02275, 1.7.A	Geomembrane Crystallinity
25B		02275, 1.7.A	Geomembrane/Admix Friction Angle
25.1		02275, 1.7.A	Geomembrane QA/QC Data
25.2		02275, 1.7.A	Extrudate Certification
25.3		02275, 1.7.A	Geomembrane QA/QC Data
25.3A		02275, 1.7.A	Geomembrane Resin Certification
25.4		02275, 1.7.A	Geomembrane QA/QC Data
25.4B		02275, 1.7.A	Geomembrane Env. Stress Crack Certification
25.4C		02275, 1.7.A	Geomembrane Env. Stress Crack Certification
25.5		02275, 1.7.A	Geomembrane Low Temp. Welding Certification
25.6		02275, 1.7.A	Geomembrane QA/QC Documentation
25.6A		02275, 1.7.A	Geomembrane Direct Shear Test Report
25.7		02275, 1.7.A	Geomembrane Resin Documentation
25.8		02275, 1.7.A	Geomembrane Resistance to Soil Burial Data
25.9		02275, 1.7.A	Geomembrane Resistance to Soil Burial Cert.
28		02275, 1.7.B	Geosynthetic Samples

NOTES:
1. Operations layer shall consist of soils excavated from on site, meeting the requirements of Section 02228 of the Technical Specifications.
2. Compacted admix shall meet the compaction, compaction, and permeability requirements as described in Section 02224 of the Technical Specifications.
3. Primary and secondary base liner drainage gravel shall meet the requirements of Section 02225 of the Technical Specifications.
4. Geosynthetic materials shall meet the requirements of Section 02275 of the Technical Specifications.
5. Geosynthetic layers not drawn to scale.
6. Grade shall be minimum of 1 foot above underlying layer.
7. Grade shall be minimum of 3 feet above underlying layer.
8. Overlap geomembrane as appropriate for welding method (see Specifications Section 02275).
9. Tolerances based on design grade.
10. Geotextile to be sewn (see Specifications Section 02275).
11. Grade shall be minimum of 1.5 feet above underlying layer.
12. For as-built conditions, see "Secondary HDPE Geomembrane Liner As-Built Drawing", and "Primary HDPE Geomembrane Liner As-Built Drawing" in the CQA Report, WHC-SD-W025-RPT-001.

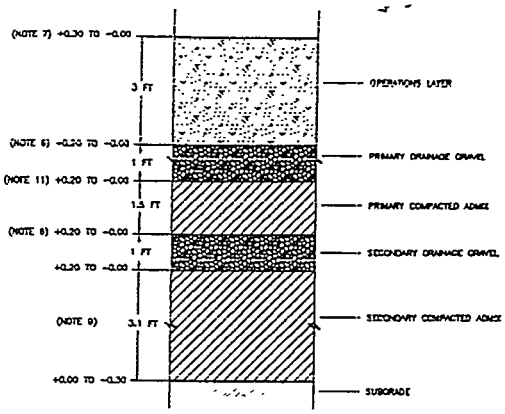
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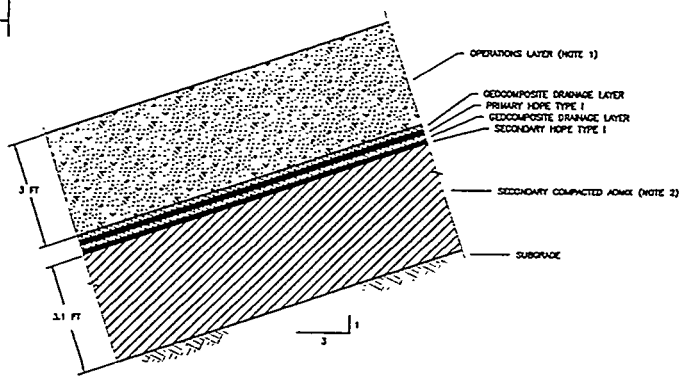
SIDESLOPE



BASE

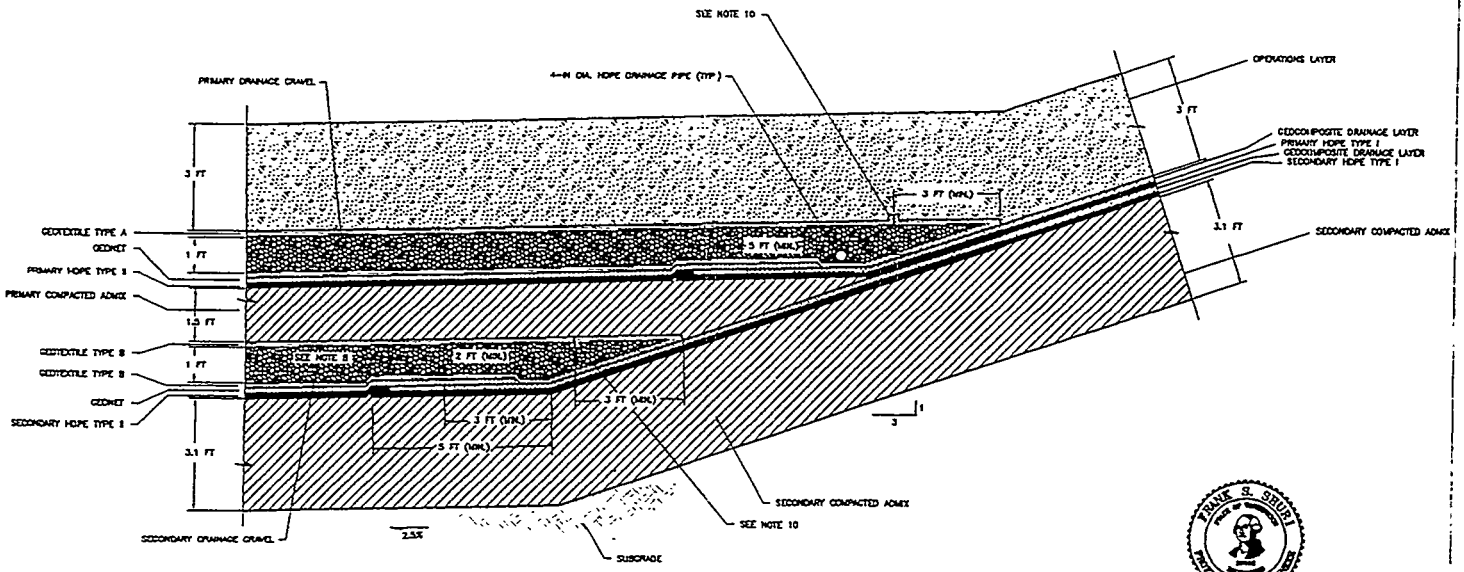
GRADING TOLERANCE DETAIL

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SCALE IN FEET



SIDESLOPE LINER DETAIL (3, H-2-131579)

0 2 4
SCALE IN FEET



TYPICAL TOE OF SLOPE DETAIL (4, H-2-131579)

0 2 4
SCALE IN FEET

FRANK S. SHURI — STATE OF WASHINGTON — PROFESSIONAL ENGINEER — EXPIRES 3-31-96

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT PER ECN; ADD CM CROSS REFS AND NOTE 12 PER ECN 9402-402; OPTIONAL PER ECN 9402-008, ECN 9402-172	SGH	F. SHURI	8/26/94
2	MINOR EDITORIAL PER ECN 9402-001	JPP	F. SHURI	5/21/93
1	INCREASE OPERATIONS LAYER THICKNESS TO 3 FT AND "PRIMARY" CHECK LAYER	JPP	F. SHURI	5/1/92

REVISIONS

CAD FILE B131579A | CAD CODE 2W;IBM;ACD2;10;SS

DESIGNED	T. MARTIN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / FOR / DATE

IMPACT LEVEL | ECF 151814

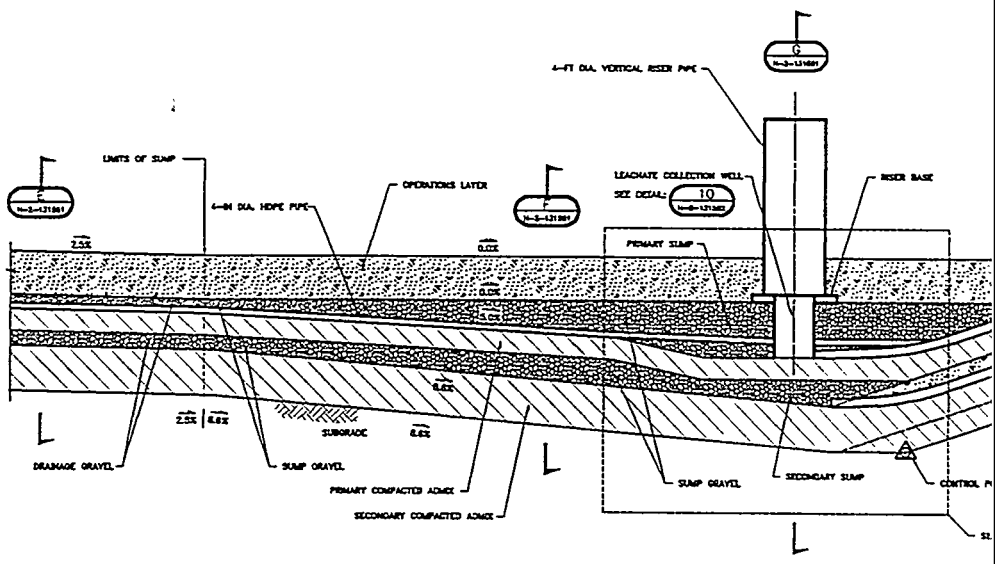
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

LINER SYSTEM DETAILS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | BLDG. NO. 216-B-6 | MDCS NO. 0401 | DWG. NO. H-2-131579 | REV. NO. 3
SCALE AS SHOWN | JOB NO. 903-1166.035 | SHEET 1 OF 1

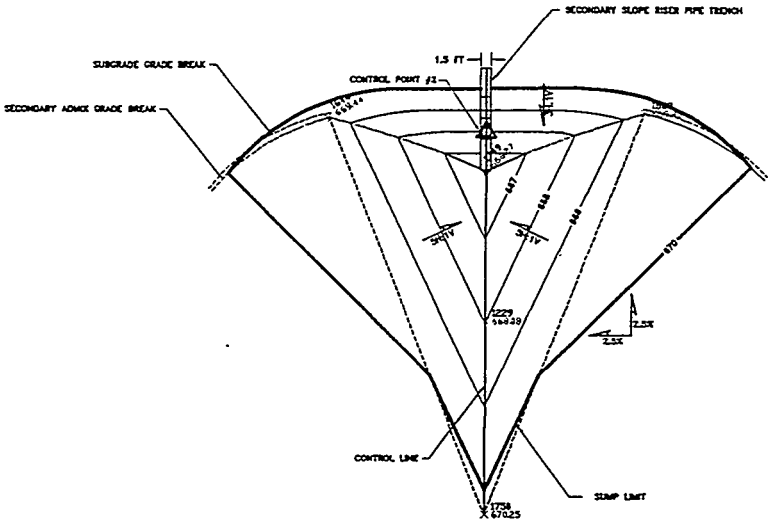
TITLE	NUMBER	TITLE
DRAWING TRACEABILITY LIST	REFERENCES	

111.4 131731



SUMP PROFILE (7, H-2-131580)

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SCALE IN FEET



SUMP SECONDARY ADMIX (2, H-2-131580)

0 10 20
SCALE IN FEET

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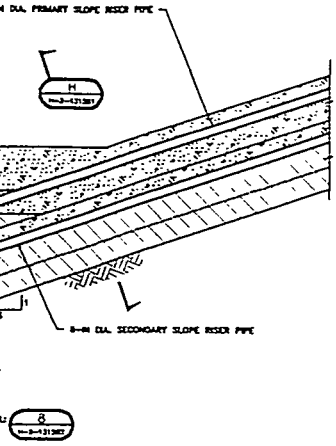
CVI SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
21		02226, 1.2	Granular Drainage Layers Gravel Samples

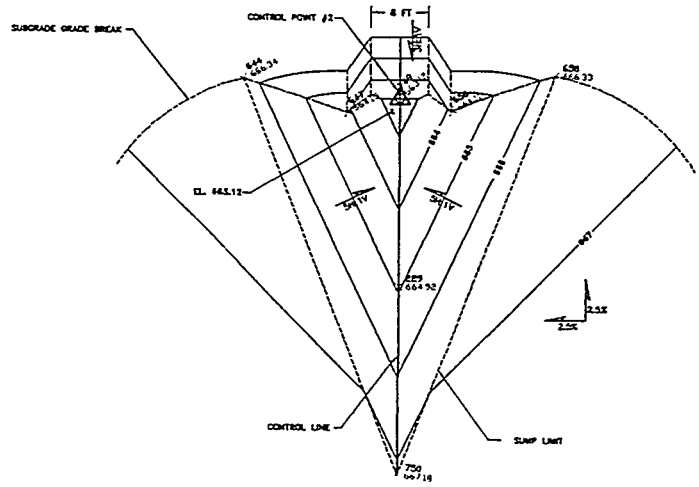
W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

NOTE:

1. Control points defined on Drawing H-2-131575.
2. For as-built conditions, see "Top of Secondary Gravel As-Built", "Top of Primary Admix As-Built", and "Top of Primary Gravel As-Built" drawings in the CCA Report, WHC-SD-W025-RPT-001.

KEY: (5, H-2-131581) — 5 = DESIGNATION, H-2-131581 = SHEET NUMBER

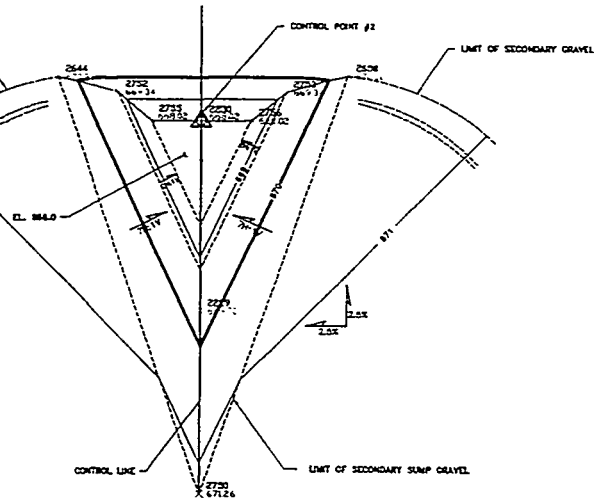


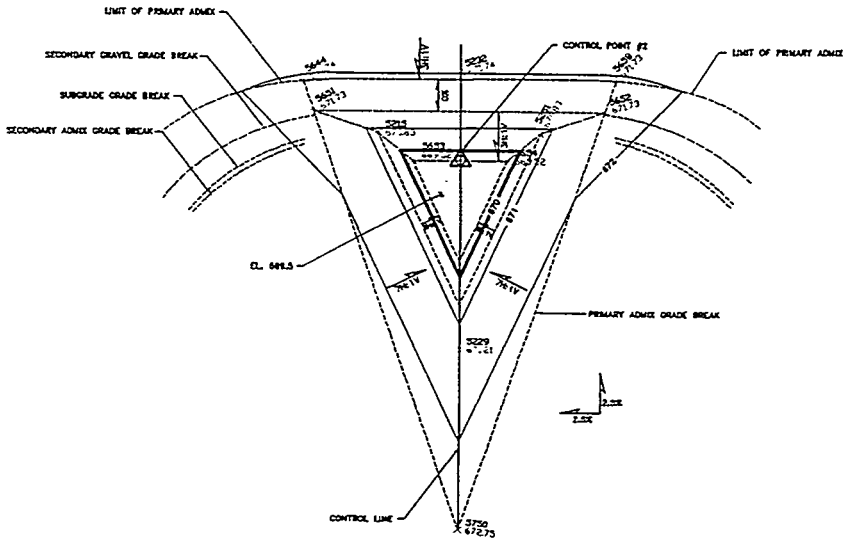


SUMP SUBGRADE (1 / H-2-131580)

0 10 20
SCALE IN FEET

N





P SECONDARY GRAVEL (3 / H-2-131580)

0 10 20
SCALE IN FEET

N

SUMP PRIMARY ADMIX (4 / H-2-131580)

0 10 20
SCALE IN FEET

N

FRANK S. SHURI — STATE OF ILLINOIS — PROFESSIONAL ENGINEER

NO	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT PER ECN; ADD DA CROSS INDEX AND NOTE 2 PER ECN	SGH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	INCREASE OPERATIONS LAYER THICKNESS TO 3 FEET	KGH	F. SHURI	5/5/92

REVISIONS

OD FILE B131580A — OD CODE 2W:IBM:ACD2:10:SS

DESIGNED	T. MARTIN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / DATE

IMPACT LEVEL — CF 151914

U. S. DEPARTMENT OF ENERGY
ROCKY FLATS OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

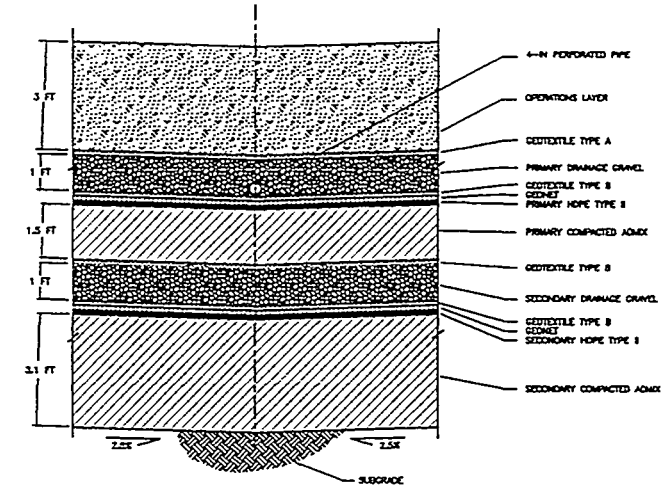
SUMP LAYOUT
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025

SIZE	BLDG. NO.	SCALE NO.	DWG. NO.	REV NO.
F	730-9-6	0401	H-2-131580	3

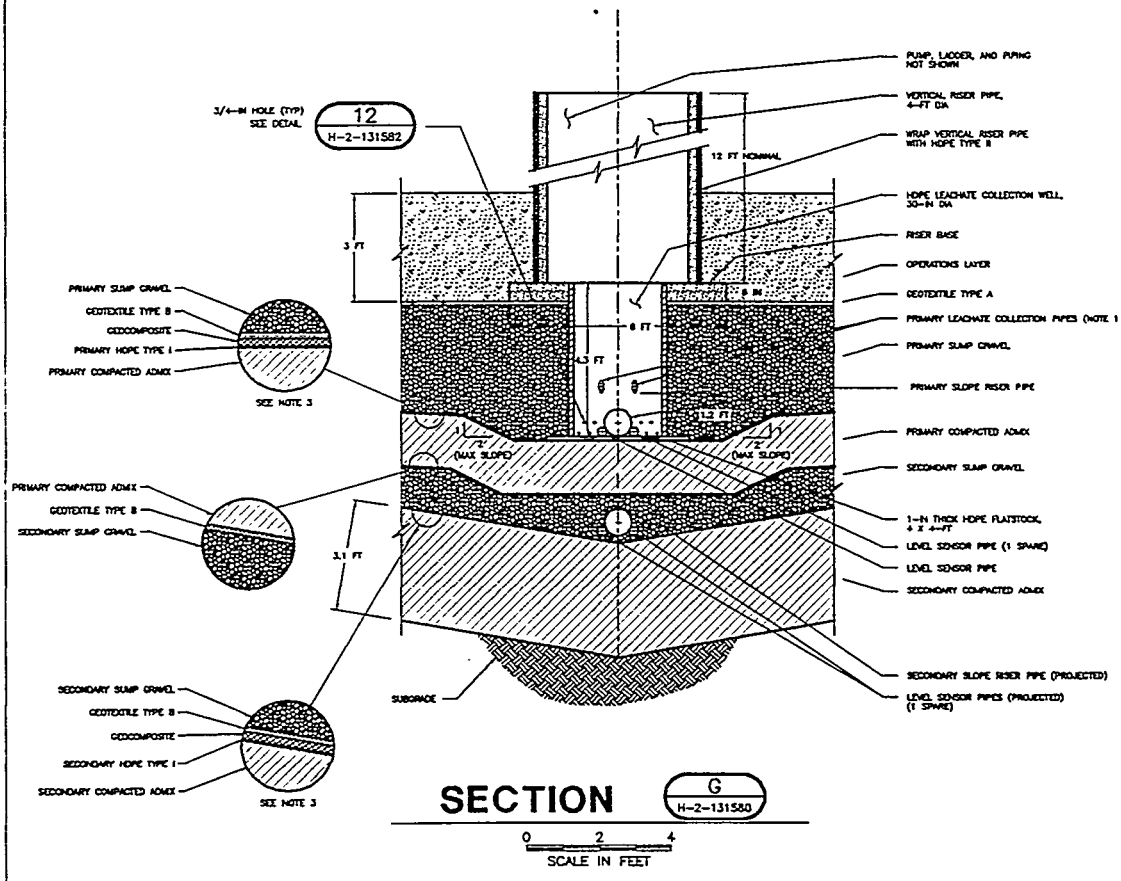
SCALE AS SHOWN — JOB NO. 8903-1166.035 — SHEET 1 OF 1

TITLE	NUMBER	TITLE
WING TRACEABILITY LIST	REFERENCES	
	NEXT USED ON	



SECTION E / H-2-131580

0 2 4
SCALE IN FEET



SECTION G / H-2-131580

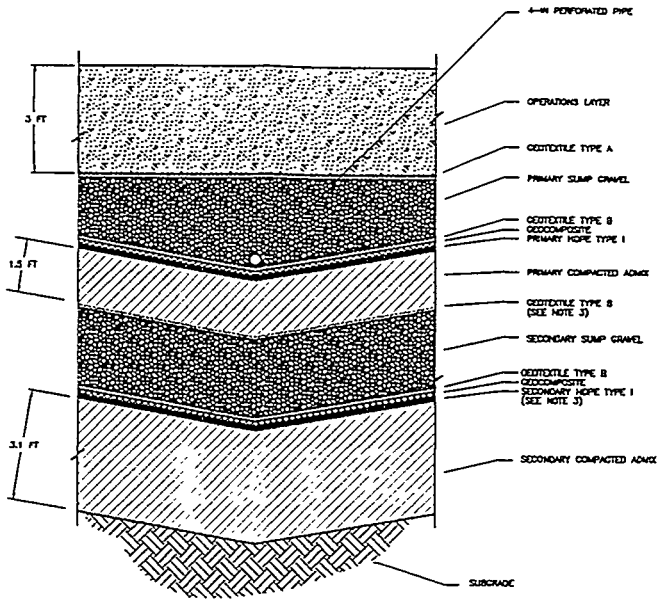
0 2 4
SCALE IN FEET

NOTES:

1. Pipes may be bent to match holes. Tack weld to HDPE collection well to secure.
2. Geosynthetic materials shall meet the requirements of Section 02275 of the Technical Specifications.
3. Type I HDPE and geocomposite drainage layer extend across entire sump area.
4. Round all trench corners as required.

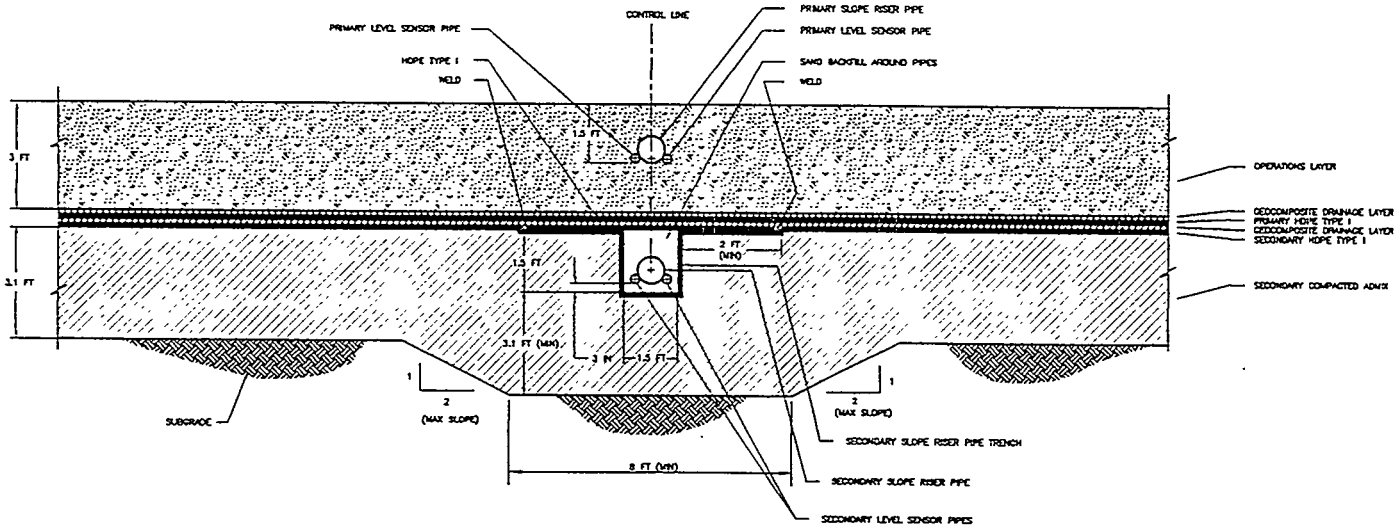
KEY:

G / H-2-131582 — DESIGNATION; SHEET NUMBER



SECTION F
H-2-131580

0 2 4
SCALE IN FEET



SECTION H
H-2-131580

0 2 4
SCALE IN FEET

FRANK S. BEDIKT
STATE OF WASHINGTON
PROFESSIONAL ENGINEER

NUMBER	TITLE
REFERENCES	

REV	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT PER ECN	SGH	F. SHURI	8/25/94
2	MINOR CHANGE	JPP	F. SHURI	5/21/93
1	MINIMUM OPERATING LAYER THICKNESS TO 3 FT. AND PRIMARY HDPE LAYER	KGH	F. SHURI	5/1/92

REVISIONS

DESIGNED	T. MARTIN	04/02/90
DRAWN	J. SNOW	04/02/90
CHECKED	F. SHURI	04/02/90
REVIEWED	P. CORSER	04/02/90
QA	M. J. OTTERSON	04/02/90
APPROVED FOR IMPLEMENTATION	BY	

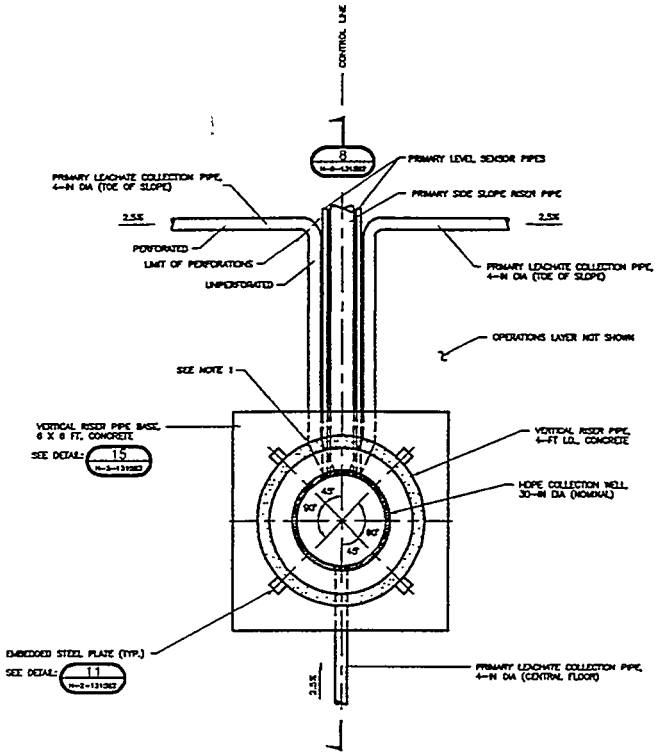
IMPACT LEVEL | ECF 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SUMP CROSS-SECTIONS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

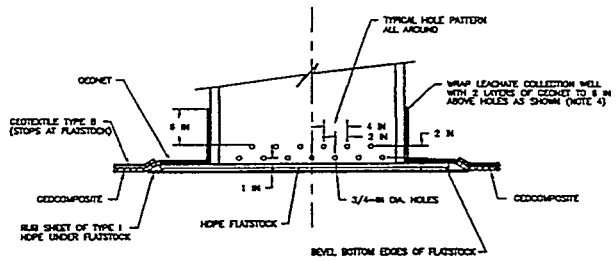
PROJECT W-025

SIZE	BLDG. NO.	MDLE NO.	DWG. NO.	REV. NO.
F	24-12-4	0401	H-2-131581	3



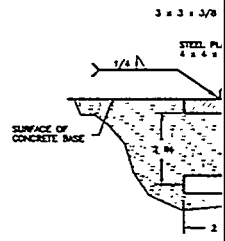
PRIMARY LEACHATE COLLECTION SUMP PLAN VIEW (10, H-2-131582)

0 2 4
SCALE IN FEET



LEACHATE COLLECTION WELL HOLE DETAIL (12, H-2-131582)

0 1 2
SCALE IN FEET



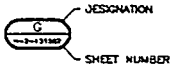
CVI SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
35		02725, 1.3.B	HDPE Pipe Fabrication Drawings
35A		02725, 1.3.B	HDPE Collection Well Fabrication Drawings

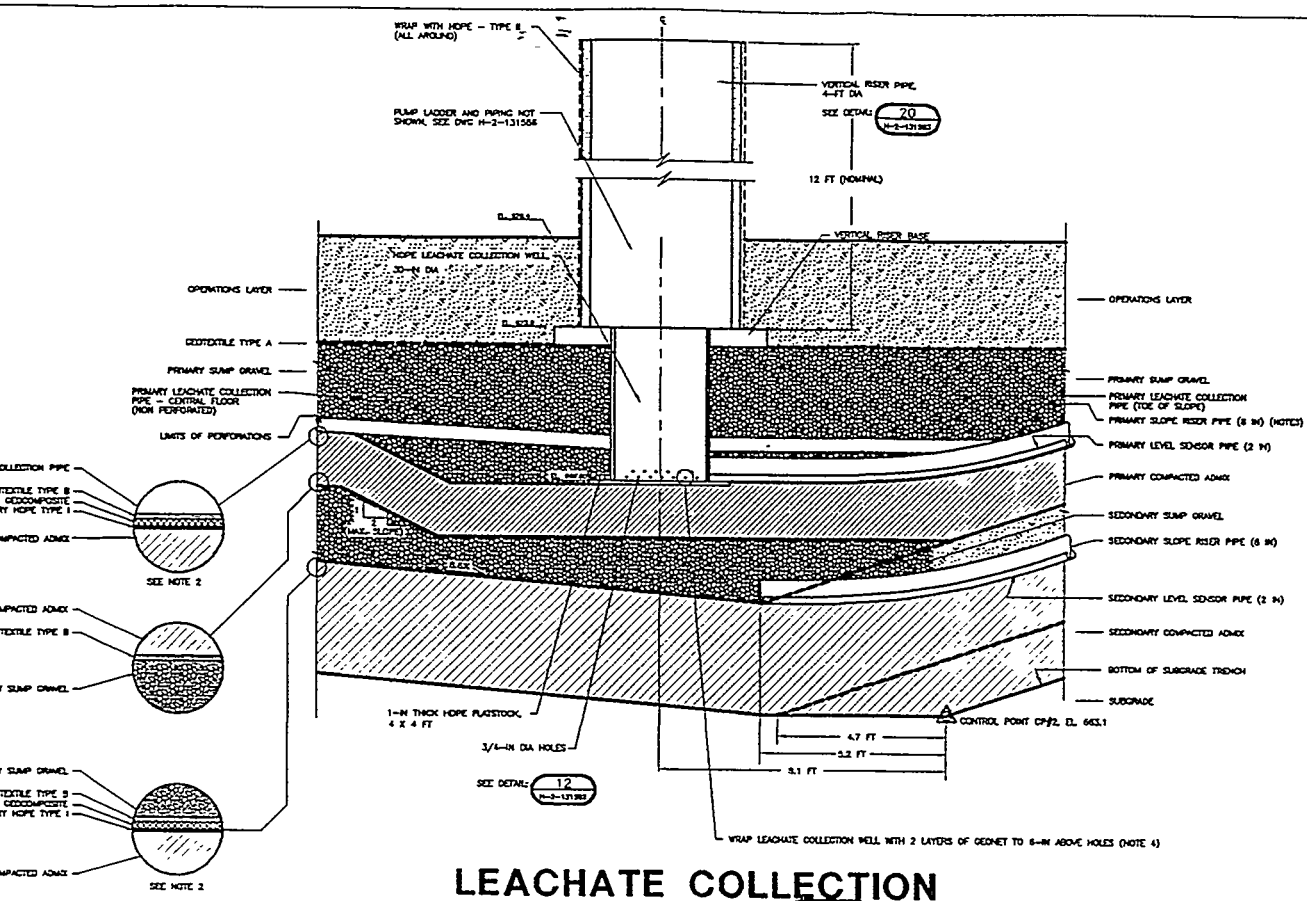
W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

NOTES:

1. Pipes may be bent to match holes. Tack weld to HDPE collection well to secure
2. Type I HDPE and geocomposite drainage layer extend across entire sump area.
3. Install primary submersible pump within 3 feet of end of slope riser pipe.
4. Offset layers of geonet so that ribs intermesh.

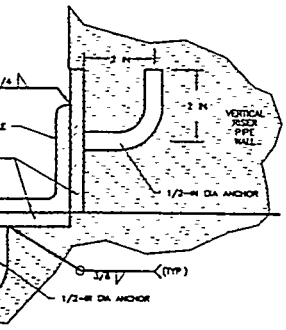
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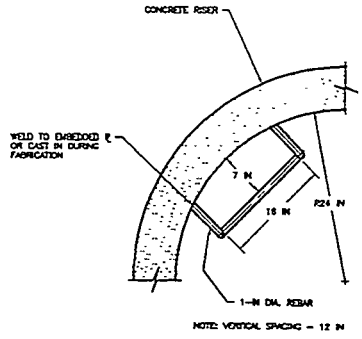


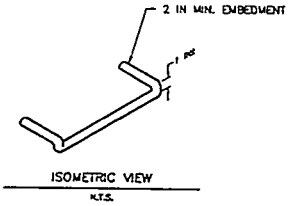


LEACHATE COLLECTION SUMP DETAIL 8 H-2-131582

0 2 4
SCALE IN FEET







ISOMETRIC VIEW
N.T.S.

TAIL 11 H-2-131582

0 2 4
SCALE IN INCHES

OSHA LADDER RUNG PLAN DETAIL 14 H-2-131582

0 10 20
SCALE IN INCHES

FRANK S. SHURI PROFESSIONAL ENGINEER

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT FOR SOW ADD OR CROSS HATCH FOR SOW H-2-131582 OPTIONAL FOR SOW H-2-131566, SOW H-2-131582	SGH	8/26/94	F. SHURI
2	MINOR EDITORIAL	UPP	5/21/93	F. SHURI
1	INCREASE OPERATIONS LAYER THICKNESS TO 2 FT ADD CERAMIC BOWL DETAIL	KGH	5/1/92	F. SHURI

REVISIONS

CAD FILE B131582A CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED T. MARTIN 04/02/90
DRAWN J. SHOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
QA M. J. OTTERSON 04/02/90
APPROVED FOR RELEASE BY FOR DATE

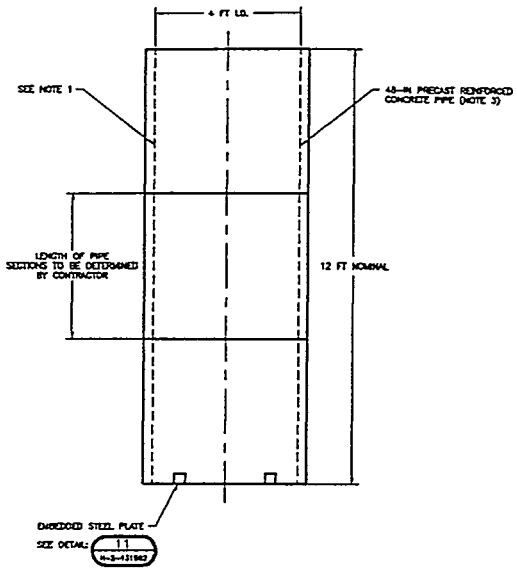
IMPACT LVL EST 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SUMP LEACHATE COLLECTION PIPES
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

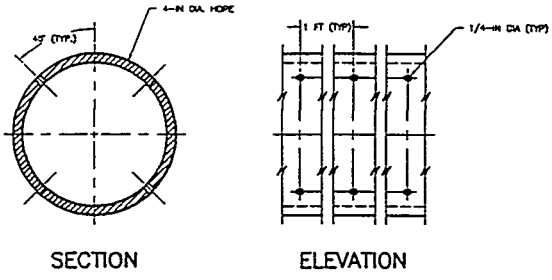
PROJECT W-025
SIZE F | BLDG. NO. 200-W-6 | MDCC NO. 0401 | DWG. NO. H-2-131582 | REV NO. 3
SCALE AS SHOWN | JOB NO. 893-1166.038 | SHEET 1 OF 1

TITLE | NUMBER | TITLE
REFERENCES
DRAWING TRACEABILITY LIST



VERTICAL RISER CONCRETE PIPE DETAIL (20, H-2-131583)

0 2 4
SCALE IN FEET



SECTION ELEVATION

PRIMARY DRAINAGE PIPE

SECTION

SECONDARY SLO

PIPE PERFORATION PATTERN DETAIL (19, H-2-131583)

N.T.S.

B-11

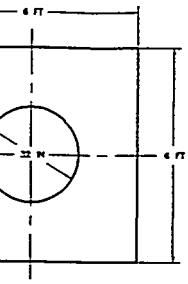
W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
37		02727, 1.2.A	Vertical Riser Pipe
37A		02727, 1.2.A	Precast Concrete Pipe
38		03210, 1.2.A	Rebar and Coating
48		09900, 1.6.A.2	Manhole Coating Manufacturer's Specification's
48A		09900, 1.6.A.2	Manhole Coating Manufacturer's Specification's

KEY: G (DESIGNATION) H-2-131583 (SHEET NUMBER)

NOTES:

1. Concrete to have protective coating. See Specifications, Section 09900.
2. See Specifications, Section 03210 for concrete reinforcement.
3. For vertical riser pipe, see Specifications, Section 02727.
4. Lifting hooks at discretion of contractor.
5. Pump shall be installed within 1.5 feet of end of slope riser pipe.



PLAN

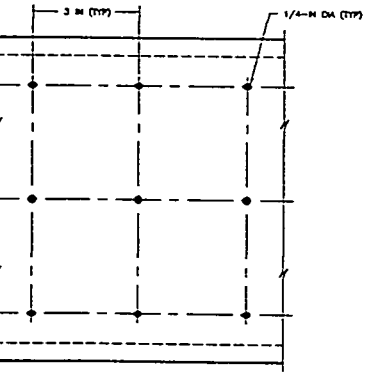


ECTION

ER REINFORCED SE DETAIL (15, H-2-131583)

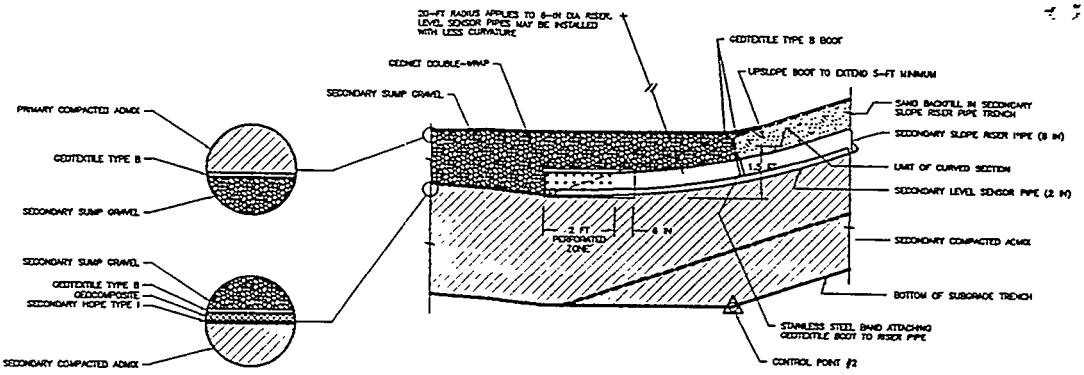
2 4

E IN FEET

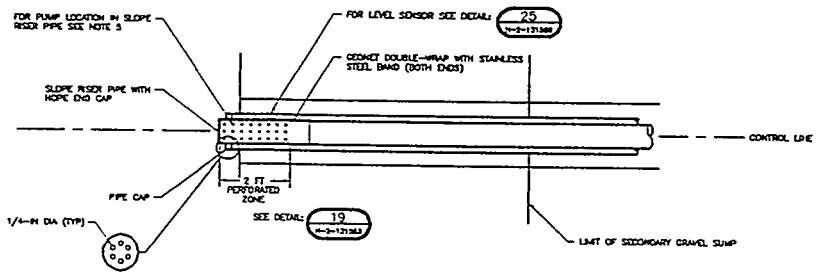


ELEVATION

RISER PIPE



ELEVATION

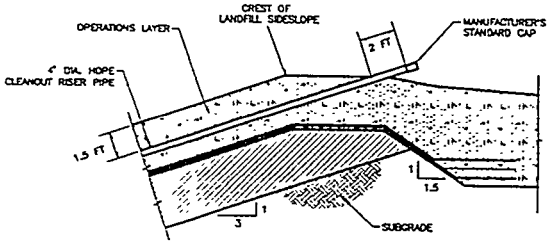


HOLE DETAIL
IN LEVEL SENSOR
PIPE CAP

PLAN

SECONDARY SLOPE RISER PIPE (21, H-2-131583)

N.T.S.



SIDESLOPE CLEANOUT RISER DETAIL (40, H-2-131583)

N.T.S.

FRANK J. SHURI — STATE OF ILLINOIS — PROFESSIONAL ENGINEER

NUMBER	TITLE
	REFERENCES
NG TRACEABILITY LIST	NEXT USED ON

NO.	DESCRIPTION	DATE	APPROVALS
3	[illegible] ADD DETAIL NO. ADD DA CROSS INDEX FOR RDA [illegible] EXTERNAL REF FOR H-2-131583, RDA H-2-131579	SGH	F. SHURI 10/6/94
2	MINOR EDITORIAL	JPP	F. SHURI 5/21/93
1	ADD PRIMARY ADMIX LAYER	KGH	F. SHURI 5/1/92

REVISIONS

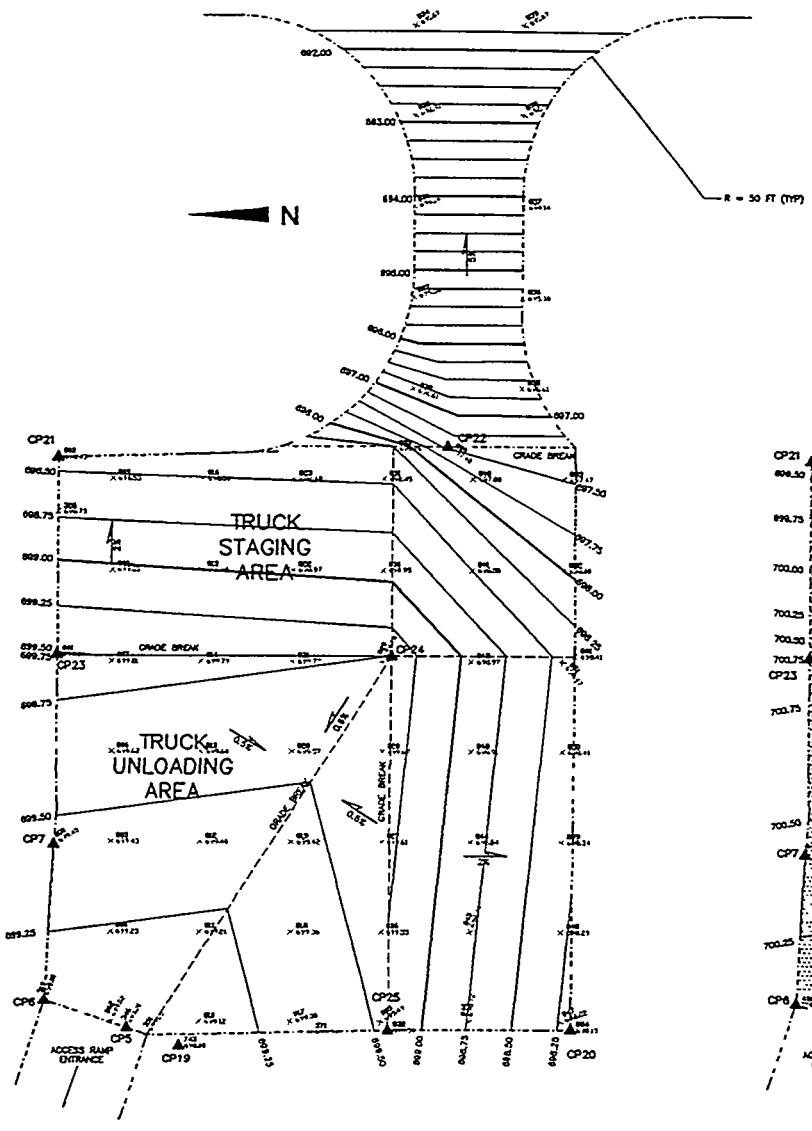
OLD FILE B131583A | OLD CODE 2W:IBM:ACD2:10:SS

IMPACT LEVEL	CIF 151914
DESIGNED	M. BROWN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / FOR / DATE

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SIDE SLOPE AND VERTICAL RISER PIPES
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

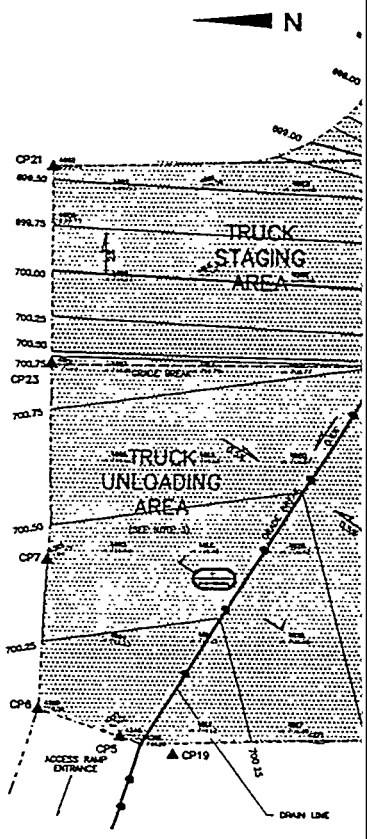
PROJECT W-025
SIZE F | BLDG. NO. 218-E-4 | MODEL NO. 0401 | DWG. NO. H-2-131583 | REV NO. 3
SCALE AS SHOWN | JOB NO. 983-1180.036 | SHEET 1 OF 1



GRADING PLAN OF TRUCK STAGING AND UNLOADING AREAS

16
H-2-131584

0 20 40
SCALE IN FEET



PAVED C OF TRUCK AND UNLOA

0 2
SCALE

D

W-025 LCRE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
31		02511, 1.5	Asphalt Mix Design
31.1		02511, 1.5	Asphalt Seal Coat

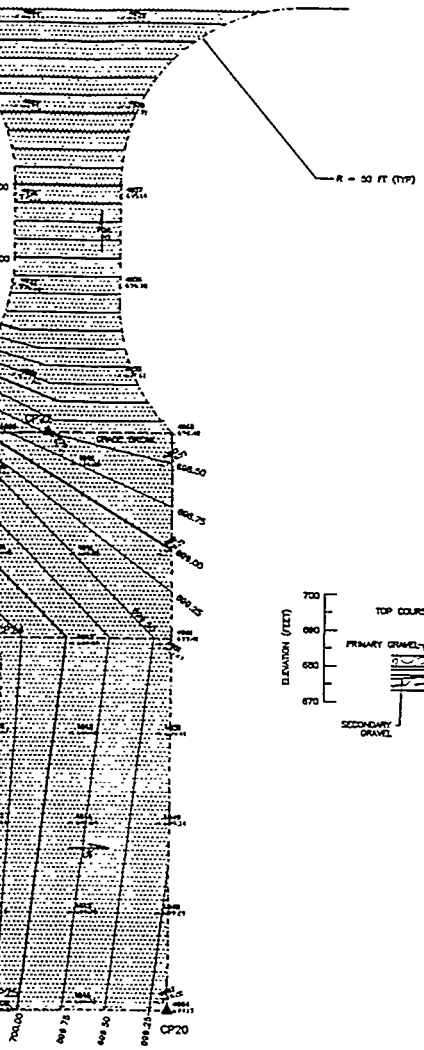
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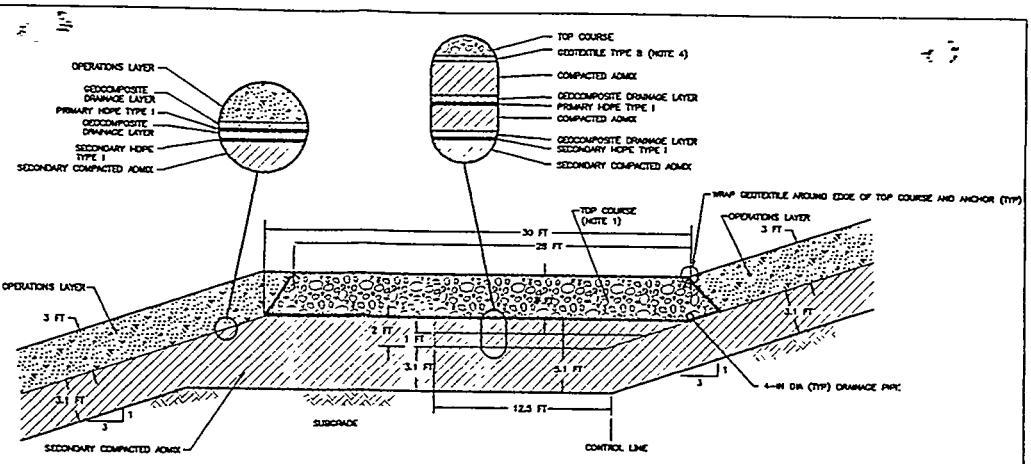
1. Asphalt and top course requirements are described in Section 02511 of the Technical Specifications.
2. Drainpipe requirements are described in Section 02725 of the Technical Specifications.
3. Truck unloading area to be lined and paved as shown in Section I. Truck Staging Area and entrance road to be paved with 8 inches of top course and 4 inches of asphalt concrete only.
4. Geotextile (Type B) to limits of top course only.
5. Geosynthetic material requirements are described in Section 02275 of the Technical Specifications.
6. Control point locations shown on Drawing H-2-131575.
7. For as-built conditions see drawings in the CQA report, WHC-SD-W025-RPT-001.

KEY:

D — DESIGNATION
H-2-131581 — SHEET NUMBER

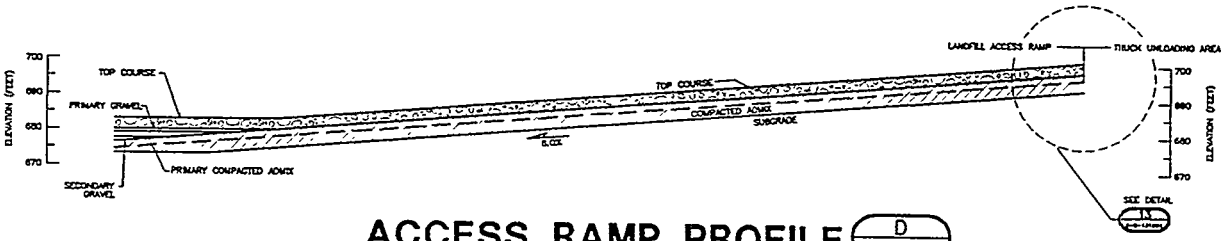
DAYTON AVENUE





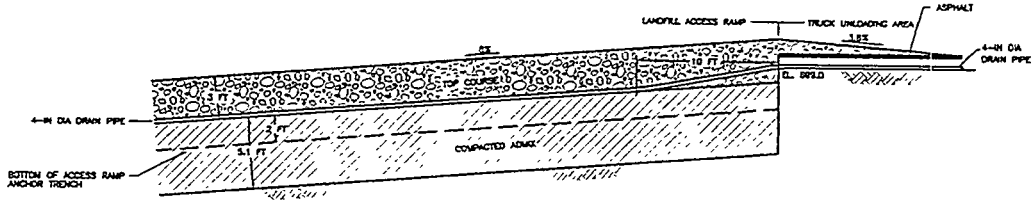
ACCESS RAMP SECTION (C, H-2-131584)

0 5 10
SCALE IN FEET



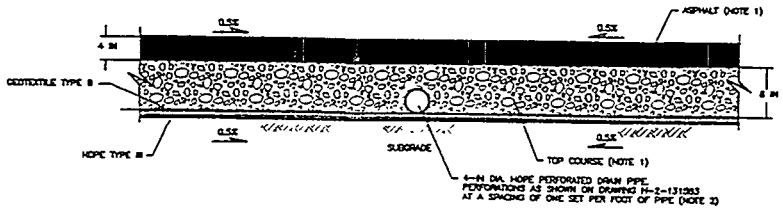
ACCESS RAMP PROFILE (D, H-2-131584)

0 20 40
SCALE IN FEET



ACCESS RAMP/TRUCK UNLOADING AREA DETAIL (13, H-2-131584)

0 5 10
SCALE IN FEET



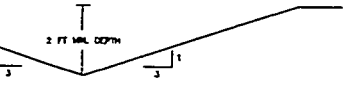
SECTION (1, H-2-131584)

0 1 2
SCALE IN FEET

FRANK S. SHURI PROFESSIONAL ENGINEER

NTOURS STAGING NG AREAS (35, H-2-131584)

40
FEET



TAIL (17, H-2-131584)

0 2 4
SCALE IN FEET

NO	DESCRIPTION	BY	DATE	APPROVALS
3	HDPE TRUCK ENTRANCE PER ECN 9-025-001	SCH	F. SHURI	8/26/94
2	HDPE TRUCK ENTRANCE PER ECN 9-025-001	JPP	F. SHURI	5/21/93
1	ADD ASPHALT CURB, LAYER AND TOP COURSE THICK. TO 2 FT.	KGH	F. SHURI	5/1/92

REVISIONS

DESIGNED T. MARTIN 04/02/90
DRAWN J. SHOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
QA M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION BY ____ FOR ____ DATE

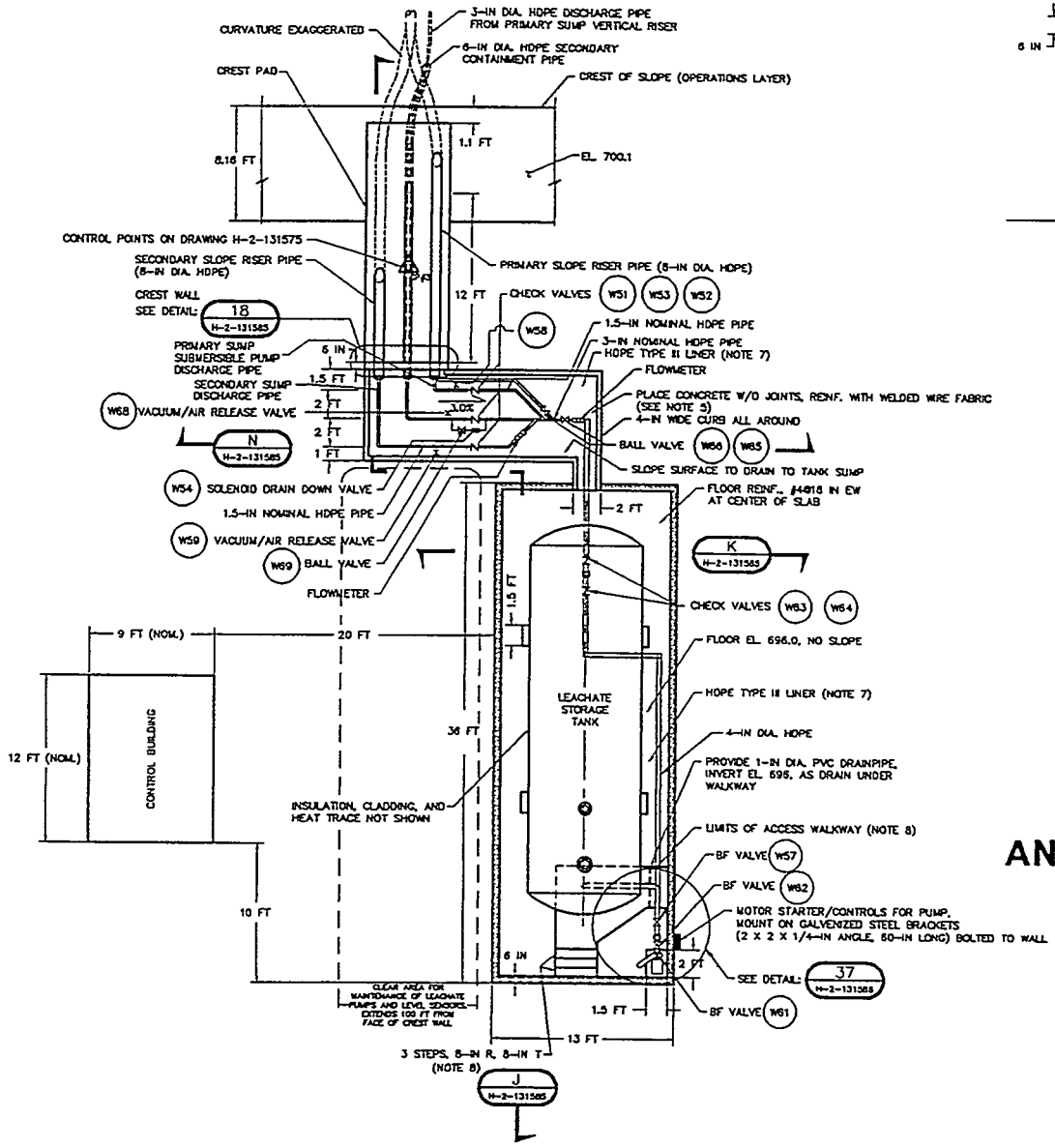
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.
TRUCK STAGING AREA & ACCESS RAMP
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

IMPACT LEVEL | EDT 151914
PROJECT W-025
SIZE F | BLDG. NO. 200-W-4 | MDCT NO. 0401 | DWG. NO. H-2-131584 | REV NO 3
SCALE AS SHOWN | JOB NO. 863-1166.036 | SHEET 1 OF 1

REFERENCES
TITLE | NUMBER | TITLE
NG TRACEABILITY LIST | MDCT USED ON

CAD FILE B131584A | CAD CODE 2W:IRM:ACD2:10:SS

H-2-131584 C 11



LEACHATE TEMPORARY STORAGE TANK & PIPING PLAN

0 5 10
SCALE IN FEET

N

NOTES:

1. Tank shall be ASME Code stamped for 25 psi design pressure.
2. All welds shall be continuous and ground smooth.
3. Tank paint systems shall meet the requirements of Specification Section 09900.
4. All connectors necessary for support of electrical conduits around tank shall be welded to tank prior to sandblasting and painting.
5. Welded wire fabric shall meet the requirements of Specification Section 03210.
6. Utilize bond breaker around all pipes through crest wall.
7. Weld HDPE Type III to form continuous watertight liner on floor and sides of concrete containment areas. Details of liner around tank saddles to be provided by contractor and shown on fabrication drawings.
8. Construct steps and access walkway over HDPE liner. Do not damage liner.
9. Heat trace and insulate all exposed piping in accordance with specifications.

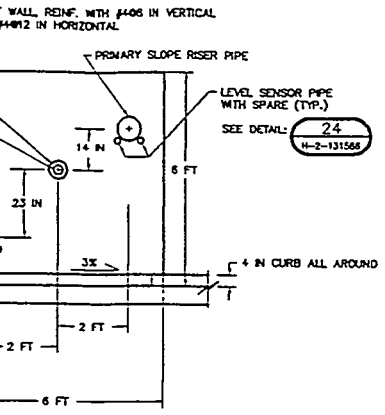
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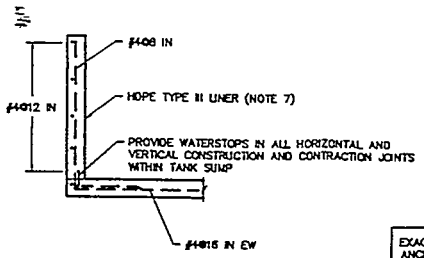
J — DESIGNATION
H-2-131565 — SHEET NUMBER

W57 VALVE AND PUMP IDENTIFICATION

ANC

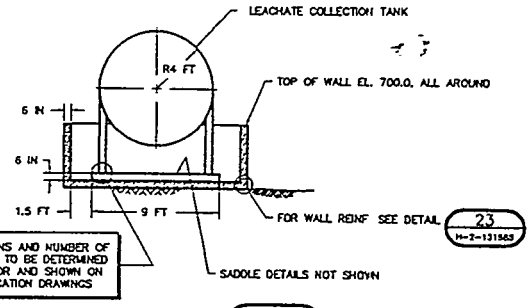
B-13





DETAIL 23 (H-2-131585)

0 2 4
SCALE IN FEET

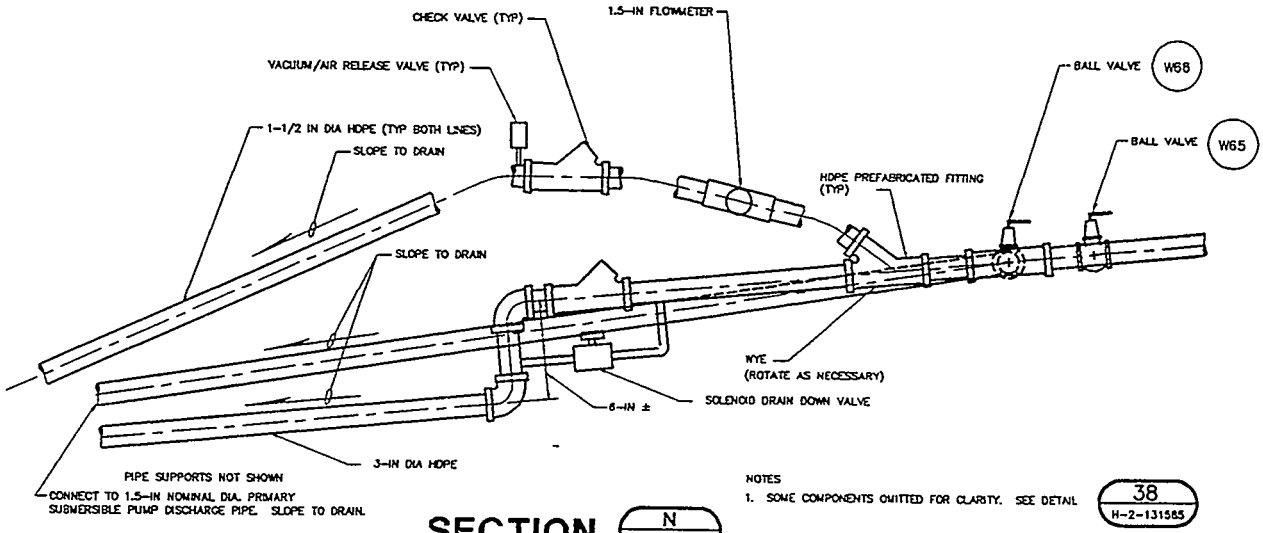


SECTION K (H-2-131585)

0 5 10
SCALE IN FEET

ETAIL 18 (H-2-131585)

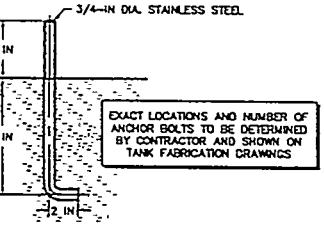
0 2 4
SCALE IN FEET



NOTES
1. SOME COMPONENTS OMITTED FOR CLARITY. SEE DETAIL 38 H-2-131585

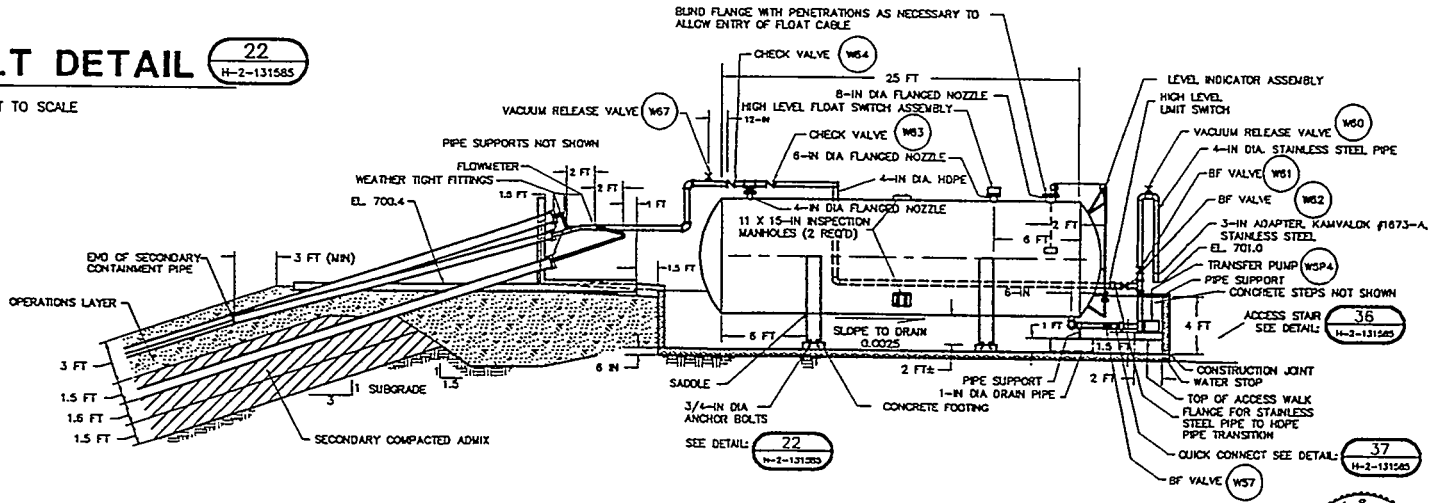
SECTION N (H-2-131585)

NOT TO SCALE
SLOPE EXAGGERATED FOR CLARITY



OR BOLT DETAIL 22 (H-2-131585)

NOT TO SCALE



LEACHATE TEMPORARY STORAGE TANK & PIPING

SECTION J (H-2-131585)

0 5 10
SCALE IN FEET

FRANK S. SHURI — STATE OF IDAHO — PROFESSIONAL ENGINEER

NO.	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT PER ECN; REVISE FLOWRATES PER ECN 9-025-031; ADD HEAT TRACE DETAILS PER ECN 9-025-029 (SEE SHEET 2); REVISE PIPING LAYOUT PER ECN 9-025-023; ADD REQUIREMENT FOR HDPE TRACE AND INSULATION, VALVE VAULT PER ECN 9-025-023; ADD SUMP CROSS-CONNECT PIPING, LEACHATE RECIRCULATION LINE PER ECN 9-025-018; EDITORIAL PER ECN 9-025-018, ECN 9-025-029	SJS	F. SHURI	10/6/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	INCREASE OPERATIONS LAYER THICKNESS TO 3 FOOT; ADD NOTE THAT 6 LAYER TO GEOSYNTHETIC LAYER	JPP	F. SHURI	5/1/92

REVISIONS

DESIGNED	M. BROWN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / FOR / DATE

IMPACT LEVEL: ET 151914

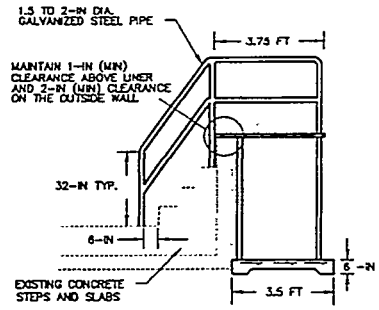
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

LEACHATE COLLECTION TANK & PIPING
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

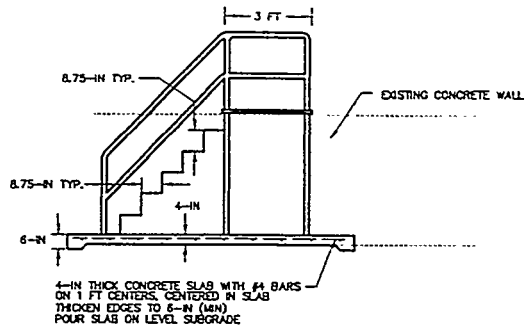
PROJECT W-025
SIZE F | BLDG. NO. 216-B-6 | MODEL NO. 0401 | DWG. NO. H-2-131585 | REV. NO. 3
SCALE AS SHOWN | JOB NO. 593-1166.036 | SHEET 1 OF 2

OLD FILE B131585A | CAD CODE 2W:IBM:ACD2:10:SS

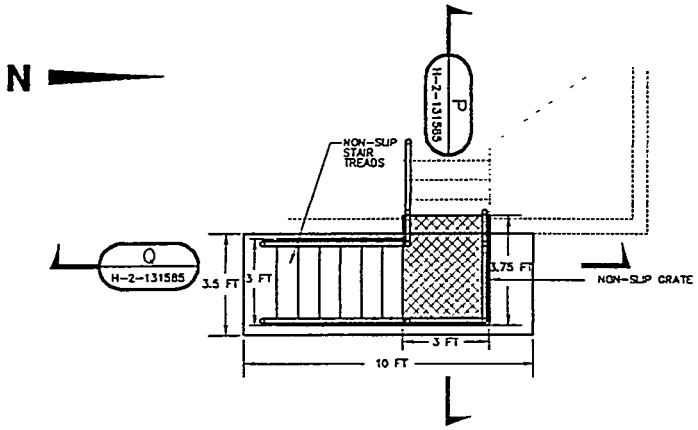
TITLE	NUMBER	TITLE
WING TRACEABILITY LIST	REFERENCES	NEXT USED ON



SECTION P H-2-131585



SECTION Q H-2-131585

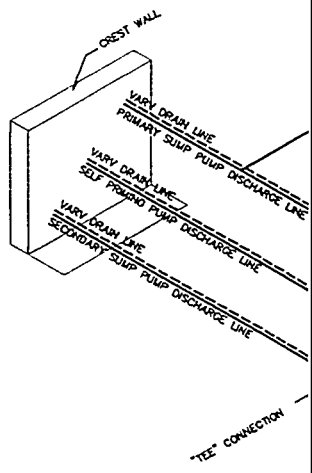


CONTAINMENT BASIN ACCESS STAIR PLAN

36 H-2-131585

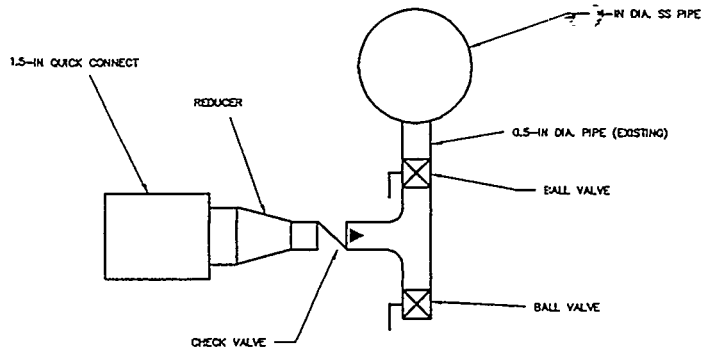
0 10

SCALE IN FEET



W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

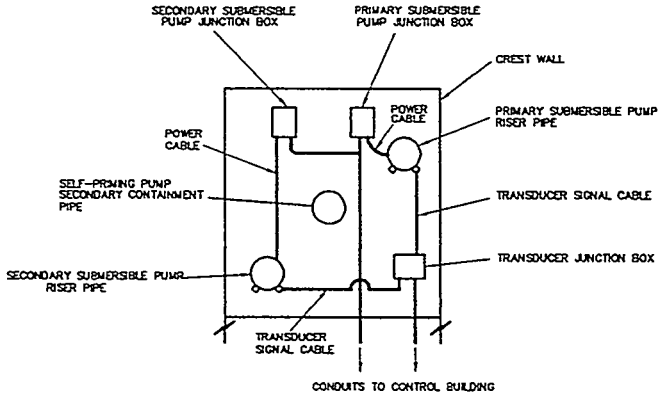
SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
39		03310, 1.2.B	Rebar Shop Drawings
40		03310, 1.2.A	Air-entraining Admix Certification
41		03310, 1.2	Concrete Mix Design
42		03310, 1.2	Anchor Bolts
43		03310, 1.2	Waterstops
47		09900, 1.6.A.1	Paint Coating Systems List
48.1		09900, 1.6.A.2	Tank Coating Manufacturer's Specifications
53		13205, 1.3.A	Leachate Tank Fabrication Drawings
53A		13205, 1.3.A	Leachate Tank Fabrication Drawings
53B		13205, 1.3.A	Leachate Tank Liquid Level Indicator
53.1		13205, 1.3.A	Leachate Tank Fabrication Drawings - Vent
53.2		13205, 1.3.A	Leachate Tank Fabrication Drawings - Elbow
53.3		13205, 1.3.A	Leachate Tank Fabrication Drawings
53.4		13205, 1.3.A	Leachate Tank Fabrication Drawings - Letter
53.5		13205, 1.3.A	Leachate Tank Fabrication Dwgs. - ASME Cert.
53.6		13205, 1.3.A	Leachate Tank Fabrication Drawings
57		15060, 1.4.A	Discharge Pipe - Shop Drawings
58		15060, 1.4.B	Leachate Tank Fabrication Drawings - Shell
59		15060, 1.4.C	Stainless Steel Pipe Certification
60		15166, 1.2.A	Hangers, Brackets, Supports - Technical Data
61		15066, 1.2.B	Pipe and Conduit Support System - Calculations
62		15066, 1.2.C	Pipe and Conduit Support System - Drawings
63		15100, 1.3.A	Vacuum Release Valves
64		15100, 1.3.A	Solenoid Drain Valve and Piping
64.1		15100, 1.3.A	Solenoid Drain Valve and Piping
65		15100, 1.3.A	Sample Cocks
66		15100, 1.3.A	Check Valves
67		15100, 1.3.A	Automatic Drain Valve - Manuf. Information
68		15104, 1.2.A	Butterfly Valves
68A		15104, 1.2.A	Butterfly Valves
69		15150, 1.2.A	Flowmeter
		15150, 1.3.A	Flowmeter - Installation, Operation, and Maintenance Manual
70		15250, 1.3.A	Leachate Tank and Pipe Insulation - Manuf. Info.
71		15250, 1.3.B	Leachate Tank and Pipe Insulation - Certification



QUICK CONNECT SECTION (0 / H-2-131585)

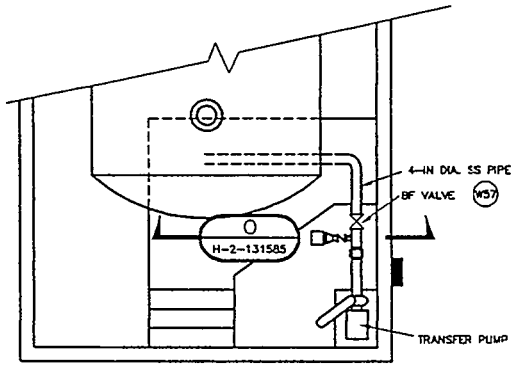
NOT TO SCALE

NOTE: INSULATE EXTENSION PIPES, CHECK VALVE, BALL VALVES, AND QUICK CONNECT.



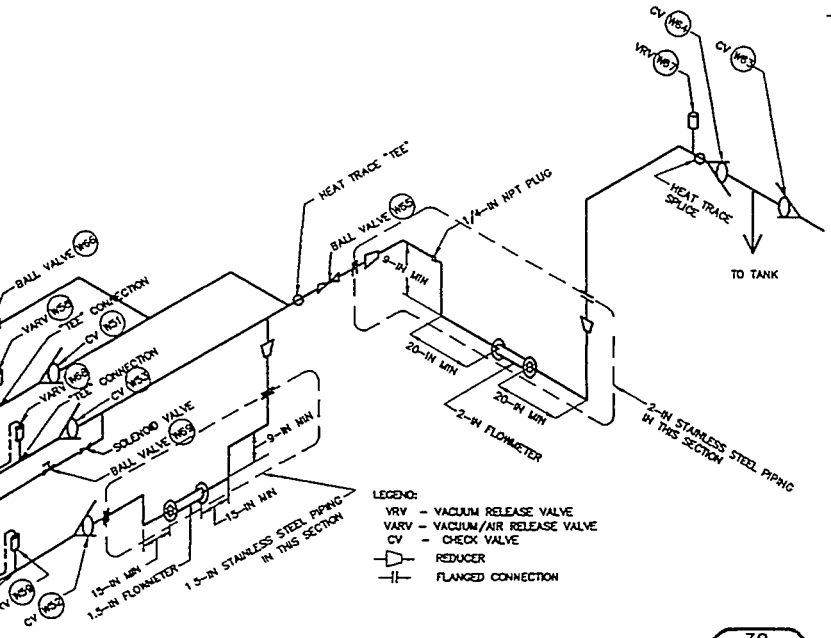
CREST WALL CABLE LAYOUT (39 / H-2-131585)

NOT TO SCALE



HOSE CONNECTION DETAIL (37 / H-2-131585)





DISCHARGE PIPING SCHEMATIC (38 / H-2-131585)

NOT TO SCALE

FRANK S. SHURI — STATE OF WASHINGTON — PROFESSIONAL ENGINEER

EXPIRES 2-11-91

				IMPACT LEVEL	EDT EDN W-025-026
				DESIGNED M. BROWN 04/02/90	U. S. DEPARTMENT OF ENERGY RICHLAND OPERATIONS OFFICE
				DRAWN J. SNOW 04/02/90	GOLDER ASSOCIATES INC.
				CHECKED F. SHURI 04/02/90	LEACHATE COLLECTION TANK & PIPING
				REVIEWED P. CORSER 04/02/90	RMW LAND DISPOSAL FACILITY
				QA M. J. OTTERSON 04/02/90	NON-DRAG-OFF
					PROJECT W-025
				APPROVED FOR IMPLEMENTATION BY	SIZE F; BLDG. NO. 719-0-6; MDCC NO. 0401; DWG. NO. H-2-131585; REV NO 0
				FOR DATE	SCALE AS SHOWN; JOB NO. 803-1186.036; SHEET 2 OF 2

NO	DESCRIPTION	BY	DATE	APPROVALS
0	AS-BUILT PER EDN	SJS	10/06/94	F. SHURI

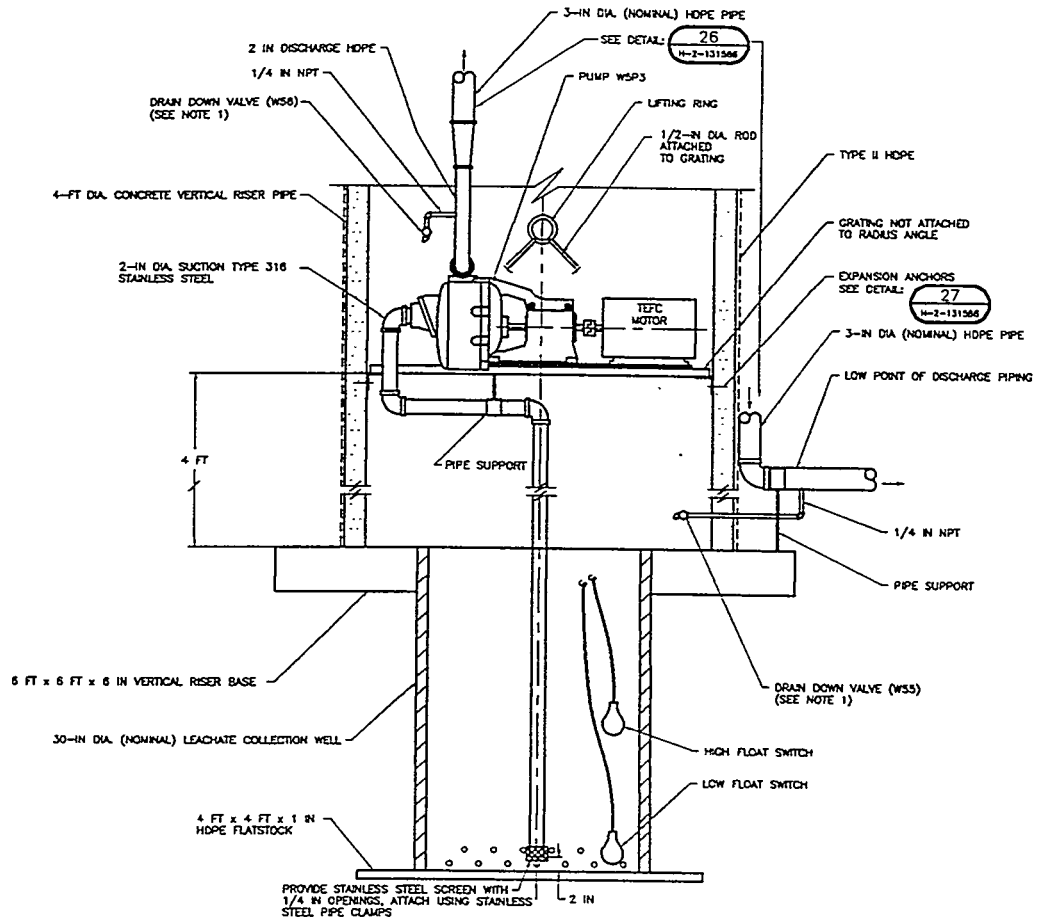
REVISIONS

NUMBER	TITLE	REFERENCES

TITLE	NEXT USED ON

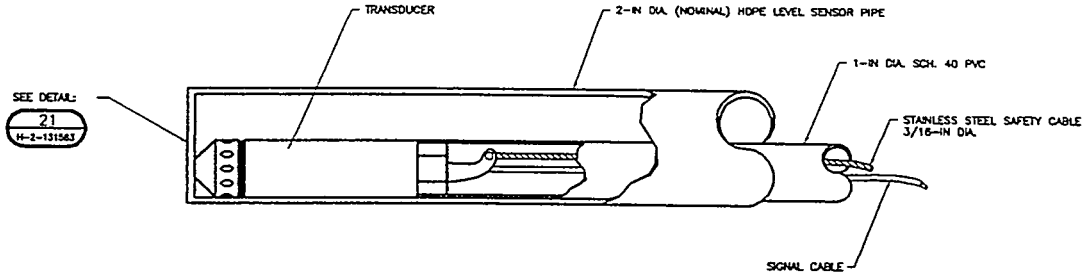
WING TRACEABILITY LIST

CAD FILE B131585B; CAD CODE 2W:IBM:ACD2:10:SS



PRIMARY LEACHATE SELF PRIMING PUMP (W5P3) SECTION L H-2-131585

0 10 20
SCALE IN INCHES



LEVEL SENSOR TRANSDUCER DETAIL 25 H-2-131583

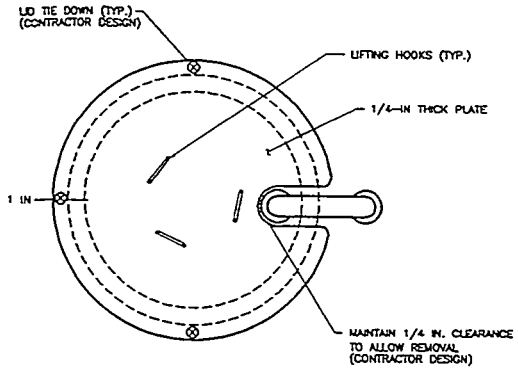
NOT TO SCALE

NOTES:

1. Requirements for drain down valve and associated piping are described in Section 15100 of the Technical Specifications.
2. Requirements for float switch housing are described in Section 13340 of the Technical Specifications.
3. Signal cables from transducers shall be terminated in junction box. Identical wires shall continue from terminal strip to control building.

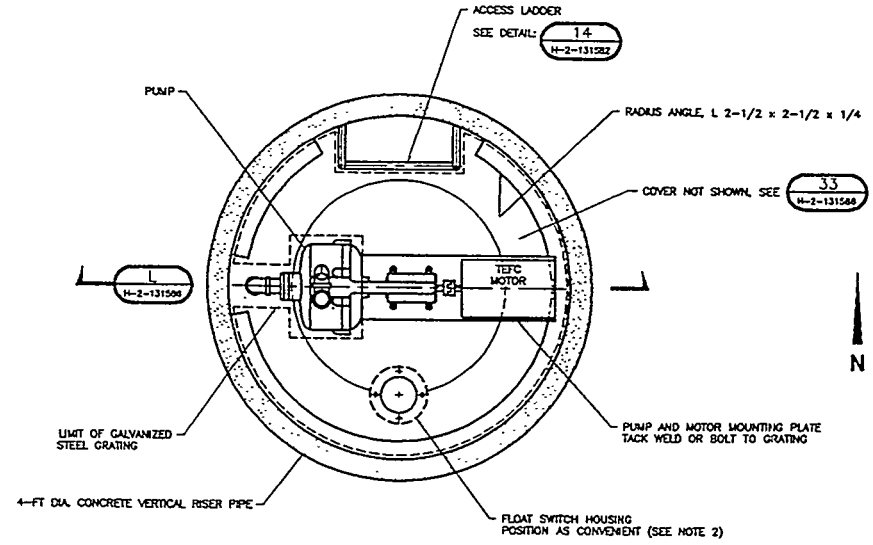
W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUB#	SPEC REF.	DESCRIPTION
44		05500, 1.2	Vertical Riser Lid and Grating Fab. Drawings
50		11210, 1.2A	Pumps, Motors, Materials Information
50.1		11210, 1.2A	Pumps, Motors, Materials Information
50.1A		11210, 1.2A	Pumps, Motors, Materials Information
		11210	Transfer Pump - Installation, Operation, and Maintenance Instructions; Parts and Disassembly Instructions
		11210	Submersible Pump - Installation and Operation Instructions
		11210, 1-2, A; 2-1	Submersible Pump - Performance Curves
		11210	Self-Priming Pump - Installation, Operation, and Maintenance Instructions; Spare Parts List; Maintenance Manual
		11210, 1-2	Volume-Depth Curve: Primary Sump
300		11210	Final Self-Priming Pump Piggy-Back Arrangement



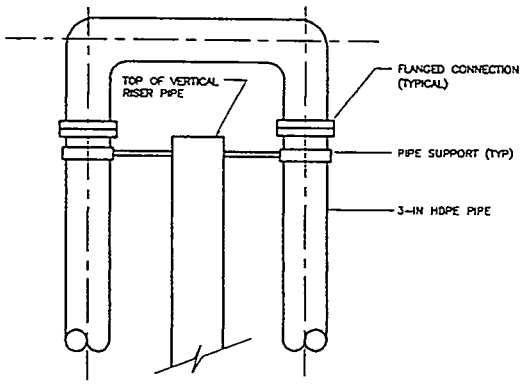
VERTICAL RISER PIPE COVER PLAN DETAIL (33 / H-2-131586)

NOT TO SCALE



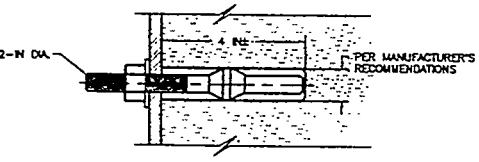
PRIMARY LEACHATE SELF PRIMING PUMP PLAN VIEW

0 10 20
SCALE IN INCHES



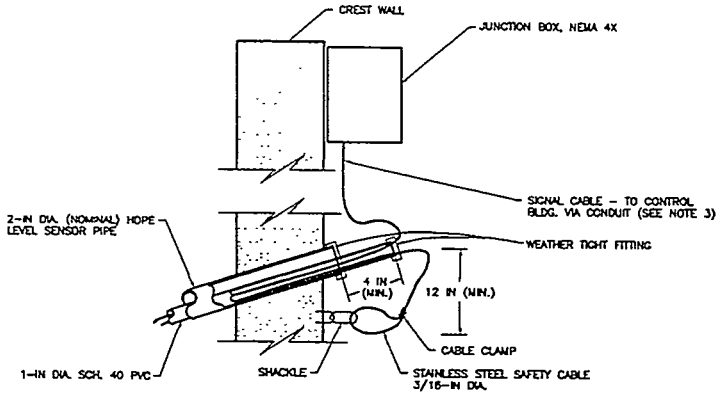
DETAIL (26 / H-2-131586)

NOT TO SCALE



PICAL EXPANSION ANCHOR DETAIL (27 / H-2-131586)

NOT TO SCALE



TERMINATION OF LEVEL SENSOR PIPE AT CREST WALL DETAIL (24 / H-2-131585)

0 5 10
SCALE IN INCHES

RENATE S. SHURI
STATE OF WASHINGTON
PROFESSIONAL ENGINEER

5 — DESIGNATION
H-2-131579 — SHEET NUMBER

NUMBER	TITLE
FILE	REFERENCES
WING TRACEABILITY LIST	NOT USED ON

NO	DESCRIPTION	BY	DCE	APPROVALS
3	AS-BUILT P.E. SEAL ADD CH CT 135 INDEX PER ECN W-025-003 ADD REPLACEMENT FOR JUNCTION BOX PER ECN W-025-004 SYSTEMS PER ECN W-025-005, ECN W-025-007	SCH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	MINOR EDITORIAL	JPP	F. SHURI	5/1/92

REVISIONS

WF — ADD ON FILE: B131586A — CHK CODE: 2W;IBM;ACD2;10;SS

DESIGNED	M. BROWN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / FOR / DATE

IMPACT LEVEL — CCF 151914

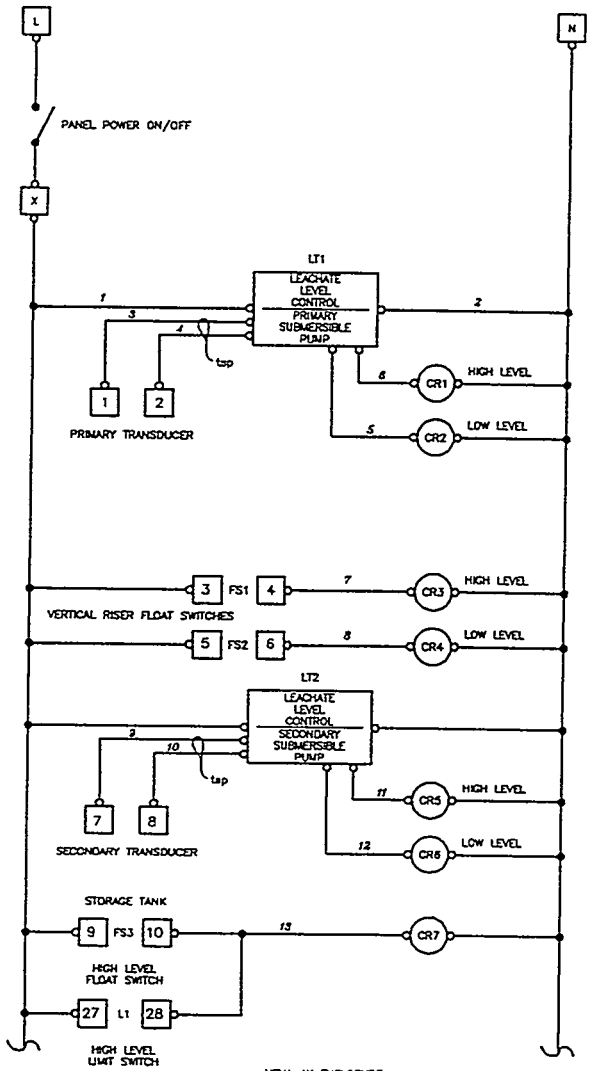
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

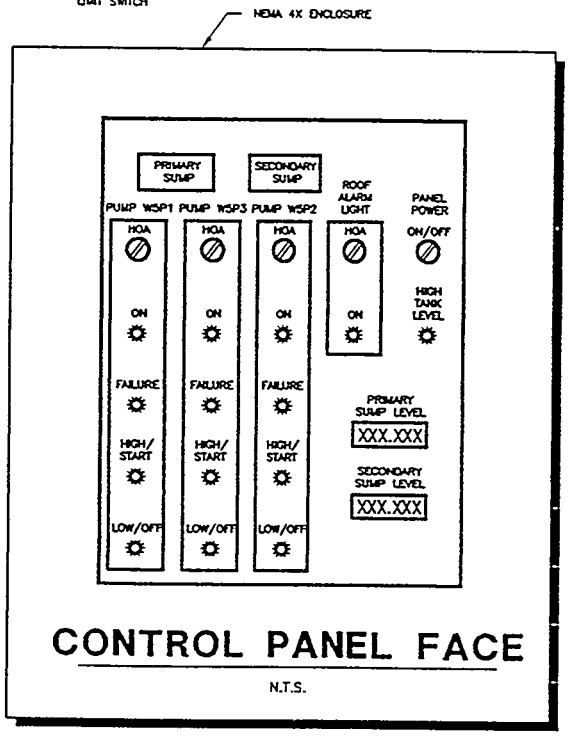
SUMP PUMP DETAILS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025

SIZE	BLDG. NO.	MODE NO.	DWG. NO.	REV NO.
F	219-H-4	0401	H-2-131586	3

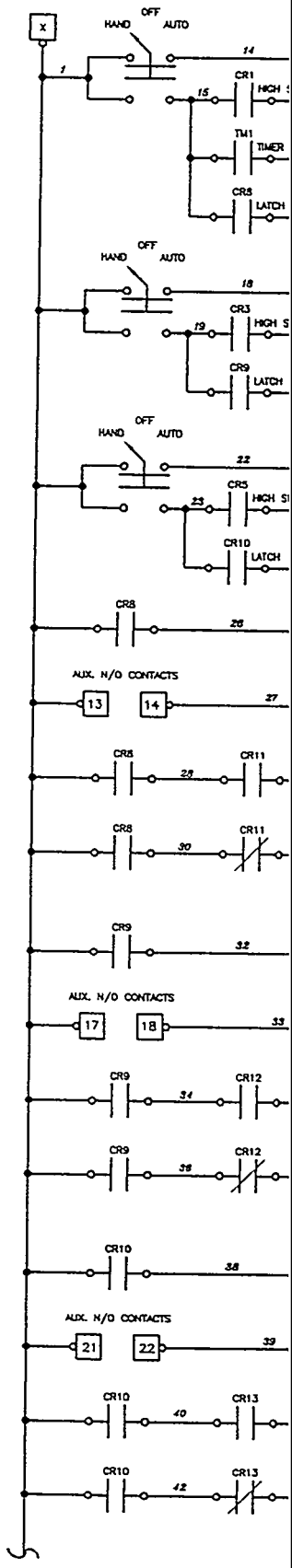
SCALE AS SHOWN — JOB NO. 803-1186.036 — SHEET 1 OF 1





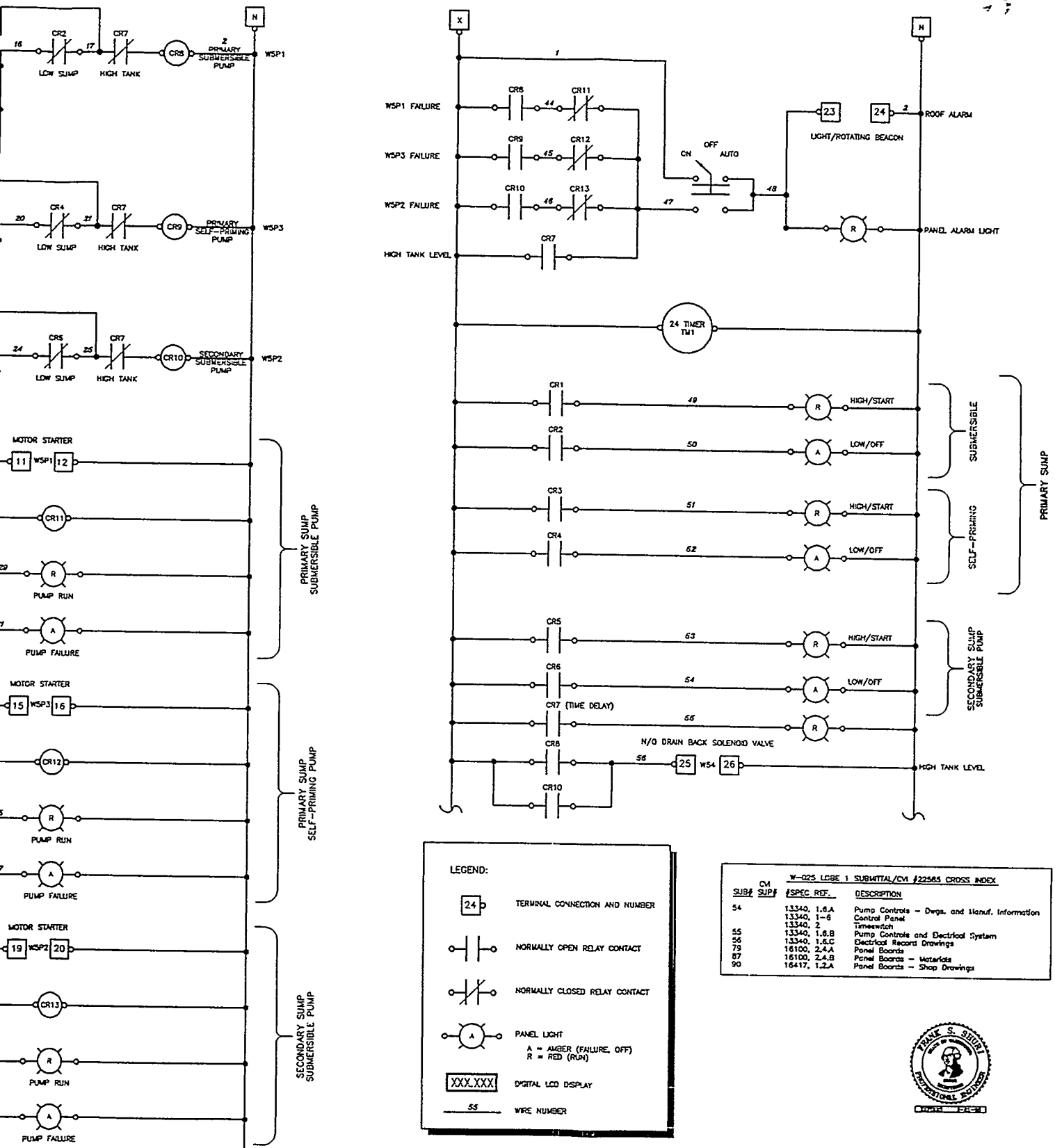
CONTROL PANEL FACE

N.T.S.



NOTES:

1. For as-built configuration see "Pump Control Wiring Diagram" in the COA Report, WHC-SD-W025-RPT-001.



LEGEND:

- 24 — TERMINAL CONNECTION AND NUMBER
- NORMALLY OPEN RELAY CONTACT
- NORMALLY CLOSED RELAY CONTACT
- A — PANEL LIGHT (A = AMBER (FAILURE, OFF); R = RED (RUN))
- XXX.XXX — DIGITAL LCD DISPLAY
- 55 — WIRE NUMBER

W-025 LCBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
54		13340, 1.6.A	Pump Controls – Dwgs. and Manuf. Information
		13340, 1-6	Control Panel
		13340, 2	Timeswitch
55		13340, 1.6.B	Pump Controls and Electrical System
56		13340, 1.6.C	Electrical Record Drawings
79		16100, 2.4.A	Panel Boards
87		16100, 2.4.B	Panel Boards – Materials
90		16417, 1.2.A	Panel Boards – Shop Drawings

FRANK S. SHURI — PROFESSIONAL ENGINEER

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	EXTERNAL PER ECN 9-025-005, ECN 9-025-029	SGH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	ADD WIRE, TERMINAL NOS., DET PANEL LIGHT	JPP	M. BROWN	5/1/92

REVISIONS

CAD FILE B131587A — CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED M. BROWN 04/02/90
DRAWN J. SNOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
DA M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION BY ____ FOR ____ DATE

IMPACT LEVEL — CVI 151214

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

ELECTRICAL – CONTROL
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F — BLDG NO. 5903 — DWG NO. H-2-131587 — REV NO 3
SCALE NONE — JOB NO. 193-1166.036 — SHEET 1 OF 4

ENG TRACEABILITY LIST — TITLE — NUMBER — REFERENCES — NOT USED ON

CONDUIT SCHEDULE

CONDUIT NUMBER	DESCRIPTION
W25 W5001	FLOAT SWITCHES IN VERTICAL RISER
W25 W5002	POWER TO SELF PRIMING PUMP WSP3
W25 W5003	POWER TO SUBMERSIBLE PUMPS WSP1 AND WSP2
W25 W5004	SIGNAL WIRES TO PRIMARY AND SECONDARY LEVEL SENSOR TRANSDUCERS
W25 W5005	POWER TO SOLENOID VALVE WS4
W25 W5006	FLOAT SWITCH IN LEACHATE TEMPORARY STORAGE TANK
W25 W5007	POWER TO TRANSFER PUMP WSP4
W25 W5008	HEAT TRACE POWER

SEE DETAIL: 28 / H-2-131587

CONTROL BUILDING

POLE

SEE DETAIL: 29 / H-2-131587

OH

OH

LEACHATE TEMPOR
SCHEMATIC C

SCALE

N

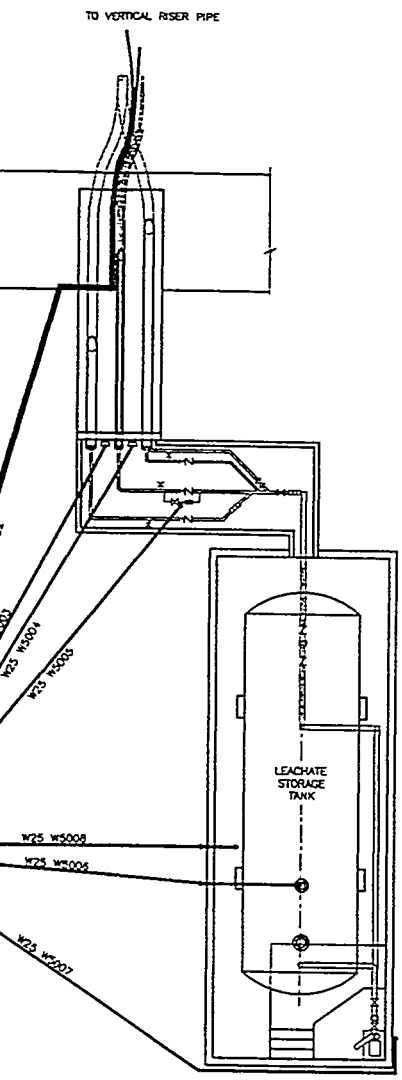
W-025 LOBE 1 SUBMITTAL/CVI #22583 CROSS INDEX

SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
46		08100, 1.2.A	Control Building Door Manuf. Information
46.1		08100, 1.2.A	Control Building Door Frame Manuf. Information
51		13120, 1.3.A	Control Building, Hardware Manuf. Information
51.1		13120, 1.3.A	Control Building Foundation
51.2		13120, 1.3.A	Control Building, Hardware Manuf. Information
52		13120, 1.3.B	Control Building Design Load

KEY:

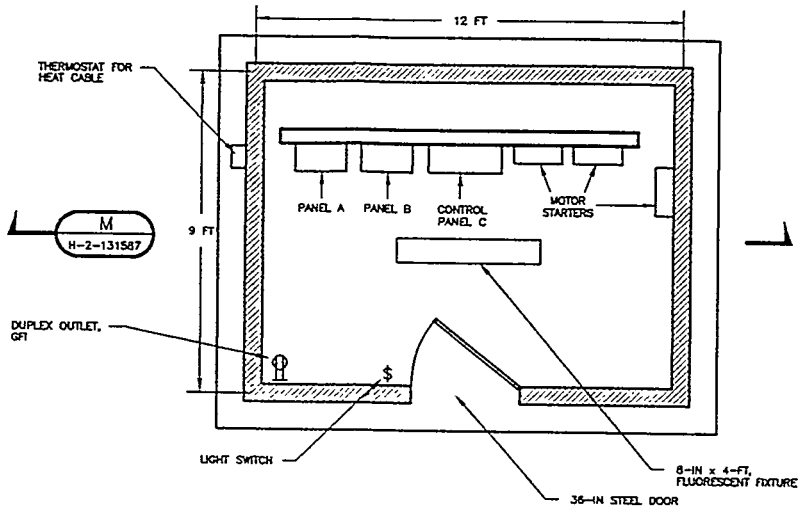
DESIGNATION

SHEET NUMBER



RY STORAGE TANK
NDUIT LAYOUT

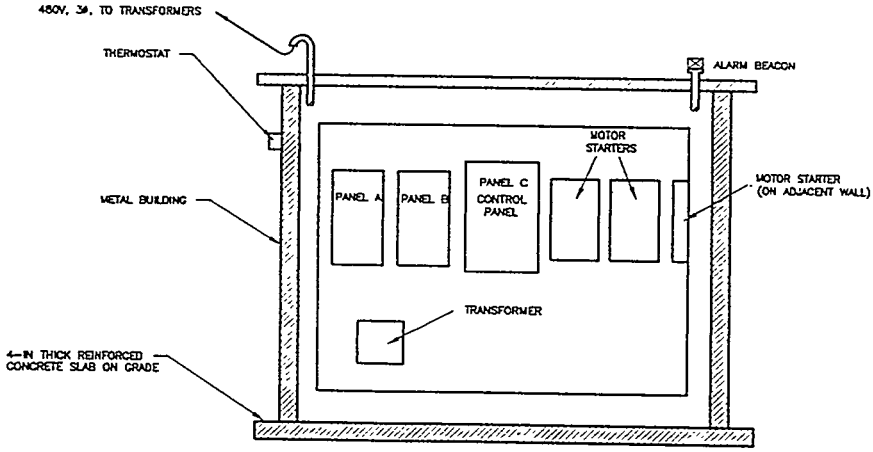
10
FEET



CONTROL BUILDING DETAIL

28
H-2-131587

N.T.S.



CONTROL BUILDING SECTION

M
H-2-131587

N.T.S.



REV NO	DESCRIPTION	BY	DATE	APPROVALS
4	AS-BUILT PER EDNs [illegible]	DWM	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	MODIFY CONDUIT LAYOUT ADD THERMOSTAT AND PHOTOCELL	JPP	F. SHURI	5/1/92

REVISIONS

CAD FILE B131587B | CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED	M. BROWN 04/02/90
DRAWN	J. SNOW 04/02/90
CHECKED	F. SHURI 04/02/90
REVIEWED	P. CORSER 04/02/90
QA	M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION	BY / DATE / FOR

IMPACT LVL: | EDT 151914

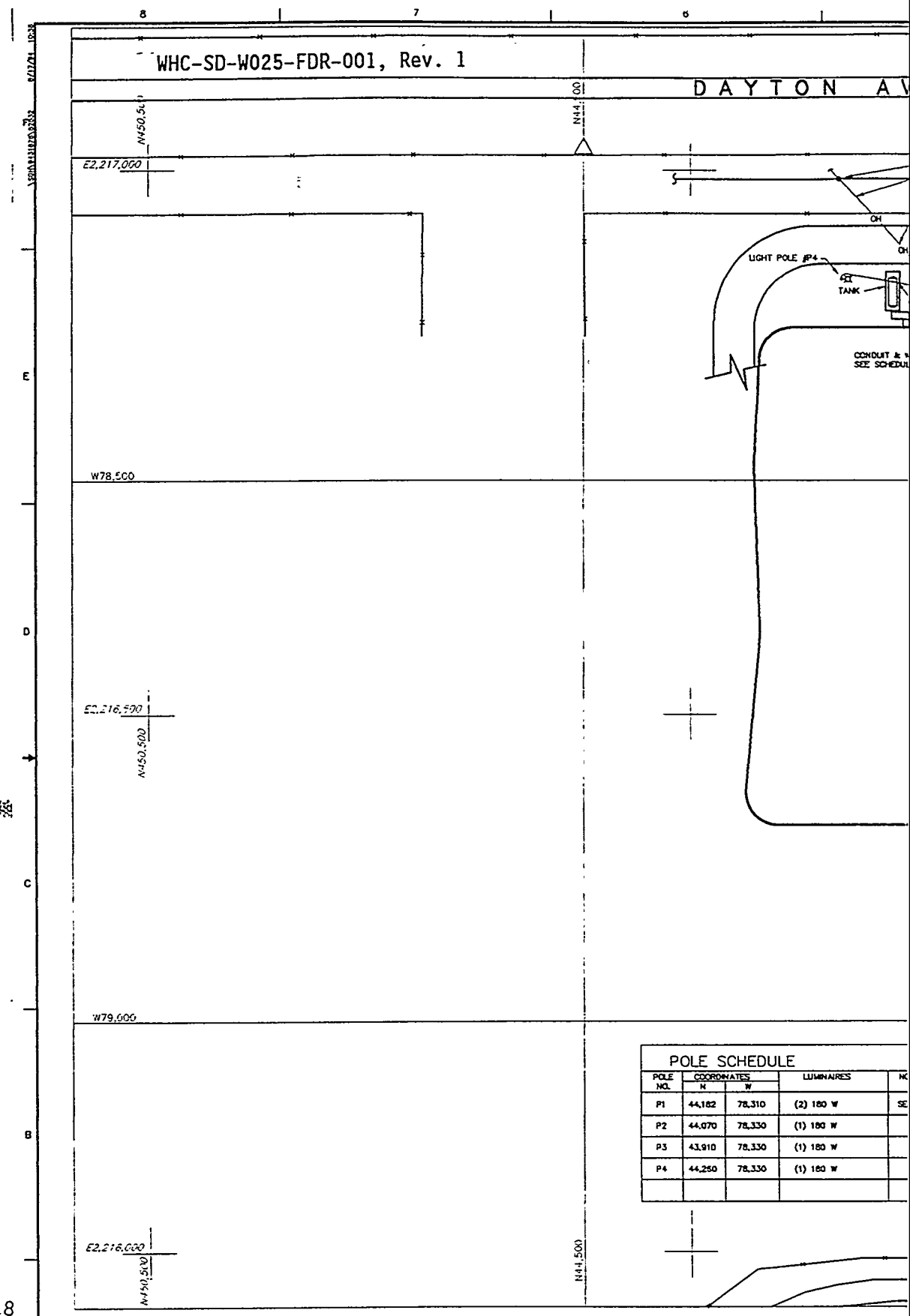
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

ELECTRICAL – TANK & CONTROL BLDG.
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | BLDG. NO. 7100 | DRWG. NO. H-2-131587 | REV NO 3
SCALE AS SHOWN | JOB NO [illegible] | SHEET 2 OF 4

WIRING TRACEABILITY LIST

TITLE	NUMBER	TITLE
	REFERENCES	
	NEXT ASSY USED ON	

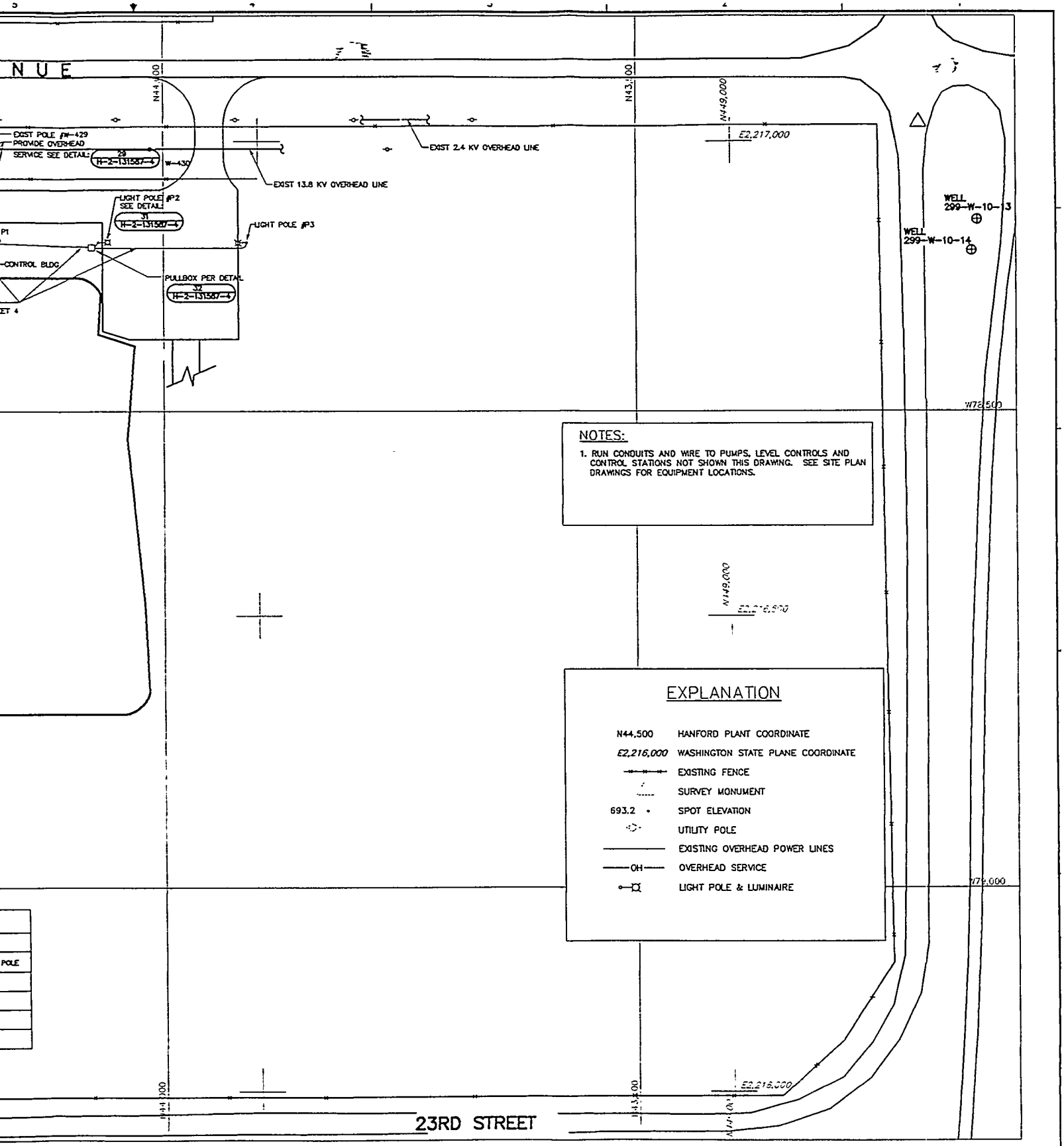


POLE SCHEDULE

POLE NO.	COORDINATES N	W	LUMINAIRES	NO
P1	44,182	78,310	(2) 180 W	SE
P2	44,070	78,330	(1) 180 W	
P3	43,910	78,330	(1) 180 W	
P4	44,250	78,330	(1) 180 W	

N

0 50 100
FEET



NOTES:

1. RUN CONDUITS AND WIRE TO PUMPS, LEVEL CONTROLS AND CONTROL STATIONS NOT SHOWN THIS DRAWING. SEE SITE PLAN DRAWINGS FOR EQUIPMENT LOCATIONS.

EXPLANATION

N44,500	HANFORD PLANT COORDINATE
E2,216,000	WASHINGTON STATE PLANE COORDINATE
	EXISTING FENCE
	SURVEY MONUMENT
693.2	SPOT ELEVATION
	UTILITY POLE
	EXISTING OVERHEAD POWER LINES
—OH—	OVERHEAD SERVICE
	LIGHT POLE & LUMINAIRE

Sparling & Associates, Inc.
1920 Eastlake Avenue East
Seattle, Washington 98102

NO.	DESCRIPTION	BY	DATE	APPROVALS
3	AS-BUILT PER ECN [illegible]	SGH	F. SHURI 10/6/94	
2	ECN [illegible] TITLE BLOCK CORRECTIONS	FSS	FSS, GM, MJG 10-30-9[illegible]	
1	ECN [illegible] GENERAL REVISIONS	EGE		
	REVISE LIGHT POLE LOCATIONS	KGH	F. SHURI 5/1/92	

REVISIONS

CAD FILE B131587C CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED R. SPARLING 04/02/90
DRAWN J. LOSHECARD 04/02/90
CHECKED R. SPARLING 04/02/90
REVIEWED F. SHURI 04/02/90
QA M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION BY FOR DATE

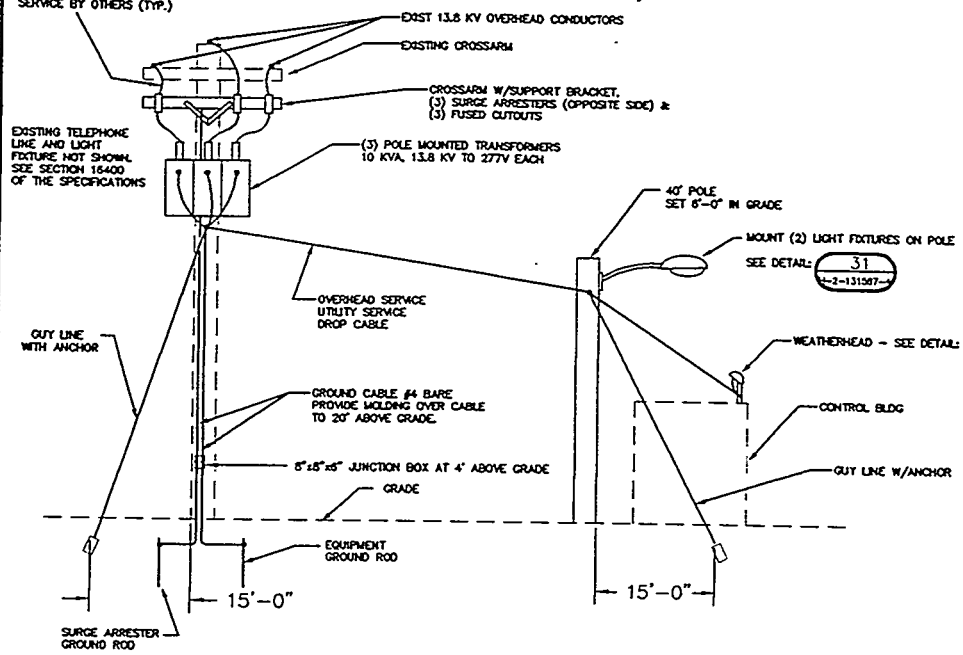
WHQC LEVEL EDT 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

ELECTRICAL SITE PLAN
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SCALE F | BLDG. NO. 299-W-10 | ACCT. NO. 7401 | DWG. NO. H-2-131587 | REV. NO. 3
SCALE AS SHOWN | JOB NO. 893-1186.030 | SHEET 3 OF 4

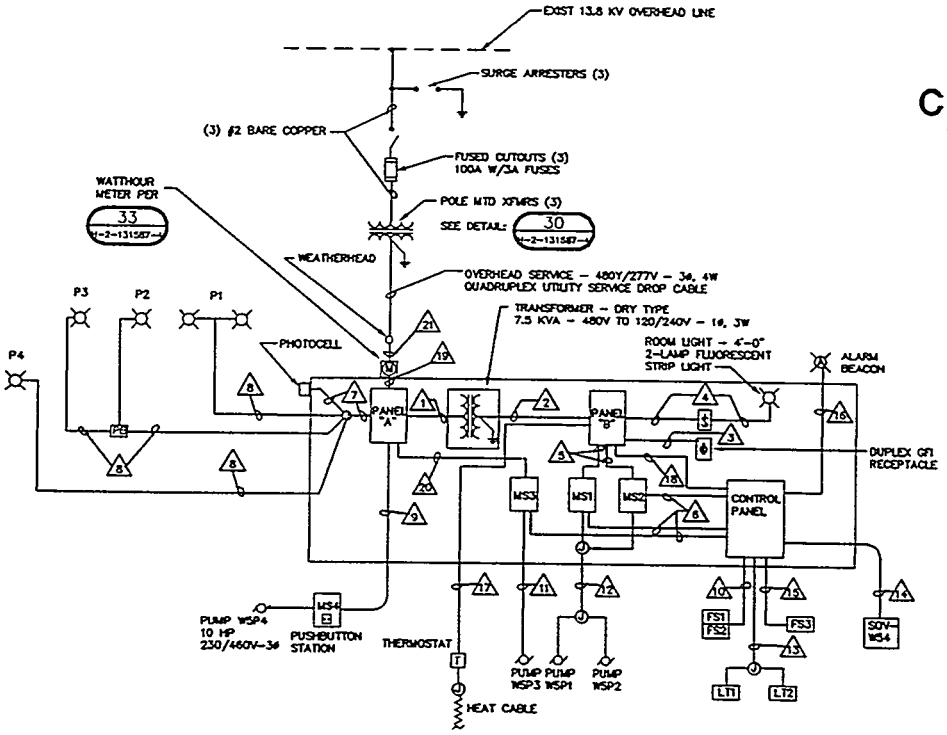
TITLE	NUMBER	TITLE
	REFERENCES	
DRAWING TRACEABILITY LIST	NOT USED ON	



POWER SERVICE ELEVATION DETAIL 29

N.T.S.

NOTE: 6'-0" MIN BETWEEN GROUND RODS. TOP OF GROUND RODS 12" MIN BELOW GRADE.



ELEMENTARY DIAGRAM

N.T.S.

Sparling & Associates, Inc.
1920 Eastlake Avenue East
Seattle, Washington 98102

PANEL SCHEDULE

LOCATION: CONTROL BLDG
SERVING:

480Y/277 VOLTS, 3 PHASE, 4 WIRE
100 AMP MAIN BREAKER

KVA	AMP			AMP	KVA	CIRCUIT DESCRIPTION
11.2	30	1	2	20	7.5	TRANSFORMER
		3	4	2		
	3	5	6	20	1.2	SITE LIGHTS
3.3	15	7	8	20		SPARE
		9	10			SPACE
	3	11	12			
		13	14			
		15	16			
		17	18			
		19	20			
		21	22			
		23	24			

NOTES:

DEMAND LOAD: ____ KVA ____ AMPS

PANEL SCHEDULE

PANEL: B
LOCATION: CONTROL BLDG
SERVING:

SURFACE ■ FLUSH □
A.I.C. 10K

120/240 VOLTS, 1 PHASE, 3 WIRE
30 AMP MAIN BREAKER

CIRCUIT DESCRIPTION	KVA	AMP			AMP	KVA	CIRCUIT DESCRIPTION
PUMP - 1/2 HP	1.1	15	1	2	20	.3	CONTROLS
WSP1		2	3	4	20	2	RECEPTACLE
PUMP - 1/2 HP	1.1	15	5	6	20	.1	ROOM LIGHT
WSP2		2	7	8	40	3.5	HEAT TRACE CABLE
SPARE		15	9	10	20		SPARE
		2	11	12	20		
SPACE			13	14	20		
			15	16			SPACE
			17	18			

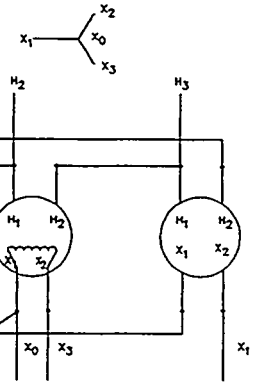
CONNECTED LOAD: 6.3 KVA 26 AMPS

NOTES:

DEMAND LOAD: ____ KVA ____ AMPS

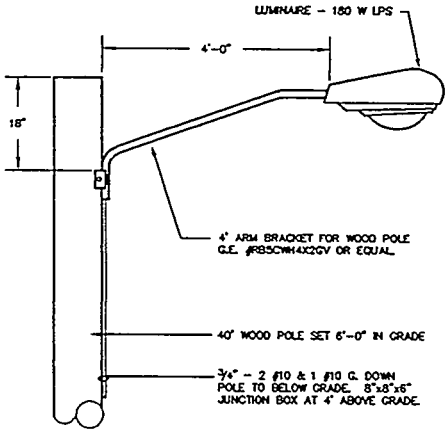
MATERIAL LIST

#	DESCRIPTION	REFERENCE
1	WATTHOUR DEMAND METER, SOCKET TYPE, 7 TERMINAL 240 VOLT TYPE FOR USE ON 480Y/277V. CLASS 200 FOR MAX 200 AMP SERVICE COMPLETE WITH M-100 DEMAND REGISTER AND PULSE INITIATOR OUTPUT BOARD. METER TO BE CALIBRATED BY WHC PRIOR TO INSTALLATION.	GE NO. VM4-65-S
2	METER SOCKET AND ENCLOSURE, 7 TERMINAL WITH MANUAL OR AUTOMATIC CIRCUIT CLOSING DEVICES, 600 VOLT RATED NEMA TYPE 3R CONSTRUCTION, FOR 200 AMP SERVICE.	CIRCLE AW NO. U-287
3	BUSHING, INSULATED GROUNDING TYPE.	OZ/GEDNEY CO. TYPE BLG
4	GROUNDING LUG (BOLT TO ENCLOSURE).	CIRCLE AW 4-25145



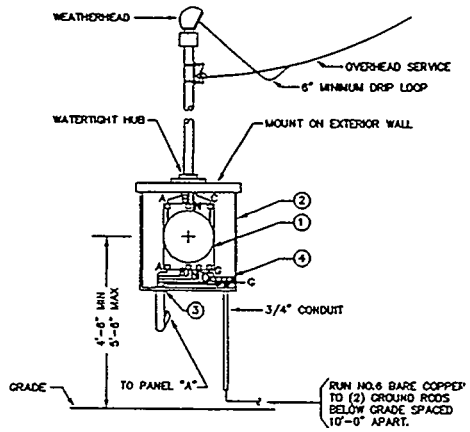
SFORMER IS DETAIL 30 (H-2-131587-4)

N.T.S.



LIGHT FIXTURE DETAIL 31 (H-2-131587-3)

N.T.S.

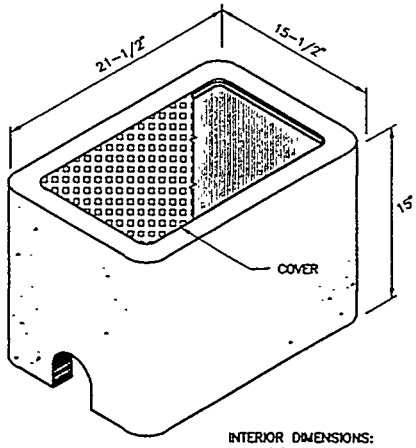


METER & GROUNDING ELEVATION 33 (H-2-131587-4)

N.T.S.

CONDUIT AND WIRE SCHEDULE

CONDUIT SIZE	WIRE	CIRCUIT NO.	DESCRIPTION	NOTES	CONDUIT NO.
3/4"	2#12 & 1#12 G.	A-2	TRANSFORMER		CW45819
3/4"	3#10 & 1#12 G.	-	PANEL 'B'		CW45811
3/4"	2#12 & 1#12 G.	B-4	RECEPTACLE		CW45812
3/4"	2#12 & 1#12 G.	B-6	LIGHT, SWITCH		CW45813
3/4"	3#12 & 1#12 G.	B-1 (5)	TO MS1 (MS2)	TO EACH	CW45814 (15)
3/4"	4 #14	-	TO CONTROL PANEL FROM MS1 (MS2,MS3)	TO EACH	CW45817 (18,19)
3/4"	2#12 & 1#12 G.	A-6	TO PHOTOCELL J-BOX	3 #12 TO PHOTOCELL	CW45820
3/4"	2#10 & 1#10 G.	A-6	TO OUTDOOR LIGHTS	BELOW GRADE	CW45821
3/4"	3#12 & 1#12 G.	A-1	TO MS4	BELOW GRADE	CW45807
3/4"	4#14 & 1#14 G.	-	TO FS1 & FS2	BELOW GRADE	CW45801
3/4"	3#12 & 1#12 G.	-	TO PUMP WSP3	BELOW GRADE	CW45802
3/4"	6#12 & 1#12 G.	-	TO PUMPS WSP1 & WSP2	BELOW GRADE	CW45803
3/4"	AS REQUIRED	-	TO LEVEL SENSORS	BELOW GRADE	CW45804
3/4"	2#12 & 1#12 G.	-	TO SOLENOID VALVE MS4	BELOW GRADE	CW45805
3/4"	2#12 & 1#12 G.	-	TO FS3	BELOW GRADE	CW45806
3/4"	2#12 & 1#12 G.	-	TO ALARM BEACON		CW45809
3/4"	2#8 & 1#10 G.	B-8	TO HEAT CABLE	BELOW GRADE	CW45808
3/4"	2#12 & 1#12 G.	B-2	TO CONTROL PANEL		CW45822
1 1/2"	4#2 & 1#6 G.	-	SERVICE		CW45823
3/4"	3#12 & 1#12 G.	A-7	TO MS3		CW45816
1 1/2"	4#2	-	SERVICE		CW45824



INTERIOR DIMENSIONS:
18-3/4"L. X 12-3/4"W.

PULLBOX DETAIL 32 (H-2-131587-3)

N.T.S.

NOTE: SET WITH COVER FLUSH WITH GRADE.

W-025 LOBE 1 SUBMITTAL/CVI #22585 CROSS INDEX

SUB#	CVI SUP#	SPEC REF.	DESCRIPTION
72		16010, 1.7	Electrical System Certification (As-Built Drawings for Underground Conduits)
		16100	Electrical and Instrumentation Operation and Maintenance
73		16100, 2.4.A	Raceways
73A		16100, 2.4.A	Coating and Conduits
74		16100, 2.4.A	Wire and Cables
75		16100, 2.4.A	Connectors and Terminations
76		16100, 2.4.A	Motor Controllers
76A		16100, 2.4.A	Motor Controllers
77		16100, 2.4.A	Power Service Equipment
78		16100, 2.4.A	Transformers
78.1		16100, 2.4.A	Transformers
80		16100, 2.4.A	Lighting Fixtures and Ballasts - Manuf. Info.
81		16100, 2.4.A	Wood Poles
82		16100, 2.4.A	Open Fuse Cutouts
83		16100, 2.4.A	Surge Arrestors - Manuf. Information
84		16160, 1.2	Push Button Stations
85		16100, 2.4.A	Pullbox
86		16100, 2.4.A	Heat Trace
86A		16100, 2.4.A	Heat Trace
88		16160, 1.2	Motor Controllers
88A		16160, 1.2	Motor Controllers
89		16460, 1.2	Transformers - Shop Drawings
89.1		16460, 1.2	Dry Type Transformers - Shop Drawings
91		16500, 1.4.A	Lighting - Certified Lab. Test Report
91A		16500, 1.4.A	Lighting - Certified Lab. Test Report
92		16500, 1.4.B	Lighting
93		16859, 1.3.A	Heat Trace - Manuf. Information
93A		16859, 1.3.A	Heat Trace
94		16859, 1.3.B	Heat Trace - QA/QC Documentation
94A		16859, 1.3.B	Heat Trace - QA/QC Documentation

FRANK S. SHURI
REGISTERED ENGINEER

NO.	DESCRIPTION	BY	DATE
3	AS-BUILT PER ECN: ADD CH CIRCUIT NOTE PER ECN W-025-028; REMOVE PHOTOCELL PER ECN W-025-033; OPTIONAL PER ECN W-025-035; FOR W-025-038	SCH	F. SHURI 8/26/94
2	REVISE PUMPS AND WIRING PER ECN W-025-031.	JPP	F. SHURI 5/21/93
1	RELOCATE MS4 PUSH BUTTON STATE, ADD JUNCTION BOX.	JPP	M. BROWN 5/1/92

REVISIONS

DESIGNED	R. SPARLING 04/02/90
DRAWN	J. LOSHEGARD 04/02/90
CHECKED	R. SPARLING 04/02/90
REVIEWED	F. SHURI 04/02/90
QA	M. J. OTTERSON 04/02/90

APPROVED FOR FABRICATION
BY
FOR DATE

IMPACT LEVEL: ____ EDT 151914

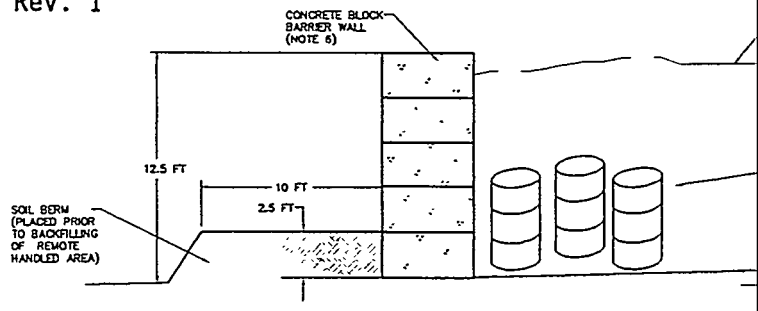
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.
ELECTRICAL DETAILS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
DRAWING NO. H-2-131587
SHEET NO. 4 OF 4
REV. NO. 3
SCALE AS SHOWN
JOB NO. 863-1166.038

DRAWING TRACEABILITY LIST

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	REFERENCES	
	NOT USED ON	

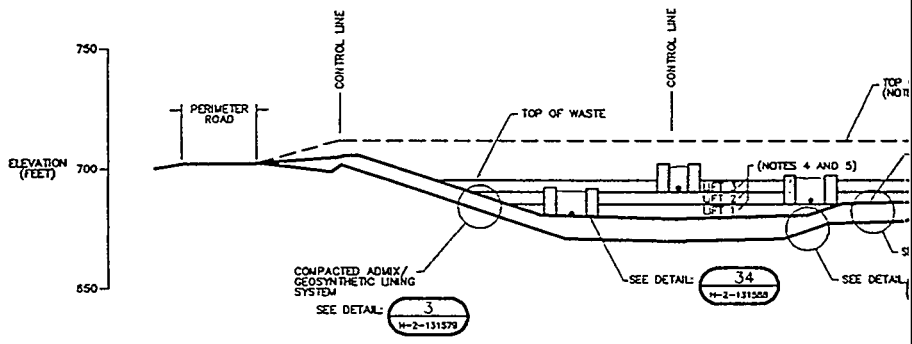
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DGN CODE 2W:IBM:ACD2:10:SS



DETAIL
BARRIER BLOCK WALL

34 / H-2-131588

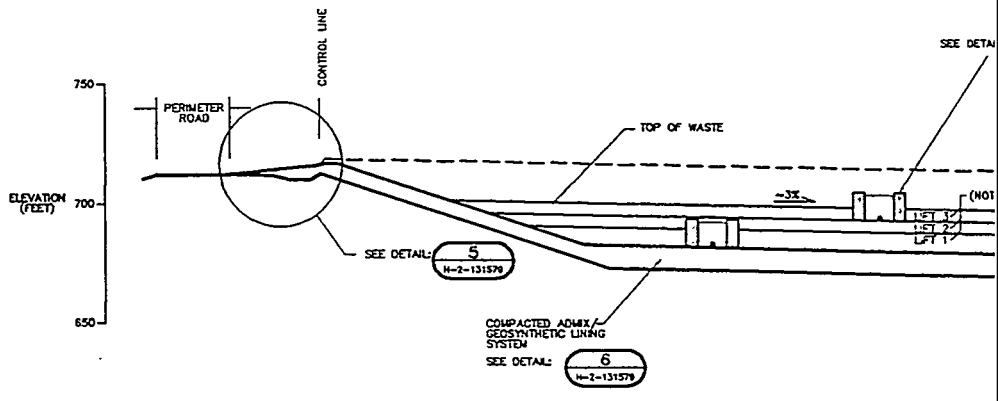
0 4 8
SCALE IN FEET



SECTION

A / H-2-131588

0 30 60
SCALE IN FEET



NOTES:

1. See Sheet H-2-131585 for tank area details.
2. Control point locations shown on Drawing H-2-131575.
3. Final cover design not included in this contract and will be performed at a later date.
4. Each waste lift shall be placed across entire floor of facility prior to start of subsequent lift.
5. Filling Plan shall be prepared by operating contractor and submitted to the engineer for review prior to waste placement.
6. Concrete block barrier wall shall consist of precast blocks, 2.5-ft x 2.5-ft x 5-ft.

SECTION

B / H-2-131588

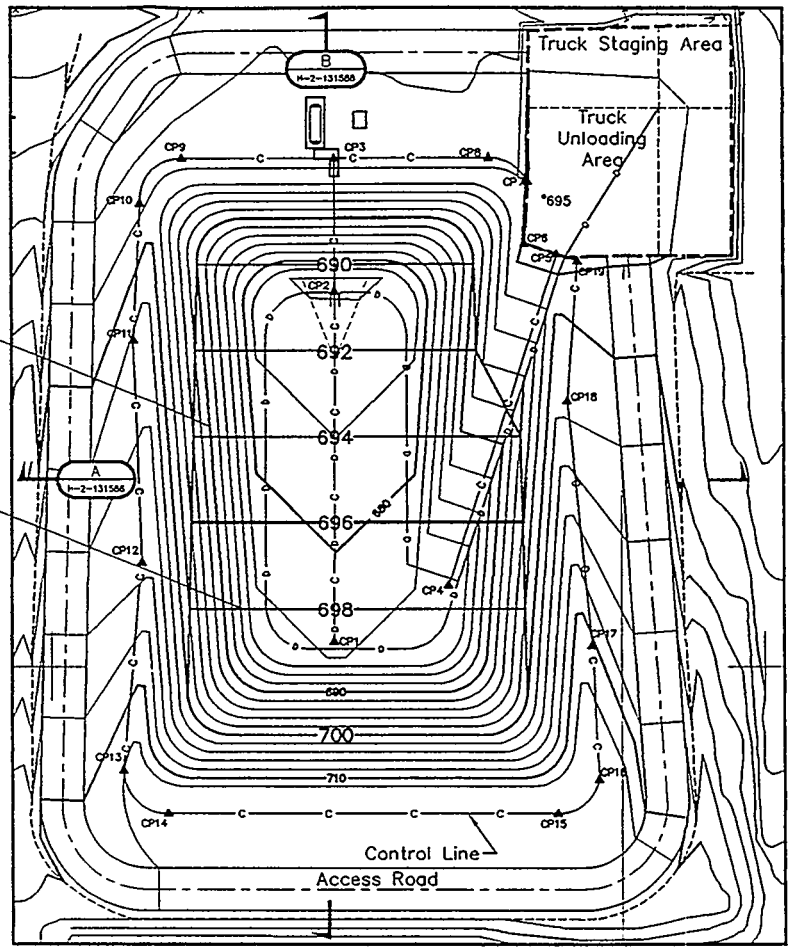
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SCALE IN FEET

5 4 3 2 1

BACKFILL COVER

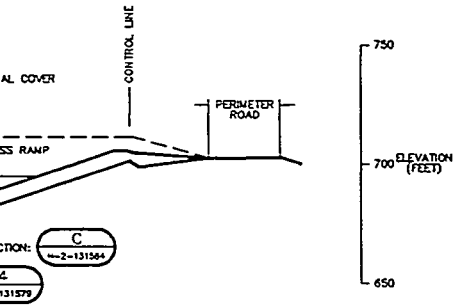
REMOTE HANDLED WASTE

OPERATIONS LAYER OR INTERMEDIATE COVER



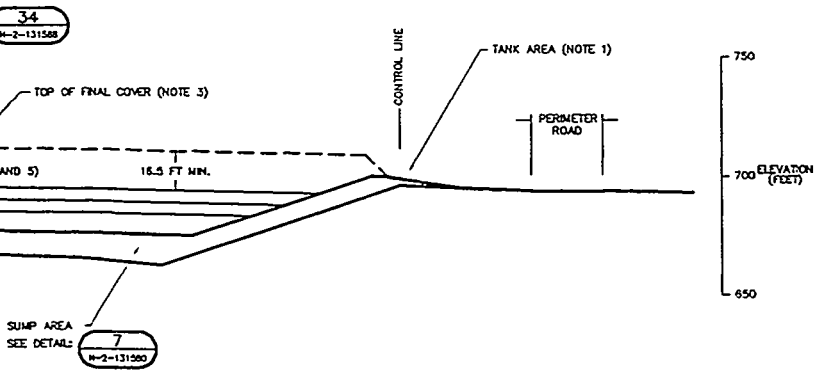
TOP OF OPERATIONS LAYER CONTOURS

TOP OF WASTE CONTOURS



CONCEPTUAL FILL PLAN
TOP OF WASTE CONTOURS

0 50 100
SCALE IN FEET



KEY:

C H-2-131577 — DESIGNATION / SHEET NUMBER

LEGEND

- REMOTE HANDLED WASTE PACKAGE
- CONCRETE BLOCK RADIATION BARRIER WALL

FRANK S. SHURI PROFESSIONAL ENGINEER

			3	SCH F. SHURI 8/28/94	
			2	JPP F. SHURI 5/21/93	
			1	JPP F. SHURI 5/1/92	
			MF	REV NO	DESCRIPTION / APPROVALS
NUMBER	TITLE				REVISIONS
TITLE	REFERENCES				
AWING TRACEABILITY LIST	NEXT USED ON				

CAD FILE B131588A CAD CODE 2W;IBM;ACD2;10;SS

IMPACT LEVEL | CAT 151914

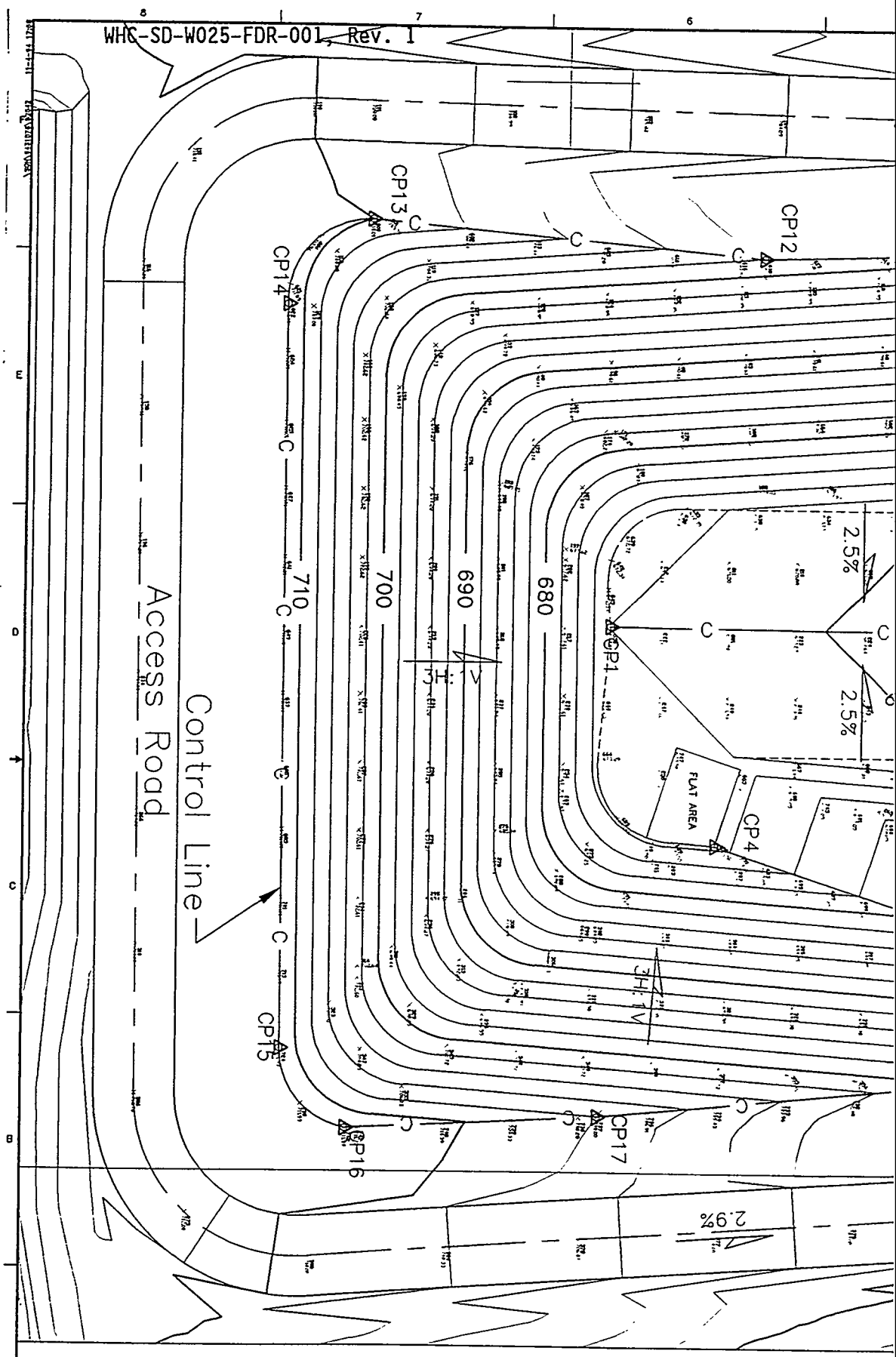
DESIGNED T. MARTIN 04/02/90
DRAWN J. SNOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
QA M. J. OTTERSON 04/02/90

APPROVED FOR IMPLEMENTATION BY ____ FOR ____ DATE

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

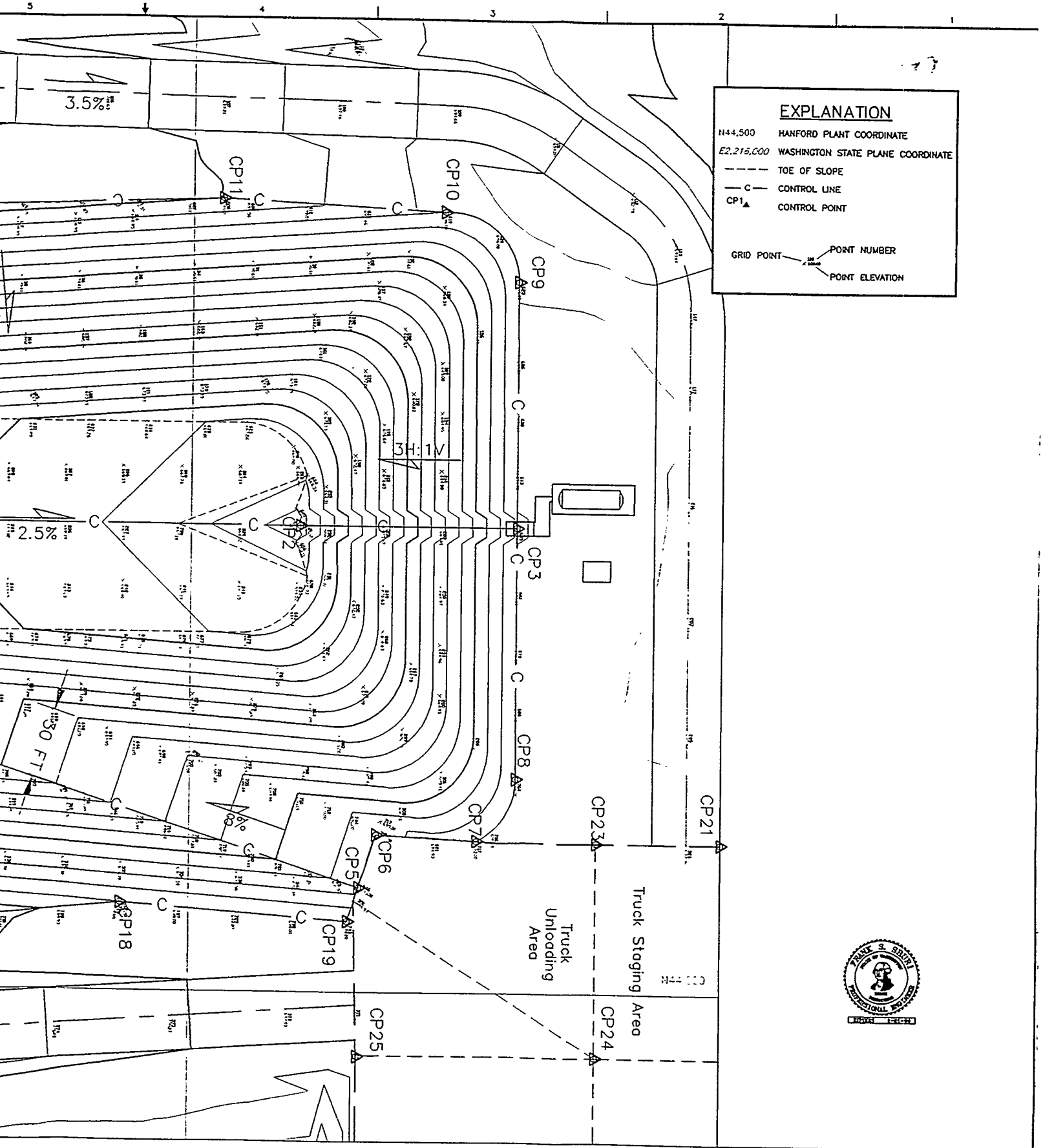
CONCEPTUAL FILL PLAN
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | BLDG. NO. 219-W-6 | DRWG NO. 0401 | DWG NO. H-2-131588 | REV NO 3
SCALE AS SHOWN | JOB NO. 893-1186.038 | SHEET 1 OF 1



NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degross Aerial Mapping based on July 18, 1988, aerial photography.



3	AS-BUILT PER ECN [illegible]	SGH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	MODIFY LANDFILL GEOMETRY AND CONTROL POINTS	SGH	F. SHURI	5/5/92
REV NO	DESCRIPTION	BY	DATE	APPROVALS

REVISIONS

CAD FILE B131589A CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED T. MARTIN 04/02/90
DRAWN J. SNOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
BY W. J. OTTERSON 04/02/90

APPROVED FOR IMPLEMENTATION
BY
FOR DATE

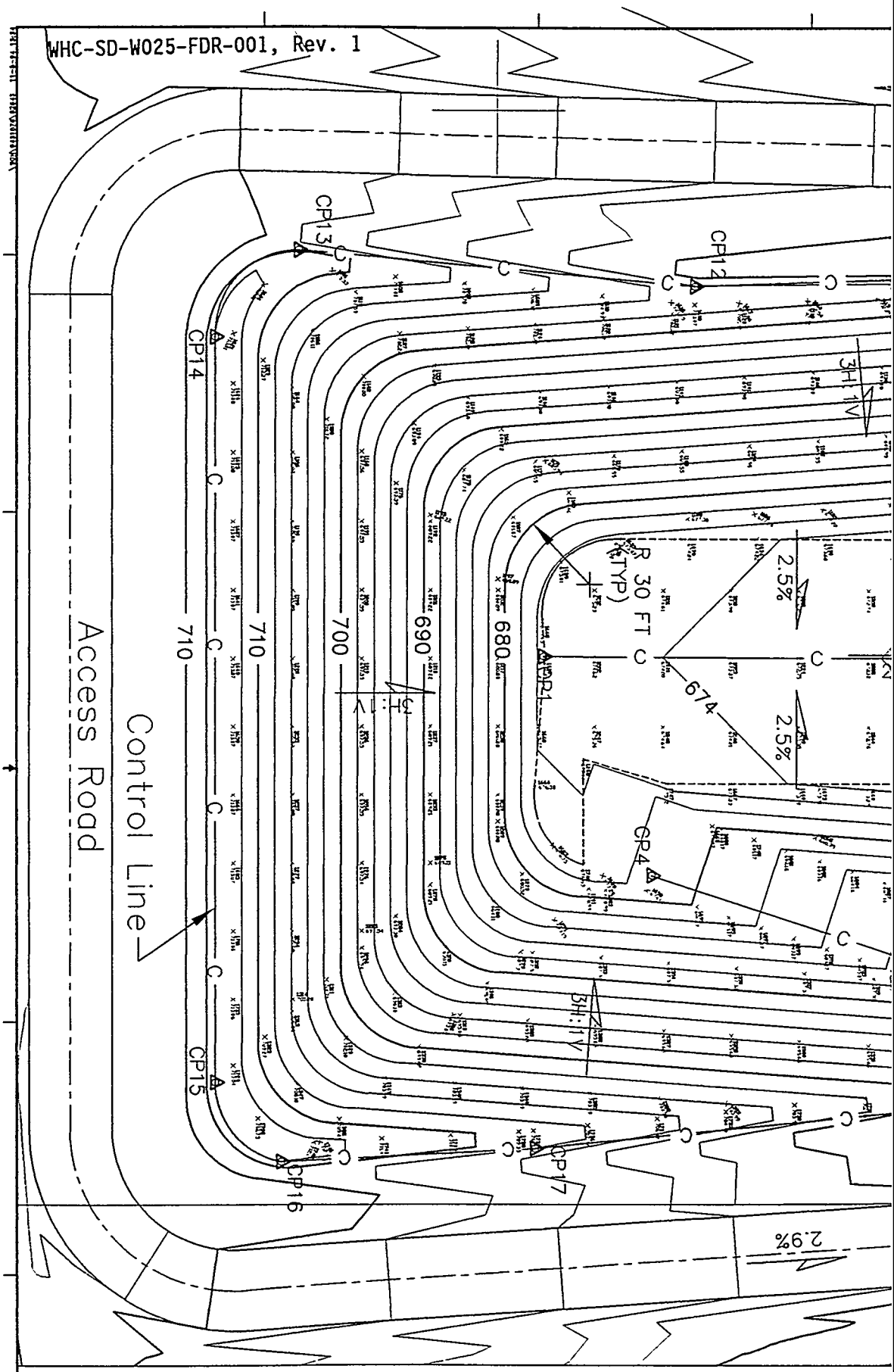
IMPACT LEVEL CI# 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SUBGRADE GRID POINTS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | BLDG. NO. [illegible] | MDL NO. 0401 | DWG. NO. H-2-131589 | REV. NO. 3
SCALE AS SHOWN JOB NO. [illegible] SHEET 1 OF 4

NUMBER TITLE
REFERENCES
WING TRACEABILITY LIST NEXT USED ON

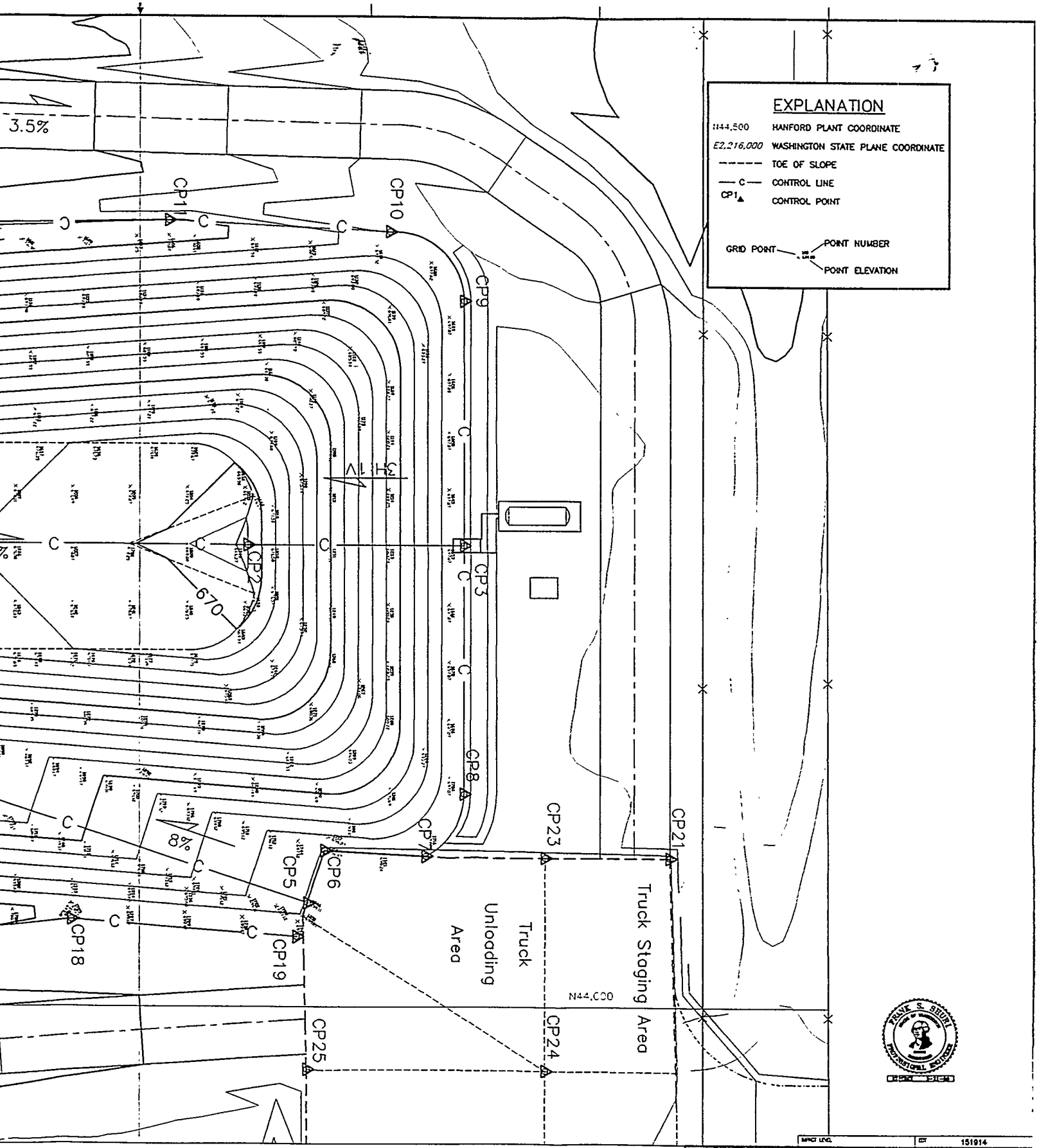


NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degross Aerial Mapping based on July 18, 1988, aerial photography.
3. Indicated elevations do not include geosynthetic thickness and grading tolerances. Maintain required layer thickness at all locations.

0 20 40
FEET

N



DESIGNED	T. MARTIN	04/02/90	
DRAWN	J. SNOW	04/02/90	
CHECKED	F. SHURB	04/02/90	
RECOMMD	P. CORSER	04/02/90	
QA	M. J. OTTERSON	04/02/90	

IMPACT LEVEL — EDT 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

SECONDARY ADMIX GRID POINTS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025

SIZE	BLDG. NO.	MICR. NO.	DWG. NO.	REV NO.
F	236-R-4	0401	H-2-131590	3

SCALE AS SHOWN — 7/1 VOL 893-1168.0351 — SHEET 1 OF 1

REV NO	DESCRIPTION	BY	DATE	APPROVALS
3	CO-LOCATE PGB ECRH HOME ACCESS ROAD 76 FT NORTH PER ECN W-025-9412 EDITORIAL PER ECN W-025-008, ECN W-025-008	SGH	F. SHURB	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURB	5/21/93
1	MODIFY LANDFILL GEOMETRY AND CONTROL POINTS	JPP	F. SHURB	5/5/92

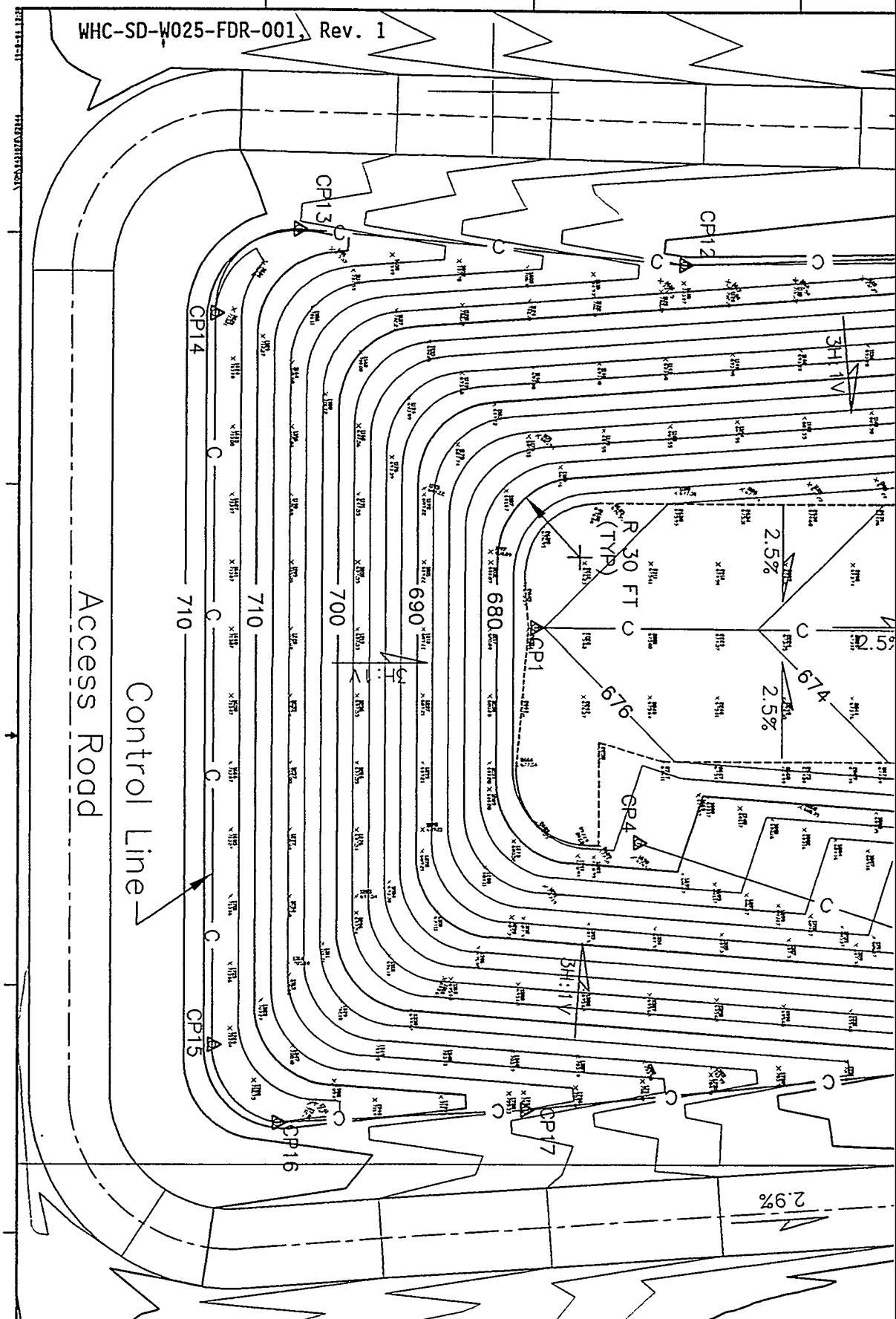
REVISIONS

APPROVED FOR RELEASE.COM — BY — FOR — DATE

NUMBER	TITLE
REFERENCES	
NEXT USED ON	

DRAWING TRACEABILITY LIST

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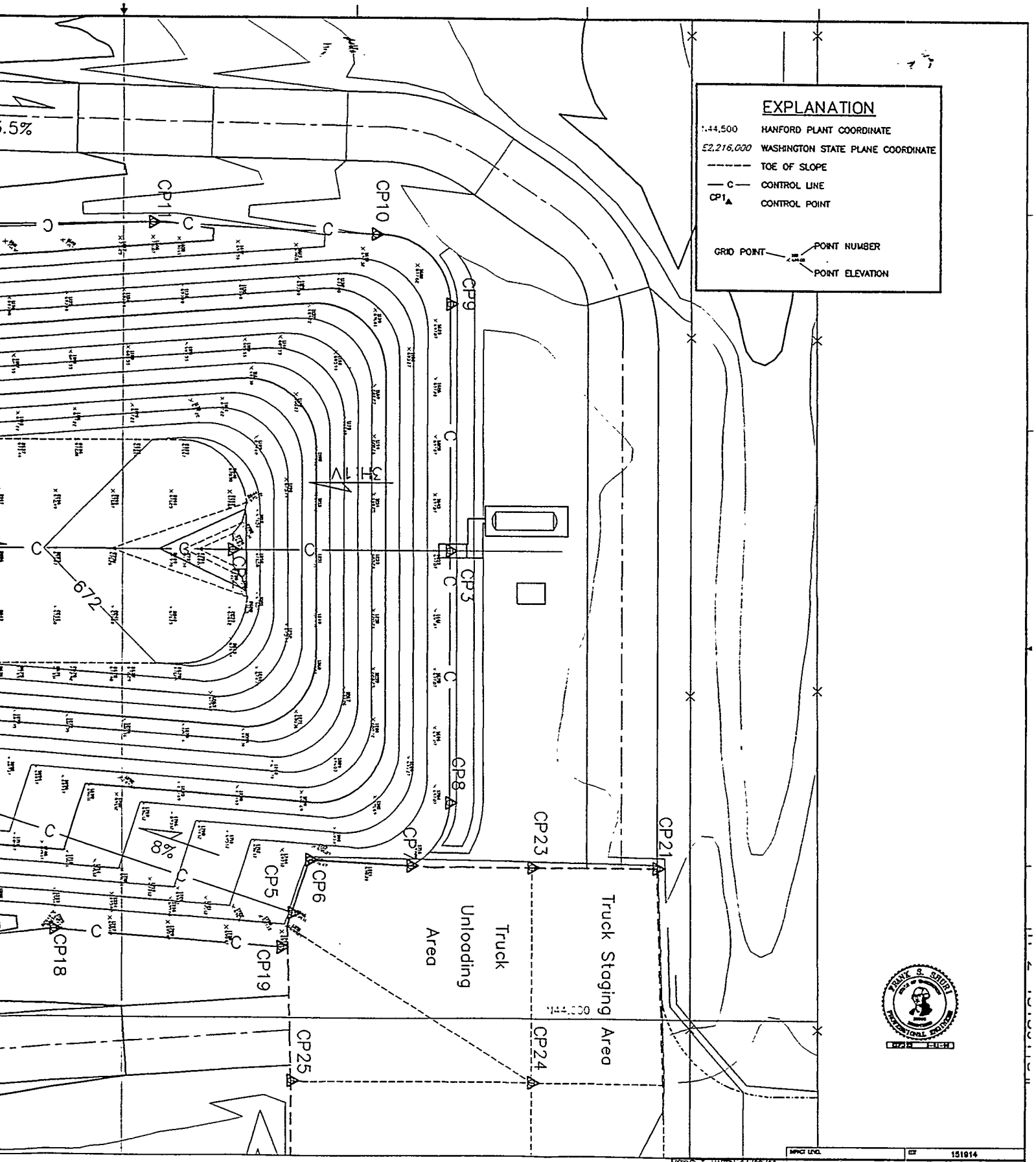


NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degross Aerial Mapping based on July 18, 1988, aerial photography.
3. Indicated elevations do not include geosynthetic thickness and grading tolerances. Maintain required layer thickness at all locations.

0 20 40
FEET

N



3	AS-BUILT PER ECN	SSH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	MODIFY LANDFILL GEOMETRY AND CONTROL POINTS	JPP	F. SHURI	5/5/92

DESCRIPTION | BY | DATE | APPROVALS

REVISIONS

OD FILE B131591A | OD CODE 2W;IBM;ACD2;10;SS

DESIGNED T. MARTIN 04/02/90
DRAWN J. SNOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
QA M. J. OTTERSON 04/02/90

APPROVED FOR SPECIFICATION
BY
FOR DATE

IMPACT LEVEL | ECR 151914

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES, INC.

SECONDARY GRAVEL GRID POINTS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

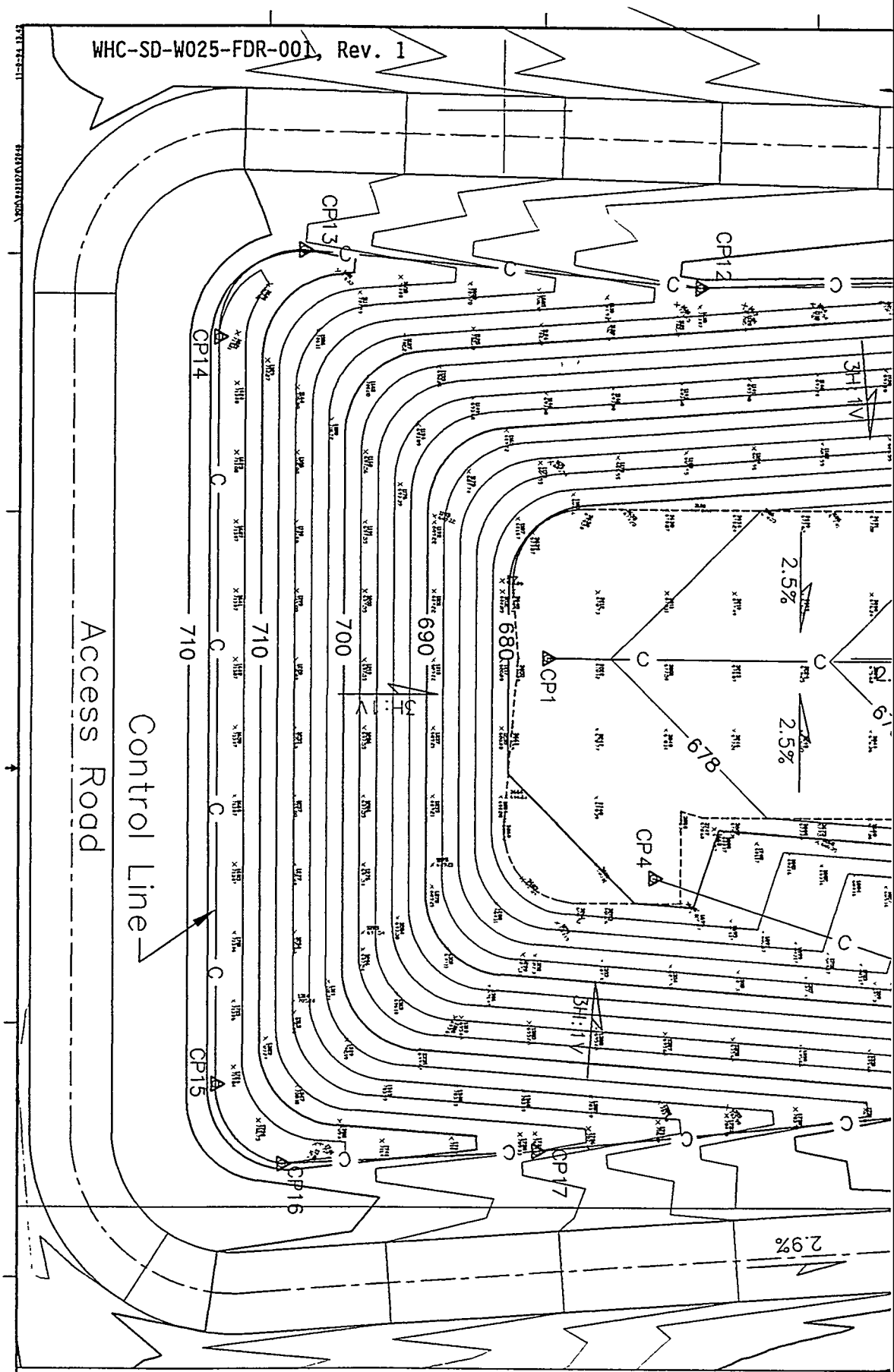
PROJECT W-025

DATE	BLDG NO.	BOOK NO.	DWG NO.	REV NO.
F 09-94		0401	H-2-131591	3

SCALE AS SHOWN | JOB NO. 8903-1166.035 | SHEET 1 OF 1

NUMBER | TITLE
REFERENCES
NOT USED ON

TITLE
NG TRACEABILITY LIST

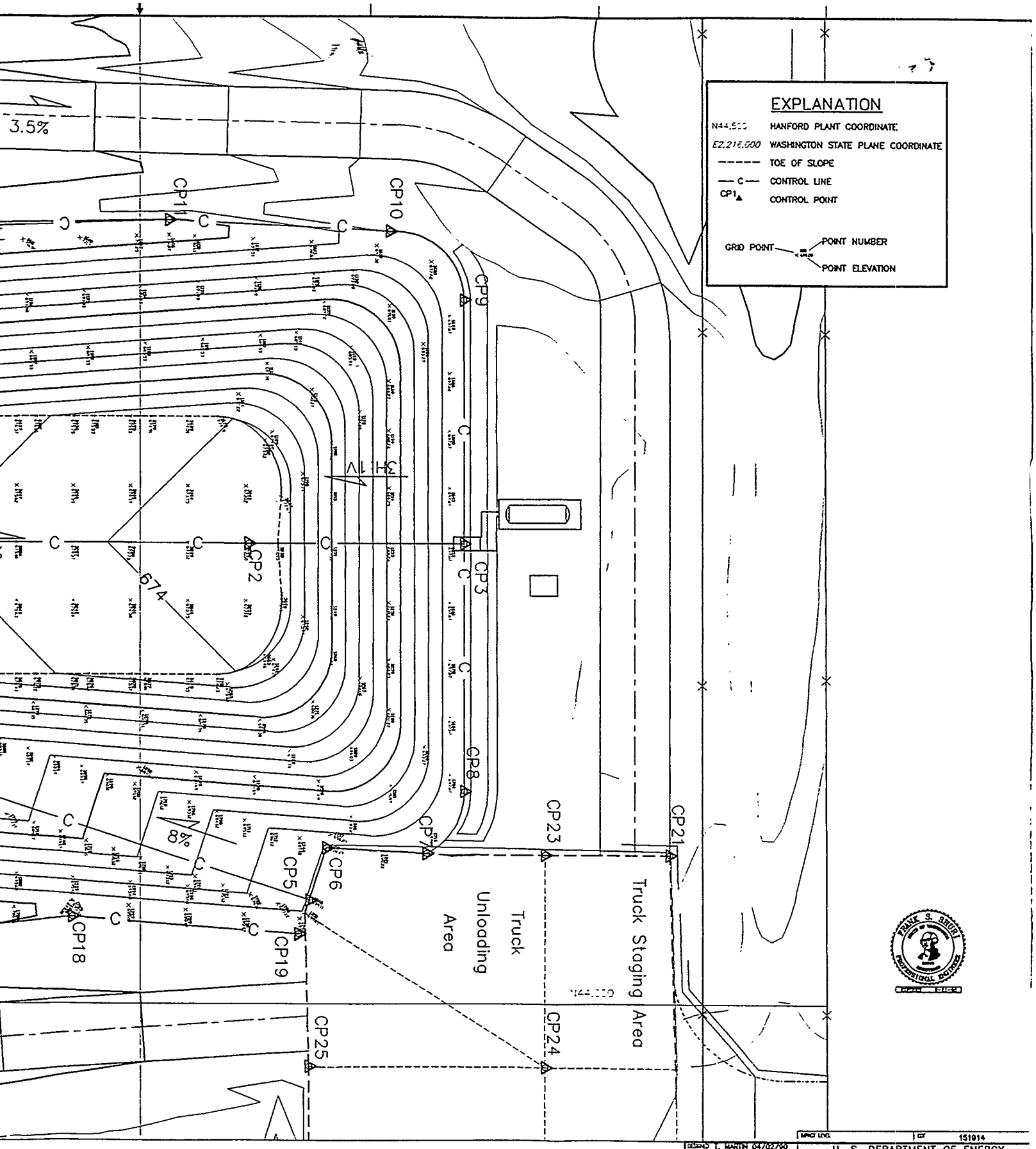


NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degross Aerial Mapping based on July 18, 1988, aerial photography.
3. Indicated elevations do not include geosynthetics thickness and grading tolerances. Maintain required layer thickness at all locations.

0 20 40
FEET

N



DESIGNED	T. MARTIN	04/02/90	
DRAWN	J. SNOW	04/02/90	
CHECKED	F. SHURI	04/02/90	
REVIEWED	P. CORSER	04/02/90	
QA	M. J. OTTERSON	04/02/90	
APPROVED FOR IMPLEMENTATION			

NO.	DESCRIPTION	APPROVALS
3	ADD ACCESS ROAD 16 FT NORTH FOR ECR W-025-111 OPTIONAL FOR ECR W-025-104, ECR W-025-108	SCH F. SHURI 8/25/94
2	MINOR EDITORIAL	JPP F. SHURI 5/21/93
1	MODIFY LANDFILL GEOMETRY AND CONTROL POINTS	JPP F. SHURI 5/5/92

REVISIONS

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC

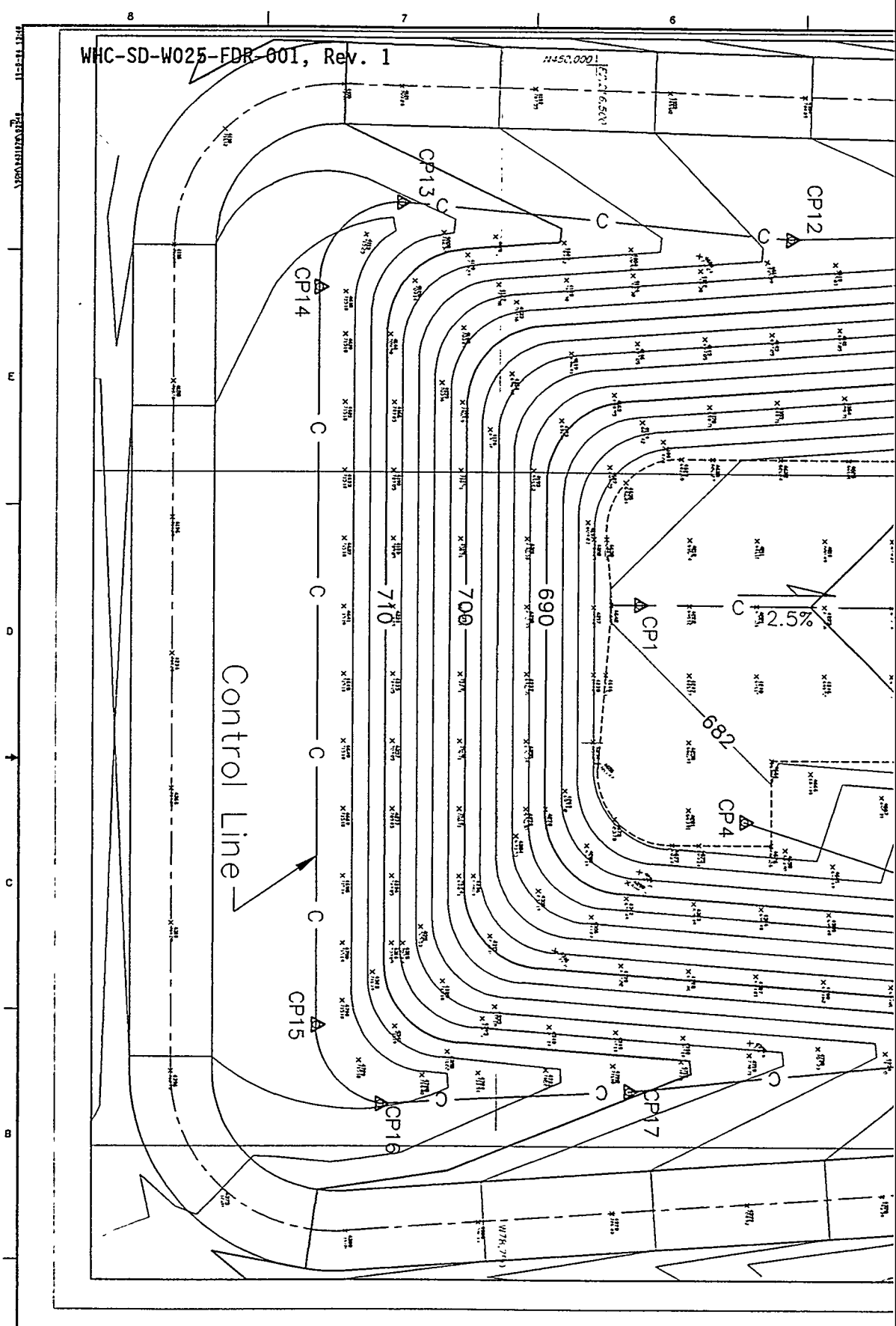
PRIMARY GRAVEL GRID POINTS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | BLDG. NO. 0401 | DRG. NO. H-2-131592 | REV. NO. 3
SCALE AS SHOWN | SHEET 1 OF 1

DWG FILE B131592A | CAD SYSTEM 2W:IBM:ACD2:10:SS

REFERENCES

DRAWING TRACEABILITY LIST

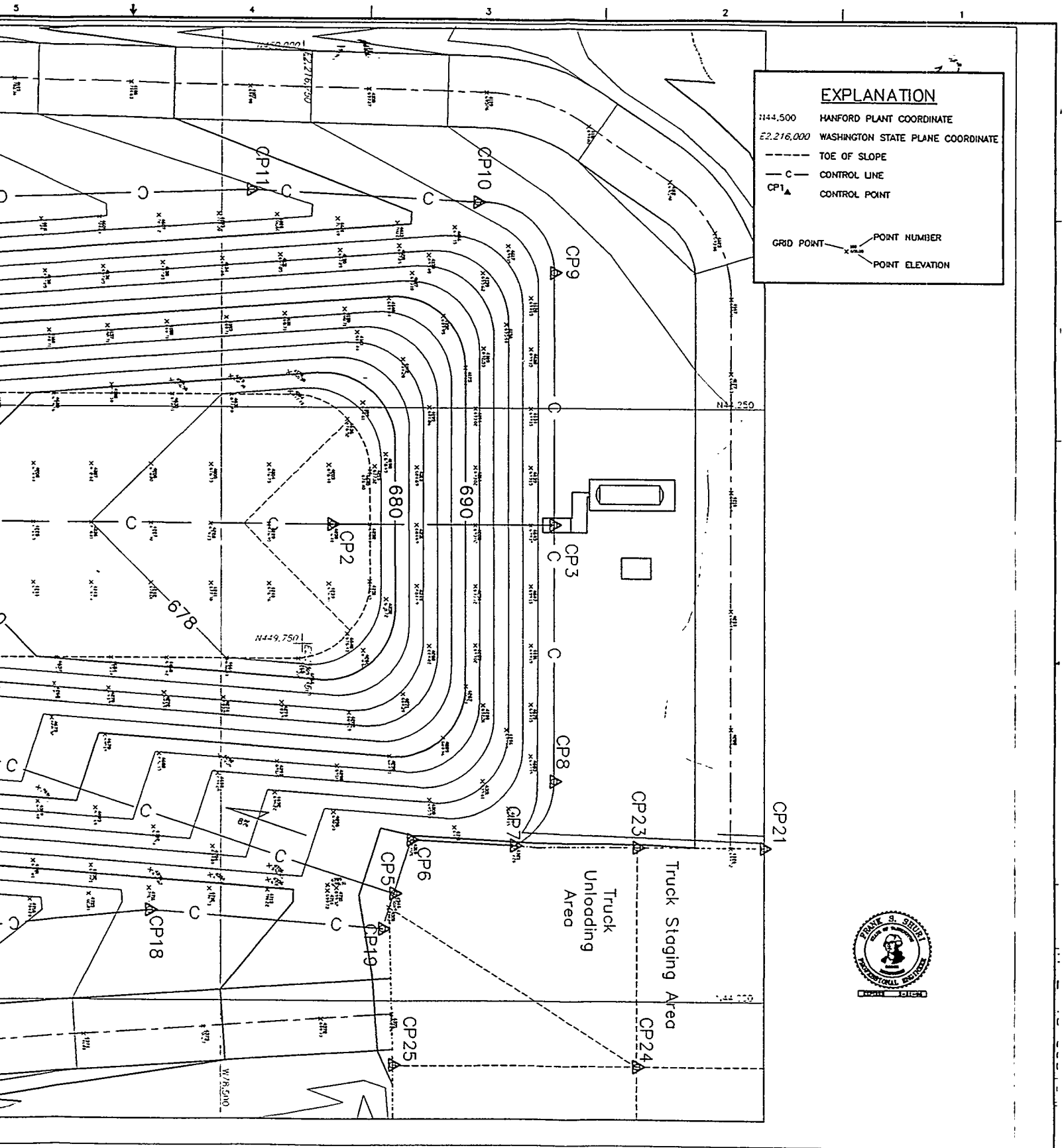


NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degross Aerial Mapping based on July 18, 1988, aerial photography.
3. Indicated elevations do not include geosynthetic thickness and grading tolerances. Maintain required layer thickness at all locations.

0 20 40
FEET

N



3	ADDED POINTS CP24, CP25, REVISED POINTS CP1, CP2, CP3, CP4, CP5, CP6, CP7, CP8, CP9, CP10, CP11, CP18, CP19, CP21, CP23	SGH	F. SHURI	8/26/94
2	MINOR EDITORIAL	JPP	F. SHURI	5/21/93
1	MODIFY LANDFILL GEOMETRY AND CONTROL POINTS	KGH	F. SHURI	5/5/92
NO.	DESCRIPTION	BY	APPROVALS	DATE

REVISIONS

CAD FILE B131593A CAD CODE 2W:IBM:ACD2:10:SS

DESIGNED T. MARTIN 04/02/90
DRAWN J. SNOW 04/02/90
CHECKED F. SHURI 04/02/90
REVIEWED P. CORSER 04/02/90
QA M. J. OTTERSON 04/02/90
APPROVED FOR IMPLEMENTATION
BY
FOR DATE

IMAGE LEVEL CDT 151914

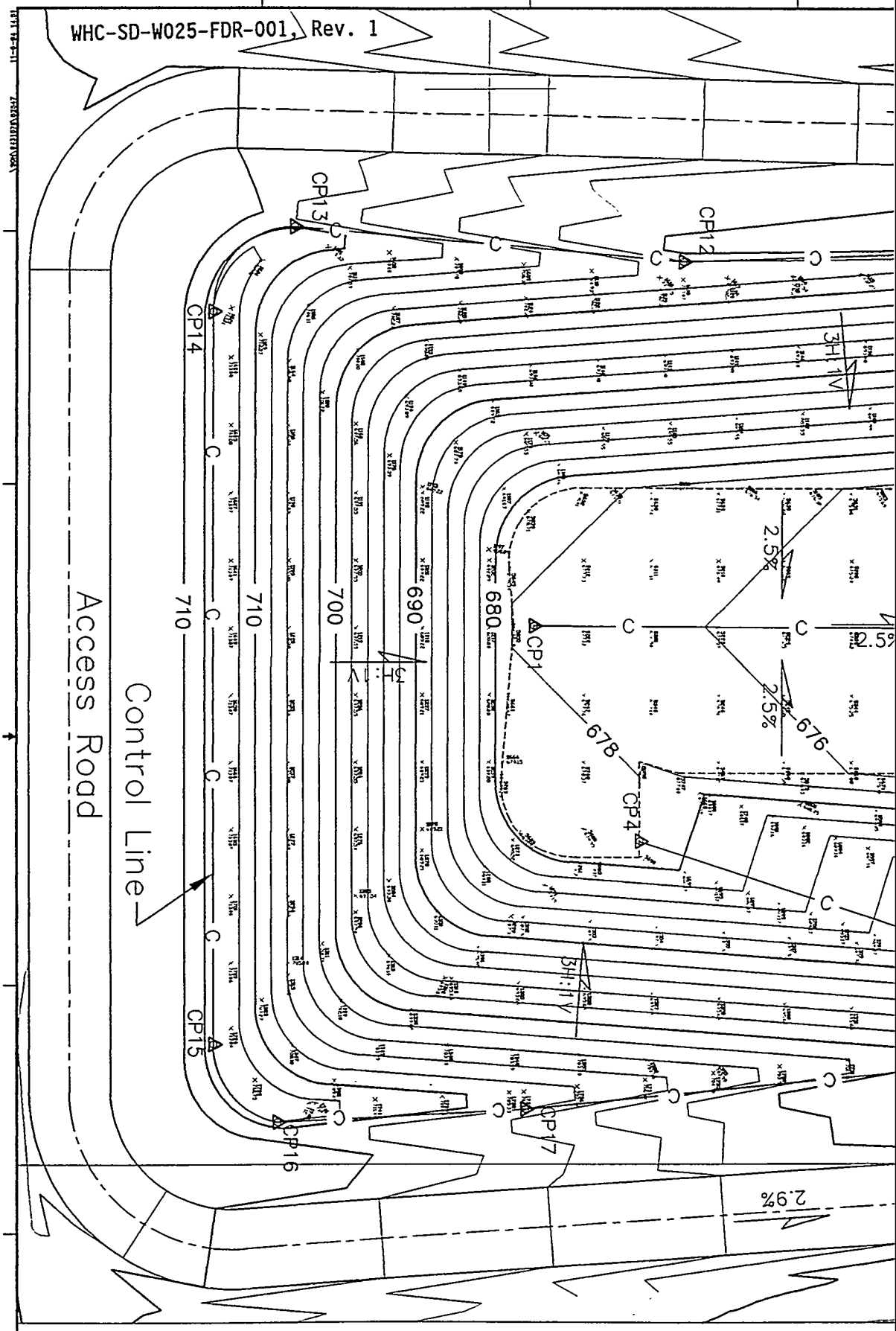
U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

OPERATIONS LAYER GRID POINTS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | DRAWING NO. 0401 | H-2-131593 | REV NO 3
SCALE AS SHOWN | JOB NO. 893-1166.035 | SHEET 1 OF 1

TITLE
WING TRACEABILITY LIST

NUMBER TITLE
REFERENCES
MOST USED ON

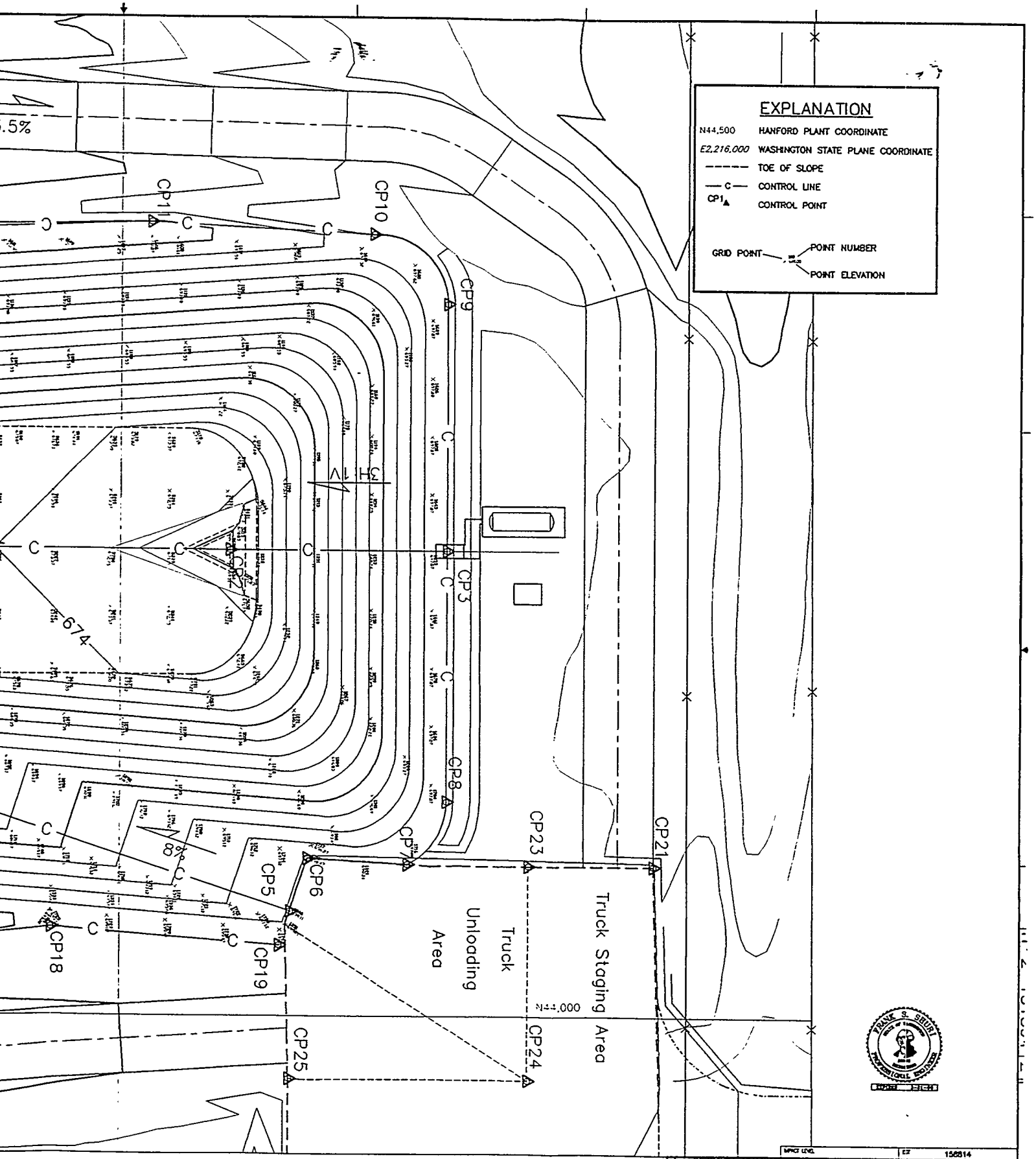


NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degrass Aerial Mapping based on July 18, 1988, aerial photography.
3. Indicated elevations do not include geosynthetic thickness and grading tolerances. Maintain required layer thickness at all locations.

0 20 40
FEET

N



DESIGNED	K. HASKELL	05/01/92	
DRAWN	K. HASKELL	05/01/92	
CHECKED	F. SHURI	05/06/92	
REVIEWED	G. MAJOR	05/06/92	
QA	J. GORMAN	05/07/92	
APPROVED FOR IMPLEMENTATION	BY		
	FOR	DATE	

U. S. DEPARTMENT OF ENERGY
RICHLAND OPERATIONS OFFICE
GOLDER ASSOCIATES INC.

PRIMARY ADMIX GRID POINTS
RMW LAND DISPOSAL FACILITY
NON-DRAG-OFF

PROJECT W-025
SIZE F | BLDG. NO. 0401 | DWG. NO. H-2-131594 | REV. NO. 2
SCALE AS SHOWN | JOB NO. 923-1030 | SHEET 1 OF 1

NO.	DESCRIPTION	BY	DATE
2	AS-BUILT PER ECN [illegible]	SGH	F. SHURI 8/28/94
1	MINOR CHANGE	JPP	F. SHURI 5/21/93

APPROVALS

REVISIONS

CAD FILE B131594A | CAD CODE 2W:IBM:ACD2:10:SS

REFERENCES

NUMBER	TITLE

NG TRACEABILITY LIST

IMPACT LEVEL | EZ 156814

DESIGN REPORT
PROJECT W-025, RADIOACTIVE MIXED WASTE (RMW)
LAND DISPOSAL FACILITY
NON-DRAG-OFF

REVISION 1

VOLUME II

Prepared for the
U.S. Department of Energy-Richland Operations Office
Contract No. DE-AC06-89RL11615

by
Golder Associates Inc.
Seattle (Redmond), Washington
Richland, Washington

Approval:

Project Director	Date 11-17-94
Project Manager	Date 11-17-94
QA Manager	Date
Design Engineer	Date 11-17-94
Westinghouse Hanford Company	Date

APPENDIX C

DESIGN CALCULATIONS

VOLUME II

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C.1 Surface Water Hydrology/HELP Model
C.2 Leachate Tank/Secondary Containment System
C.3 Primary Leachate Collection System
C.4 Tank Secondary Containment System/Concrete Structural Design
C.5 Secondary Leachate Collection Sump/Leak Detection System
C.6 Vertical Riser Pipe Foundation - Structural Design
C.7 Sump Design
C.8 Subgrade Side Slope Stability
C.9 Liner Side Slope Stability
C.10 Liner Tension Caused by Thermal Contraction
C.11 Anchor Trench Pull Out Resistance
C.12 Ramp Stability Analysis
C.13 Bearing Capacity of Subgrade Soils
C.14 Bearing Capacity of Liner Soils
C.15 Stability Checks on Barrier Block Wall
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C.28 Vertical Riser Pipe Seismic Design Analysis

Calculation Cover Sheet

Sheet 1 of 75

Project Title: WHC- Non Drag Off

Project No. 893- 1186.032

Calculation Title: Surface Water Hydrology + Help Runs - Appendices B.21 and B.11

	Name	Date
Prepared By:	MICHAEL BROWN	Dec. 1989
Checked By:	JOHN VERMEULEN	JANUARY 19, 1990
Approved By:	[signature]	8 FEB 90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

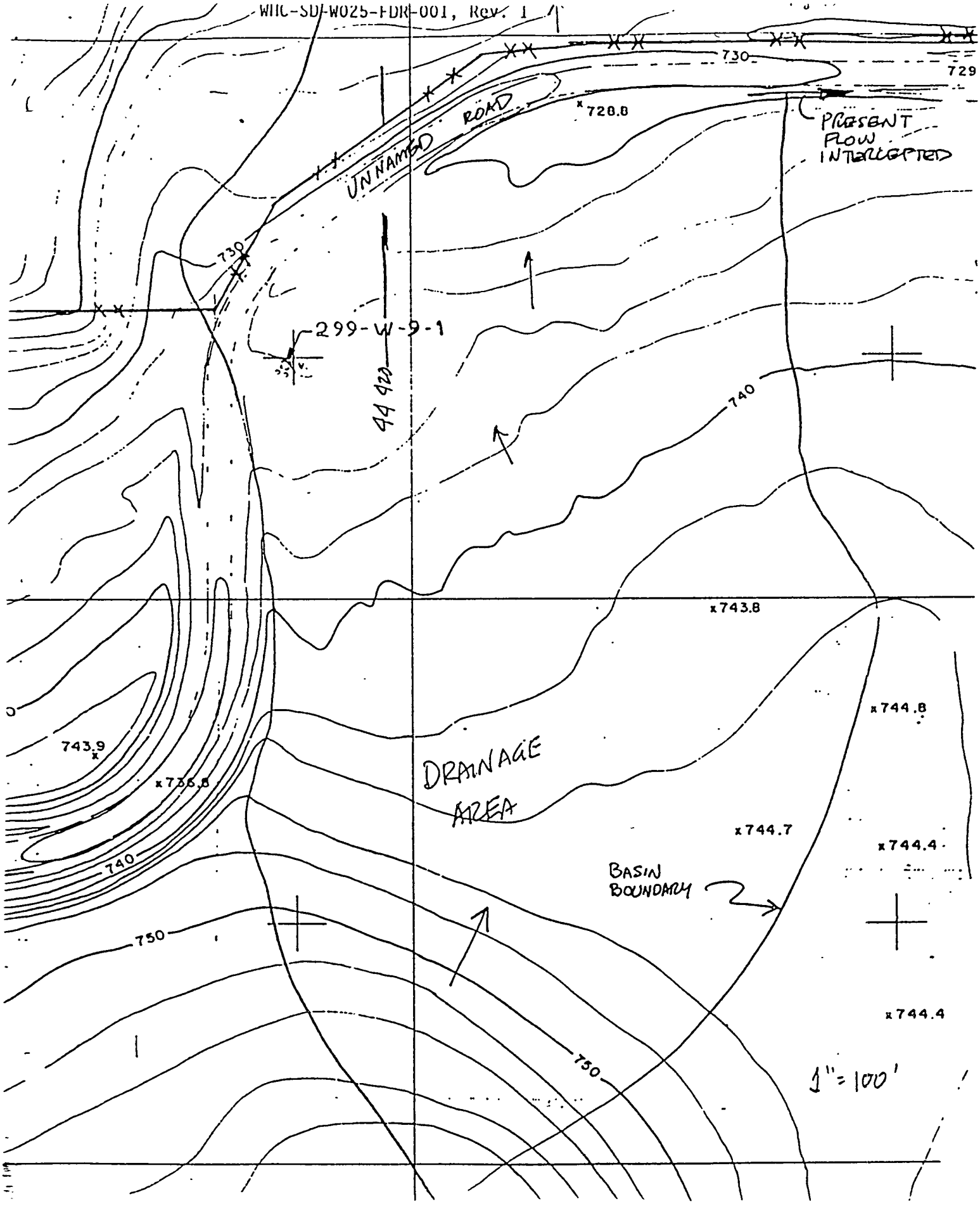
Golder Associates

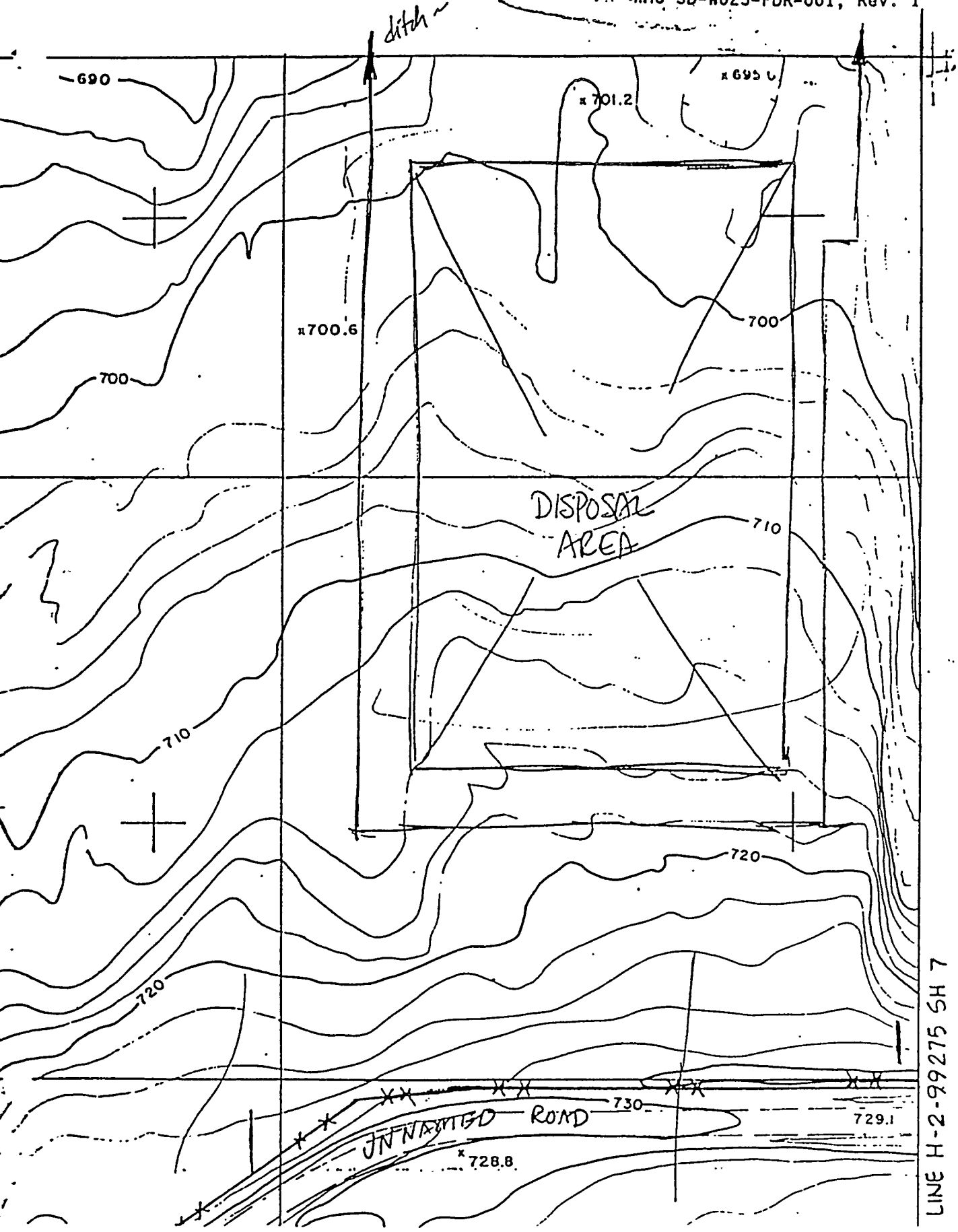
SUBJECT	NON DRAG OFF / SURFACE WATER		
Job No. 893-1186.032 Ref.	Made by MWB Checked JSV Reviewed [illegible]	Date 11/13/89 Sheet 2 of [illegible] 75	

OBJECTIVES:

The waste pit is to be surrounded by 2 V ditch which will intercept flow - both run-on and run off. During operation it will intercept off-site runoff and convey it to a designated discharge point. Following closure ditch will intercept flow from both on sit and offsit to discharge point.

Estimate the peak runoff from the tributary basin at key locations from the design flood.


730
729
728.8
PRESENT FLOW INTERCEPTED
UNNAMED ROAD
730
299-W-9-1
740
743.8
744.8
743.9
736.8
DRAINAGE AREA
744.7
744.4
740
BASIN BOUNDARY
750
744.4
750
1" = 100'


ditch
690
695
701.2
700.6
700
700
DISPOSAL AREA
710
710
720
720
UNNAMED ROAD
730
729.1
728.8
LINE H-2-99275 SH 7

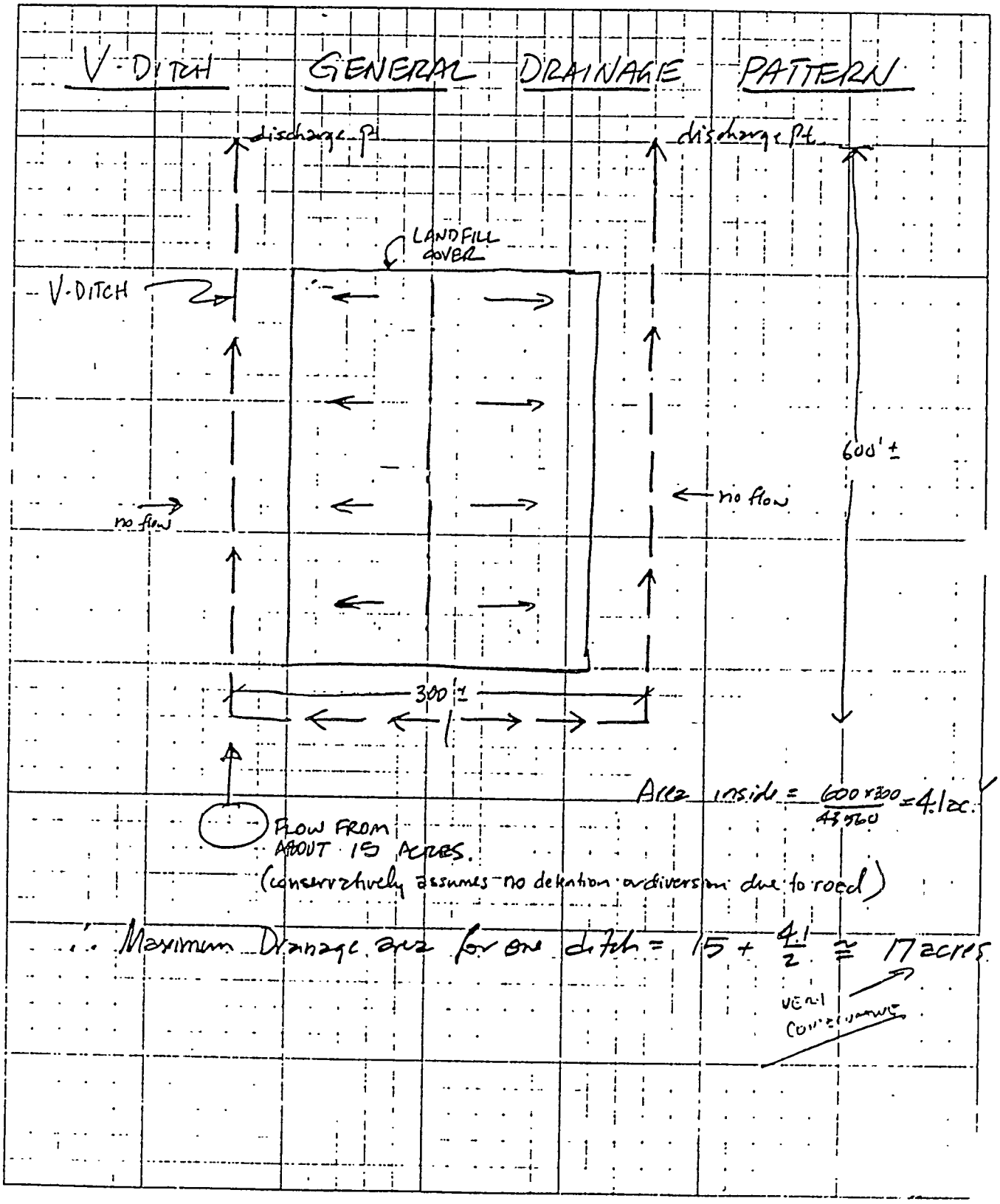
Golder Associates

SUBJECT Non Drag Off V-DITCH

Job No. 873-1186.032 Ref.

Made by [illegible] Checked JSV Reviewed [illegible]

Date 11/13/89 Sheet 5 of 75



pg 6 of 75

TO USE THIS CHART, SELECT ANY DESIRED RAINFALL INTENSITY AND DURATION AND READ FROM THE DIAGONAL LINES THE EXPECTED FREQUENCY OF SUCH INTENSITY AND DURATION. FOR EXAMPLE, RAINFALL INTENSITY OF 1.3 INCHES PER HOUR FOR 10 MINUTES CAN BE EXPECTED TO OCCUR, ON AVERAGE, ONCE EVERY 5 YEARS (POINT A). HOWEVER, SUCH INTENSITY CAN BE EXPECTED FOR 30 MINUTES DURATION ONLY ABOUT ONCE IN 100 YEARS (POINT B). THE RETURN PERIOD FOR THIS INTENSITY FOR 60 MINUTES DURATION IS GREATER THAN 1000 YEARS (POINT C).

THERE ARE, OF COURSE, VARIATIONS IN USE OF THE CHART. SUPPOSE, FOR EXAMPLE, IT IS DESIRED TO FIND THE "100-YEAR STORM" FOR 60 MINUTES. THIS IS 0.8 INCH (POINT D).

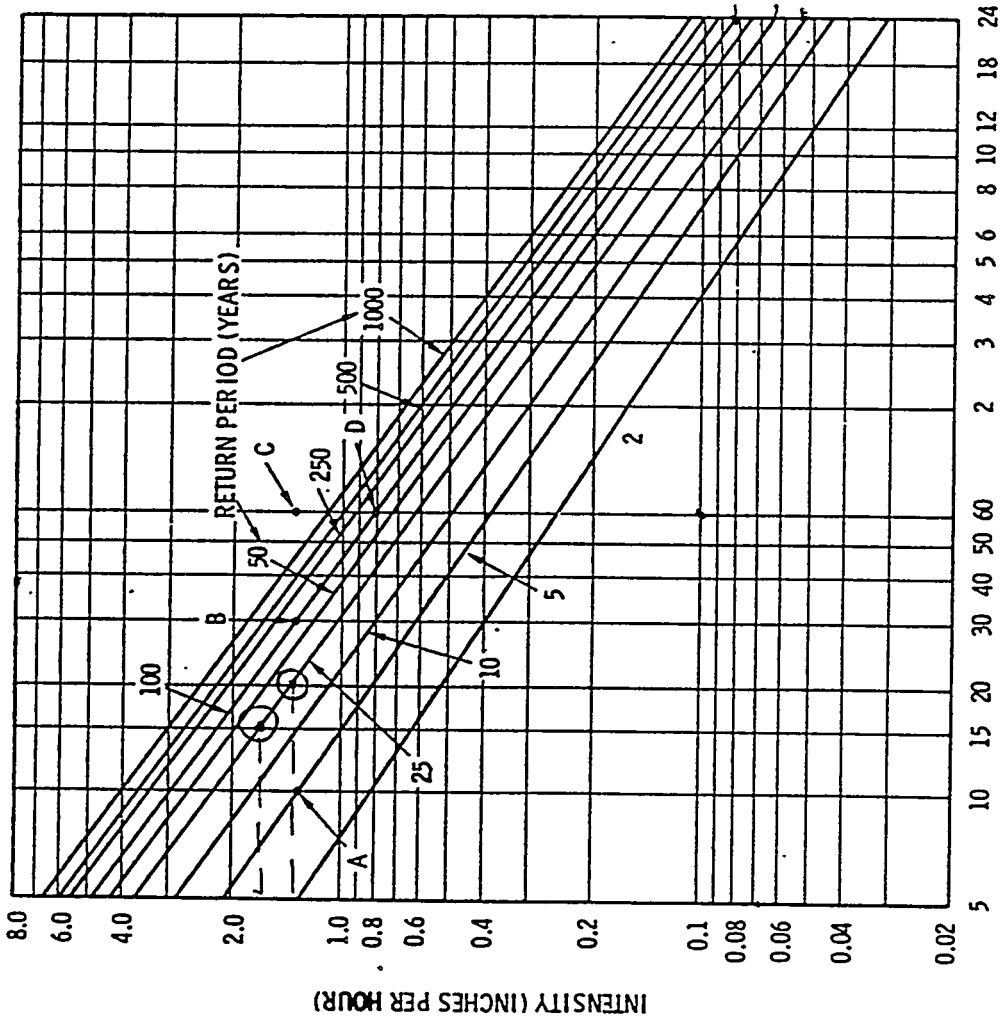


FIGURE 36. Rainfall Intensity, Duration and Frequency Based on the Period 1947 to 1969 at Hanford

TABLE 61. Average Return Period (R) and Existing Record (ER) for Various Precipitation Amounts and Intensity During Specified Time Periods at Hanford (Based on Extreme Value Analysis of 1947 Through 1969 Records)

	AMOUNT (INCHES)							INTENSITY (INCHES PER HOUR)						
	TIME PERIOD							TIME PERIOD						
R (YEARS)	20 MIN	60 MIN	2 HRS	3 HRS	6 HRS	12 HRS	24 HRS	20 MIN	60 MIN	2 HRS	3 HRS	6 HRS	12 HRS	24 HRS
2	0.16	0.26	0.30	0.36	0.48	0.62	0.72	0.49	0.26	0.15	0.12	0.08	0.052	0.030
5	0.24	0.40	0.48	0.55	0.77	0.95	1.06	0.72	0.40	0.24	0.18	0.13	0.079	0.044
10	0.37	0.50	0.59	0.67	0.96	1.17	1.28	1.1	0.50	0.30	0.22	0.16	0.098	0.053
25	0.47	0.62	0.74	0.83	1.21	1.45	1.56	1.4	0.62	0.37	0.28	0.20	0.121	0.065
50	0.53	0.72	0.85	0.96	1.40	1.66	1.77	1.6	0.72	0.42	0.32	0.23	0.138	0.074
100	0.60	0.81	0.96	1.07	1.59	1.87	1.99	1.8	0.81	0.48	0.36	0.27	0.156	0.083
250	0.68	0.93	1.11	1.22	1.82	2.13	2.26	2.0	0.93	0.55	0.41	0.30	0.177	0.094
500	0.73	1.02	1.22	1.33	2.00	2.34	2.47	2.2	1.02	0.61	0.44	0.33	0.195	0.103
1000	0.80	1.11	1.33	1.45	2.20	2.55	2.68	2.4	1.11	0.67	0.48	0.37	0.212	0.112
ER	*	0.59	0.88	1.08	1.68	1.88	1.91	*	0.59	0.44	0.36	0.28	0.157	0.080
		6/12	10/1	10/1	10/1-2	10/1-2	10/1-2		6/12	10/1	10/1	10/1-2	10/1-2	10/1-2
DATE	--	1969	1957	1957	1957	1957	1957	--	1969	1957	1957	1957	1957	1957

* NO RECORDS HAVE BEEN KEPT FOR TIME PERIODS OF LESS THAN 60 MINUTES. HOWEVER, THE RAIN GAGE CHART FOR 6-12-69 SHOWS THAT 0.55 INCH OCCURRED DURING A 20-MINUTE PERIOD FROM 1835 TO 1855 PST. AN ADDITIONAL 0.04 INCH OCCURRED BETWEEN 1855 AND 1910 TO ACCOUNT FOR THE RECORD 60-MINUTE AMOUNT OF 0.59 INCH.

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SUBJECT NONDRAW OFF V-DITCH		
Job No. 893-1186.032 Ref.	Made by MJS Checked JSV Reviewed RL	Date 11/13/89 Sheet 8 of 24 75

NATURAL GROUND SLOPE / MAX RAINFALL INTENSITY.

Approx Slope of natural ground from DWG H-2-99275
from EL 720 & EL 710 @ N 44 300 = $\frac{10'}{350'}$ = 2.9% ✓

Assume overland flow velocity ≅ 2 fps ✓ OK (NEARLY BARE GROUND 3% → 1.7 fps W/ SCS UPLAND METHOD)

Distance from Western Edge of Basin ditch discharge ≅ 2000 ft

Time to flow = $\frac{2000}{2 fps}$ = 1000 sec ≅ 15 minutes ✓

∴ use rainfall duration of 15 minutes to compute intensity in in/hr ✓

from Climatological Summary for Hanford (Table 61)
the 20 min precip (25 yr) = 0.47 in = 1.4 in/hr ✓

∴ use 1.5 in/hr as design intensity OK

POSSIBLE LOW IF tc RETURNED CLOSER TO 10 MIN

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SUBJECT NONDRAIN OFF - V DITCH DESIGN

Job No. 883-1186.032 Ref.

Made by [illegible] Checked JSU Reviewed [illegible]

Date 11/13/89 Sheet 4 of 9

Compute Q_{max} RATIONAL FORMULA

$Q = CIA$ = Rational formula [applicable for $A < 200$ acres]

$I = 1.5$ in/hr

$A = 17$ acres

C = coefficient dependent on basin characteristics,

for flat lawn (2%) on sandy soil $C = 0.05$ to 0.10

Assume sandy soil — slope is ~ 2%

Assume C double since there is no lawn

$C = 0.20$ — OTHER JUSTIFICATION?

$Q = CIA = (0.2)(1.5)(17) = 5.1$ cfs.

TR-55 w/
CN = 80
t_c = .16 hr = 10 min
Q_{peak} = 6 cfs
10 Flow Duration
Soil
free

NON DRAG OFF V-DITCH

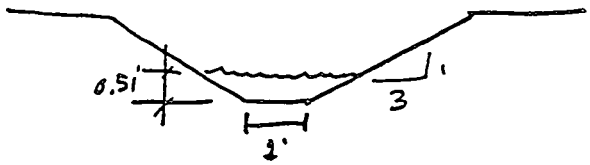
Sheet 10 of 75

Name - mlb
Date - 11/13/89
Project number - 893-1186

w = 5.00 cfs
Bottom width = 1.00 ft
Side slope = 3.00:1
Channel bed slope = .02000
Mannings n = .0250
Depth - .51 ft
Velocity - 3.84 ft/s OK
Froude number - 1.20

Critical depth - .56 ft

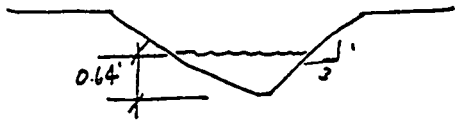
Critical Slope = .0137



Flow = 5.10 cfs
Bottom width = .10 ft
Side slope = 3.00:1
Channel bed slope = .02000
Mannings n = .0250
Depth - .64 ft
Velocity - 3.90 ft/s OK
Froude number - 1.20

Critical depth - .69 ft

Critical Slope = .0138



Assume:
1) Grass lined ditch → n = .025
2) Clear water → max. velocity - use 5.10 cfs
3) Supercritical flow does not go subcritical anywhere in ditch section

w = 5.10 cfs
Bottom width = .10 ft
Side slope = 3.00:1
Channel bed slope = .01000
Mannings n = .0200
Depth - ~~.67 ft~~ .55
Velocity - ~~3.56 ft/s~~ 5.37 fps ← too fast, keep slope less than 2.5% without erosion protection or drops.
Froude number - ~~1.07~~ 1.71

Critical depth - .69 ft

Critical Slope = .0088

Flow = 5.10 cfs
Bottom width = .10 ft
Side slope = 3.00:1
Channel bed slope = .00200 ← minimum slope.
Mannings n = .0250
Depth - 1.00 ft ← maximum depth.
Velocity - 1.64 ft/s OK
Froude number - .41

Critical depth - .69 ft

Critical Slope = .0138

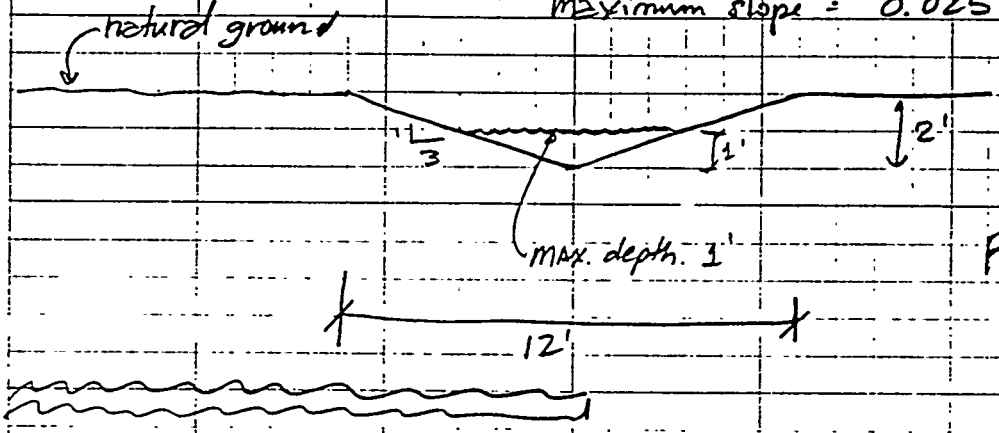
Check:
1) Subcritical flow → ok depth at

Golder Associates

SUBJECT NON DRAG OFF			
Job No. 893-1186	Made by MPBrown	Date 11-13-89	
Ref. .032	Checked JSV	Sheet 11 of 75	
	Reviewed PE		

V-DITCH CROSS SECTION

Minimum slope = 0.002
Maximum slope = 0.025



FB = 1.0 [illegible] MORE THAN REQUIRED FOR Q = 5.1 CFS
Q = 6 CFS

Check parameters for culvert pipe

for Diam = 2' S = .02 n = .02
depth = 0.67' V = 5.49 fps (supercritical)

Dia = 2 1 = .81
S = .01 V = 4.27 (still supercritical, but just barely)
n = .02

OK.

Use 24"Φ culvert pipe,
bed slope = 0.01 (1%)

? INLET OR OUTLET CONTROL ?
IF INLET CTRL $h_w/D = 0.5 - 0.52$
2 SERIES CURVES
12" ACCEPTABLE

Golder Associates

SUBJECT NON DRAG OFF HYDROLOGY-Flood		
Job No. 883-1186.032 Ref. 893	Made by ML. Brown Checked JSU Reviewed	Date 11/22/89 Sheet 12 of 75

REGRESSION EQN's

REFERENCE 1 – Hydrology Studies within the Columbia Plateau, Wash. An Integration of Current knowledge, Oct 1979 Rockwell International

Ref 1 References PAGE III-41 Cummans et al 1975, USGS

COMPUTE PEAK 25YR FLOW FOR SMALL D.A.

PASCO BASIN & HANFORD SITE ARE REGION XII

$$Q_T = a A^{b_1} P^{b_2} F^{b_3}$$

where Q_T = cfs peak flow

A = drainage area, mi^2 [minimum 1.8 mi^2]

P = mean annual Precip. [USE 10]

F = % forest cover, [0.01 min]

a = 1.76 for Region XII, 25 yr flood

b_1 = 0.87

b_2 = 1.03

b_3 = −0.25

use A = 1.0

$Q = 1.76\ A^{0.87}\ 10^{1.03}\ 0.01^{-0.25}$

$Q = 1.76\ A^{0.87}\ 10.72 \cdot 3.16 = 59.66\ A^{0.87}$

Because these values are based on measured streamflow they include consideration of snowmelt.

Golder Associates

SUBJECT	NON DRAG OFF		FLOOD HYDROLOGY
Job No. 883-1186.032	Made by M.L. BROWN	Date 11/22/89	
Ref. 893	Checked JSV	Sheet 12 of 13	75
	Reviewed		

$Q = 60 A^{0.87}$

A ACRES	A SQ MI	Q cfs	cfs/mi²
1280	2	110	55
640	1	60	60
320	0.5	33	66
160	0.25	18	72
64	0.1	8	80
10	0.0156	1.6	103
17	0.0266	2.5	94 ←

ALTHOUGH THE D.A IS OUTSIDE RANGE (<1.8 mi²)

IT DOES APPEAR TO BE A REASONABLE APPROXIMATION.

[THIS INCLUDES SNOWMELT]

Using the rational method again —

time of concentration = $\frac{L}{V}$ L = basin hydraulic length; V = average velocity

Estimate L = 1200', V = 2 fps → $\frac{1200}{2}$ = 600 sec = 10 min

Max rainfall intensity for 10 min interval (from Fig 36, Hanford) $i_{10\,min}^{2\,yr} \approx 2.3$ in/hr.

Estimate C = 0.2

$Q = CIA = (0.2)(2.3)(10) = 4.6$ cfs. (10 acres with ? (as one image?))

Using HEC1 with CN = 65 Q = 0 because CN 65 produces no runoff with 1.56" precip storm.

CN = 85 Q_{max} = 9 cfs; QUERLY comparable

CN=65

pg 14 of 75

****

FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
****

$Q_{MAX} = 0$

17 CMS

THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3

1 HEC-1 INPUT PAGE 1

```
      LINE          ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10
        1           ID   WHC NON-DRAGOFF
        2           ID   V-Ditch Inflow Hydrology
        3           IT       6 14NOV89    1200     300
        4           IN       6 14NOV89    1206
        5           IO       1        1
*** FREE ***
        6           KK 25_year 24-hour storm, P = 1.56"
        7           BA  0.0266
        8           KM   TYPE II DISTRIBUTION 24 HOUR DURATION
        9           PB    1.56
       10           PC   0.001   0.002   0.003   0.004   0.005   0.006   0.007   0.008   0.010   0.011
       11           PC   0.012   0.013   0.014   0.015   0.016   0.017   0.018   0.020   0.021   0.022
       12           PC   0.023   0.024   0.026   0.027   0.028   0.029   0.030   0.032   0.033   0.034
       13           PC   0.036   0.037   0.038   0.039   0.041   0.042   0.043   0.045   0.046   0.048
       14           PC   0.049   0.050   0.052   0.053   0.055   0.056   0.058   0.059   0.061   0.062
       15           PC   0.064   0.065   0.067   0.069   0.070   0.072   0.074   0.075   0.077   0.079
       16           PC   0.080   0.082   0.084   0.086   0.088   0.089   0.091   0.093   0.095   0.097
       17           PC   0.099   0.101   0.103   0.105   0.108   0.110   0.112   0.114   0.117   0.119
       18           PC   0.121   0.124   0.126   0.129   0.131   0.134   0.137   0.140   0.142   0.145
       19           PC   0.148   0.151   0.155   0.158   0.161   0.165   0.168   0.172   0.176   0.180
       20           PC   0.184   0.188   0.193   0.197   0.202   0.207   0.213   0.219   0.225   0.232
       21           PC   0.239   0.247   0.256   0.266   0.277   0.290   0.307   0.329   0.365   0.500
       22           PC   0.635   0.671   0.693   0.710   0.723   0.734   0.744   0.753   0.761   0.768
       23           PC   0.775   0.781   0.787   0.793   0.798   0.803   0.807   0.812   0.816   0.820
       24           PC   0.824   0.828   0.832   0.835   0.839   0.842   0.845   0.849   0.852   0.855
       25           PC   0.858   0.860   0.863   0.866   0.869   0.871   0.874   0.876   0.879   0.881
       26           PC   0.883   0.886   0.888   0.890   0.892   0.895   0.897   0.899   0.901   0.903
       27           PC   0.905   0.907   0.909   0.911   0.912   0.914   0.916   0.918   0.920   0.921
       28           PC   0.923   0.925   0.926   0.928   0.930   0.931   0.933   0.935   0.936   0.938
       29           PC   0.939   0.941   0.942   0.944   0.945   0.947   0.948   0.950   0.951   0.952
       30           PC   0.954   0.955   0.957   0.958   0.959   0.961   0.962   0.963   0.964   0.966
       31           PC   0.967   0.968   0.970   0.971   0.972   0.973   0.974   0.976   0.977   0.978
       32           PC   0.979   0.980   0.982   0.983   0.984   0.985   0.986   0.987   0.988   0.989
       33           PC   0.990   0.992   0.993   0.994   0.995   0.996   0.997   0.998   0.999   1.000
       34           LS       0      65
       35           UD   0.150
       36           ZZ
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1

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FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
****

WHC NON-DRAGOFF
V-Ditch Inflow Hydrology

5 ID OUTPUT CONTROL VARIABLES

IPRNT	1	PRINT CONTROL
IPLOT	1	PLOT CONTROL
QSCAL	0.	HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA

NMIN	6	MINUTES IN COMPUTATION INTERVAL
IDATE	14NOV89	STARTING DATE
ITIME	1200	STARTING TIME
NQ	300	NUMBER OF HYDROGRAPH ORDINATES
NDDATE	15NOV89	ENDING DATE
NDTIME	1754	ENDING TIME

```
                COMPUTATION INTERVAL     .10 HOURS
                    TOTAL TIME BASE   29.90 HOURS

        ENGLISH UNITS

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 6 KK     *  25_year   *      24-hour storm, P = 1.56"
          *            *
          **************
                       TYPE II DISTRIBUTION 24 HOUR DURATION

 4 IN          TIME DATA FOR INPUT TIME SERIES
                    JXMIN           6  TIME INTERVAL IN MINUTES
                   JXDATE     14NOV89  STARTING DATE
                   JXTIME        1206  STARTING TIME

             SUBBASIN RUNOFF DATA

 7 BA          SUBBASIN CHARACTERISTICS
                    TAREA         .03  SUBBASIN AREA

               PRECIPITATION DATA

 9 PB               STORM        1.56  BASIN TOTAL PRECIPITATION

10 PI            INCREMENTAL PRECIPITATION PATTERN
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .01   .00   .01   .00   .01   .01   .01   .01
                   .01   .01   .01   .01   .01   .01   .02   .02   .04   .13
                   .13   .04   .02   .02   .01   .01   .01   .01   .01   .01
                   .01   .01   .01   .01   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                   .00   .00   .00   .00   .00   .00   .00   .00   .00   .00

34 LS          SCS LOSS RATE
                    STRTL        1.08  INITIAL ABSTRACTION
                   CRVNBR       65.00  CURVE NUMBER
                    RTIMP         .00  PERCENT IMPERVIOUS AREA

35 UD          SCS DIMENSIONLESS UNITGRAPH
                    TLAG         .15  LAG

                                                    ***

WARNING *** TIME INTERVAL IS GREATER THAN .29*LAG

                                                UNIT HYDROGRAPH
                                          10 END-OF-PERIOD ORDINATES
              30.      65.      44.      18.       8.       4.       2.       1.       0.       0.

 ************************************************************************************************************************

                                          HYDROGRAPH AT STATION  25_year

 ************************************************************************************************************************
                                                    *
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DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA	MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
14	NOV	1200	1	.00	.00	.00	0.	*	15	NOV	0300	151	.00	.00	.00	0.
14	NOV	1206	2	.00	.00	.00	0.	*	15	NOV	0306	152	.00	.00	.00	0.
14	NOV	1212	3	.00	.00	.00	0.	*	15	NOV	0312	153	.00	.00	.00	0.
14	NOV	1218	4	.00	.00	.00	0.	*	15	NOV	0318	154	.00	.00	.00	0.
14	NOV	1224	5	.00	.00	.00	0.	*	15	NOV	0324	155	.00	.00	.00	0.
14	NOV	1230	6	.00	.00	.00	0.	*	15	NOV	0330	156	.00	.00	.00	0.
14	NOV	1236	7	.00	.00	.00	0.	*	15	NOV	0336	157	.00	.00	.00	0.
14	NOV	1242	8	.00	.00	.00	0.	*	15	NOV	0342	158	.00	.00	.00	0.
14	NOV	1248	9	.00	.00	.00	0.	*	15	NOV	0348	159	.00	.00	.00	0.
14	NOV	1254	10	.00	.00	.00	0.	*	15	NOV	0354	160	.00	.00	.00	0.
14	NOV	1300	11	.00	.00	.00	0.	*	15	NOV	0400	161	.00	.00	.00	0.
14	NOV	1306	12	.00	.00	.00	0.	*	15	NOV	0406	162	.00	.00	.00	0.
14	NOV	1312	13	.00	.00	.00	0.	*	15	NOV	0412	163	.00	.00	.00	0.
14	NOV	1318	14	.00	.00	.00	0.	*	15	NOV	0418	164	.00	.00	.00	0.
14	NOV	1324	15	.00	.00	.00	0.	*	15	NOV	0424	165	.00	.00	.00	0.
14	NOV	1330	16	.00	.00	.00	0.	*	15	NOV	0430	166	.00	.00	.00	0.
14	NOV	1336	17	.00	.00	.00	0.	*	15	NOV	0436	167	.00	.00	.00	0.
14	NOV	1342	18	.00	.00	.00	0.	*	15	NOV	0442	168	.00	.00	.00	0.
14	NOV	1348	19	.00	.00	.00	0.	*	15	NOV	0448	169	.00	.00	.00	0.
14	NOV	1354	20	.00	.00	.00	0.	*	15	NOV	0454	170	.00	.00	.00	0.
14	NOV	1400	21	.00	.00	.00	0.	*	15	NOV	0500	171	.00	.00	.00	0.
14	NOV	1406	22	.00	.00	.00	0.	*	15	NOV	0506	172	.00	.00	.00	0.
14	NOV	1412	23	.00	.00	.00	0.	*	15	NOV	0512	173	.00	.00	.00	0.
14	NOV	1418	24	.00	.00	.00	0.	*	15	NOV	0518	174	.00	.00	.00	0.
14	NOV	1424	25	.00	.00	.00	0.	*	15	NOV	0524	175	.00	.00	.00	0.
14	NOV	1430	26	.00	.00	.00	0.	*	15	NOV	0530	176	.00	.00	.00	0.
14	NOV	1436	27	.00	.00	.00	0.	*	15	NOV	0536	177	.00	.00	.00	0.
14	NOV	1442	28	.00	.00	.00	0.	*	15	NOV	0542	178	.00	.00	.00	0.
14	NOV	1448	29	.00	.00	.00	0.	*	15	NOV	0548	179	.00	.00	.00	0.
14	NOV	1454	30	.00	.00	.00	0.	*	15	NOV	0554	180	.00	.00	.00	0.
14	NOV	1500	31	.00	.00	.00	0.	*	15	NOV	0600	181	.00	.00	.00	0.
14	NOV	1506	32	.00	.00	.00	0.	*	15	NOV	0606	182	.00	.00	.00	0.
14	NOV	1512	33	.00	.00	.00	0.	*	15	NOV	0612	183	.00	.00	.00	0.
14	NOV	1518	34	.00	.00	.00	0.	*	15	NOV	0618	184	.00	.00	.00	0.
14	NOV	1524	35	.00	.00	.00	0.	*	15	NOV	0624	185	.00	.00	.00	0.
14	NOV	1530	36	.00	.00	.00	0.	*	15	NOV	0630	186	.00	.00	.00	0.
14	NOV	1536	37	.00	.00	.00	0.	*	15	NOV	0636	187	.00	.00	.00	0.
14	NOV	1542	38	.00	.00	.00	0.	*	15	NOV	0642	188	.00	.00	.00	0.
14	NOV	1548	39	.00	.00	.00	0.	*	15	NOV	0648	189	.00	.00	.00	0.
14	NOV	1554	40	.00	.00	.00	0.	*	15	NOV	0654	190	.00	.00	.00	0.
14	NOV	1600	41	.00	.00	.00	0.	*	15	NOV	0700	191	.00	.00	.00	0.
14	NOV	1606	42	.00	.00	.00	0.	*	15	NOV	0706	192	.00	.00	.00	0.
14	NOV	1612	43	.00	.00	.00	0.	*	15	NOV	0712	193	.00	.00	.00	0.
14	NOV	1618	44	.00	.00	.00	0.	*	15	NOV	0718	194	.00	.00	.00	0.
14	NOV	1624	45	.00	.00	.00	0.	*	15	NOV	0724	195	.00	.00	.00	0.
14	NOV	1630	46	.00	.00	.00	0.	*	15	NOV	0730	196	.00	.00	.00	0.
14	NOV	1636	47	.00	.00	.00	0.	*	15	NOV	0736	197	.00	.00	.00	0.
14	NOV	1642	48	.00	.00	.00	0.	*	15	NOV	0742	198	.00	.00	.00	0.
14	NOV	1648	49	.00	.00	.00	0.	*	15	NOV	0748	199	.00	.00	.00	0.
14	NOV	1654	50	.00	.00	.00	0.	*	15	NOV	0754	200	.00	.00	.00	0.
14	NOV	1700	51	.00	.00	.00	0.	*	15	NOV	0800	201	.00	.00	.00	0.
14	NOV	1706	52	.00	.00	.00	0.	*	15	NOV	0806	202	.00	.00	.00	0.
14	NOV	1712	53	.00	.00	.00	0.	*	15	NOV	0812	203	.00	.00	.00	0.
14	NOV	1718	54	.00	.00	.00	0.	*	15	NOV	0818	204	.00	.00	.00	0.
14	NOV	1724	55	.00	.00	.00	0.	*	15	NOV	0824	205	.00	.00	.00	0.
14	NOV	1730	56	.00	.00	.00	0.	*	15	NOV	0830	206	.00	.00	.00	0.
14	NOV	1736	57	.00	.00	.00	0.	*	15	NOV	0836	207	.00	.00	.00	0.
14	NOV	1742	58	.00	.00	.00	0.	*	15	NOV	0842	208	.00	.00	.00	0.
14	NOV	1748	59	.00	.00	.00	0.	*	15	NOV	0848	209	.00	.00	.00	0.
14	NOV	1754	60	.00	.00	.00	0.	*	15	NOV	0854	210	.00	.00	.00	0.
14	NOV	1800	61	.00	.00	.00	0.	*	15	NOV	0900	211	.00	.00	.00	0.
14	NOV	1806	62	.00	.00	.00	0.	*	15	NOV	0906	212	.00	.00	.00	0.
14	NOV	1812	63	.00	.00	.00	0.	*	15	NOV	0912	213	.00	.00	.00	0.
14	NOV	1818	64	.00	.00	.00	0.	*	15	NOV	0918	214	.00	.00	.00	0.
14	NOV	1824	65	.00	.00	.00	0.	*	15	NOV	0924	215	.00	.00	.00	0.
14	NOV	1830	66	.00	.00	.00	0.	*	15	NOV	0930	216	.00	.00	.00	0.
14	NOV	1836	67	.00	.00	.00	0.	*	15	NOV	0936	217	.00	.00	.00	0.
14	NOV	1842	68	.00	.00	.00	0.	*	15	NOV	0942	218	.00	.00	.00	0.
14	NOV	1848	69	.00	.00	.00	0.	*	15	NOV	0948	219	.00	.00	.00	0.
14	NOV	1854	70	.00	.00	.00	0.	*	15	NOV	0954	220	.00	.00	.00	0.
14	NOV	1900	71	.00	.00	.00	0.	*	15	NOV	1000	221	.00	.00	.00	0.
14	NOV	1906	72	.00	.00	.00	0.	*	15	NOV	1006	222	.00	.00	.00	0.
14	NOV	1912	73	.00	.00	.00	0.	*	15	NOV	1012	223	.00	.00	.00	0.
14	NOV	1918	74	.00	.00	.00	0.	*	15	NOV	1018	224	.00	.00	.00	0.
14	NOV	1924	75	.00	.00	.00	0.	*	15	NOV	1024	225	.00	.00	.00	0.
14	NOV	1930	76	.00	.00	.00	0.	*	15	NOV	1030	226	.00	.00	.00	0.
14	NOV	1936	77	.00	.00	.00	0.	*	15	NOV	1036	227	.00	.00	.00	0.
14	NOV	1942	78	.00	.00	.00	0.	*	15	NOV	1042	228	.00	.00	.00	0.

14 NOV 1948	79	.00	.00	.00	0.	*	15 NOV 1048	229	.00	.00	.00	0.
14 NOV 1954	80	.00	.00	.00	0.	*	15 NOV 1054	230	.00	.00	.00	0.
14 NOV 2000	81	.00	.00	.00	0.	*	15 NOV 1100	231	.00	.00	.00	0.
14 NOV 2006	82	.00	.00	.00	0.	*	15 NOV 1106	232	.00	.00	.00	0.
14 NOV 2012	83	.00	.00	.00	0.	*	15 NOV 1112	233	.00	.00	.00	0.
14 NOV 2018	84	.00	.00	.00	0.	*	15 NOV 1118	234	.00	.00	.00	0.
14 NOV 2024	85	.00	.00	.00	0.	*	15 NOV 1124	235	.00	.00	.00	0.
14 NOV 2030	86	.00	.00	.00	0.	*	15 NOV 1130	236	.00	.00	.00	0.
14 NOV 2036	87	.00	.00	.00	0.	*	15 NOV 1136	237	.00	.00	.00	0.
14 NOV 2042	88	.00	.00	.00	0.	*	15 NOV 1142	238	.00	.00	.00	0.
14 NOV 2048	89	.00	.00	.00	0.	*	15 NOV 1148	239	.00	.00	.00	0.
14 NOV 2054	90	.00	.00	.00	0.	*	15 NOV 1154	240	.00	.00	.00	0.
14 NOV 2100	91	.00	.00	.00	0.	*	15 NOV 1200	241	.00	.00	.00	0.
14 NOV 2106	92	.00	.00	.00	0.	*	15 NOV 1206	242	.00	.00	.00	0.
14 NOV 2112	93	.00	.00	.00	0.	*	15 NOV 1212	243	.00	.00	.00	0.
14 NOV 2118	94	.01	.01	.00	0.	*	15 NOV 1218	244	.00	.00	.00	0.
14 NOV 2124	95	.00	.00	.00	0.	*	15 NOV 1224	245	.00	.00	.00	0.
14 NOV 2130	96	.00	.00	.00	0.	*	15 NOV 1230	246	.00	.00	.00	0.
14 NOV 2136	97	.01	.01	.00	0.	*	15 NOV 1236	247	.00	.00	.00	0.
14 NOV 2142	98	.00	.00	.00	0.	*	15 NOV 1242	248	.00	.00	.00	0.
14 NOV 2148	99	.01	.01	.00	0.	*	15 NOV 1248	249	.00	.00	.00	0.
14 NOV 2154	100	.01	.01	.00	0.	*	15 NOV 1254	250	.00	.00	.00	0.
14 NOV 2200	101	.01	.01	.00	0.	*	15 NOV 1300	251	.00	.00	.00	0.
14 NOV 2206	102	.01	.01	.00	0.	*	15 NOV 1306	252	.00	.00	.00	0.
14 NOV 2212	103	.01	.01	.00	0.	*	15 NOV 1312	253	.00	.00	.00	0.
14 NOV 2218	104	.01	.01	.00	0.	*	15 NOV 1318	254	.00	.00	.00	0.
14 NOV 2224	105	.01	.01	.00	0.	*	15 NOV 1324	255	.00	.00	.00	0.
14 NOV 2230	106	.01	.01	.00	0.	*	15 NOV 1330	256	.00	.00	.00	0.
14 NOV 2236	107	.01	.01	.00	0.	*	15 NOV 1336	257	.00	.00	.00	0.
14 NOV 2242	108	.01	.01	.00	0.	*	15 NOV 1342	258	.00	.00	.00	0.
14 NOV 2248	109	.01	.01	.00	0.	*	15 NOV 1348	259	.00	.00	.00	0.
14 NOV 2254	110	.01	.01	.00	0.	*	15 NOV 1354	260	.00	.00	.00	0.
14 NOV 2300	111	.01	.01	.00	0.	*	15 NOV 1400	261	.00	.00	.00	0.
14 NOV 2306	112	.01	.01	.00	0.	*	15 NOV 1406	262	.00	.00	.00	0.
14 NOV 2312	113	.01	.01	.00	0.	*	15 NOV 1412	263	.00	.00	.00	0.
14 NOV 2318	114	.01	.01	.00	0.	*	15 NOV 1418	264	.00	.00	.00	0.
14 NOV 2324	115	.02	.02	.00	0.	*	15 NOV 1424	265	.00	.00	.00	0.
14 NOV 2330	116	.02	.02	.00	0.	*	15 NOV 1430	266	.00	.00	.00	0.
14 NOV 2336	117	.02	.02	.00	0.	*	15 NOV 1436	267	.00	.00	.00	0.
14 NOV 2342	118	.03	.03	.00	0.	*	15 NOV 1442	268	.00	.00	.00	0.
14 NOV 2348	119	.03	.03	.00	0.	*	15 NOV 1448	269	.00	.00	.00	0.
14 NOV 2354	120	.06	.06	.00	0.	*	15 NOV 1454	270	.00	.00	.00	0.
15 NOV 0000	121	.21	.21	.00	0.	*	15 NOV 1500	271	.00	.00	.00	0.
15 NOV 0006	122	.21	.21	.00	0.	*	15 NOV 1506	272	.00	.00	.00	0.
15 NOV 0012	123	.06	.06	.00	0.	*	15 NOV 1512	273	.00	.00	.00	0.
15 NOV 0018	124	.03	.03	.00	0.	*	15 NOV 1518	274	.00	.00	.00	0.
15 NOV 0024	125	.03	.03	.00	0.	*	15 NOV 1524	275	.00	.00	.00	0.
15 NOV 0030	126	.02	.02	.00	0.	*	15 NOV 1530	276	.00	.00	.00	0.
15 NOV 0036	127	.02	.02	.00	0.	*	15 NOV 1536	277	.00	.00	.00	0.
15 NOV 0042	128	.02	.02	.00	0.	*	15 NOV 1542	278	.00	.00	.00	0.
15 NOV 0048	129	.01	.01	.00	0.	*	15 NOV 1548	279	.00	.00	.00	0.
15 NOV 0054	130	.01	.01	.00	0.	*	15 NOV 1554	280	.00	.00	.00	0.
15 NOV 0100	131	.01	.01	.00	0.	*	15 NOV 1600	281	.00	.00	.00	0.
15 NOV 0106	132	.01	.01	.00	0.	*	15 NOV 1606	282	.00	.00	.00	0.
15 NOV 0112	133	.01	.01	.00	0.	*	15 NOV 1612	283	.00	.00	.00	0.
15 NOV 0118	134	.01	.01	.00	0.	*	15 NOV 1618	284	.00	.00	.00	0.
15 NOV 0124	135	.01	.01	.00	0.	*	15 NOV 1624	285	.00	.00	.00	0.
15 NOV 0130	136	.01	.01	.00	0.	*	15 NOV 1630	286	.00	.00	.00	0.
15 NOV 0136	137	.01	.01	.00	0.	*	15 NOV 1636	287	.00	.00	.00	0.
15 NOV 0142	138	.01	.01	.00	0.	*	15 NOV 1642	288	.00	.00	.00	0.
15 NOV 0148	139	.01	.01	.00	0.	*	15 NOV 1648	289	.00	.00	.00	0.
15 NOV 0154	140	.01	.01	.00	0.	*	15 NOV 1654	290	.00	.00	.00	0.
15 NOV 0200	141	.01	.01	.00	0.	*	15 NOV 1700	291	.00	.00	.00	0.
15 NOV 0206	142	.01	.01	.00	0.	*	15 NOV 1706	292	.00	.00	.00	0.
15 NOV 0212	143	.01	.01	.00	0.	*	15 NOV 1712	293	.00	.00	.00	0.
15 NOV 0218	144	.01	.01	.00	0.	*	15 NOV 1718	294	.00	.00	.00	0.
15 NOV 0224	145	.00	.00	.00	0.	*	15 NOV 1724	295	.00	.00	.00	0.
15 NOV 0230	146	.01	.01	.00	0.	*	15 NOV 1730	296	.00	.00	.00	0.
15 NOV 0236	147	.00	.00	.00	0.	*	15 NOV 1736	297	.00	.00	.00	0.
15 NOV 0242	148	.00	.00	.00	0.	*	15 NOV 1742	298	.00	.00	.00	0.
15 NOV 0248	149	.01	.01	.00	0.	*	15 NOV 1748	299	.00	.00	.00	0.
15 NOV 0254	150	.00	.00	.00	0.	*	15 NOV 1754	300	.00	.00	.00	0.

TOTAL RAINFALL = 1.56, TOTAL LOSS = 1.52, TOTAL EXCESS = .04

PEAK FLOW (CFS)	TIME (HR)	MAXIMUM AVERAGE FLOW 6-HR	24-HR	72-HR	29.90-HR

0.	13.50	(CFS)	0.	0.	0.	0.
		(INCHES)	.024	.040	.040	.040
		(AC-FT)	0.	0.	0.	0.

CUMULATIVE AREA = .03 SQ MI

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	25_year	0.	13.50	0.	0.	0.	.03		

*** NORMAL END OF HEC-1 ***

14 NOV 2224	105	.01	.01	.00	0.	*	15 NOV 1324	255	.00	.00	.00	0.
14 NOV 2230	106	.01	.01	.00	0.	*	15 NOV 1330	256	.00	.00	.00	0.
14 NOV 2236	107	.01	.01	.00	0.	*	15 NOV 1336	257	.00	.00	.00	0.
14 NOV 2242	108	.01	.01	.00	0.	*	15 NOV 1342	258	.00	.00	.00	0.
14 NOV 2248	109	.01	.01	.00	0.	*	15 NOV 1348	259	.00	.00	.00	0.
14 NOV 2254	110	.01	.01	.00	0.	*	15 NOV 1354	260	.00	.00	.00	0.
14 NOV 2300	111	.01	.01	.00	0.	*	15 NOV 1400	261	.00	.00	.00	0.
14 NOV 2306	112	.01	.01	.00	0.	*	15 NOV 1406	262	.00	.00	.00	0.
14 NOV 2312	113	.01	.01	.00	0.	*	15 NOV 1412	263	.00	.00	.00	0.
14 NOV 2318	114	.01	.01	.00	0.	*	15 NOV 1418	264	.00	.00	.00	0.
14 NOV 2324	115	.02	.01	.00	0.	*	15 NOV 1424	265	.00	.00	.00	0.
14 NOV 2330	116	.02	.02	.00	0.	*	15 NOV 1430	266	.00	.00	.00	0.
14 NOV 2336	117	.02	.02	.00	0.	*	15 NOV 1436	267	.00	.00	.00	0.
14 NOV 2342	118	.03	.02	.00	0.	*	15 NOV 1442	268	.00	.00	.00	0.
14 NOV 2348	119	.03	.03	.00	0.	*	15 NOV 1448	269	.00	.00	.00	0.
14 NOV 2354	120	.06	.05	.01	1.	*	15 NOV 1454	270	.00	.00	.00	0.
15 NOV 0000	121	.21	.15	.06	3.	*	15 NOV 1500	271	.00	.00	.00	0.
15 NOV 0006	122	.21	.12	.09	7.	*	15 NOV 1506	272	.00	.00	.00	0.
15 NOV 0012	123	.06	.03	.03	9.	*	15 NOV 1512	273	.00	.00	.00	0.
15 NOV 0018	124	.03	.02	.02	7.	*	15 NOV 1518	274	.00	.00	.00	0.
15 NOV 0024	125	.03	.01	.01	5.	*	15 NOV 1524	275	.00	.00	.00	0.
15 NOV 0030	126	.02	.01	.01	3.	*	15 NOV 1530	276	.00	.00	.00	0.
15 NOV 0036	127	.02	.01	.01	2.	*	15 NOV 1536	277	.00	.00	.00	0.
15 NOV 0042	128	.02	.01	.01	2.	*	15 NOV 1542	278	.00	.00	.00	0.
15 NOV 0048	129	.01	.01	.01	2.	*	15 NOV 1548	279	.00	.00	.00	0.
15 NOV 0054	130	.01	.01	.01	1.	*	15 NOV 1554	280	.00	.00	.00	0.
15 NOV 0100	131	.01	.01	.01	1.	*	15 NOV 1600	281	.00	.00	.00	0.
15 NOV 0106	132	.01	.00	.01	1.	*	15 NOV 1606	282	.00	.00	.00	0.
15 NOV 0112	133	.01	.00	.01	1.	*	15 NOV 1612	283	.00	.00	.00	0.
15 NOV 0118	134	.01	.00	.01	1.	*	15 NOV 1618	284	.00	.00	.00	0.
15 NOV 0124	135	.01	.00	.01	1.	*	15 NOV 1624	285	.00	.00	.00	0.
15 NOV 0130	136	.01	.00	.00	1.	*	15 NOV 1630	286	.00	.00	.00	0.
15 NOV 0136	137	.01	.00	.00	1.	*	15 NOV 1636	287	.00	.00	.00	0.
15 NOV 0142	138	.01	.00	.00	1.	*	15 NOV 1642	288	.00	.00	.00	0.
15 NOV 0148	139	.01	.00	.00	1.	*	15 NOV 1648	289	.00	.00	.00	0.
15 NOV 0154	140	.01	.00	.00	1.	*	15 NOV 1654	290	.00	.00	.00	0.
15 NOV 0200	141	.01	.00	.00	1.	*	15 NOV 1700	291	.00	.00	.00	0.
15 NOV 0206	142	.01	.00	.00	1.	*	15 NOV 1706	292	.00	.00	.00	0.
15 NOV 0212	143	.01	.00	.00	1.	*	15 NOV 1712	293	.00	.00	.00	0.
15 NOV 0218	144	.01	.00	.00	1.	*	15 NOV 1718	294	.00	.00	.00	0.
15 NOV 0224	145	.00	.00	.00	1.	*	15 NOV 1724	295	.00	.00	.00	0.
15 NOV 0230	146	.01	.00	.00	1.	*	15 NOV 1730	296	.00	.00	.00	0.
15 NOV 0236	147	.00	.00	.00	1.	*	15 NOV 1736	297	.00	.00	.00	0.
15 NOV 0242	148	.00	.00	.00	1.	*	15 NOV 1742	298	.00	.00	.00	0.
15 NOV 0248	149	.01	.00	.00	1.	*	15 NOV 1748	299	.00	.00	.00	0.
15 NOV 0254	150	.00	.00	.00	1.	*	15 NOV 1754	300	.00	.00	.00	0.
						*						

****

TOTAL RAINFALL = 1.56, TOTAL LOSS = 1.07, TOTAL EXCESS = .49

PEAK FLOW (CFS)	TIME (HR)		MAXIMUM AVERAGE FLOW 6-HR	24-HR	72-HR	29.90-HR
9.	12.20	(CFS)	1.	0.	0.	0.
		(INCHES)	.400	.490	.490	.490
		(AC-FT)	1.	1.	1.	1.

CUMULATIVE AREA = .03 SQ MI

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	25_year	9.	12.20	1.	0.	0.	.03		

NORMAL END OF HEC-1 ***

U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616 Pg 20 of 75

****

CN = 85
Q_{max} = 9 cfs.

THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3 17 zones

HEC-1 INPUT PAGE 1

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      LINE          ID.....1.....2.....3.....4.....5.....6.....7.....8.....9.....10

        1           ID   WHC NON-DRAGOFF
        2           ID   V-Ditch Inflow Hydrology
        3           IT       6 14NOV89    1200     300
        4           IN       6 14NOV89    1206
        5           IO       1        1
*** FREE ***

        6           KK 25_year 24-hour storm, P = 1.56"
        7           BA  0.0266
        8           KM   TYPE II DISTRIBUTION 24 HOUR DURATION
        9           PB    1.56
       10           PC   0.001   0.002   0.003   0.004   0.005   0.006   0.007   0.008   0.010   0.011
       11           PC   0.012   0.013   0.014   0.015   0.016   0.017   0.018   0.020   0.021   0.022
       12           PC   0.023   0.024   0.026   0.027   0.028   0.029   0.030   0.032   0.033   0.034
       13           PC   0.036   0.037   0.038   0.039   0.041   0.042   0.043   0.045   0.046   0.048
       14           PC   0.049   0.050   0.052   0.053   0.055   0.056   0.058   0.059   0.061   0.062
       15           PC   0.064   0.065   0.067   0.069   0.070   0.072   0.074   0.075   0.077   0.079
       16           PC   0.080   0.082   0.084   0.086   0.088   0.089   0.091   0.093   0.095   0.097
       17           PC   0.099   0.101   0.103   0.105   0.108   0.110   0.112   0.114   0.117   0.119
       18           PC   0.121   0.124   0.126   0.129   0.131   0.134   0.137   0.140   0.142   0.145
       19           PC   0.148   0.151   0.155   0.158   0.161   0.165   0.168   0.172   0.176   0.180
       20           PC   0.184   0.188   0.193   0.197   0.202   0.207   0.213   0.219   0.225   0.232
       21           PC   0.239   0.247   0.256   0.266   0.277   0.290   0.307   0.329   0.365   0.500
       22           PC   0.635   0.671   0.693   0.710   0.723   0.734   0.744   0.753   0.761   0.768
       23           PC   0.775   0.781   0.787   0.793   0.798   0.803   0.807   0.812   0.816   0.820
       24           PC   0.824   0.828   0.832   0.835   0.839   0.842   0.845   0.849   0.852   0.855
       25           PC   0.858   0.860   0.863   0.866   0.869   0.871   0.874   0.876   0.879   0.881
       26           PC   0.883   0.886   0.888   0.890   0.892   0.895   0.897   0.899   0.901   0.903
       27           PC   0.905   0.907   0.909   0.911   0.912   0.914   0.916   0.918   0.920   0.921
       28           PC   0.923   0.925   0.926   0.928   0.930   0.931   0.933   0.935   0.936   0.938
       29           PC   0.939   0.941   0.942   0.944   0.945   0.947   0.948   0.950   0.951   0.952
       30           PC   0.954   0.955   0.957   0.958   0.959   0.961   0.962   0.963   0.964   0.966
       31           PC   0.967   0.968   0.970   0.971   0.972   0.973   0.974   0.976   0.977   0.978
       32           PC   0.979   0.980   0.982   0.983   0.984   0.985   0.986   0.987   0.988   0.989
       33           PC   0.990   0.992   0.993   0.994   0.995   0.996   0.997   0.998   0.999   1.000
       34           LS       0      85
       35           UD   0.150
       36           ZZ
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FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616

****

WHC NON-DRAGOFF
V-Ditch Inflow Hydrology

5 ID OUTPUT CONTROL VARIABLES

IPRNT	1	PRINT CONTROL
IPLOT	1	PLOT CONTROL
QSCAL	0.	HYDROGRAPH PLOT SCALE

IT HYDROGRAPH TIME DATA

NMIN	6	MINUTES IN COMPUTATION INTERVAL
IDATE	14NOV89	STARTING DATE
ITIME	1200	STARTING TIME
NQ	300	NUMBER OF HYDROGRAPH ORDINATES
NDDATE	15NOV89	ENDING DATE
NDTIME	1754	ENDING TIME

COMPUTATION INTERVAL .10 HOURS
TOTAL TIME BASE 29.90 HOURS

ENGLISH UNITS

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          **************
          *            *
6 KK      *  25_year   *      24-hour storm, P = 1.56"
          *            *
          **************

                    TYPE II DISTRIBUTION 24 HOUR DURATION

- IN          TIME DATA FOR INPUT TIME SERIES
                   JXMIN          6  TIME INTERVAL IN MINUTES
                  JXDATE   14NOV89  STARTING DATE
                  JXTIME       1206  STARTING TIME

             SUBBASIN RUNOFF DATA

7 BA         SUBBASIN CHARACTERISTICS
                   TAREA        .03  SUBBASIN AREA

             PRECIPITATION DATA

9 PB               STORM       1.56  BASIN TOTAL PRECIPITATION

0 PI          INCREMENTAL PRECIPITATION PATTERN
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .01   .00   .01   .00   .01   .01   .01   .01
                .01   .01   .01   .01   .01   .01   .02   .02   .04   .13
                .13   .04   .02   .02   .01   .01   .01   .01   .01   .01
                .01   .01   .01   .01   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00
                .00   .00   .00   .00   .00   .00   .00   .00   .00   .00

X LS         SCS LOSS RATE
                   STRTL        .35  INITIAL ABSTRACTION
                  CRVNBR       85.00  CURVE NUMBER
                  RTIMP         .00  PERCENT IMPERVIOUS AREA

5 UD         SCS DIMENSIONLESS UNITGRAPH
                    TLAG        .15  LAG

                                  ***

.NING *** TIME INTERVAL IS GREATER THAN .29*LAG

                                UNIT HYDROGRAPH
                            10 END-OF-PERIOD ORDINATES
              30.     65.     44.     18.      8.      4.      2.      1.      0.      0.

*****************************************************************************************************

                            HYDROGRAPH AT STATION  25_year

*****************************************************************************************************

   DA MON HRMN  ORD   RAIN   LOSS  EXCESS   COMP Q      DA MON HRMN  ORD   RAIN   LOSS  EXCESS   COMP Q
   14 NOV 1200    1    .00    .00    .00       0.      15 NOV 0300  151    .00    .00    .00       1.
   14 NOV 1206    2    .00    .00    .00       0.      15 NOV 0306  152    .00    .00    .00       0.
   14 NOV 1212    3    .00    .00    .00       0.      15 NOV 0312  153    .00    .00    .00       0.
   14 NOV 1218    4    .00    .00    .00       0.      15 NOV 0318  154    .00    .00    .00       0.
   14 NOV 1224    5    .00    .00    .00       0.      15 NOV 0324  155    .00    .00    .00       0.
   14 NOV 1230    6    .00    .00    .00       0.      15 NOV 0330  156    .00    .00    .00       0.
   14 NOV 1236    7    .00    .00    .00       0.      15 NOV 0336  157    .00    .00    .00       0.
   14 NOV 1242    8    .00    .00    .00       0.      15 NOV 0342  158    .00    .00    .00       0.
   14 NOV 1248    9    .00    .00    .00       0.      15 NOV 0348  159    .00    .00    .00       0.
   14 NOV 1254   10    .00    .00    .00       0.      15 NOV 0354  160    .00    .00    .00       0.
   14 NOV 1300   11    .00    .00    .00       0.      15 NOV 0400  161    .00    .00    .00       0.
   14 NOV 1306   12    .00    .00    .00       0.      15 NOV 0406  162    .00    .00    .00       0.
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14	NOV	1336	17	.00	.00	.00	0.	*	15	NOV	0436	167	.00	.00	.00	0.	
	V	1342	18	.00	.00	.00	0.	*	15	NOV	0442	168	.00	.00	.00	0.	
	/	1348	19	.00	.00	.00	0.	*	15	NOV	0448	169	.00	.00	.00	0.	
14	NOV	1354	20	.00	.00	.00	0.	*	15	NOV	0454	170	.00	.00	.00	0.	
14	NOV	1400	21	.00	.00	.00	0.	*	15	NOV	0500	171	.00	.00	.00	0.	
14	NOV	1406	22	.00	.00	.00	0.	*	15	NOV	0506	172	.00	.00	.00	0.	
14	NOV	1412	23	.00	.00	.00	0.	*	15	NOV	0512	173	.00	.00	.00	0.	
14	NOV	1418	24	.00	.00	.00	0.	*	15	NOV	0518	174	.00	.00	.00	0.	
14	NOV	1424	25	.00	.00	.00	0.	*	15	NOV	0524	175	.00	.00	.00	0.	
14	NOV	1430	26	.00	.00	.00	0.	*	15	NOV	0530	176	.00	.00	.00	0.	
14	NOV	1436	27	.00	.00	.00	0.	*	15	NOV	0536	177	.00	.00	.00	0.	
14	NOV	1442	28	.00	.00	.00	0.	*	15	NOV	0542	178	.00	.00	.00	0.	
14	NOV	1448	29	.00	.00	.00	0.	*	15	NOV	0548	179	.00	.00	.00	0.	
14	NOV	1454	30	.00	.00	.00	0.	*	15	NOV	0554	180	.00	.00	.00	0.	
14	NOV	1500	31	.00	.00	.00	0.	*	15	NOV	0600	181	.00	.00	.00	0.	
14	NOV	1506	32	.00	.00	.00	0.	*	15	NOV	0606	182	.00	.00	.00	0.	
14	NOV	1512	33	.00	.00	.00	0.	*	15	NOV	0612	183	.00	.00	.00	0.	
14	NOV	1518	34	.00	.00	.00	0.	*	15	NOV	0618	184	.00	.00	.00	0.	
14	NOV	1524	35	.00	.00	.00	0.	*	15	NOV	0624	185	.00	.00	.00	0.	
14	NOV	1530	36	.00	.00	.00	0.	*	15	NOV	0630	186	.00	.00	.00	0.	
14	NOV	1536	37	.00	.00	.00	0.	*	15	NOV	0636	187	.00	.00	.00	0.	
14	NOV	1542	38	.00	.00	.00	0.	*	15	NOV	0642	188	.00	.00	.00	0.	
14	NOV	1548	39	.00	.00	.00	0.	*	15	NOV	0648	189	.00	.00	.00	0.	
14	NOV	1554	40	.00	.00	.00	0.	*	15	NOV	0654	190	.00	.00	.00	0.	
14	NOV	1600	41	.00	.00	.00	0.	*	15	NOV	0700	191	.00	.00	.00	0.	
14	NOV	1606	42	.00	.00	.00	0.	*	15	NOV	0706	192	.00	.00	.00	0.	
14	NOV	1612	43	.00	.00	.00	0.	*	15	NOV	0712	193	.00	.00	.00	0.	
14	NOV	1618	44	.00	.00	.00	0.	*	15	NOV	0718	194	.00	.00	.00	0.	
14	NOV	1624	45	.00	.00	.00	0.	*	15	NOV	0724	195	.00	.00	.00	0.	
14	NOV	1630	46	.00	.00	.00	0.	*	15	NOV	0730	196	.00	.00	.00	0.	
14	NOV	1636	47	.00	.00	.00	0.	*	15	NOV	0736	197	.00	.00	.00	0.	
14	NOV	1642	48	.00	.00	.00	0.	*	15	NOV	0742	198	.00	.00	.00	0.	
14	NOV	1648	49	.00	.00	.00	0.	*	15	NOV	0748	199	.00	.00	.00	0.	
14	NOV	1654	50	.00	.00	.00	0.	*	15	NOV	0754	200	.00	.00	.00	0.	
14	NOV	1700	51	.00	.00	.00	0.	*	15	NOV	0800	201	.00	.00	.00	0.	
14	NOV	1706	52	.00	.00	.00	0.	*	15	NOV	0806	202	.00	.00	.00	0.	
14	NOV	1712	53	.00	.00	.00	0.	*	15	NOV	0812	203	.00	.00	.00	0.	
14	NOV	1718	54	.00	.00	.00	0.	*	15	NOV	0818	204	.00	.00	.00	0.	
14	NOV	1724	55	.00	.00	.00	0.	*	15	NOV	0824	205	.00	.00	.00	0.	
14	NOV	1730	56	.00	.00	.00	0.	*	15	NOV	0830	206	.00	.00	.00	0.	
		1736	57	.00	.00	.00	0.	*	15	NOV	0836	207	.00	.00	.00	0.	
	/	1742	58	.00	.00	.00	0.	*	15	NOV	0842	208	.00	.00	.00	0.	
14	NOV	1748	59	.00	.00	.00	0.	*	15	NOV	0848	209	.00	.00	.00	0.	
14	NOV	1754	60	.00	.00	.00	0.	*	15	NOV	0854	210	.00	.00	.00	0.	
14	NOV	1800	61	.00	.00	.00	0.	*	15	NOV	0900	211	.00	.00	.00	0.	
14	NOV	1806	62	.00	.00	.00	0.	*	15	NOV	0906	212	.00	.00	.00	0.	
14	NOV	1812	63	.00	.00	.00	0.	*	15	NOV	0912	213	.00	.00	.00	0.	
14	NOV	1818	64	.00	.00	.00	0.	*	15	NOV	0918	214	.00	.00	.00	0.	
14	NOV	1824	65	.00	.00	.00	0.	*	15	NOV	0924	215	.00	.00	.00	0.	
14	NOV	1830	66	.00	.00	.00	0.	*	15	NOV	0930	216	.00	.00	.00	0.	
14	NOV	1836	67	.00	.00	.00	0.	*	15	NOV	0936	217	.00	.00	.00	0.	
14	NOV	1842	68	.00	.00	.00	0.	*	15	NOV	0942	218	.00	.00	.00	0.	
14	NOV	1848	69	.00	.00	.00	0.	*	15	NOV	0948	219	.00	.00	.00	0.	
14	NOV	1854	70	.00	.00	.00	0.	*	15	NOV	0954	220	.00	.00	.00	0.	
14	NOV	1900	71	.00	.00	.00	0.	*	15	NOV	1000	221	.00	.00	.00	0.	
14	NOV	1906	72	.00	.00	.00	0.	*	15	NOV	1006	222	.00	.00	.00	0.	
14	NOV	1912	73	.00	.00	.00	0.	*	15	NOV	1012	223	.00	.00	.00	0.	
14	NOV	1918	74	.00	.00	.00	0.	*	15	NOV	1018	224	.00	.00	.00	0.	
14	NOV	1924	75	.00	.00	.00	0.	*	15	NOV	1024	225	.00	.00	.00	0.	
14	NOV	1930	76	.00	.00	.00	0.	*	15	NOV	1030	226	.00	.00	.00	0.	
14	NOV	1936	77	.00	.00	.00	0.	*	15	NOV	1036	227	.00	.00	.00	0.	
14	NOV	1942	78	.00	.00	.00	0.	*	15	NOV	1042	228	.00	.00	.00	0.	
14	NOV	1948	79	.00	.00	.00	0.	*	15	NOV	1048	229	.00	.00	.00	0.	
14	NOV	1954	80	.00	.00	.00	0.	*	15	NOV	1054	230	.00	.00	.00	0.	
14	NOV	2000	81	.00	.00	.00	0.	*	15	NOV	1100	231	.00	.00	.00	0.	
14	NOV	2006	82	.00	.00	.00	0.	*	15	NOV	1106	232	.00	.00	.00	0.	
14	NOV	2012	83	.00	.00	.00	0.	*	15	NOV	1112	233	.00	.00	.00	0.	
14	NOV	2018	84	.00	.00	.00	0.	*	15	NOV	1118	234	.00	.00	.00	0.	
14	NOV	2024	85	.00	.00	.00	0.	*	15	NOV	1124	235	.00	.00	.00	0.	
14	NOV	2030	86	.00	.00	.00	0.	*	15	NOV	1130	236	.00	.00	.00	0.	
14	NOV	2036	87	.00	.00	.00	0.	*	15	NOV	1136	237	.00	.00	.00	0.	
14	NOV	2042	88	.00	.00	.00	0.	*	15	NOV	1142	238	.00	.00	.00	0.	
14	NOV	2048	89	.00	.00	.00	0.	*	15	NOV	1148	239	.00	.00	.00	0.	
14	NOV	2054	90	.00	.00	.00	0.	*	15	NOV	1154	240	.00	.00	.00	0.	
14	NOV	2100	91	.00	.00	.00	0.	*	15	NOV	1200	241	.00	.00	.00	0.	
14	NOV	2106	92	.00	.00	.00	0.	*	15	NOV	1206	242	.00	.00	.00	0.	
14	NOV	2112	93	.00	.00	.00	0.	*	15	NOV	1212	243	.00	.00	.00	0.	
14	NOV	2118	94	.01	.01	.00	0.	*	15	NOV	1218	244	.00	.00	.00	0.	
		2124	95	.00	.00	.00	0.	*	15	NOV	1224	245	.00	.00	.00	0.	
		2130	96	.00	.00	.00	0.	*	15	NOV	1230	246	.00	.00	.00	0.	
14	NOV	2136	97	.01	.01	.00	0.	*	15	NOV	1236	247	.00	.00	.00	0.	
14	NOV	2142	98	.00	.00	.00	0.	*	15	NOV	1242	248	.00	.00	.00	0.	
14	NOV	2148	99	.01	.01	.00	0.	*	15	NOV	1248	249	.00	.00	.00	0.	
14	NOV	2154	100	.01	.01	.00	0.	*	15	NOV	1254	250	.00	.00	.00	0.	

Golder Associates

SUBJECT NON-DRAG-OFF / SNOW MELT

Job No. 89-1186 Ref. 843-486.032 | Made by M. BROWN Checked JSV Reviewed | Date JAN 8 90 Sheet 23 of 75

ESTIMATE PEAK SNOW MELT

Assume, very conservatively, that 25 year snow on ground is subjected to 25 year precip.

25 yr. Snow = 12" ✓ see attached graph, ref. 2

Assume WATER CONTENT = 10% ⇒ 1.2" WATER

Assume 32°F, frozen.

During peak 20 minute precip rate = 1.5 in/hr ⇒ 0.5 in total

Assume temp = 50°F.

Heat for thawing = 90 BTU/lb.
Heat available = (50-32) or 18 BTU/lb

∴ Precip will melt $\frac{18}{90}$ of its depth = $0.5" \times \frac{18}{90} = 0.10"$

Therefore this very very conservative assumption yields an increase in precip. of 20%.

Given level of uncertainty in estimate, ignore snowmelt.

This is not an analysis of the potential runoff of rain on frozen ground. The probability of that event is addressed by the regression eqns since they are based on stream flow. ✓

Pg 24 of 75

12" snow

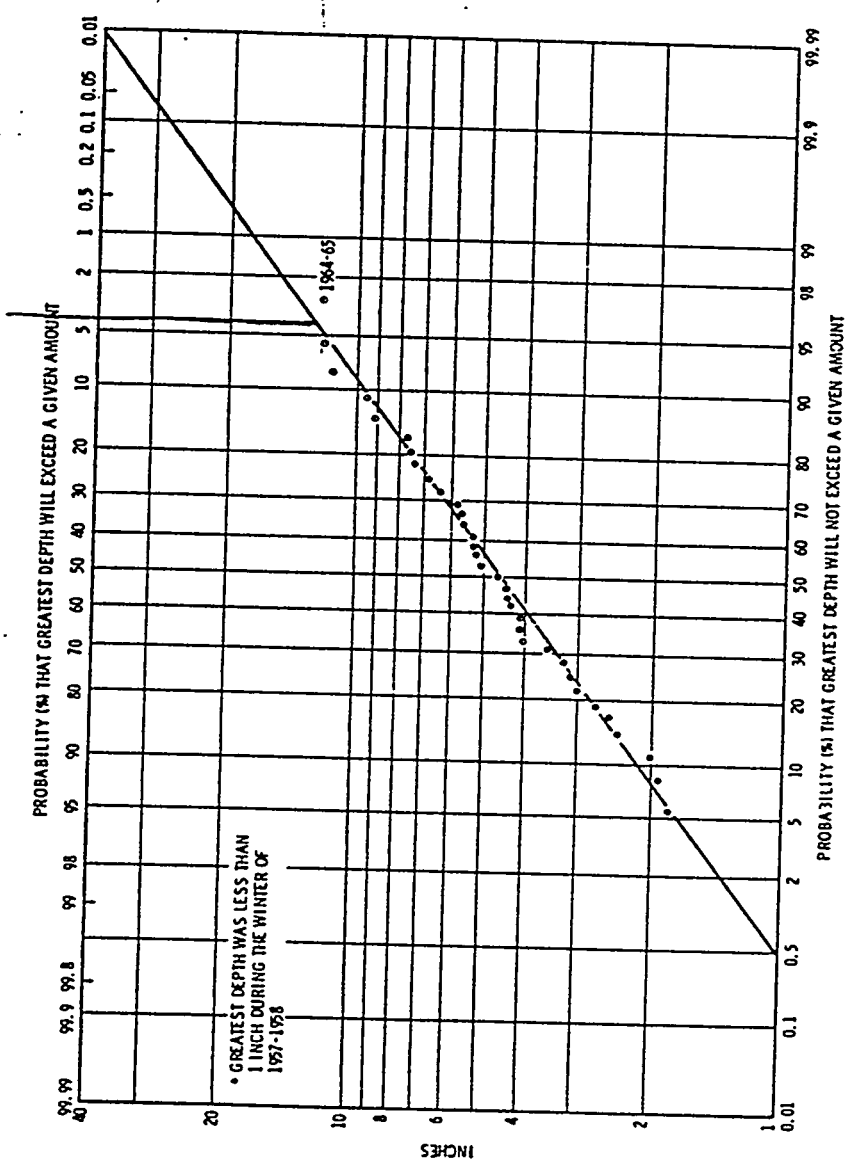


FIGURE 18. Greatest Depth of Snow on Ground During Winters of Record at Hanford 1946-47 Through 1980-81*

25 year ⇒ 4% prob.

APPENDIX/CALCULATION B.11 HELP RUNS

Golder Associates

SUBJECT NON-DRAG-OFF LEACHATE PRODUCTION

Job No. 893-1186.032 Ref.

Made by M. BROWN Checked JSV Reviewed

Date JAN 11 1990 Sheet 25 of 75

OBJECTIVE

Estimate quantity of leachate which will reach the sump on an average annual basis and a peak Daily basis. Compare results to previous work by EDASCO.

METHODOLOGY

USE HELP MODEL, version 2

Hanford Town site Precip

Yakima Solar Radiation & Temperature data.

— Use 10 years of data for average annual estimate

— Use wet period for peak daily estimate.

See attached HELP OUTPUT —

AVERAGE ANNUAL LEACHATE PRODUCTION

from Run HIST10-1 20671 ft³ = 154620 gal.

max year = 1983 39592 ft³ = 296148 gal.

Peak day for this run = 2013 ft³ = 15000 gallons

↳ 15057 gal

PEAK DAY - FROM RUN PEAK5-2

Peak daily 14531 ft³ = 108692 gal.

I. Comparison of Golder and Ebasco "HELP" Modelling

During the Preliminary Design Review meeting, Golder Associates Inc. was supplied with a copy of a study by Ebasco Services (Ref 4) which addresses the problem of treatment of leachate produced at the Hanford Site. As part of their study Ebasco used the HELP model to estimate annual and peak daily leachate production. Because Ebasco's results were substantially different from Golder's preliminary estimates, a comparative study of the results of the two estimates was initiated.

The "HELP" model [Hydrologic Evaluation of Landfill Performance], simulates the one dimensional movement of precipitation through a series of soil layers; the time step is one day. The precipitation data used in the model was actual Hanford precipitation. The model uses temperature, solar radiation to estimate loses to the atmosphere; for this data Yakima was used as the source. Estimates of average annual leachate production were made by running the model with the local climatological data together with input data which simulated the open landfill.

To simulate a 25 year flood event, both Golder and Ebasco introduced the 25yr/24hr precipitation event (1.56") into the precipitation record but at different locations. The HELP model was then run again with input data which simulated the open landfill. The wettest day during the simulation period was October 28, 1982; however, the wettest period was in December of 1983. Ebasco substituted the 25-year 24-hour precipitation for the largest daily the precipitation in the record which was .92 inches on Oct 28 1982. Golder inserted the 25-year precipitation near the end of December 1983, following a period of rain.

The results obtained by Ebasco and Golder were substantially different. To resolve these differences the modelling assumptions and the data used by each were examined to the extent possible. Further, attempts were made to duplicate Ebasco's results. This was followed by additional model runs with variations in one parameter at a time. The results of this comparison is summarized on Table III. The significant findings of our studies are summarized below:

1. The versions of HELP used are different; version 2 has significant differences in the way loses are computed. These differences have little significance to the peak daily production (6,065 gal to 5,702 gal) but a major impact on the average annual values (88,960 gal to 175,256 gal). If the leaf area index were the same (zero) the difference would be even greater.

2. There are apparently some small differences in the official Hanford precipitation record; Golder's average annual precipitation data is 0.4 inches higher than Ebasco's for the same period.

3. Ebasco used a drainage layer consisting of 12 inches with a permeability of 10^{-2} cm/sec, whereas Golder, assuming the use of drainage gravel and geonet, used a 12 inch layer with a permeability of 1 cm/sec. This difference apparently has little impact on the results.

4. Changing the date of the 25-year precipitation event to late December 1983 significantly increase the peak daily leachate production (5,702 gal to 44,780 gal).

5. Similarly the changing the thickness of the operations layer, the leaf area index and the area of the landfill all cause increases in peak and average annual leachate production.

In summary all the major differences between Ebasco's analysis and Golder's analysis can be explained. For the purposes of this design the estimates of leachate production made by Golder were used.

COMPARISON OF ANALYSES USING "HELP"

DATA INPUT & OUTPUT	RUN COMPUTED BY ==> EBASCO RUN NAME ==> NORMAL PRECIP	STORM PRECIP	GOLDER EBASCO-	EBASCO-	EBASCO-	EBASCO-	EBASCO-
INPUT DATA							
HELP MODEL USED	Ver 1	Ver 1	Ver 2	Ver 2	Ver 2	Ver 2	Ver 2
SURFACE LAYER							
Thickness (inches)	36	36	36	36	36	24	36
Porosity	0.389	0.389	0.389	0.389	0.389	0.389	0.389
Field Capacity	0.199	0.199	0.199	0.199	0.199	0.199	0.199
Wilting Point	0.066	0.066	0.066	0.066	0.066	0.066	0.066
Eff. Hydr.Condtvty (cm/sec)	0.010	0.010	0.010	0.010	0.010	0.010	0.010
Area (sq ft)	134,925	134,925	134,925	134,925	134,925	134,925	134,925
25-YEAR STORM EVENT							
Depth (inches)	0	1.5	0	1.56	1.56	1.56	1.56
Date	---	10/28/82	---	10/28/82	12/28/83	12/28/83	12/28/83
Leaf Area Index	0	0	0.25	0.25	0.25	0.25	0
MODEL RESULTS							
Ave Annual Precip. (in)	8.23	8.34	8.63	8.76	8.94	8.94	8.94
Ave. Annual ET (in)	7.019	7.129	6.67	6.67	6.67	6.67	6.37
Average Annual Leachate (gal)	88,960	89,483	175,256	185,840	201,520	199,760	219,530
Peak Daily Leachate (gal)	6,065	6,065	5,702	5,702	44,780	85,085	99,090

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WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
10 YEAR SIMULATION - HISTORICAL DATA
24" Ops Layer; Leaf INDEX = 0; ML BROWN - 1-11-90; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL

INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	80.00
TOTAL AREA OF COVER	=	148000. SQ FT
EVAPORATIVE ZONE DEPTH	=	18.00 INCHES
POTENTIAL RUNOFF FRACTION	=	0.000000
UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.3848 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	21.1200 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	124
END OF GROWING SEASON (JULIAN DATE)	=	276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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MONTHLY TOTALS FOR YEAR 79

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.54	0.17	0.54	0.52	0.10	0.00
	0.09	0.38	0.20	0.67	1.43	0.99
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

EVAPOTRANSPIRATION (INCHES)	0.940 0.090	0.565 0.160	0.194 0.215	0.386 0.217	0.323 0.590	0.256 0.451
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.3620 0.0098	0.1417 0.0081	0.0436 0.0067	0.0233 0.0060	0.0164 0.1069	0.0120 0.5023
PERCOLATION FROM LAYER 3 (INCHES)	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.00

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ANNUAL TOTALS FOR YEAR 79

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	5.63	69437.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	4.387	54110.	77.93
LATERAL DRAINAGE FROM LAYER 2	1.2388	15279.	22.00
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.004	48.	0.07
SOIL WATER AT START OF YEAR	19.94	245918.	
SOIL WATER AT END OF YEAR	19.94	245966.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 80

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	1.32	1.30	0.30	0.86	1.43	0.96
	0.00	0.02	0.85	0.33	0.44	1.89
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.556	1.296	1.067	0.422	1.653	0.629
	0.458	0.085	0.220	0.282	0.250	0.265
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.7342	0.4646	0.1153	0.0372	0.0226	0.0213
	0.0154	0.0117	0.0091	0.0101	0.0245	0.7009
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

[illegible]. DAILY HEAD ON AYER 3 (INCHES)	0.01	0.01	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01

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ANNUAL TOTALS FOR YEAR 80

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	9.70	119633.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	7.185	88615.	74.07
LATERAL DRAINAGE FROM LAYER 2	2.1670	26726.	22.34
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.348	4292..	3.59
SOIL WATER AT START OF YEAR	19.94	245966.	

SOIL WATER AT END OF YEAR	20.29	250258.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 81

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.56	0.60	0.70	0.02	0.99	0.43
	0.19	0.03	0.60	0.39	1.08	1.45
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.839	1.175	0.315	0.405	0.205	0.579
	0.269	0.247	0.126	0.217	0.577	0.475
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6925	0.1097	0.0409	0.0224	0.0160	0.0199
	0.0870	0.0738	0.0388	0.0230	0.3858	0.3442
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.00
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.00

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ANNUAL TOTALS FOR YEAR 81

	(INCHES)	(CU. FT.)	PERCENT

PRECIPITATION	7.04	86827.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	5.427	66939.	77.09
LATERAL DRAINAGE FROM LAYER 2	1.8539	22865.	26.33
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.241	-2977.	-3.43
SOIL WATER AT START OF YEAR	20.29	250258.	
SOIL WATER AT END OF YEAR	20.02	246871.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.03	410.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 82

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.38 0.22	0.57 0.20	0.30 0.55	0.75 1.37	0.28 0.91	0.75 1.79
RUNOFF (INCHES)	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000	0.000 0.000
EVAPOTRANSPIRATION (INCHES)	0.775 0.572	0.872 0.250	0.234 0.242	0.440 0.436	0.354 1.302	0.305 0.574
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.5530 0.0101	0.1165 0.0165	0.0408 0.0267	0.0224 0.0907	0.0160 0.4631	0.0117 0.7036
PERCOLATION FROM LAYER 3 (INCHES)	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000	0.0000 0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01 0.00	0.00 0.00	0.00 0.00	0.00 0.00	0.00 0.01	0.00 0.01

STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.01	0.02

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ANNUAL TOTALS FOR YEAR 82

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	8.07	99530.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	6.355	78374.	78.74
LATERAL DRAINAGE FROM LAYER 2	2.0711	25543.	25.66
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.356	-4388.	-4.41
SOIL WATER AT START OF YEAR	20.02	246871.	
SOIL WATER AT END OF YEAR	19.69	242893.	
SNOW WATER AT START OF YEAR	0.03	410.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 83

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	1.44	1.36	1.00	0.42	0.52	0.68
	0.31	0.12	0.46	0.52	2.12	2.12
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.715	1.290	1.648	0.453	0.374	0.343
	0.304	0.277	0.234	0.239	0.889	0.554

LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6846	0.3034	0.1557	0.0463	0.0254	0.0165
	0.0127	0.0100	0.0080	0.0071	0.7179	1.2227
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.02
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.02

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ANNUAL TOTALS FOR YEAR 83

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	11.07	136530.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	7.320	90277.	66.12
LATERAL DRAINAGE FROM LAYER 2	3.2102	39592.	29.00
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.540	6661.	4.88
SOIL WATER AT START OF YEAR	19.69	242893.	
SOIL WATER AT END OF YEAR	20.23	249554.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

MAX ANNUAL.

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MONTHLY TOTALS FOR YEAR 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.23	0.94	1.01	0.60	0.55	0.99
	0.06	0.00	0.42	0.07	1.83	0.57
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.602	1.351	1.022	0.497	0.441	0.564
	0.296	0.258	0.252	0.228	0.400	0.665
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6849	0.1184	0.0414	0.0226	0.0161	0.0118
	0.0097	0.0081	0.0070	0.0064	0.0142	0.5402
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 84

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	7.27	89663.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	6.576	81101.	90.45
LATERAL DRAINAGE FROM LAYER 2	1.4808	18263.	20.37
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.787	-9701.	-10.82
SOIL WATER AT START OF YEAR	20.23	249554.	
SOIL WATER AT END OF YEAR	19.45	239853.	
SNOW WATER AT START OF YEAR	0.00	0.	

SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 85

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.34	0.82	0.36	0.01	0.12	0.15
	0.12	0.01	0.63	0.46	1.24	0.86
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.828	0.958	0.331	0.038	0.120	0.150
	0.011	0.119	0.208	0.213	0.221	0.697
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.1440	0.0589	0.0330	0.0196	0.0145	0.0108
	0.0091	0.0076	0.0063	0.0057	0.1731	0.9585
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.01

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ANNUAL TOTALS FOR YEAR 85

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	5.12	63147.	100.00
RUNOFF	0.000	0.	0.00

EVAPOTRANSPIRATION	3.893	48017.	76.04
LATERAL DRAINAGE FROM LAYER 2	1.4410	17773.	28.15
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.214	-2643.	-4.18
SOIL WATER AT START OF YEAR	19.45	239853.	
SOIL WATER AT END OF YEAR	19.23	237210.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 86

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	1.76	1.21	0.76	0.00	0.30	0.00
	0.21	0.02	0.96	0.29	0.65	0.77
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.743	1.695	0.854	0.071	0.300	0.000
	0.210	0.020	0.152	0.215	0.201	0.469
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.2029	0.6176	0.1470	0.0384	0.0228	0.0152
	0.0120	0.0095	0.0077	0.0072	0.0196	0.4476
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.01	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 86

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	6.93	85470.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	4.929	60792.	71.13
LATERAL DRAINAGE FROM LAYER 2	1.5474	19085.	22.33
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.453	5593.	6.54
SOIL WATER AT START OF YEAR	19.23	237210.	
SOIL WATER AT END OF YEAR	19.69	242803.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 87

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.80	0.55	1.05	0.14	0.39	0.08
	0.50	0.07	0.01	0.00	0.40	1.63
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.797	0.952	1.309	0.136	0.244	0.229
	0.275	0.264	0.041	0.000	0.208	0.312
ERAL DRAINAGE FROM LAYER 2 (INCHES)	0.2573	0.1239	0.0474	0.0244	0.0170	0.0123
	0.0101	0.0083	0.0068	0.0061	0.0052	0.2406
PERCOLATION FROM	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON	0.00	0.00	0.00	0.00	0.00	0.00
LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
STD. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR 87

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	5.62	69313.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	4.767	58796.	84.83
LATERAL DRAINAGE FROM LAYER 2	0.7592	9364.	13.51
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.094	1154.	1.66
SOIL WATER AT START OF YEAR	19.69	242803.	
SOIL WATER AT END OF YEAR	19.77	243852.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.01	105.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 88

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC

PRECIPITATION (INCHES)	0.48	0.00	0.60	1.12	0.33	0.11
	0.13	0.00	0.39	0.01	0.82	0.40
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.713	0.351	0.375	0.339	0.589	0.256
	0.235	0.164	0.094	0.191	0.218	0.269
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.3943	0.1568	0.0457	0.0239	0.0540	0.1235
	0.0821	0.0461	0.0248	0.0172	0.0124	0.0103
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00

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ANNUAL TOTALS FOR YEAR (88)

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	4.39	54143.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	3.795	46807.	86.45
LATERAL DRAINAGE FROM LAYER 2	0.9910	12222.	22.57
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.396	-4886.	-9.02
SOIL WATER AT START OF YEAR	19.77	243852.	
SOIL WATER AT END OF YEAR	19.38	239071.	
SNOW WATER AT START OF YEAR	0.01	105.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 79 THROUGH 88

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.78	0.75	0.66	0.44	0.50	0.42
	0.18	0.09	0.51	0.41	1.09	1.25
STD. DEVIATIONS	0.53	0.46	0.29	0.40	0.41	0.40
	0.14	0.12	0.28	0.40	0.57	0.60
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.751	1.051	0.735	0.319	0.460	0.331
	0.272	0.184	0.178	0.224	0.486	0.473
STD. DEVIATIONS	0.113	0.396	0.515	0.170	0.439	0.202
	0.161	0.089	0.071	0.105	0.367	0.155
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.4710	0.2212	0.0711	0.0281	0.0221	0.0255
	0.0258	0.0200	0.0142	0.0179	0.1923	0.5671
STD. DEVIATIONS	0.2258	0.1836	0.0483	0.0091	0.0118	0.0346
	0.0310	0.0222	0.0116	0.0262	0.2476	0.3504
PERCOLATION FROM LAYER 3						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 79 THROUGH 88

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	7.08	(2.085)	87369.	100.00
RUNOFF	0.000	(0.000)	0.	0.00
EVAPOTRANSPIRATION	5.463	(1.317)	67383.	77.12
LATERAL DRAINAGE FROM LAYER 2	1.6760	(0.6987)	20671.	23.66
PERCOLATION FROM LAYER 3	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.056	(0.422)	-685.	-0.78

20671. => 154 620 gallons

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PEAK DAILY VALUES FOR YEARS 79 THROUGH 88

	(INCHES)	(CU. FT.)
PRECIPITATION	0.93	11470.0
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	0.1633	2013.8
PERCOLATION FROM LAYER 3	0.0000	0.0
HEAD ON LAYER 3	0.1	
SNOW WATER	0.76	9431.8

2013.8 = 15 063 gallons

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1800
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0630

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FINAL WATER STORAGE AT END OF YEAR 88

LAYER	(INCHES)	(VOL/VOL)
1	2.58	0.1077
2	0.60	0.0500

3	16.20	0.4500
SNOW WATER	0.00	

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FILE: PEAK5-2

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WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
5 YEAR SIMULATION - HISTORICAL DATA + 25 year Event on 12/28/83
24" Ops Layer; Leaf INDEX = 0; ML BROWN - 1-11-90; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL

INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	80.00
TOTAL AREA OF COVER	=	148000. SQ FT
EVAPORATIVE ZONE DEPTH	=	18.00 INCHES
POTENTIAL RUNOFF FRACTION	=	0.000000
UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.3801 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	21.1200 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	124
END OF GROWING SEASON (JULIAN DATE)	=	276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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MONTHLY TOTALS FOR YEAR 80

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	1.32	1.30	0.30	0.86	1.43	0.96
	0.00	0.02	0.85	0.33	0.44	1.89
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

EVAPOTRANSPIRATION (INCHES)	0.860	1.380	0.667	0.410	1.637	0.629
	0.458	0.091	0.219	0.279	0.247	0.517
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	1.1470	0.2496	0.0509	0.0254	0.0181	0.0256
	0.0176	0.0130	0.0099	0.0109	0.0261	0.7119
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.02	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01

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ANNUAL TOTALS FOR YEAR 80

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	9.70	119633.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	7.393	91186.	76.22
LATERAL DRAINAGE FROM LAYER 2	2.3060	28440.	23.77
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.001	7.	0.01
SOIL WATER AT START OF YEAR	20.04	247128.	
SOIL WATER AT END OF YEAR	20.04	247135.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 81

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.56	0.60	0.70	0.02	0.99	0.43
	0.19	0.03	0.60	0.39	1.08	1.45
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.527	1.398	0.325	0.394	0.210	0.671
	0.273	0.250	0.106	0.219	0.648	0.398
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.5017	0.1228	0.0473	0.0244	0.0170	0.0155
	0.0455	0.0525	0.0317	0.0202	0.3824	0.2646
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.00
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.00

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ANNUAL TOTALS FOR YEAR 81

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	7.04	86827.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	5.420	66847.	76.99
LATERAL DRAINAGE FROM LAYER 2	1.5257	18817.	21.67
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.094	1163.	1.34
SOIL WATER AT START OF YEAR	20.04	247135.	

SOIL WATER AT END OF YEAR	20.06	247413.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.07	885.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 82

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.38	0.57	0.30	0.75	0.28	0.75
	0.22	0.20	0.55	1.37	0.91	1.79
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.916	0.720	0.242	0.559	0.331	0.753
	0.482	0.182	0.263	0.323	0.513	0.405
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6428	0.1052	0.0390	0.0218	0.0157	0.0115
	0.0096	0.0079	0.0065	0.0062	0.4399	1.4204
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.02
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.02

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ANNUAL TOTALS FOR YEAR 82

	(INCHES)	(CU. FT.)	PERCENT

PRECIPITATION	8.07	99530.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	5.688	70151.	70.48
LATERAL DRAINAGE FROM LAYER 2	2.7265	33627.	33.79
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.344	-4247.	-4.27
SOIL WATER AT START OF YEAR	20.06	247413.	
SOIL WATER AT END OF YEAR	19.79	244050.	
SNOW WATER AT START OF YEAR	0.07	885.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 83

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	1.44	1.36	1.00	0.42	0.52	0.68
	0.31	0.12	0.46	0.52	2.12	3.68
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION (INCHES)	0.704	1.260	1.650	0.450	0.376	0.349
	0.307	0.279	0.236	0.255	0.838	0.428
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.7502	0.3449	0.1805	0.0508	0.0268	0.0171
	0.0131	0.0103	0.0082	0.0072	0.6822	2.5577
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.01	0.01	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.01	0.07

. DEV. OF DAILY HEAD	0.00	0.00	0.00	0.00	0.00	0.00
N LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.01	0.22

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ANNUAL TOTALS FOR YEAR 83

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	12.63	155770.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	7.131	87954.	56.46
LATERAL DRAINAGE FROM LAYER 2	4.6491	57339.	36.81
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	0.849	10477.	6.73
SOIL WATER AT START OF YEAR	19.79	244050.	
SOIL WATER AT END OF YEAR	20.64	254527.	
NOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

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MONTHLY TOTALS FOR YEAR 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.23	0.94	1.01	0.60	0.55	0.99
	0.06	0.00	0.42	0.07	1.83	0.57
RUNOFF (INCHES)	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
E OTRANSPIRATION (INCHES)	0.788	1.186	0.974	0.547	0.444	0.869
	0.389	0.000	0.171	0.312	0.314	0.244

LATERAL DRAINAGE FROM LAYER 2 (INCHES)	1.0859	0.1051	0.0379	0.0214	0.0155	0.0114
	0.0095	0.0079	0.0065	0.0058	0.0073	0.5297
PERCOLATION FROM LAYER 3 (INCHES)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

MONTHLY SUMMARIES FOR DAILY HEADS

AVG. DAILY HEAD ON LAYER 3 (INCHES)	0.02	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD ON LAYER 3 (INCHES)	0.03	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00

ANNUAL TOTALS FOR YEAR 84

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	7.27	89663.	100.00
RUNOFF	0.000	0.	0.00
EVAPOTRANSPIRATION	6.238	76933.	85.80
LATERAL DRAINAGE FROM LAYER 2	1.8439	22741.	25.36
PERCOLATION FROM LAYER 3	0.0000	0.	0.00
CHANGE IN WATER STORAGE	-0.812	-10011.	-11.16
SOIL WATER AT START OF YEAR	20.64	254527.	
SOIL WATER AT END OF YEAR	19.83	244516.	
SNOW WATER AT START OF YEAR	0.00	0.	
SNOW WATER AT END OF YEAR	0.00	0.	
ANNUAL WATER BUDGET BALANCE	0.00	0.	0.00

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 80 THROUGH 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.79	0.95	0.66	0.53	0.75	0.76
	0.16	0.07	0.58	0.54	1.28	1.88
STD. DEVIATIONS	0.56	0.37	0.35	0.33	0.46	0.23
	0.13	0.08	0.17	0.49	0.69	1.13
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.759	1.189	0.772	0.472	0.599	0.654
	0.382	0.161	0.199	0.277	0.512	0.398
STD. DEVIATIONS	0.152	0.276	0.571	0.077	0.586	0.194
	0.091	0.115	0.062	0.043	0.242	0.098
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.8255	0.1855	0.0711	0.0288	0.0186	0.0162
	0.0191	0.0183	0.0126	0.0101	0.3076	1.0968
STD. DEVIATIONS	0.2807	0.1077	0.0614	0.0125	0.0047	0.0058
	0.0152	0.0192	0.0108	0.0060	0.2884	0.9221
PERCOLATION FROM LAYER 3						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 80 THROUGH 84

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	8.94	(2.310)	110285.	100.00
RUNOFF	0.000	(0.000)	0.	0.00
EVAPOTRANSPIRATION	6.374	(0.868)	78614.	71.28

LATERAL DRAINAGE FROM LAYER 2	2.6102 (1.2274)	32193. ≈240880	29.19
PERCOLATION FROM LAYER 3	0.0000 (0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.042 (0.612)	-522.	-0.47

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PEAK DAILY VALUES FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	19240.0
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	1.1782	14531.0 ≈ 108692
PERCOLATION FROM LAYER 3	0.0000	0.0
HEAD ON LAYER 3	1.1	
SNOW WATER	0.89	10969.9

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1913
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0632

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FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)
1	3.02	0.1260
2	0.60	0.0502
3	16.20	0.4500
SNOW WATER	0.00	

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EBASCO-1; WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
SIMULATION OF EBASCO MODEL - TEST OF HELP VERSIONS
M.L. BROWN - January 9, 1990; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.6468 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	23.2800 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	= 0.25
START OF GROWING SEASON (JULIAN DATE)	= 124
END OF GROWING SEASON (JULIAN DATE)	= 276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 80 THROUGH 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.79	0.95	0.66	0.53	0.75	0.76
	0.16	0.07	0.58	0.54	1.28	1.56
STD. DEVIATIONS	0.56	0.37	0.35	0.33	0.46	0.23
	0.13	0.08	0.17	0.49	0.69	0.61
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.694	1.197	0.797	0.480	0.609	0.650
	0.468	0.186	0.193	0.303	0.659	0.434
STD. DEVIATIONS	0.119	0.192	0.583	0.114	0.595	0.321
	0.188	0.171	0.089	0.088	0.422	0.136
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.7553	0.3269	0.1588	0.0827	0.0575	0.0435
	0.0413	0.0352	0.0277	0.0245	0.0684	0.4619
STD. DEVIATIONS	0.1682	0.1628	0.0651	0.0214	0.0104	0.0066

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 80 THROUGH 84

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	8.63	(1.717)	97034.	100.00
RUNOFF	0.000	(0.000)	0.	0.00
EVAPOTRANSPIRATION	6.670	(0.829)	74999.	77.29
LATERAL DRAINAGE FROM LAYER 2	2.0838	(0.6102)	23430.	24.15
PERCOLATION FROM LAYER 3	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.124	(0.409)	-1395.	-1.44

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PEAK DAILY VALUES FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)
PRECIPITATION	0.93	10456.7
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	0.0678	762.4
PERCOLATION FROM LAYER 3	0.0000	0.0
HEAD ON LAYER 3	0.0	
SNOW WATER	0.76	8598.5

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1800

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0630

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FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)

3	16.20	0.4500
SNOW WATER	0.00	

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-EBASCO-2; WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
SIMULATION OF EBASCO MODEL - INCLUDES 25yr PRECIP EVENT
M.L. BROWN - January 9, 1990; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.6468 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	23.2800 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	= 0.25
START OF GROWING SEASON (JULIAN DATE)	= 124
END OF GROWING SEASON (JULIAN DATE)	= 276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 80 THROUGH 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.79	0.95	0.66	0.53	0.75	0.76
	0.16	0.07	0.58	0.66	1.28	1.56
STD. DEVIATIONS	0.56	0.37	0.35	0.33	0.46	0.23
	0.13	0.08	0.17	0.77	0.69	0.61
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.694	1.197	0.797	0.480	0.609	0.649
	0.468	0.186	0.193	0.303	0.659	0.434
STD. DEVIATIONS	0.119	0.192	0.583	0.114	0.595	0.321
	0.188	0.171	0.089	0.088	0.422	0.136
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.7619	0.3270	0.1588	0.0827	0.0575	0.0435
	0.0413	0.0352	0.0277	0.0307	0.1477	0.4957
STD. DEVIATIONS	0.1709	0.1629	0.0651	0.0214	0.0104	0.0066

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TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 80 THROUGH 84

	(INCHES)		(CU. FT.)	PERCENT
PRECIPITATION	8.76	(1.688)	98450.	100.00
RUNOFF	0.000	(0.000)	0.	0.00
EVAPOTRANSPIRATION	6.670	(0.830)	75001.	76.18
LATERAL DRAINAGE FROM LAYER 2	2.2096	(0.5408)	24845.	25.24
PERCOLATION FROM LAYER 3	0.0000	(0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.124	(0.396)	-1395.	-1.42

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PEAK DAILY VALUES FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	17540.2
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	0.0678	762.4
PERCOLATION FROM LAYER 3	0.0000	0.0
HEAD ON LAYER 3	0.0	
SNOW WATER	0.76	8598.5

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1800

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0630

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FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)

3	16.20	0.4500
SNOW WATER	0.00	

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EBASCO-3; WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
SIMULATION OF EBASCO MODEL - INCLUDES 25yr PRECIP EVENT IN LATE 83
M.L. BROWN - January 9, 1990; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	80.00
TOTAL AREA OF COVER	=	134925. SQ FT
EVAPORATIVE ZONE DEPTH	=	18.00 INCHES
POTENTIAL RUNOFF FRACTION	=	0.000000
UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.6468 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	23.2800 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	= 0.25
START OF GROWING SEASON (JULIAN DATE)	= 124
END OF GROWING SEASON (JULIAN DATE)	= 276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 80 THROUGH 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.79	0.95	0.66	0.53	0.75	0.76
	0.16	0.07	0.58	0.54	1.28	1.88
STD. DEVIATIONS	0.56	0.37	0.35	0.33	0.46	0.23
	0.13	0.08	0.17	0.49	0.69	1.13
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

STD. DEVIATIONS	0.119	0.192	0.583	0.114	0.595	0.321
	0.188	0.171	0.089	0.088	0.422	0.136
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.9809	0.3289	0.1593	0.0829	0.0576	0.0435
	0.0413	0.0352	0.0277	0.0246	0.0684	0.5457
STD. DEVIATIONS	0.5387	0.1613	0.0647	0.0213	0.0104	0.0065
	0.0131	0.0134	0.0086	0.0062	0.0440	0.5354
PERCOLATION FROM LAYER 3						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	8.94 (2.310)	100542.	100.00
RUNOFF	0.000 (0.000)	0.	0.00
EVAPOTRANSPIRATION	6.670 (0.830)	74995.	74.59
LATERAL DRAINAGE FROM LAYER 2	2.3961 (0.7233)	26941.	26.80
PERCOLATION FROM LAYER 3	0.0000 (0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.124 (1.205)	-1395.	-1.39

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PEAK DAILY VALUES FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	17540.2
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	0.5324	5986.6
PERCOLATION FROM LAYER 3	0.0000	0.0

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MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1927

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0630

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FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)
1	4.46	0.1239
2	0.60	0.0501
3	16.20	0.4500
SNOW WATER	0.00	

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EBASCO-4; WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
SIMULATION OF EBASCO MODEL - INCLUDES 25yr PRECIP EVENT IN LATE 83
USES 24 inch Ops Layer; ML BROWN - 1-9-90; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	80.00
TOTAL AREA OF COVER	=	134925. SQ FT
EVAPORATIVE ZONE DEPTH	=	18.00 INCHES
POTENTIAL RUNOFF FRACTION	=	0.000000
UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.6468 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	21.1200 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	=	0.25
START OF GROWING SEASON (JULIAN DATE)	=	124
END OF GROWING SEASON (JULIAN DATE)	=	276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 80 THROUGH 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.79	0.95	0.66	0.53	0.75	0.76
	0.16	0.07	0.58	0.54	1.28	1.88
STD. DEVIATIONS	0.56	0.37	0.35	0.33	0.46	0.23
	0.13	0.08	0.17	0.49	0.69	1.13
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

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STD. DEVIATIONS	0.119	0.192	0.583	0.114	0.595	0.321
	0.188	0.171	0.089	0.088	0.422	0.136
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.8347	0.2232	0.0802	0.0304	0.0193	0.0158
	0.0235	0.0181	0.0113	0.0094	0.1904	0.9189
STD. DEVIATIONS	0.2661	0.1567	0.0559	0.0113	0.0046	0.0039
	0.0262	0.0192	0.0083	0.0049	0.1716	0.8028
PERCOLATION FROM LAYER 3						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	8.94 (2.310)	100542.	100.00
RUNOFF	0.000 (0.000)	0.	0.00
EVAPOTRANSPIRATION	6.670 (0.830)	74995.	74.59
LATERAL DRAINAGE FROM LAYER 2	2.3752 (0.9428)	26706.	26.56
PERCOLATION FROM LAYER 3	0.0000 (0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.103 (0.705)	-1160.	-1.15

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PEAK DAILY VALUES FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	17540.2
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	1.0117	11374.9
PERCOLATION FROM LAYER 3	0.0000	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1927

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0630

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FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)
1	2.97	0.1239
2	0.60	0.0501
3	16.20	0.4500
SNOW WATER	0.00	

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EBASCO-5; WHC NON-DRAG OFF MIXED RADIOACTIVE WASTE LANDFILL DESIGN
SIMULATION OF EBASCO MODEL - INCLUDES 25yr PRECIP EVENT IN LATE 83
24" Ops Layer; Leaf INDEX = 0; ML BROWN - 1-9-90; 893-1186.032

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LAYER 1

VERTICAL PERCOLATION LAYER

THICKNESS	=	24.00 INCHES
POROSITY	=	0.3890 VOL/VOL
FIELD CAPACITY	=	0.1990 VOL/VOL
WILTING POINT	=	0.0660 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1800 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.009999999776 CM/SEC

LAYER 2

LATERAL DRAINAGE LAYER

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4200 VOL/VOL
FIELD CAPACITY	=	0.0500 VOL/VOL
WILTING POINT	=	0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	1.000000000000 CM/SEC
SLOPE	=	2.00 PERCENT
DRAINAGE LENGTH	=	50.0 FEET

LAYER 3

BARRIER SOIL LINER WITH FLEXIBLE MEMBRANE LINER

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4500 VOL/VOL
FIELD CAPACITY	=	0.3800 VOL/VOL
WILTING POINT	=	0.3000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4500 VOL/VOL
SATURATED HYDRAULIC CONDUCTIVITY	=	0.000000010000 CM/SEC
LINER LEAKAGE FRACTION	=	0.00001000

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GENERAL SIMULATION DATA

SCS RUNOFF CURVE NUMBER	=	80.00
TOTAL AREA OF COVER	=	134925. SQ FT
EVAPORATIVE ZONE DEPTH	=	18.00 INCHES
POTENTIAL RUNOFF FRACTION	=	0.000000
UPPER LIMIT VEG. STORAGE	=	7.0020 INCHES
INITIAL VEG. STORAGE	=	2.3801 INCHES
INITIAL SNOW WATER CONTENT	=	0.0000 INCHES
INITIAL TOTAL WATER STORAGE IN SOIL AND WASTE LAYERS	=	21.1200 INCHES

SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

USER SPECIFIED RAINFALL WITH SYNTHETIC DAILY TEMPERATURES AND SOLAR RADIATION FOR YAKIMA WASHINGTON

MAXIMUM LEAF AREA INDEX	= 0.00
START OF GROWING SEASON (JULIAN DATE)	= 124
END OF GROWING SEASON (JULIAN DATE)	= 276

NORMAL MEAN MONTHLY TEMPERATURES, DEGREES FAHRENHEIT

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 80 THROUGH 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.79	0.95	0.66	0.53	0.75	0.76
	0.16	0.07	0.58	0.54	1.28	1.88
STD. DEVIATIONS	0.56	0.37	0.35	0.33	0.46	0.23
	0.13	0.08	0.17	0.49	0.69	1.13
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000

STD. DEVIATIONS	0.152	0.276	0.571	0.077	0.586	0.194
	0.091	0.115	0.062	0.043	0.242	0.098
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.8255	0.1855	0.0711	0.0288	0.0186	0.0162
	0.0191	0.0183	0.0126	0.0101	0.3076	1.0968
STD. DEVIATIONS	0.2807	0.1077	0.0614	0.0125	0.0047	0.0058
	0.0152	0.0192	0.0108	0.0060	0.2884	0.9221
PERCOLATION FROM LAYER 3						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	8.94 (2.310)	100542.	100.00
RUNOFF	0.000 (0.000)	0.	0.00
EVAPOTRANSPIRATION	6.374 (0.868)	71669.	71.28
LATERAL DRAINAGE FROM LAYER 2	2.6102 (1.2274)	29349.	29.19
PERCOLATION FROM LAYER 3	0.0000 (0.0000)	0.	0.00
CHANGE IN WATER STORAGE	-0.042 (0.612)	-476.	-0.47

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PEAK DAILY VALUES FOR YEARS 80 THROUGH 84

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	17540.2
RUNOFF	0.000	0.0
LATERAL DRAINAGE FROM LAYER 2	1.1782	13247.2
PERCOLATION FROM LAYER 3	0.0000	0.0

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1913

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0632

****

****

FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)
1	3.02	0.1260
2	0.60	0.0502
3	16.20	0.4500
SNOW WATER	0.00	

****
****

Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 4

Project Title: NON DRAG OFF

Project No. 893-1186

Calculation Title: LEACHATE TANK SECONDARY CONTAINMENT SYSTEM B.24

	Name	Date
Prepared By:	M. BROWN	1/22/90
Checked By:	Frank S. Shuri	2-7-90
Approved By:	[illegible]	2-8-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT NDO Tank Sump		
Job No. 893-1186 Ref.	Made by M. BROWN Checked FR Reviewed	Date 1/22/90 Sheet 1 of 4

OBJECTIVE:

SIZE SUMP

CRITERIA: 1.0 tank Volume + Ave Annual Precip.

Approx. Tank dimensions = 8'6" x 25' long ⇒ Vol. = $\frac{\pi D^2}{4} \times L$ = 1256 ft³

From Spec. 2

Tank Volume = 10,000 gal. = 1337 ft³ ✓

Area of bottom of Sump [from dwg., attached]

Interior 12 wide x 35' long = 420 ft² ✓

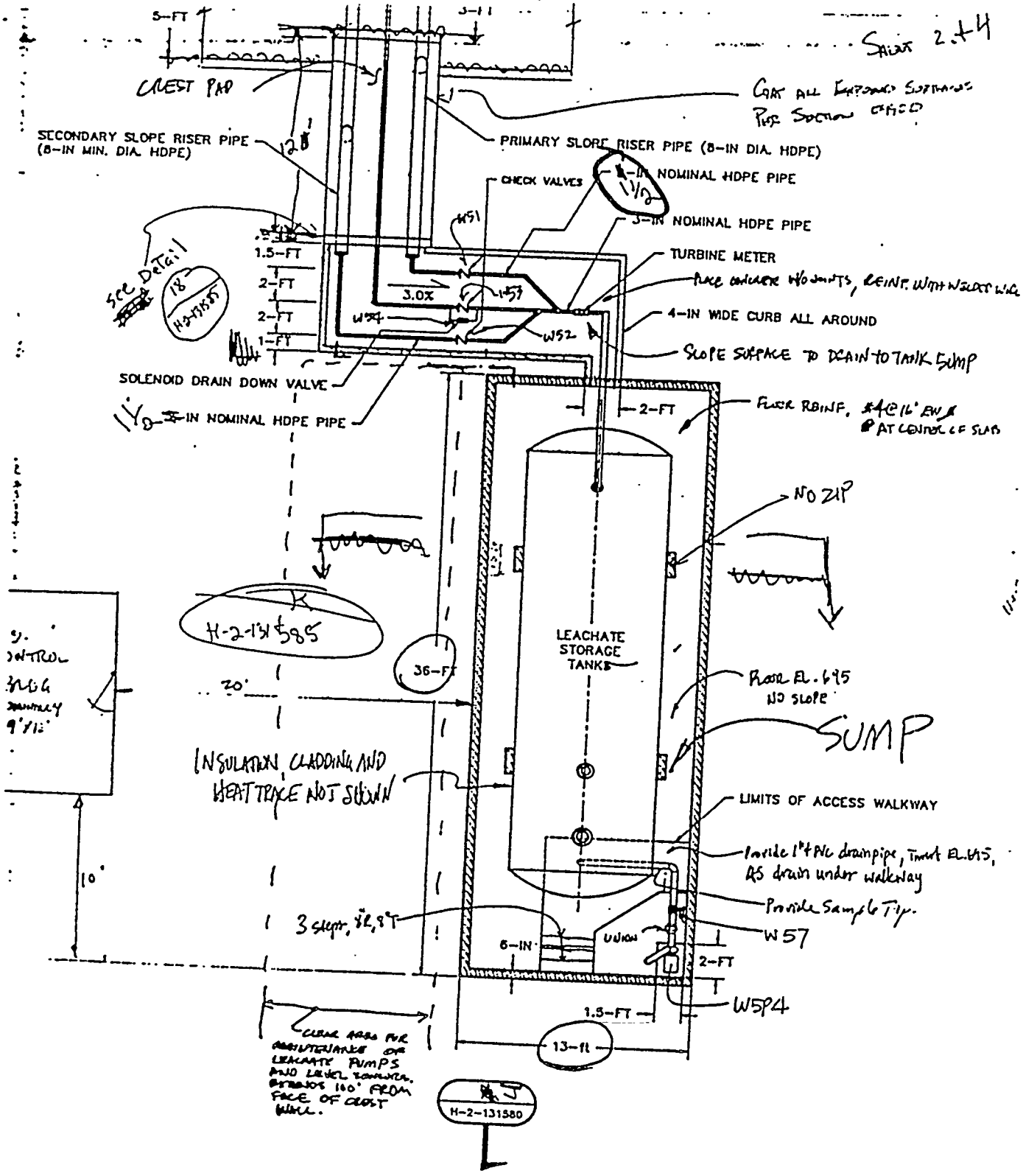
depth to hold 1 tank volume = $\frac{1337\ ft^3}{420\ ft^2}$ = 3.18 ft.

Ave. Annual Precip = 6.3" say 0.525 ft. ✓

[See attached record]

Depth required = 3.18 + 0.525 = 3.705' ft ✓

USE WALL HEIGHT OF 4.0 ft. ✓

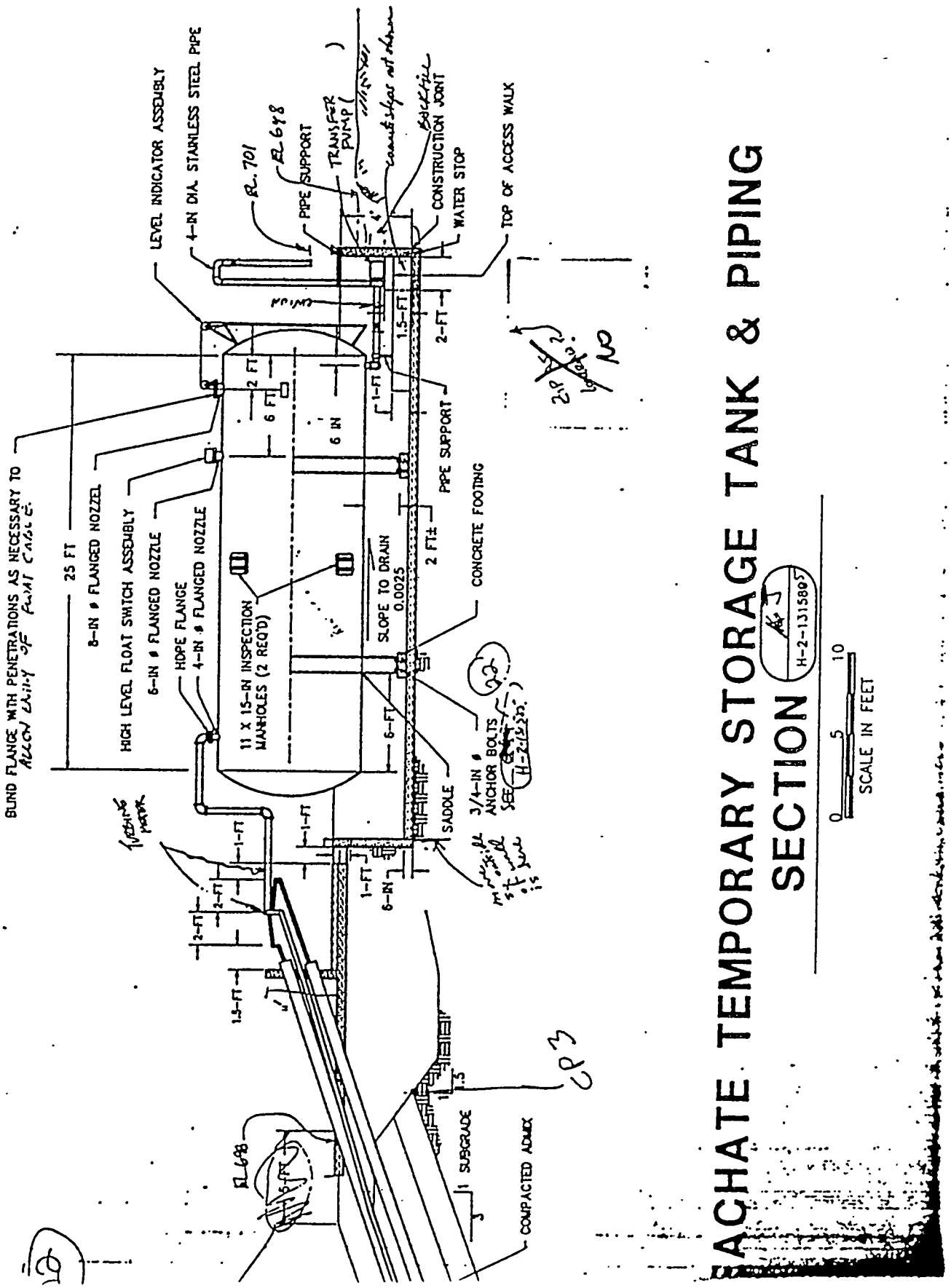


IATE TEMPORARY STORAGE TANK & PIP

PLAN

0 5 10

SCALE IN FEET



LEACHATE TEMPORARY STORAGE TANK & PIPING SECTION

H-2-131580

0 5 10

SCALE IN FEET

from Climatological Summary for Hanford Area. Sheet 4 of 4

TABLE 10. Composite Record of Monthly and Annual Precipitation (Inches) for Hanford Townsite (Near) and HMS: 1912 Through 1980

YEAR	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	TOTAL
1912	---	---	---	---	0.93	0.72	0.09	0.93	0.04	0.55	1.19	0.46	14.41)
1913	1.39	0.16	0.05	0.16	0.61	1.11	0	T	0.03	1.42	0.75	0.76	6.44
1914	1.12	1.15	T	0.54	0.60	0.29	0.73	0	0.32	1.75	0.58	0.74	7.82
1915	0.65	0.61	0.58	0.39	0.90	0.76	0.21	T	T	0.20	1.56	1.18	6.96
1916	0.69	1.54	0.55	0.77	0.01	---	---	0	0.02	0	1.20	0.87	(5.65)
1917	1.00	0.28	0.31	0.77	0.37	0.09	T	0	0.87	0	0.81	0.50	4.35
1918	0.63	0.50	0.22	0	0.34	0.44	0.28	0.36	0.15	0.54	0.69	0.49	4.90
1919	1.04	1.09	0.38	0.32	0.44	0	T	T	0.34	0.21	0.88	1.00	6.30
1920	1.46	T	T	0.78	0.05	0.69	T	1.18	0.75	0.51	0.99	1.72	7.63
1921	0.36	1.25	T	0.60	0.33	0.12	T	0.34	0.06	0.79	1.61	0.69	5.85
1922	0.37	0.45	0.26	0.14	T	0.65	0	0.26	0.26	0.72	0.28	1.03	4.42
1923	0.37	0.40	0.05	0.85	0.27	1.11	0.33	T	0.43	1.03	0.46	0.56	5.46
1924	1.02	0.78	0.10	0.01	T	0.14	0.37	0.17	0.83	0.77	1.36	1.16	(6.71
1925	0.46	0.78	0.06	0.80	0.87	0.34	0	T	0.49	0.07	0.96	0.66	5.41
1926	0.93	0.57	0.08	0.04	0.26	0.80	0	0.10	0.30	0.40	3.05	0.78	7.31
1927	1.79	0.77	0.45	0.05	0.09	T	T	0.79	1.14	0.62	1.76	0.60	6.34
1928	1.34	0.09	0.63	0.71	0.17	0.06	0.35	0.05	0.01	0.58	0.49	1.33	5.56
1929	1.37	0.07	0.02	0.05	0.25	0.89	T	0.02	0.15	0.87	T	0.84	3.76
1930	0.58	0.78	0.14	0.24	0.43	0.09	T	0.06	0.48	0.01	0.32	0.30	3.38
1931	1.35	0.17	0.99	0.11	0	0.51	0.19	0	0.16	0.47	1.15	2.33	7.43
1932	1.52	0.15	0.63	0.96	0.79	0.06	0.03	0.10	T	0.77	0.65	0.74	5.40
1933	0.66	0.50	0.43	0	1.06	0.17	0.10	0.13	0.44	0.59	0.48	1.20	5.96
1934	0.41	0.10	0.57	0.79	1.56	0.12	0	0.45	0.71	0.34	1.05	0.50	6.50
1935	0.41	0.31	0.05	0.08	0.13	0.37	0.46	0	0.08	0.39	0.30	0.87	3.42
1936	1.54	0.65	0	0.35	0.33	0.50	0.04	0.01	0.78	0.06	T	0.98	4.74
1937	0.80	0.63	0.76	0.73	0.01	1.09	0.24	0.06	0.16	0.39	1.84	1.08	7.79
1938	0.35	0.77	1.15	0.18	0.11	1.78	0.06	0.14	0.02	0.79	0.67	0.71	6.73
1939	0.59	0.80	0.57	T	0.07	0.01	0	0.25	0.11	0.25	T	1.69	4.34
1940	0.64	1.48	0.47	0.76	0.78	0.11	0.34	0	0.08	2.03	1.45	1.57	11.76
1941	1.87	0.65	0.16	0.31	0.83	1.16	0.06	0.59	0.57	0.41	0.46	0.40	7.44
1942	0.46	0.47	0	0.70	1.07	1.78	0.02	0	T	0.54	1.34	1.19	6.57
1943	0.48	0.37	0.33	---	---	---	---	---	---	---	---	0.23	(1.33)
1944	0.39	0.56	0.83	0.77	---	---	---	---	---	---	---	---	(1.75)
1945	---	---	---	---	---	---	---	---	---	---	---	---	---
1946	---	---	---	---	---	---	0.15	0.35	0.52	0.65	0.66	0.11	(2.44)
1947	0.32	0.27	0.42	0.70	0.02	1.07	0.71	0.46	1.36	2.20	0.81	0.75	9.79
1948	1.36	0.49	0.07	0.95	1.71	1.47	0.40	0.39	0.16	0.45	0.95	1.11	9.71
1949	0.13	0.68	1.12	0.02	0.16	0.01	0.01	0.05	0.23	0.10	1.47	0.16	4.12
1950	1.80	1.06	0.87	0.47	0.77	2.92	0.07	T	0.01	2.46	0.55	0.97	11.45
1951	0.84	0.51	0.46	0.53	0.43	1.38	0.37	0.15	0.10	0.71	0.87	0.70	7.00
1952	0.65	0.50	0.06	0.13	0.58	1.07	T	0.08	0.08	0.04	0.20	0.77	4.16
1953	2.16	0.25	0.17	0.77	0.78	0.55	T	0.96	0.13	0.70	0.96	0.49	6.92
1954	1.48	0.76	0.59	0.07	0.41	0.18	0.72	0.42	0.51	0.47	0.86	0.35	5.71
1955	0.56	0.27	0.11	0.40	0.59	0.78	0.57	0	0.77	0.40	1.54	2.03	7.53
1956	1.71	0.56	0.10	T	0.27	0.84	T	0.34	0.04	1.03	0.15	0.58	5.60
1957	0.48	0.73	1.84	0.38	0.67	0.47	0.05	0.02	0.34	1.72	0.39	0.53	8.79
1958	1.74	1.48	0.46	0.64	0.74	0.81	0.02	T	0.05	0.19	0.77	1.84	8.74
1959	2.05	1.17	0.48	0.70	0.58	0.73	T	0.08	1.76	0.34	0.41	0.76	7.80
1960	0.51	0.38	0.67	0.53	0.71	0.14	T	0.76	0.23	0.23	0.92	0.64	5.47
1961	0.33	2.18	1.02	0.48	0.80	0.42	0.15	0.89	T	0.07	0.49	0.88	6.83
1962	0.13	0.90	0.14	0.34	1.35	0.12	T	0.50	0.36	0.95	0.62	0.60	6.03
1963	0.95	0.69	0.53	1.17	0.43	0.78	0.31	0.01	0.02	0.04	0.74	1.14	6.31
1964	0.37	0.01	0.05	0.11	0.01	0.90	0.84	0.24	0.89	0.28	0.94	2.34	5.39
1965	0.93	0.34	0.03	0.89	0.15	0.49	0.11	0.08	0.11	0.04	1.17	0.39	3.85
1966	0.68	0.05	0.39	0.03	0.05	0.43	0.11	T	0.27	0.39	2.25	0.60	5.93
1967	0.32	T	0.14	0.92	0.56	0.57	T	T	0.05	0.13	0.16	0.43	3.26
1968	0.68	0.34	0.02	0.01	0.06	0.19	0.04	0.51	0.25	0.93	1.23	1.25	5.95
1969	1.24	0.34	0.10	1.22	0.51	0.75	T	T	0.48	0.10	0.13	1.29	6.36
1970	2.41	0.75	0.27	0.43	0.34	0.25	0.01	T	0.08	0.24	0.31	0.61	6.33
1971	0.78	0.18	1.02	0.07	0.56	0.71	0.13	0.09	1.13	0.18	0.46	1.07	6.30
1972	0.19	0.27	0.31	0.10	2.01	0.46	0.16	0.56	0.83	T	0.55	1.27	6.39
1973	0.90	0.71	0.08	T	0.24	0.01	T	0.02	0.43	1.72	2.64	2.02	8.77
1974	0.90	0.41	0.52	0.46	0.78	0.12	0.71	T	0.01	0.21	0.71	0.97	5.30
1975	1.43	0.98	0.33	0.42	0.36	0.24	0.32	1.16	0.02	0.07	0.60	0.78	7.46
1976	0.56	0.36	0.73	0.41	0.08	0.11	0.33	0.06	T	0.64	T	0.11	2.99
1977	0.08	0.57	0.41	T	0.65	0.32	0.06	1.36	0.66	0.15	0.63	1.02	6.41
1978	1.22	0.92	0.30	0.06	0.41	0.09	0.52	0.10	0.11	T	1.21	0.76	6.57
1979	0.54	0.17	0.54	0.52	0.10	T	0.09	0.36	0.20	0.67	1.36	0.99	5.56
1980	1.32	1.30	0.30	0.86	1.41	0.06	T	0.02	0.85	0.33	0.64	1.89	9.68
AVG	0.92	0.60	0.37	0.39	0.48	0.54	0.15	0.24	0.31	0.56	0.85	0.89	6.30
*NORMAL	0.97	0.36	0.40	0.39	0.53	0.65	0.16	0.29	0.29	0.46	0.80	0.95	6.25

← Ave. Annual.

THE HIGHEST, AND IF MEASURABLE, THE LOWEST TOTALS IN EACH COLUMN ARE UNDERSCORED

T DENOTES A TRACE (<0.005 INCH)

( ) DENOTES AN INCOMPLETE ANNUAL TOTAL

* NORMALS ARE AVERAGES FOR THE PERIOD 1951-1980

--- DENOTES MISSING DATA

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 20

Project Title: WHC Non Drag Off

Project No. 893-1186

Calculation Title: Primary Leachate Collection System
B.12

	Name	Date
Prepared By:	M. Brown	1/22/90
Checked By:	Frank S. Shuri	2-7-90
Approved By:	Terry Cook	2-8-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

CALCULATION B.12

Golder Associates

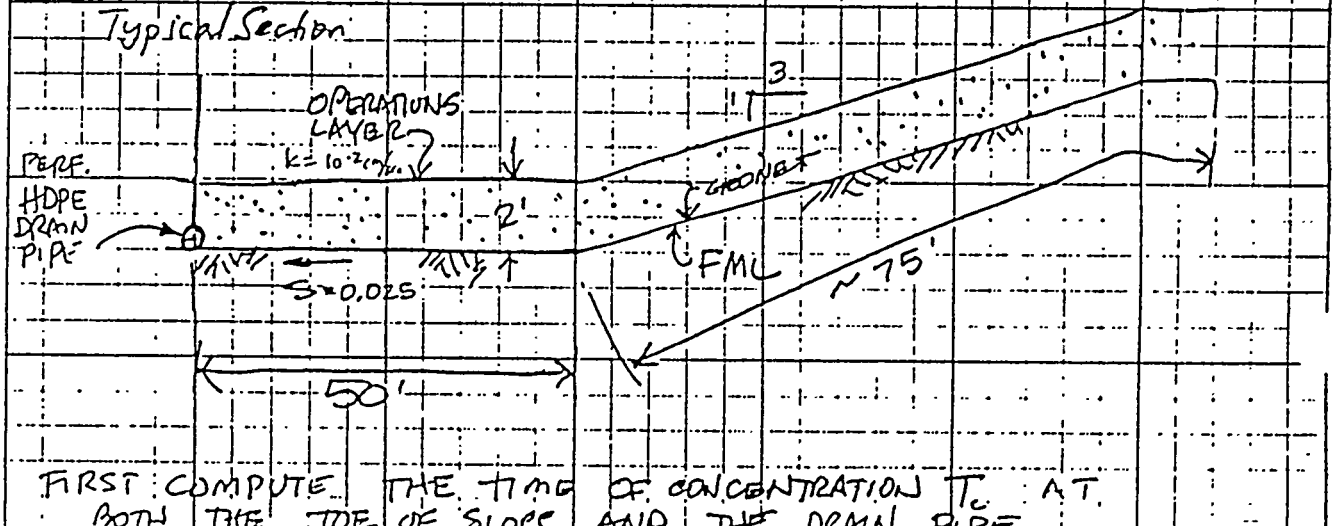
SUBJECT			
SUBJECT NDO PRIMARY LEACHATE COLECTION SYSTEM			
Job No. 893-1186	Made by M. BROWN	Date 1/22/90	
Ref.	Checked FSS	Sheet 2 of 20	
	Reviewed RAS		

OBJECTIVES

Select capacities of drainage elements

Method

Typical Section



FIRST COMPUTE THE TIME OF CONCENTRATION T_c AT BOTH THE TOE OF SLOPE AND THE DRAIN PIPE.

Travel time through operations layer

$Q = kiA$ $\quad \frac{Q}{A} = ki$ = apparent velocity

actual velocity $= \frac{ki}{e} = \frac{10^{-2}\text{cm/sec} \times 1}{0.38} = 0.0263\text{cm/sec}$ ✓

e: effective porosity, see attached TA ✓

Distance = 24" × 2.54cm/in. = 60.96cm ✓

TIME = 60.96 / 0.0263cm/sec = 38.6 minutes ✓

Travel time in geonet ~ $\frac{\text{distance}}{\text{velocity}} = \frac{75'}{0.6\text{fps}} = 125\text{sec} \approx 2\text{ min}$ ✓

See attached sheet 1B

Author's First Draft 10/21/88

Ref pg 3 of 2

THE HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL

Volume IV. Documentation for Version 2

by

P. R. Schroeder, B. M. McEnroe, R. L. Peyton, and J. W. Sjostrom
US Army Engineer Waterways Experiment Station
Vicksburg, MS 39181-0631

Interagency Agreement Number DW21931425-01-3

Project Officer

R. Landreth
Landfill Pollution Control Division
Hazardous Waste Engineering Research Laboratory
Cincinnati, OH 45268

HAZARDOUS WASTE ENGINEERING RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
US ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OH 45268

OFFICE OF SOLID WASTE AND EMERGENCY RESPONSE
US ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, DC 20460

Sheet ~~NA~~ 4 of 4

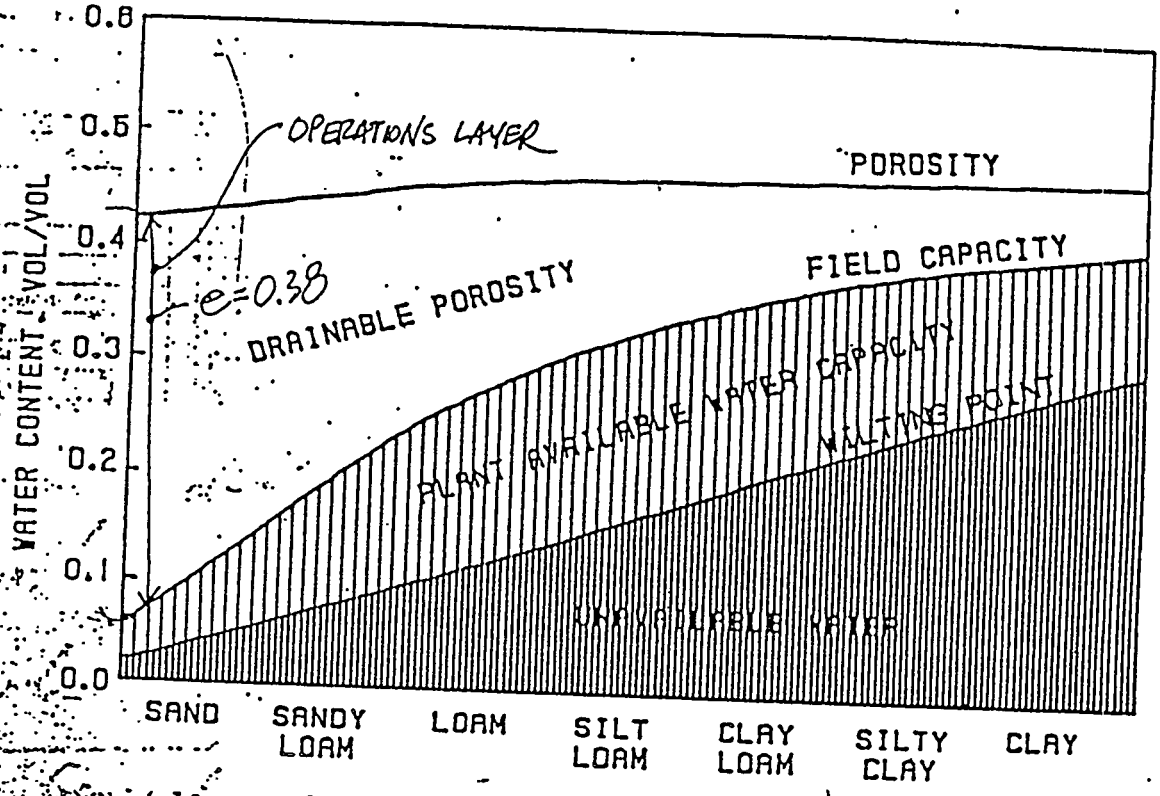


Figure 6. General relation between soil water properties and soil texture (Rawls et al., 1982)

FROM "HELP" MODEL DOCUMENTATION

40

sheet 5 of 20

REFERENCES

Arnold, J. G., J. R. Williams, and A. D. Nicks. 1986. SWRRB, A Simulator for Water Resources in Rural Basins. Agricultural Research Service, USDA.

Brooks, R. H. and A. T. Corey. 1964. Hydraulic Properties of Porous Media. Hydrology Paper No. 3, Colorado State University. 27pp.

Brown, K. W., J. C. Thomas, R. L. Lytton, P. Jayawikrama, and S. C. Bahrt. 1987. Quantification of Leak Rates Through Holes in Landfill Liners. EPA/600/ S2-87-062. U.S. Environmental Protection Agency, Cincinnati, OH.

Campbell, G. S. 1974. A Simple Method for Determining Unsaturated Hydraulic Conductivity from Moisture Retention Data. Soil Science, Vol. 117, No. 6. pp. 311-314.

Chow, V. T., Editor. 1964 Handbook of Applied Hydrology. McGraw-Hill Book Company, New York, New York.

Horton, R. E. 1919. Rainfall Interception. Monthly Weather Review, U.S. Weather Bureau, Vol. 47, No. 9. pp. 603-623.

IBM. 1974. IBM OS FORTRAN IV (H extended). Compiler Programmer's Guide. International Business Machines Corporation, New York, New York.

Knisel, W. G., Editor. 1980. CREAMS, A Field Scale Model for Chemical Runoff and Erosion from Agricultural Management Systems. Vols. I, II, and III, Draft Copy, USDA-SEA, AR, Cons. Res. Report 24. 643 pp.

Li, E. A. 1975. A Model to Define Hydrologic Response Units Based on Characteristics of the Soil-Vegetative Complex Within a Drainage Basin. M.S. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA. 124 pp.

Linsley, R. K., M. A. Kohler, and J. L. H. Paulhus. 1982 Hydrology for Engineers, Third Edition, McGraw-Hill Book Company, New York, New York. 508 pp.

Lutton, R. J., G. L. Regan, and L. W. Jones. 1979. Design and Construction of Covers for Solid Waste Landfills. EPA-600/2-79-165. U.S. Environmental Protection Agency, Cincinnati, OH.

Perrier, E. R. and A. C. Gibson. 1980. Hydrologic Simulation on Solid Waste Disposal Sites. EPA-SW-868, U.S. Environmental Protection Agency, Cincinnati, OH. 111 pp.

Rawls, W. J., D. L. Brakensiek, and K. E. Saxton. 1982. Estimation of Soil Water Properties. Transactions of the American Society of Civil Engineers. pp. 1316-1320.

Richardson, C. W. 1981. Stochastic Simulation of Daily Precipitation, Temperature, and Solar Radiation. Water Resources Research, Vol. 17, No. 1, pp. 182-190.

Golder Associates

SUBJECT NDO Primary

Job No. 893-1186 Ref.

Made by M. BROWN Checked FBS Reviewed RCL

Date 1/28-90 Sheet 18 of 20

Estimated travel velocity in geonet.

We know that geonet transmissivity is 3 to 6 gpm/ft.

Conservatively use 6 gpm/ft, this is for i=1

for i=0.33 on slope of facility (6 x 0.33)

T= 2 gpm/ft ✓ (POINT T= 7.68 gpm/ft.) OK

for our calculation in feet + seconds: T= 0.0045 cfs./ft ✓

Estimate effective porosity at 0.5 (by examination of material)

$$\text{travel velocity} = \frac{0.0045 \text{ cfs}}{(0.5)(1 \text{ ft})(0.015 \text{ ft})} = 0.6 \text{ fps} \checkmark$$

↑ approx. thickness

Golder Associates

SUBJECT NDO Primary Leachate

Job No. | Made by | Date

Ref. | Checked FSS | Sheet 7 of 20

Reviewed RE

Time of concentration at toe of slope = 38.6 + 2 = 40.6 min.

Max. Rainfall intensity (Fig 36 of Hanford Climatological Data) [ATTACHED] ✓

for T = 40.6 min ≅ 41 min

i = 0.8 in/hour

for 1 acre Q = 0.8 cfs = 358 gpm

for 1 ft wide strip, 75 ft long

$Q_{toe} = \frac{75}{43560} \times 358 \text{ gpm} = 0.62 \text{ gpm}$ ✓

Flow at center of facility $Q_c = \frac{75+50}{43560} \times 358 = 7.02 \text{ gpm}$ ✓

↑ Actual Q is slightly lower due to increased T_c and resulting reduced rainfall intensity, conservatively use 358.

Cuspate capacity of geonet (see attached page 2A.)

From manufacturer's info transm ≅ 6 gpm/ft (i = 1.0)

for i = 0.025 T = 0.15 gpm/ft < 0.62 gpm NOT OK on Floor

· ti

on Slope i = 0.33 T = 6 × .33 = 2 gpm/ft > 0.62 gpm

∴ OK on Slope

AN

USE ADDITIONAL HDPE PERF PIPE AT TOE OF SLOPE. ✓

Golder Associates

SUBJECT: NOO PRIMARY LEACHATE

Job No.
Ref.

Made by: M. FRANK
Checked: FSS
Reviewed: [illegible]

Date: 1-30-90
Sheet: 8 of 20

The text-net which we propose to use is TN1001 or equal. The transmissivity for TN1001 = 0.0015 m²/sec (see attached geofabrics report). However, this will be specified as an "or equal".

Assume Transmissivity of final product is on step low - similar to TN3001, T = 0.0012 m²/sec.

$$T = 0.0012\ \frac{m^2}{sec} \times \left(3.28\ \frac{ft}{m}\right)^2 \times 449\ \frac{gpm}{ft^3/sec} = 5.8\ gpm/ft$$

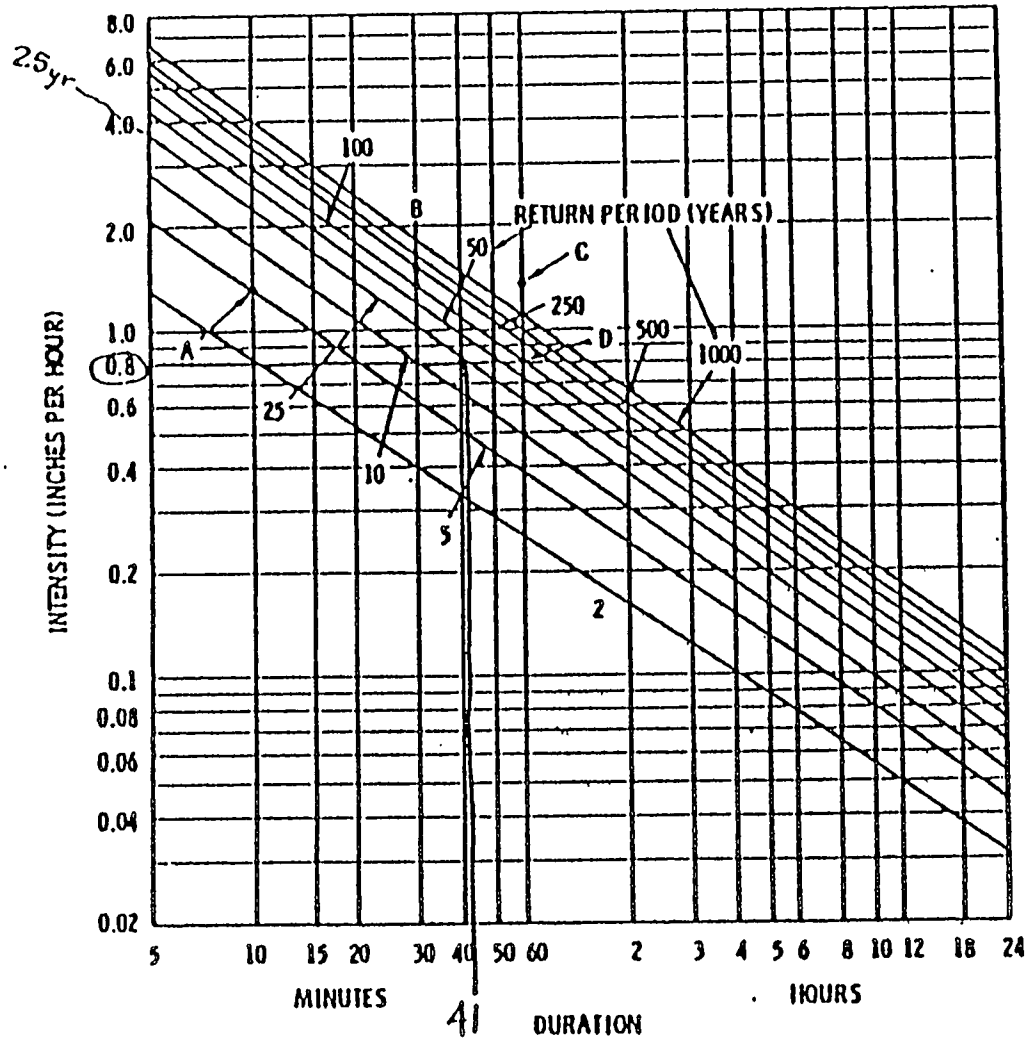
∴ Transmissivity of geonet ≈ 6.0 gpm/ft

DRAINAGE PRODUCTS

Product Name	Structure[7]	Polymer Composition	Dimensional Properties: Width/ Length m (ft)	Dimensional Properties: Thickness ASTM D1777[8] mm (mils)	Crush Strength ASTM D1621 kPa (psi)	Transmissivity: Geotextile Product if attached[9]	Transmissivity: ASTM D4716-87[4] 10 kPa (1.45 psi)	Transmissivity: ASTM D4716-87[4] 100 kPa (14.5 psi)
							M^2/S (gal/min/ft)	M^2/S (gal/min/ft)
Contech Construction Products Inc.								
C-112	geonet with laminated geotextile	polyethylene	1.6/7.6-30.5 (5.2/25-100)	5.6 (220)	na	Treves 1112	na	na
C-114	geonet with laminated geotextile	polyethylene	1.6/7.6-30.5 (5.2/25-100)	5.6 (220)	na	Treves 1114	na	na
C-212	geonet with laminated geotextile	polyethylene	1.6/7.6-30.5 (5.2/25-100)	5.6 (220)	na	Treves 1112	na	na
C-214	geonet with laminated geotextile	polyethylene	1.6/7.6-30.5 (5.2/25-100)	5.6 (220)	na	Treves 1114	na	na
STRIPDRAIN 100	cuspated core & overwrapped geotextile	HDPE	.30-.46-.61-.76-.91/122 (1.1-1.5-2-2.5-3/400)	25.4 (1000)	379 (55)	Treves 1114	5.11 (24.7)	4.66 (22.5)
STRIPDRAIN 75	cuspated core & laminated geotextile	HDPE	.55-1.1/55 (1.8-3.6/180)	19.1 (750)	276 (40)	Treves 1114	.62 (3)	.41 (2)
Conwed Plastics								
LXE-9000	thermoformed net	polyethylene	1.2/30.5 (4/100)	15.2 (600)	280 (40)	various	7.4 (35.8)[A,B]	6.6 (31.9)[B]
XB8110	geonet	polyethylene	2.1/91.4 (6.9/300)	6.1 (240)	>900 (130)	various	3.4 (16.4)[A,B]	3.0 (14.5)[B]
XB8210	geonet	polyethylene	2.1/91.4 (6.9/300)	4.1 (160)	>900 (130)	various	1.9 (9.2)[A,B]	1.8 (8.7)[B]
XB8310	geonet	polyethylene	2.1/91.4 (6.9/300)	4.8 (190)	>900 (130)	various	2.7 (13.0)[A,B]	2.6 (12.6)[B]
XB8315CN	geonet	polyethylene	2.1/91.4 (6.9/300)	5.1 (200)	>900 (130)	various	2.6 (12.6)[A,B]	2.6 (12.6)[B]
XB8410	geonet	polyethylene	2.1/67.1 (6.9/220)	7.6 (300)	>900 (130)	various	4.3 (20.8)[A,B]	4.3 (20.8)[B]
Fluid Systems Inc.								
TEX-NET TN1001	geonet composite	polyethylene	(7.5/250)	5.7 (225)	>900 (130)	Treves 1120[B]	.0015	.0015
TEX-NET TN3001	geonet composite	polyethylene	(7.5/300)	5.1 (200)	>900 (130)	Treves 1120[B]	.0012	.0012
TEX-NET TN3001CN	geonet composite	polyethylene	(7.5/300)	5.1 (200)	<900 (130)	Treves 1120[B]	.0010	.0010
TEX-NET TN4001	geonet composite	polyethylene	(7.5/250)	6.9 (270)	<900 (130)	Treves 1120[B]	.0025	.0025
Greenstreak Inc.								
Deck Drain	composite single-side core	polystyrene	1.2/3.0 (4/10)	9.6 (380)	958 (139)	Amoco 1198 woven polypropylene	1.5-4.8 (7-23)[C]	1.2-4.6 (6-22)[C]
Sheet Drain	composite single-side core	polystyrene	1.2/3.0 (4/10)	9.6 (380)	717 (104)	Amoco 9140 NP polypropylene	1.2-3.9 (6-19)[C]	1-3.7 (5-18)[C]
Sheet Drain HS	composite single-side core	polystyrene	1.2/3.0 (4/10)	5.3 (210)	1199 (174)	Amoco 9140 NP polypropylene	.3-1.1 (1.5-5.2)[C]	.2-10 (1.1-4.9)[C]

[A] 48 kPa - 6.9 psi
[B] Available on both sides
[C] Gradient range of .1 - 1.0
[D] Gradient of .25

All values were requested to be minimum average roll values and all claims are the responsibility of the manufacturer. All product data are intended as a guide. *Geotechnical Fabrics Report* recommends you contact manufacturers before making any specifying/purchasing decisions.



TO USE THIS CHART, SELECT ANY DESIRED RAINFALL INTENSITY AND DURATION AND READ FROM THE DIAGONAL LINES THE EXPECTED FREQUENCY OF SUCH INTENSITY AND DURATION. FOR EXAMPLE, RAINFALL INTENSITY OF 1.3 INCHES PER HOUR FOR 10 MINUTES CAN BE EXPECTED TO OCCUR, ON AVERAGE, ONCE EVERY 5 YEARS (POINT A). HOWEVER, SUCH INTENSITY CAN BE EXPECTED FOR 30 MINUTES DURATION ONLY ABOUT ONCE IN 100 YEARS (POINT B). THE RETURN PERIOD FOR THIS INTENSITY FOR 60 MINUTES DURATION IS GREATER THAN 1000 YEARS (POINT C).

THERE ARE, OF COURSE, VARIATIONS IN USE OF THE CHART. SUPPOSE, FOR EXAMPLE, IT IS DESIRED TO FIND THE "100-YEAR STORM" FOR 60 MINUTES. THIS IS 0.8 INCH (POINT D).

FIGURE 36. Rainfall Intensity, Duration and Frequency Based on the Period 1947 to 1969 at Hanford

Golder Associates

SUBJECT N00

Job No. 873-1186 Ref.

Made by M. BROWN Checked FGS Reviewed RE

Date 1/28/90 Sheet 3/11 of 20

CHECK DRAIN PIPE CAPACITY.

FIRST - ESTIMATE PEAK FLOW WITH FULL SUMP EVACUATION

Estimate travel time to most distant point.

From South West corner. - Thru ops layer 38.6 min

140 ft on slope - velocity ≈ 0.6 fps [Page 18]

Travel time = 140 ft / 0.6 fps = 3.9 min

by experience (constant in relation to assumption) Assume velocity in pipe is about 5 fps. → 1 min

Length of pipe at the of slope = 320'

T_c = 43.5 min

Time of concentration is still ≈ 40.6 min = T_c

Intensity i = 0.9 from rainfall intensity-duration-frequency fig. 36

Intercepting Area - See Attached plan view

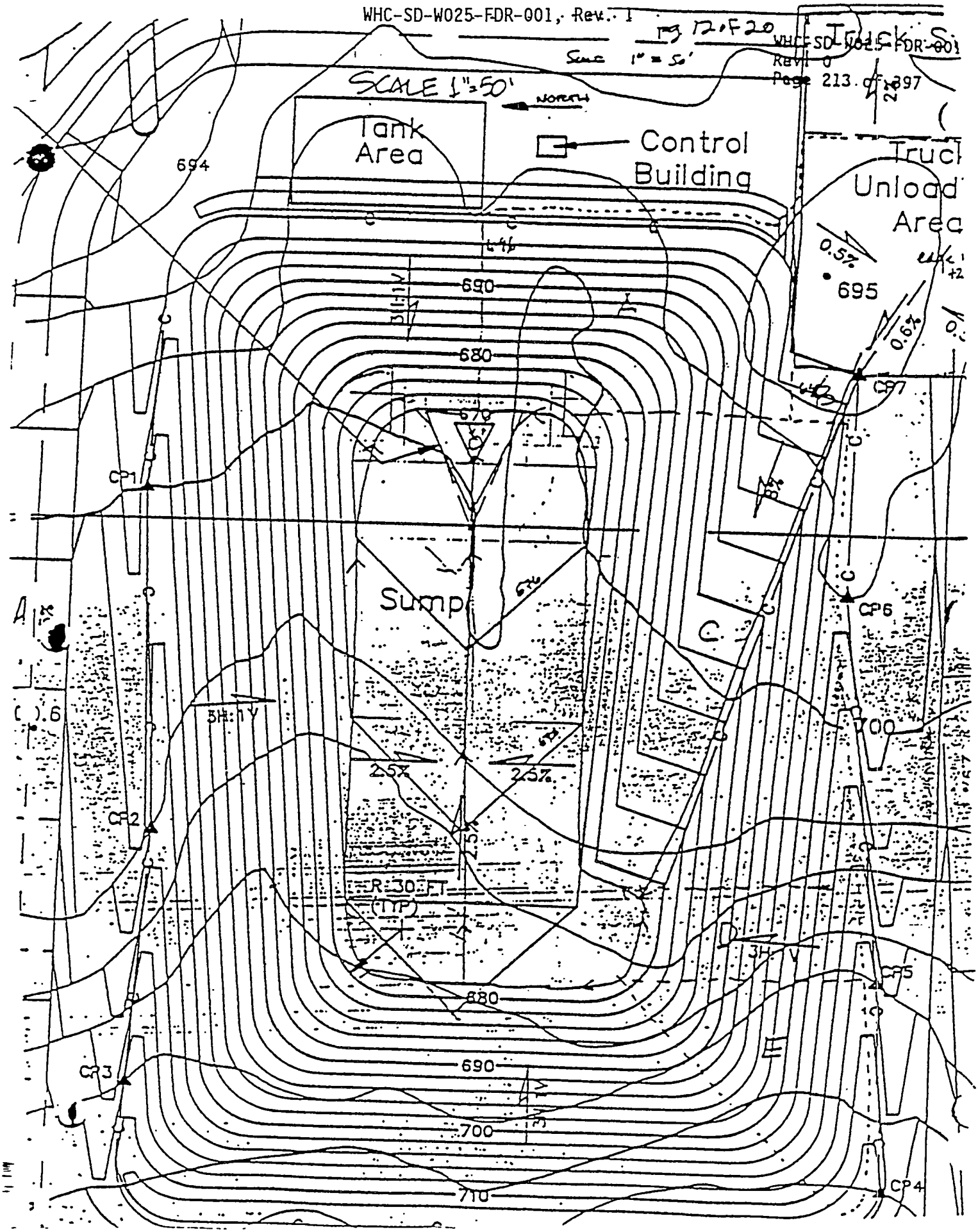
	Area		
Truck Staging		100 x 100 =	10000 ft²
	A	130 x 90 =	11700
	B	25 x 25 =	625
	C	110 x 210 =	23100
	D	90 x 40 =	3600
	E	½ 110' x 110' =	6050
	TOTAL		55075 ft² = 1.26 acres

PEAK FLOW FROM RATIONAL FORMULA

$Q = CIA$ where C = 1 ← no losses

$Q = (1)(0.9 \text{ in/hour})(1.26 \text{ acres}) = 1.008$ cfs

= 454 gpm


SCALE 1"=50'
NORTH
Tank Area
Control Building
Truck Unload Area
694
690
680
670
695
0.5%
0.6%
CP1
CP2
CP3
CP4
CP5
CP6
Sump
3H:1V
25%
25%
R 30 FT (TYP)
3H:1V
680
690
700
710

Golder Associates

SUBJECT NDO

Job No. 897-1186 Ref.

Made by M. Brown Checked TSO Reviewed RR

Date 1/28/00 Sheet 4 of 20

Check capacity of Pipe -

USE Either 6" SDR 11 or 4" SDR11

Using Hazen Williams per Manufacturers info (attached)

C=150 ✓ $Q = 1.008 \text{ cfs} \times \frac{448 \text{ gpm}}{\text{cfs}} = 452$

4" Id = 3.68" ✓

$\Delta P = \frac{452 \times (1 \times 448)^{1.85}}{150^{1.85} \times 3.68^{4.86}} = 6.1 \text{ psi}/100'$ ✓

$\frac{6.1 \text{ psi}}{100'} \times \frac{1}{0.43 \text{ psi/ft}} = 0.141$ friction slope

6" Id = 5.42" ✓

$\Delta P = \frac{452 (1 \times 448)^{1.85}}{150^{1.85} \times 5.42^{4.86}} = 0.93 \text{ psi}/100'$ ✓

$\Delta P = 0.0216$ friction slope

ACTUAL PIPE SLOPE IS 0.025

∴ 4" pipe will cause backwater effects, 6"ϕ will not.

Check capacity of 4" pipe at S = 0.025, use n = 0.009 ✓ [see attached chart]

Manning eqn: from USBR Hydraulic and Excavation Tables, Table 21 attached.

for pipe flowing full: $\frac{Qn}{D^{8/3} S^{1/2}} = 0.463$ ✓ D = pipe ϕ

$Q = \frac{0.463 (0.025)^{1/2} \left(\frac{3.68}{12}\right)^{8/3}}{0.009} = 0.3479 \text{ cfs} = 155 \text{ gpm}$ ✓

There are 3 pipes flowing into primary sump - each with a capacity of 155 gpm. Sump contains 4 pumps with combined discharge capacity of 155 gpm (150+5).

Golder Associates

SUBJECT NDO Primary Leachate Collection

Job No. 893-1186 Ref.

Made by M. BROWN Checked FSS Reviewed TE

Date 1/30/90 Sheet ~~13A~~ 14 of 20

During preliminary design the question of leachate collection capacity was addressed. For the 25 yr event it was decided to allow ponding of water. The pump discharge capacity was limited to 155 gpm.

∴ A 4"ϕ leachate collection pipe on the toe will provide satisfactory performance even if leachate exits the pipe (to be stored in the drainage/ops layer) while it is flowing full. When pump is running the maximum rq'd discharge per pipe is 155/3 = 52 gpm

Capacity from previous pg is 155 gpm > 52 gpm ∴ OK

Backwater effects in pipe acceptable.

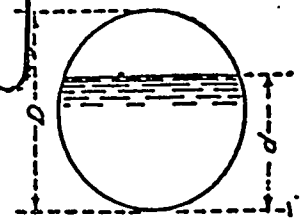
Table 21.—*Uniform flow in circular sections flowing partly full*

d = Depth of flow
D = Diameter of pipe
A = Area of flow
R = Hydraulic radius
Q = Discharge in second-feet by Manning's formula
n = Manning's coefficient
S = Slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.236	1.442
0.02	0.0037	0.0132	0.00031	16.37	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	17.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	18.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	19.15	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	19.85	0.56	0.4525	0.2676	0.278	1.311
0.07	0.0242	0.0451	0.00455	20.47	0.57	0.4623	0.2703	0.285	1.286
0.08	0.0294	0.0513	0.00604	21.09	0.58	0.4722	0.2728	0.292	1.262
0.09	0.0350	0.0573	0.00775	21.78	0.59	0.4819	0.2753	0.298	1.238
0.10	0.0409	0.0635	0.00967	22.29	0.60	0.4920	0.2778	0.311	1.215
0.11	0.0470	0.0695	0.01181	22.83	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0753	0.01417	23.34	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	23.88	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	24.39	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	24.91	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	25.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	25.93	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	26.47	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	27.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	27.65	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	28.27	0.71	0.5964	0.2975	0.395	0.983
0.22	0.1281	0.1312	0.0492	28.79	0.72	0.6054	0.2987	0.402	0.963
0.23	0.1365	0.1364	0.0537	29.31	0.73	0.6143	0.2998	0.409	0.943
0.24	0.1449	0.1416	0.0585	29.93	0.74	0.6231	0.3008	0.416	0.923
0.25	0.1535	0.1466	0.0634	30.56	0.75	0.6319	0.3017	0.422	0.902
0.26	0.1623	0.1516	0.0686	31.18	0.76	0.6405	0.3024	0.428	0.881
0.27	0.1711	0.1566	0.0739	31.82	0.77	0.6489	0.3031	0.433	0.860
0.28	0.1800	0.1614	0.0793	32.36	0.78	0.6573	0.3036	0.441	0.839
0.29	0.1890	0.1662	0.0849	33.00	0.79	0.6655	0.3039	0.447	0.818
0.30	0.1982	0.1709	0.0907	33.65	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	34.28	0.81	0.6815	0.3043	0.458	0.801
0.32	0.2167	0.1802	0.1027	34.96	0.82	0.6893	0.3043	0.463	0.781
0.33	0.2260	0.1847	0.1089	35.59	0.83	0.6968	0.3041	0.466	0.760
0.34	0.2355	0.1891	0.1153	36.25	0.84	0.7043	0.3038	0.473	0.739
0.35	0.2450	0.1935	0.1218	36.90	0.85	0.7115	0.3033	0.477	0.718
0.36	0.2546	0.1978	0.1284	37.58	0.86	0.7186	0.3026	0.481	0.697
0.37	0.2642	0.2020	0.1351	38.25	0.87	0.7254	0.3018	0.485	0.676
0.38	0.2739	0.2062	0.1420	38.93	0.88	0.7320	0.3007	0.488	0.655
0.39	0.2836	0.2102	0.1490	39.63	0.89	0.7384	0.2995	0.491	0.634
0.40	0.2934	0.2142	0.1561	40.32	0.90	0.7445	0.2980	0.494	0.613
0.41	0.3032	0.2182	0.1633	41.03	0.91	0.7504	0.2963	0.496	0.592
0.42	0.3130	0.2220	0.1705	41.74	0.92	0.7560	0.2944	0.497	0.571
0.43	0.3229	0.2258	0.1779	42.47	0.93	0.7612	0.2921	0.498	0.550
0.44	0.3328	0.2295	0.1854	43.19	0.94	0.7662	0.2895	0.498	0.528
0.45	0.3428	0.2331	0.1929	43.92	0.95	0.7707	0.2865	0.498	0.507
0.46	0.3527	0.2366	0.201	44.68	0.96	0.7749	0.2829	0.498	0.486
0.47	0.3627	0.2401	0.208	45.44	0.97	0.7785	0.2787	0.497	0.465
0.48	0.3727	0.2433	0.216	46.20	0.98	0.7817	0.2735	0.489	0.443
0.49	0.3827	0.2468	0.224	46.97	0.99	0.7841	0.2666	0.483	0.422
0.50	0.3927	0.2500	0.232	47.74	1.00	0.7854	0.2500	0.463	0.463

Table 22.—*Velocity head and discharge at critical depths and static pressures in circular conduits partly full*

D = Diameter of circle.
d = Depth of water.
h = Velocity head for a critical depth of d.
Q = Discharge when the critical depth is d.
P = Pressure on cross section of water prism in cubic units of water. To get P in pounds, when d and D are in feet, multiply by 62.5.



$\frac{d}{D}$	$\frac{h}{D}$	$\frac{Q}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h}{D}$	$\frac{Q}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h}{D}$	$\frac{Q}{D^{5/2}}$	$\frac{P}{D^3}$
1	2	3	4	1	2	3	4	1	2	3	4
.01	.0003	.0006	.0000	.34	.1263	.0857	.0332	.67	.2974	2.4464	.1644
.02	.0007	.0023	.0000	.35	.1294	.0940	.0356	.68	.3048	2.5183	.1700
.03	.0014	.0053	.0001	.36	.1326	.1023	.0381	.69	.3125	2.5912	.1758
.04	.0024	.0096	.0002	.37	.1358	.1108	.0407	.70	.3204	2.6656	.1816
.05	.0038	.0153	.0003	.38	.1411	.1196	.0434	.71	.3286	2.7414	.1875
.06	.0053	.0220	.0005	.39	.1454	.1287	.0462	.72	.3371	2.8188	.1935
.07	.0071	.0298	.0007	.40	.1493	.1381	.0491	.73	.3459	2.8977	.1996
.08	.0091	.0389	.0010	.41	.1541	.1543	.0520	.74	.3552	2.9783	.2058
.09	.0114	.0491	.0013	.42	.1586	.9998	.0551	.75	.3648	3.0607	.2121
.10	.0141	.0605	.0017	.43	.1631	1.0438	.0583	.76	.3749	3.1450	.2185
.11	.0170	.0731	.0021	.44	.1676	1.0929	.0616	.77	.3855	3.2314	.2249
.12	.0201	.0868	.0026	.45	.1723	1.1410	.0650	.78	.3967	3.3200	.2314
.13	.0236	.1016	.0032	.46	.1768	1.1898	.0684	.79	.4085	3.4112	.2380
.14	.0272	.1176	.0038	.47	.1817	1.2398	.0720	.80	.4210	3.5050	.2447
.15	.0311	.1347	.0045	.48	.1865	1.2908	.0757	.81	.4343	3.6019	.2515
.16	.0353	.1530	.0053	.49	.1914	1.3427	.0795	.82	.4485	3.7031	.2584
.17	.0389	.1724	.0061	.50	.1964	1.3955	.0833	.83	.4638	3.8091	.2653
.18	.0434	.1928	.0070	.51	.2014	1.4493	.0873	.84	.4803	3.9144	.2723
.19	.0482	.2144	.0080	.52	.2065	1.5041	.0914	.85	.4982	4.0278	.2794
.20	.0529	.2371	.0091	.53	.2117	1.5598	.0956	.86	.5177	4.1465	.2865
.21	.0576	.2609	.0103	.54	.2170	1.6164	.0998	.87	.5392	4.2721	.2938
.22	.0623	.2857	.0115	.55	.2224	1.6733	.1042	.88	.5632	4.4056	.3011
.23	.0671	.3116	.0128	.56	.2279	1.7317	.1087	.89	.5900	4.5486	.3084
.24	.0720	.3386	.0143	.57	.2333	1.7913	.1133	.90	.6204	4.7033	.3158
.25	.0767	.3667	.0157	.58	.2388	1.8530	.1179	.91	.6555	4.8725	.3233
.26	.0815	.3957	.0173	.59	.2431	1.9146	.1227	.92	.6966	5.0603	.3308
.27	.0863	.4259	.0190	.60	.2511	1.9773	.1276	.93	.7450	5.2726	.3384
.28	.0912	.4571	.0207	.61	.2572	2.0408	.1326	.94	.8065	5.5183	.3460
.29	.0962	.4893	.0226	.62	.2633	2.1057	.1376	.95	.8841	5.8118	.3537
.30	.1011	.5223	.0245	.63	.2695	2.1716	.1428	.96	.9885	6.1787	.3615
.31	.1121	.5568	.0266	.64	.2755	2.2386	.1481	.97	1.1410	6.6802	.3692
.32	.1161	.5921	.0287	.65	.2833	2.3067	.1534	.98	1.3958	7.4063	.3770
.33	.1212	.6284	.0309	.66	.2903	2.3760	.1589	.99	1.9700	8.8263	.3848
								1.00	.....	.....	.3927

WHC-SD-W025-FDR-001, Rev. 1
From USBR Hydraulic + Excavation Tables Eighth Edition 1957

DuraTuff®
polyethylene pipe

pg 16 of 20

STRAIGHT LENGTHS

Polaris Pipe Co.
Manufacturers of Plastic Pipe

		SDR 7.3			SDR 9			SDR 11			SDR 13.5		
		255 P.S.I.			200 P.S.I.			160 P.S.I.			130 P.S.I.		
Nominal Pipe Size (Inches)	Actual Outside Diameter	Average Inside Diameter	Minimum Wall Thickness	Weight (lb./ft.)	Average Inside Diameter	Minimum Wall Thickness	Weight (lb./ft.)	Average Inside Diameter	Minimum Wall Thickness	Weight (lb./ft.)	Average Inside Diameter	Minimum Wall Thickness	Weight (lb./ft.)
1½"	1.90	1.38	.260	.59	1.48	.211	.49	1.55	.173	.41	1.62	.141	.34
2"	2.375	1.71	.325	.91	1.83	.264	.77	1.93	.216	.64	2.01	.176	.53
3"	3.500	2.51	.479	1.97	2.70	.389	1.65	2.86	.318	1.38	2.97	.260	1.15
4"	4.500	3.23	.616	3.27	3.47	.500	2.73	3.68	.409	2.28	3.81	.333	1.89
5.375"	5.375	3.86	.736	4.72	4.18	.597	4.00	4.39	.489	3.26	4.55	.398	2.75
5"	5.563	3.99	.762	4.99	4.29	.618	4.16	4.55	.506	3.49	4.71	.412	2.89
6"	6.625	4.76	.908	7.08	5.11	.736	5.92	5.42	.602	4.95	5.61	.491	4.10
7.125"	7.125	5.11	.976	8.30	5.49	.792	6.90	5.82	.648	5.78	6.04	.528	4.82
8"	8.625	6.19	1.182	11.99	6.65	.958	10.03	7.05	.784	8.39	7.31	.639	6.96
10"	10.750	7.72	1.473	18.63	8.29	1.194	15.57	8.79	.977	13.03	9.11	.796	10.81
12"	12.750	9.15	1.747	26.21	9.83	1.417	21.90	10.43	1.159	18.33	10.81	.944	15.21
13.38"	13.380	9.58	1.832	29.27	10.32	1.486	24.45	10.87	1.216	20.40	11.34	.991	16.99
14"	14.000	10.05	1.918	32.00	10.80	1.556	26.40	11.38	1.273	22.09	11.86	1.037	18.34
16"	16.000	11.48	2.192	41.80	12.34	1.778	34.48	13.00	1.455	28.95	13.56	1.183	23.95
18"	18.000	12.92	2.466	53.00	13.88	2.000	44.28	14.63	1.636	37.05	15.25	1.333	30.30
20"	20.000	14.35	2.740	65.45	15.42	2.222	54.66	16.26	1.818	45.75	16.95	1.481	37.32
22"	22.000	—	—	—	16.96	2.444	66.15	17.88	2.000	55.35	18.64	1.630	45.18
24"	24.000	—	—	—	18.50	2.666	78.70	19.50	2.182	65.87	20.33	1.778	54.67
26"	26.000	—	—	—	20.05	2.888	92.36	21.13	2.364	77.32	22.03	1.926	64.20
28"	28.000	—	—	—	—	—	—	22.75	2.545	89.64	23.72	2.074	74.40
30"	30.000	—	—	—	—	—	—	24.38	2.727	102.91	25.42	2.222	85.41
800mm	31.496	—	—	—	—	—	—	—	—	—	26.67	2.34	94.50
32"	32.000	—	—	—	—	—	—	—	—	—	27.11	2.370	97.20

ALL WEIGHTS ARE CALCULATED IN ACCORDANCE WITH PPI-TR-7.

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Calculation of the pressure-drop in a system pumping water and fluids of like viscosity are based on the Williams and Hazen formula.

$$\Delta P_f = \frac{453 \times Q^{1.85}}{C^{1.85} \times d^{4.86}}$$

POLYPIPE INDUSTRIES DESIGN & ENGR. GUIDE (6)

FLUID FLOW

Where ΔP_f – friction pressure loss per 100 feet, psi
C – pipe coefficient
Q – flow rate, gpm
d – inside diameter of pipe, inches

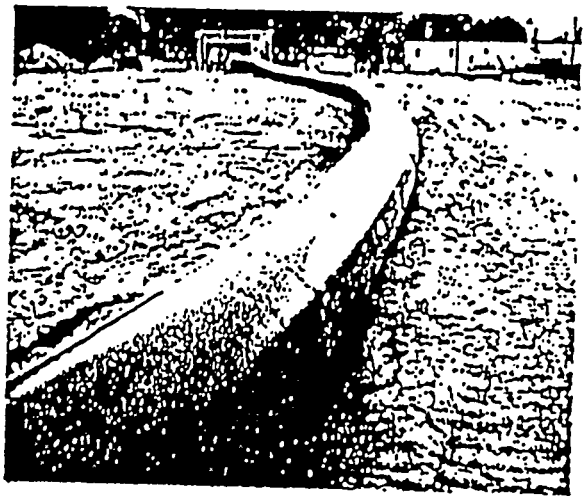
Coefficient C for POLY PIPE is recommended as 150. There are various quoted values for the pipe coefficient for polyethylene ranging from 150 to 160. However, Plastic Pipe Institute Technical Report Number 14, Water Flow Characteristic of Thermoplastic Pipe states "The value of C = 150 for thermoplastic pipe is based on engineering measurement made with new and used thermoplastic pipe made in several laboratories. The test results ranged from C = 155 to C = 165 with the average above C = 160. No significant differences were observed between new and used pipe. Thus, the value of C = 150 has a conservative bias." The designer may want to refer to Fig. 10 for some C factor references.

The Williams and Hazen formula can be used to calculate any one of the variables; volume flow rate, flow velocity, required pipe diameters or friction pressure loss. The flow chart 12 is based on the Williams and Hazen formula and can be used to obtain the required information for a system design.

It should be noted the pipe sizes quoted in the flow charts are minimum bore sizes only and the specific outside diameter of the pipe can be found from the pipe dimension chart on page A-9.

Fig. 10
William and Hazen Coefficients for Flow in various pipes

Pipe Description	"C" Value	Mannings n*
Poly Pipe	150	
Very smooth and straight, steel, glass	130-140	0.009-0.013
Smooth wood and wood stave	120	0.010-0.014
New rivited steel, cast iron	110	0.011-0.015
Old cast iron	95	
Old pipes in bad condition	60-80	
Small pipes, badly corroded	40-50	



* from King's Handbook, pg. 182, . . .

Handbook of Hydraulics
Horace King
Third Edition.
McGraw-Hill, 1939

E-3

Golder Associates

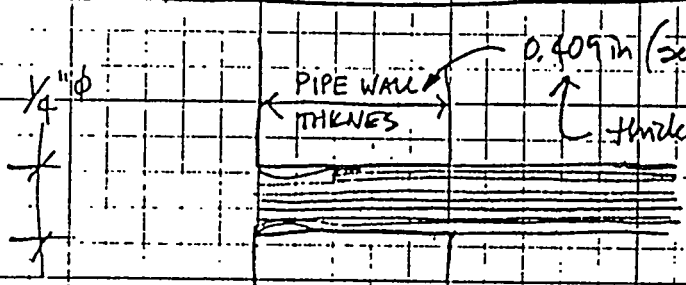
SUBJECT NDO - Primary Leachate Collection System

Job No. 893-1186 Ref.

Made by M. BROWN Checked FSS Reviewed RLV

Date 1/31/90 Sheet 44 of 48 20

COMPUTE NO. OF HOLES AND/OR SLOTS PER FOOT REQ'D IN LEACHATE COLLECTION PIPE



C = 0.82 from Kings Handbook

$Q = Ca\sqrt{2gh}$ (see attached page)

a for 1/4" ϕ hole $= \frac{\pi D^2}{4} = \frac{\pi\left(\frac{1/4}{12}\right)^2}{4} = 0.000341 \text{ ft}^2$ ✓

If h = 1 ft ← reasonable assumption – op's layer slightly more than half saturated

$Q = 0.82(.000341)(\sqrt{2 \times 32.2 \times 1}) = 0.0022 \text{ cfs/hole}$ ✓

25yr inflow max. = 0.62 gpm/ft of width ✓

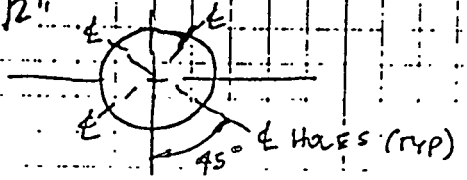
$0.62 \frac{\text{gpm}}{\text{ft}} \div 448.5 \frac{\text{gpm}}{\text{cfs}} = 0.0014 \text{ cfs/ft}$ ✓

Geonet Capacity 2 gpm/ft → 0.0045 cfs/ft ✓

Set criteria such that drain holes have capacity of geonet ✓

$\text{No. Holes} = \frac{0.0045 \text{ cfs/ft}}{0.0022 \text{ cfs/hole}} = 2 \frac{\text{holes}}{\text{ft}}$ ← MIN. REQ'D 1/4" HOLES. ✓

SINCE HOLES ARE INEXPENSIVE AND PIPE MIGHT ROTATE OR CLOG. ✓
PROVIDE 4 holes/ft, 4 holes every 12"



From: Handbook of Hydraulics
by Horace King.
Third Edition.
1934

$Q = Ca\sqrt{2gh}$

$C = 0.82$

if

C.3-19

its length, about $\frac{1}{2}d$, such that the jet will not touch the sides of the tube. The following are average coefficients:

$C = 0.51, \quad C_v = 0.98 \quad C_c = 0.52$

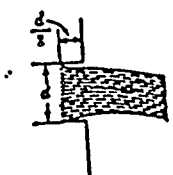


Fig. 30.—Borda's mouthpiece.

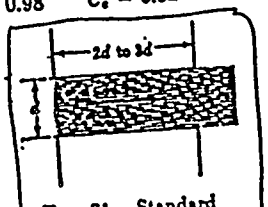


Fig. 31.—Standard short tube.

Standard Short Tube.—A cylindrical tube having a length of 2 to 3 diameters with the inner end flush with a flat wall so as to form a sharp-cornered entrance is called a standard short tube. In such tubes (Fig. 31) the issuing jet first contracts and then expands, filling the tube. The coefficient of contraction is considered unity. The coefficient of discharge varies from 0.78 to 0.83. The mean value generally used is

$C = 0.82$

Short tubes projecting inward, as shown in Fig. 32, have coefficients of discharge varying from 0.72 to 0.80. The average value is

$C = 0.75$

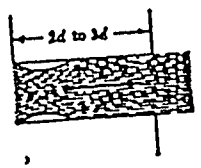


Fig. 32.—Short tube projecting inward.

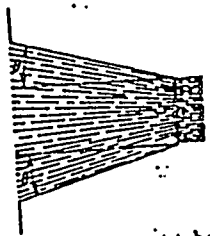


Fig. 33.—Convergent tube with sharp corner at entrance.

Convergent short tubes are frustums of cones, as shown in Figs. 33 and 34. Figure 33 has the larger base flush with a flat wall forming a sharp-cornered entrance. Figure 34 has the entrance to the tube slightly rounded. The sides of the tube make an angle θ with the axis of the cone.

Experiments for these tubes give conflicting results. Fair average values are given by Unwin[1] as follows:

Angle θ	0°	5¼°	11¼°	22½°	45°
C_c for Fig. 33	0.83	0.94	0.92	0.85	
C_c for Fig. 34	0.97	0.95	0.92	0.88	0.75

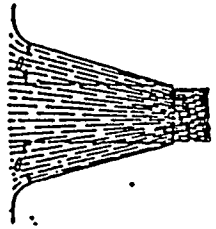


Fig. 34.—Convergent tube with rounded corner at entrance.

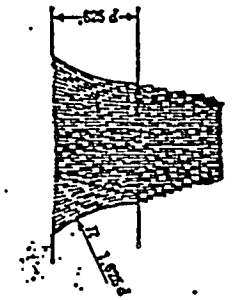


Fig. 35.—Converging bell-mouthed orifice.

Converging Bell-mouthed Orifice.—If the surface of the opening is rounded to conform to the shape of the contracted jet (Fig. 35), C approaches unity. The following are coefficients by Weisbach[2] for $d = 0.033$ foot. Other experiments indicate that these results hold approximately for larger orifices.

h in feet	0.066	1.640	11.480	55.770	337.930
C	0.959	0.967	0.975	0.994	0.994

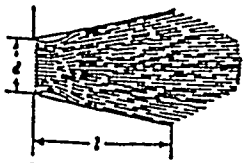


Fig. 36.—Diverging tube with sharp corner at entrance.

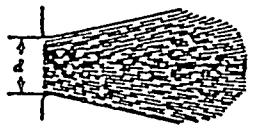


Fig. 37.—Diverging tube with rounded corner at entrance.

Diverging conical tubes are shown in Figs. 36 and 37. The coefficient of discharge varies with the angle of divergence and length of tube. Experiments by Venturi showed discharge to be a maximum with $l = 9d$ and angle of divergence equal to 5°.

[1] Unwin, W. C., "Treatise on Hydraulics," p. 89.

[2] Weisbach, Julius, "Ingenieur und Maschinen-Mechanik," p. 969 (ed. 1855).

WHC-SD-W025-FDR-001, Rev. 1

Fig 12 of 20

Handbook of Hydraulics, Horace King, 3rd Ed, Michigan, 1939.

7. The discharge of a circular orifice under any given head is the same, whether the jet be horizontal, vertical, or at any intermediate angle.

It is probable that with proper modification the above comments will apply to square or rectangular orifices. The approximate coefficient, 0.60 for orifices above 2½ inches in diameter and for heads greater than 17 inches, can be easily remembered.

A table of coefficients of discharge for rectangular orifices has been prepared by Fanning[1] from experiments by Michelotti, Bossut, Rennie, Castel, Lespinasse, and Ellis. Fanning's results to three decimal places are given in Table 28 (p. 71). The coefficients given are for orifices 1 foot wide and from 0.125 foot to 4 feet high under heads of from 0.3 foot to 50 feet.

Table 29 (p. 72), prepared by Bovey[2] from experiments on orifices of different shapes, having the same area as a circle ½ inch in diameter, gives the effect of shape of opening on the coefficient of discharge. It does not necessarily follow that a similar relation will hold for orifices of larger areas.

Orifices with Suppressed Contractions.—Experimental data for determining coefficients of discharge for orifices with contractions wholly or partially suppressed do not cover a wide range of conditions. Table 30 (p. 73) has been prepared from results obtained by Smith[3] from experiments by Lebro[illegible]. Though the orifices experimented upon were small, they should form a guide for selecting coefficients for larger orifices. It is probable that coefficients of discharge for orifices with contractions suppressed will decrease slightly as the size of the opening increases the same as for sharp-edged orifices. In Table 30 suppressed contraction means that the side of the channel coincides with the edge of the orifice, and partial suppressed contraction means that the distance between the side of the channel and edge of the orifice is 0.066 foot.

Williams[4] has determined coefficients for rectangular orifices 30 inches wide, with heights of opening of 2 to 4 inches and heads of 1 to 2.5 feet. The average coefficient of discharge with full contraction at the top and contractions suppressed on the two sides and bottom was 0.607. With contraction

[1] FANNING, J. T., "Water Supply Engineering," pp. 205–206.
[2] BOVEY, HENRY T., "Hydraulics," p. 40.
[3] SMITH, HAMILTON, JR., "Hydraulics," pp. 65–67, 1886.
[4] WILLIAMS, GARDNER S., Experiments performed at University of Michigan in 1923. Detailed results to be published.

suppressed on the sides and bottom and with a beveled top edge the average coefficient was 0.776. With full contraction on the top, one side a sharp edge 6 inches from the side of the channel, and contractions suppressed on one side and the bottom $C = 0.611$. With the latter arrangement but with beveled top $C = 0.755$.

Submerged and Partially Submerged Orifices.—The discharge through a submerged orifice (Fig. 24) is given by the formula

$$Q = Ca\sqrt{2gh} \qquad (20)$$

h being the differences in elevation of water surfaces, a the area of opening, and C the coefficient of discharge.

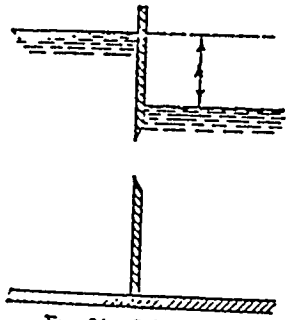


FIG. 24.—Submerged orifice.

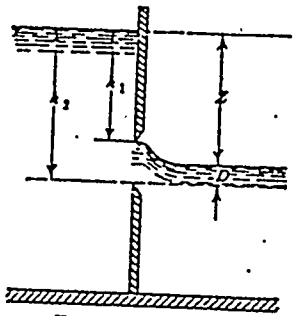


FIG. 25.—Partially submerged orifice.

The few experiments for determining C for submerged orifices that are available indicate that the value of the coefficient is not greatly affected by submergence. Table 31 (p. 74) gives values of C for sharp-edged submerged orifices, and Table 32 (p. 74) contains values of C for a submerged orifice 1 foot square with rounded corners.

Figure 25 represents a partially submerged orifice. Assume a rectangular opening with the top and bottom edges h_1 and h_2 respectively, below the upstream water surface. D, Z, and L are, respectively, depth of submergence, difference in elevation of water surfaces, and width of opening. The theoretical discharge formula for this orifice is interesting because of its relation to the submerged weir. The discharge Q is the combined discharge of the upper portion Q_1, which has free

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C.3-20

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Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 7

Project Title: NON DRAG OFF

Project No. 893-1186.060

Calculation Title: Tank Secondary Containment System Concrete Structural Design B.25

	Name	Date
Prepared By:	M. Brown	1/16/90
Checked By:	Frank S. Shuri	2-7-90
Approved By:	FRANK CSHURI	2-8-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT NDO Tank Secondary Containment System

Job No. 893-1186.060 Ref. | Made by M. Brown | Checked FSS | Reviewed | Date 1/16/90 | Sheet 1 of 7

OBJECTIVE: Design concrete floor slab and walls for TSCS concrete sump.

LOADINGS: Active earthpressure, tank weight

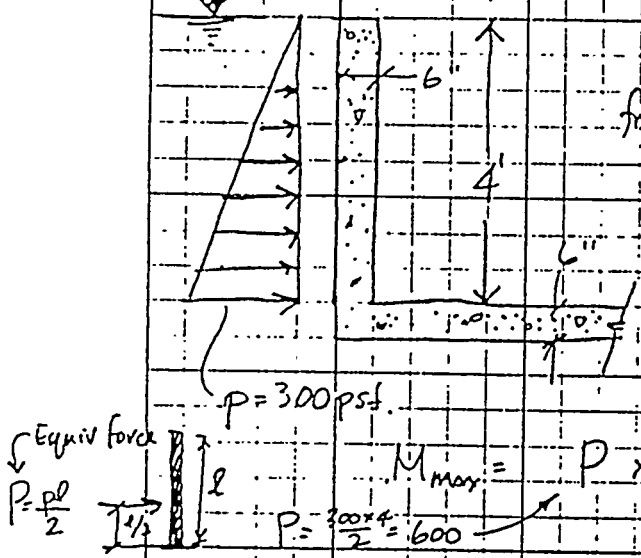
Assumption of high groundwater is very conservative; actual water table is ~200' below surface.

Assume: $\gamma_d = 100$ pcf (dry density), $\phi = 30°$ (typical of onsite soils)

from Terzaghi & Peck (attached) $K_a = \tan^2(45° - \frac{\phi}{2}) = 0.33$ ✓

$p = 0.33(4')(100-62) + \gamma_w 4'$

$p = 50 + 250 = 300$ psf ✓



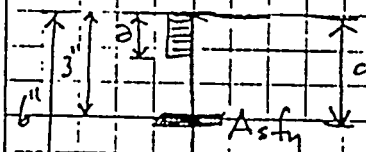
Equiv force $P = \frac{pl}{2}$, $P = \frac{300 \times 4}{2} = 600$

$M_{max} = \frac{P \times 4}{3} = 800 \frac{ft\text{-}lbs}{ft} = 0.8 \frac{k\text{-}ft}{ft}$.

Treat all load as Live load

$M_u = 0.8 \times 1.7 = 1.36$ k-ft ✓

Ultimate moment, ultimate strength design. (See code attached)



assume $a = 1"$ ✓

use ultimate strength design - see Winter, Nilson attached.

$M_u = A_s f_y (d - \frac{a}{2})$ ✓

$A_s = \frac{M_u}{f_y (d - \frac{a}{2})} = \frac{1.36 \text{ k-ft} \times 12}{60 \frac{k}{in} (3 - \frac{1}{2}) in} = 0.109 \text{ in}^2/\text{ft}$ ✓

#4 bars @ 8" provide 0.29 in²/ft ✓ See attached table.

Conservative practice

Golder Associates

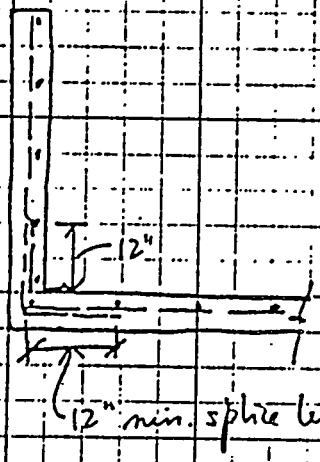
SUBJECT	NDO Tank Secondary Containment System		
Job No. 893-1186.060 Ref.	Made by M. Brown	Checked [illegible]	Reviewed [illegible]
Date 1/16/90	Sheet 2 of 7		

checking assumed a: $\Sigma F_H = 0 \Rightarrow a \times 0.85 f'_c \times 12''/ft = F_y A_s$

$a = \frac{F_y A_s}{0.85 f'_c \times 12} = \frac{60\ ksi \times 0.2\ in^2}{0.85 \times 3\frac{1}{2}\ ksi \times 12} = 0.56'' < 1'' \therefore OK$

Provide Steel Reinforcement as follows:

Vertical Wall Steel #4's @ 8"
Horizontal Wall Steel #4's @ 12"
Floor Slab Steel #4's @ 18" E.W.
} at center of 6" THK slab.



Weight of 1/4" THK Steel Plate:

$\frac{0.25}{12} \times 490\ lb/ft^3 = 10.2\ lb/ft^2$ ← steel density

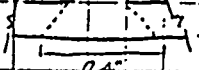
From dwgs:
tank diam. = 8'
side shell length = 25'

Check load under tank footings -

Tank weight = water + steel : water = $1000\ gal \times 8.5\ lb/gal \approx 85000\ lbs.$

steel = area $\times$ $10\ lb/ft^2 = \{(\pi D \times 25) + \pi r^2\} 10 = 7400\ lbs.$ ← 8'

weight = 93 kips

Area = 2 footings  Length = 10' Area per footing = $2 \times 10 = 20\ ft^2$

TOTAL AREA = $40\ ft^2$

foundation pressure = 2.32 ksf

← no problem under this. *

*SEE FURTHER DISCUSSION - NEXT PAGE

SAIC Investigation Report Recommended 250075 Recommended Rec

Golder Associates

SUBJECT NDO Tank Sec. Containment System
Job No. 893-1186.060 Ref.
Made by M. Brown Checked FSS Reviewed
Date 2/7/90 Sheet 3 of 7

Computation of soil bearing pressure on previous page shows that even if surrounding slab were not present the bearing pressure beneath the tank footing would be allowable. This is a limiting condition. Actual condition will involve moderate slab reinforcement which will spread load somewhat until slab cracks (crack occurance). The deflection resulting from cracking will reduce the slab moment. All of this is expected to occur within reasonable deflection limits, and therefore slab moments.

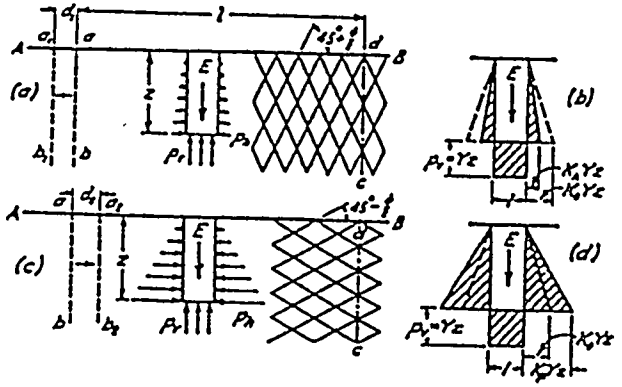


Fig. 27.1. (*a* and *b*) Diagrams illustrating active Rankine state in semi-infinite mass of sand. (*c* and *d*) Corresponding diagrams for passive Rankine state.

Since the vertical principal stress p_v in the mass of sand shown in Fig. 27.1*a* can be either the major or the minor principal stress, the ratio $K = p_h/p_v$ can assume any value between the limits,

$$K_A = \frac{p_h}{p_v} = \frac{1}{N_\phi} = \tan^2\left(45° - \frac{\phi}{2}\right) \tag{27.2}$$

and

$$K_P = \frac{p_h}{p_v} = N_\phi = \tan^2\left(45° + \frac{\phi}{2}\right) \tag{27.3}$$

After a mass of sand has been deposited by either a natural or an artificial process, K has a value K_0 intermediate between K_A and K_P, and

$$p_h = K_0 p_v \tag{27.4}$$

wherein K_0 is an empirical coefficient known as the *coefficient of earth pressure at rest*. Its value depends on the relative density of the sand and the process by which the deposit was formed. If this process did not involve artificial compaction by tamping, the value of K_0 ranges from about 0.40 for dense sand to 0.50 for loose sand. Tamping in layers may increase the value to about 0.8.

In order to change the value of K for a mass of sand from K_0 to some other value, it is necessary to give the entire mass an opportunity either to stretch or to be compressed in a horizontal direction. Since the weight of sand above any horizontal section remains unchanged, the vertical pressure p_v is unaltered. The horizontal pressure $p_h = Kp_v$, however, decreases if the mass stretches and increases if it compresses.

As the mass stretches, any two vertical sections such as *ab* and *cd* move apart, and the value of K decreases until it becomes equal to K_A (Eq. 27.2). The sand is then in what is known as the *active Rankine state*. In this state the intensity of the horizontal pressure at any depth z is equal to

$$p_h = K_A p_v = K_A \gamma z = \gamma z \frac{1}{N_\phi} \tag{27.5}$$

in which K_A is called the *coefficient of active earth pressure*. The distribution of pressure over the sides and base of an element such as E is shown in Fig. 27.1*b*. Further stretching of the mass has no effect on p_h (Eq. 27.5), but sliding occurs along two sets of plane surfaces as indicated on the right-hand side of Fig. 27.1*a*. According to Eq. 16.3, such surfaces of sliding intersect the direction of the minor principal stress at the angle $45° + \phi/2$. Since the minor principal stresses in the active Rankine state are horizontal, the shear planes rise at an angle of $45° + \phi/2$ with the horizontal. The pattern formed by the traces of the shear planes on a vertical section parallel to the direction of stretching is known as the *shear pattern*.

A horizontal compression of the entire mass of sand causes *ab* to move toward *cd*, as shown in Fig. 27.1*c*. As a consequence, the ratio $K = p_h/p_v$ increases. As soon as K becomes equal to K_P (Eq. 27.3) the sand is said to be in the *passive Rankine state*. At any depth z the horizontal pressure is

$$p_h = K_P p_v = K_P \gamma z = \gamma z N_\phi \tag{27.6}$$

in which K_P is the *coefficient of passive earth pressure*. Since the minor principal stress in the passive Rankine state is vertical, the surfaces of sliding rise at an angle of $45° - \phi/2$ with the horizontal, as shown in Fig. 27.1*c*.

The active and the passive Rankine states constitute the two limiting states for the equilibrium of the sand. Every intermediate state, including the state of rest, is referred to as a *state of elastic equilibrium*.

Local States of Plastic Equilibrium

The Rankine states illustrated by Fig. 27.1 were produced by uniformly stretching or compressing every part of a semi-infinite mass of sand. They are known as *general states of plastic equilibrium*. However, in a stratum of real sand, no general state of equilibrium can be produced except by a geological process such as the horizontal compression by tectonic forces of the entire rock foundation of the sand strata. Local events, such as the yielding of a retaining wall, cannot produce a radical change in the state of stress in the sand except in the immediate vicinity

Soil Mechanics in Engr. Practice Terzaghi & Peck, 2nd Ed, Wiley, 1968
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α_m = average value of α for all beams on edges of a panel

β = ratio of clear spans in long to short direction of two-way slabs

β_s = ratio of length of continuous edges to total perimeter of a slab panel

ϕ = strength reduction factor. See Section 9.3

9.1 – General

9.1.1 – Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this code

9.1.2 – Members also shall meet all other requirements of this code to insure adequate performance at service load levels.

9.2 – Required strength

9.2.1 – Required strength U to resist dead load D and live load L shall be at least equal to

$$U = 1.4D + 1.7L \quad (9\text{-}1)$$

9.2.2 – If resistance to structural effects of a specified wind load W are included in design, the following combinations of D, L, and W shall be investigated to determine the greatest required strength U

$$U = 0.75(1.4D + 1.7L + 1.7W) \quad (9\text{-}2)$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W \quad (9\text{-}3)$$

but for any combination of D, L, and W, required strength U shall not be less than Eq. (9-1).

9.2.3 – If resistance to specified earthquake loads or forces E are included in design, load combinations of Section 9.2.2 shall apply, except that 1.1 E shall be substituted for W.

9.2.4 – If resistance to lateral earth pressure H is included in design, required strength U shall be at least equal to

$$U = 1.4D + 1.7L + 1.7H \quad (9\text{-}4)$$

except that where D or L reduce the effect of H, $0.9D$ shall be substituted for $1.4D$ and zero value of L shall be used to determine the greatest required strength U. For any combination of D, L, and H, required strength U shall not be less than Eq. (9-1).

9.2.5 – If resistance to lateral liquid pressure F is included in design, load combinations of Section 9.2.4 shall apply, except that $1.4F$ shall be substituted for $1.7H$. Vertical liquid pressure shall be considered as dead load D, with due regard to variation in liquid depth.

9.2.6 – If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

9.2.7 – Where structural effects T of differential settlement, creep, shrinkage, or temperature change may be significant in design, required strength U shall be at least equal to

$$U = 0.75(1.4D + 1.4T + 1.7L) \quad (9\text{-}5)$$

but required strength U shall not be less than

$$U = 1.4(D + T) \quad (9\text{-}6)$$

Estimations of differential settlement, creep, shrinkage, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.3 – Design strength

9.3.1 – Design strength provided by a member or cross section in terms of load, moment, shear, or stress shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by a strength reduction factor ϕ.

9.3.2 – Strength reduction factor ϕ shall be as follows:

(a) Flexure, with or without axial tension . . 0.90

(b) Axial tension 0.90

(c) Axial compression, with or without flexure:
Members with spiral reinforcement conforming to Section 10.9.3. . . . . 0.75
Other reinforced members 0.70

except that for low values of axial load, ϕ may be increased in accordance with the following:

For members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ to zero.

For other reinforced members, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ or ϕP_b, whichever is smaller, to zero.

(d) Shear and torsion 0.85

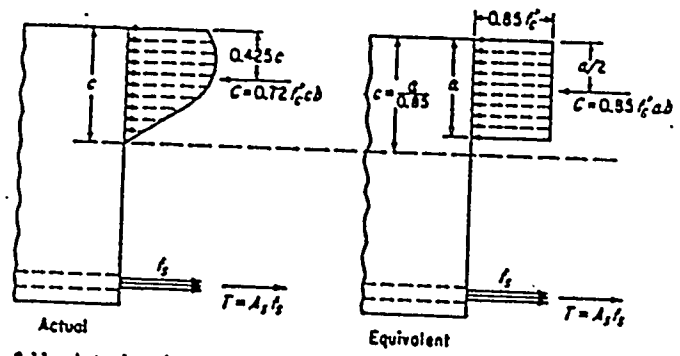


Fig. 2.11 Actual and equivalent rectangular stress distributions at ultimate load (for f'_c less than 4000 psi).

country was first proposed by C. S. Whitney and subsequently elaborated and checked experimentally by others (see, e.g., Ref. 2.4). For concrete with cylinder strength f'_c not exceeding 4000 psi, an actual stress distribution and the fictitious equivalent distribution are shown in Fig. 2.11.

It is seen that the actual distribution is replaced by a fictitious equivalent one of simple rectangular outline. The intensity of this uniform and constant equivalent compression stress is $0.85f'_c$, and it is thought of as acting over a part of the section of depth $a = 0.85c$, where c is the distance to the actual neutral axis. It is in no way maintained that the compression stresses are actually distributed in this most unlikely manner. It is maintained, however that this equivalent distribution gives the answers just derived for the actual situation. Of this one can easily convince oneself. In a rectangular beam of width b the equivalent uniform compression stress $0.85f'_c$ acts on an area ab, so that the total compression force is

$$C = 0.85f'_cab \tag{2.30}$$

If one expresses the depth a of the rectangular stress block in terms of the distance c to the neutral axis as assumed, that is, $a = 0.85c$, one obtains for the compression force $C = 0.85f'_c \times 0.85c \times b = 0.72f'_cb$, which is the same as Eq. (2.21) for $\alpha = 0.72$. Further, since the centroid of a rectangle is at middepth, the distance from the compression edge to C is evidently $a/2$. Again expressing a in terms of c as above, one obtains the distance to the compression resultant as $0.5 \times 0.85c = 0.425c$, which is the correct distance for $\beta = 0.425$.

Since the chosen equivalent rectangular stress distribution satisfies the stated requirements (1) and (2), it will give exactly the same answers as were derived previously. This is so for f'_c not exceeding 4000 psi because, in this range of concrete strengths, the parameters α and β are constants

equal, respectively, to 0.72 and 0.425. For concretes of higher strength it has been stated that α and β decrease because of the greater brittleness of such concretes. In the equivalent rectangular stress distribution, this is reflected in the following stipulation:

$$a/c = k_1 = 0.85 \text{ for } f'_c \leq 4000 \text{ psi and decreases by 0.05 for every 1000 psi above 4000} \tag{2.31}$$

This equivalent rectangular stress distribution can be used for deriving the equations which have been developed in the previous section. The failure criteria, of course, are the same as before: yielding of the steel at $f_s = f_y$ or crushing of the concrete at $\epsilon_c = 0.003$. More important, however, because the rectangular stress block is easily visualized and its geometrical properties are extremely simple, many calculations are conveniently carried out directly without reference to formally derived equations.

Example Using the equivalent rectangular stress distribution, calculate directly the ultimate strength of the previously analyzed beam.

The distribution of stresses, internal forces, and strains is as shown in Fig. 2.12. The depth a of the equivalent stress block is found from the equilibrium condition $C = T$. Hence $ab \times 0.85f'_c = A_sf_y$ or $a \times 10 \times 2550 = 2.35 \times 40{,}000$, from which $a = 3.70$ in. The distance to the neutral axis, by the definition of the rectangular stress block, is $c = a/0.85 = 3.70/0.85 = 4.35$ in. One way of determining whether the strength of the beam is governed by yielding of the steel or by crushing of the concrete is to calculate whether the concrete strain in the top fiber is smaller or larger than 0.003 at the instant the steel starts yielding. At that point the strain in the steel is $\epsilon_s = f_s/E_s = 40{,}000/29{,}000{,}000 = 0.00138$. From the similar triangles of the strain distribution, $\epsilon_c = \epsilon_sc/(d - c) = 0.00138(4.35/18.65) = 0.00032$. This value is smaller than the assumed crushing strain of the concrete, and hence failure will be initiated

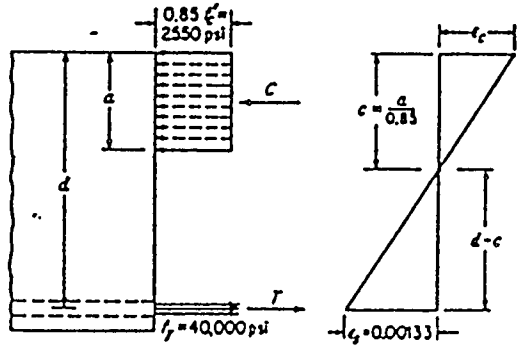


Fig. 2.12

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Design of Concrete Structures, Winter, Urquhart, O'Rourke, Nilson, McGraw Hill 8th Ed. 1964

Design of Concrete Structures
Winter, Urquhart, O'Rourke, Nilson
McGraw Hill, 7th Ed. 1964

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appendix B

TABLES AND DIAGRAMS

Table 1 Designations, areas, perimeters, and weights of standard bars

Bar No.*	Diameter, in.	Cross-sectional area, in.²	Perimeter, in.	Unit weight per foot, lb
2	⅛ = 0.250	0.05	0.79	0.167
3	⅜ = 0.375	0.11	1.18	0.376
4	½ = 0.500	0.20	1.57	0.668
5	⅝ = 0.625	0.31	1.96	1.043
6	¾ = 0.750	0.44	2.36	1.502
7	⅞ = 0.875	0.60	2.75	2.044
8	1 = 1.000	0.79	3.14	2.670
9	1⅛ = 1.128†	1.00	3.54	3.400
10	1¼ = 1.270†	1.27	3.99	4.303
11	1⅜ = 1.410†	1.56	4.43	5.313
14S	1¾ = 1.693†	2.25	5.32	7.650
18S	2¼ = 2.257†	4.00	7.09	13.600

* Based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar. Bar No. 2 is available in plain rounds only. All others are available in deformed rounds.

† Approximate to the nearest ⅛ in.

Table 2 Areas of groups of standard bars in square inches

Bar No.	Number of bars												
	2	3	4	5	6	7	8	9	10	11	12	13	14
4	0.39	0.58	0.78	0.98	1.18	1.37	1.57	1.77	1.96	2.16	2.36	2.55	2.75
5	0.61	0.91	1.23	1.53	1.84	2.15	2.45	2.76	3.07	3.37	3.68	3.99	4.30
6	0.88	1.32	1.77	2.21	2.65	3.09	3.53	3.98	4.42	4.86	5.30	5.74	6.19
7	1.20	1.80	2.41	3.01	3.61	4.21	4.81	5.41	6.01	6.61	7.22	7.82	8.42
8	1.57	2.35	3.14	3.93	4.71	5.50	6.28	7.07	7.85	8.64	9.43	10.21	11.00
9	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00
10	2.53	3.79	5.06	6.33	7.59	8.86	10.12	11.39	12.66	13.92	15.19	16.45	17.72
11	3.12	4.68	6.25	7.81	9.37	10.94	12.50	14.06	15.62	17.19	18.75	20.31	21.87
14S	4.50	6.75	9.00	11.25	13.50	15.75	18.00	20.25	22.50	24.75	27.00	29.25	31.50
18S	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00	52.00	56.00

Table 3 Perimeters of groups of standard bars in inches

Bar No.	Number of bars												
	2	3	4	5	6	7	8	9	10	11	12	13	14
4	3.1	4.7	6.2	7.8	9.4	11.0	12.6	14.1	15.7	17.3	18.8	20.4	22.0
5	3.9	5.9	7.8	9.8	11.8	13.7	15.7	17.7	19.5	21.6	23.6	25.5	27.5
6	4.7	7.1	9.4	11.8	14.1	16.5	18.8	21.2	23.6	25.9	28.3	30.6	33.0
7	5.5	8.2	11.0	12.7	16.5	19.2	22.0	24.7	27.5	30.2	33.0	35.7	38.5
8	6.3	9.4	12.6	15.7	18.9	22.0	25.1	28.3	31.4	34.6	37.7	40.9	44.0
9	7.1	10.6	14.2	17.7	21.3	24.8	28.4	31.9	35.4	39.0	42.5	46.0	49.6
10	8.0	12.0	16.0	20.0	23.9	27.9	31.9	35.9	39.9	43.9	47.9	51.9	55.9
11	8.9	13.3	17.7	22.2	26.6	31.0	35.4	39.9	44.3	48.7	53.2	57.6	62.0
14S	10.6	16.0	21.3	26.6	31.9	37.2	42.6	47.9	53.2	58.5	63.8	69.2	74.5
18S	14.2	21.3	28.4	35.5	42.5	49.6	56.7	63.8	70.9	78.0	85.1	93.2	100.3

Table 4 Areas of bars in slabs in square inches per foot

Spacing, in.	Bar No.								
	3	4	5	6	7	8	9	10	11
3	0.44	0.78	1.23	1.77	2.40	3.14	4.00	5.06	6.25
3½	0.38	0.67	1.05	1.51	2.06	2.69	3.43	4.34	5.36
4	0.33	0.59	0.92	1.32	1.80	2.36	3.00	3.80	4.68
4½	0.29	0.52	0.82	1.18	1.60	2.09	2.67	3.37	4.17
5	0.26	0.47	0.74	1.06	1.44	1.88	2.40	3.04	3.75
5½	0.24	0.43	0.67	0.96	1.31	1.71	2.18	2.76	3.41
6	0.22	0.39	0.61	0.88	1.20	1.57	2.00	2.53	3.12
6½	0.20	0.36	0.57	0.82	1.11	1.45	1.85	2.34	2.89
7	0.19	0.34	0.53	0.76	1.03	1.35	1.71	2.17	2.68
7½	0.18	0.31	0.49	0.71	0.96	1.26	1.60	2.02	2.50
8	0.17	0.29	0.46	0.66	0.90	1.18	1.50	1.89	2.34
9	0.15	0.26	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	0.13	0.24	0.37	0.53	0.72	0.94	1.20	1.52	1.87
12	0.11	0.20	0.31	0.44	0.60	0.78	1.00	1.27	1.56

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Golder Associates Inc.

Calculation Cover Sheet

Sheet Ø of 10

Project Title: WHC NON DRAG :OFF

Project No. 893-1186

Calculation Title: Secondary Leachate Collection Sump - B.13
Leak Detection System.

	Name	Date
Prepared By:	M. Brown	1/22/90
Checked By:	Frank S. Shuri	2-7-90
Approved By:	FRANK CORSER	2-8-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT NDO – SECONDARY SUMP		
Job No. 893-1186 Ref.	Made by M. BROWN Checked Reviewed	Date 1/22/90 Sheet 1 of 10

Check Accuracy of Secondary Sump Monitoring

Minimum water depth = 6"

Area at depth = 6" = 30 ft² (from spreadsheet comps by Barkesfield, et al) Attached.
(min. depth = 6" because pump cannot remove more)

Maximum accuracy of transducer = 0.5% of full scale (AMETEK)
See attached manuf. info.

Full Scale = 6 psi = 14' ∴ Sensitivity = 14 × 0.005 = 0.07 ft. ✓

Volume of space necessary to raise leachate level from 0.5 ft to 0.57 ft = 0.07 × 30 = 2.1 ft³

Since this is part drainage gravel e = 0.40 (see attached)

leachate volume = 2.1 ft³ × e = 0.84 ft³ = 6.3 gal.

∴ the accuracy = 6.3 gallons

Proposed EPA leak detection rule [Reg. for Haz Waste ... pg. 122]

ALR requires sensitivity of 5 gal/acre/day × 3 acres = 15 gal/day
(actually 5 to 20)

↑ Action Leakage Rate

6.3 < 15 gal/day ∴ OK

↑ AREA OF THE LINED WASTE DISPOSAL FACILITY, SEE 'HELP' MODEL COMP'S CALCS B.11 and B.21.

Elevation ft	Secondary Drainage Gravel Eff Porosity = 0.45					Primary Drainage Gravel Eff Porosity = 0.50					Primary Operations Layer Eff Porosity = 0.35					Free Water Pond Eff Porosity = 1.00					Total Primary Water Storage gallons
	Area sq ft	Increment Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increment Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increment Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increment Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	
0.00	0.00		0.00	0.00	0	0.00		0.00	0.0	0	0.0		0.00	0.0	0	0.00		0.00	0.0	0	0
0.25	8.6	1.1	1.1	0.5	4	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
0.50	30.0	4.8	5.9	2.7	20	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
0.75	62.1	11.5	17.4	7.8	59	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.00	107.3	21.2	38.6	17.4	130	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.25	171.0	34.8	73.4	33.0	247	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.50	245.3	52.0	125.4	56.4	422	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.75	326.3	71.5	196.9	88.6	663	14.5	1.8	1.8	0.9	7	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
2.00	456.2	97.8	294.7	132.6	992	28.6	5.4	7.2	3.6	27	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	7
2.25	187.1	80.4	375.1	168.8	1263	58.4	10.9	18.1	9.0	68	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	27
2.50	229.8	52.1	427.2	192.2	1438	77.0	16.9	35.0	17.5	131	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	68
2.75	281.8	64.0	491.2	221.0	1653	105.1	22.8	57.8	28.9	216	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	131
3.00	336.4	77.3	568.4	255.8	1913	135.2	30.0	87.8	43.9	328	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	216
3.25	304.8	80.2	648.6	291.9	2183	247.8	47.9	135.7	67.8	507	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	328
3.50	226.0	66.4	714.9	321.7	2406	347.5	74.4	210.1	105.0	786	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	507
3.75	168.0	49.3	764.2	343.9	2572	460.8	101.0	311.1	155.6	1164	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	786
4.00	0.0	21.0	785.2	353.3	2643	585.0	130.7	441.9	220.9	1653	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	1164
4.25	0.0	0.0	785.2	353.3	2643	750.3	166.9	608.8	304.4	2277	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	1653
4.50	0.0	0.0	785.2	353.3	2643	911.3	207.7	816.5	408.2	3054	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	2277
4.75	0.0	0.0	785.2	353.3	2643	1073.1	248.0	1064.5	532.3	3981	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	3054
5.00	0.0	0.0	785.2	353.3	2643	1240.8	289.2	1353.8	676.9	5063	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	3981
5.25	0.0	0.0	785.2	353.3	2643	1405.4	330.8	1684.5	842.3	6300	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	5063
5.50	0.0	0.0	785.2	353.3	2643	1900.0	413.2	2097.7	1048.9	7845	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	6300
5.75	0.0	0.0	785.2	353.3	2643	2100.0	500.0	2597.7	1298.9	9715	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	7845
6.00	0.0	0.0	785.2	353.3	2643	0.0	262.5	2860.2	1430.1	10697	2500.0	312.5	312.5	109.4	818	0.00	0.0	0.0	0.0	0	9715
6.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	3000.0	687.5	1000.0	350.0	2618	0.00	0.0	0.0	0.0	0	11515
6.50	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	3500.0	812.5	1812.5	634.4	4745	0.00	0.0	0.0	0.0	0	13315
6.75	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	4000.00	937.5	2750.0	962.5	7200	0.00	0.0	0.0	0.0	0	15442
7.00	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	4600.00	1075.0	3825.0	1338.8	10014	0.00	0.0	0.0	0.0	0	17897
7.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	5100.00	1212.5	5037.5	1763.1	13188	0.00	0.0	0.0	0.0	0	20711
7.50	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	5500.00	1325.0	6362.5	2226.9	16657	0.00	0.0	0.0	0.0	0	23885
7.75	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	6000.00	1437.5	7800.0	2730.0	20420	0.00	0.0	0.0	0.0	0	27354
8.00	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	6850.00	1606.3	9406.3	3292.2	24626	0.00	0.0	0.0	0.0	0	31118
8.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	11600.00	2306.3	11712.5	4099.4	30663	7600.00	950.0	950.0	950.0	7106	35323
8.50	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	13400.00	3125.0	14837.5	5193.1	38845	8500.00	2025.0	2975.0	2975.0	22253	48466
8.75	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	14300.00	3462.5	18300.0	6405.0	47909	9700.00	2287.5	5262.5	5262.5	39364	71795
9.00	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	16000.00	3787.5	22087.5	7730.6	57825	11000.00	2587.5	7850.0	7850.0	58718	97970
9.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	18900.00	4362.5	26450.0	9257.5	69246	12100.00	2887.5	10737.5	10737.5	80317	127240

MODEL 57 SN/SP SUBMERSIBLE LEVEL SENSORS

3.10

SPECIFICATIONS

(Millivolt output transducers, for use with 572 motor.)

RANGES

FEET OF WATER: 0/14, 0/35, 0/69, 0/138, 0/230, 0/340, 0/460, 0/690

PSI: 0/8, 0/15, 0/30, 0/60, 0/100, 0/150, 0/200, 0/300

TEMPERATURE RANGES

MAX. OPERATING: −40° to 80°C (−40° to 176°F)

COMPENSATED: 0° to 50°C (32° to 122°F)

TEMP. EFFECTS: ±2% output change for ±25°C temp. change within compensated range (for ranges over 30 psi)

ACCURACY: ±0.5% O.S. (including linearity, hysteresis and repeatability, B.S.L.)

IMPEDANCE:

Typical Excitation	10 ±5VDC
Input Impedance	8K ohms typical
Output Impedance	5K ohms typical

EXCITATION VOLTAGE: 16 VDC max; 10 VDC (regulated) typical, 2 VDC min. 571 motor provides 8 VDC excitation std.

OUTPUT SENSITIVITY: typically 5 mV/volt for 0 to 30 psi ranges. 10 mV/volt for 60 to 300 psi units

ELECTRICAL CONNECTION: Attached 4-wire, 20 ga. polyethylene shielded cable. Unspliced lengths to 5000 ft.

CABLE COLOR CODE: RED (+) excitation, WHITE (−) output, BLACK (−) excitation, GREEN (+) output

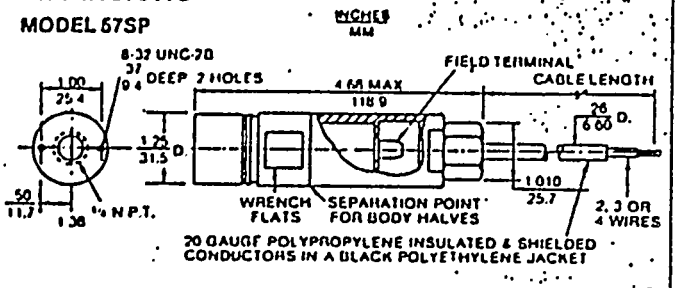
PROCESS CONNECTION: ¼" NPT, female (Model 57SP only)

CONSTRUCTION: 300 series stainless steel, neoprene, BUNA-N, high density polypropylene

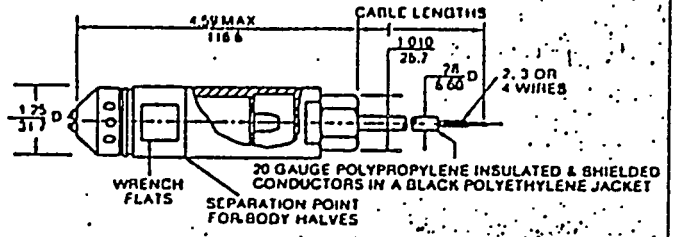
OTHER OUTPUTS: 4-20 ma, 1-5 VDC (with and without built-in voltage regulation). For complete specifications on these units contact your local representative or the factory.

DIMENSIONS

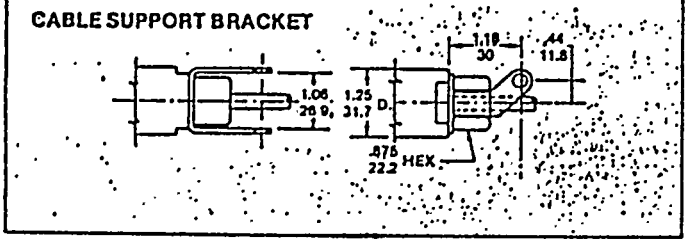
MODEL 57SP



MODEL 57SN



CABLE SUPPORT BRACKET



ORDERING INFORMATION

MODEL	RANGE	INPUT/ OUTPUT	WETTED PARTS	FILL FLUID	OPTIONS	CABLE LENGTH
57 SP 57 SN	ooo	•	•	•	••	(Specify in feet)

MODEL
57SP Transducer (¼" NPT Process Conn.)
57SN Transducer (Non-Clogging)

RANGE	M.V. OUTPUT
018 = 0-8 PSI	50 mV
018 = 0-15 PSI	50 mV
010 = 0-30 PSI	50 mV
005 = 0-60 PSI	100 mV
006 = 0-100 PSI	100 mV
007 = 0-150 PSI	100 mV
008 = 0-200 PSI	100 mV
009 = 0-300 PSI	100 mV

INPUT/OUTPUT
A = 10 VDC (regulated)/see range code
*B = 15-45 VDC/4-20 mA output
*D = 15-45 VDC/1-5 VDC

*Contact Rep. or Factory Per Specs.

WETTED PARTS
2 = 316 St. Stl.

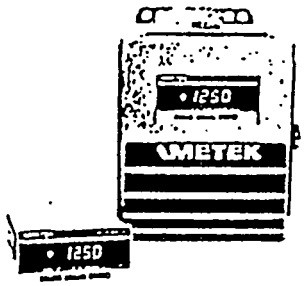
FILL FLUID
S = Dow Corning 200 Silicone Oil
M = Mineral Oil

OPTIONS
CS = Support Cable Bracket

CABLE LENGTH
(Specify length in feet)

MODEL 572
MONITOR/CONTROLLER

For continuous display/control of liquid levels, AMETEK provides the Model 572 digital meter/controller. This instrument can be directly attached to the transducer to display sensor output, transmit a 4-20 mA signal and control ancillary equipment via two built-in control relays. The Model 572 is available in a panel mounted case or a weatherproof housing; RS 232 also available.



AMETEK
CONTROLS DIVISION • 820 PENNSYLVANIA BLVD., FEASTERVILLE, PA 19047
TELEPHONE (215) 355-6900 TELEX 29-1769 FAX (215) 355-2937

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THE HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL

Volume IV. Documentation for Version 2

by

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Interagency Agreement Number DW21931425-01-3

Project Officer

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OFFICE OF SOLID WASTE AND EMERGENCY RESPONSE
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WASHINGTON, DC 20460

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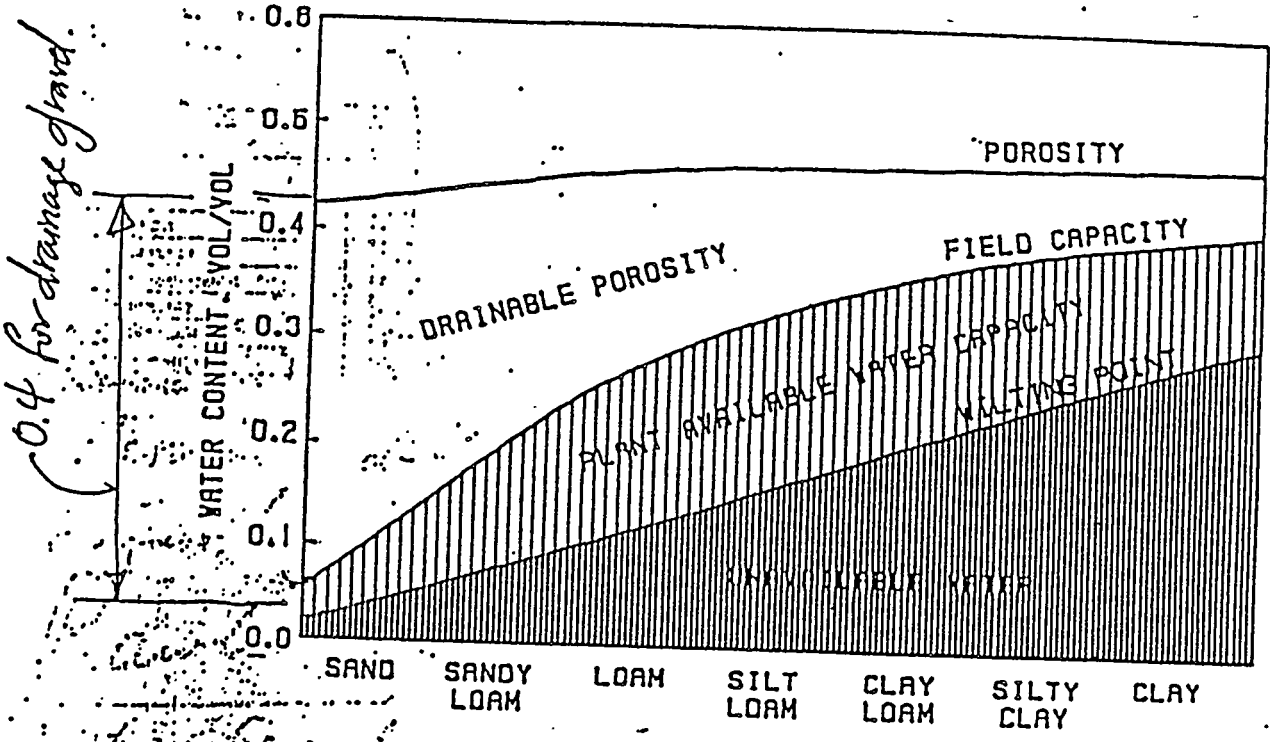


Figure 6. General relation between soil water properties and soil texture (Rawls et al., 1982)

↑ See next page.

From HELP MODEL DOCUMENTATION.

↳ See previous page.

40

6.f 10

REFERENCES

Arnold, J. G., J. R. Williams, and A. D. Nicks. 1986. SWRRB, A Simulator for Water Resources in Rural Basins. Agricultural Research Service, USDA.

Brooks, R. H. and A. T. Corey. 1964. Hydraulic Properties of Porous Media. Hydrology Paper No. 3, Colorado State University. 27pp.

Brown, K. W., J. C. Thomas, R. L. Lytton, P. Jayawikrama, and S. C. Bahrt. 1987. Quantification of Leak Rates Through Holes in Landfill Liners. EPA/600/ S2-87-062. U.S. Environmental Protection Agency, Cincinnati, OH.

Campbell, G. S. 1974. A Simple Method for Determining Unsaturated Hydraulic Conductivity from Moisture Retention Data. Soil Science, Vol. 117, No. 6. pp. 311-314.

Chow, V. T., Editor. 1964 Handbook of Applied Hydrology. McGraw-Hill Book Company, New York, New York.

Horton, R. E. 1919. Rainfall Interception. Monthly Weather Review, U.S. Weather Bureau, Vol. 47, No. 9. pp. 603-623.

IBM. 1974. IBM OS FORTRAN IV (H extended). Compiler Programmer's Guide. International Business Machines Corporation, New York, New York.

Knisel, W. G., Editor. 1980. CREAMS, A Field Scale Model for Chemical Runoff and Erosion from Agricultural Management Systems. Vols. I, II, and III, Draft Copy, USDA-SEA, AR, Cons. Res. Report 24. 643 pp.

Li, E. A. 1975. A Model to Define Hydrologic Response Units Based on Characteristics of the Soil-Vegetative Complex Within a Drainage Basin. M.S. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA. 124 pp.

Linsley, R. K., M. A. Kohler, and J. L. H. Paulhus. 1982 Hydrology for Engineers, Third Edition, McGraw-Hill Book Company, New York, New York. 508 pp.

Lutton, R. J., G. L. Regan, and L. W. Jones. 1979. Design and Construction of Covers for Solid Waste Landfills. EPA-600/2-79-165. U.S. Environmental Protection Agency, Cincinnati, OH.

Perrier, E. R. and A. C. Gibson. 1980. Hydrologic Simulation on Solid Waste Disposal Sites. EPA-SW-868, U.S. Environmental Protection Agency, Cincinnati, OH. 111 pp.

Rawls, W. J., D. L. Brakensiek, and K. E. Saxton. 1982. Estimation of Soil Water Properties. Transactions of the American Society of Civil Engineers. pp. 1316-1320.

Richardson, C. W. 1981. Stochastic Simulation of Daily Precipitation, Temperature, and Solar Radiation. Water Resources Research, Vol. 17, No. 1, pp. 182-190.

Source.

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7.f10

Source:

Requirements for Hazardous Waste Landfill Design, Construction and Closure.
EPA/625/4-89/022
August 1989

10. LEAK RESPONSE ACTION PLANS

This final chapter reviews proposed requirements for Response Action Plans, or RAPs, that are contained in the proposed leak detection rule issued in May, 1987. It focuses on the concepts behind the RAPs and the preliminary, technical calculations used in developing them. The main topics of discussion will be the technical basis for the two response action triggers, action leakage rate (ALR) and rapid and large leakage (RLL) rate; the RAPs themselves; and the RAP submittal process.

Background

In the Hazardous and Solid Waste Amendments (HSWA) of 1984, Congress required that leaks from new land disposal facilities be detected at the earliest practical time. However, HSWA did not require or specify actions to be taken once a leak is detected in the leak detection system. Therefore, EPA proposed requirements for response action plans to deal with leaks detected in the leak detection system between the two liners. EPA realizes that even with a good construction quality assurance plan, flexible membrane liners (FMLs) will allow some liquid transmission either through water vapor permeation of an intact FML, or through small pinholes or tears in a slightly flawed FML. Leakage rates resulting from these mechanisms can range from less than 1 to 300 gallons per acre per day (gal/acre/day). If unchecked, these leak rates may result in increased hydraulic heads acting on the bottom liner and potential subsequent damage to the liner system.

The idea behind the RAP is to be prepared for any leaks or clogging of the drainage layer in the leak detection system that may occur during the active life or post-closure care period of a waste facility. The first step is to identify the top liner leak rates that would require response actions. Therefore, in the proposed leak detection rule of May 29, 1987, EPA established two triggers for response actions: the Action Leakage Rate (ALR) and the Rapid and Large Leakage (RLL) rate. The ALR is a low-level leak rate that would indicate the presence of a small hole or defect in the top liner. The RLL is indicative of a severe breach or large tear in the top liner. A different level of responsiveness would be required for leakage rates above these two triggers. RAPs developed by owners or operators may have more than two triggers as appropriate to cover the range of leak rates expected for a landfill unit. In addition to triggers, the proposed rule also defines the elements of a RAP, gives an example of one, and discusses the procedures for submitting and reviewing a RAP.

Action Leakage Rate (ALR)

EPA has historically used the term de minimus leakage when referring to leaks resulting from permeation of an intact FML. Action leakage rate (ALR) was developed to distinguish leak rates due to holes from mere permeation of an intact FML, and to initiate early interaction between the owner/operator of the unit and the Agency. The ALR essentially defines top liner leakage in a landfill, and the proposed value is based on calculated leak rates through a 1 to 2 mm hole in a FML subject to low hydraulic heads on the order of 1 inch. The proposed ALR, therefore, is representative of well-designed and operated landfills, although, as proposed, it would also apply to surface impoundments and waste piles.

Because EPA is considering setting a single ALR value applicable to landfills, surface impoundments, and waste piles, the Agency calculated top liner leak rates for different sizes of holes and for different hydraulic heads. In addition, EPA compared leak rates for a FML top liner with that for a composite top liner, since many new facilities have double composite liner systems. Table 10-1 shows the results of these calculations for FML and composite top liners. Even for FMLs with very small holes (i.e., 1 to 2 mm in diameter), leak rates can be significant depending on the hydraulic head acting on the top liner. The addition of the compacted low permeability soil layer to the FML significantly reduces these leak rates to less than 10 gal/acre/day, even for large hydraulic heads that are common in surface impoundments. These results indicate that,

8d-10

at least for deep surface impoundments with large hydraulic heads, double composite liner systems may be the key to reducing the leak rates to de minimus levels that are below the proposed ALR.

Table 10-1. Calculated Leakage Rates through FML and Composite Liners (gal/acre/day)

FML Alone

Leakage Mechanism	Hydraulic Head, ft 0.1	1	10
Small Hole (1-2 mm)	30	100	300
Standard Hole (1 cm²)	300	1,000	3,000

Composite Liner (good contact)

Leakage Mechanism	Hydraulic Head, ft 0.1	1	10
Small Hole (1-2 mm)	0.01	0.1	2
Standard Hole (1 cm²)	0.01	0.2	3

Source: U.S. EPA. 1987. Background document on proposed liner and leak detection rule. EPA/530-SW-87-015.

EPA's proposed rule sets the ALR at 5 to 20 gal/acre/day, a difficult range to achieve with a primary FML alone (especially for surface impoundments). The proposed rule also enables the owner/operator to use a site-specific ALR value that would take into account meteorological and hydrogeological factors, as well as design factors that might result in leak rates that would frequently exceed the ALR value. Using these factors, a surface impoundment that meets the minimum technological requirements of a FML top liner could conceivably apply for a site-specific ALR value.

Daily leakage rates through top liners can vary by 10 to 20 percent or more, even in the absence of major precipitation events. Because of these variations, EPA may allow the landfill owner/operator to average daily readings over a 30-day period, as long as the leakage rate does not exceed 50 gal/acre/day on any 1 day. If the average daily leak rate does not exceed the ALR, then the owner/operator does not have to implement a RAP.

Rapid and Large Leakage (RLL)

The Rapid and Large Leakage (RLL) rate is the high-level trigger that indicates a serious malfunction of system components in the double-lined unit and that warrants immediate action. In developing the proposed rule, EPA defined the RLL as the maximum design leakage rate that the leak detection system can accept. In other words, the RLL is exceeded when the fluid head is greater than the thickness of the secondary leachate collection and removal system (LCRS) drainage layer. The visible expression of RLL leakage in surface impoundments is the creation of bubbles, or "whales," as the FML is lifted up under the fluid pressure. See Chapter Three for further discussion of "whales".

Because the RLL is highly dependent on the design of the leak detection system, EPA's proposed rule requires that owners/operators calculate their own site-specific RLL values. EPA also proposes to require that owners/operators submit a RAP for leakage rates exceeding that value prior to beginning operation of a unit. The EPA Regional Administrator must approve the RAP before a facility can receive wastes.

The following equations represent EPA's preliminary attempt to define a range of potential RLL values for a hypothetical leak detection system, which consists of a 1-foot granular drainage layer with 1 cm/sec hydraulic conductivity. These calculations are for two-dimensional rather than three-dimensional flow. In addition, the equations apply to flow from a single defect in the FML, rather than multiple defects. Therefore, results from this analysis are only preliminary ones, and the EPA will develop guidance on calculating RLL values in the near future.

RLL values can be calculated using the following equation:

$$h = (Q_d/B)/(k_d \tan\beta) \qquad (1)$$

where:
- h = hydraulic head
- Q_d = flow rate entering into the drainage layer
- B = width of the drainage layer
- k_d = hydraulic conductivity of the drainage layer
- β = slope of the drainage layer perpendicular to, and in the plane of, flow toward the collection pipe

When the value for h exceeds the thickness of the drainage layer (1 foot in this example), the leakage rate is greater than the RLL value for the unit.

In reality, a leak from an isolated source, i.e., a tear or a hole in the FML, results in a discreet zone of saturation as the liquids flow toward the collection pipe (see Figure 10-1). The appropriate variable representing the width of flow, then, is not really B, the entire width of the drainage layer perpendicular to flow, but b, the width of saturated flow perpendicular to the flow direction. If b were known, the equation could be solved. But to date, the data has

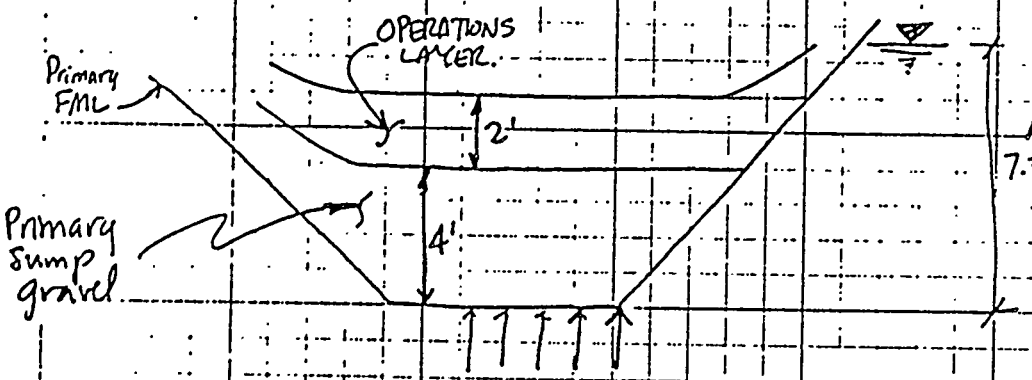
Golder Associates

SUBJECT NDO			
Job No. 893-1186 Ref.	Made by M. BROWN Checked FSS Reviewed	Date 2/7/90 Sheet 9 of 10	

Check uplift on Primary Liner

Design Condition: If secondary pump fails with water in secondary sump and none in primary, will uplift occur? Assume 25 yr. flood occurred, primary leaked, and then was rapidly evacuated.

Max. Water Level above bottom of Primary Liner = 7.5' (See attached)



Uplift = 7.5' x 62.4 pcf = 468 psf ✓

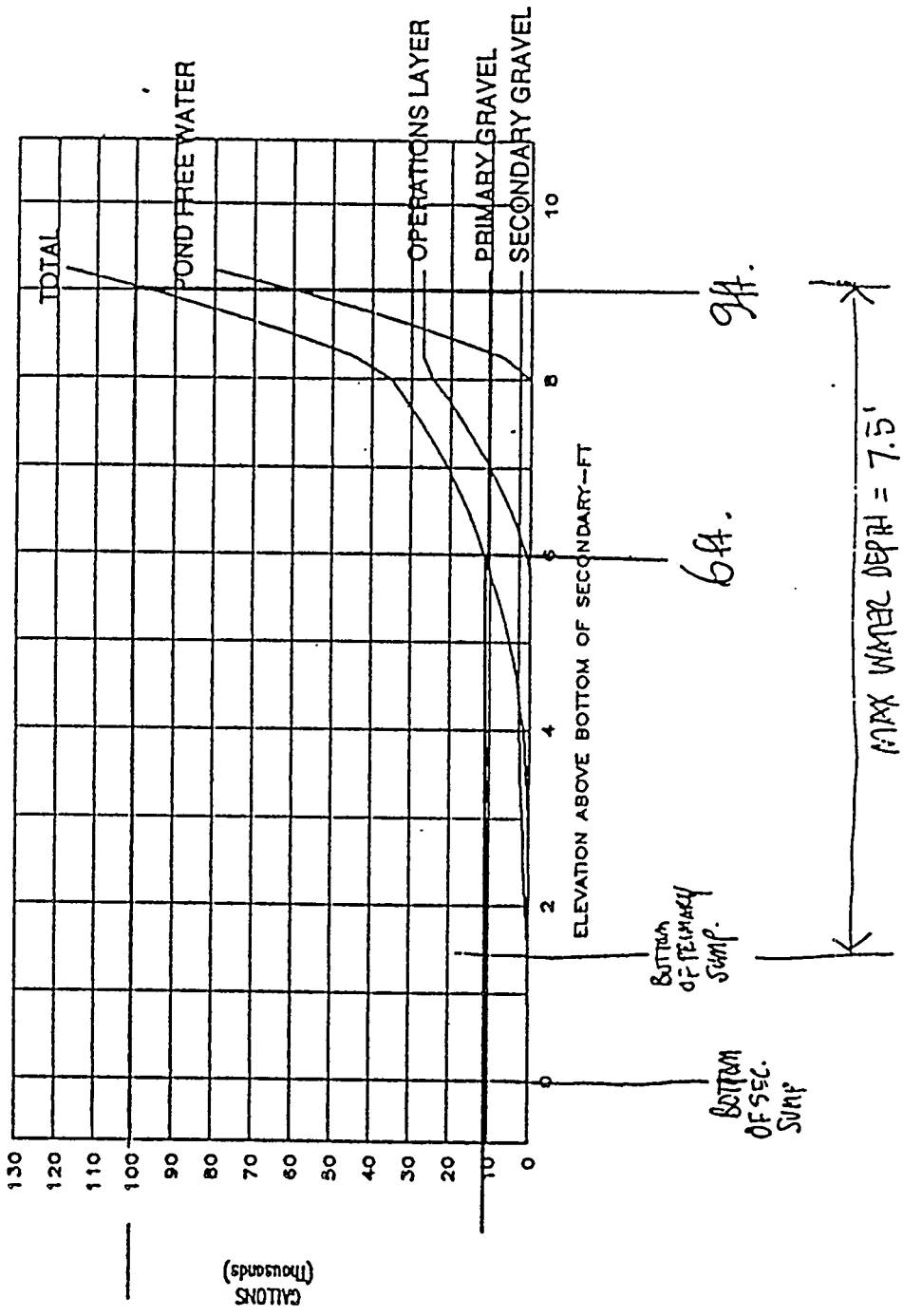
Soil pressure down = 6' x 110 pcf = 660 psf > 468 ∴ OK

(est. moist density)

10 of 10

TABLE III

LEACHATE STORAGE VOLUME AVAILABLE



Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 16

Project Title: WHC NON DRAG OFF

Project No. 893-1186

Calculation Title: Vertical Riser Pipe Foundation - Structural Design.

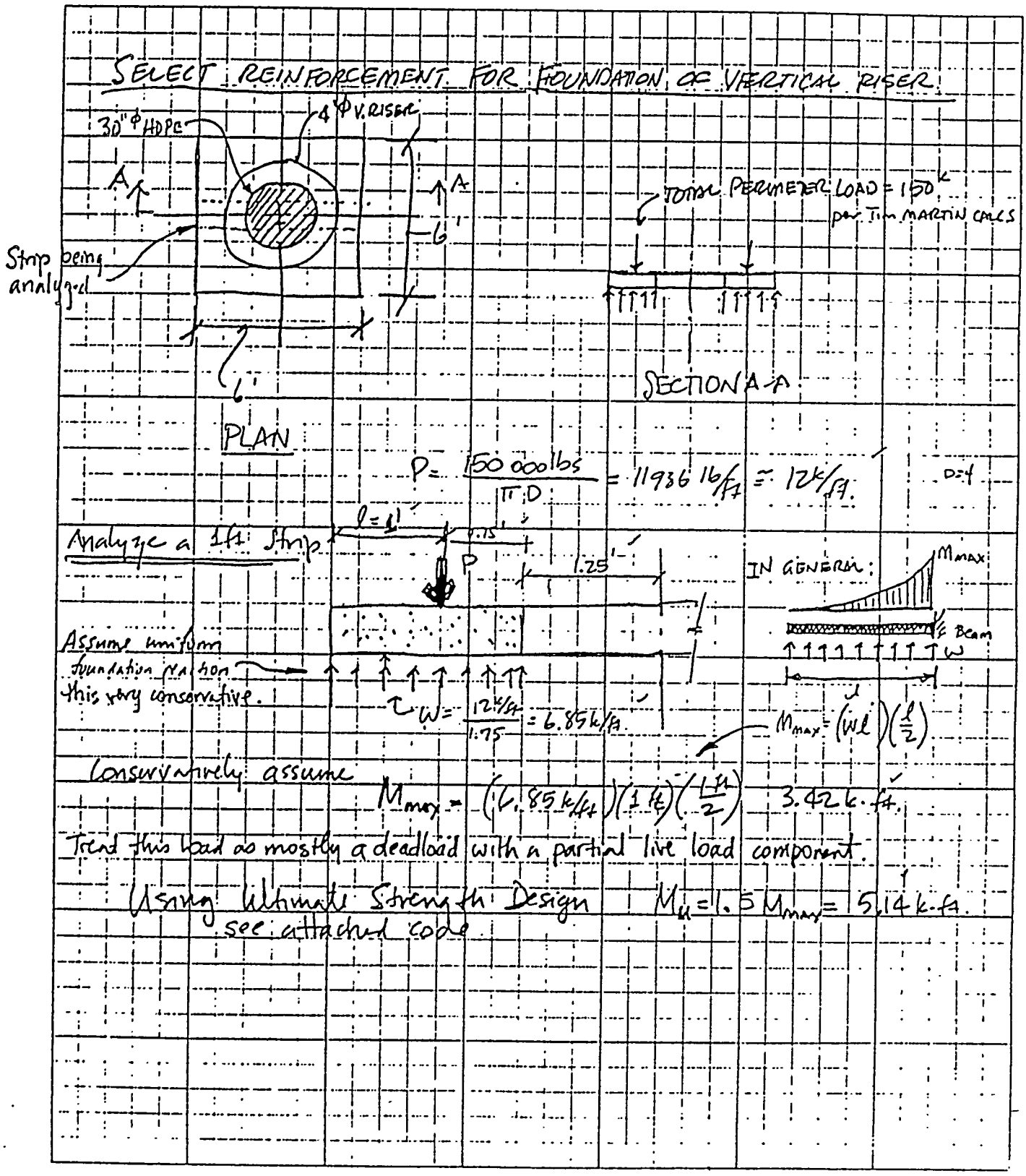
	Name	Date
Prepared By:	M. BROWN	1-24-90
Checked By:	Frank S. Shui	2-7-90
Approved By:	[illegible]	2-8-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT WHC / NDO VERTICAL RISER CONCRETE FD.
Job No. 913-1182 Ref.
Made by M. Brown Checked TSS Reviewed
Date 1/24/90 Sheet 1 of 6

SELECT REINFORCEMENT FOR FOUNDATION OF VERTICAL RISER.



α_m = average value of α for all beams on edges of a panel

β = ratio of clear spans in long to short direction of two-way slabs

β_s = ratio of length of continuous edges to total perimeter of a slab panel

ϕ = strength reduction factor. See Section 9.3

9.1 – General

9.1.1 – Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this code

9.1.2 – Members also shall meet all other requirements of this code to insure adequate performance at service load levels.

9.2 – Required strength

9.2.1 – Required strength U to resist dead load D and live load L shall be at least equal to

$$U = 1.4D + 1.7L \quad (9\text{-}1)$$

9.2.2 – If resistance to structural effects of a specified wind load W are included in design, the following combinations of D, L, and W shall be investigated to determine the greatest required strength U

$$U = 0.75(1.4D + 1.7L + 1.7W) \quad (9\text{-}2)$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W \quad (9\text{-}3)$$

but for any combination of D, L, and W, required strength U shall not be less than Eq. (9-1).

9.2.3 – If resistance to specified earthquake loads or forces E are included in design, load combinations of Section 9.2.2 shall apply, except that 1.1 E shall be substituted for W.

9.2.4 – If resistance to lateral earth pressure H is included in design, required strength U shall be at least equal to

$$U = 1.4D + 1.7L + 1.7H \quad (9\text{-}4)$$

except that where D or L reduce the effect of H, 0.9D shall be substituted for 1.4D and zero value of L shall be used to determine the greatest required strength U. For any combination of D, L, and H, required strength U shall not be less than Eq. (9-1).

9.2.5 – If resistance to lateral liquid pressure F is included in design, load combinations of Section 9.2.4 shall apply, except that 1.4F shall be substituted for 1.7H. Vertical liquid pressure shall be considered as dead load D, with due regard to variation in liquid depth.

9.2.6 – If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

9.2.7 – Where structural effects T of differential settlement, creep, shrinkage, or temperature change may be significant in design, required strength U shall be at least equal to

$$U = 0.75(1.4D + 1.4T + 1.7L) \quad (9\text{-}5)$$

but required strength U shall not be less than

$$U = 1.4(D + T) \quad (9\text{-}6)$$

Estimations of differential settlement, creep, shrinkage, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.3 – Design strength

9.3.1 – Design strength provided by a member or cross section in terms of load, moment, shear, or stress shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by a strength reduction factor ϕ.

9.3.2 – Strength reduction factor ϕ shall be as follows:

(a) Flexure, with or without axial tension . . 0.90

(b) Axial tension 0.90

(c) Axial compression, with or without flexure:
Members with spiral reinforcement conforming to Section 10.9.3. . . . . 0.75
Other reinforced members 0.70

except that for low values of axial load, ϕ may be increased in accordance with the following:

For members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ to zero.

For other reinforced members, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ or ϕP_b, whichever is smaller, to zero.

(d) Shear and torsion 0.85

Golder Associates

SUBJECT WHC/NDO VERTICAL PUSH FOUND. SLAB

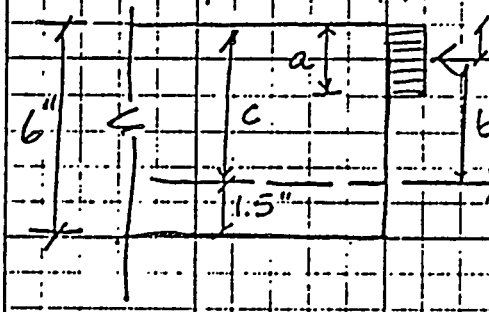
Job No. 983186 Ref. | Made by M. Brown | Checked FSS | Reviewed | Date 1/24/90 | Sheet 3 of 6

Because bars are epoxy coated and concrete is coated allow a reduced clear cover of 1.5" ✓ See attached code

USE ULTIMATE STRENGTH DESIGN

See Attached page from Text by Winter Urquhart O'Rourke + Nilson

$F_y = 60$ ksi, $F_c' = 3$ ksi



$c = 6 - 1.5 = 4.5"$ ✓

assume $a = 1"$ $b = c - \frac{a}{2} = 4"$ ✓

$M_u = A_s F_y .85\, b$

$A_s = \frac{M_u}{F_y .85\, 4/12}$

$A_s = \frac{5.14 \text{ k-ft.}}{60 \text{ ksi} \times .85 \times .33} = 0.305 \text{ in}^2/\text{ft.}$ ✓

Checking a:

$\Sigma F_x = 0$

$0.85 a f_c' 12" = A_s f_y = 0.305 \times 60 = 18.3$ kips ✓

$a = \frac{18.3}{3 \times .85 \times 12} = 0.598" < 1.0$ ✓ so OK

Steel needed to provide 0.3 in²/ft. ? — ✓

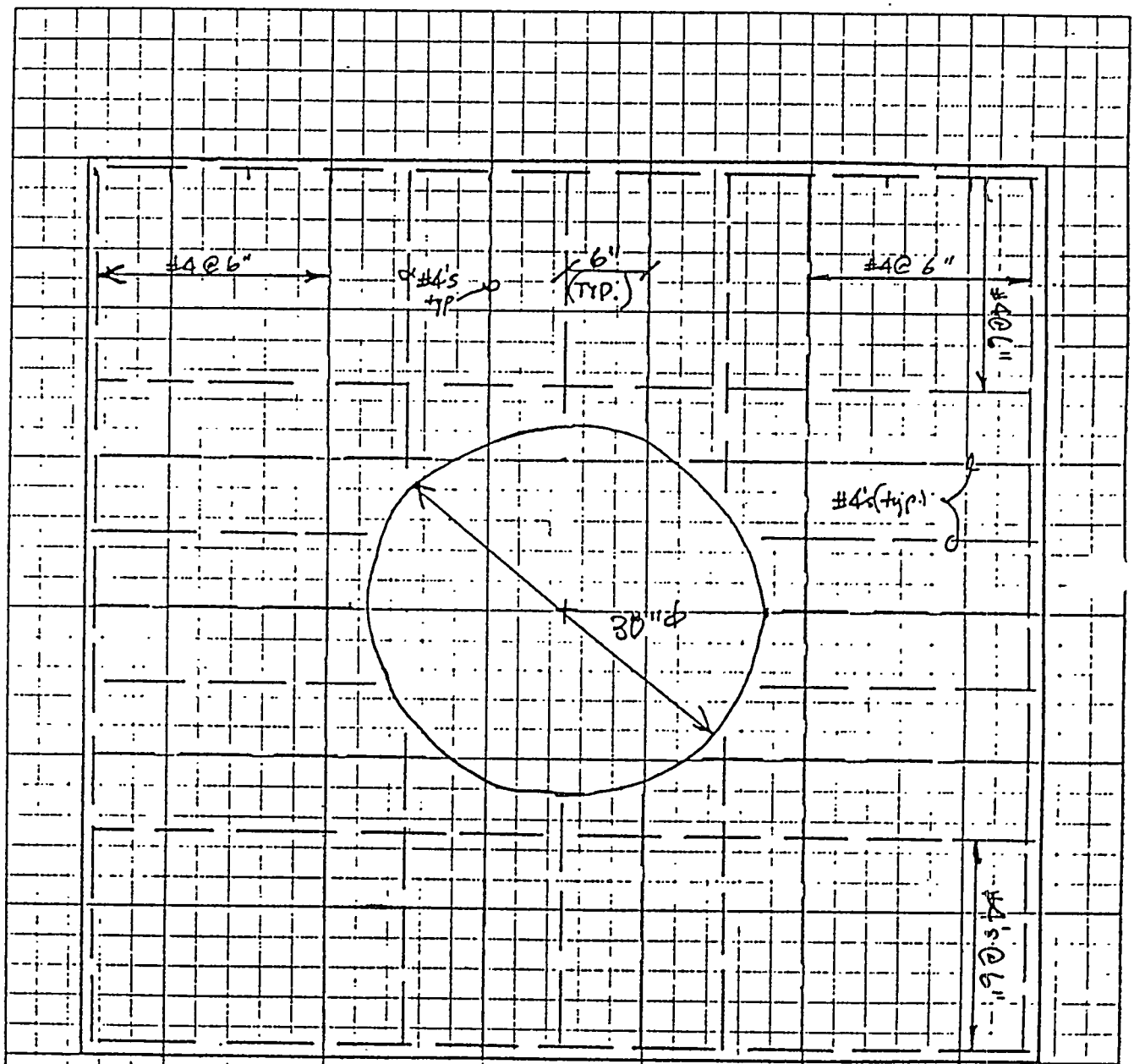
Use #4 bars @ 6" ⇒ 0.39 in²/ft.

Place in bottom of slab, 1½" clear cover

Since handling of slab may cause it to be inverted — place #4's at 6" each face (top + bottom).

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SUBJECT WHC/ND 0 SLAB.			
Job No. 893-1186 Ref.	Made by M. BROWN Checked FSS Reviewed	Date Sheet 4 of 6	



PLAN SHOWING REINFORCEMENT

NOTE:

1. Provide 1½" clear cover all around
2. Provide identical reinforcement each face, only one face is shown.
3. Lifting hooks to be located by contractor.

7.5.3 – Welded wire fabric (with wire size not greater than W5 or D5) used in slabs not exceeding 10 ft in span may be curved from a point near the top of slab over the support to a point near the bottom of slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.

7.5.4 – Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the Engineer.

7.6 – Spacing limits for reinforcement

7.6.1 – Clear distance between parallel bars in a layer shall not be less than d_b nor 1 in. See also Section 3.3.3.

7.6.2 – Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 1 in.

7.6.3 – In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall not be less than $1.5d_b$ nor 1½ in. See also Section 3.3.3.

7.6.4 – Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

7.6.5 – In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than 3 times the wall or slab thickness, nor 18 in.

7.6.6 – Bundled bars

7.6.6.1 – Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to 4 in any one bundle.

7.6.6.2 – Bundled bars shall be enclosed within stirrups or ties.

7.6.6.3 – Bars larger than #11 shall not be bundled in beams.

7.6.6.4 – Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

7.6.6.5 – Where spacing limitations and minimum concrete cover are based on bar diameter d_b, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

7.6.7 – Prestressing tendons and ducts

7.6.7.1 – Clear distance between pretensioning tendons at each end of a member shall not be less than $4d_b$ for wire, nor $3d_b$ for strands. See also Section 3.3.3. Closer vertical spacing and bundling of strands may be permitted in the middle portion of a span.

7.6.7.2 – Post-tensioning ducts may be bundled if shown that concrete can be satisfactorily placed and if provision is made to prevent the tendons, when tensioned, from breaking through the duct.

7.7 – Concrete protection for reinforcement

7.7.1 – Cast-in-place concrete (nonprestressed).

The following minimum concrete cover shall be provided for reinforcement:

	Minimum cover, in.
(a) Concrete cast against and permanently exposed to earth ..	3
(b) Concrete exposed to earth or weather:	
#6 through #18 bars	2
#5 bar, W31 or D31 wire, and smaller	1½
(c) Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
#14 and #18 bars.....	1½
#11 bar and smaller	3/4
Beams, columns: Primary reinforcement, ties, stirrups, spirals	1½
Shells, folded plate members:	
#6 bar and larger.....	3/4
#5 bar, W31 or D31 wire, and smaller	1/2

7.7.2 – Precast concrete (manufactured under plant control conditions)

The following minimum concrete cover shall be provided for reinforcement:

(a) Concrete exposed to earth or weather:	
Wall panels:	
#14 and #18 bars.....	1½
#11 bar and smaller	3/4
Other members:	
#14 and #18 bars.....	2
#6 through #11 bars	1½
#5 bar, W31 or D31 wire, and smaller.....	1¼

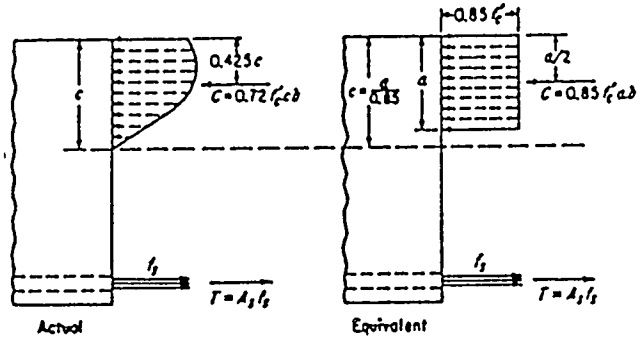


Fig. 2.11 Actual and equivalent rectangular stress distributions at ultimate load (for f'_c less than 4000 psi).

country was first proposed by C. S. Whitney and subsequently elaborated and checked experimentally by others (see, e.g., Ref. 2.4). For concrete with cylinder strength f'_c not exceeding 4000 psi, an actual stress distribution and the fictitious equivalent distribution are shown in Fig. 2.11.

It is seen that the actual distribution is replaced by a fictitious equivalent one of simple rectangular outline. The intensity of this uniform and constant equivalent compression stress is $0.85f'_c$, and it is thought of as acting over a part of the section of depth $a = 0.85c$, where c is the distance to the actual neutral axis. It is in no way maintained that the compression stresses are actually distributed in this most unlikely manner. It is maintained, however that this equivalent distribution gives the answers just derived for the actual situation. Of this one can easily convince oneself. In a rectangular beam of width b the equivalent uniform compression stress $0.85f'_c$ acts on an area ab, so that the total compression force is

$$C = 0.85f'_c ab \tag{2.30}$$

If one expresses the depth a of the rectangular stress block in terms of the distance c to the neutral axis as assumed, that is, $a = 0.85c$, one obtains for the compression force $C = 0.85f'_c \times 0.85c \times b = 0.72f'_c cb$, which is the same as Eq. (2.21) for $\alpha = 0.72$. Further, since the centroid of a rectangle is at middepth, the distance from the compression edge to C is evidently $a/2$. Again expressing a in terms of c as above, one obtains the distance to the compression resultant as $0.5 \times 0.85c = 0.425c$, which is the correct distance for $\beta = 0.425$.

Since the chosen equivalent rectangular stress distribution satisfies the stated requirements (1) and (2), it will give exactly the same answers as were derived previously. This is so for f'_c not exceeding 4000 psi because, in this range of concrete strengths, the parameters α and β are constants

equal, respectively, to 0.72 and 0.425. For concretes of higher strength it has been stated that α and β decrease because of the greater brittleness of such concretes. In the equivalent rectangular stress distribution, this is reflected in the following stipulation:

$$a/c = k_1 = 0.85 \text{ for } f'_c \leq 4000 \text{ psi and decreases by 0.05 for every 1000 psi above 4000} \tag{2.31}$$

This equivalent rectangular stress distribution can be used for deriving the equations which have been developed in the previous section. The failure criteria, of course, are the same as before: yielding of the steel at $f_s = f_y$ or crushing of the concrete at $\epsilon_c = 0.003$. More important, however, because the rectangular stress block is easily visualized and its geometrical properties are extremely simple, many calculations are conveniently carried out directly without reference to formally derived equations.

Example Using the equivalent rectangular stress distribution, calculate directly the ultimate strength of the previously analyzed beam.

The distribution of stresses, internal forces, and strains is as shown in Fig. 2.12. The depth a of the equivalent stress block is found from the equilibrium condition $C = T$. Hence $ab \times 0.85f'_c = A_s f_y$, or $a \times 10 \times 2550 = 2.35 \times 40{,}000$, from which $a = 3.70$ in. The distance to the neutral axis, by the definition of the rectangular stress block, is $c = a/0.85 = 3.70/0.85 = 4.35$ in. One way of determining whether the strength of the beam is governed by yielding of the steel or by crushing of the concrete is to calculate whether the concrete strain in the top fiber is smaller or larger than 0.003 at the instant the steel starts yielding. At that point the strain in the steel is $\epsilon_s = f_y/E_s = 40{,}000/29{,}000{,}000 = 0.00138$. From the similar triangles of the strain distribution, $\epsilon_c = \epsilon_s c/(d - c) = 0.00138(4.35/18.65) = 0.00032$. This value is smaller than the assumed crushing strain of the concrete, and hence failure will be initiated

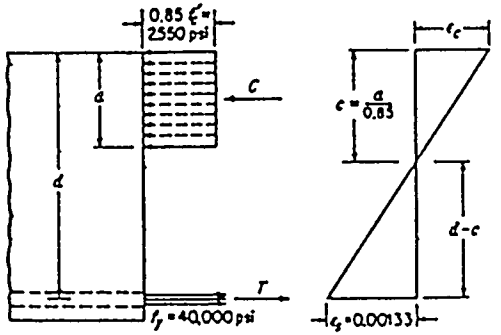


Fig. 2.12

Design of Concrete Structures, Winter, Urquhart, O'Rourke and Nilson, 7th Ed, McGraw Hill 1964,
C.6-7
WHC-SD-W025-FDR-001, Rev. 1

Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 10

Project Title: WHC Non Drag Off

Project No. 893-1186

Calculation Title: B.14 Sump Design.

	Name	Date
Prepared By:	M. Brown	1/22/90
Checked By:	Frank S. Shuri	2-7-90
Approved By:	PATRICK GIBB	2-8-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT NON DRAW OFF – PUMP DISCH. PIPE DIAM.

Job No. 893-1186 Ref.

Made by M. BROWN Checked FSS Reviewed

Date 1/22/90 Sheet 1 of 10

OBJECTIVE:

CALCULATE MINIMUM AND MAXIMUM PIPE DIAMETERS FOR DISCHARGE PIPING

Use ASTM Standard F714/D 3035 Pipe Specs + Dimensions

Velocity should be less than 8fps* under normal operation

Submersible Pump Capacities 5 gpm. = 0.011 cfs

Smallest pipe available Nominal 1½"ϕ

in SDR 13.5 I.D. = 1.62" Velocity = V = 0.77 fps, OK.

$V = Q/A = \frac{0.011}{\pi\left(\frac{1.62}{2\times12}\right)^2}$

Large pump 150 gpm. = 0.333 cfs

Using SDR 17 ID = 3.08 V = 6.46 fps < 8 OK

3" nominal,

$V = Q/A = \frac{0.333}{\pi\left(\frac{3.08}{2\times12}\right)^2}$

* The selection of 8 fps as a maximum velocity is based on generally accepted engineering practice regarding the maximum economic flow velocity and therefor minimum pipe diameter. See attached. Constant operation systems have lower flow velocities because of the importance of head loss.

Occassional operation systems have higher velocities because of the importance of capital cost.

2 of 10

head available, the character of the conduit material, and limiting velocities.

Locating the Aqueduct. The location of an aqueduct is based on engineering and economic considerations. Since the terminal locations are dictated by the source of supply and the region to be served, the problem becomes one of finding the most practical and economic route between them. The choice of location has an obvious bearing on the type of aqueduct that can be built. Aqueducts built to grade require topography such that cut-and-cover operations can be closely balanced. Pressure aqueducts, on the other hand, can follow the topography and often permit the choice of a direct route. Pumping costs in such cases would have to be included in making economic comparisons with conduits constructed along the hydraulic grade line. Material and construction costs must also be balanced against length. The most direct route is not always the least expensive.

Determining the Size and Capacity of the Conduit. The size and configuration of the aqueduct finally selected will in all probability be variable along the route. The choice of section depends on the hydraulic loading, external loading and economic considerations.

Once the stage has been reached where the actual conduit design is required, a decision will have been made as to the quantities of water to be transported along its various reaches. This information will be derived by the methods outlined in Chap. 3 and will answer the question of the required capacity.

For a given type aqueduct (pipe, tunnel, flume, canal) the size will usually be determined on the basis of hydraulic, economic and construction considerations. Occasionally construction practices dictate a minimum size in excess of that required to handle the flow under the prevailing hydraulic conditions (available head). This condition would most likely be encountered where a tunnel was involved. Hydraulic factors which control the design are the head available and permissible velocities. Available heads are affected by reservoir drawdown and local pressure requirements. Where the head is in excess of that needed to transport the water, power development might be economical. Limiting velocities are based on the character of the water to be transported and the need to protect transmission lines against excessive pressures which might be developed through hydraulic surge. Where silt is transported with the water, minimum velocities of about 2.5 fps should be maintained. Maximum velocities must preclude pipe erosion or hydraulic surge problems. Upper limits are usually between 10 and 20 fps. A normal range of 4 to 6 fps is common.

Where power generation is involved, pumping costs or power values and conduit costs jointly determine the conduit size. For single gravity-

Clark + Viessman
Water Supply and Pollution Control

Golder Associates

SUBJECT WHC NON DRAG OFF

Job No. 893-1186.032 Ref.

Made by M. Brown Checked [illegible] Reviewed

Date 12/14/89 Sheet 3 of 10

ESTIMATE THE DRAIN-BACK VOLUMES TO THE SUMP PUMPS

Assume #2 Primary Sump Pump Q_{max} = 150 gpm

Try nominal 4" HDPE piping SDR17

from pipe dimension chart I.D = 3.97" ⇒ V = 3.89 fps.

try 3" nominal, SDR17 ID = 3.08 V = 6.46 fps OK

Assume this is used

Even if we use 3"ϕ SDR11 ID = 2.86" V = 7.49 fps < 8 fps OK.

Assume #1 primary & secondary submersible pumps

are 10 gpm = 0.022 cfs

Try nominal 1½" diam. SDR 13.5 (lightest avail 266)

ID = 1.62" V = 1.55 fps

for SDR 7.3 ID = 1.38" V = 2.15 fps. ← USE THIS.

for #2 Primary SDR17, 3"ϕ $A = \frac{\pi D^2}{4} = \frac{\pi\ 3.08^2}{4} \times 7.45\ in^2 = 0.0517\ ft^2$

length 20'

see plan view in calc. 8.12

L = 40 + 100 + 50 = 190'

(slope correction negligible)

100' 50'

Volume = 0.0517 ft² × 190' = 9.82 ft³ = 73.5 gallons.

no vertical loop for submersible discharge line

for Submersibles L = 100 + 50 = 150' $A = \frac{\pi D^2}{4} = \frac{\pi(1.38)^2}{4} = 1.5\ in^2 = 0.0104\ ft^2$

V = .0104 × 150 = 1.56 ft³ = 12 gallons. 01

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SUBJECT NOO PRIMARY SUMP		
Job No. 893-1186 Ref.	Made by M. BROWN Checked [illegible] Reviewed	Date 1/28/10 Sheet 4 of 10

From attached volume-elevation information:

Elevation above bottom of Primary Sump	Usable Leachate Storage Volume gallons	
0'	0	
0.25'	7	
0.5'	27	← minimum water level, pump diam = 4"
0.75'	68	
1.00'	131	

drain back volume from submersible pump will raise water level

$$\frac{12 \text{ gal}}{(68-27)_{\text{gal}}} \times 3'' = 0.88'' \approx 1''$$

from self priming pump

$$3'' + \left[\frac{73.5 - (68-27)}{131-68} \times 3''\right] = 4.5''$$

Float switches and current sensitive relays on pump controls should be set to reflect these drain back levels.

VOLUME - ELEVATION SUMP

200 AREA TREATED EFFLUENT DISPOSAL FACILITY

B93-1186.032

Elevation ft	Secondary Drainage Gravel Eff Porosity = 0.45					Primary Drainage Gravel Eff Porosity = 0.50					Primary Operations Layer Eff Porosity = 0.35					Free Water Pond Eff Porosity = 1.00					Total Primary Water Storage gallons
	Area sq ft	Increm Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increm Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increm Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increm Volume cu ft	Accum Volume cu ft	Eff Pore Volume cu ft	Eff Pore Volume gallons	
0.00	0.00		0.00	0.00	0	0.00		0.00	0.0	0	0.0		0.00	0.0	0	0.00		0.00	0.0	0	0
0.25	8.6	1.1	1.1	0.5	4	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
0.50	30.0	4.8	5.9	2.7	20	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
0.75	62.1	11.5	17.4	7.8	59	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.00	107.3	21.2	38.6	17.4	130	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.25	171.0	34.8	73.4	33.0	247	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.50	245.3	52.0	125.4	56.4	422	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
1.75	326.3	71.5	196.9	88.6	663	14.5	1.8	1.8	0.9	7	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	0
2.00	456.2	97.8	294.7	132.6	992	28.6	5.4	7.2	3.6	27	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	7
2.25	187.1	80.4	375.1	168.8	1263	58.4	10.9	18.1	9.0	68	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	27
2.50	229.8	52.1	427.2	192.2	1438	77.0	16.9	35.0	17.5	131	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	68
2.75	281.8	64.0	491.2	221.0	1653	105.1	22.8	57.8	28.9	216	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	131
3.00	336.4	77.3	568.4	255.8	1913	135.2	30.0	87.8	43.9	328	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	216
3.25	304.8	80.2	648.6	291.9	2183	247.8	47.9	135.7	67.8	507	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	328
3.50	226.0	66.4	714.9	321.7	2406	347.5	74.4	210.1	105.0	786	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	507
3.75	168.0	49.3	764.2	343.9	2572	460.8	101.0	311.1	155.6	1164	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	786
4.00	0.0	21.0	785.2	353.3	2643	585.0	130.7	441.9	220.9	1653	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	1164
4.25	0.0	0.0	785.2	353.3	2643	750.3	166.9	608.8	304.4	2277	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	1653
4.50	0.0	0.0	785.2	353.3	2643	911.3	207.7	816.5	408.2	3054	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	2277
4.75	0.0	0.0	785.2	353.3	2643	1073.1	248.0	1064.5	532.3	3981	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	3054
5.00	0.0	0.0	785.2	353.3	2643	1240.8	289.2	1353.8	676.9	5063	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	3981
5.25	0.0	0.0	785.2	353.3	2643	1405.4	330.8	1684.5	842.3	6300	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	5063
5.50	0.0	0.0	785.2	353.3	2643	1900.0	413.2	2097.7	1048.9	7845	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	6300
5.75	0.0	0.0	785.2	353.3	2643	2100.0	500.0	2597.7	1298.9	9715	0.0	0.0	0.0	0.0	0	0.00	0.0	0.0	0.0	0	7845
6.00	0.0	0.0	785.2	353.3	2643	0.0	262.5	2860.2	1430.1	10697	2500.0	312.5	312.5	109.4	818	0.00	0.0	0.0	0.0	0	9715
6.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	3000.0	687.5	1000.0	350.0	2618	0.00	0.0	0.0	0.0	0	11515
6.50	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	3500.0	812.5	1812.5	634.4	4745	0.00	0.0	0.0	0.0	0	13315
6.75	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	4000.00	937.5	2750.0	962.5	7200	0.00	0.0	0.0	0.0	0	15442
7.00	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	4600.00	1075.0	3825.0	1338.8	10014	0.00	0.0	0.0	0.0	0	17897
7.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	5100.00	1212.5	5037.5	1763.1	13188	0.00	0.0	0.0	0.0	0	20711
7.50	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	5500.00	1325.0	6362.5	2226.9	16657	0.00	0.0	0.0	0.0	0	23885
7.75	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	6000.00	1437.5	7800.0	2730.0	20420	0.00	0.0	0.0	0.0	0	27354
8.00	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	6850.00	1606.3	9406.3	3292.2	24626	0.00	0.0	0.0	0.0	0	31118
8.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	11600.00	2306.3	11712.5	4099.4	30663	7600.00	950.0	950.0	950.0	7106	35323
8.50	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	13400.00	3125.0	14837.5	5193.1	38845	8600.00	2025.0	2975.0	2975.0	22253	48466
8.75	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	14300.00	3462.5	18300.0	6405.0	47909	9700.00	2287.5	5262.5	5262.5	39364	71795
9.00	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	16000.00	3787.5	22087.5	7730.6	57825	11000.00	2587.5	7850.0	7850.0	58718	127240
9.25	0.0	0.0	785.2	353.3	2643	0.0	0.0	2860.2	1430.1	10697	18900.00	4362.5	26450.0	9257.5	69246	12100.00	2887.5	10737.5	10737.5	80317	160260

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Golder Associates

SUBJECT WHC-NDO Leachate Collection Riser Pipes
Job No. 893-1186 Ref. | Made by M. BROWN Checked FJS Reviewed | Date 1/24/90 Sheet 6 of 10

Calculate load necessary to bend slope riser pipes to required radius at toe of slope

PIPE = 8" DIAM. SDR = 11 from T. Martin calcs.

E = 136000 psi ✓ from Poly pipe Manual Attached

OD = 8.625 ✓

ID = 7.05 ✓

From AISC manual, attached

$M = EI\frac{d^2y}{dx^2}$

From AISC Steel Construction Manual

$I = 0.049087(d^4 - d_i^4)$

$I = 150\ in^4$ ✓

Required radius of curvature ≈ 20' from drawings. ✓

from math handbook, attached

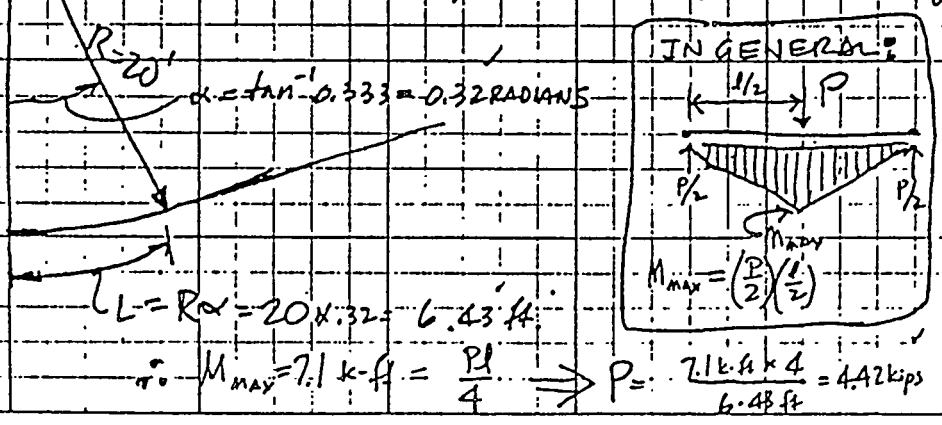
$R = \frac{1}{\frac{d^2y}{dx^2}} = \frac{EI}{M}$

R = 20' = 240" ✓

$240" = \frac{136000\ \frac{lb}{in} \times 150\ in^4}{M} \Rightarrow M = 85{,}000\ in\text{-}lbs$ ✓

$M = 7.1\ k\text{-}ft$ ✓

Determine point load which would produce this moment, in order to get a sense of magnitude



$M_{MAX} = 7.1\ k\text{-}ft = \frac{Pl}{4} \Rightarrow P = \frac{7.1\ k\text{-}ft \times 4}{6.43\ ft} = 4.42\ kips$ ✓

POLY PIPE INDUSTRIES
DESIGN & ENGINEERING GUIDE

7 & 10

Fig. 1
NOMINAL PHYSICAL PROPERTIES
POLY PIPE PE 3408 Pipe Material

Property	ASTM Reference	Nominal Value
Density, black	D 1505	0.956 gm/cc
Density, natural	D 1505	0.945 gm/cc
Melt flow	D 1238 (cond. E)	0.10 gm/10min.
Vicat softening temperature	D 1525	255°F
Brittleness temperature	D 746	◄–180°F
Environmental stress cracking	D 1693 (cond. C)	►5000 hrs.
Tensile, yield (2"/min.)‡	D 638	►3200 psi
Tensile, break (2"/min.)	D 638	►5000 psi
Elongation (2"/min.)	D 638	►800%
Stiffness	D 747	62,000 psi
Modulus of elasticity	D 638	113,000 psi
Flexural modulus	D 3350	136,000 psi ←
Impact strength, 0.25"	D 256 (method A)	12
Impact strength, 0.125"	D 256 (method A)	Non-break
Hardness, shore D	D 2240	66
Thermal conductivity	C 177	2.7 Btu-in/ft.²/hr./°F
Thermal expansion coefficient	D 696	9×10^{-5} in/in/°F
Long term hydrostatic strength	D 2837	►1600 psi
Hydrostatic design basis	D 2837	1600 psi
Hydrostatic design stress	D 2837	800 psi
Material cell* classification	D 3350	PE 345434C

‡Rate of cooling of test sheet per ASTM D1928, procedure C.

*For explanation of Poly Pipe cell classification see page A-10.

A-7

BEAM DIAGRAMS AND FORMULAS

Frequently used formulas

The formulas given below are frequently required in structural designing. They are included herein for the convenience of those engineers who have infrequent use for such formulas and hence may find reference necessary. Variation from the standard nomenclature on page 2 - 100 is noted.

BEAMS

Flexural stress at extreme fiber:

$$f = Mc/I = M/S$$

Flexural stress at any fiber:

$$f = My/I$$

y = distance from neutral axis to fiber.

Average vertical shear (for maximum see below):

$$v = V/A = V/dt \text{ (for beams and girders)}$$

Horizontal shearing stress at any section A-A:

$$v = VQ/I\,b$$

Q = statical moment about the neutral axis of the entire section of that portion of the cross-section lying outside of section A-A.

b = width at section A-A

(Intensity of vertical shear is equal to that of horizontal shear acting normal to it at the same point and both are usually a maximum at mid-height of beam.)

Slope and deflection at any point:

$$EI\frac{d^2y}{dx^2} = M$$

x and y are abscissa and ordinate respectively of a point on the neutral axis, referred to axes of rectangular coordinates through a selected point of support.

(First integration gives slopes; second integration gives deflections. Constants of integration must be determined.)

CONTINUOUS BEAMS (THE THEOREM OF THREE MOMENTS)

Uniform load:

$$M_a\frac{l_1}{I_1} + 2M_b\left(\frac{l_1}{I_1} + \frac{l_2}{I_2}\right) + M_c\frac{l_2}{I_2} = -\frac{1}{4}\left(\frac{w_1 l_1^3}{I_1} + \frac{w_2 l_2^3}{I_2}\right)$$

Concentrated loads:

$$M_a\frac{l_1}{I_1} + 2M_b\left(\frac{l_1}{I_1} + \frac{l_2}{I_2}\right) + M_c\frac{l_2}{I_2} = -\frac{P_1 a_1 b_1}{l_1}\left(1 + \frac{a_1}{l_1}\right) - \frac{P_2 a_2 b_2}{l_2}\left(1 + \frac{b_2}{l_2}\right)$$

Considering any two consecutive spans in any continuous structure:

M_a, M_b, M_c = moments at left, center, and right supports respectively, of any pair of adjacent spans.

l_1 and l_2 = length of left and right spans respectively, of the pair.

I_1 and I_2 = moment of inertia of left and right spans respectively.

w_1 and w_2 = load per unit of length on left and right spans respectively.

P_1 and P_2 = concentrated loads on left and right spans respectively.

a_1 and a_2 = distance of concentrated loads from left support in left and right spans respectively.

b_1 and b_2 = distance of concentrated loads from right support in left and right spans respectively.

The above equations are for beams with moment of inertia constant in each span but differing in different spans, continuous over three or more supports. By writing such an equation for each successive pair of spans and introducing the known values (usually zero) of end moments, all other moments can be found.

Manual of Steel Construction, 7th Ed. 1970

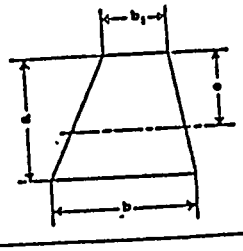
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

PROPERTIES OF GEOMETRIC SECTIONS

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TRAPEZOID

Axis of moments through center of gravity



$$A = \frac{d(b + b_1)}{2}$$

$$c = \frac{d(2b + b_1)}{3(b + b_1)}$$

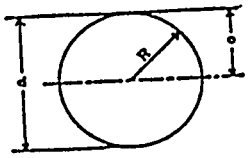
$$I = \frac{d^3 (b^2 + 4bb_1 + b_1^2)}{36 (b + b_1)}$$

$$S = \frac{d^3 (b^2 + 4bb_1 + b_1^2)}{12 (2b + b_1)}$$

$$r = \frac{d}{6(b + b_1)} \sqrt{2 (b^2 + 4bb_1 + b_1^2)}$$

CIRCLE

Axis of moments through center



$$A = \frac{\pi d^2}{4} = \pi R^2 = .785398\, d^2 = 3.141593\, R^2$$

$$c = \frac{d}{2} = R$$

$$I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4} = .049087\, d^4 = .785398\, R^4$$

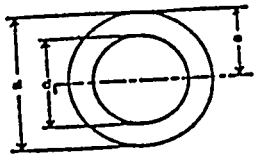
$$S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4} = .098175\, d^3 = .785398\, R^3$$

$$r = \frac{d}{4} = \frac{R}{2}$$

$$Z = \frac{d^3}{6}$$

HOLLOW CIRCLE

Axis of moments through center



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\, (d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\, (d^4 - d_1^4)$$

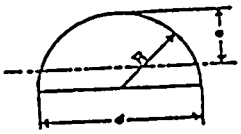
$$S = \frac{\pi(d^4 - d_1^4)}{32d} = .098175\, \frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

HALF CIRCLE

Axis of moments through center of gravity



$$A = \frac{\pi R^2}{2} = 1.570796\, R^2$$

$$c = R\left(1 - \frac{4}{3\pi}\right) = .575587\, R$$

$$I = R^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right) = .109757\, R^4$$

$$S = \frac{R^3}{24}\,\frac{(9\pi^2 - 64)}{(3\pi - 4)} = .190687\, R^3$$

$$r = R\,\frac{\sqrt{9\pi^2 - 64}}{6\pi} = .264336\, R$$

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Manual of Steel Construction, 7th Ed. 1970

113. DERIVATIVE OF ARC LENGTH. RADIUS OF CURVATURE (see Secs. 75, 107, 126) Let s be the length of arc measured along the curve $y = f(x)$ [or in polar coordinates $r = \phi(\theta)$] from some fixed point to any point $P(x,y)$ and α be the angle of inclination of the tangent line at P with OX. Then

$$\frac{dx}{ds} = \cos\alpha = \frac{1}{\sqrt{1 + \left(\frac{dy}{dx}\right)^2}}, \quad \frac{dy}{ds} = \sin\alpha = \frac{1}{\sqrt{1 + \left(\frac{dx}{dy}\right)^2}}$$

$$ds = \sqrt{dx^2 + dy^2} = \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\,dx = \sqrt{1 + \left(\frac{dx}{dy}\right)^2}\,dy$$

$$ds = \sqrt{dr^2 + r^2 d\theta^2} = \sqrt{r^2 + \left(\frac{dr}{d\theta}\right)^2}\,d\theta = \sqrt{1 + r^2\left(\frac{d\theta}{dr}\right)^2}\,dr$$

If $x = r\cos\theta$ and $y = r\sin\theta$

$$dx = \cos\theta\,dr - r\sin\theta\,d\theta, \quad dy = \sin\theta\,dr + r\cos\theta\,d\theta$$

The *radius of curvature* R at any point $P(x,y)$ of the curve $y = f(x)$ is

$$R = \frac{ds}{d\alpha} = \frac{\left[1 + \left(\frac{dy}{dx}\right)^2\right]^{3/2}}{\frac{d^2y}{dx^2}} = \frac{\{1 + [f'(x)]^2\}^{3/2}}{f''(x)}$$

$$= \frac{\left[r^2 + \left(\frac{dr}{d\theta}\right)^2\right]^{3/2}}{r^2 + 2\left(\frac{dr}{d\theta}\right)^2 - r\frac{d^2r}{d\theta^2}}$$

The *curvature* (K) at (x,y) is $K = 1/R$.

The *center* of *curvature* corresponding to the point (x_1,y_1) on $y = f(x)$ is (h,k) where

$$h = x_1 - \frac{f'(x_1)\{1 + [f'(x_1)]^2\}}{f''(x_1)}$$

$$k = y_1 + \frac{1 + [f'(x_1)]^2}{f''(x_1)}$$

114. THEOREM OF THE MEAN. ROLLE'S THEOREM If $y = f(x)$ and its derivative $f'(x)$ are continuous on the interval (a,b), there exists a value of x somewhere between a and b such that

$$f(b) = f(a) + (b - a)f'(x_1) \qquad a < x_1 < b$$

Rolle's theorem is a special case of the theorem of the mean with $f(a) = f(b) = 0$; that is, there exists at least one value of x between a and b for which $f'(x_1) = 0$.

115. EVALUATION OF INDETERMINATE FORMS If $f(x)$ and $F(x)$ are two continuous functions of x having continuous derivatives, $f'(x)$ and $F'(x)$, then

(*a*) If $\lim_{x\to a} f(x) = 0$, $\lim_{x\to a} F(x) = 0$, and $\lim_{x\to a} F'(x) \neq 0$ [or if $\lim_{x\to a} f(x) = \lim_{x\to a} F(x) = \infty$], then

$$\lim_{x\to a}\frac{f(x)}{F(x)} = \lim_{x\to a}\frac{f'(x)}{F'(x)}$$

(*b*) If $\lim_{x\to a} f(x) = 0$ and $\lim_{x\to a} F(x) = \infty$ [that is, $F(x)$ becomes infinite as x approaches a as a limit], then $\lim_{x\to a}[f(x)F(x)]$ may often be determined by writing

$$f(x)F(x) = \frac{f(x)}{1/F(x)}$$

thus expressing the functions in the form of (*a*).

(*c*) If $\lim_{x\to a} f(x) = \infty$ and $\lim_{x\to a} F(x) = \infty$, then $\lim_{x\to a}[f(x) - F(x)]$ may often be determined by writing

$$f(x) - F(x) = \frac{[1/F(x)] - [1/f(x)]}{1/[f(x)F(x)]}$$

and using (*a*).

(*d*) The $\lim_{x\to a}[f(x)^{F(x)}]$ may frequently be evaluated upon writing

$$f(x)^{F(x)} = e^{F(x)\log_e f(x)}$$

When one factor of the last exponent approaches zero and the other becomes infinite, the exponent is of the type considered in (*b*).

Thus we are led to the indeterminate forms which are symbolized by $0/0$, ∞/∞, 0^0, 1^∞, ∞^0.

Handbook of Mathematical Tables and Formulas, 4th Ed. Richard Burington, McGraw-Hill 1965.

for $dy/dx = 0$

$R = \frac{1}{\frac{d^2y}{dx^2}}$

C.7-11

WHC-SD-W025-FDR-001, Rev. 1

10-5-17

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 5

Project Title: Non-Drag-Off Landfill Design Project

Project No. 893-1186.032

Calculation Title: Subgrade Side Slope Stability

	Name	Date
Prepared By:	Timothy L. Martin	1-13-90
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT: Subgrade Side Slope Stability

Job No. 993-1186.032 | Made by TLM | Date 1-13-90
Ref. | Checked FBS | Sheet 2 of 5
Reviewed RL

Objective: To determine side slope stability in the subgrade soils.

Reference: Stability Analysis of Earth Slopes, Huang, Y.H., 1983.

Approach: The subgrade soil is a cohesionless, drained, fine to medium sand with some silt-sized particles. Stability analysis can conservatively be performed using an infinite slope analysis.

Assumptions:
1) The side slope will be cut to a 3H:1V angle (18.4° ≈ 18°)
2) The slope will be fully drained.
3) Based on laboratory test data, $c = 0$, $\bar{\phi} = 38°$
4) Dynamic horizontal acceleration of 0.12g to be used in the analysis.

Analysis:

Infinite slope equation:

$$F.S. = \frac{(\bar{c}/\gamma d)\sec\alpha + [1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\phi'}{\sin\alpha + C_s \cos\alpha}$$

where $\bar{c}$ = effective cohesion
γ = unit weight of sliding mass
d = thickness (vertical) of sliding mass
α = slope angle
r_u = pore pressure ratio
C_s = horizontal acceleration
ϕ' = Effective Friction angle of slide mass

since $\bar{c} = 0$ and $r_u = 0$, and For a static analysis $C_s = 0$

Static Analysis:

$$FS = \frac{\tan\phi'}{\tan\alpha} = \frac{\tan 38}{\tan 18} = 2.4$$

For a dynamic Analysis, $C_s = 0.12g$

$$FS = \frac{[\cos\alpha - C_s \sin\alpha]\tan\phi}{\sin\alpha + C_s \cos\alpha} = \frac{[\cos 18 - (0.2)\sin 18]\tan 38}{\sin 18 + (0.12)\cos 18}$$

$FS = 1.69$ ✓

Golder Associates

SUBJECT Subgrade Side Slope Stability

Job No. 893-1186.012 | Made by TLM | Date 1-13-90

Ref. | Checked [illegible] | Sheet 3 of 5

Reviewed [illegible]

To determine the critical acceleration, i.e., the acceleration which will cause a FS = 1.0; insert FS = 1.0 and solve for C_s

$$FS = \frac{(\cos\alpha - C_s \sin\alpha)\tan\phi'}{\sin\alpha + C_s \cos\alpha}$$

$$1.0 = \frac{(\cos 18 - C_s \sin 18)\tan 38}{\sin 18 + C_s \cos 18}$$

$$1.0 = \frac{(0.951 - C_s\, 0.309)\, 0.781}{0.309 + C_s\, 0.951}$$

$$1.0 = \frac{0.743 - C_s\, 0.241}{0.309 + C_s\, 0.951}$$

$$0.309 + C_s\, 0.951 = 0.743 - C_s\, 0.241$$

$$C_s\, 1.192 = 0.434$$

$$C_s = 0.364 \approx \underline{0.36}$$ ✓ critical acceleration

Summary:

For slope stability, the minimum recommended factor of safety is 1.5

Static Analysis: FS = 2.40

Dynamic Analysis: FS = 1.69

Critical Acceleration C_s = 0.36g

Conclusion: Subgrade sideslope stability does not appear to be a problem. ✓

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6
Simplified Methods for Plane Failure

6.1 INFINITE SLOPES

The purpose of this chapter is to present some simple equations for determining the safety factor of slopes with plane failure surfaces. Only three simple cases will be considered: one involving an infinite slope with a failure plane parallel to the slope surface, one involving a triangular cross section with a single failure plane, and the other involving a trapezoidal cross section with two failure planes (Huang, 1977a, 1978b). For three failure planes, the computerized method presented in Chap. 8 should be used.

Figure 6.1 shows an infinite slope underlaid by a rock surface with an inclination, α. The slope is considered as infinite because it has a length much greater than the depth, d. If a free body of width a is taken, the forces on the two vertical sides are the same because every plane is considered the same in an infinite slope. As the side forces neutralize each other, the only forces to be considered are the weight, W, and the seismic force, C_sW. In contrast to the cylindrical failure, the effect of seismic force on the force

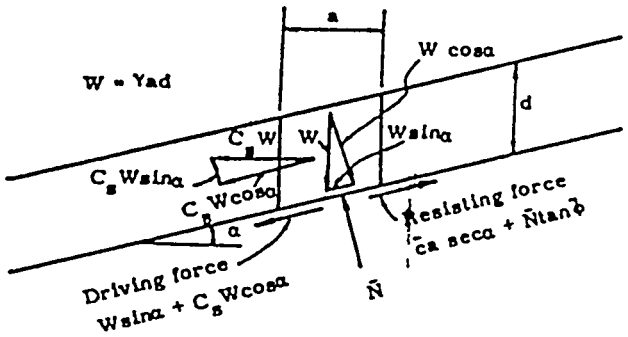


FIGURE 6.1. Analysis of infinite slope.

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normal to the failure plane and thus on the shear resistance is considered. In all the derivations which follow, only the effective stress analysis will be presented. The equations can also be applied to a total stress analysis by simply replacing the effective strength parameters by the total strength parameters.

The factor of safety is defined as a ratio of the resisting force due to the shear strength of soil along the failure surface to the driving force due to the weight of the sliding mass. The resisting force is composed of two parts: one due to cohesion and equal to $\bar{c}a$ sec α and the other due to friction and equal to $\bar{N}$ tan $\bar{\phi}$, where $\bar{N}$ is the effective force normal to the failure plane. With a pore pressure ratio, r_u,

$$\bar{N} = W\,[(1 - r_u)\cos\alpha - C_s \sin\alpha] \tag{6.1}$$

The driving force is always equal to the component of weight and seismic force parallel to the failure surface, or $W \sin\alpha + C_s W \cos\alpha$, regardless of whether seepage exists or not. Therefore, the factor of safety, F, can be written as

$$F = \frac{\bar{c}a \sec\alpha + W\,[(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{W(\sin\alpha + C_s \cos\alpha)} \tag{6.2}$$

Replacing W by $\gamma a d$, where γ is the unit weight of the sliding mass

$$F = \frac{(\bar{c}/\gamma d)\sec\alpha + [(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{\sin\alpha + C_s \cos\alpha} \tag{6.3}$$

Equation 6.3 is applicable to an infinite slope possessing both cohesion and angle of internal friction. The factor of safety decreases with the increase in d, so the most critical plane is along the rock surface.

If there is no cohesion ($\bar{c} = 0$), from Eq. 6.3

$$F = \frac{[(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{\sin\alpha + C_s \cos\alpha} \tag{6.4}$$

Equation 6.4 shows that the factor of safety for a cohesionless material is independent of d. Therefore, every plane parallel to the slope is a critical plane and has the same factor of safety. The failure will start from the slope surface where the soil particles will roll down the slope. If there is no seismic force, Eq. 6.4 can be simplified to

$$F = (1 - r_u)\frac{\tan\bar{\phi}}{\tan\alpha} \tag{6.5}$$

6.2 TRIANGULAR CROSS

Figure 6.2 shows a triangular
failure plane is along the bott
failure are the spoil banks c
ground surface is not properly
of the fill. In addition to the v
to $C_s W$ is also applied.

Similar to Eq. 6.2 for infi
is H csc α instead of a sec α

$$F = \frac{\bar{c}H \csc\alpha + }{}$$

and

$$W = \tfrac{1}{2}\gamma H^2 \csc\beta$$

in which γ is the unit weight

Substituting W from Eq. 6.7 i

$$F = \frac{2 \sin\beta \csc(\beta - \alpha)(\bar{c}/$$

Note that $\bar{c}$, γ, and H can
determining the factor of safet
height, H,

$$F = \frac{\bar{c}W_f \sin\beta/\sin(\beta - \alpha) +}{W \sin}$$

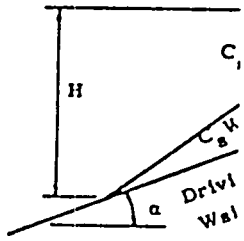


FIGURE 6.2

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11

Project Title: Non-Dmg-aFF LandFill Design

Project No. 893-1186.032

Calculation Title: Liner Side Slope Stability

	Name	Date
Prepared By:	Timothy L. Martin	January 24, 1990
Checked By:	Frank S. Shuri	January 30, 1990
Approved By:	MARK CORR	JAN 31, 1990
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

ER SIDE SLOPE STABILITY
Date: 30-Jan-90

Made by: TLM
Checked by: FSS
Reviewed by:

893-1186

Objective: To determine liner side slope stability independently of anchor trench capacity.

Reference: Stability Analysis of Earth Slopes, Huang, Y.H., 1983

Approach: Infinite slope analysis comparison of geosynthetic interface shear strengths.
Shear strengths from laboratory interface direct shear tests.

Assumptions:
1. Side slopes will be 3H:1V
2. The slope will be fully drained.
3. Dynamic acceleration of 0.12g used for dynamic analysis.

Analysis:

Infinite slope equation: FS = ((c/Gd)sec a + (cos a - Cssina)tan P)/(sin a + Cscos a)

Where: c = effective cohesion
G = unit weight of sliding mass
d = vertical thickness of sliding mass
a = slope angle
Cs = horizontal acceleration
P = friction angle

Based on current design:

	G =	120	pcf
	d =	2	ft
	a =	18	
	Cs =	0.12	g
Liner interface: Soil/geotextile	c =	0	CRITICAL INTERFACE ✓
	P =	30	

Note: c,P are residual strength parameters

Static Analysis:	FS =	1.78 ✓
Dynamic Analysis:	FS =	1.25 ✓

ER SIDE SLOPE STABILITY
Date: 30-Jan-90

Made by: TLM
Checked by: FSB
Reviewed by:

893-1186

Objective: To determine liner side slope stability independently of anchor trench capacity.

Reference: Stability Analysis of Earth Slopes, Huang, Y.H., 1983

Approach: Infinite slope analysis comparison of geosynthetic interface shear strengths. Shear strengths from laboratory interface direct shear tests.

Assumptions:
1. Side slopes will be 3H:1V
2. The slope will be fully drained.
3. Dynamic acceleration of 0.12g used for dynamic analysis.

Analysis:

Infinite slope equation: FS = $(c/Gd)\sec a + (\cos a - Cs \sin a \tan P)/(\sin a + Cs \cos a)$

Where:
c = effective cohesion
G = unit weight of sliding mass
d = vertical thickness of sliding mass
a = slope angle
Cs = horizontal acceleration
P = friction angle

Based on current design:

	G =	120 pcf
	d =	2 ft
	a =	18
	Cs =	0.12 g
Liner interface: Acrylic/HDPE (text)	c =	320
	P =	36

Note: c,P are residual strengths

Static Analysis:	FS =	6.77	✓
Dynamic Analysis:	FS =	4.88	✓

Pg 4 of 11

893-1186

ER SIDE SLOPE STABILITY
Date: 30-Jan-90

Made by: TJM
Checked by: FSS
Reviewed by:

Objective: To determine liner side slope stability independently of anchor trench capacity.

References: Stability Analysis of Earth Slopes, Huang, Y.H., 1983

Approach: Infinite slope analysis comparison of geosynthetic interface shear strengths. Shear strengths from laboratory interface direct shear tests.

Assumptions:
1. Side slopes will be 3H:1V
2. The slope will be fully drained.
3. Dynamic acceleration of 0.12g used for dynamic analysis.

Analysis:

Infinite slope equation: FS = (c/G*sec a + (cos a - Cs*sin a*tan P)/(sin a + Cs*cos a)

Where:
c = effective cohesion
G = unit weight of sliding mass
t = vertical thickness of sliding mass
a = slope angle
Cs = horizontal acceleration
P = friction angle

Based on current design:

	G =	120 pcf
	c =	2 ft
	a =	18
	Cs =	0.12 g
Liner interface: Geocomp/HDPE textc =		188
	P =	22

Note: c,P peak = residual, no distinct peak

Static Analysis:	FS =	3.91	✓
Dynamic Analysis:	FS =	2.82	✓

Pg 5 of 11

893-1186

NON-DRAG-OFF LANDFILL DESIGN
(ER SIDE SLOPE STABILITY
Date: 24-Jan-90

Project No.: 893-1186
Made by: TLM
Checked by: FSS
Reviewed by:

Objective: To determine liner side slope stability independently of anchor trench capacity.

Reference: Stability Analysis of Earth Slopes, Huang, Y.H., 1983

Approach: Infinite slope analysis comparison of geosynthetic interface shear strengths.
Shear strengths from laboratory interface direct shear tests.

Assumptions: 1. Side slopes will be 3H:1V
2. The slope will be fully drained.
3. Dynamic acceleration of 0.12g used for dynamic analysis.

Analysis:

Infinite slope equation: FS = (c/Gdsec a + (cos a - Cs*sin a)tan P)/(sin a + Cs*cos a)

Where: c = effective cohesion
G = unit weight of sliding mass
d = vertical thickness of sliding mass
a = slope angle
Cs = horizontal acceleration
P = friction angle

Based on current design:

G = 120 pcf
d = 2 ft
a = 18
Cs = 0.12 g

Liner interface: Geotextile (text)
c = 195
P = 12

Note: c, P are residual strengths

Static Analysis:	FS =	3.42 ✓
Dynamic Analysis:	FS =	2.48 ✓

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C.9-6

6
Simplified Methods for Plane Failure

6.1 INFINITE SLOPES

The purpose of this chapter is to present some simple equations for determining the safety factor of slopes with plane failure surfaces. Only three simple cases will be considered: one involving an infinite slope with a failure plane parallel to the slope surface, one involving a triangular cross section with a single failure plane, and the other involving a trapezoidal cross section with two failure planes (Huang, 1977a, 1978b). For three failure planes, the computerized method presented in Chap. 8 should be used.

Figure 6.1 shows an infinite slope underlaid by a rock surface with an inclination, α. The slope is considered as infinite because it has a length much greater than the depth, d. If a free body of width a is taken, the forces on the two vertical sides are the same because every plane is considered the same in an infinite slope. As the side forces neutralize each other, the only forces to be considered are the weight, W, and the seismic force, C_sW. In contrast to the cylindrical failure, the effect of seismic force on the force

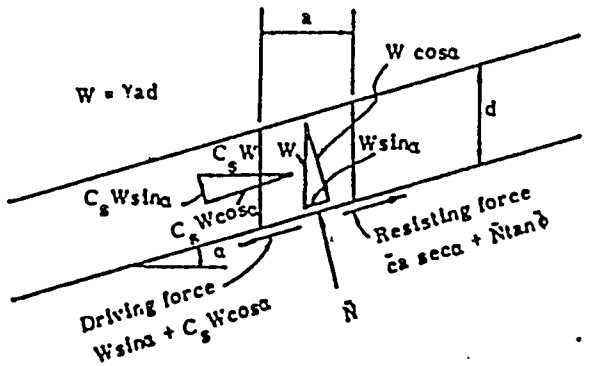


FIGURE 6.1. Analysis of infinite slope.

normal to the failure plane and thus on the shear resistance is considered. In all the derivations which follow, only the effective stress analysis will be presented. The equations can also be applied to a total stress analysis by simply replacing the effective strength parameters by the total strength parameters.

The factor of safety is defined as a ratio of the resisting force due to the shear strength of soil along the failure surface to the driving force due to the weight of the sliding mass. The resisting force is composed of two parts: one due to cohesion and equal to $\bar{c}a$ sec α and the other due to friction and equal to $\bar{N}$ tan $\bar{\phi}$, where $\bar{N}$ is the effective force normal to the failure plane. With a pore pressure ratio, r_u,

$$\bar{N} = W\,[(1 - r_u)\cos\alpha - C_s \sin\alpha] \tag{6.1}$$

The driving force is always equal to the component of weight and seismic force parallel to the failure surface, or $W \sin\alpha + C_s W \cos\alpha$, regardless of whether seepage exists or not. Therefore, the factor of safety, F, can be written as

$$F = \frac{\bar{c}a \sec\alpha + W\,[(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{W(\sin\alpha + C_s \cos\alpha)} \tag{6.2}$$

Replacing W by $\gamma a d$, where γ is the unit weight of the sliding mass

$$F = \frac{(\bar{c}/\gamma d)\sec\alpha + [(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{\sin\alpha + C_s \cos\alpha} \tag{6.3}$$

Equation 6.3 is applicable to an infinite slope possessing both cohesion and angle of internal friction. The factor of safety decreases with the increase in d, so the most critical plane is along the rock surface.

If there is no cohesion ($\bar{c} = 0$), from Eq. 6.3

$$F = \frac{[(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{\sin\alpha + C_s \cos\alpha} \tag{6.4}$$

Equation 6.4 shows that the factor of safety for a cohesionless material is independent of d. Therefore, every plane parallel to the slope is a critical plane and has the same factor of safety. The failure will start from the slope surface where the soil particles will roll down the slope. If there is no seismic force, Eq. 6.4 can be simplified to

$$F = (1 - r_u)\frac{\tan\bar{\phi}}{\tan\alpha} \tag{6.5}$$

6.2 TRIANGULAR CROSS SECTION

Figure 6.2 shows a triangular fill on a sloping surface. It is assumed that the failure plane is along the bottom of the fill. Good examples for this type of failure are the spoil banks created by surface mining, where the original ground surface is not properly scalped and a weak layer exists at the bottom of the fill. In addition to the vertical weight, a horizontal seismic force equal to $C_s W$ is also applied.

Similar to Eq. 6.2 for infinite slope except that the length of failure plane is H csc α instead of a sec α, the factor of safety is

$$F = \frac{\bar{c}H \csc\alpha + W[(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{W \sin\alpha + C_s W \cos\alpha} \tag{6.}$$

and

$$W = \tfrac{1}{2}\gamma H^2 \csc\beta \csc\alpha \sin(\beta - \alpha) \tag{6.}$$

in which γ is the unit weight of soil and β is the angle of outslope.

Substituting W from Eq. 6.7 into Eq. 6.6

$$F = \frac{2 \sin\beta \csc(\beta - \alpha)(\bar{c}/\gamma H) + [(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{\sin\alpha + C_s \cos\alpha} \tag{6.}$$

Note that $\bar{c}$, γ, and H can be lumped together as a single parameter in determining the factor of safety. If the fill width, W_f, is given instead of the height, H,

$$F = \frac{\bar{c}W_f \sin\beta/\sin(\beta - \alpha) + W[(1 - r_u)\cos\alpha - C_s \sin\alpha]\tan\bar{\phi}}{W \sin\alpha + C_s W \cos\alpha} \tag{6.}$$

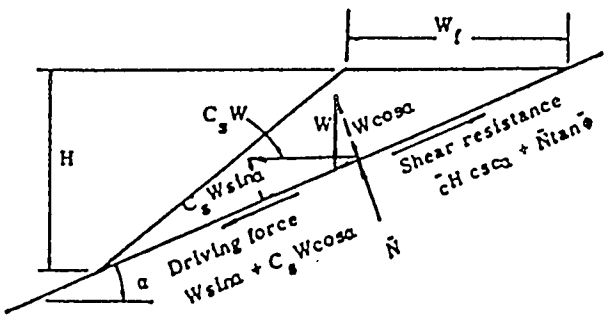
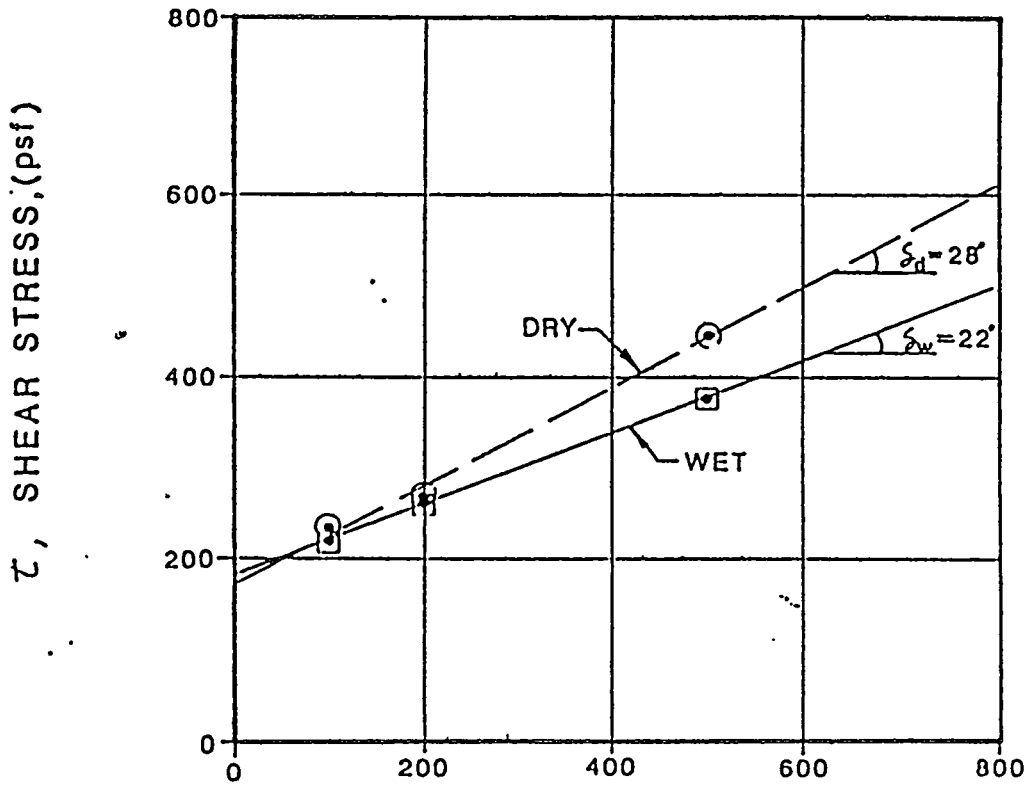


FIGURE 6.2. Plane failure of triangular fill.

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893-1186

TEXNET GEOCOMPOSITE vs HDT GEOMEMBRANE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – TEXNET GEOCOMPOSITE
LOWER SPECIMEN – HDT GEOMEMBRANE
SUBSTRATUM – PLATES

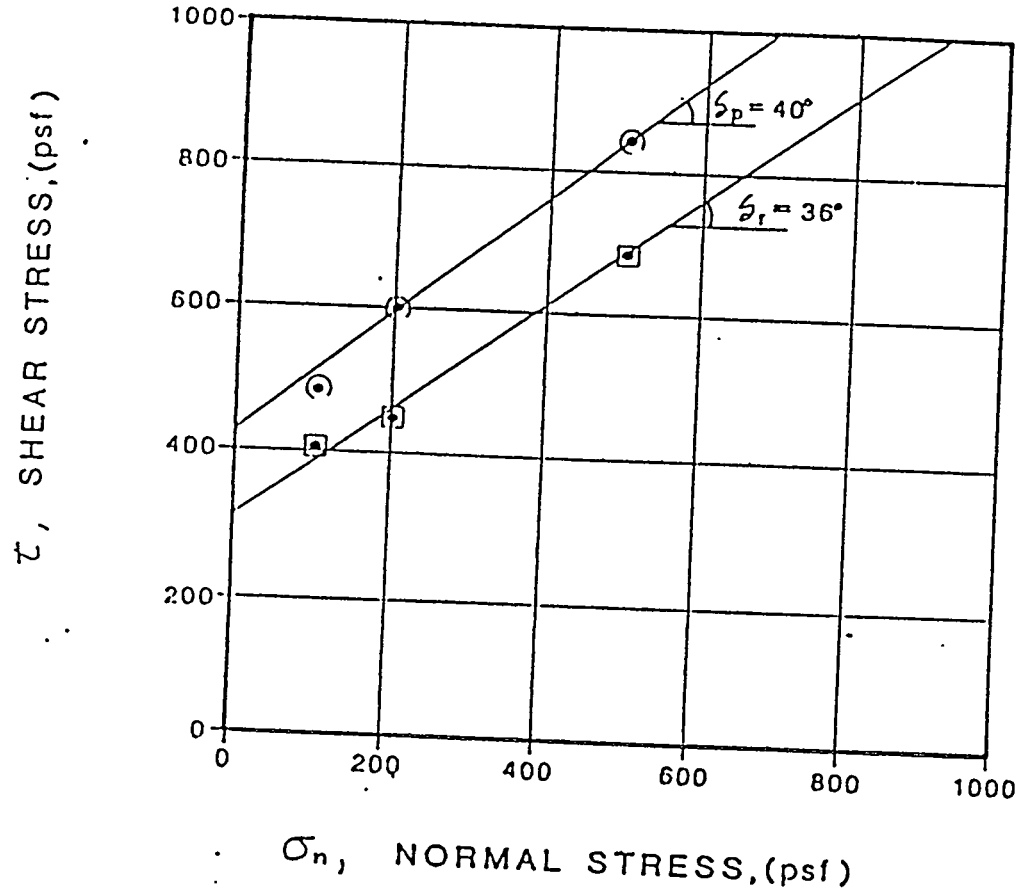
GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

Westinghouse

PROJ. NO. 4147-90-104 FIG NO. 3

893-1186

ADMIX vs HDT GEOMEMBRANE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – ADMIX
LOWER SPECIMEN – HDT GEOMEMBRANE
SUBSTRATUM – PLATES

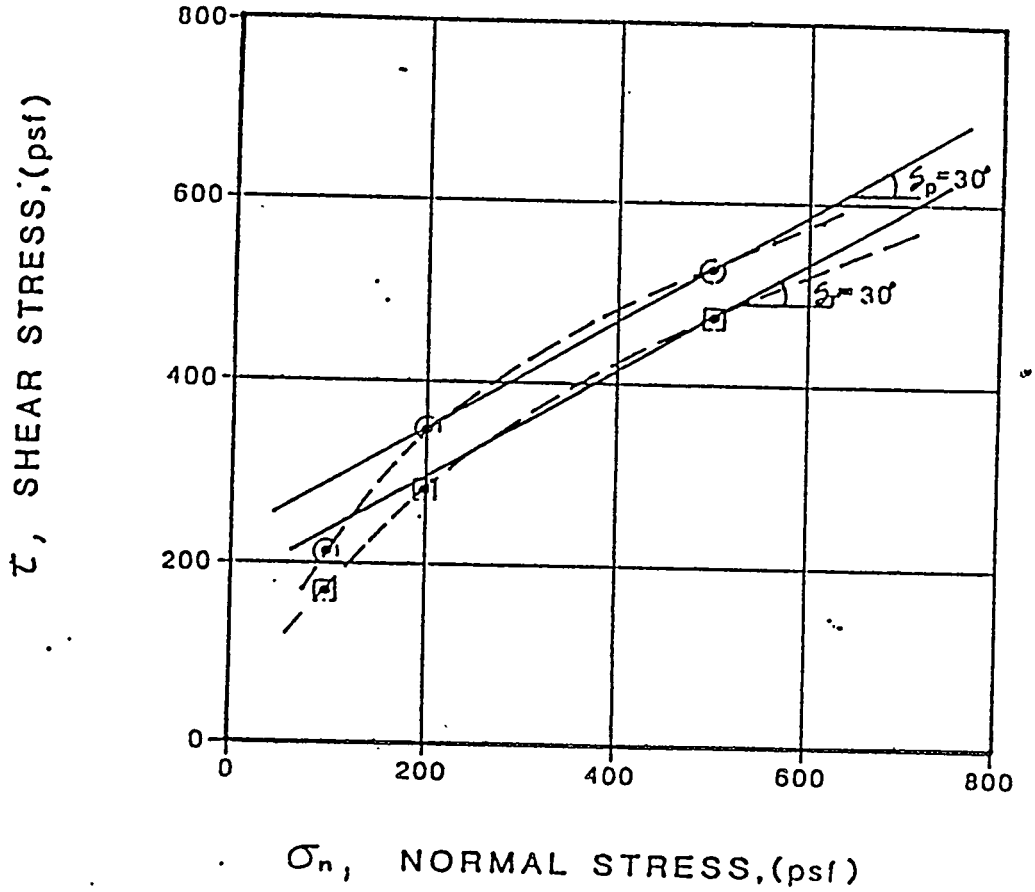
GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

Westinghouse

PROJ. NO. 4147-90-104 FIG. NO. 2

893-1186

SANDY SOIL vs TEXNET GEOCOMPOSITE



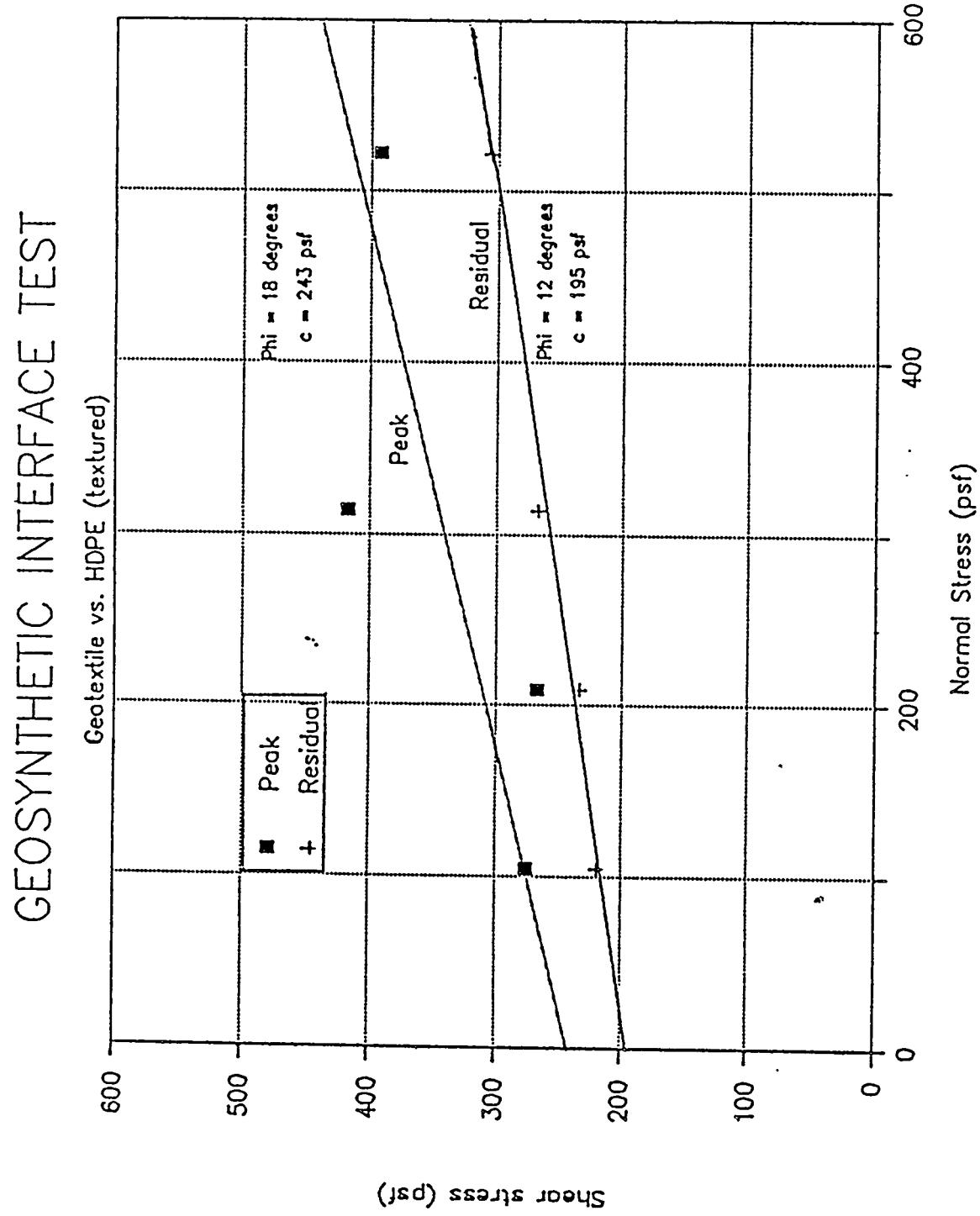
SUPERSTRATUM – PLATES
UPPER SPECIMEN – SANDY SOIL
LOWER SPECIMEN – TEXNET GEOCOMPOSITE
SUBSTRATUM – PLATES

GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

Westinghouse

PROJ. NO. 4147-90-104 FIG. NO. 2

893-1186


GEOSYNTHETIC INTERFACE TEST
Geotextile vs. HDPE (textured)
Peak
Residual
Phi = 18 degrees
c = 243 psf
Phi = 12 degrees
c = 195 psf
Peak
Residual
Shear stress (psf)
Normal Stress (psf)
0
100
200
300
400
500
600
0
200
400
600

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 5

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Liner Tension Caused by Thermal Contraction

	Name	Date
Prepared By:	Timothy L. Martin	December 14, 1989
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

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SUBJECT: Liner tension caused by thermal contraction		
Job No. 893-1186.032	Made by RM	Date 12-14-89
Ref.	Checked FS	Sheet 2 of 5
	Reviewed RL	

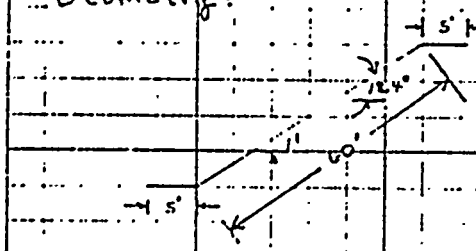
Objective: To determine the contraction expansion potential of HDPE and to estimate the maximum stress caused in the liner.

Approach: To estimate the expansion potential of the HDPE liner over a typical sideslope liner length using published thermal expansion coefficient for HDPE.

Reference: Poly Pipe Design and Engineering manual, Fig. 1 Nominal Physical Properties HDPE.

Assumptions:
- The maximum slope length for HDPE slope panel is approximately 70 Ft.
- Coefficient of Thermal expansion $9 \times 10^{-5} \frac{in}{in \cdot °F}$ -> E = 113,000 psi
- Assume a temperature differential of 100 F.
- 60 mil HDPE (textured) > S.G. = 0.945 — w/o carbon black, but Gundle specs 0.94 (typ.), so OK. FS
- ϕ HDPE/Admix ≈ 20° — Assumed conservative FS

Geometry:



total panel slope length: 70 Ft

(70 Ft)(12 in) = 840 in

$\Delta L = (840 in)(9 \times 10^{-5} \frac{in}{in \cdot °F})(100 °F)$

$\Delta L = 7.56$ inches

$\frac{\Delta L}{L} = \frac{7.56}{840} = 0.009 < 1\%$, 0-10% strain is the elastic range, See ASTM D638 table.

As the liner heats up and expands it moves down the slope as it contracts it must pull itself up the slope. The anchor trench must have enough capacity to hold the liner in place during the contraction phase. The calculation of that capacity is as follows:

Assume not anchored at toe.

The Weight of the liner: $W = (60 mils)(\frac{1 in}{1000 mils})(\frac{1 Ft}{12 in})(0.945 \times 62.4)(70)(1)$

$= 20.64 \frac{lbs}{Ft}$ of width

tangential force,	W_T =	20.64 sin 18.4 = 6.52 lbs	Down slope force
normal force	W_n =	20.64 cos 18.4 = 19.58 lbs	Acts like drag force
Resisting force	F_R =	19.58 tan 20 = 7.13 lbs	
Total force =	$W_T + F_R$ =	6.52 + 7.13 = 13.65 lbs/Ft	

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SUBJECT: Liner tension caused by thermal contraction

Job No. 893-1186.032 | Made by TLM | Date 12-14-89

Ref. | Checked FES | Sheet 3 of 5 Mod

Reviewed TLK

Assume for a moment that the liner was placed and anchored in hot weather so that the liner could not slip at either anchorage point. Then the temperature dropped 100°F in the liner, not an uncommon occurance. What stress would the liner exhibit?

anchored at toe and crest

$E = \frac{\sigma}{\varepsilon}$ given that $E = 113{,}000$ psi

$\varepsilon = 0.009$

$\sigma = (113{,}000)(0.009)$

$\sigma = 1{,}017$ psi ✓

In terms of anchor trench capacity:

$(0.060\ in)(12) = 0.72\ in^2$

$(0.72\ in^2)\left(1{,}017\ \frac{lbs}{in^2}\right) = 732\ \frac{lbs}{Ft}$ ✓

Conclusion: During construction it is not practical to construct an anchor trench with ≈700 lbs/Ft capacity. In fact, some movement would mitigate the thermal stress potential. Recommend leaving some slack in the liner when deploying in hot weather. ✓

Note: The thermal expansion and contraction of the HDPE liner will impact the anchor trench capacity during the construction phase of the landfill. When the operations layer is placed on the liner system it will insulate the HDPE, protecting it from extreme temperature fluctuations.

Even under a worst case scenario with the liner firmly anchored at the toe and crest and stretched tight during hot weather, the yield within the liner is approximately 1% which is much less than the 10% elongation at yield. For this reason it is not necessary to design the anchor trench to allow the HDPE liner to slip before yield strength is reached. ✓

made by: TKM
checked: FSS
Reviewed: PK

TABLE 5. MATERIAL PROPERTIES

HIGH DENSITY POLYETHYLENE (HDPE)

Property	Test Method	Unsupported(U) 30	36	40	50	60	80	100
Gauge (nominal)	—	30	36	40	50	60	80	100
Thickness, mils minimum ±10%	ASTM D1593 Para 8.1.3	27	32	36	45	54	72	90
Specific Gravity (minimum)	ASTM D792 Method A	0.940	0.940	0.940	0.940	0.940	0.940	0.940
Minimum Tensile Properties (each direction)	ASTM D638							
1. Tensile Strength Yield (lb/in width)		60	60	70	90	120	140	180
2. Tensile Strength at Break (lb/in width)		90	108	120	150	180	240	300
3. Elongation at Yield (percent)		10	10	10	10	10	10	10
4. Elongation at Break (percent)		500	500	500	500	500	500	500
5. Modulus of Elasticity (lbs/sq in)		80,000	80,000	80,000	80,000	80,000	80,000	80,000
Tear Resistance (lb, minimum)	ASTM D1004 Die C	15	18	20	25	30	40	50
Low Temperature, °F	ASTM D746 Procedure B	-40	-40	-40	-40	-40	-40	-40
Dimensional Stability (each direction, percent change maximum)	ASTM D1204 212°F, 15 min.	±3	±3	±3	±3	±3	±3	±3
Resistance to Soil Burial (percent change maximum in original value)	ASTM D3083 (as modified in Appendix A)							
1. Tensile Strength Yield		10	10	10	10	10	10	10
2. Tensile Strength at Break		10	10	10	10	10	10	10
3. Elongation at Yield		10	10	10	10	10	10	10
4. Elongation at Break		10	10	10	10	10	10	10
5. Modulus of Elasticity		10	10	10	10	10	10	10
Environmental Stress Crack (minimum, hours)	ASTM D1693 (as modified in Appendix A)	500	500	500	500	500	500	500
Factory and Field Seam Requirements[1]								
Bonded Seam Strength (factory seam, breaking factor, ppi width)	ASTM D3083 (as modified in Appendix A)	45	54	63	81	108	108	135
Peel Adhesion (lb/in minimum)	ASTM D413 (as modified in Appendix A)	FTB[2]	FTB[2]	FTB[2]	FTB[2]	FTB[2]	FTB[2]	FTB[2]
Dead Load								
Room Temperature 73°F 50% Bonded Seam Load	See Appendix A	Pass	Pass	Pass	Pass	Pass	Pass	Pass
Elevated Temperature 158°F 25% Bonded Seam Load	See Appendix A	Pass	Pass	Pass	Pass	Pass	Pass	Pass
Resistance to Soil Burial	ASTM D3083 (as modified in Appendix A)							
Peel Adhesion		FTB[2]	FTB[2]	FTB[2]	FTB[2]	FTB[2]	FTB[2]	FTB[2]
Bonded Seam Strength (percent change maximum in original value)		-10	-10	-10	-10	-10	-20	-20

[1]Factory bonded seam strength is the responsibility of the fabricator. Factory seams are further idscussed in Item 4.2.

[2]FTB - Film Tear Bond

This table provides a reliable and dependable means for industry to furnish such materials of known and consistent quality. Before selection of any material for an FML, however, the user should consult with appropriate manufacturers, because these materials may not be appropriate for every application. Specific information should be obtained from the manufacturers garding installation requirements, exposure conditions, performance expectations, and experience factor.

Job No: [illegible]
made by: TLM
Checked: FJS
Reviewed: JDL
5 of 45

Fig. 1
NOMINAL PHYSICAL PROPERTIES
POLY PIPE PE 3408 Pipe Material

Property	ASTM Reference	Nominal Value
Density, black	D 1505	0.956 gm/cc
Density, natural	D 1505	0.945 gm/cc
Melt flow	D 1238 (cond. E)	0.10 gm/10min.
Vicat softening temperature	D 1525	255°F
Brittleness temperature	D 746	◄– 180°F
Environmental stress cracking	D 1693 (cond. C)	►5000 hrs.
Tensile, yield (2"/min.)‡	D 638	►3200 psi
Tensile, break (2"/min.)	D 638	►5000 psi
Elongation (2"/min.)	D 638	►800%
Stiffness	D 747	62,000 psi
Modulus of elasticity	D 638	113,000 psi
Flexural modulus	D 3350	136,000 psi
Impact strength, 0.25"	D 256 (method A)	12
Impact strength, 0.125"	D 256 (method A)	Non-break
Hardness, shore D	D 2240	66
Thermal conductivity	C 177	2.7 Btu-in/ft.²/hr./°F
Thermal expansion coefficient	D 696	$9x10^{-5}$in/in/°F
Long term hydrostatic strength	D 2837	►1600 psi
Hydrostatic design basis	D 2837	1600 psi
Hydrostatic design stress	D 2837	800 psi
Material cell* classification	D 3350	PE 345434C

‡Rate of cooling of test sheet per ASTM D1928, procedure C.

*For explanation of Poly Pipe cell classification see page A-10.

A-7

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Calculation Cover Sheet

Sheet 1 of 3

Project Title: Non-Drag-OFF Landfill Design

Project No. 893-1186.032

Calculation Title: Anchor Trench Pullout Resistance

	Name	Date
Prepared By:	Timothy L. Martin	December 14,15, 1989
Checked By:	Fayek S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

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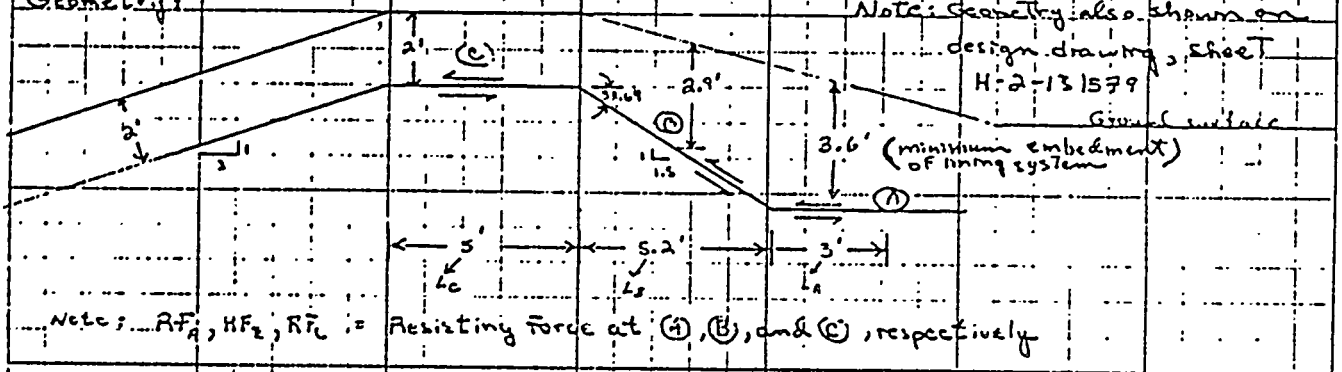
SUBJECT Calculation of Anchor Trench Pullout Resistance		
Job No. 893-1186.035 Ref.	Made by TLM Checked RS see note Reviewed [illegible]	Date 12-14-89 Sheet 2 of 3

Objective: To calculate the anchor trench pullout resistance on the completed anchor trench system.

Approach: To calculate the frictional resistance along the anchor trench and crest sections of the liner system.

Ref: Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments., Richardson, G.N. and Koerner, R.M.

Geometry:



Note: Geometry also shown on design drawing, Sheet H-2-131579

Note: RF_A, RF_B, RF_C = Resisting force at (A), (B), and (C), respectively

Assumptions: The weakest shear interface has a friction angle of 20°.

γ_s = 120 pcf

F.S. = 1.5

Calculate Resisting Forces

$RF_A = q L_A \tan\delta$ where, $q = (120)(3.6) = 432$ psf

$= (432)(3.0)(\tan 20)$

$RF_A = 471.7$ lbs/ft ✓

$RF_B = q L_B \tan\delta$ where $q = (120)(2.9) = 348$ psf

$= (348)\left(\frac{5.2}{\cos 33.69}\right)(\tan 20)$

$RF_B = 791.6$ lbs/ft ✓

$RF_C = q L_C \tan\delta$ where, $q = (120)(2) = 240$ psf ✓

$= (240)(5.0)(\tan 20)$

$RF_C = 436.8$ ✓

Total Resisting Force = ΣRF_{A+B+C} = 1700.1 lbs/ft

Driving Force

The driving force both during and after construction will be non-existent unless an interface friction angle on the 3H:1V slope is less than 18.4 degrees, (i.e., $\tan^{-1}(\frac{1}{3}) = 18.4°$). The one possibility for a driving

Golder Associates

SUBJECT Calculations of Anchor Trench Pullout Resistance

Job No. 891-1186.025 | Made by TLM | Date 12-15-89
Ref. | Checked FGS | Sheet 3 of 3
Reviewed

(continued)

Force is the expansion-contraction potential caused by installing a liner tight during construction in hot weather and anchoring both ends prior to the arrival of cooler temperatures. See the attached calculations and conclusions entitled "Liner Tension caused by thermal Contraction". After construction and placement of the operations layer thermal expansion and contraction is expected to be negligible due to the insulating nature of the soil cover.

Note: If the geomembrane is pulled and the overlying soil does not move with it, the membrane/soil interaction is a double shear situation, and the anchorage capacity would be $2 \times 1700 = 3400$ lbs/ft. Although the behavior in place would probably not be a full double shear situation, this does indicate that 1700 lbs/ft is a lower bound for pullout capacity. On the other hand, the tensile strength at yield (10% strain - which may be unacceptably high) is 1440 lbs/ft for 60 mil HDPE (see spec sheet attached to thermal expansion calcs). Strength would probably be less for 60 mil textured, and allowable load would certainly be less at lower elongations. Hence, even this modest anchor trench design has far too much capacity to allow slippage should the membrane be leaked. As discussed above, this is not expected to be a problem if placement of the operations layer is from the bottom upwards and if the interface friction angles are as assumed. Anchorage that would allow slip is not considered practical from a construction point-of-view. Consequently, the proposed design is considered acceptable subject to the cautions discussed here.

FGS

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11

Project Title: Non-Drag-OFF Land Fill Design

Project No. 893-1186.032

Calculation Title: Ramp Stability Analysis

	Name	Date
Prepared By:	Timothy L. Martin	January 30, 1990
Checked By:	Frank S. Shuri	January 31, 1990
Approved By:	[illegible]	Jan 31, 1990
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

RAMP STABILITY ANALYSIS
Jan-90
...OJECT NUMBER: 893-1186

Prepared by: TLM
Checked by: FSC
Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments, Richardson, G.N. and Koerner, R.M. EPA/600/2-87/097

DEFINE DRIVING FORCES:

Ws = Weight of the road base
Wv = Weight of the vehicle
Fb = Vehicle braking force (0.3*Wv)

DEFINE RESISTING FORCES:

Fr = Frictional resistance at base of roadway
= (Wv+Ws)(cos slope angle)(tan Phi)
Fa = Adhesion force
= roadway width * roadway length * cohesion

FACTOR OF SAFETY = (Fr + Fa)/((Ws+Wv)*(sin slope angle) + Fb)

ANALYSIS DATA: (Based on laboratory geosynthetic interface testing program)

Equipment weight: 200000 pounds (100 ton load; largest load estimate by WHC)

Friction/Adhesion:		Phi	C	
	soil/geotextile	30	0	Residual Strength Parameters CRITICAL INTERFACE

Ramp Geometry:

width:	30	ft
length:	250	ft
thickness	2	ft
slope:	4.57	degrees (8 percent)
density:	130	pcf

Ramp Subbase:

density:	130	pcf
Phi:	36	degrees
c:	0	psf

ANALYSIS:

Ws =	1950000	pounds
Wv =	200000	pounds
Fb =	60000	pounds
Fa =	0	pounds
Fr =	1237357	pounds
FS =	7.22 ✓	static stability
FS =	5.35 ✓	dynamic stability (braking force)

Recommended minimum FS = 3 static analysis
FS = 2 dynamic analysis

RAMP STABILITY ANALYSIS
Jan-90
...JECT NUMBER: 893-1186

Prepared by: TJM
Checked by: FSS
Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments, Richardson, G.N. and Koerner, R.M.

DEFINE DRIVING FORCES:

Ws = Weight of the road base
Wv = Weight of the vehicle
Fb = Vehicle braking force (0.3*Wv)

DEFINE RESISTING FORCES:

Fr = Frictional resistance at base of roadway
= (Wv+Ws)(cos slope angle)(tan Phi)
Fa = Adhesion force
= roadway width * roadway length * cohesion

FACTOR OF SAFETY = (Fr + Fa)/((Ws+Wv)*(sin slope angle) + Fb)

ANALYSIS DATA: (Based on laboratory geosynthetic interface testing program)

Equipment weight: 200000 pounds (100 ton load; largest load estimate by WHC)

Friction/Adhesion:		Phi	C
	admix/HDPE (tex)	36	320 Residual Strength Parameters

Ramp Geometry:

width:	30 ft
length:	250 ft
thickness	2 ft
slope:	4.57 degrees (8 percent)
density:	130 pcf

Ramp Subbase:

density:	130 pcf
Phi:	36 degrees
c:	0 psf

ANALYSIS:

Ws =	1950000 pounds	
Wv =	200000 pounds	
Fb =	60000 pounds	
Fa =	2400000 pounds	
Fr =	1557100 pounds	
FS =	22.10 ✓	static stability
FS =	17.11 ✓	dynamic stability (braking force)

Recommended minimum FS = 3 static analysis
FS = 2 dynamic analysis

STABILITY ANALYSIS — Prepared by: TZM
Jan-90 — Checked by: FDS
PROJECT NUMBER: B93-1186 — Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments, Richardson, G.N. and Koerner, R.M. EPA/600/2-87/097

DEFINE DRIVING FORCES:

Ws = Weight of the road base
Wv = Weight of the vehicle
Fb = Vehicle braking force (0.3*Wv)

DEFINE RESISTING FORCES:

Fr = Frictional resistance at base of roadway
= (Wv+Ws)(cos slope angle)(tan Phi)
Fa = Adhesion force
= roadway width * roadway length * cohesion

FACTOR OF SAFETY = (Fr + Fa)/((Ws+Wv)*(sin slope angle) + Fb)

ANALYSIS DATA: (Based on laboratory geosynthetic interface testing program)

Equipment weight: 200000 pounds (100 ton load; largest load estimate by WHC)

Friction/Adhesion:	Phi	C	
HDPE/geotextile	12	195	Residual Strength Parameters

Ramp Geometry:

width:	30	ft
length:	250	ft
thickness	2	ft
slope:	4.57	degrees (8 percent)
density:	130	pcf

Ramp Subbase:

density:	130	pcf
Phi:	38	degrees
c:	0	psi

ANALYSIS:

Ws =	1950000	pounds	
Wv =	200000	pounds	
Fb =	60000	pounds	
Fa =	1462500	pounds	
Fr =	455544	pounds	
FS =	11.20	✓	static stability
FS =	8.29	✓	dynamic stability (braking force)

Recommended minimum FS = 3 static analysis
FS = 2 dynamic analysis

RAMP STABILITY ANALYSIS
Jan-90
...OJECT NUMBER: B93-1186

Prepared by: TLM
Checked by: RBS
Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments, Richardson, G.N. and Koerner, R.M.

DEFINE DRIVING FORCES:

Ws = Weight of the road base
Wv = Weight of the vehicle
Fb = Vehicle braking force (0.3*Wv)

DEFINE RESISTING FORCES:

Fr = Frictional resistance at base of roadway
= (Wv+Ws)(cos slope angle)(tan Phi)
Fa = Adhesion force
= roadway width * roadway length * cohesion

FACTOR OF SAFETY = (Fr + Fa)/((Ws+Wv)*(sin slope angle) + Fb)

ANALYSIS DATA: (Based on laboratory geosynthetic interface testing program)

Equipment weight: 200000 pounds (100 ton load; largest load estimate by WHC)

Friction/Adhesion:		Phi	C
	HDPE/geocomposite	22	188 Residual ~ to peak strength parameters

Ramp Geometry:

width:	30	ft
length:	250	ft
thickness	2	ft
slope:	4.57	degrees (8 percent)
density:	130	pcf

Ramp Subbase:

density:	130	pcf
Phi:	38	degrees
c:	0	psf

ANALYSIS:

Ws =	1950000	pounds
Wv =	200000	pounds
Fb =	60000	pounds
Fa =	1410000	pounds
Fr =	805895	pounds
FS =	13.29 ✓	static stability
FS =	9.84 ✓	dynamic stability (braking force)

Recommended minimum FS = 3 static analysis
FS = 2 dynamic analysis

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SUBJECT Ramp stability Analysis

Job No. 893-1186.032 | Made by PSS | Date 1-8-90

Ref. | Checked TLM | Sheet 6 of 11

Reviewed

What ϕ gives FS = 3 for static?

$$FS = \frac{(W_1+W_3)\cos\alpha \tan\phi}{(W_1+W_3)\sin\alpha}$$

$$FS = \frac{\tan\phi}{\tan i}$$

$$\phi = ATN(FS \tan i)$$

$$= ATN(3 \times \tan 4.57)$$

$$= 13.5°$$

What ϕ gives FS = 2 for dynamic?

$$FS = \frac{(W_1+W_3)\cos\alpha \tan\phi}{(W_1+W_3)\sin\alpha + F_B}$$

$$\tan\phi = \frac{FS((W_1+W_3)\sin\alpha + F_B)}{(W_1+W_3)\cos\alpha}$$

$$= 12.2°$$

So any system w/ $\phi > 13.5°$ is OK.

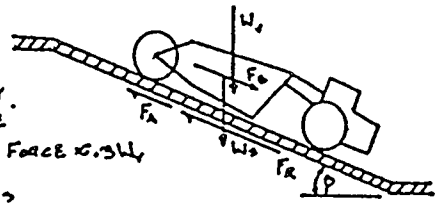
Lab results are much higher than this, even allowing for uncertainty, so design is acceptable.

Cell Component: RAMP

Consideration: SLIDING - VERIFY THAT RAMP SUB-BASE IS STABLE UNDER LOAD.

Required Material Properties	Range	Test	Standard
Soil-LCR FRICTION ANGLE, δ_{sl}	20 - 35°	DIRECT SHEAR	BOTH PERFORMED
Soil - FML FRICTION ANGLE, δ_{FML}	14 - 22°		
LCR - FML FRICTION ANGLE, δ_{FL}	10-18°		

Analysis Procedure:



(1) DEFINE DRIVING FORCES

W_R = WEIGHT OF ROADWAY
W_v = WEIGHT OF VEHICLE
F_B = VEHICLE BRAKING FORCE = $0.3W_v$

(2) DEFINE RESISTING FORCES

F_R = FRICTIONAL FORCE @ BASE OF ROADWAY
$= (W_R + W_v) \times \cos\beta \times \tan\delta_{MIN}$

WHERE δ_{MIN} IS THE MINIMUM FRICTION INTERFACE ANGLE

F_A = ADHESION FORCE
$= WL \times C_{MIN}$ W = RAMP WIDTH L = RAMP LENGTH

WHERE C_{MIN} IS THE MINIMUM INTERFACE ADHESION

(3) DEFINE DESIGN RATIO, DR

$$DR = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

$$DR = \frac{F_A + F_R}{(W_R + W_v)\cos\beta + F_B} \quad \text{(DYNAMIC W/ } F_B\text{)}$$

Design Ratio:

DR_{MIN} = 3.0 WITH STATIC LOADS
2.0 WITH DYNAMIC LOADS

References:

Example:

GIVEN:

- EQUIPMENT = 55 TON PLN
- FRICTION/ADHESION:

	S	C	
SOIL-LCR	31°	200 PSF	
SOIL-FML	18°	50 PSF	
FML-LCR	12°	0	← FAILURE CRITERIA

- RAMP GEOMETRY:
  WIDTH = 18' LENGTH = 150' THICKNESS = 24" β = 8°
- RAMP SUBBASE:
  γ = 130 PCF φ = 36° C = 1100 PSF

(1) DEFINE DRIVING FORCES

W_R = WEIGHT OF ROADWAY = $150 \times 18 \times \frac{24}{12} \times 130 = 702$ LOADED TOF KIP
W_v = VEHICLE WEIGHT = 55 TON = 110 KIP
F_B = BRAKING FORCE = $.3 \times 110$ KIP = $\frac{33}{2}$ KIP

(2) DEFINE RESISTING FORCES

$\delta_{MIN} = 12°$ $C_{MIN} = 0 \Rightarrow F_A = 0$

$F_R = (702 + 110) \times \cos 8° \times \tan 12°$
$= 170.9$ KIPS

(3) DEFINE DESIGN RATIOS, DR

$$DR_{STATIC} = \frac{170.9}{(702+110)\sin 8°} = \frac{170.9}{113} = 1.51 \quad \text{NG}$$

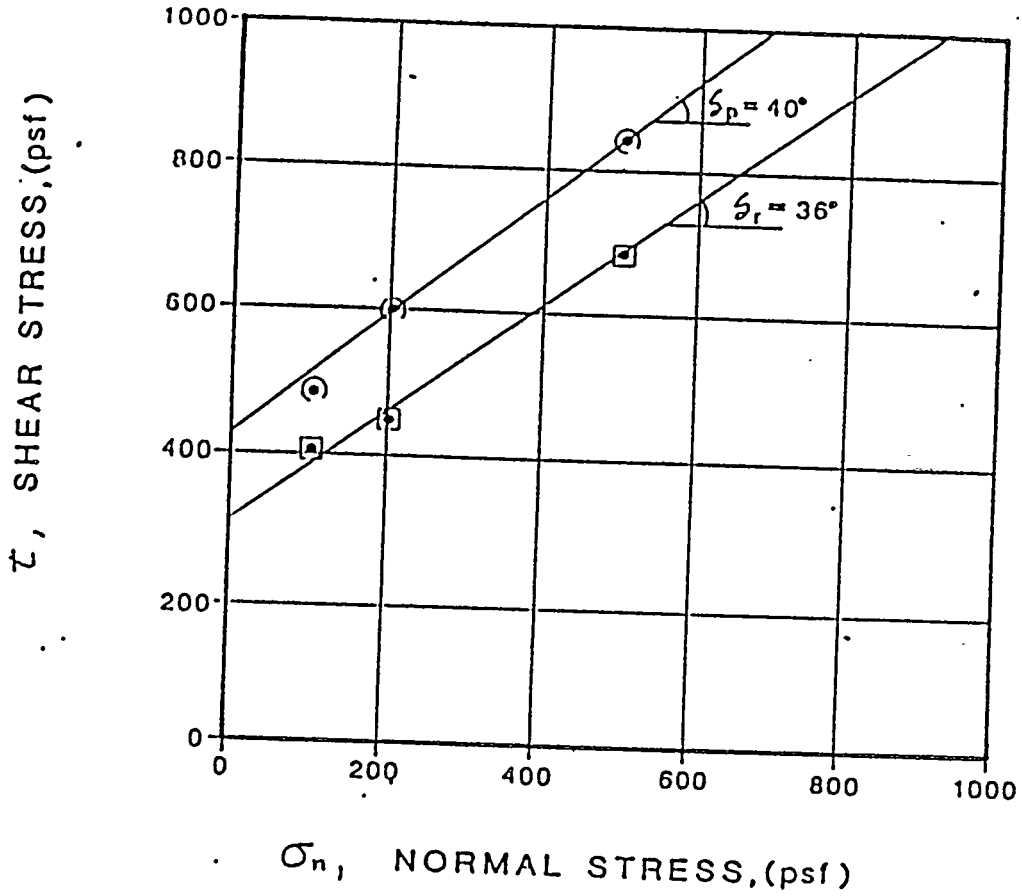
$$DR_{DYNAM} = \frac{170.9}{113 + 33} = \frac{170.9}{146} = 1.17 \quad \text{NG}$$

RECOMMEND PLACING SMALL LAYER OF SAND BETWEEN LCR AND FML TO INCREASE FRICTION ANGLE δ_{MIN}

Example No. 4.1

893-1186

ADMIX vs HDT GEOMEMBRANE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – ADMIX
LOWER SPECIMEN – HDT GEOMEMBRANE
SUBSTRATUM – PLATES

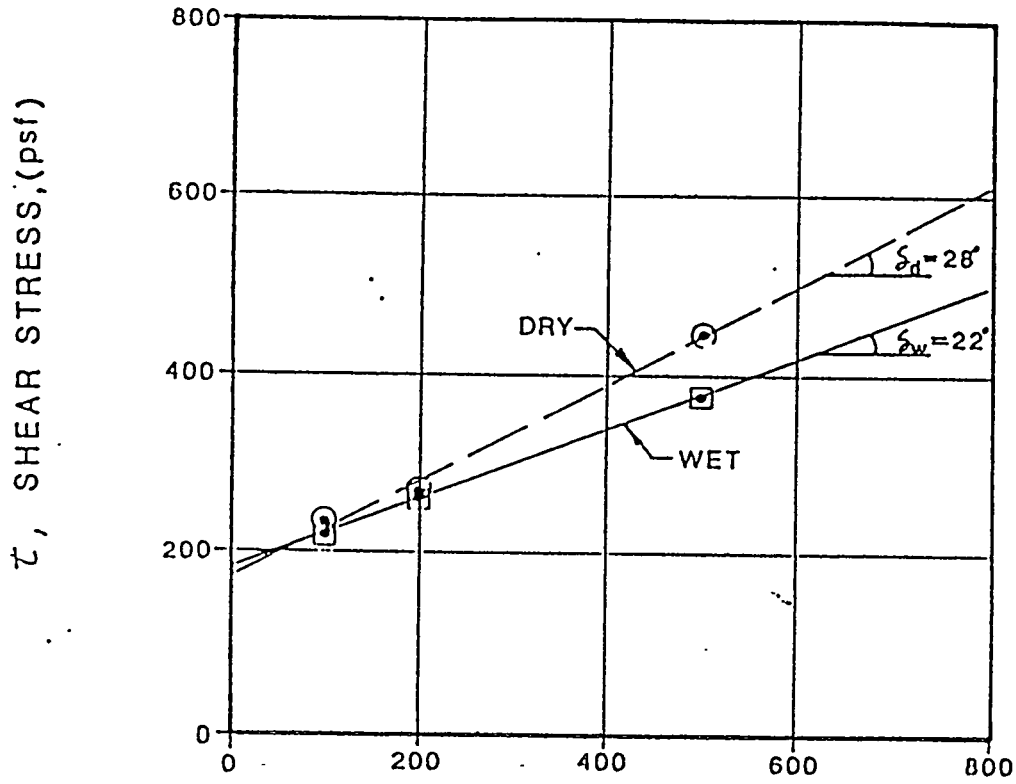
GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

Westinghouse

PROJ. NO. 4147-90-104 FIG. NO. 2

893-1186

TEXNET GEOCOMPOSITE vs HDT GEOMEMBRANE



SUPERSTRATUM – PLATES
UPPER SPECIMEN – TEXNET GEOCOMPOSITE
LOWER SPECIMEN – HDT GEOMEMBRANE
SUBSTRATUM – PLATES

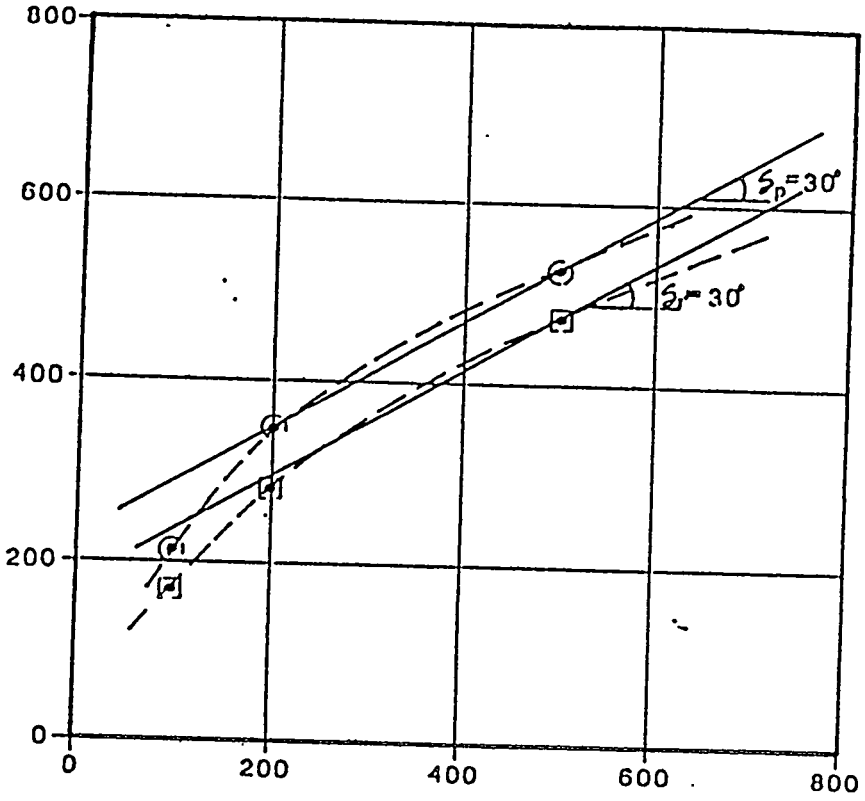
GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

Westinghouse

PROJ. NO. 4147-90-104 FIG NO. 3

893-1186

SANDY SOIL vs TEXNET GEOCOMPOSITE



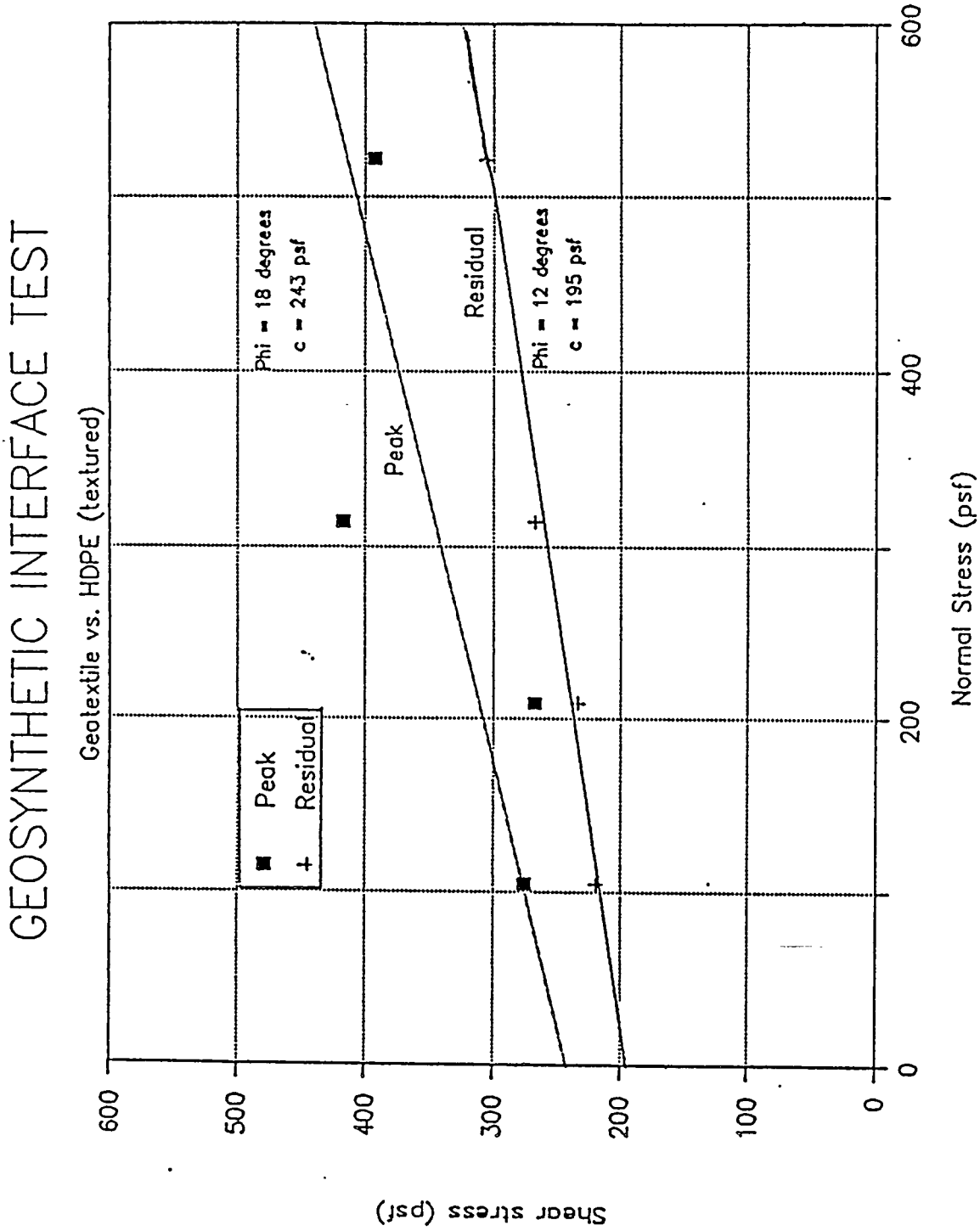
SUPERSTRATUM – PLATES
UPPER SPECIMEN – SANDY SOIL
LOWER SPECIMEN – TEXNET GEOCOMPOSITE
SUBSTRATUM – PLATES

GEOSYNTHETICS LABORATORY
DIRECT SHEAR/FRICTION TEST
GOLDER ASSOCIATES

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PROJ. NO. 4147-90-104 FIG. NO. 2

893-1186


GEOSYNTHETIC INTERFACE TEST
Geotextile vs. HDPE (textured)
Peak
Residual
Phi = 18 degrees
c = 243 psf
Phi = 12 degrees
c = 195 psf
Shear stress (psf)
Normal Stress (psf)
0
100
200
300
400
500
600
0
200
400
600

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 4

Project Title: Non-Drag-OFF LandFill Design

Project No. 893-1186.032

Calculation Title: Bearing Capacity of Subgrade soils

	Name	Date
Prepared By:	Timothy L. Martin	11-30-89
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT: Bearing Capacity of subgrade soil — Non-Drag-off

Job No. 893-1186 | Made by TLM | Date 11-30-89

Ref. | Checked | Reviewed | Sheet 2 of 4

Objective: To determine maximum allowable bearing capacity of onsite native soils.

Reference: Principles of Foundation Engineering
Braja M. Das
page 112, Table 3.2

Foundations and Earth Structures
Naval Facilities Engineering Command
Design Manual 7.02a
page 7.2-42, Table 1

$q_u = CN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$

(from Prin. of Found. Engr.)

where: $C = 0$

GNI lab tests: $\gamma = 120$ pcf (moist); $\phi = 38°$

$N_c = 61.35$, $N_q = 48.93$, $N_\gamma = 78.03$ } Eolian Sands

Assumptions: 1) Embedment Depth of foundation = 1.5 Ft
2) FS = 3
3) B = 1 ft

$q = (120)(.5) = 180$ psf

$q_u = (180)(48.93) + (\frac{1}{2})(120)(1)(78.03)$

$= 8,807.4 + 4,681.8$

$q_u = 13,489$ psf

$q_{net} = q_u - q$

$= 13,489 - 180$

$= 13,309$ psf

$q_{allowable} = \frac{q_{net(u)}}{FS}$

$= \frac{13,309}{3}$

$q_{allowable} = 4,436$ psf $\Rightarrow$ 4000 psf. for compacted structural fill in eolian sands, 95 percent of ASTM: D1557

other soils:

Dense native sands and gravels (Hanford Formation) = $q_{allow} = 6000$ psf *

Proof rolled Silty sands (Eolian sands) = 5,000 psf *

* after DM 7.02 page 7.2-142

3 of 4

Table 3.2 Bearing Capacity Factors*

ϕ	N_c	N_q	N_γ	N_q/N_c	tan ϕ
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	1.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.07	762.89	1.20	1.19

*After Vesic (1973)

Principles of Foundation Engineering
Braja M. Das

4 of 4

TABLE 1
Presumptive Values of Allowable Bearing Pressures for Spread Fou 1.

Type of Bearing Material	Consistency In Place	Allowable Bearing Pressure Tons Per sq ft	
		Range	Recomm Value Usc
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks).	Hard, sound rock	60 to 100	80.0
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks).	Medium hard sound rock	30 to 40	35.0
Sedimentary rock; hard cemented shales, siltstone, sandstone, limestone without cavities.	Medium hard sound rock	15 to 25	20.0
Weathered or broken bed rock of any kind except highly argillaceous rock (shale). RQD less than 25.	Soft rock	8 to 12	10.0
Compaction shale or other highly argillaceous rock in sound condition.	Soft rock	8 to 12	10.0
Well graded mixture of fine and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	8 to 12	10.0
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact Medium to compact Loose	6 to 10 4 to 7 2 to 6	7.0 5.0 3.0
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	4 to 6 2 to 4 1 to 3	4.0 3.0 1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	3 to 5 2 to 4 1 to 2	3.0 2.5 1.5

DM-7.02

7.2-142

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 7

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Bearing Capacity of Liner Soils

	Name	Date
Prepared By:	Timothy L. Martin	January 10, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT Bearing Capacity of Liner Soils

Job No. 893-1186.032 Ref.

Made by TLM Checked FSS Reviewed RL

Date 1-10-90 Sheet 2 of 7

Objective: To determine the bearing capacity of the liner soils.

Approach: Analysis for sand over clay.
1.) check q_u for sand
2.) check q_u for clay at depth
3.) use smallest q_u

Reference: DM-7.2 pg 131, 145

Sand Layer Bearing Capacity Analysis

Calculate for a continuous footing and for a rectangular footing.

A) Continuous footing

$q_{ult} = \gamma D N_q + \frac{1}{2}\gamma B N_\gamma$ (Figure 1) /

assume: γ 120 pcf

$D = 0$ (depth of embedment)

$B = 5.0$ ft (From attached figure sketch) - inside barrier wall

$\phi = 38°$ (From laboratory tests) /

$N_\gamma = 80$ (From fig 1, attached, DM 7.2 /

$q_{ult} = (120)\left(\frac{5}{2}\right)(80) = 24{,}000$ psf /

B) Rectangular footing

$q_{ult} = \gamma D N_q + 0.4 \gamma B N_\gamma$ ... (Figure 1) /

(Same conditions as per continuous footing)

$q_{ult} = (0.4)(120)(5)(80)$

$q_{ult} = 19{,}200$ psf /

Allowable Bearing capacities for Factor of Safety = 3.0

Sand Layer

A) Continuous Footing

$q_{allow} = \frac{24000}{3} = 8000$ psf /

B) Rectangular Footing

$q_{allow} = \frac{19{,}200}{3} = 6{,}400$ psf /

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SUBJECT: Bearing Capacity of Liner Soils

Job No. 893-1186.032 | Made by TLM | Date 1-10-90
Ref. | Checked FSS | Sheet 3 of 7
Reviewed KL

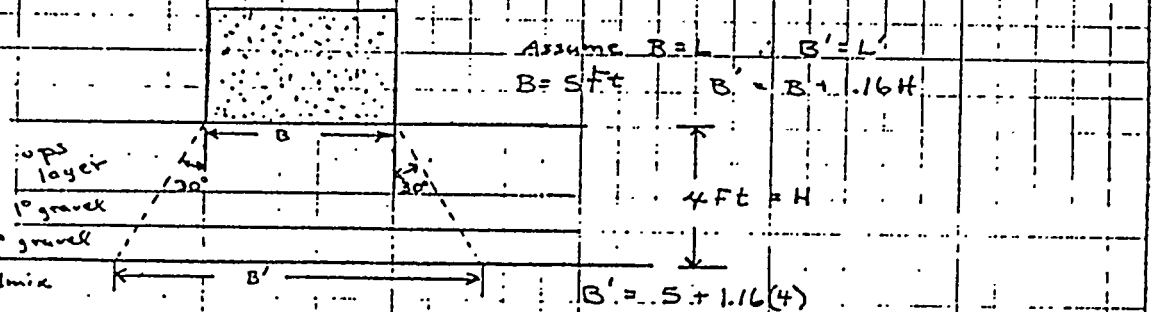
Admix layer Bearing Capacity

Calculate for a continuous footing and for a rectangular footing.

Assume worst case, i.e., undrained conditions, $\phi = 0$

A) Continuous footing

Assume: $C = 1000$ psf (Based on pocket pen. = 2000 psf, $S_u = 2/2 = 1$)
$D = 0$
$B' = 9.6$ ft (From sketch below, Ref. DM-7.2 pg (45))
$N_c = 5.53$ (From Fig 1, attached, DM 7.2)



$q_{ult} = CN_c + \gamma D$
$= (1000)(5.53)$
$q_{ult} = 5{,}530$ psf

B) Rectangular footing

$q_{ult} = CN_c\left(1 + 0.3\frac{B}{L}\right) + \gamma D$

$q_{ult} = (1000)(5.53)(1 + 0.3)$
$= 7{,}189$ psf

Allowable Bearing Capacities for factor of safety = 3.0

Clay layer

A) Continuous footing

$q_{allow} = \frac{5{,}530 \text{ psf}}{3} = 1{,}843$ psf

B) Rectangular footing

$q_{allow} = \frac{7{,}189 \text{ psf}}{3} = 2{,}396$ psf

Note: $(B + 1.16H)(L + 1.16H)$ = Area stressed by foundation load on the admix surface. (Only true for a 5 ft by 5 ft surface footing.)

$(B')(L') = 92.93 \text{ ft}^2$

If Q = foundation load, then

$\frac{Q \text{ pounds}}{92.93 \text{ ft}^2} \leq q_{allow}$ psf

Golder Associates

SUBJECT: Bearing Capacity of Liner Soils

Job No. 893-1186.032 | Made by TLM | Date 1-10-90
Ref. | Checked FSS | Sheet 4 of 7
Reviewed RL

Calculate load due to concrete barrier blocks

Assume: $\gamma_{concrete} \cong 150$ pcf (Civil Engineering Reference Manual, Lindeburg, M.R.)
Height of stacked blocks = 12.5 Ft (See attached sketch)

$q_{concrete} = (150\ pcf)(12.5\ ft)$
$= 1,875$ psf ✓

The $q_{concrete}$ stress is transfered directly to the sand layer since the foundation is in direct contact with the sand; however, the $q_{concrete}$ stress will be reduced on the admix since the admix is four feet down from the base of the concrete. The actual stress experienced on the admix surface will be reduced by the ratio $\left(\frac{BL}{B'L'}\right)$.

Calculate the foundation pressure applied to the admix; (assume B=L)

$q_{actual\ on\ admix} = (q_{concrete})\left(\frac{BL}{B'L'}\right) = \left(1,875\ \frac{lbs}{ft^2}\right)\frac{(5)(5)}{(9.64)(9.64)} = 504$ psf ✓

Summary of Bearing Capacities

Soil	q_{ult} continuous	q_{ult} Rectangular	q_{allow} (FS=3.0) continuous	q_{allow} (FS=3.0) Rectangular	
Sand	24,000 psf	19,200 psf	8,000 psf	6,400 psf	✓
Admix	5,530 psf	7,189 psf	1,843 psf	2,396 psf	✓

Factor of Safety Summary

Barrier Block Load: 1,875 psf

Sand:

Continuous: $FS = \frac{24000}{1,875} = 12.8$ ✓

Rectangular: $FS = \frac{19,200}{1,875} = 10.2$ ✓ (Note: all FS > 3.0) ✓

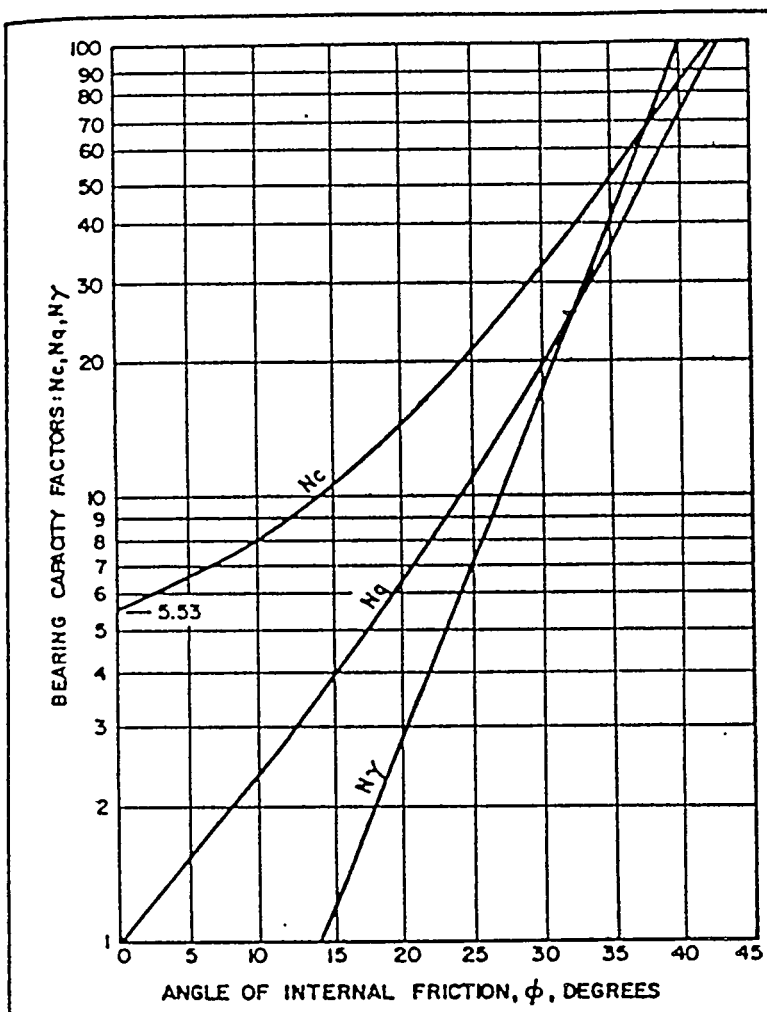
Admix:

Continuous: $FS = \frac{5,530}{504} = 11.0$ ✓

Rectangular: $FS = \frac{7,189}{504} = 14.3$ ✓

Conclusion: The concrete barrier blocks stacked to 12.5 Ft will not cause bearing failure in the underlying liner soils. ✓

Non-Drag-off
893-1186 5 of 17



ULTIMATE BEARING CAPACITY = q_{ult}

CONTINUOUS FOOTING; GENERAL CASE

$q_{ult} = q' + q''$

q' = PORTION OF BEARING CAPACITY ASSUMING WEIGHTLESS FOUNDATION SOIL

q'' = PORTION OF BEARING CAPACITY FROM WEIGHT OF FOUNDATION SOILS

$q' = cN_c + \gamma D N_q$

$q'' = \gamma \frac{B}{2} N_\gamma$

$q_{ult} = cN_c + \gamma D N_q + \frac{\gamma B}{2} N_\gamma$

SQUARE OR RECTANGULAR FOOTING

$q_{ult} = cN_c \left(1 + .3\frac{B}{L}\right) + \gamma D N_q + 0.4 \gamma B N_\gamma$

CIRCULAR FOOTING: R = B/2

$q_{ult} = 1.3\, cN_c + \gamma D N_q + 0.6 \gamma R N_\gamma$

FOR COHESIONLESS FOUNDATION SOILS ($c = 0$)

CONTINUOUS FOOTING:

$q_{ult} = \gamma D N_q + \frac{\gamma B}{2} N_\gamma$

SQUARE OR RECTANGULAR FOOTING:

$q_{ult} = \gamma D N_q + 0.4 \gamma B N_\gamma$

CIRCULAR FOOTING:

$q_{ult} = \gamma D N_q + 0.6 \gamma R N_\gamma$

FOR COHESIVE FOUNDATION SOILS ($\phi = 0$)

CONTINUOUS FOOTING:

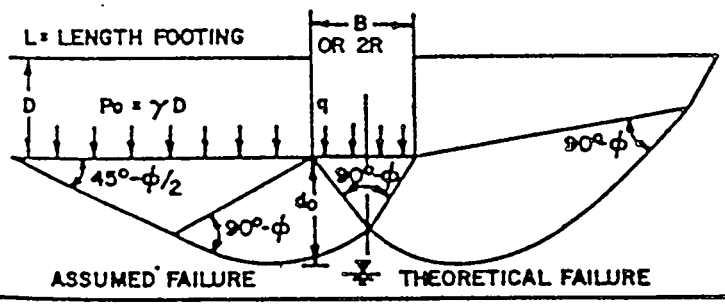
$q_{ult} = cN_c + \gamma D$

SQUARE OR RECTANGULAR FOOTING:

$q_{ult} = cN_c \left(1 + .3\frac{B}{L}\right) + \gamma D$

CIRCULAR FOOTING:

$q_{ult} = 1.3\, cN_c + \gamma D$



ASSUMED CONDITIONS:

1. $D \leq B$
2. SOIL IS UNIFORM TO DEPTH $d_0 > B$.
3. WATER LEVEL LOWER THAN d_0 BELOW BASE OF FOOTING.
4. VERTICAL LOAD CONCENTRIC.
5. FRACTION AND ADHESION ON VERTICAL SIDES OF FOOTING ARE NEGLECTED.
6. FOUNDATION SOIL WITH PROPERTIES C, ϕ, γ

FIGURE 1
Ultimate Bearing Capacity of Shallow Footings With Concentric Loads

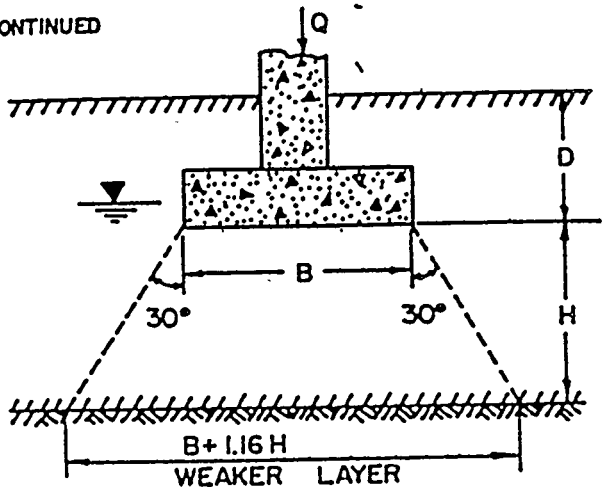
7.2-131

Non-Dog-off
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TABLE 2 (continued)
Selection of Allowable Bearing Pressures for Spread Foundations

8. CONTINUED



Q = applied load, not including weight of foundation itself.

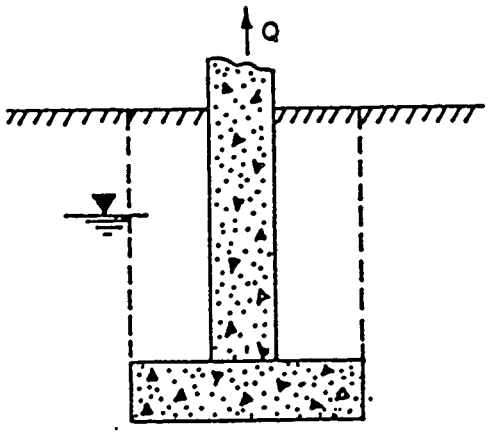
L = length of foundation.

(B + 1.16H) (L + 1.16H) = area stressed in weaker layer.

$$\frac{Q}{(B + 1.16H)(L + 1.16H)} \leq \text{nominal value of allowable bearing pressure.}$$

Area stressed in weaker layer shall not extend beyond intersection of 30° planes extending downward from adjacent foundations.

9. Where the footing is subjected to a sustained uplift force, compute ultimate resistance to uplift as follows:

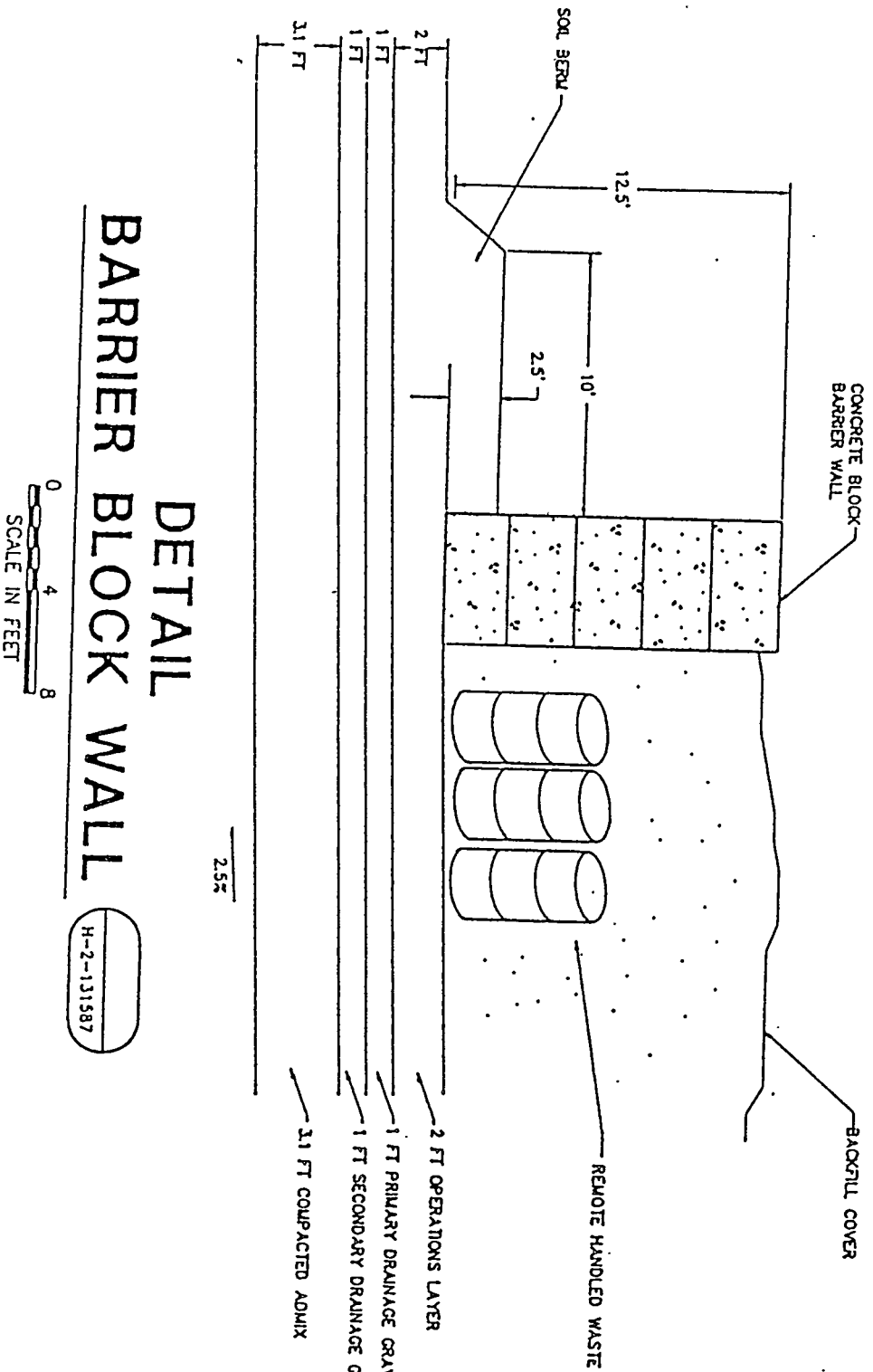


Q = applied uplift load.

W = total effective weight of soil and concrete located within prism bounded by vertical lines at base of foundation. Use total unit weights above water table and buoyant unit weights below.

$$\text{Safety Factor} = \frac{W}{Q} \geq 2$$

(This is a conservative procedure; see text for reference on more detailed analyses procedures.)



DETAIL
BARRIER BLOCK WALL

H-2-131587

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11

Project Title: Non-Drag-off Landfill Design Project

Project No. 893-1186.032

Calculation Title: Stability Checks on Barrier Block Wall

	Name	Date
Prepared By:	Timothy L. Martin	January 10-13, 1990
Checked By:	Frank S. Slusi	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT Stability Checks on Barrier Block Wall

Job No. 893-1186.032 | Made by TLM | Date 1-10-90
Ref. | Checked FJS | Sheet 2 of 11
Reviewed RCL

Objective: To check for overturning and sliding of barrier wall. Note that bearing capacity analysis was conducted separately under the title "Bearing capacity of liner soils"

Approach: For overturning analysis, assume the wall is flexible and can move slightly creating an active force. The active force will be calculated using the Rankine active force method.

Reference: Fundamentals of Foundation engineering, Das, B.J.

Assumptions: $\gamma = 115$ pcf (assuming loose condition)
$\phi = 33°$ (assuming loose condition)
$H = 12.5$ ft (height of barrier wall)
$\gamma_{concrete} = 150$ pcf

Rankine Active force per unit length of barrier wall:

$P_a = \frac{1}{2} \gamma H^2 K_a$ K_a = active earth pressure coefficient

$= \tan^2(45 - \phi/2)$

$= \tan^2(45 - 33/2)$

$= 0.295$

$P_a = (\frac{1}{2})(115)(12.5)^2(0.295)$

$P_a = 2,648.6 \approx 2,650$ lbs/ft of wall length

Check for overturning

$FS_{(overturning)} = \frac{\Sigma m_R}{\Sigma m_O}$ ..... about Point A

Σm_O = (force)(moment arm)

$= (P_a)(H/3)$

$= (2,650)(12.5/3)$

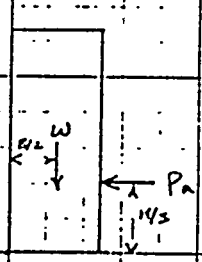
$= 11,041.67$ lbs

Σm_R = (force)(moment arm)

$= (5)(12.5)(150)(2.5)$

$= 23,437.5$ lbs

$FS_{overturning} = \frac{23,437.5}{11,041.7} \approx 2.1$



Check for sliding

A) At base of the wall

$FS_{sliding} = \frac{\Sigma F_R}{\Sigma F_d}$

$\Sigma F_d = P_a$ (Assuming horizontal backslope) $= 2,650$ lbs/ft of wall length.

$\Sigma F_R = \Sigma V \tan\phi + B\phi^\circ + P\phi^\circ$

$= (5)(12.5)(150)\tan 33°$

$= 6,088$ lbs/ft

$FS_{sliding} = \frac{6,088}{2,650} = 2.3$

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SUBJECT Stability Checks on Barrier Block Wall

Job No. 693-1126.032 | Made by TLM | Date 1-10-90
Ref. | Checked FSS | Sheet 3 of 11
Reviewed RC

B) at First Geosynthetics/HDPE contact
Assume: Geotextile/HDPE $\phi \approx 8°$ (- Seed et al 1988 testing of Kettleman Hills geosynthetics)

$FS = \frac{\Sigma F_R}{\Sigma F_d}$

$\Sigma F_d = P_a = \frac{1}{2} \gamma H^2 K_a$ where $K_a = 0.295$
$\gamma = 115$ p.c.f
$H = 12.5 + 3 = 15.5$

$P_a = \frac{1}{2}(115)(15.5^2)(0.295)$

$P_a = 4,075.2$ lbs/ft of wall length ✓

$\Sigma F_R = \Sigma V \tan\phi + B\hat{c}^0 + P_p$ ✓

where $K_p = \tan^2(45 + \phi/2)$
$= \tan^2(45 + 33/2)$
$K_p = 3.39$ ✓

$= [(12.5)(150) + (3)(120)] \tan 8° + 1830.6$

$= 2,144.7$ lbs/ft ✓

$P_p = \frac{1}{2}\gamma D^2 K_p = \frac{1}{2}(120)(3^2)(3.39)$
$= 1,830.6$ lbs/ft of wall length ✓

$FS_{sliding} = \frac{2144.7}{4,075.2} = 0.53$ Not acceptable

Calculate Berm height to provide passive force to obtain FS ≥ 1.5
i.e., $F_R \geq (1.5)(4,075.2) = 6,112.8$ lbs/ft
Try Berm 2.5 ft High and 10 ft wide
$\Sigma F_R = \Sigma V \tan\phi + B\hat{c} + P_p$
$= [(12.5)(150) + (3)(120)] \tan 8° + 6,152.9$
$\Sigma F_R = 6,467$ lbs/ft ✓

$P_p = \frac{1}{2}\gamma D^2 K_p$
$= \frac{1}{2}(120)(5.5^2)(3.39)$
$= 6,152.9$ lbs/ft of wall length ✓

$FS_{sliding} = \frac{\Sigma F_R}{\Sigma F_d} = \frac{6,467}{4,075.2} = 1.59$ OK with added berm 2.5 ft high and 10 ft wide along barrier wall, shown on attached sketch.

C) Worst case sliding conditions: The base of the landfill in some areas can slope up to 3.5% (Resultant of 2.5% → and 2.5% ↓)
Analyze stability by reducing the critical interface friction angle.
3.5% ≅ 2°, therefore analyze stability using 8° - 2° = 6° for critical interface.
$P_p = 6,152.9$ lbs/ft of wall length as in part B above.
$\Sigma F_d = 4,075.2$ lbs/ft of wall length as in part B above.
$\Sigma F_R = [(2.5)(150) + (3)(120)] \tan(8° - 2°) + 6,152.9 = 6,387.8$ lbs/ft of wall length

$FS_{sliding} = \frac{6,387.8}{4,075.2} = 1.57$ OK (includes berm as shown on attached sketch)

Golder Associates

SUBJECT Stability Checks on Barrier Block Wall

Job No. 893-1186.032 | Made by TLM | Date 1-13-90

Ref. | Checked FSS | Sheet 4 of 11

Reviewed TCL

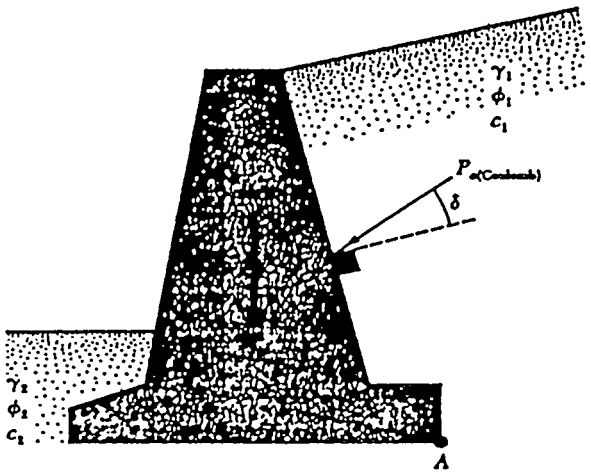
Conclusions:

1) Overturning Stability - The barrier block wall as configured in the attached sketch has a calculated $FS_{overturning} = 2.1$ which is above the usual minimum desirable factor of safety with respect to overturning of from 1.5 to 2 (Ref. Principles of Foundation Engineering, page 246, attached).

2) Sliding Stability - The barrier block wall as configured in the attached sketch requires a 2.5 Ft thick, 10 Ft wide soil berm to maintain a greater than factor of safety of greater than 1.5 which is the general required minimum factor of safety against sliding (Ref. Principles of Foundation Engineering page 247, attached). The worst case condition (part c of the analysis) still provides a factor of safety of 1.57 on the weakest interface, on a sloping base. This analyses was concerned with the first lift of waste placed on the initial operations layer. The factor of safety against sliding will increase as subsequent lifts of waste are placed.

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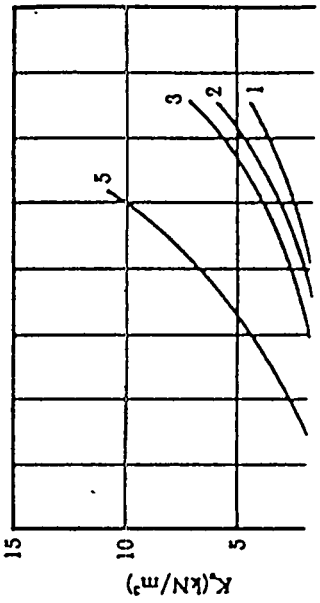
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(c)

Figure 5.18 (Continued)

backfills with plane surfaces, and Figure 5.20 is for backfills that slope upward from the crest of the wall for a limited distance and then become horizontal. Note that, in these figures, $\frac{1}{2}K_a H^2$ is the vertical component of the active force on the plane AB; similarly, $\frac{1}{2}K_h H'^2$ is the horizontal force. The numerals on the curves indicate the type of soil described in Table 5.8 on page 244.



5.14 Stability Checks

To check the stability of a retaining wall, the following steps are necessary:

1. Check for *overturning* about its toe
2. Check for *sliding failure* along its base
3. Check for *bearing capacity failure* of the base
4. Check for *settlement*
5. Check for *overall stability*

This section will describe the procedure for checking for overturning and sliding and bearing capacity failure. The principles of investigation for settlement have been covered in Chapter 3 and will not be repeated here. Some problems regarding the overall stability of retaining walls have been discussed in Section 5.15.

Check for Overturning

Figure 5.21 on page 245 shows the forces acting on a cantilever and gravity retaining wall with the assumption that the Rankine active pressure is acting along a vertical plane AB drawn through the heel. P_p is the Rankine passive pressure; its magnitude can be given as

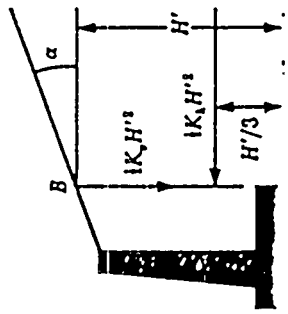


Figure 5.19 Chart for esti... walls supporting backfills ... *Engineering Practice*, 2nd ... right 1967 by John Wiley ...

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Table 5.8 Types of Backfill for Retaining Walls*

1. Coarse-grained soil without admixture of fine soil particles, very permeable (clean sand or gravel).
2. Coarse-grained soil of low permeability due to admixture of particles of silt size.
3. Residual soil with stones, fine silty sand, and granular materials with conspicuous clay content.
4. Very soft or soft clay, organic silts, or silty clays.
5. Medium or stiff clay, deposited in chunks and protected in such a way that a negligible amount of water enters the spaces between the chunks during floods or heavy rains. If this condition of protection cannot be satisfied, the clay should not be used as backfill material. With increasing stiffness of the clay, danger to the wall due to infiltration of water increases rapidly.

*From *Soil Mechanics in Engineering Practice*, 2nd ed., by K. Terzaghi and R. B. Peck. Copyright 1967 by John Wiley and Sons. Reprinted by permission.

$$P_p = \tfrac{1}{2}K_p\gamma_2 D^2 + 2c_2\sqrt{K_p}D \tag{5.22}$$

where γ_2 = unit weight of soil in front of the heel and under the base slab
K_p = Rankine passive earth pressure coefficient
$= \tan^2(45 + \phi_2/2)$
c_2, ϕ_2 = cohesion and soil friction angle, respectively

The factor of safety against overturning about the toe—that is, about point C in Figure 5.21—can be expressed as

$$FS_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_O} \tag{5.50}$$

where ΣM_O = sum of the moments of forces tending to overturn about point C
ΣM_R = sum of the moments of forces tending to resist overturning about point C

The overturning moment can be given as

$$\Sigma M_O = P_h\left(\frac{H'}{3}\right) \tag{5.51}$$

where $P_h = P_a \cos\alpha$

For calculation of the resisting moment, ΣM_R (neglecting P_p), a table (such as Table 5.9) can be prepared. The weight of the soil above the heel and the weight of the concrete (or masonry) are both forces that contribute to the resisting moment. Note that the force P_v also contributes to the resisting moment. P_v is the vertical component of the active force P_a, or

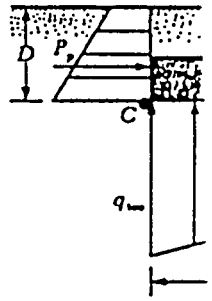
$$P_v = P_a \sin\alpha \tag{5.52}$$

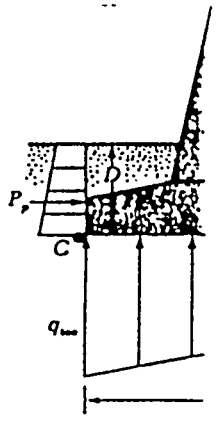
The moment of the force P_v about C is equal to

$$M_v = P_v B = P_a \sin\alpha \cdot B \tag{5.53}$$

where B = width of the base slab

Once ΣM_R is known, the factor of safety can be calculated as





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permeable (clean sand or

les of silt size.
with conspicuous clay

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B. Peck. Copyright 1967 by John

(5.22)

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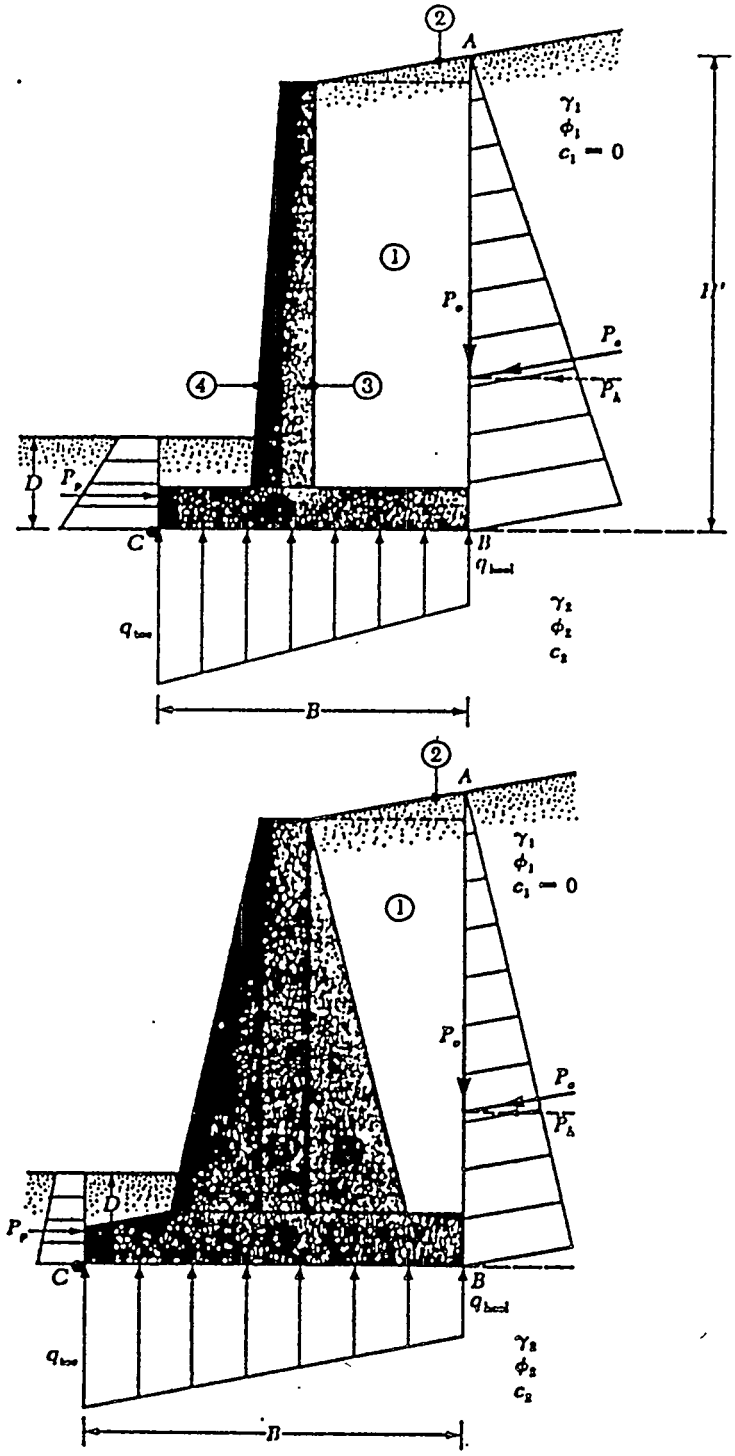


Figure 5.21 Check for overturning; assume Rankine pressure is valid

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Table 5.9 Procedure for Calculation of ΣM_o

Section (1)	Area (2)	Weight/unit length of wall (3)	Moment arm measured from C (4)	Moment about C (5)
1	A_1	$W_1 = \gamma_1 \times A_1$	X_1	M_1
2	A_2	$W_2 = \gamma_2 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_c \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_c \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_c \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_c \times A_6$	X_6	M_6
		P_v	B	M_o
		ΣV		ΣM_R

Note: γ_1 = unit weight of backfill
γ_c = unit weight of concrete

$$FS_{(\text{overturning})} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_o}{P_a \cos \alpha \dfrac{H'}{3}} \tag{5.54}$$

The usual minimum desirable value of the factor of safety with respect to overturning is 1.5 to 2.

Some designers prefer to use the following equation for determination of the factor of safety against overturning:

$$FS_{(\text{overturning})} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6}{(P_a \cos \alpha) \dfrac{H'}{3} - M_o} \tag{5.55}$$

Check for Sliding Along the Base

The factor of safety against sliding may be expressed by the equation

$$FS_{(\text{sliding})} = \frac{\Sigma F_{R'}}{\Sigma F_d} \tag{5.56}$$

where $\Sigma F_{R'}$ = sum of the horizontal resisting forces
ΣF_d = sum of the horizontal driving forces

Referring to Figure 5.22, one sees that the shear strength of the soil below the base slab can be represented as

$$s = \sigma \tan \phi_2 + c_2$$

Thus, the maximum resisting force that can be derived from the soil per unit length of the wall along the bottom of the base slab is

$$R' = s \text{ (area of cross section)} = s(B \times 1) = B\,\sigma \tan \phi_2 + Bc_2$$

However

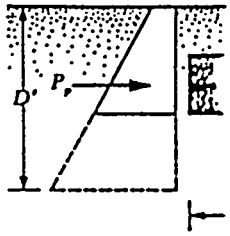


Figure 5.22 Check for

$B\sigma$ = sum

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R' = (Σ V)

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$\Sigma F_{R'}$ = (Σ

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force) is the horizo

$\Sigma F_d = P_a$

Combining Eqs. (5

$FS_{(\text{sliding})}$ =

A minimum factor

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to the value of 0.5

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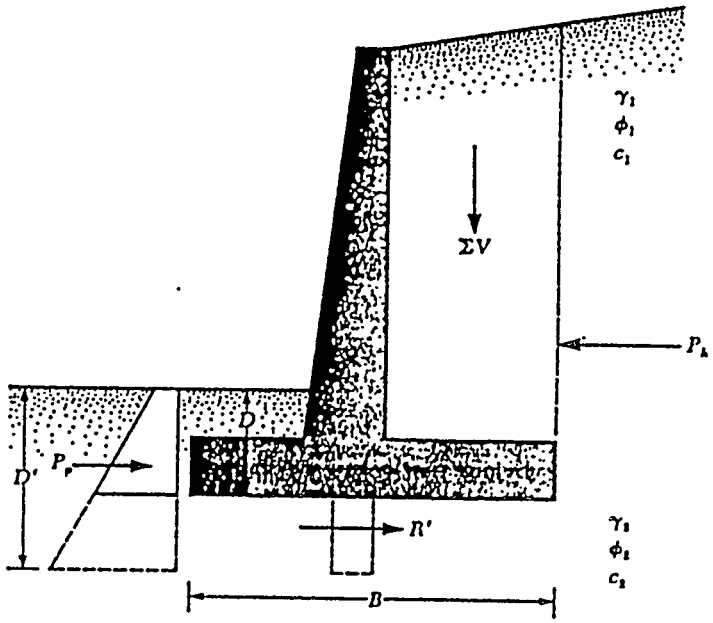


Figure 5.22 Check for sliding along the base

$B\sigma$ = sum of the vertical force = ΣV (see Table 5.9)

So

$$R' = (\Sigma V) \tan \phi_2 + Bc_2$$

As can be seen in Figure 5.22, the passive force P_p is also a horizontal resisting force. The expression for P_p is given in Eq. (5.22). Hence

$$\Sigma F_{R'} = (\Sigma V) \tan \phi_2 + Bc_2 + P_p \tag{5.57}$$

The only horizontal force that will tend to cause sliding of the wall (*driving force*) is the horizontal component of the active force P_a, so

$$\Sigma F_d = P_a \cos \alpha \tag{5.58}$$

Combining Eqs. (5.56), (5.57), and (5.58)

$$FS_{(\text{sliding})} = \frac{(\Sigma V)\tan \phi_2 + Bc_2 + P_p}{P_a \cos \alpha} \tag{5.59}$$

A minimum factor of safety of 1.5 against sliding is generally required.

In many cases, the passive force P_p is ignored for calculation of the factor of safety with respect to sliding. The friction angle, ϕ_2, is also reduced in several instances. The reduced soil friction angle may be in the order of one-half to two-thirds of the angle ϕ_2. In a similar manner, the cohesion c_2 may be reduced to the value of 0.5 to 0.67 c_2. Thus

$$FS_{(sliding)} = \frac{(\Sigma V)\tan(k_1\phi_2) + Bk_2c_2 + P_p}{P_a \cos \alpha} \tag{5.60}$$

where k_1 and k_2 are in the range of 1/2 to 2/3

In some instances, certain walls may not yield a desired factor of safety of 1.5. In order to improve their resistance to sliding, a base key may be used. Base keys are illustrated by broken lines in Figure 5.22. This figure indicates that the passive force at the toe *without the key* can be given as

$$P_p = \tfrac{1}{2}\gamma_2 D^2 K_p + 2c_2 D\sqrt{K_p}$$

However, if a key is included, the passive force per unit length of the wall will be

$$P_p = \tfrac{1}{2}\gamma_2 D'^2 K_p + 2c_2 D'\sqrt{K_p}$$

where $K_p = \tan^2(45 + \phi_2/2)$

Because $D' > D$, it is obvious that a key will help increase the passive resistance at the toe and, hence, the factor of safety against sliding. Usually the base key is constructed below the stem, and some main steel is run into the key.

Check for Bearing Capacity Failure

The vertical pressure as transmitted to the soil by the base slab of the retaining wall should be checked against the ultimate bearing capacity of the soil. The nature of variation of the vertical pressure transmitted by the base slab into the soil is shown in Figure 5.23. Note that q_{toe} and q_{heel} are the *maximum* and the *minimum* pressures occurring at the ends of the toe and heel sections, respectively. The magnitudes of q_{toe} and q_{heel} can be determined in the following manner:

The sum of the vertical forces acting on the base slab is equal to ΣV (see Col. 3, Table 5.9), and the horizontal force is $P_a \cos \alpha$. Let R be the resultant force, or

$$\vec{R} = \overrightarrow{\Sigma V} + \overrightarrow{(P_a \cos \alpha)} \tag{5.61}$$

The net moment of these forces about the point C (Figure 5.23) is equal to

$$M_{net} = \Sigma M_R - \Sigma M_O \tag{5.62}$$

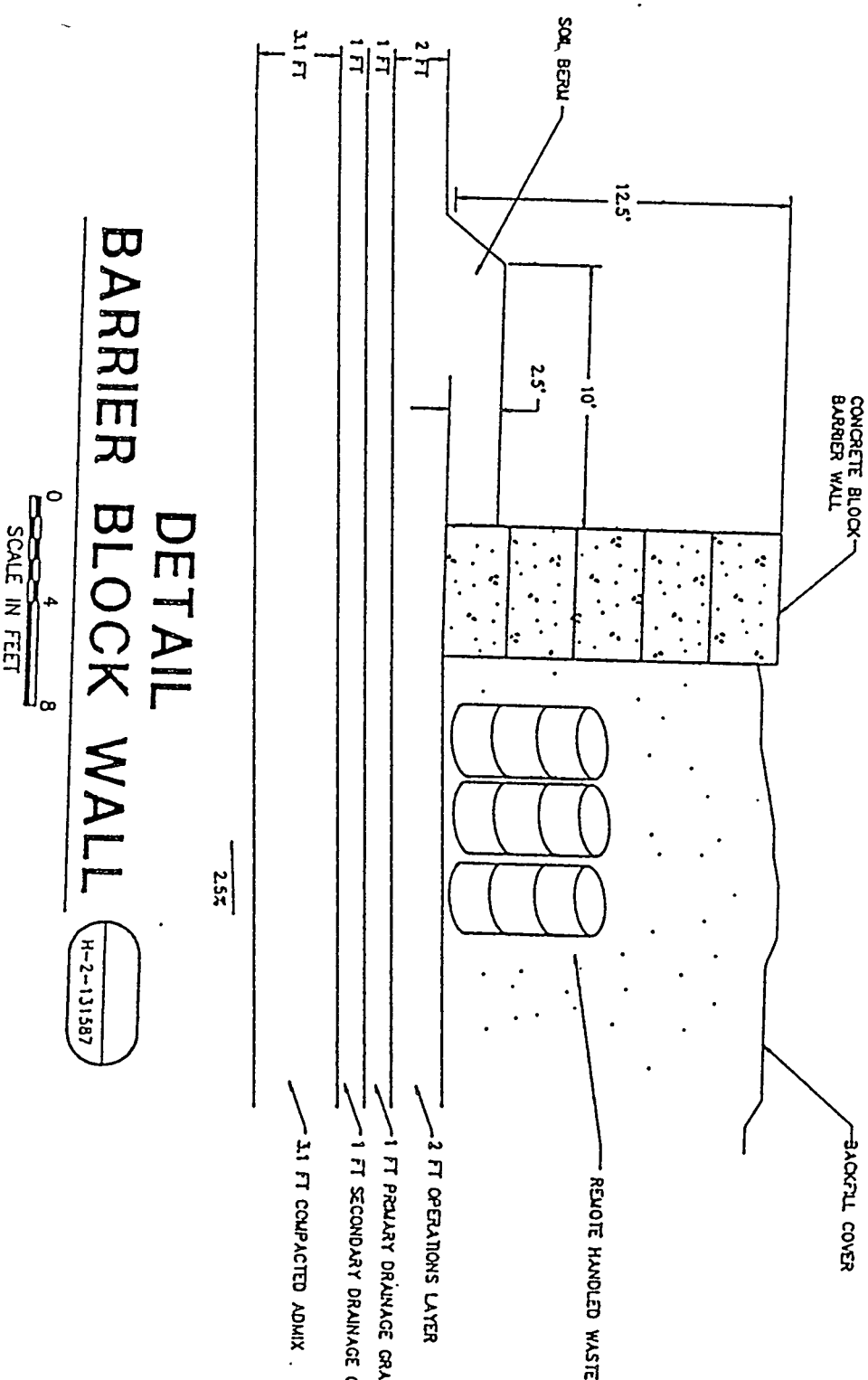
Note that the values of ΣM_R and ΣM_O have been previously determined [see Col. 5, Table 5.9 and Eq. (5.51)]. Let the line of action of the resultant, R, intersect the base slab at E, as shown in Figure 5.23. The distance CE can now be determined as

$$\overline{CE} = \overline{X} = \frac{M_{net}}{\Sigma V} \tag{5.63}$$

Hence, the eccentricity of the resultant, R, can be expressed as

$$e = \frac{B}{2} - \overline{CE} \tag{5.64}$$

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DETAIL
BARRIER BLOCK WALL

H-2-131587

0 4 8
SCALE IN FEET

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 6

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Elastic Settlement of Foundation Sand

	Name	Date
Prepared By:	Timothy L. Martin	December 13, 1989
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT: Calculation of Elastic Settlement of Foundations

Job No. 893-1186.032 | Made by TCM | Date 12-13-89
Ref. Non-Dang-off | Checked TSS | Sheet 2 of 6
Reviewed

Objective: To estimate the maximum differential settlement of the non-cohesive foundation soils of the Non-Dang-off landfill.

Approach: To calculate the settlement at the base of the landfill which will be subject to the largest normal load. Then calculate the settlement at the top of the slope which would be subject to the lowest normal load, i.e., the cover section only. Compare the base and top of slope settlements to estimate the maximum differential elastic settlement.

The Schmertmann method was developed for initial loading, not reloading. This means that the Schmertmann method will be conservative in that the soil has been preloaded and some settlement has already occurred.

References: Soil Mechanics Design Manual 7.1, DM-7.1 pg 7.1-220 - 223. 1982

Assumptions: Foundation width ≈ 100 Ft
Embankment: none
Load: 4200 psf (base), (35 Ft @ 120 pcf, includes cover section)
1800 psf (Top of Slope), (15 Ft @ 120 pcf, cover section only)

Soil Properties

Properties based on borehole GPH-1

Depth Below Ground Surface	Soil Description	SPT Blow Count, N
0.0 - 17.5 Ft	Compact F-m Sand, some silt	13
17.5 - 27.5	Compact, F-C Sand and F Gravel	12
27.5 - 46.0	V. Dense, F-C Sand	60
46.0 - 200*	V. Dense F-C Sand and F-C Gravel	100

*Assume soil conditions after 53.3 Ft based on well log 699-45-78

Side Slope

Layer	Δz (in)	N (thickness of layer / SPT Blow Count)	E_s/N (Fig 7)	E_s (tsf) (soil modulus)	z_c (depth to center of z_i layer) (in)	I_z (attenuation) (influence factor for top of layer)	$\frac{I_z}{E_s}(\Delta z)$ (in/tsf)
1	210	13	4	52	105	0.100	0.404
2	120	12	7	84	270	0.275	0.393
3	222	60	10	600	441	0.440	0.63
4	1848	100	12	1200	1476	0.308	0.474
						Σ	1.434

(continued next page)

Golder Associates

SUBJECT Calculation of Elastic Settlements in Foundation Sand

Job No. 893-1186.032 | Made by TLM | Date 12-13-89
Ref. | Checked TSS | Sheet 3 of 6
Reviewed

(Side Slope settlement calculations continued)

Total Settlement = $\Delta H = C_1 C_2 \Delta P \sum_0^{2B} \left(\frac{I_z}{E_s}\right)(\Delta z)$

where $C_1 = 1 - 0.5(P_o/\Delta P)$, $C_1 \geq 0.5$

$C_2 = 1 + 0.2 \log(10\,t)$

P_o = overburden pressure at foundation level

ΔP = net foundation pressure increase

t = elapsed time in years

$C_1 = 1.0$ since $P_o = 0$

$C_2 = 1 + 0.2 \log(10 \times 30 \text{ years}) = 1.495$ (30 years is monitored life).

$\Delta P = 1800$ psf $= 0.90$ tsf

$\Delta H = (1.0)(1.495)(0.90)(1.434)$

$\Delta H = 1.93$ inch (for top of slope settlement)

Base

layer	Thickness of layer Δz (in)	SPT Blowcount N	Soil modulus E_s/N	E_s	Depth to center of Δz layer z_c (in)	Influence factor I_z	$\frac{I_z}{E_s}(\Delta z)$ (in/tsf)
1	120	12	7	84	60	0.065	0.093
2	222	60	10	600	231	0.230	0.085
3a	264	100	12	1200	474	0.470	0.103
3b	1794	100	12	1200	1503	0.300	0.449
							0.730

$C_1 = 1.0$

$C_2 = 1.495$

$\Delta P = 4200$ psf $= 2.10$ tsf

$\Delta H = (1.0)(1.495)(2.10)(0.730)$

$\Delta H = 2.29$ inch (For base settlement)

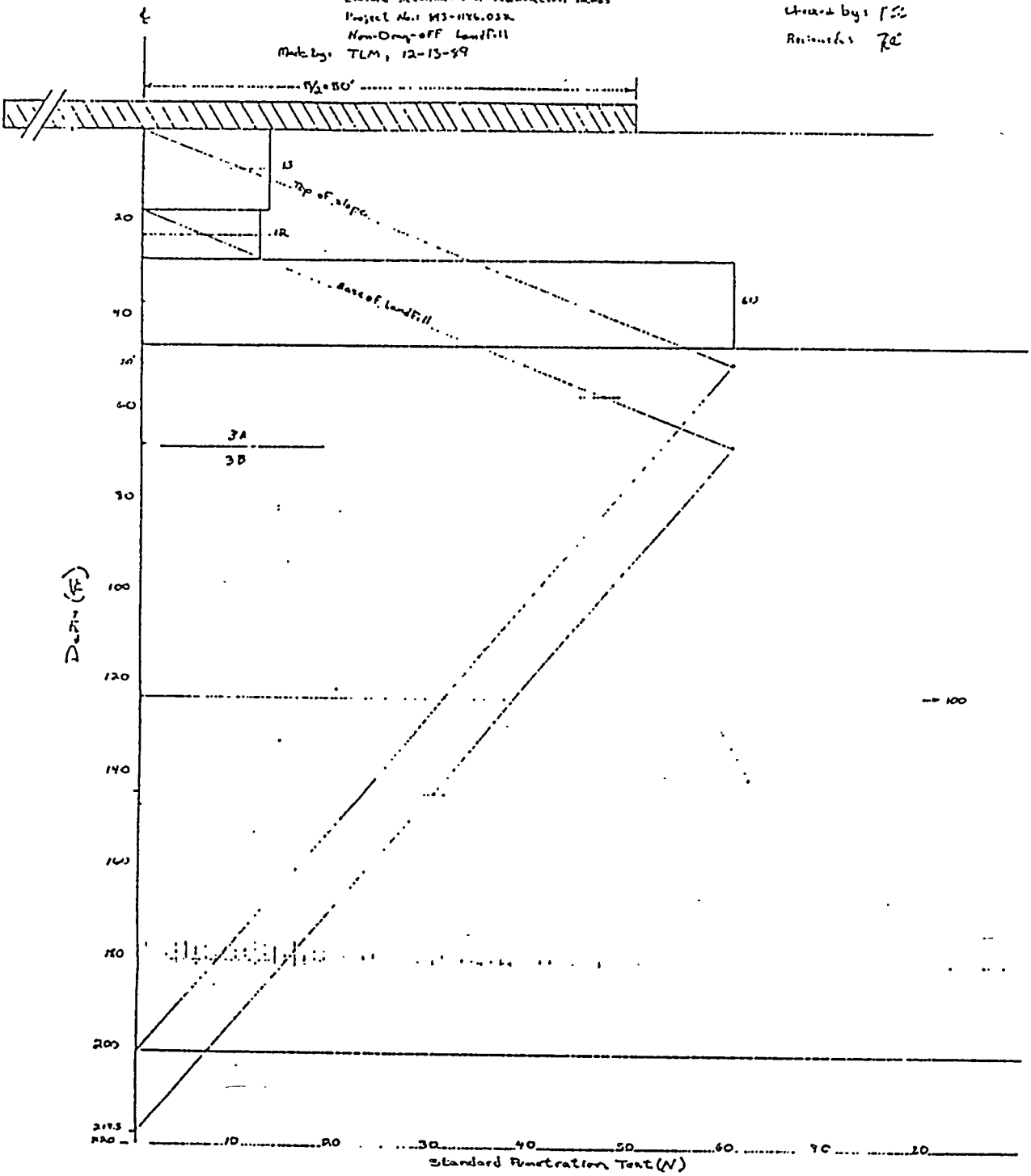
Conclusion:

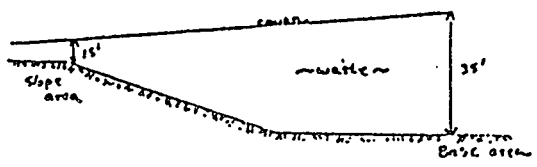
Maximum differential settlement = $2.29 - 1.93 = 0.36 \approx 0.4$ inch

Differential settlement of the subgrade is not anticipated to impact the design of the Land Fill.

Elastic Settlement of Foundation Sands
Project No: H3-1126.03R
Non-Dmg-off Landfill
Made by: TLM, 12-13-89

Pg 4 of 6
Checked by: [illegible]
Reviewed: RC





DATA REQUIRED:

1. A profile of standard penetration resistance N (blows/ft) versus depth, from the proposed foundation level to a depth of 2B, or to boundary of an incompressible layer, whichever occurs first. Value of soil modulus E_s is established using the following relationships.

Soil Type	E_s/N
Silts, sands silts, slightly cohesive silt-sand mixtures	4
Clean, fine to med, sands & slightly silty sands	7
Coarse sands & sands with little gravel	10
Sandy gravels and gravel	12

2. Least width of foundation = B, depth of embedment = D, and proposedaverage contact pressure = P.

3. Approximate unit weights of surcharge soils, and position of water table if within D.

4. If the static cone bearing value q_c is measured compute E_s based on $E_s = 2\ q_c$.

ANALYSIS PROCEDURE:

Refer to table in example problem for column numbers referred to by parenthesis:

1. Divide the subsurface soil profile into a convenient number of layers of any thickness, each with constant N over the depth interval 0 to 2B below the foundation.

2. Prepare a table as illustrated in the example problem, using the indicated column headings. Fill in columns 1, 2, 3 and 4 with the layering assigned in Step 1.

3. Multiply N values in column 3 by the appropriate factor E_s/N (col. 4) to obtain values of E_s; place values in column 5.

4. Draw an assumed 2B-0.6 triangular distribution for the strain influence factor I_z, along a scaled depth of 0 to 2B below the foundation. Locate the depth of the mid-height of each of the layers assumed in Step 2, and place in column 6. From this construction, determine the I_z value at the mid-height of each layer, and place in column 7.

FIGURE 7
Settlement of Footings Over Granular Soils: Example Computation Using Schmertmann's Method

7.1-220

5. Calculate $(I_z/E_s)\ \Delta z$, and place in column 8. Determine the sum of all values in column 8.

6. Total settlement = $\Delta H = C_1 C_2 \Delta p \sum_{0}^{2B} (\frac{I_z}{E_s}) \Delta z$,

where $C_1 = 1 - 0.5\ (p_o/\Delta p)$; $C_1 \geq 0.5$ embedment correction factor

$C_2 = 1 + 0.2 \log (10t)$ creep correction factor

p_o = overburden pressure at foundation level

Δp = net foundation pressure increase

t = elapsed time in years.

EXAMPLE PROBLEM:

GIVEN THE FOLLOWING SOIL SYSTEM AND CORRESPONDING STANDARD PENETRATION TEST (SPT) DATA, DETERMINE THE AMOUNT OF ULTIMATE SETTLEMENT UNDER A GIVEN FOOTING AND FOOTING LOAD:

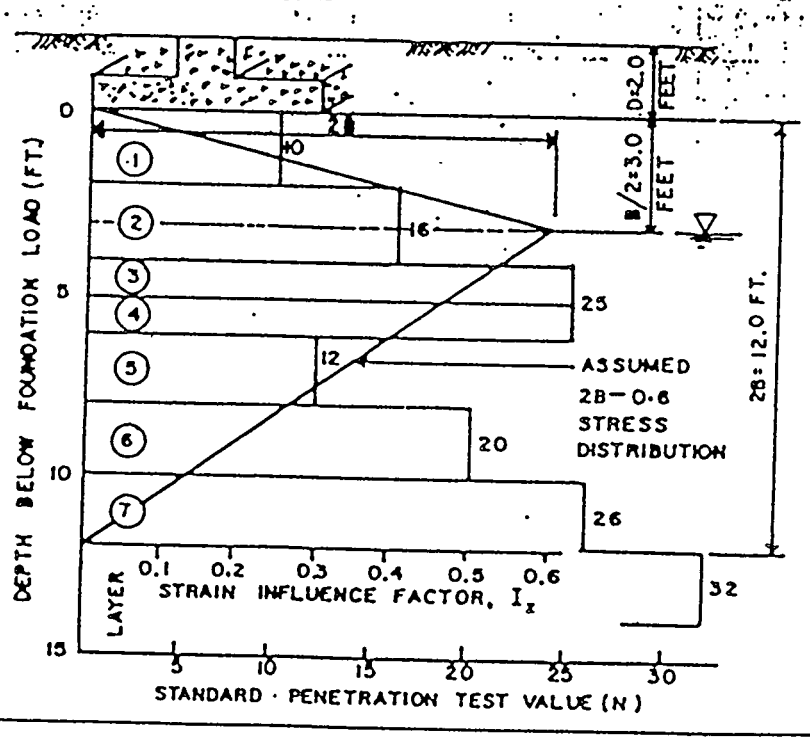


FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Footing Details:

Footing width: 6.0 ft. (min.) by 8.0 ft. (max.)

Depth of Embedment: 2.0 ft. Load (Dead + Live): 120 tons

Soil Properties:

Depth Below Surface (ft.)	Depth Below Base of Footing (ft.)	Unit Wt. (pcf) Moist	Sat.	Soil Description
0 - 5	<5	95	105	Fine sandy silt
5 - 10	3 - 8	105	120	Fine to medium sand
10 - 17	8 - 15	120	130	Coarse sand

Solution:

Layer (1)	ΔZ (in.) (2)	N (3)	E_s/N (4)	E_s (tsf) (5)	Z_c (in.) (6)	I_z (7)	$\frac{I_z}{E_s}\Delta Z$ (in./tsf) (8)
1	24	10	4	40	12	.20	0.120
2	24	16	4	64	36	.60	0.225
3	12	25	4	100	54	.50	0.060
4	12	25	7	175	66	.43	0.029
5	24	12	7	84	84	.33	0.094
6	24	20	7	140	108	.20	0.034
7	24	26	10	260	132	.07	0.006

$\Sigma = 0.568$

p_o = (2.0 ft)(95 pcf) = 190 psf = 0.095 tsf
Δp = 120 tons/(6 ft.)(8 ft.) = 2.50 tsf

At t = 1 yr,

$C_1 = 1 - 0.5(.095/2.50) = 0.981$

$C_2 = 1 + 0.2 \log (10)(1) = 1.20$

$\Delta H = (0.981)(1.20)(2.50)(0.568) =$ 1.67 in.

FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Page 6-026

Section 4. PRIMARY AND SECONDARY SETTLEMENTS

1. PRIMARY CONSOLIDATION.

a. Consolidation Settlement. For conditions where excess pore pressures are developed during the application of load and if preconsolidation stress is determined reliably, total settlement can be predicted with reasonable accuracy. The percentage error is greatest for settlement from recompression only. In this case an overestimate may result unless high quality undisturbed samples are used for consolidation tests.

(1) Typical Loading Cycle. See Figure 3 for loading sequence in building construction. Foundation excavation can cause swell and heave. Application of a structural load recompresses subsoil and may extend consolidation into the virgin compression range. Stress changes are plotted on a semilogarithmic pressure-void ratio e-log p curve similar to that shown in Figure 3.

(2) Pressure-Void Ratio Diagram. Determine the appropriate e-log p curve to represent average properties of compressible stratum from consolidation tests. The e-log p curve may be interpreted from straight line virgin compression and recompression slopes intersecting at the preconsolidation stress. Draw e-log p curve to conform to these straight lines as shown in Figure 3.

(3) Magnitude of Consolidation Settlement. Compute settlement magnitude from change in void ratio corresponding to change in stress from initial to final conditions, obtained from the e-log p curve (Figure 3). To improve the accuracy of computations divide the clay layer into a number of sublayers for computing settlement. Changes in compressibility of the stratum and existing and applied stresses can be dealt with more accurately by considering each sublayer independently and then finding their combined effect.

(4) Preliminary estimates of C_c can be made using the correlations in Table 3.

b. Corrections to Magnitude of Consolidation Settlements. Settlements computed for overconsolidated clays by the above procedures may give an overestimate of the settlement. Correct consolidation settlement estimate as follows:

$$H_c = \alpha \ (\Delta H)_{oc}$$

H_c = corrected consolidation settlement

α = function of overconsolidation ratio (OCR) and the width of loaded area and thickness of compressible stratum (See Figure 8 for values and Reference 6, Estimating Consolidation Settlements of Shallow Foundation on Overconsolidated Clay, by Leonards.)

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 3

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Consolidation Settlement of Compacted Admix

	Name	Date
Prepared By:	Timothy L. Martini	December 19, 1989
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT: Consolidation Settlement of the Compacted Admix

Job No. 893-1186.032 | Made by TLM | Date 12-19-89

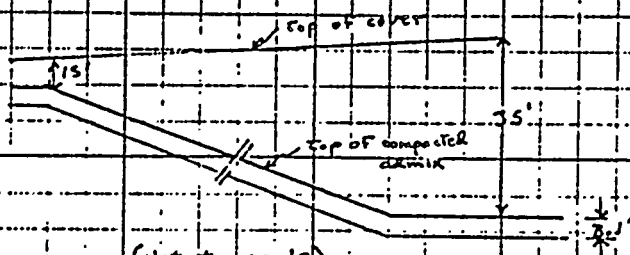
Ref. | Checked FSS | Sheet 2 of 3

Reviewed RDK

Objective: To determine the consolidation settlement of the compacted admix at the base of the landfill and at the top of slope.

Approach: Utilize laboratory consolidation test curve to calculate consolidation settlements within the compacted admix.

Geometry: Primary m/y - OK



Assumptions:
1) Maximum vertical overburden stress is 35 Ft from base of landfill
2) Vertical overburden stress at top of slope is 15 ft
3) Unit weight of waste and cover, approximately 120 pcf
4) Assume one-way drainage path, i.e., H_d = 37.2 Ft = 37.20 inches

Base consolidation settlement

$\Delta\sigma_v = (35)(120) = 4{,}200$ psf

From consolidation curve at 4,200 psf $\Rightarrow$ $\varepsilon = 0.04$ in/in

$\Delta H = (37.2 \text{ inches})\left(0.04 \frac{\text{in}}{\text{in}}\right) = 1.49$ inches

Top of slope consolidation settlement

$\Delta\sigma_v = (15)(120) = 1{,}800$ psf

From consolidation curve at 1,800 psf $\Rightarrow$ $\varepsilon = 0.029$ in/in

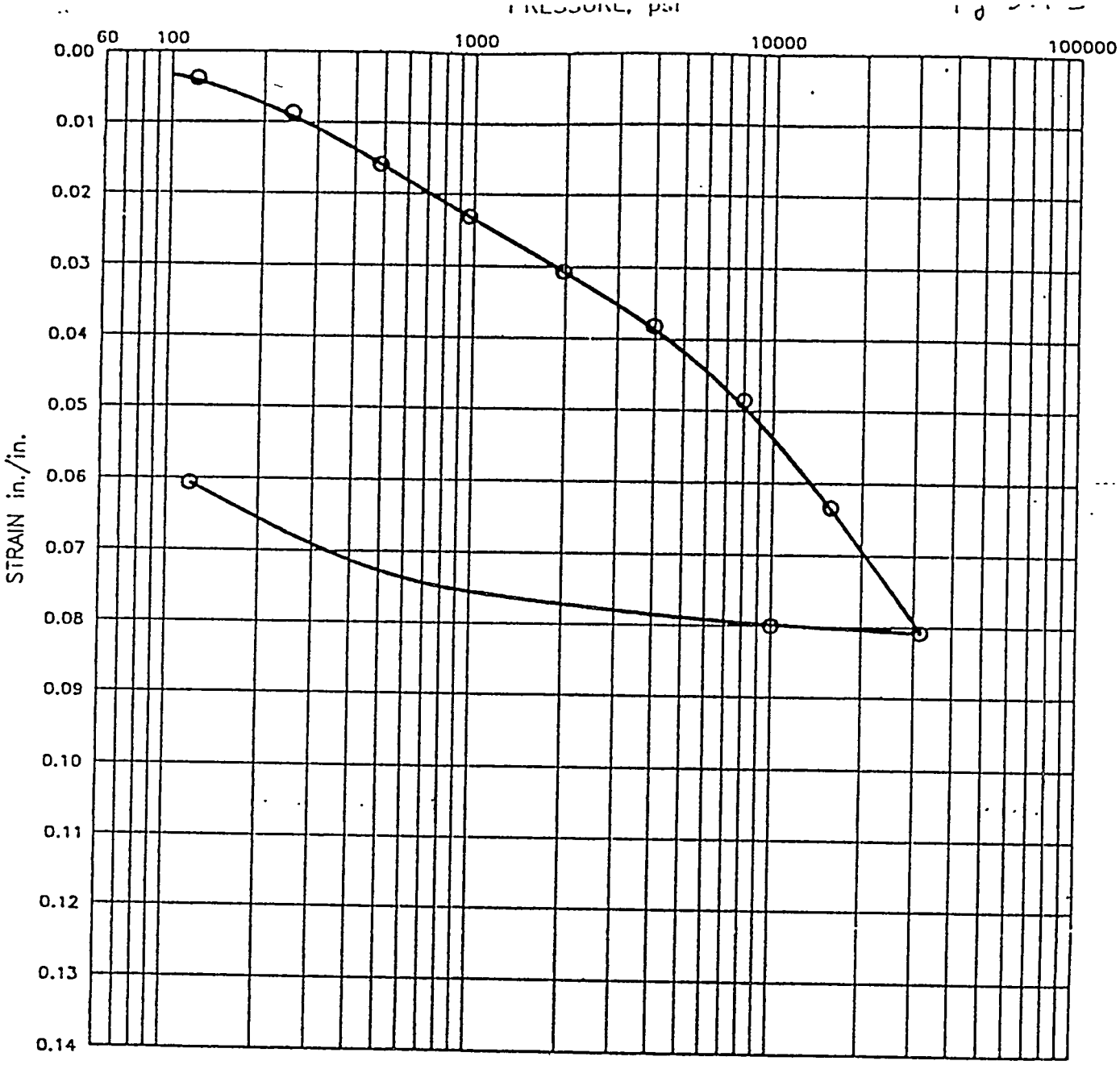
$\Delta H = (37.2 \text{ inches})(0.029 \text{ in/in})$

$\Delta H = 1.08$ inches

Maximum Differential settlement = 1.49 − 1.08

= 0.41 inches $\approx$ 0.5 inches

Conclusion: Consolidation settlement does not appear to be a problem



Boring	Boring	Soil Classification	Moisture Content Before	Moisture Content After	Dry Density in. lbs./cu. ft.
GTP1-7	8% Bent. Admix	Light olive gray, medium to fine SAND, some silty clay (SC)	19.6	21.7	104.2

CONSOLIDATION TEST

Project DOE-RL/NON-DRAGOFF/WA

Project No. 893-1186.035 Date 1/11/90 Tested By DN Approved By TM

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Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 2

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Settlement Summary - elastic and consolidation settlements.

	Name	Date
Prepared By:	Timothy L. Martino	January 10, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	FRANK. SHURI	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT Settlement Summary (Combined)

Job No. 893-1186 | Made by TLM | Date 1-10-90

Ref. | Checked FSS | Sheet 2 of 2

Reviewed gd

Objective: To summarize subgrade elastic settlement and liner consolidation settlement.

References: The Elastic Subgrade Settlement Analysis and the Consolidation Settlement of the Compacted Admix Analysis.

	Base Settlement	Top of Slope Settlement	Differential
Subgrade	2.29 inches ✓	1.93 inches ✓	0.36 inches ✓
Compacted Admix	1.49 inches ✓	1.08 inches ✓	0.41 inches ✓
Combined	3.78 inches ✓	3.01 inches ✓	0.77 inches ✓

Conclusions: The combined total and differential settlements are not anticipated to adversely affect the design and operation of the landfill. ✓

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 2

Project Title: Non-Drag-OFF Landfill Design

Project No. 893-1186.032

Calculation Title: Subsidence and Sinkhole Potential

	Name	Date
Prepared By:	Timothy L. Martin	January 9, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Project Name: Non-Drag-Off Landfill Design
Project Number: 893-1186
Date: January 9, 1990
Subject: SUBSIDENCE AND SINKHOLE POTENTIAL

Prepared by: TLM
Checked by:
Reviewed by:

Site Geology

Approximately 85 percent of the proposed Non-Drag-Off landfill will be constructed in Recent eolian sands. The sands extend to approximately 17 feet below the surface. Underlying the eolian sands is the coarse-grained sequence of the Hanford formation which consists of an interstratified deposit of coarse sand, gravelly sand, and/or sandy gravel. This coarse grained deposit is part of the Cold Creek Bar which was formed during the Pleistocene by proglacial floods. Based on previous borings, ("Hydrology of the 200 Areas Low-Level Burial Grounds - An Interm Report", Last, G.V., et al, January 1989) the Hanford formation is about 100 feet thick at the landfill site. A fine-grained sequence consisting of mostly silty sand to sandy silt is present at the base of the coarse-grained sequence and is often referred to as the basal slack water sequence of the Hanford formation. Units underlying the Hanford formation are: the early Palouse soil, the Plio-Pleistocene unit, the middle Ringold unit, and the Elephant Mountain Member. Ground water depths are approximately 270 feet below the ground surface.

Subsidence and Sinkhole Potential

The potential for subsidence and/or sinkhole development is negligible based on the following information:

- The soil underlying the landfill site tend to be coarse-grained sands and gravels which are not subject to piping which can cause transport of soil and resulting subsidence.
- The groundwater level is deep and lying approximately 250 feet below the base of the landfill and as a result will not impact bearing soils.
- No mining or tunneling has been noted in the area. No subsidence or sinkhole activity has been noted in the area. Extensive borings in the landfill area have not encountered any soluble materials in the foundation materials.
- No reserves of pertroleum have been noted and it is our understanding that there are no plans for withdrawing large quantities of ground water. Even if the ground water level was lowered substantially and consolidation occured in the aquifer, local site specific subsidence would be negligible due to the depth of the ground water below the landfill.
- Soils are well graded and relatively dense.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 2

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Potential For uplift From Gas or Hydraulic Pressures

	Name	Date
Prepared By:	Timothy L. Martin	January 9, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Project Name: Non-Drag-Off Landfill Design
Project Number: 893-1186
Date: January 9, 1990
Subject: POTENTIAL FOR UPLIFT FROM GAS OR HYDRAULIC PRESSURES

Prepared by: TLM
Checked by: TSB
Reviewed by: RW

Site Geology

Approximately 85 percent of the proposed Non-Drag-Off landfill will be constructed in Recent eolian sands. The sands extend to approximately 17 feet below the surface. Underlying the eolian sands is the coarse-grained sequence of the Hanford formation which consists of an interstratified deposit of coarse sand, gravelly sand, and/or sandy gravel. This coarse grained deposit is part of the Cold Creek Bar which was formed during the Pleistocene by proglacial floods. Based on previous borings, ("Hydrology of the 200 Areas Low-Level Burial Grounds - An Interm Report", Last, G.V., et al, January 1989) the Hanford formation is about 100 feet thick at the landfill site. A fine-grained sequence consisting of mostly silty sand to sandy silt is present at the base of the coarse-grained sequence and is often referred to as the basal slack water sequence of the Hanford formation. Units underlying the Hanford formation are: the early Palouse soil, the Plio-Pleistocene unit, the middle Ringold unit, and the Elephant Mountain Member. Ground water depths are approximately 270 feet below the ground surface.

Uplift Potential Due to Gas or Hydraulic Pressures

The potential for uplift on the composite liner system is very low. The static groundwater level is approximately 250 feet below the design depth of the landfill. Higher elevation perched groundwater from surface water infiltration is unlikely to develop due to the absence of aquitards in the coarse-grained Hanford formation underlying the landfill. The coarse-grained nature of the Hanford formation also promotes rapid, primarily vertical, infiltration which means it is unlikely that infiltration from outside the landfill boundary will be transported laterally underneath the landfill liner.

Gas pressures are similarly unlikely to develop due to the absence of any noted subsurface gas generation (from organic material decomposition) and the coarse-grained highly permeable sands and gravels underlying the landfill. Even in the unlikely event that the groundwater level would increase rapidly by 50 feet, the excess gas pressure generated by such an event would be dissipated by the coarse grained Hanford formation.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 10

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Earth Load - Primary Slope Riser Pipe

	Name	Date
Prepared By:	Timothy L. Martin	January 5, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT: Calculation of Earth Load – Primary Slope Riser

Job No. 893-1186.032 | Ref. | Made by TLM | Checked PES | Reviewed [illegible] | Date 1-5-90 | Sheet 2 of 10

Objective: To calculate the earth load being applied to the side slope riser pipes and to select a diameter and SDR rating for the pipes.

Approach: Use the modified Marston Formula to calculate earth loadings.

Reference: Design and Engineering Guide, Poly Pipe Industries, Inc. Sect. 6 "Earthloading"

Analysis:

Modified Marston equation:

$W = C_d \times w_s \times B_d \times D$ where: W = earth load in pounds per linear foot of pipe, lb/Ft.

C_d = coefficient dependent on soil type and height of backfill to width of trench ratio, (see Fig. 16, attached)

D = outside diameter of pipe, Ft.

B_d = trench width at top of pipe, Ft.

w_s = weight of soil, lbs/Ft³

Trial 1

Assume:

1) That the primary pipe is the worst case condition since it sits on the surface without being placed in an excavated trench.

2) Propose to compact soil during placement of the operation's layers to within 3 feet of the pipe which effectively results in the pipe being in a 6 Ft wide trench, i.e., B_d = 6 Ft ✓

3) Assume the fill is class II fill

4) $\gamma_{soil} \approx 120$ pcf ✓

5) The maximum height of the waste fill H is approximately 35 Ft.

6) Assume 18 inch nominal diameter, SDR 11, i.e., $D = 8.625/12 = 0.719$ ✓

$H/B_d = 35/6 \approx 5.8 \approx 6 \Rightarrow$ From Fig. 16 $C_d \approx 3.1$ ✓

$$W = (3.1)\left(120 \frac{lbs}{Ft^3}\right)(6\ Ft)(0.719\ Ft) = 1,604.35\ lbs/Ft \approx 1,604\ lbs/Ft$$ ✓

For Flexible HDPE Pipe, the deflection under earth loading is:

$$\Delta X = \frac{DL \times K \times W}{\frac{2E}{3(SDR)^3} + 0.061 E'}$$

where: ΔX = vertical deflection of pipe in inches

DL = deflection lag factor ≈ 1.50

K = bedding constant ≈ 0.10

W = earth load on pipe in lbs per linear foot

E = modulus of elasticity of polyethylene, psi

E' = soil modulus, psi

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SUBJECT Calculation of Earth Load - Primary Slope Riser Pipe

Job No. 893-1186.032 | Made by TLM | Date 1-5-90

Ref. | Checked PDS - w/ min corrections | Sheet 3 of 10

Reviewed TLM

$$\Delta X = \frac{(1.50)(0.10)(134\ lbs/in)}{(2)(30{,}000) + (0.061)(300)} \quad / \quad (3)(11^3)$$

Note: E = 30,000 psi assumed hand tamped to 65% of max density

$w = (160\ lbs/ft^3)(\frac{1\ ft}{12\ in}) = 13.4\ lbs/in$

$$\Delta X = \frac{20.1}{33.3} = 0.603\ in$$

$$\frac{\Delta X}{X} = \frac{0.603}{8} \times 100\% = 7.6\% > 5\% \quad N.G.$$

Trial 2; 8 inch nominal, SDR11, 3 Ft. wide trench.

Assume can compact the operations layer on both sides of the primary riser pipes to in effect create a 3 Ft wide trench, i.e., B_d = 3 Ft. All other conditions and assumptions remain the same.

$H/B_d = \frac{35}{3} \cong 12 \Rightarrow$ from Fig 16 $C_d \cong 3.7$

$w = (3.7)(120)(3)(0.719)$

$w = 957.7 \cong 958\ lbs/Ft \cong 80\ lbs/in$

$$\Delta X = \frac{(1.50)(0.10)(80)}{(2)(30{,}000) + (0.061)(300)} = \frac{12}{33.3} = 0.36\ inch$$

$$\frac{\Delta X}{X} = \frac{0.36}{8} = 4.5\% < 5\%^* \therefore OK$$

Conclusion:

Recommend using 8 inch nominal HDPE pipe, SDR11 with operations layer placed and compacted (track rolling OK) within 18 inches of the pipe to in effect, form a 3 Ft wide trench. Material placed directly over the pipes should be hand compacted with a walk-along vibratory roller after at least one Ft of soil is placed over the pipe.

* The maximum acceptable deflection of 5% is based on the recommendation in the design manual pg C-5 (pg 7 of 9 in this analysis package) which states: "A deflection of 15% would be acceptable for a butt fused polyethylene system although a reduction in flow would be noted. It would also be difficult to utilize conventional cleaning equipment with this severe deflection. For these reasons, Poly Pipe recommends maximum deflections of 5 to 7½%. Calculations in this guide have been based on 5%."

Non-Drag-off
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EARTHLOADING

POLY PIPE, due to its flexibility, will deflect when it is buried and is subjected to loads caused by the compressive forces of the compacted soil. The degree of deflection will depend upon the soil conditions, trench width, and the depth of burial. The degree of deflection of the pipe is limited by the soil around its periphery especially in the lateral direction. When the soil compacts around the pipe there is a supportive effect from the soil itself, and as compaction occurs there is a soil arch formed over the pipe which reduces the direct load on the pipe.

POLY PIPE, as do other flexible conduits, depends on the surrounding soil for support and has to be considered as one component in a pipe/soil system. Therefore, performance of POLY PIPE is determined as much by the quality and density of the soil underneath the pipe haunches and at its sides as by the properties of the pipe itself.

The presence of the soil arch and the support derived from the lateral movement limitations are highly beneficial to the efficiency of the system. The flexibility of POLY PIPE is the major reason for these advantages. As has been stated the durability of polyethylene is the reason for its resistance to high levels of mechanical abuse and this is no less true for buried systems where forced deflections may occur due to subsidence, washout and settlement and to surface traffic.

The earth load being applied to a buried system can be determined by the use of the modified Marston formula.

$$W = C_d \times w \times B_d \times D \qquad (10)$$

Where W – earth load in pounds per linear feet of pipe, lb./ft.
C_d – coefficient dependent on soil type and height of backfill to width of trench ratio. (see Fig. 16 for C_d)
w – weight of soil, lbs./cu.ft.
D – outside diameter of pipe, ft.
B_d – trench width at top of pipe, ft.

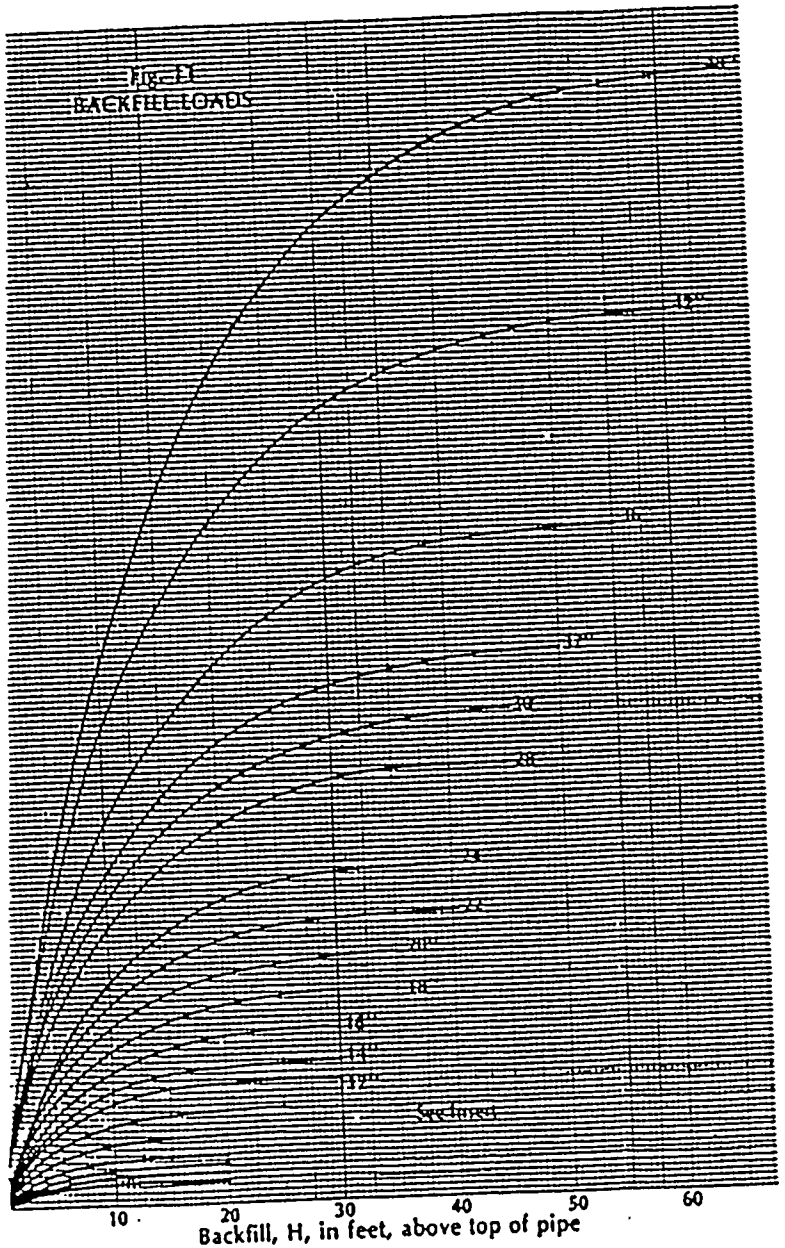
In general practice the trench width can be kept to a minimum of 6 inches per side greater than the pipe diameter itself. Although this may seem narrow in comparison to trenching of conventional materials it must be noted that POLY PIPE can be preassembled above ground and later placed into the trench. The trench width should be maintained as narrow as possible as the soil loading on the pipe is a relationship of the trench width

The loads applied to the pipe as a result of the backfill are given in Fig. 13.

In situations where the pipe is buried below the water table the effect is to relieve the load on the pipe to some extent due to buoyancy of the pipe. Although this can be a considerable effect, it should not be taken into account when designing a system as the water table can change to a position below the pipe. In the event the water level does not change, the reduction in stress will serve as an additional safety factor.

In calculating the soil load placed on a buried pipe, the designer must be able to calculate to some degree of accuracy the type and condition of the backfill material. Saturated clay would be more difficult to place and adequately compact than would coarse granular material that would not stick together. It is important in the pipe/soil system that the backfill material utilized for haunching and initial backfill (see Fig. 18 for explanation of terminology) be granular and non-cohesive, free of debris, organic matter, frozen earth and rocks larger than 1½ inch in diameter. This material can best be described as Class I or II of ASTM D2321 "angular ¼ to 1½ inch graded stone, slag, cinders, crushed shells and stone or sands and gravels containing small percentages of fines, generally granular and non-cohesive, wet or dry." This material can easily be worked into the pipe haunch, compacted in approximately 4 to 6 inch lifts to above the pipe.

C-1

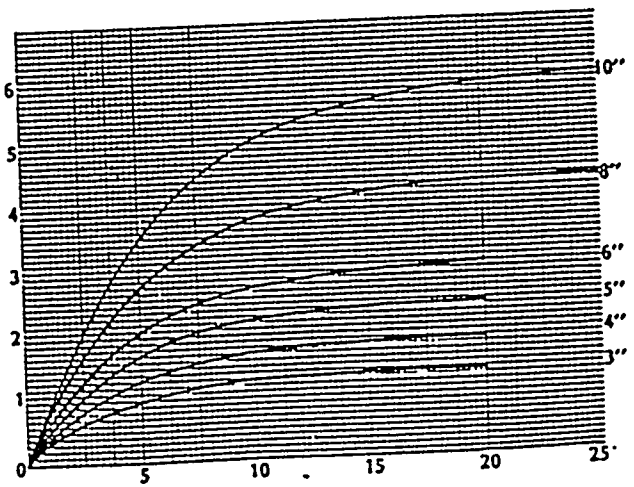


These curves were determined by using a 120 pound per cubic foot soil weight, trench widths of one foot greater than the pipe diameter, and a Class II backfill material as noted in Fig. 16.

Class II backfill materials can, on occasion be taken directly from the trench and replaced in an acceptable manner. Class I materials, in virtually all cases, would require off-site acquisition. Class III and IV materials, while time-consuming and difficult to work into the haunch and embedment zone at an acceptable compaction level, are generally available from the existing spoil.

Selection of these conditions was made as Class II backfill is considered optimum for backfilling in general. Class II soil at 90% density will effectively support the pipe/soil unit through the range of thinner wall pipes – SDR 17 to SDR 32.5. The designer should consider heavier wall pipe – SDR 13.5 and SDR 11 – when Class III and IV soils are utilized in the embedment zone.

Fig. 14 presents the designer with the estimated loading to obtain a maximum of 5% deflection in a round, flexible pipe. Should a more conservative deflection be desired the values in this table can be multiplied by the appropriate correction factor. It may, on occasion, indicate that the selection of a heavier wall pipe for a particular installation will present an economic advantage over a thinner wall pipe and improved compaction.



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deflection can be calculate

Where: ΔX = deflectio
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K = bedding
W = load on
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Modified for flexible

Where: Δx = vertica
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K = beddi
W = earth l
E = modul
E' = soil mo
SDR = OD/t

Note: Values of DI = de
Manual of Practice No.
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From the above for
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Where: Δx = defec
D = nomi

G-2

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Hand or mechanical tamping of the side-fill to an 85% Standard Proctor Density will significantly reduce the soil pressure on the pipe. If the side-fill is more yielding than the pipe, the pipe will carry a larger part of the soil load. If the side-fill is stiffer than the pipe, a soil-arch or "bridge" will form over the pipe that will support a greater portion of the total load. This compacted side-fill also adds stiffness to the pipe. With adequate stiffness in the side-fill, the pipe is restrained from deflecting due to soil loads in the vertical and expansion in the horizontal. Therefore, it may be said the performance of a buried flexible pipe is dependent upon the selection, placement and compaction of the earthen envelope.

Under normal circumstance it is essential to limit the deflection of the pipe to a designed quantity. The deflection can be calculated from the following equation.

$$\Delta X = \frac{DI \times K \times W}{\frac{EI}{R^3} + 0.061\ E'}$$

Where: ΔX – deflection in inches
DI – lag factor, assumed between 1.25 and 1.50. (1.50)
K – bedding class factor, conservatively K – 0.1
W – load on pipe in pounds per linear inch
R – mean radius in inches
E – modulus of elasticity of pipe (30,000 psi conservatively)
I – moment of inertia – $T^3/12$
where T – wall thickness, in.
E' – modulus of passive soil resistance

This equation is known as the Spangler equation.

Modified for flexible **POLY PIPE** this formula becomes:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3\ (SDR)^3} + 0.061\ E'} \qquad (11)$$

Where: Δx – vertical deflection of pipe, inches
DI – deflection lag factor
K – bedding constant
W – earth load on pipe in pounds per linear inch
E – modulus of elasticity of polyethylene, psi
E' – soil modulus, psi
SDR – OD/t

Note: Values of DI – deflection lag factor 1.50 and K – bedding constant of 0.10 are taken from WPCF Manual of Practice No. 9 as is the selection of the soil modulus of 300 psi when the side fill is hand tamped to 65% of maximum density and 700 psi when side fill is mechanically tamped to 90% or more maximum density.

From the above formula the deflection, Δx, of a pipe can be calculated. From this the percent deflection can then be calculated by

$$\% \text{ def} = \frac{\Delta x}{D} \times 100 \qquad (12)$$

Where: Δx – deflection in inches
D – nominal pipe outside diameter, inches

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Fig. 14

Maximum recommended external loading with 5% deflection, pounds per linear foot of pipe.

Nominal Pipe Size, in.	Soil Modulus, psi	SDR 32.5	26	21	17	13.5
3	300			285	315	370
	700			630	655	710
4	300		350	370	405	475
	700		790	810	840	915
5	300	420	435	455	500	590
	700	965	975	1000	1040	1130
6	300	500	515	540	595	700
	700	1145	1160	1190	1240	1345
8	300	650	670	705	770	910
	700	1495	1510	1550	1615	1755
10	300	810	835	880	960	1135
	700	1860	1885	1930	2010	2185
12	300	965	990	1045	1140	1350
	700	2205	2235	2290	2385	2590
14	300	1055	1090	1145	1255	1480
	700	2425	2455	2510	2620	2845
16	300	1210	1245	1310	1430	1690
	700	2770	2805	2870	2995	3255
18	300	1360	1400	1475	1610	1905
	700	3115	3155	3230	3370	3660
20	300	1510	1555	1635	1790	2115
	700	3465	3505	3590	3740	4065
22	300	1660	1710	1800	1970	2325
	700	3810	3855	3950	4115	4475
24	300	1815	1865	1965	2150	2535
	700	4155	4210	4305	4490	4880
28	300	2115	2175	2290	2505	2960
	700	4845	4910	5025	5240	5695
30	300	2265	2335	2455	2685	3170
	700	5195	5260	5385	5615	6100
32	300	2415	2490	2620	2865	3385
	700	5540	5610	5740	2990	6505
36	300	2720	2780	2945	3220	
	700	6235	6310	6460	6735	
42	300	3170	3265	3435		
	700	7270	7365	7535		
48	300	3625	3730			
	700	8310	8415			

G-4

Listed below are the recommended maximum allowable design limits for deflection of polyethylene pipe for the different available standard dimension ratios.

Dimension Ratio	Safe Deflection (% of diameter)
32.5	8.5
26	7.0
21	6.0
17	5.0
13.5	4.0

Fig. 14 depicts the load imposed on a pipe when recommended installation practices are followed. In this Fig. a value of 120 #/ft³ soil and for Class II soils a ratio of lateral to vertical pressures of 0.17. For other classes of soil as noted in Fig. 16, Class I - 0.19, Class III - 0.15, Class IV - 0.13, and Class V - 0.11. As can be seen from Fig. 13, there is a depth at which increased loading on the pipe ceases and the load level remains constant, regardless of the height of backfill.

POLY PIPE through its inherent physical properties of flexibility, resilience, and toughness can withstand significant deflection without failure. It can be flattened without causing a fracture of the pipe wall. However, this condition is unacceptable as far as service is concerned. A deflection of 15% would be acceptable for a butt fused polyethylene system although a reduction in flow would be noted. It would also be difficult to utilize conventional cleaning equipment with this severe a deflection. For these reasons, POLY PIPE recommends maximum deflections of 5 to 7½%. Calculations in this guide have been based on 5%.

Compaction or quality of backfill above the pipe zone, while important for many reasons, does not effect the predicted deflection of the pipe. The designer may wish to review Sec. H, Underground Installations, for further considerations.

Some design considerations to be recognized in the installation of buried pipelines is the exercise of caution in the tamping of the side-fill so as to not force granular, free flowing, non-cohesive Class I & II bedding materials under the haunch in sufficient force to "float" the pipe. It is imperative that buried pipe be on a firm, continuous support. Final backfill material, while unimportant to the pipe zone, should be free of large rocks, boulders or other similar materials that could project down into the pipe.

The designer may wish to review Sec. C, Critical Buckling, of this design guide in conjunction with this section.

For a full discussion of the formula and the derivative work involved the reader is referred to Bulletin 153 of the Iowa State College Bulletin "The Structural Design of Flexible Pipe Culverts" by M.G. Spangler.

Fig. 15

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Classification of Backfill Material

per ASTM D2321*

Class	Comments
Class I — Angular graded stone, ¼ to 1½ in., including a number of fill materials that have regional significance such as coral, slag, cinders, crushed stone, crushed gravel, and crushed shells.	100 - 120 pounds per cubic foot. Pipe sizes less than 10-inch should limit maximum particle size to ½ to ¾-inch for ease of placement.
Class II — Coarse sands and gravels with maximum particle size of 1½ in., including variously graded sands and gravels containing small percentages of fines, generally granular and non-cohesive, either wet or dry.	110 - 130 pounds per cubic foot. Pipe sizes less than 10-inch should limit maximum particle size to ½ to ¾-inch for ease of placement.
Class III — Fine sand and clayey gravels, including fine sands, sand-clay mixtures, and gravel-clay mixtures.	140 - 150 pounds per cubic foot.
Class IV — Silt, silty clays, and clays, including inorganic clays and silts of medium to high plasticity and liquid limits.	150 - 180 pounds per cubic foot.
Class V — Includes organic soils as well as soils containing frozen earth, debris, rocks larger than 1½ in. in diameter and other foreign materials.	Not recommended for backfill except in the final backfill zone.

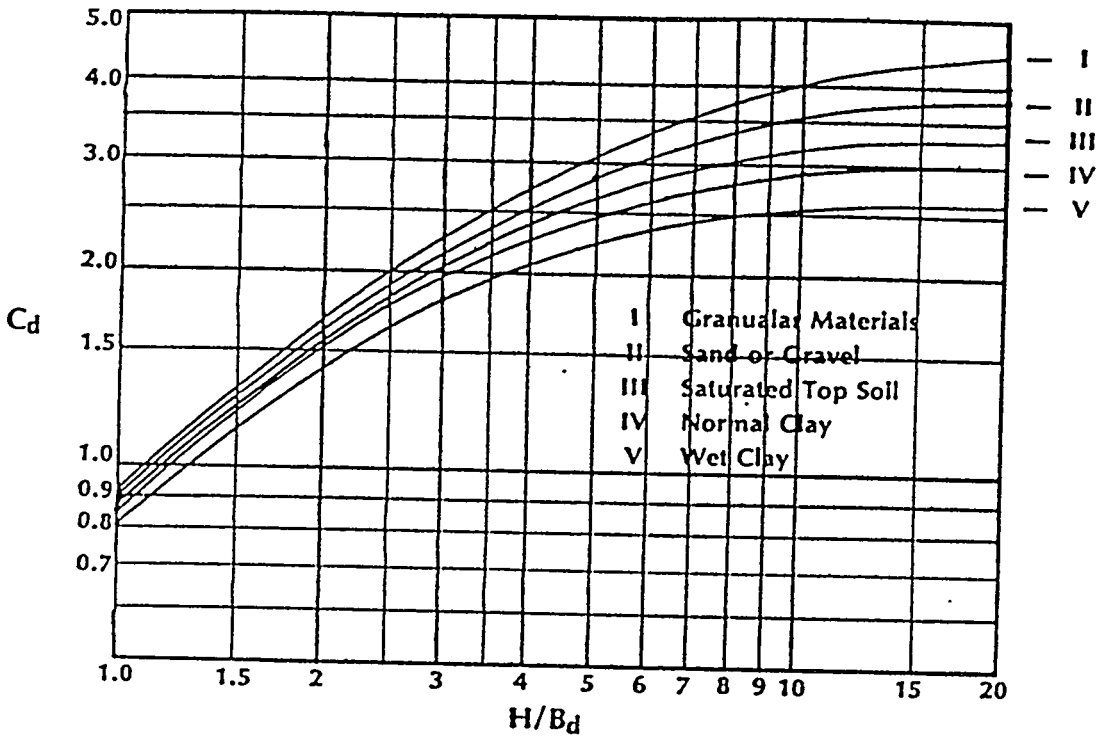
*For further classification of soils the designer may want to review ASTM D2487, Standard Test Method for Classification of Soils for Engineering Purposes.

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Fig. 16

Coefficient, C_d, dependent on soil type and ditch configuration.



Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 10

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Earth Load - Primary Drainage Pipe

	Name	Date
Prepared By:	Timothy L. Martin	January 11, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[signature]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT Calculation of Earth Load - Primary Drainage Pipe

Job No. 893-1186.032 | Made by TLM | Date 1-11-90

Ref. | Checked [illegible] | Sheet 2 of 10

Reviewed [illegible]

Objective: To calculate the earth load being applied to the primary drainage pipes which are located at the toe of slope and on the center floor drainage break.

Approach: Use the modified Marston formula to calculate earth loading.

Reference: Design and Engineering Guide, Poly Pipe Industries, Inc. Section 6 "Earthloading"

Analysis:

Modified Marston Equation:

$W = C_d \cdot w_s \cdot B_d \cdot D$ where W = earthload in pounds per linear foot of pipe (lbs/ft).

C_d = coefficient dependent on soil type and height of backfill to width of trench ratio, (see Fig 16, attached)

D = outside diameter of pipe, Ft.

B_d = trench width at top of pipe, ft.

w_s = weight of soil, lbs/ft^3

Trial 1:

Assume:

1) The pipe will be placed on surface and the operations layer will be compacted to within 18 inches of the pipe which creates a virtual "trench" for the pipe 3 ft wide.

2) Assume Fill is Class II fill.

3) $\gamma_{soil} \approx 120$ pcf

4) The maximum height of soil/waste is approximately 35 ft.

5) Assume 4 inch nominal diameter, SDR11, i.e., D = 4.5 inches (.375 Ft)

$H/B_d = \frac{35}{3} = 11.67 \approx 12 \Rightarrow$ from Fig 16, $C_d \approx 3.7$

$W = (3.7)(120)(3)(0.375) = 499.5$ lbs/ft $\approx$ 500 lbs/ft $\approx$ 42 lbs/inch

For flexible HDPE pipe, the deflection under Earth loading is:

$$\Delta X = \frac{DL \times K \times W}{\frac{2E}{3(SDR^2)} + 0.061E'}$$

where: ΔX = vertical deflection of Pipe in inches

DL = deflection lag factor = 1.50

K = bedding constant = 0.10

W = earth load on pipe in lbs per linear inch.

E = modulus of elasticity of polyethylene, psi

E' = soil modulus, psi

SDR = OD/t

Golder Associates

SUBJECT Calculation of Earth Load - Primary Drainage Pipe

Job No. 893-1186.032 | Made by TLM | Date 1-11-90

Ref. | Checked [illegible] | Sheet 3 of 10

Reviewed [illegible]

(continued)

$$\Delta X = \frac{(1.50)(0.10)(42)}{(2)(30{,}000) + 0.061(30^3)} \qquad \text{Note: } E = 30{,}000 \text{ psi}$$

$$\frac{3(12^3)}{}$$ $E' = 300$ psi (assumed hand tamped to 65% of max. density.)

$\Delta X = 0.189$ inch

$\frac{\Delta X}{X} = \frac{0.189}{4} = 0.047 \times 100\% = 4.7\% < 5\%^{*}$ ∴ OK

Conclusion:

Recommend using 4 inch nominal HDPE pipe, SDR11 with operations layer placed and compacted (track rolling, OK) within 18 inches of the pipe which will create a virtual 3 Ft wide "trench" for the pipe material placed directly over the pipe should be hand compacted with a walk-along vibrating roller after at least one Ft of soil is placed over the pipe.

* The maximum acceptable deflection of 5% is based on the recommendation in the design manual pg G-5 (pg 7 of 9 in this analysis package) which states: "A deflection of 15% would be acceptable for a butt fused polyethylene system although a reduction in flow would be noted. It would also be difficult to utilize conventional cleaning equipment with this severe a deflection. For these reasons, Poly Pipe recommends maximum deflections of 5 to 7½%. Calculations in this guide have been based on 5%."

Non-Drag-off
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EARTHLOADING

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POLY PIPE, due to its flexibility, will deflect when it is buried and is subjected to loads caused by the compressive forces of the compacted soil. The degree of deflection will depend upon the soil conditions, trench width, and the depth of burial. The degree of deflection of the pipe is limited by the soil around its periphery especially in the lateral direction. When the soil compacts around the pipe there is a supportive effect from the soil itself, and as compaction occurs there is a soil arch formed over the pipe which reduces the direct load on the pipe.

POLY PIPE, as do other flexible conduits, depends on the surrounding soil for support and has to be considered as one component in a pipe/soil system. Therefore, performance of POLY PIPE is determined as much by the quality and density of the soil underneath the pipe haunches and at its sides as by the properties of the pipe itself.

The presence of the soil arch and the support derived from the lateral movement limitations are highly beneficial to the efficiency of the system. The flexibility of POLY PIPE is the major reason for these advantages. As has been stated the durability of polyethylene is the reason for its resistance to high levels of mechanical abuse and this is no less true for buried systems where forced deflections may occur due to subsidence, washout and settlement and to surface traffic.

The earth load being applied to a buried system can be determined by the use of the modified Marston formula.

$$W = C_d \times w \times B_d \times D \qquad (10)$$

Where W – earth load in pounds per linear feet of pipe, lb./ft.
C_d – coefficient dependent on soil type and height of backfill to width of trench ratio. (see Fig. 16 for C_d)
w – weight of soil, lbs./cu.ft.
D – outside diameter of pipe, ft.
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In general practice the trench width can be kept to a minimum of 6 inches per side greater than the pipe diameter itself. Although this may seem narrow in comparison to trenching of conventional materials it must be noted that POLY PIPE can be preassembled above ground and later placed into the trench. The trench width should be maintained as narrow as possible as the soil loading on the pipe is a relationship of the trench width

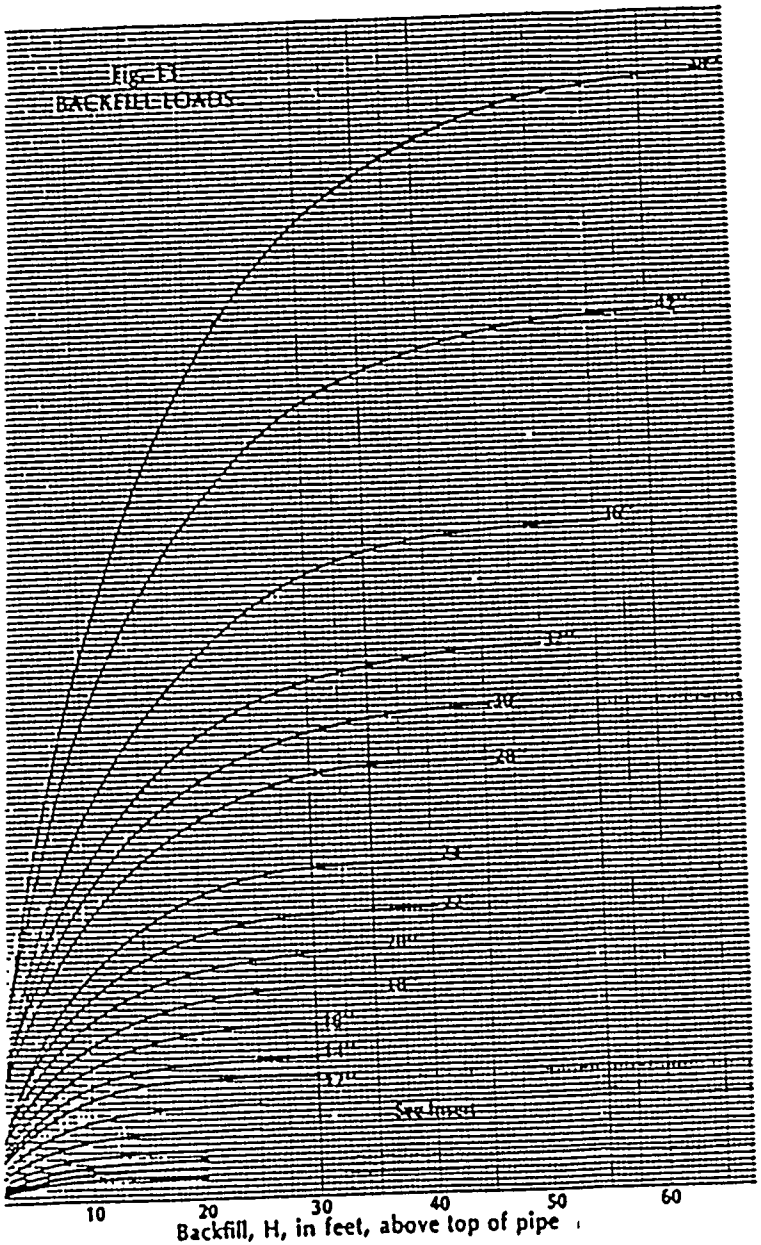
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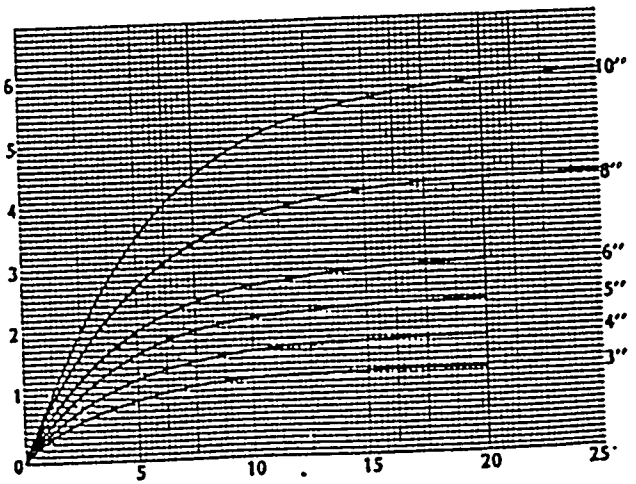


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Under normal circumstance it is essential to limit the deflection of the pipe to a designed quantity. The deflection can be calculated from the following equation.

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Modified for flexible POLY PIPE this formula becomes:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3\,(SDR)^3} + 0.061\, E'} \qquad (11)$$

Where: Δx – vertical deflection of pipe, inches
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E – modulus of elasticity of polyethylene, psi
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Note: Values of DI – deflection lag factor 1.50 and K – bedding constant of 0.10 are taken from WPCF Manual of Practice No. 9 as is the selection of the soil modulus of 300 psi when the side fill is hand tamped to 65% of maximum density and 700 psi when side fill is mechanically tamped to 90% or more maximum density.

From the above formula the deflection, Δx, of a pipe can be calculated. From this the percent deflection can then be calculated by

$$\%\ \text{def} = \frac{\Delta x}{D} \times 100 \qquad (12)$$

Where: Δx – deflection in inches
D – nominal pipe outside diameter, inches

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Fig. 14

Maximum recommended external loading with 5% deflection, pounds per linear foot of pipe.

Nominal Pipe Size, in.	Soil Modulus, psi	SDR 32.5	26	21	17	13.5
3	300			285	315	370
	700			630	655	710
4	300		350	370	405	475
	700		790	810	840	915
5	300	420	435	455	500	590
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	700	1145	1160	1190	1240	1345
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	700	2425	2455	2510	2620	2845
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	700	2770	2805	2870	2995	3255
18	300	1360	1400	1475	1610	1905
	700	3115	3155	3230	3370	3660
20	300	1510	1555	1635	1790	2115
	700	3465	3505	3590	3740	4065
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	700	8310	8415			

G-4

Listed below are the recommended maximum allowable design limits for deflection of polyethylene pipe for the different available standard dimension ratios.

Dimension Ratio	Safe Deflection (% of diameter)
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17	5.0
13.5	4.0

Fig. 14 depicts the load imposed on a pipe when recommended installation practices are followed. In this Fig. a value of 120 #/ft³ soil and for Class II soils a ratio of lateral to vertical pressures of 0.17. For other classes of soil as noted in Fig. 16, Class I – 0.19, Class III – 0.15, Class IV – 0.13, and Class V – 0.11. As can be seen from Fig. 13, there is a depth at which increased loading on the pipe ceases and the load level remains constant, regardless of the height of backfill.

POLY PIPE through its inherent physical properties of flexibility, resilience, and toughness can withstand significant deflection without failure. It can be flattened without causing a fracture of the pipe wall. However, this condition is unacceptable as far as service is concerned. A deflection of 15% would be acceptable for a butt fused polyethylene system although a reduction in flow would be noted. It would also be difficult to utilize conventional cleaning equipment with this severe a deflection. For these reasons, POLY PIPE recommends maximum deflections of 5 to 7½%. Calculations in this guide have been based on 5%.

Compaction or quality of backfill above the pipe zone, while important for many reasons, does not effect the predicted deflection of the pipe. The designer may wish to review Sec. H, Underground Installations, for further considerations.

Some design considerations to be recognized in the installation of buried pipelines is the exercise of caution in the tamping of the side-fill so as to not force granular, free flowing, non-cohesive Class I & II bedding materials under the haunch in sufficient force to "float" the pipe. It is imperative that buried pipe be on a firm, continuous support. Final backfill material, while unimportant to the pipe zone, should be free of large rocks, boulders or other similar materials that could project down into the pipe.

The designer may wish to review Sec. C, Critical Buckling, of this design guide in conjunction with this section.

For a full discussion of the formula and the derivative work involved the reader is referred to Bulletin 153 of the Iowa State College Bulletin "The Structural Design of Flexible Pipe Culverts" by M.G. Spangler.

Fig. 15

Non-Drag-off
893-1186

Pg 9 of 10

Classification of Backfill Material

per ASTM D2321*

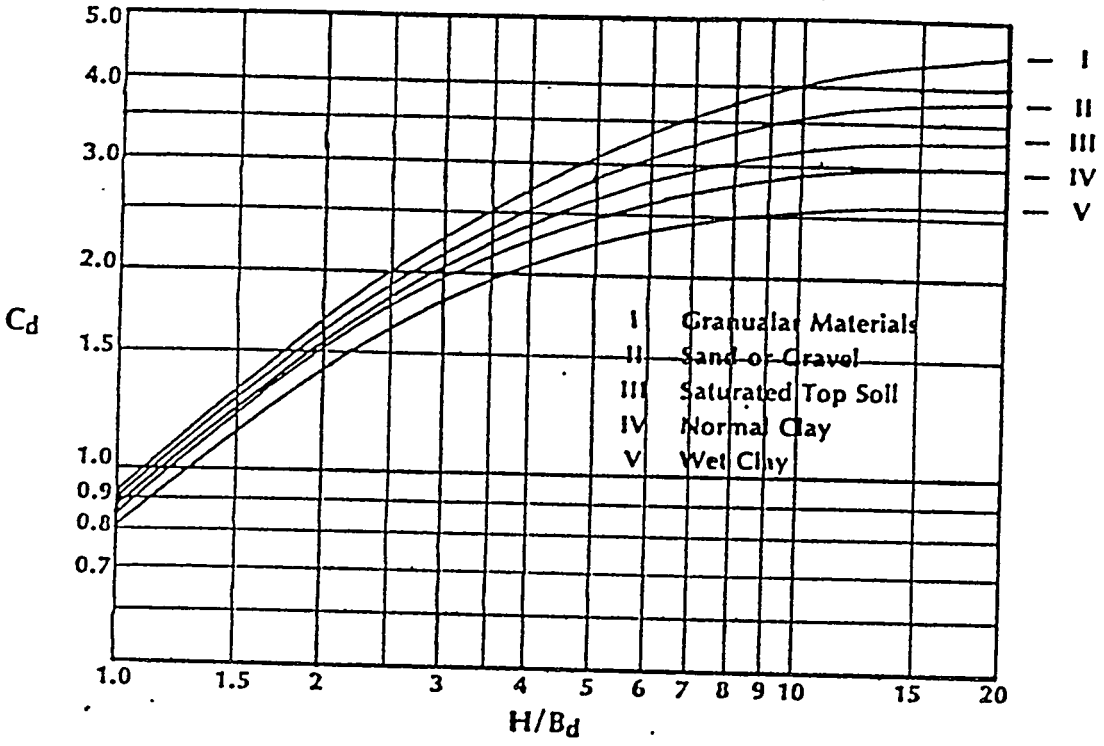
Class	Comments
Class I — Angular graded stone, ¼ to 1½ in., including a number of fill materials that have regional significance such as coral, slag, cinders, crushed stone, crushed gravel, and crushed shells.	100 - 120 pounds per cubic foot. Pipe sizes less than 10-inch should limit maximum particle size to ½ to ¾-inch for ease of placement.
Class II — Coarse sands and gravels with maximum particle size of 1½ in., including variously graded sands and gravels containing small percentages of fines, generally granular and non-cohesive, either wet or dry.	110 - 130 pounds per cubic foot. Pipe sizes less than 10-inch should limit maximum particle size to ½ to ¾-inch for ease of placement.
Class III — Fine sand and clayey gravels, including fine sands, sand-clay mixtures, and gravel-clay mixtures.	140 - 150 pounds per cubic foot.
Class IV — Silt, silty clays, and clays, including inorganic clays and silts of medium to high plasticity and liquid limits.	150 - 180 pounds per cubic foot.
Class V — Includes organic soils as well as soils containing frozen earth, debris, rocks larger than 1½ in. in diameter and other foreign materials.	Not recommended for backfill except in the final backfill zone.

*For further classification of soils the designer may want to review ASTM D2487, Standard Test Method for Classification of Soils for Engineering Purposes.

Non-Drag. off
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Fig. 16

Coefficient, C_d, dependent on soil type and ditch configuration.



Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 3

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Down-Drag Analysis For Vertical Riser Pipe

	Name	Date
Prepared By:	Timothy L. Martin	December 8, 1989
Checked By:	Frank S. Shuri	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT: Down-Drag Analysis For vertical riser pipe

Job No. 893-1186.032 | Made by TLM | Date 12-8-89

Ref. | Checked [illegible] | Sheet 2 of 3

Objective: To determine the downward drag force on the vertical riser pipe.

Reference: Principles of Foundation Engineering, Braja M. Das, pg 408.

Assumptions:
- Outside diameter of the vertical riser pipe is 4.5 ft.
- Smooth HDPE will be placed against the riser pipe to reduce skin friction.
- Assume a friction angle between the concrete and the HDPE of 18 degrees. (0.5ϕ)
- Assume 35 ft total fill height

$f_n = k' \sigma_v' \tan\delta$, where f_n = downward skin stress

k' = earth pressure coefficient = $K_o = 1 - \sin\phi$

σ_v' = vertical effective stress at any depth $Z = \gamma_f' Z$

γ_f' = Effective unit weight of fill

δ = HDPE - concrete friction angle

Total downward drag force, $Q_n = \frac{\pi d k' \gamma_f' H_f^2 \tan\delta}{2}$

where H_f = height of the fill

$k_o = 1 - \sin 38° = 0.38$ ✓

$Q_n = \frac{\pi (4.5)(1-\sin 38°)(120)(35)^2 \tan 18°}{2}$

Q_n = 129,760 pounds ✓

Weight of concrete riser pipe sections ✓ ✓ ✓

4 Ft nominal diameter — $(4.5)\left(\frac{2.5}{12}\right)(4)(150) = 1,571$ pounds/section

2.5 inches thick

36 Ft high (9 sections) — Total of 9 sections = 14,139 pounds

$\gamma_{concrete} = 150$ pcf — Base = $(6' \times 6' \times 0.5')(150)$ = 2,700 pounds ✓

Total Force = self weight + downdrag

= 16,839 + 129,760

= 146,599 pounds

Total stress on soil = 146,599/36 = 4072 psf

Stress on clay layer assuming a 1:1 stress distribution, clay under 1.5' gravel

146,599/(9^2) = 1,810 psf ✓

Note: overburden stress of fill 35 ft thick = 4200 psf = (35')(120 p.c.f.)

Conclusion: Downdrag plus self weight will not exceed overburden stress of fill.

Job No. 893-1186.032 Sheet 3 of 3
Date 12-8-89
Made By: TKM
Checked By: PSE
Reviewed By:

Clay Fill Over Granular Soil (Figure 8.35a)

Similar to the β method presented in Section 8.6, the negative (downward) skin stress on the pile can be given by the relation

$$f_n = K'\sigma_o' \tan \delta \tag{8.109}$$

where K' = earth pressure coefficient = $K_o = 1 - \sin \phi$
σ_o' = vertical effective stress at any depth $z = \gamma_f' z$
γ_f' = effective unit weight of fill
δ = soil-pile friction angle ≈ 0.5–0.7ϕ

Hence, the total downward drag force, Q_n, on a pile is

$$Q_n = \int_0^{H_f} (pK'\gamma_f' \tan \delta) z \, dz = \frac{pK'\gamma_f' H_f^2 \tan \delta}{2} \tag{8.110}$$

where H_f = height of the fill

If the fill is above the water table, the effective unit weight γ_f' should be replaced by the moist unit weight.

Granular Soil Fill Over Clay (Figure 8.35b)

In this case, there is enough evidence to show that the negative skin stress on the pile may exist from $z = 0$ to $z = L_1$, which is referred to as the *neutral depth* (see Vesic, 1977, pp. 25–26 for discussion). The neutral depth may be given as (Bowles, 1982)

$$L_1 = \frac{(L - H_f)}{L_1}\left[\frac{L - H_f}{2} + \frac{\gamma_f' H_f}{\gamma'}\right] - \frac{2\gamma_f' H_f}{\gamma'} \tag{8.111}$$

where γ_f' and γ' = effective unit weights of the fill and the underlying clay layer, respectively

Once the value of L_1 is determined, the downward drag force can be obtained in the following manner: The unit negative skin friction at any depth from $z = 0$ to $z = L_1$ is

$$f_n = K'\sigma_o' \tan \delta \tag{8.112}$$

where $K' = K_o = 1 - \sin \phi$
$\sigma_o' = \gamma_f' H_f + \gamma' z$
$\delta = 0.5$–0.7ϕ

Hence, the total drag force

$$Q_n = \int_0^{L_1} p f_n \, dz = \int_0^{L_1} pK'(\gamma_f' H_f + \gamma' z) \tan \delta \cdot dz$$

$$= (pK'\gamma_f' H_f \tan \delta) L_1 + \frac{L_1^2 pK'\gamma' \tan \delta}{2} \tag{8.113}$$

If the soil and the fill are above the water table, the effective unit weights should be replaced by moist unit weights. In some cases, the piles can be coated with asphalt in the down-drag zone to avoid this problem.

Example 8.15

Refer to Figur
diameter of 0.305 m
and $\phi = 32°$. Deter

Solution

From Eq. (8.110)

$$Q_n = \frac{pK'\gamma_f'}$$

$$p = \pi(0.3$$

$$K' = 1 - \sin$$

$$\delta = (0.6)(3$$

$$Q_n = \frac{(0.305}$$

Example 8.16

Refer to Figu
$\phi_{clay} = 34°$, $\gamma_{moist(clay)}$
top of the clay laye

Solution

The depth to the n

$$L_1 = \frac{L -$$

Note that γ_f' in Ec
table. So

$$L_1 = \frac{(20 -}{L}$$

$$L_1 = \frac{242.}{L_1}$$

or

$$L_1 = 11.7$$

Now, referring to

$$Q_n = (pK$$

$$p = \pi(0$$

$$K' = 1 -$$

$$Q_n = (0.$$

$$+ ($$

$$= 60.$$

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186.032

Calculation Title: Earth load - Leachate Collection well

	Name	Date
Prepared By:	Timothy L. Martin	January 19, 1990
Checked By:	Frank S. Shen	2-5-90
Approved By:	[illegible]	2-5-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder Associates

SUBJECT Calculation of Earthload - Leachate Collection Well

Job No. 893-1186.032 | Made by TLM | Date 1-19-90

Ref. | Checked TSS | Sheet 2 of 11

Reviewed RDL

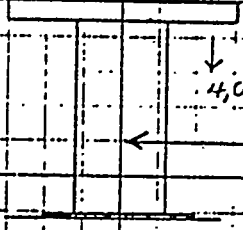
Objective: To calculate the earth load being applied to the leachate collection well and to select the SDR rating for the pipe.

Approach: Use horizontal earth load to calculate the pipe loading.

Reference: Principles of Foundation Engineering, Das, B. M.
Design and Engineering Guide, Poly Pipe Industries, Inc.
Section 6, Earthloading.

Assumptions: 1) The pipe soil interaction model Ko conditions, i.e., the pipe will not move (at least enough to develop Ka conditions). Analysis will be conservative when based on Ko conditions.
2) The vertical load is the same as calculated on the downdrag analysis on the vertical riser, i.e., 4,072 psf.
3) Assume ϕ sump gravel = 40°.

Geometry:



Analysis:

K_o = lateral earth pressure coefficient at rest

$K_o = 1 - \sin\phi$

$K_o = 1 - \sin 40°$

$K_o = 0.36$

σ_h = horizontal stress

σ_v = vertical stress = 4,072 psf (as per Assumption 2)

$\sigma_h = K_o \sigma_v$

$\sigma_h = (0.36)(4,027)$

$\sigma_h = 1,449.7 \approx 1,450$ psf

w = earthload in lbs/ft of pipe

$w = \left(1,450 \frac{lbs}{ft^2}\right)\left(\frac{30 \text{ in}}{12 \text{ in/ft}}\right) = 3,625 \frac{lbs}{ft}$

Golder Associates

SUBJECT Calculation of Earthload - Leachate collection well

Job No. 893-1186.032 | Made by TLM | Date 1-19-90
Ref. | Checked FSS | Sheet 3 of 11
Reviewed

For Flexible HDPE pipe, the deflection under earth loading:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3(SDR)} + 0.061E'}$$

Δx = vertical deflection of pipe in inches.
DI = deflection lag factor ≈ 1.50
K = bedding constant = 0.10
W = Earthload on pipe in lbs/linear inch
E = modulus of elasticity of polyethylene (psi)
E' = Soil modulus (psi) (700 psi)
SDR = OD/t = 13.5 (assumed OK)

$$\Delta x = \frac{(1.5)(0.10)\left(\frac{3625}{12}\right)}{\frac{2(30)(0.9)}{3(13.5^2)} + 0.061(700)}$$

$$\Delta x = \frac{45.31}{8.13 + 42.7} = 0.89 \text{ inches}$$

$$\left|\frac{\Delta x}{x}\right| = \frac{0.89}{30} \times 100\% = 2.97\% < 5\% \text{ OK}$$

Conclusion:
Recommend using the 30 inch nominal HDPE pipe, SDR 13.5 as per the design for the leachate collection well. The maximum acceptable deflection of 5% was based on the design manual pg G-5, attached.

Deflection assuming E_{soil} = 300 is 1.24" = 4.5%, also OK.

Note: this analysis is very conservative, as it assumes loading is essentially uniaxial, rather than uniform radial. As such, it is closer to a buckling analysis. FSS

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EARTHLOADING

POLY PIPE, due to its flexibility, will deflect when it is buried and is subjected to loads caused by the compressive forces of the compacted soil. The degree of deflection will depend upon the soil conditions, trench width, and the depth of burial. The degree of deflection of the pipe is limited by the soil around its periphery especially in the lateral direction. When the soil compacts around the pipe there is a supportive effect from the soil itself, and as compaction occurs there is a soil arch formed over the pipe which reduces the direct load on the pipe.

POLY PIPE, as do other flexible conduits, depends on the surrounding soil for support and has to be considered as one component in a pipe/soil system. Therefore, performance of POLY PIPE is determined as much by the quality and density of the soil underneath the pipe haunches and at its sides as by the properties of the pipe itself.

The presence of the soil arch and the support derived from the lateral movement limitations are highly beneficial to the efficiency of the system. The flexibility of POLY PIPE is the major reason for these advantages. As has been stated the durability of polyethylene is the reason for its resistance to high levels of mechanical abuse and this is no less true for buried systems where forced deflections may occur due to subsidence, washout and settlement and to surface traffic.

The earth load being applied to a buried system can be determined by the use of the modified Marston formula.

$$W = C_d \times w \times B_d \times D \qquad (10)$$

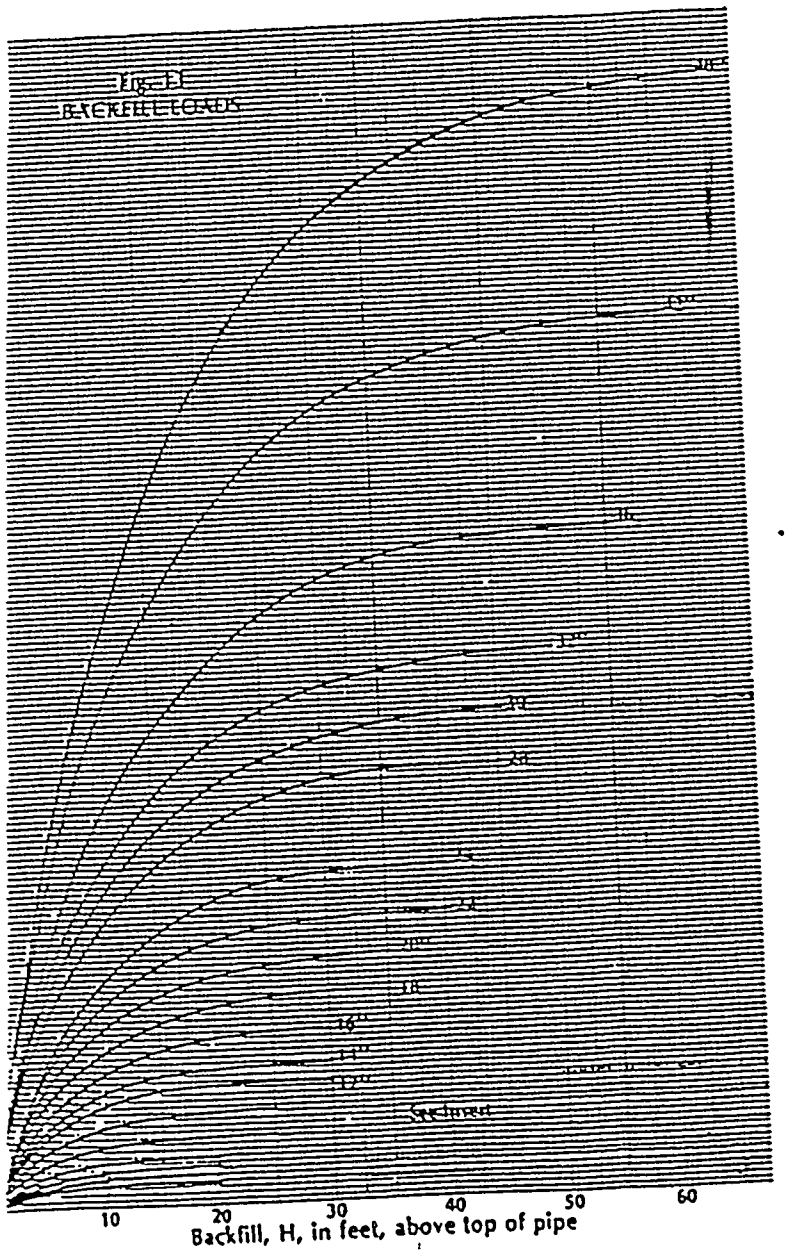
Where W – earth load in pounds per linear feet of pipe, lb./ft.
C_d – coefficient dependent on soil type and height of backfill to width of trench ratio. (see Fig. 16 for C_d)
w – weight of soil, lbs./cu.ft.
D – outside diameter of pipe, ft.
B_d – trench width at top of pipe, ft.

In general practice the trench width can be kept to a minimum of 6 inches per side greater than the pipe diameter itself. Although this may seem narrow in comparison to trenching of conventional materials it must be noted that POLY PIPE can be preassembled above ground and later placed into the trench. The trench width should be maintained as narrow as possible as the soil loading on the pipe is a relationship of the trench width

The loads applied to the pipe as a result of the backfill are given in Fig. 13.

In situations where the pipe is buried below the water table the effect is to relieve the load on the pipe to some extent due to buoyancy of the pipe. Although this can be a considerable effect, it should not be taken into account when designing a system as the water table can change to a position below the pipe. In the event the water level does not change, the reduction in stress will serve as an additional safety factor.

In calculating the soil load placed on a buried pipe, the designer must be able to calculate to some degree of accuracy the type and condition of the backfill material. Saturated clay would be more difficult to place and adequately compact than would coarse granular material that would not stick together. It is important in the pipe/soil system that the backfill material utilized for haunching and initial backfill (see Fig. 18 for explanation of terminology) be granular and non-cohesive, free of debris, organic matter, frozen earth and rocks larger than 1½ inch in diameter. This material can best be described as Class I or II of ASTM D2321 "angular ¼ to 1½ inch graded stone, slag, cinders, crushed shells and stone or sands and gravels containing small percentages of fines, generally granular and non-cohesive, wet or dry." This material can easily be worked into the pipe haunch, compacted in approximately 4 to 6 inch lifts to above the pipe.

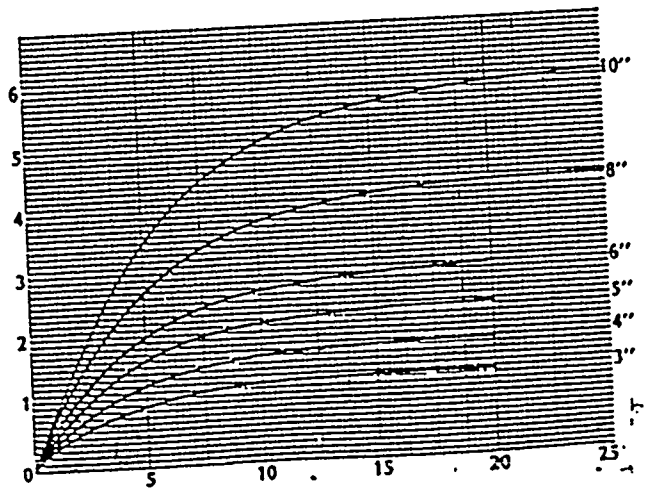


These curves were determined by using a 120 pound per cubic foot soil weight, trench widths of one foot greater than the pipe diameter, and a Class II backfill material as noted in Fig. 16.

Class II backfill materials can, on occasion be taken directly from the trench and replaced in an acceptable manner. Class I materials, in virtually all cases, would require off-site acquisition. Class III and IV materials, while time-consuming and difficult to work into the haunch and embedment zone at an acceptable compaction level, are generally available from the existing spoil.

Selection of these conditions was made as Class II backfill is considered optimum for backfilling in general. Class II soil at 90% density will effectively support the pipe/soil unit through the range of thinner wall pipes – SDR 17 to SDR 32.5. The designer should consider heavier wall pipe – SDR 13.5 and SDR 11 – when Class III and IV soils are utilized in the embedment zone.

Fig. 14 presents the designer with the estimated loading to obtain a maximum of 5% deflection in a round, flexible pipe. Should a more conservative deflection be desired the values in this table can be multiplied by the appropriate correction factor. It may, on occasion, indicate that the selection of a heavier wall pipe for a particular installation will present an economic advantage over a thinner wall pipe and improved compaction.



Hand or mechanical tam
the soil pressure on the pipe
the soil load. If the sidefill is
support a greater portion of
adequate stiffness in the side
expansion in the horizontal.
upon the selection, placeme

Under normal circumsta
deflection can be calculated

Where: ΔX = deflection
DI = lag factor
K = bedding
W = load on
R = mean rad
E = modulus
I = moment
where T
E' = modulus

This equation is known as

Modified for flexible P

Where: Δx = vertical
DI = deflecti
K = bedding
W = earth lo
E = modulu
E' = soil mo
SDR = OD/t

Note: Values of DI = de
Manual of Practice No.
to 65% of maximum de
density.

From the above form
can then be calculated t

Where: Δx = deflec
D = nomi

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Hand or mechanical tamping of the side-fill to an 85% Standard Proctor Density will significantly reduce the soil pressure on the pipe. If the side-fill is more yielding than the pipe, the pipe will carry a larger part of the soil load. If the side-fill is stiffer than the pipe, a soil-arch or "bridge" will form over the pipe that will support a greater portion of the total load. This compacted side-fill also adds stiffness to the pipe. With adequate stiffness in the side-fill, the pipe is restrained from deflecting due to soil loads in the vertical and expansion in the horizontal. Therefore, it may be said the performance of a buried flexible pipe is dependent upon the selection, placement and compaction of the earthen envelope.

Under normal circumstance it is essential to limit the deflection of the pipe to a designed quantity. The deflection can be calculated from the following equation.

$$\Delta X = \frac{DI \times K \times W}{\frac{EI}{R^3} + 0.061\ E'}$$

Where: ΔX – deflection in inches
DI – lag factor, assumed between 1.25 and 1.50. (1.50)
K – bedding class factor, conservatively K – 0.1
W – load on pipe in pounds per linear inch
R – mean radius in inches
E – modulus of elasticity of pipe (30,000 psi conservatively)
I – moment of inertia – $T^3/12$
where T – wall thickness, in.
E' – modulus of passive soil resistance

This equation is known as the Spangler equation.

Modified for flexible POLY PIPE this formula becomes:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3\ (SDR)^3} + 0.061\ E'} \qquad (11)$$

Where: Δx – vertical deflection of pipe, inches
DI – deflection lag factor
K – bedding constant
W – earth load on pipe in pounds per linear inch
E – modulus of elasticity of polyethylene, psi
E' – soil modulus, psi
SDR – OD/t

Note: Values of DI – deflection lag factor 1.50 and K – bedding constant of 0.10 are taken from WPCF Manual of Practice No. 9 as is the selection of the soil modulus of 300 psi when the side fill is hand tamped to 65% of maximum density and 700 psi when side fill is mechanically tamped to 90% or more maximum density.

From the above formula the deflection, Δx, of a pipe can be calculated. From this the percent deflection can then be calculated by

$$\% \text{ def} = \frac{\Delta x}{D} \times 100 \qquad (12)$$

Where: Δx – deflection in inches
D – nominal pipe outside diameter, inches

Fig. 14

Maximum recommended external loading with 5% deflection, pounds per linear foot of pipe.

Nominal Pipe Size, in.	Soil Modulus, psi	SDR 32.5	26	21	17	13.5
3	300			285	315	370
	700			630	655	710
4	300		350	370	405	475
	700		790	810	840	915
5	300	420	435	455	500	590
	700	965	975	1000	1040	1130
6	300	500	515	540	595	700
	700	1145	1160	1190	1240	1345
8	300	650	670	705	770	910
	700	1495	1510	1550	1615	1755
10	300	810	835	880	960	1135
	700	1860	1885	1930	2010	2185
12	300	965	990	1045	1140	1350
	700	2205	2235	2290	2385	2590
14	300	1055	1090	1145	1255	1480
	700	2425	2455	2510	2620	2845
16	300	1210	1245	1310	1430	1690
	700	2770	2805	2870	2995	3255
18	300	1360	1400	1475	1610	1905
	700	3115	3155	3230	3370	3660
20	300	1510	1555	1635	1790	2115
	700	3465	3505	3590	3740	4065
22	300	1660	1710	1800	1970	2325
	700	3810	3855	3950	4115	4475
24	300	1815	1865	1965	2150	2535
	700	4155	4210	4305	4490	4880
28	300	2115	2175	2290	2505	2960
	700	4845	4910	5025	5240	5695
30	300	2265	2335	2455	2685	3170
	700	5195	5260	5385	5615	6100
32	300	2415	2490	2620	2865	3385
	700	5540	5610	5740	2990	6505
36	300	2720	2780	2945	3220	
	700	6235	6310	6460	6735	
42	300	3170	3265	3435		
	700	7270	7365	7535		
48	300	3625	3730			
	700	8310	8415			

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Listed below are the recommended maximum allowable design limits for deflection of polyethylene pipe for the different available standard dimension ratios.

Dimension Ratio	Safe Deflection (% of diameter)
32.5	8.5
26	7.0
21	6.0
17	5.0
13.5	4.0

Fig. 14 depicts the load imposed on a pipe when recommended installation practices are followed. In this Fig. a value of 120 #/ft³ soil and for Class II soils a ratio of lateral to vertical pressures of 0.17. For other classes of soil as noted in Fig. 16, Class I – 0.19, Class III – 0.15, Class IV – 0.13, and Class V – 0.11. As can be seen from Fig. 13, there is a depth at which increased loading on the pipe ceases and the load level remains constant, regardless of the height of backfill.

POLY PIPE through its inherent physical properties of flexibility, resilience, and toughness can withstand significant deflection without failure. It can be flattened without causing a fracture of the pipe wall. However, this condition is unacceptable as far as service is concerned. A deflection of 15% would be acceptable for a butt fused polyethylene system although a reduction in flow would be noted. It would also be difficult to utilize conventional cleaning equipment with this severe a deflection. For these reasons, POLY PIPE recommends maximum deflections of 5 to 7½%. Calculations in this guide have been based on 5%.

Compaction or quality of backfill above the pipe zone, while important for many reasons, does not effect the predicted deflection of the pipe. The designer may wish to review Sec. H, Underground Installations, for further considerations.

Some design considerations to be recognized in the installation of buried pipelines is the exercise of caution in the tamping of the side-fill so as to not force granular, free flowing, non-cohesive Class I & II bedding materials under the haunch in sufficient force to "float" the pipe. It is imperative that buried pipe be on a firm, continuous support. Final backfill material, while unimportant to the pipe zone, should be free of large rocks, boulders or other similar materials that could project down into the pipe.

The designer may wish to review Sec. C, Critical Buckling, of this design guide in conjunction with this section.

For a full discussion of the formula and the derivative work involved the reader is referred to Bulletin 153 of the Iowa State College Bulletin "The Structural Design of Flexible Pipe Culverts" by M.G. Spangler.

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Fig. 15

Classification of Backfill Material

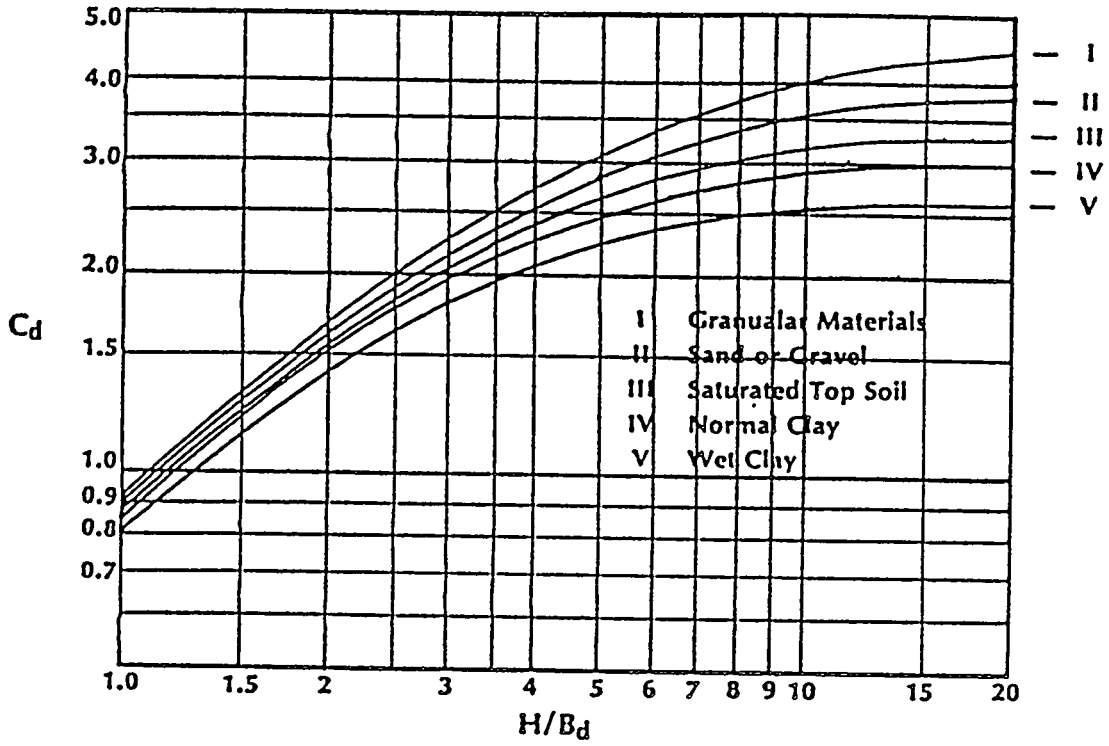
per ASTM D2321*

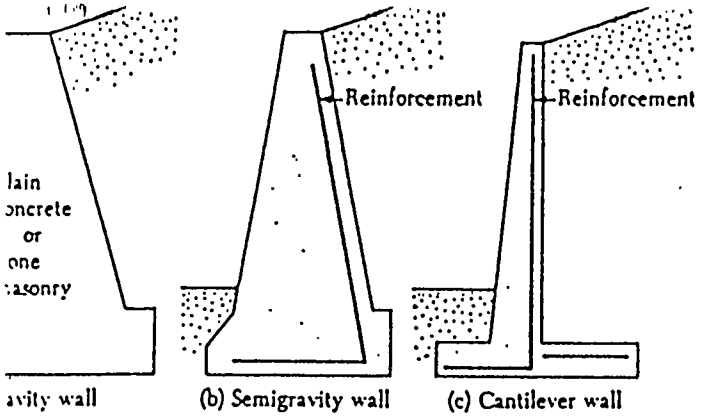
Class	Comments
Class I — Angular graded stone, ¼ to 1½ in., including a number of fill materials that have regional significance such as coral, slag, cinders, crushed stone, crushed gravel, and crushed shells.	100 - 120 pounds per cubic foot. Pipe sizes less than 10-inch should limit maximum particle size to ½ to ¾-inch for ease of placement.
Class II — Coarse sands and gravels with maximum particle size of 1½ in., including variously graded sands and gravels containing small percentages of fines, generally granular and non-cohesive, either wet or dry.	110 - 130 pounds per cubic foot. Pipe sizes less than 10-inch should limit maximum particle size to ½ to ¾-inch for ease of placement.
Class III — Fine sand and clayey gravels, including fine sands, sand-clay mixtures, and gravel-clay mixtures.	140 - 150 pounds per cubic foot.
Class IV — Silt, silty clays, and clays, including inorganic clays and silts of medium to high plasticity and liquid limits.	150 - 180 pounds per cubic foot.
Class V — Includes organic soils as well as soils containing frozen earth, debris, rocks larger than 1½ in. in diameter and other foreign materials.	Not recommended for backfill except in the final backfill zone.

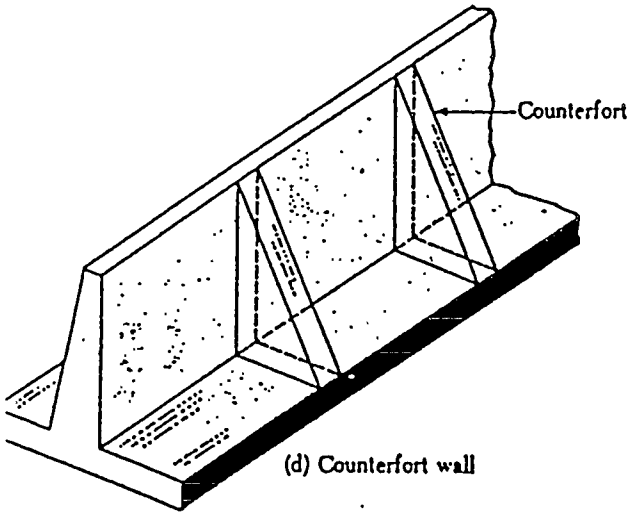
*For further classification of soils the designer may want to review ASTM D2487, Standard Test Method for Classification of Soils for Engineering Purposes.

Fig. 16

Coefficient, C_d, dependent on soil type and ditch configuration.







s of retaining wall

erly design retaining walls, one must know the basic soil that is, the *unit weight*, *angle of friction*, and *cohesion*—for the behind the wall and the soil below the base slab. Knowing the the soil behind the wall enables an engineer to determine the or which the design has to be made.

e two phases in the design of a retaining wall. First, with the lateral e known, the structure as a whole is checked for *stability*. This king for possible *overturning*, *sliding*, and *bearing capacity* fail-, each component of the structure is checked for *adequate* the *steel reinforcement* of each component is determined.

pter presents the procedures for determination of lateral earth retaining-wall stability. Checks for adequate strength of each the structures are covered in Appendix A.

Lateral Earth Pressure At-Rest

Consider a vertical wall of height H, as shown in Figure 5.2, retaining a soil having a unit weight of γ. A uniformly distributed load, q/unit area, is also applied at the ground surface. The shear strength, s, of the soil may be given by the equation

$$s = c + \sigma' \tan \phi$$

where c = cohesion
ϕ = angle of friction
σ' = effective normal stress

At any depth z below the ground surface, the vertical stress below the ground surface can be given as

$$\sigma_o = q + \gamma z \tag{5.1}$$

If the *wall is not allowed to move at all* either way from the soil mass or into the soil mass, the lateral pressure at a depth z can be given as

$$\sigma_h = K_o \sigma_o' + u \tag{5.2}$$

where u = pore water pressure
K_o = coefficient of at-rest earth pressure

For granular soil, the relation for K_o is

$$K_o = 1 - \sin \phi \tag{5.3}$$

For normally consolidated clays, the coefficient of earth pressure at-rest can be approximated as (Brooker and Ireland, 1965)

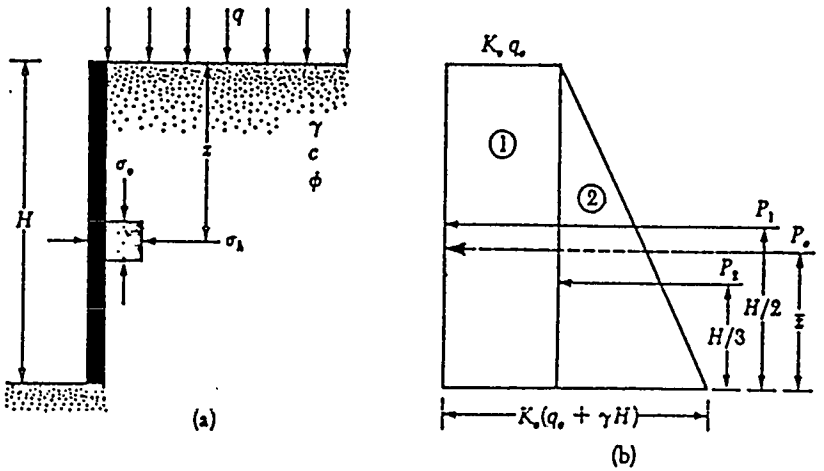


Figure 5.2 At-rest earth pressure

WHC-SD-W025-FDR-001, Rev. 1

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Calculation Cover Sheet

Sheet 1 of 1

Project Title: Non-Drag-off Landfill Designs

Project No. 893-1186.032

Calculation Title: Geotextile Selection

	Name	Date
Prepared By:	Steve Wright	1-16-90
Checked By:	Timothy L. Martin	1-18-90
Approved By:	[illegible]	1-23-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

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SUBJECT GEOTEXTILE SELECTION		
Job No. 893-1186.032	Made by SRW	Date 1-16-90
Ref. NON-DRAG-OFF	Checked TJM	Sheet 2 of 13
	Reviewed JCL	

OBJECTIVE: SELECT GEOTEXTILE(S) TO SATISFY THE FOLLOWING OBJECTIVES:

1) SEPARATION / FILTER BETWEEN OPERATION LAYER & DRAINAGE LAYER

2) CUSHION / SEPARATION BETWEEN 1° & 2° DRAINAGE MATERIALS & 1° & 2° GEOMEMBRANES

APPROACH: FOLLOW FHWA GEOTEXTILE ENGINEERING MANUAL GUIDELINES & RECOMMENDATIONS FOR OBJECTIVE # 1 & FOLLOW KOERNER'S "DESIGNING WITH GEOSYNTHETICS" FOR OBJECTIVE # 2.

REFERENCES: "FHWA GEOTEXTILE ENGINEERING MANUAL", CHRISTOPHER & HOLTZ, 1985.

"DESIGNING WITH GEOSYNTHETICS", KOERNER, 1986.

ASSUMPTIONS:

- ASSUME STEADY STATE FLOW
- γ_{WASTE} = 120 PCF
- ASSUME MAX TIRE PRESSURE = 85 PSI (= CAT 637 SCRAPER)
- ASSUME NO SLIPPAGE BETWEEN GEOTEXTILE & DRAINAGE STONE

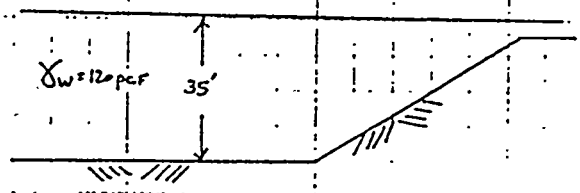
SOIL PROPERTIES

OPERATIONS LAYER	DRAINAGE LAYER*	BASE COURSE / RAMP**
D_{85} = 0.49 mm	d_{50} ≅ 1.2mm	d_{50} ≅ 6mm
D_{60} = 0.18 mm	SPHERICITY = 0.7	SPHERICITY = 0.4
D_{10} = 0.061 mm	d_{10} ≅ 0.25mm	
C_u = 2.95		

* GRADED TO WITHIN THE LIMITS SPECIFIED BY WSDOT 9.03.12(4) "1988 STANDARD SPECIFICATIONS"

** TO CONFORM TO WSDOT 9.03.9 (3) "1988 STANDARD SPECIFICATIONS"

GEOMETRY



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SUBJECT GEOTEXTILE SELECTION

Job No. 893-1186.032 | Rel. Now-Alab-DFF | Made by SAW | Checked TLM | Reviewed [illegible] | Date 1-16-90 | Sheet 3 of 13

CALCULATIONS

I. OBJECTIVE # 1 (SEPARATION/FILTER)

A. FILTERING

OPERATIONS LAYER CONTAINS < 50% PASSING U.S. # 200 SIEVE.

ACCORDING TO FHWA MANUAL, IF < 50% PASSING U.S. # 200 SIEVE

$$AOS = BD_{85} \quad \text{(ASSUMES STEADY STATE FLOW)}$$

WHERE,

$B = 1$ FOR $C_u < 2$

$B = 0.5C_u$ FOR $2 < C_u < 4$

$B = 8/C_u$ FOR $4 < C_u < 8$

FOR PROPOSED OPERATIONS LAYER MATERIAL, $C_u = 2.95$

$\therefore 2 < C_u < 4 \Rightarrow B = 0.5C_u = 1.48$

$AOS = BD_{85}$

$AOS = 1.48\ (0.49\ mm)$

$AOS = 0.72\ mm = 30$ sieve ← FOR SEPARATION/FILTER

TLM Retention

TLM FHWA manual.

Clogging: Porosity ≥ 30%

$O_{95} \geq 3D_{15}$ ... $D_{15} = 0.068$ ... $O_{95} \geq (3)(0.068) = 0.20 \Rightarrow 80$ s.t.c

B. PERMEABILITY

Require AOS between 30 and 80

ACCORDING TO FHWA MANUAL, FOR CRITICAL/SEVERE APPLICATIONS,

$K_{GEOTEXTILE} \geq 10\ (K_{SOIL})$

OPERATIONS LAYER IS ASSUMED TO HAVE $K_{ops} \cong 10^{-4}$ cm/s

$\therefore K_{GEOTEXTILE} \geq 10\ (10^{-4}\ cm/s)$

$K_{GEOTEXTILE} \geq 10^{-3}$ cm/s ← FOR SEPARATION/FI

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SUBJECT GEOTEXTILE SELECTION

Job No. 893-1106.032 Ref. NON-DWG-OFF | Made by SCW Checked TKM Reviewed RL | Date 1-16-90 Sheet 4 of 13

C. CLOGGING (BETWEEN 1° GRAVEL & OPERATIONS LAYER)

GRADIENT RATIO (GR) TEST NEEDS TO BE RUN TO EVALUATE THE COMPATIBILITY BETWEEN THE OPERATIONS LAYER MATERIAL & THE SPECIFIED GEOTEXTILE.

THE FHWA MANUAL RECOMMENDS THAT

$GR \leq 3$

D. BURST RESISTANCE (BETWEEN 1° GRAVEL & OPERATIONS LAYER)

ASSUME: LARGEST LOADING WILL BE FROM CAT 637D SCRAPER

FROM "CATERPILLAR PERFORMANCE HANDBOOK, EDITION H", 1983 (PG 460).

RADIAL TIRES

	MICHELIN		GOODYEAR		FIRESTONE		BRIDGESTONE	
	FRONT	REAR	FRONT	REAR	FRONT	REAR	FRONT	REAR
PRESSURE (PSI)	80	80	85*	80	85*	75	80	80

* WORST CASES = P_a

FROM KOERNER, PRESSURE FORCING GEOTEXTILE INTO VOID $\cong 0.75\ P_a$

$= 0.75\ (85\ psi)$

$= 63.75\ psi$

FROM FIG. 2.14 ("DESIGNING WITH GEOSYNTHETICS, KOERNER, 1986 PG 68):

BURST PRESSURE OF 17 PSI CORRESPONDS TO TIRE INFLATION OF 64 PSI & MAX STONE SIZE OF 1-INCH (SEE ATTACHED FIG)

KOERNER USES A FS = 3 IN HIS EXAMPLE,

∴ REQUIRED BURST RESISTANCE = 3(17) = 51 PSI ← Fo Separa

Pg 5 of 13

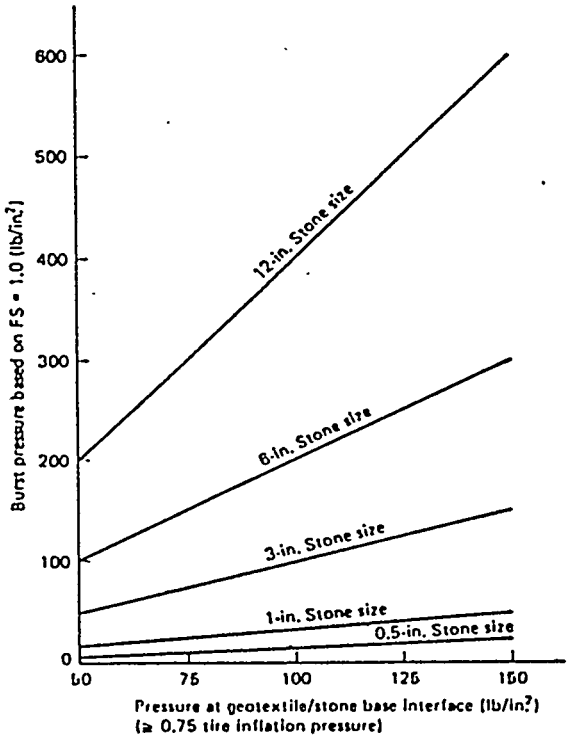


Figure 2.14 Design guide for burst analysis of geotextile used in a separation function.

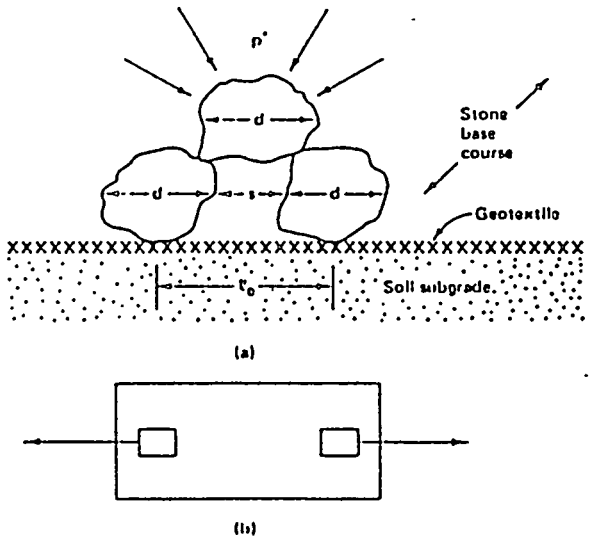


Figure 2.15 Geotextile being subjected to tensile (grab) stress as surface pressure is applied and stone base tends to spread laterally. (a) Actual situation; (b) analogous grab tension test.

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SUBJECT	GEOTEXTILE SELECTION	
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E. TENSILE STRENGTH REQ'D (SEPARATION)

GIVEN: AN ASSUMED MAX. TIRE PRESSURE OF 85 PSI (SEE REF. ABOVE) ON A BASE COURSE OF 1-INCH MINUS MATERIAL W/GEOTEXTILE BENEATH IT.

FIND: MAXIMUM GRAB TENSILE STRESS ON GEOTEXTILE ASSUMING NO SLIPPAGE BETWEEN FABRIC & STONE*.

* THIS IS A VERY CONSERVATIVE ASSUMPTION; THEREFORE, KOERNER RECOMMENDS USING A FS = 2

SOLUTION: ACCORDING TO KOERNER (Pg 69), STRAIN IN FABRIC (ε) IS 75%. ASSUMING NO STONE-TO-FABRIC SLIPPAGE.

KOERNER SHOWS (Pg 69) THAT

MOBILIZED TENSILE FORCE = PRESSURE × ε^2
= 0.75(85 PSI)(0.75)^2
= 36 LB

FOR FS = 2, REQUIRED GRAB STRENGTH = 72 LB *

* NOTE: GRAB STRENGTH TEST DIMENSIONS ARE SET-UP TO REFLECT 3-IN DIAMETER STONE SIZE. BECAUSE WE HAVE MAXIMUM PARTICLE SIZE OF 1-IN, GRAB STRENGTH NEEDS TO BE PROPORTIONATELY LARGER OR

REQUIRED GRAB STRENGTH = 3(72 LB) = 216 LB

F. PUNCTURE/TEAR RESISTANCE (SEPARATION)

GIVEN: AN ASSUMED MAX. TIRE PRESSURE OF 85 PSI (SEE REF. ABOVE) ON THE OPERATIONS LAYER WHICH OVERLAYS A GEOTEXTILE WHICH COVERS A ½-INCH MINUS DRAINAGE LAYER

FIND: REQUIRED PUNCTURE RESISTANCE OF THE GEOTEXTILE ASSUME AOS = 0.72mm (PER AOS CALCULATION)

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SUBJECT GEOTEXTILE SELECTION

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Reviewed

STRESS ON FABRIC = 0.75 P_a
= 0.75 (85 PSI)
= 64 PSI

AOS = 0.72 mm = 0.0284 in

$T_{REQ'D} = (\pi d_i d_a) P' S'$

WHERE
d_i = AOS = 0.0284 in
d_a = 1/2 INCH
P' = 64 PSI
S' = (1-S) = (1-0.7) = 0.3

$T_{REQ'D} = \pi (0.0284\ in)(0.5\ in)(64\ PSI)(0.3)$
= 0.86 LB

KOERNER USES A FS = 2 IN HIS EXAMPLE

∴ PUNCTURE/TEAR RESISTANCE = 2(0.86 LB) = 1.7 LB

6. SUMMARY OF REQUIRED PROPERTIES FOR SEPARATION (85 PSI LAYER/1" GRAVEL)

PROPERTY	REQUIRED (BASED ON CALC'S)	FINAL RECOM. MINIMUMS*
AOS	0.72 mm – 0.20 mm (TRM)	—
	30 SIEVE – 80 SIEVE	
$K_{TEXTILE}$	$\geq 10^{-3}$ cm/s	—
STRENGTH/BURST	51 PSI	320 PSI
TRAPEZOIDAL TEAR	—	50 LB
GRAB STRENGTH	216 LB	200 LB
ELONG. @ MAX σ_T	—	15 %
PUNCTURE	1.7 LB	80 LB

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SUBJECT	GEOTEXTILE SELECTION		
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TLM

LIST OF PRODUCTS MEETING ABOVE REQUIREMENTS:

MANUFACTURER	NAME	TYPE	AOS (Sieve)	mass/area oz/yd²
Delaware Valley Corp.	Tenamat 208	Non-woven	60	8
Hoechst Celanese Corp.	Trevira 1125	Non-woven	50	7.1
Nicolon Corp.	Geolon N70	Non-woven	70	7.1
	Geolon N80	Non-woven	70	7.7
Wellman Quline Inc.	Quline 100	Non-woven	70	10

II. OBJECTIVE #2 (CUSHION/SEPARATION)

A. ACCESS RAMP CUSHIONING

GIVEN: A BASE COURSE CONFORMING TO WSDOT 9.03.9 (3) WHICH IS PLACED ON TOP OF A GEOTEXTILE WHICH IS PLACED ON TOP OF THE GEOSYNTHETICS OF THE 1° DRAINAGE LAYER

FIND: REQUIRED STRENGTH OF GEOTEXTILE TO PROVIDE ADEQUATE CUSHIONING OF THE UNDERLYING HDPE

SOLUTION:

$$T_{REQ'D} = \pi d_i d_a P'$$

WHERE:

d_i = #100 SIEVE = 0.15mm = 0.006 in (ASSUME TREVIRA 1155)

$d_a \cong d_{50} \cong$ 6mm = 0.24 in

$P' = 0.75(85\ PSI) = 64\ PSI$

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SUBJECT: GEOTEXTILE SELECTION

Job No. 913-1186.032 | Rel. NON-DMG-OFF | Made by SAW | Checked TLM | Reviewed | Date 1-16-90 | Sheet 9 of 13

$T_{REQ'D} = \pi(0.006\,in)(0.24\,in)(64\,PSI)(0.6)$

$= 0.1737\ LB$

PER MANUFACTURER, TREVIRA 1155 HAS A PUNCTURE STRENGTH = 200 LB

200 LB > 0.17 LB ∴ TREVIRA 1155 OK

B. ACCESS RAMP BURST STRENGTH

GIVEN: A BASE COURSE CONFORMING TO WSDOT 9.03.9(3) (5/8" MINUS) ON TOP OF A GEOTEXTILE. ASSUME MAX TIRE INFLATION = 85 PSI. PER KOERNER, USE FS = 3.0

FIND: REQUIRED BURST STRENGTH OF GEOTEXTILE

SOLUTION: FROM FIG 2.11 (KOERNER, "DESIGNING WITH GEOSYNTHETICS", 1986, PG 68), A BURST PRESSURE OF 8 PSI CORRESPONDS TO A TIRE INFLATION OF 64 PSI WITH A 5/8" MAX. STONE SIZE

FOR FS = 3,

REQUIRED BURST STRENGTH = 3(8 PSI) = 24 PSI

PER MANUFACTURER, TREVIRA 1155 HAS A BURST STRENGTH = 765 PSI

765 PSI > 24 PSI ∴ TREVIRA 1155 OK

C. CUSHIONING OF UNDERLYING HDPE FROM DRAINAGE MATERIAL

GIVEN: A 1/2-INCH MINUS DRAINAGE MATERIAL OVERLYING A GEOMEMBRANE WITH GEOTEXTILE BETWEEN THE TWO MATERIALS.

FIND: REQUIRED STRENGTH OF GEOTEXTILE TO PROVIDE ADEQUATE CUSHIONING OF THE UNDERLYING HDPE

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SUBJECT GEOTEXTILE SELECTION

Job No. 893-1186.032 | Made by SLW | Date 1-16-90
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Revlewed RJL

SOLUTION:

$T_{REQ'D} = \pi\, di\, da\, P'\, S'$

WHERE: $di = AOS \cong 0.7\ mm = 0.0276\ IN$

$da \cong d_{50} \cong 1.2\ mm = 0.05\ in$

$P' = 35\ FT\ (120\ LB/FT^3) = 4{,}200\ LB/FT^2 = 29.2\ PSI$ *

$S' = (1-S) = (1-0.7) = 0.3$

* SEE "GEOMETRY", PG 1

$T_{REQ'D} = \pi (0.0276\ IN)(0.05\ in)(29.2\ PSI)(0.3)$
$= 0.038\ LB$

PER MANUFACTURER, TREVIRA 1155 HAS A PUNCTURE STRENGTH = 200 LB

200 LB > 0.038 LB ∴ TREVIRA 1155 OK

D. PERMEABILITY BETWEEN DRAINAGE MATERIAL & GEOTEXTILE "LYSIION"

GEOTEXTILE UNDER 1" & 2" DRAINAGE MATERIAL WILL BE USED IN A LESS SEVERE/NON-CRITICAL APPLICATION

FOR THIS CONDITION, THE FHWA MANUAL RECOMMENDS:

$K_{GEOTEXTILE} > K_{SOIL}$

FOR CLEAN COHESIONLESS SOILS, K_{SOIL} CAN BE APPROXIMATED USING HAZEN'S FORMULA:

$K = C\,(D_{10})^2$

WHERE:
K = PERMEABILITY IN mm/SEC
D_{10} = EFFECTIVE SIZE IN mm
C = CONSTANT WHOSE VALUE RANGES BETWEEN 10 & 15.

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SUBJECT GEOTEXTILE SELECTION

Job No. 893-1186.032 | Made by SAW | Date 1-16-90
Ref. NON-DWG-OFF | Checked TJM | Sheet 11 of 13
Reviewed

FOR OUR DRAINAGE MATERIAL, $D_{10} \cong 0.25$ mm

BASED UPON HAZEN'S FORMULA, K OF DRAINAGE LAYER RANGES FROM

$K = 10(0.25)^2 = 0.625$ mm/sec $= 6.25 \times 10^{-2}$ cm/sec

TO

WORST CASE $\Rightarrow$ $K = 15(0.25)^2 = 0.94$ mm/sec $= 9.4 \times 10^{-2}$ cm/sec

$\therefore K_{GEOTEXTILE} > 9.4 \times 10^{-2}$ cm/sec

PER MANUFACTURER, TREVIRA 1155 HAS A PERMITTIVITY $= 0.5\ sec^{-1}$ AND A THICKNESS $= 5.1$ mm

$\Rightarrow K_{1155} = (0.5\ sec^{-1})(5.1\ mm) = 2.55 \frac{mm}{sec} = 0.255 \frac{cm}{sec}$

$0.255 \frac{cm}{sec} > 9.4 \times 10^{-2}$ cm/sec

$\therefore$ TREVIRA 1155 OK

E. CLOGGING BETWEEN DRAINAGE MATERIAL & GEOTEXTILE "CUSHION"

1° & 2° DRAINAGE MATERIAL WILL CONSIST OF A "CLEAN" SAND. CLOGGING OF THE GEOTEXTILE SHOULD NOT BE A PROBLEM.

CONCLUSIONS

- TREVIRA 1155 CAN BE USED FOR OBJECTIVE # 2: CUSHION/SEPARATION
- IT APPEARS THAT A WOVEN GEOTEXTILE NEEDS TO BE USED FOR OBJECTIVE # 1: SEPARATION/FILTER. SEE Pg 5 FOR A LIST OF WOVEN GEOTEXTILES THAT SATISFY THE DESIGN REQUIREMENTS

GEOTEXTILE CALCS
[redacted]
393-11860
[redacted]
Pg 12 of 13

3-29

TABLE 3-3

SUMMARY OF GEOTEXTILE DESIGN AND SELECTION CRITERIA FOR DRAINAGE, FILTRATION, AND EROSION CONTROL APPLICATIONS

I. SOIL RETENTION (PIPING RESISTANCE CRITERIA)[1]

Soils	Steady State Flow	Dynamic, Pulsating, and Cyclic Flow
≤50% Passing[2] U.S. No. 200 sieve	AOS -- $O_{95} \leq B\ D_{85}$	$O_{95} \leq D_{15}$ (if soil can move beneath fabric)
	$C_u \leq 2$ or ≥ 8 $B = 1$	OR
	$2 \leq C_u \leq 4$ $B = 0.5\ C_u$	$O_{50} \leq 0.5\ D_{85}$
	$4 \leq C_u \leq 8$ $B = \frac{8}{C_u}$	
≥50% Passing: U.S. No. 200 sieve	Woven: $O_{95} \leq D_{85}$	$O_{50} \leq 0.5\ D_{85}$
	Nonwoven: $O_{95} \leq 1.8\ D_{85}$	
	AOS No. (fabric) ≥ No. 50 sieve	

1. When the protected soil contains particles from 1 inch size to those passing the U.S. No. 200 sieve, use only the gradation of soil passing the U.S. No. 4 sieve in selecting the fabric.
2. Select fabric on the basis of largest opening value required (smallest AOS)

II. PERMEABILITY CRITERIA[1]

A. Critical/Severe Applications

k (fabric) ≥ 10 k (soil)

B. Less Critical/Less Severe and (with Clean Medium to Coarse Sands and Gravels)

k (fabric) ≥ k (soil)

1. Permeability should be based on the actual fabric open area available for flow. For example, if 50% of fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50%.

III. CLOGGING CRITERIA

A. Critical/Severe Applications[1]

Select fabrics meeting I, II, IIIB, and perform soil/fabric filtration tests before specification, prequalifying the fabric, or after selection before bid closing. Alternative: use approved list specification for filtration applications. Suggested performance test method: Gradient Ratio ≤ 3

B. Less Critical/Non-Severe Applications

1. Whenever possible, fabric with maximum opening size possible (lowest AOS No.) from retention criteria should be specified.
2. Effective Open Area Qualifiers[2]: Woven fabrics: Percent Open Area: ≥ 4%.
   Nonwoven fabrics: Porosity[3] ≥ 30%
3. Additional Qualifier (Optional): $O_{95} \geq 3D_{15}$
4. Additional Qualifier (Optional): $O_{15} \geq 3D_{15}$

Notes: 1. Filtration tests are performance tests and cannot be performed by the manufacturer as they depend on specific soil and design conditions. Tests to be performed by specifying agency or his representative. Note: experience required to obtain reproducible results in gradient ratio test.

2. Qualifiers in potential clogging condition situations (e.g. gap-graded soils and silty type soils) where filtration is of concern.

3. Porosity requirement based on graded granular filter porosity.

Ref: FHWA "TS-86-203 Geotextile Engineering Manual" (1985).

Project DOE RL/NON-DOE/AD OFF/SEATTLE
Project No. 883-1186.031 Date 11/04/88 Traced By RLH Approved By KMB
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GRAIN SIZE DISTRIBUTION

Percent Finer by Weight

0 10 20 30 40 50 60 70 80 90 100

US Standard Sieve Sizes
5" 3" 2" 1" 3/4" 3/8" 4 10 20 40 60 100 200

Grain Size in Millimeters
100 10 1.0 0.1 0.01 0.001

Cobbles | Gravel (Coarse, Fine) | Sand (Coarse, Medium, Fine) | Fines (Silt or Clay)

Boring No.	Elev. or Depth	W_n	W_L	W_p	I_p	Description
GTP 1-7	Composite	2.6				Olive gray, silty fine to medium SAND, (SM).

QA's Layer

Mk - Dayoff

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 6

Project Title: Non-Drag-OFF Land Fill Design

Project No. 893-1186.032

Calculation Title: Estimated Permeabilities of Drainage Gravels

	Name	Date
Prepared By:	Timothy L. Martin	January 24, 1990
Checked By:	Frank S. Shuri	2-5-90
Approved By:	PATRICK CROSS	JAN 29, 1990
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

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SUBJECT Estimated Permeabilities of Drainage Gravels

Job No. 993-1186.032 | Made by TLM | Date 1-24-90
Ref. | Checked RSS | Sheet 2 of 6
Reviewed PE

Objective: To estimate the coefficient of permeability in the proposed drainage gravels. To ensure $K \geq 1 \times 10^{-2}$ cm/sec.

Approach: To use Hazen's approximation for clean drainage material. Hazen's approximation should be used only when less than 5 percent passes the No. 200 sieve. Hazen's equation is valid for $K \geq 10^{-1}$ cm/sec and D_{10} sizes between 0.1 and 3.0 mm.

Assumption: Assume that by analyzing the lower grain-size boundary will set a lower limit of permeability for the drainage material.

Reference: An Introduction to Geotechnical Engineering, Holtz, R.D., and Kovacs, W.D.

Analysis:

Hazens Equation: $K = CD_{10}^2$ ✓ where K = permeability (cm/sec)
C = constant which varies between 0.1 to 1.2 with an average value of 1 and takes into account the conversion of units.
D_{10} = Grain diameter (in mm) corresponding to 10% passing by weight.

Primary and secondary Sump Gravel

From attached figure, $D_{10} = 6.7$ mm* ✓

$K = (1)(6.7^2)$ ✓

$K = 4.5 \times 10^{1}$ cm/sec $>> 10^{-2}$ cm/sec ok ✓

* Note: Since $D_{10} > 3$ mm Hazen's equation is not recommended for use in estimating permeability. Actual permeability can be estimated in a laboratory constant head permeability test. For the purpose of this analysis the permeability of the sump gravel is estimated to be much greater than the desired lower limit of permeability of 10^{-2} cm/sec. ✓

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SUBJECT: Estimated Permeabilities of Drainage Gravel

Job No. 893-1186.032 | Made by TLM | Date 1-24-90
Ref. | Checked FSS | Sheet 3 of 6
Reviewed RL

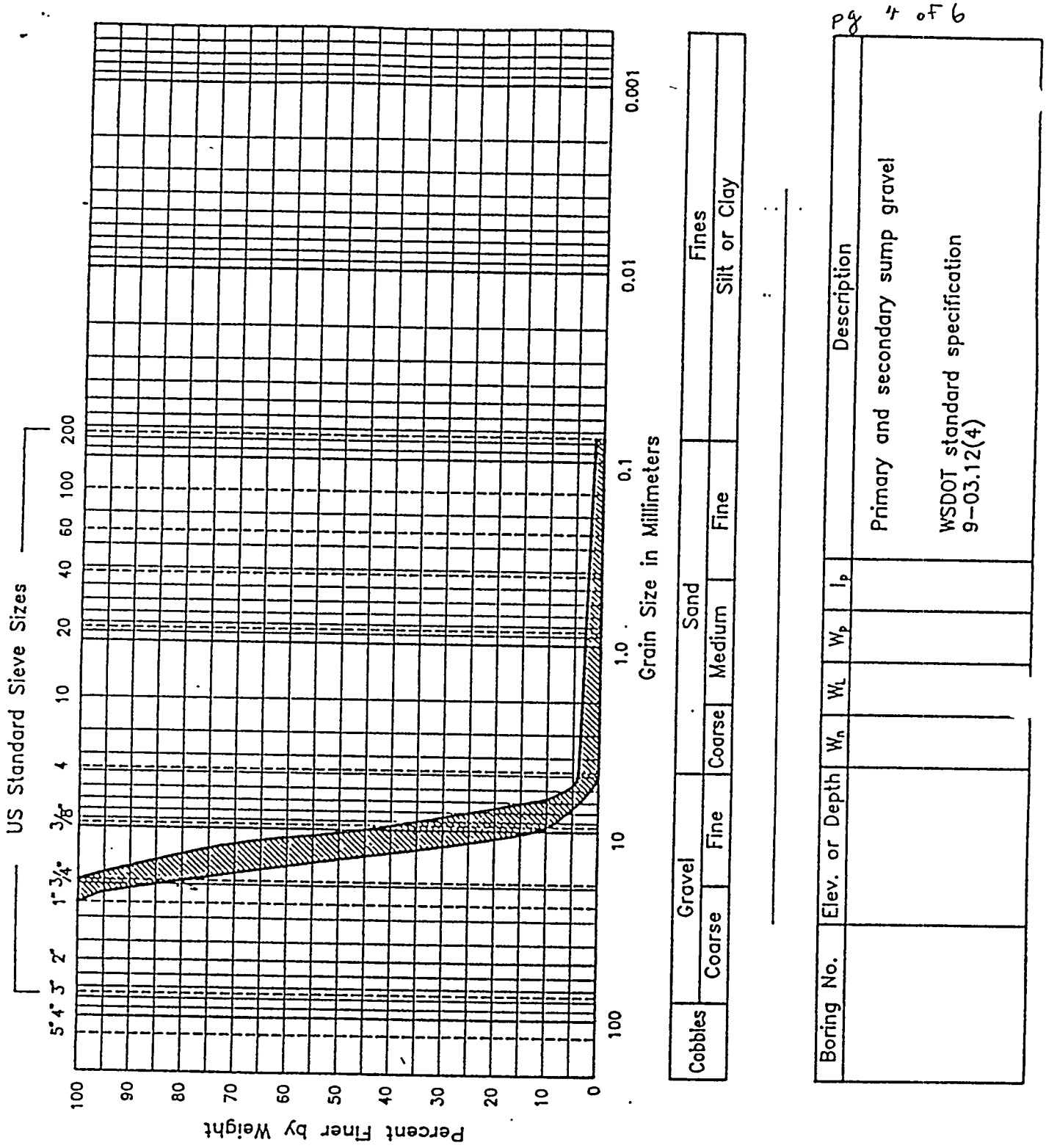
Primary and Secondary Drainage Gravel

From attached figure, $D_{10} \approx 0.425$ mm ✓

$k = (1)(0.425)^2$ ✓

$k = 1.8 \times 10^{-1}$ cm/sec $> 10^{-2}$ cm/sec ok ✓

Note: The Hazen's Equation provides an approximation of the actual permeability. All drainage gravels should have representative samples tested in the laboratory prior to use in the landfill. ✓



GRAIN SIZE DISTRIBUTION

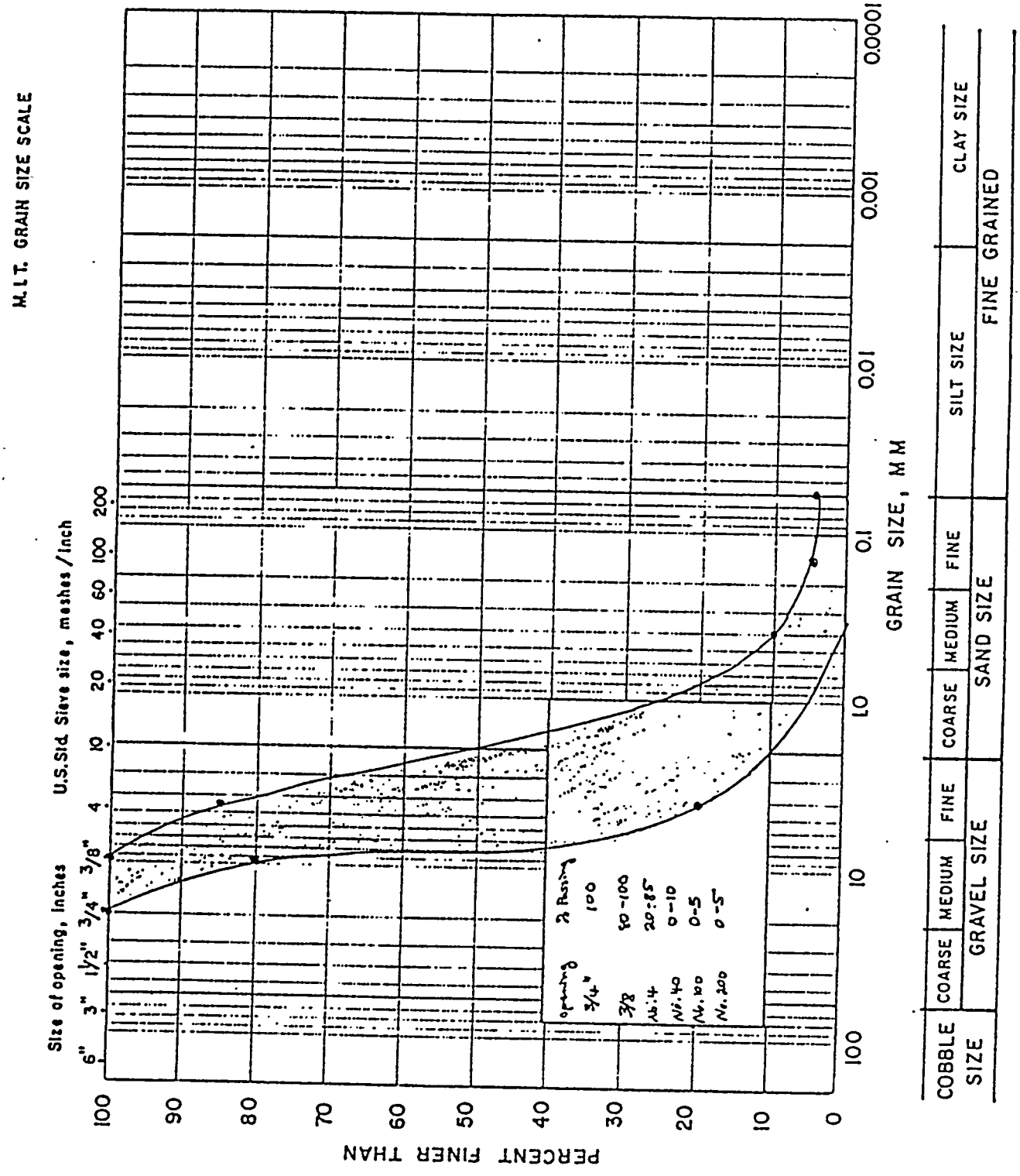
Project Non-Drag-Off
Project No. 893-1186.032 Date 1/23/90 Tested By ... Approved By ...

Golder Associates

893118B032\33257

GRAIN SIZE DISTRIBUTION

FIGURE



Project No. _____
Project _____ Golder Associates
Date _____
By _____

Pg 6 of 6

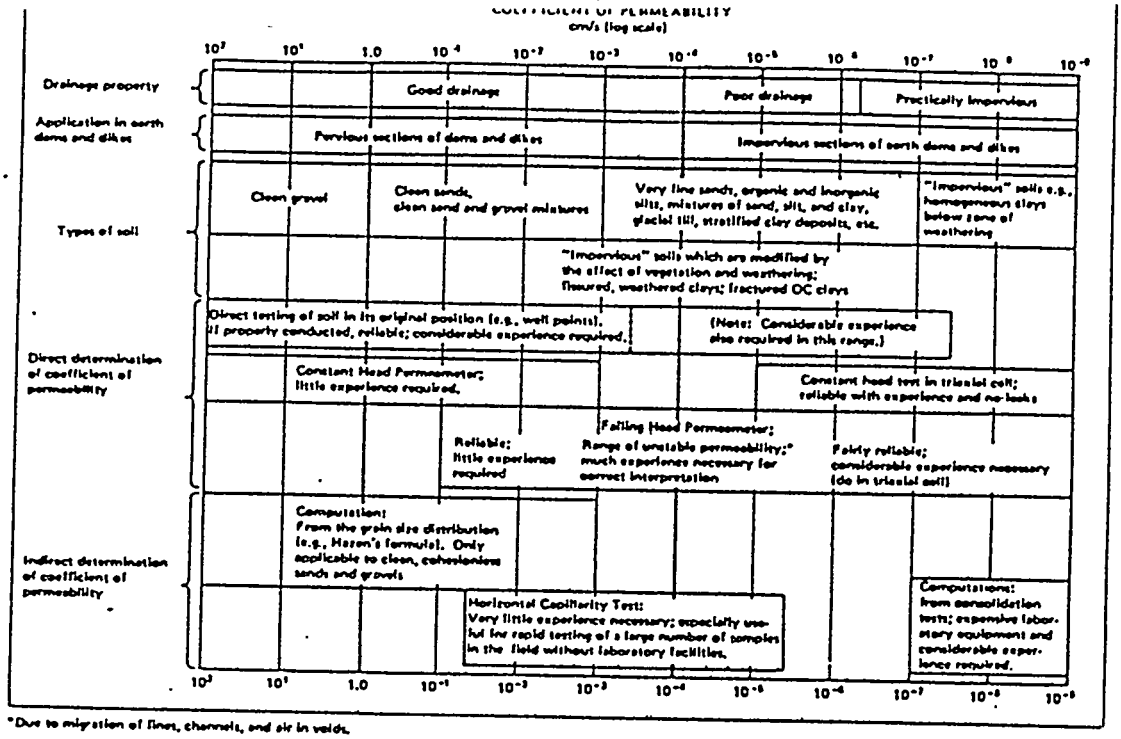


Fig. 7.6 Permeability, drainage, soil type, and methods to determine the coefficient of permeability (after A. Casagrande, 1938, with minor additions).

sands and sandy gravels. A k of 10^{-4} cm/s is the approximate bound between pervious and poorly drained soils under low gradients. Soils around this value are also highly susceptible to migration of fines or *piping*. The next boundary, 10^{-9} cm/s, is approximately the lower limit of the permeability of soils and concrete, although some recent measurements have found permeabilities as low as 10^{-11} for highly plastic clays at the shrinkage limit. Prof. Casagrande recommends that k be related to the nearest benchmark value, for example, 0.01×10^{-4} cm/s rather than 10^{-6} cm/s. For various soil types, Fig. 7.6 also indicates their general drainage properties, applications to earth dams and dikes, and the means for direct and indirect determination of the coefficient of permeability.

An empirical equation relating the coefficient of permeability to D_{10}, the *effective grain size*, was proposed by A. Hazen (1911). For *clean* sands (with less than 5% passing the No. 200 sieve) with D_{10} sizes between 0.1 and 3.0 mm, the coefficient of permeability k is

$$k = CD_{10}^2 \qquad (7\text{-}10)$$

where the units of k are in cm/s and those of the effective grain size are in mm. The constant C varies from 0.4 to 1.2, with an average value of 1, and takes into account the conversion of units. The equation is valid for $k > 10^{-3}$ cm/s (Fig. 7.6).

To estimate k at void ratios other than the test void ratio, Taylor (1948) offers the relationship

$$k_1 : k_2 = \frac{C_1 e_1^3}{1 + e_1} : \frac{C_2 e_2^3}{1 + e_2} \qquad (7\text{-}11)$$

where the coefficients C_1 and C_2, which depend on the soil structure, must be determined empirically. *Very approximately for sands*, $C_1 \approx C_2$. Another relationship which has been found to be useful for sands is

$$k_1 : k_2 = C_1' e_1^2 : C_2' e_2^2 \qquad (7\text{-}12)$$

As before, *approximately for sands*, $C_1' \approx C_2'$.

For silts and clays, neither of these relationships works very well. For kaolinites over a rather *narrow* range of permeabilities (say one order of magnitude), e versus $\log_{10} k$ has been found to be approximately linear, all other factors being equal. However for compacted silts and silty clays Garcia-Bengochea, et al. (1979) found that the relationship between void ratio e and the logarithm of permeability k is far from linear (Fig. 7.7). They showed that pore size distribution parameters provide a better relationship with the ratio for some compacted soils.

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Calculation Cover Sheet

Sheet 1 of 5

Project Title: WHC NON DRAG OFF

Project No. ~~893-1186~~ 893-1186

Calculation Title: CHECK PUNCTURE RESISTANCE OF HDPE

C.27

	Name	Date
Prepared By:	P. Gotz	18 APR 90
Checked By:	John Fillion	6/7/90
Approved By:	Timothy J. Martin	6/8/90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

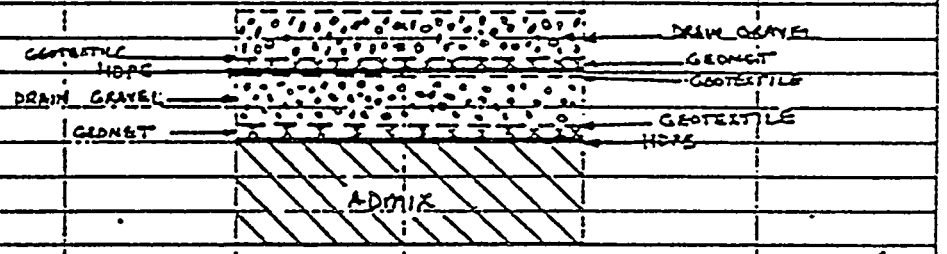
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SUBJECT NDO — CHECK PUNCTURE RESISTANCE OF HDPE		
Job No. 893-1186	Made by PCC	Date 19 APR 90
Ref.	Checked JPP	Sheet 2 of 5
	Reviewed	

OBJECTIVE: CHECK PUNCTURE RESISTANCE OF GEOMEMBRANE AT NON-DRAG-OFF FACILITY DUE TO EXPECTED LOADINGS AND ASSUMING GEOMEMBRANE IS IN CONTACT WITH DRAINAGE GRAVEL (ie. ASSUME GEOTEXTILE NOT PRESENT)

REFERENCE: GEOTEXTILES AND GEOMEMBRANES DEFINITIONS, PROPERTIES AND DESIGN BY JEAN-PIERRE GIROUD

GEOMETRY:



ASSUMPTION: Typical grader/scraper tire pressure ≈ 80 psi (conservative). JPM

CALCULATIONS

PUNCTURE RESISTANCE OF HDPE = 90 lbs

ALLOWABLE PUNCTURE RESISTANCE = 90#/3 = 30 lbs

DETERMINE MAXIMUM PARTICLE SIZE

$R_p = pd^2$

R_p = ALLOWABLE PUNCTURE RESIS (lb)
p = STATIC OR DYNAMIC PENETRATION RESIST
d = SIZE OF STONE (in.)

$p_{static} = (35\ ft)(110\ lb/ft^3) = 3850\ lb/ft^2 = 26.74\ psi$

$p_{dynamic\ or\ grader} = (3/4)(\text{TIRE PRESSURE}) + (.35)(80\ psi) = 60\ psi$

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SUBJECT NDO – CHECK PUNCTURE RESISTANCE OF HDPE

Job No. 893-1196 | Made by PGC | Date 19 APR 90

Ref. | Checked JPP | Sheet 3 of 5

Reviewed

$d = \sqrt{\frac{R_p}{p}}$

R_p = 30 lb = ALLOWABLE PUNCTURE RESIST

p = 26.74 lb/in² = OVERBURDEN STRESS

$d = \sqrt{\frac{30 \text{ lb}}{26.74 \text{ lb/in}^2}}$

$d = 1.05$ in

$d = \sqrt{\frac{30 \text{ lb}}{60 \text{ lb/in}^2}}$

$= 0.71$ in

MAXIMUM PARTICLE SIZE = 3/4 in. FOR DRAIN GRAVEL

SEE ATTACHED GRAIN SIZE SPECIFICATIONS FOR BOTH DRAINAGE GRAVEL AND SUMP GRAVEL. FOR DRAINAGE GRAVEL ALL MATERIAL LESS THAN 3/4 in. FOR SUMP GRAVEL AT WORST 20% GREATER THAN 3/4 in.

IN SUMP GRAVEL THICKNESS IS GENERALLY GREATER THAN 1 ft THEREFORE STRESS FROM TIRE PRESSURE SHOULD BE REDUCED. IN ADDITION, A GEOTEXTILE CUSHION IS BETWEEN GEOMEMBRANE AND GRAVEL AT THIS LOCATION

Note: No dynamic loading analysis was performed for the puncture resistance of the HDPE liner since it is not considered to add to the loading on the HDPE liner.

U.S. Sieve Size	% Passing
3/4 in.	100
3/8 in.	80-100
No. 4	20-85
No. 40	0-10
No. 100	0-5
No. 200	0-5

B. Material shall exhibit a permeability of $1x10^{-3}$ cm/sec or greater.

2-2 SUMP GRAVEL

A. Sump gravel shall be rounded washed gravel meeting the following gradation requirements.

1. Washington DOT standard specification 9-03.12(4).

U.S. Sieve Size	% Passing
1 in.	100
3/4 in.	80-100
3/8 in.	10-40
No. 4	0-4
No. 40	0-4
No. 200	0-2

B. Material shall exhibit a permeability of $1x10^{-3}$ cm/sec or greater.

PART 3: EXECUTION

3-1 PLACEMENT OF GRANULAR DRAINAGE GRAVEL AND SUMP GRAVEL

A. Place over secondary and primary geomembrane liners across base of landfill and in sumps as shown on Drawings.

Pat C
Gatti 1 MARCH

TA165 Y6
TA165
A51

Geotextiles and Geomembranes Definitions, Properties and Design

Selected Papers; Revisions and Comments

THIRD EDITION

Jean-Pierre Giroud

1985

TA
710
.G4
1984

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 4

Project Title: Non-Drag-off Landfill Design

Project No. 893-1186

Calculation Title: Vertical Riser Concrete Design
Seismic Design Analysis C.28

	Name	Date
Prepared By:	Michael L. Brown	6-11-90
Checked By:	Timothy L. Martin	6-11-90
Approved By:	Robert Gass	6-12-90
QA Approval:		

Revisions:	Date	By	Checked By	Approved By	QA Approval

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SUBJECT WHC /NDO VERTICAL RISER CONCRETE

Page 393 of 397

Job No. 893-1186 Ref.

Made by M. BROWN Checked TJM Reviewed

Date 6/8/90 Sheet 2 of 4

concrete foundation slab

OBJECTIVE: Check adequacy of design for seismic loadings.

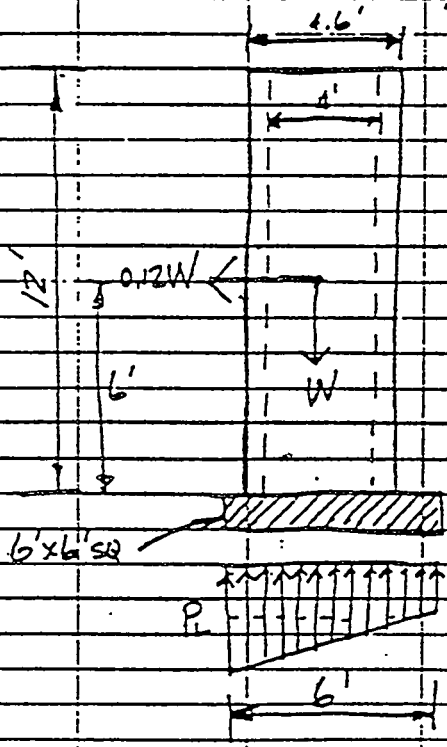
APPROACH: Check moment on base assuming linear foundation pressure distribution.

CRITERIA: Seismic load of 0.12g

Downdrag not present

Critical condition is after initial construction when surrounding waste does not provide lateral support.

From construction drawings, height = 12'



Weight of vertical riser concrete

$V_{ol} = \left[\frac{\pi 4.6^2}{4} - \frac{\pi 4^2}{4}\right] 12$

$V_{ol} = 48.6\ ft^3$ ✓

Unit Wt of concrete = 150 pcf ✓

$Wt = 48.6\ ft^3 \times 150\ pcf = 7295\ lbs.$ ✓

$= 7.3\ kips$ ✓

$\Sigma F = 0$

TOTAL UPWARD REACTION = 7.3 kips.

$= \left(\frac{P_b + P_a}{2}\right)(6' \times 6') = 18(P_b + P_a)$ *

$7.3\ kips = 18(P_b + P_a)$ Eqn (A)

* The hole for the HDPE well is ignored in this analysis. Our primary concern is with the moment on the base not the total force so ignoring the central hole has little effect on the result.

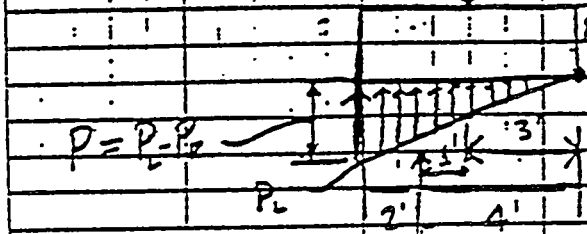
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SUBJECT WHC/NDO VERTICAL RISER CONCRETE Page

Job No. 893-1856 Ref. | Made by M. BROWN Checked TCM Reviewed [illegible] | Date 6/8/90 Sheet 3 of 4

$\Sigma M = 0$: Driving moment = $0.12W \times 6 = 0.12 \times 6 \times 7.3 = 5.26$ k-ft. $= M_D$

Restoring moment. $= \frac{1}{2} \times 4 \times P_R$ (Area of triangle)



$\Sigma M = 0 = F(4') - F(3') = F(1')$

Equivalent force $= \frac{(F \times b)}{2} b' = F$

$M_R = 1' F = (1')\left(\frac{P \times b}{2}\right)(b) = 18P = 18(P_L - P_R)$

$M_R = M_D$

$18(P_L - P_R) = 5.26$ k-ft. Eqn (B)

$18(P_L + P_R) = 7.3$ kips Eqn (A)

dividing by 18

$(P_L - P_R) = 0.29$

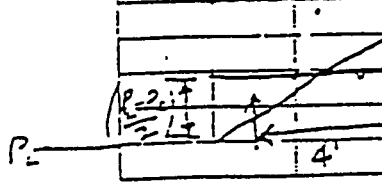
$P_L + P_R = 0.406$

Subtracting $-2P_R = -0.116 \Rightarrow P_R = 0.058$ ksf

$P_L = 0.29 + P_R = 0.29 + 0.058 = 0.348$ ksf

Checking: Upward reaction $= \left(\frac{0.348 + 0.058}{2}\right)(4')(6') = 7.3$ kips. ✓ OK

Moment $= 6\left(\frac{P_L - P_R}{2}\right)\frac{3}{3} \times 4 = 18(P_L - P_R) = 5.22$ k-ft. ✓ OK



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SUBJECT WHC/NDO

Job No. 893-1186 Ref.

Made by M. BROWN Checked TFM Reviewed

Date 6/8/90 Sheet 45 of 47

Max. foundation pressure on one edge is 0.348 ksf due to seismic overturning moments.

Original design which, included down-drag forces, used a foundation pressure of 6.85 ksf. for concrete slab design.

Since 0.348 ksf << 6.85 ksf the static condition with down drag forces controls the design. No further concrete design is necessary.

Based on the above calculation we conclude that seismic loadings on the vertical riser will not cause stresses in the concrete which exceed design criteria limits.