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Name: V. Roscha

Victor Roscha 11/13/94
Signature

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

Approval:

Maurice J. McNeill
Project Director

11-14-94
Date

Frank S. Shari
Project Manager

11-14-94
Date

John L. ...
QA Manager

11-14-94
Date

Michael L. Brown
Design Engineer

11/14/94
Date

John L. ...
Westinghouse Hanford Company

Nov. 15, 1994
Date

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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 (Contract 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/ft²
- Dense native sands and gravels: 3000 lb/ft²
- Proof-rolled silty sands (Eolian sand): 2500 lb/ft²

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 (Cummins 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 mi². 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/yd² 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/yd² 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.

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FIGURES

FIGURE 1

PERMEABILITY OF ADMX VERSUS PERCENT BENTONITE

DOE-RL/NON-DRAG-OFF/WA

Golder Associates

FIGURE 2
 MOISTURE-DENSITY-PERMEABILITY RELATIONSHIP OF ADMIX

DOE-RL/NON-DRAW-OFF/WA

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

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

WHC-SD-W025-FDR-001, Rev. 1

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	CN=65	0
HEC-1	CN=85	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

Cobbles	Gravel		Sand			Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay	

Boring No.	Elev. or Depth	W_n	W_L	W_p	I_p	Description
10% Ephrata Mix						Light olive-gray, fine to medium SAND, some silt (SM)

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

893118603533338

	Gravel		Sand		Fines
Cobbles	Coarse	Fine	Coarse	Medium	Fine
			Coarse	Medium	Silt or Clay

Boring No.	Elev. or Depth	W _n	W _L	W _p	I _p	Description
GTP1-7	8% Bentonite Composite					Light olive-gray, fine to medium SAND, some silty clay (SC)

FIGURE A-2
GRAIN SIZE DISTRIBUTION

Project DOE/Non-Drog-Off/RichlandProject No. 893-1186.035

Date 2/14/90 Tested By BFW

Approved By KRE

Gold Associates

NP3118803533337

Boring	Boring	Soil Classification	Moisture Before	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

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-DRAW-OFF/SEATTLE

Project No. 893-1186.033

Date 12/19/89

Tested by DN

Approved by KRB

Golder Associates

GOLDER ASSOCIATES
PERMEABILITY TEST SPREADSHEET

WHC-SD-W025-FDR-001, Rev. 1

Project Number: 893-1186.033
Project Description: DOX RL/NON-DRAQ-OFF/SRATYLE

Sample Number: GTP1-7 68 BENYONITE
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

TEST RESULTS

K = 3E-8 cm/second

Tested by: DMC
Reviewed by: KLB

Sample Properties : Pre Test Post Test
Net Weight (g): 583.24 611.02
Moisture (%): 19.1 24.8
Net 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

SPREADSHEET UPDATED 6/13/89, K. BROWN

Board number: L0014

Cell number: L0005

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

yr mo da hr minsec	Elapsed (min.)	CELL VOLUMES (cc)			INFLOW VOLUMES (cc)				OUTFLOW VOLUMES (cc)				K (cm/s)	Transducer Reading (psi)			Head (cm)
		Reading	Delta	Total	Reading	Refill	Delta	Total	Reading	Drain	Delta	Total		Delta	Cell	Bottom	
89 12 5 8 21 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
89 12 5 9 27 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
89 12 5 10 35 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	360.1
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	363.2
89 12 5 15 8 0 *	407.00	2.2	0.0	-0.4	18.9	0.0	1.0	18.5	5.8	0.0	1.7	18.4	2.1E-07	99.9	95.0	89.8	352.7
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	343.1
89 12 5 17 2 0 *	571.00	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	340.8
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	8.0E+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	4.0E-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 TEST RESULTS: K=3E-8 cm/second
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE *****

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

Sample Dimensions (cgs):	Initial	Final	Board number:	L0091
Height:	7.199	7.319	Cell number:	L0088
Diameter:	7.239	7.298		
Area:	41.16	41.83		
Volume:	296.3	306.2		

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:
Reviewed by:

DN

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom 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)			K (cm/s) Delta	Transducer Reading (psi)			Head (cm H2O)	
			Reading	Drain	Delta		Cell	Bottom	Top		
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 TEST RESULTS: $K=1E-8$ cm/second
Project Description: DOE RL/NON-DRAG-OFF/SEATTLE *****

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

Sample Dimensions (cgs):	Initial	Final	Board number:	L0091
Height:	7.016	7.065		
Diameter:	7.286	7.292	Cell number:	L0008
Area:	41.69	41.76		
Volume:	292.5	295.0		

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:
Reviewed by:

DN
KRIS

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom 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)				K (cm/s) Delta	Transducer Reading (psi)			Head (cm H ₂ O)	
							Reading	Drain	Delta	Total		Cell	Bottom	Top		
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 TEST RESULTS: $K=1E-8$ cm/second
Project Description: DOE RL/MON-DRAG-OFF/SEATTLE *****

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

Sample Dimensions (cgs):	Initial	Final	Board number:	L0091
Height:	7.202	7.213	Cell number:	L0007
Diameter:	7.294	7.273		
Area:	41.79	41.54		
Volume:	300.9	299.7		

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: DH
Reviewed by: *KIRB*

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom 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	min	sec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc)			Total	K (cm/s) Delta	Transducer Reading (psi)			Head (cm H ₂ O)
								Reading	Drain	Delta			Cell	Bottom	Top	
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 TEST RESULTS: K=1K-8 cm/second
Project Description: DOX RL/NON-DRAG-OFF/SEATTLE *****

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

Sample Dimensions (cgs):	Initial	Final	Board number:	L0091
Height:	7.172	7.172	Cell number:	L0089
Diameter:	7.279	7.279		
Area:	41.61	41.61		
Volume:	298.5	298.5		

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:
Reviewed by:

DR
KRS

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom 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				OUTFLOW VOLUMES (cc)			K (cm/s) Delta	Transducer Reading (psi)			Head (cm H ₂ O)
							Reading	Delta	Drain	Delta	Total	Cell	Bottom		Top			
89	12	2	12	17	0 *	0.00	17.9	22.9	0.0	0.0	0.0	0.0K+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	3.3K-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	4.6K-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	3.2K-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	2.4K-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	2.3K-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	0.0K+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	5.0K-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	2.0K-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	1.4K-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	1.3K-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	1.4K-08	100.8	95.6	90.4	367.0		

Golder Associates
Permeability Test Results

WHC-SD-W025-FDR-001, Rev. 1

Project Number: 893-1186.033 TEST RESULTS: I-2E-9 cm/second
Project Description: DOE RL/NON-DRAW-OFF/SEATTLE *****
Sample Number: GTP1-7 8XBENTONITE/23XMOISTURE
Sample Description: Lt. olive gray, med. to fine SAND,

Sample Dimensions (cgs): Initial Final Board number: L0091
Height: 7.369 7.256
Diameter: 7.259 7.211 Cell number: L0089
Area: 41.39 40.84
Volume: 305.0 296.3

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:
Reviewed by:

DM
KMB

Target Test Cond. (psig): Saturation During Test
Cell: 100.0 100.0
Top Stone: 92.5 90.0
Bottom stone: 92.5 95.0
B - coefficient: 0.99 M.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	min	sec	Elapsed (min.)	INFLOW Reading	OUTFLOW VOLUMES (cc)			I (cm/s) Delta	Transducer Reading (psi)			Head (cm H2O)	
								Reading	Drain	Delta		Cell	Bottom	Top		
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 TEST RESULTS: K=7E-5 cm/second
Project Description: DOE RL/WOM-DKAG-OFF/SEATTLE *****

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

Sample Dimensions (cgs):	Initial	Final	Board number:	L0015
Height:	7.187	7.255		
Diameter:	7.345	7.255	Cell number:	L0202
Area:	42.37	41.34		
Volume:	304.5	299.9		

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:
Reviewed by:

DN

KLS

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom 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)			K (cm/s)		Transducer Reading (psi)			Head (cm H2O)
			Reading	Drain	Delta	Total	Delta	Cell	Bottom	Top	
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 TEST RESULTS: K=5E-9 cm/second
Project Description: DOE RL/MON-DRAG-OFF/SEATTLE *****

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

Sample Dimensions (cgs):	Initial	Final	Board number:	L0091
Height:	7.204	7.174	Cell number:	L0008
Diameter:	7.247	7.286		
Area:	41.25	41.69		
Volume:	297.2	299.1		

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: *KRB*

Target Test Cond. (psig):	Saturation	During Test
Cell:	100.0	100.0
Top Stone:	92.5	90.0
Bottom 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 H2O)
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	3.8E-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 TEST RESULTS: K= 2X-9 cm/second
Project Description: DOR RL/WOM-DRAG-OFF/SEATTLE *****

Sample Number: - GYP1-7 16XBENTONITE
Sample Description: Lt. olive gray, med. to fine SAND,
some silty clay (SC).

Sample Dimensions (cgs):	Initial	Final	Board number:	L0091
Height:	7.177	7.126	Cell number:	L0007
Diameter:	7.254	7.258		
Area:	41.33	41.37		
Volume:	296.6	294.8		

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
Bottom 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)			K (cm/s)	Transducer Reading (psi)			Head	
								Reading	Drain	Delta	Total	Delta	Cell	Bottom	Top	(cm H2O)
89	12	6	7	52	0 *	0.00	9.7	22.2	0.0	0.0	0.0	0.0X+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	4.7X-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	4.1X-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	2.0X-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	1.2X-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	2.0X-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	0.0X+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	3.8X-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	2.2X-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	1.5X-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	1.6X-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	1.5X-09	100.1	94.7	90.6	299.5

GOLDER ASSOCIATES INC., REDMOND, WA

LATE ANALYSIS

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

APPENDIX A.2
INTERFACE SHEAR STRENGTH TESTING

WHC-SD-W025-FDR-001, Rev. 1

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:

<u>Normal Stress (psf)</u>	<u>Dry Density (pcf)</u>	<u>Moisture Content (%)</u>
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.

Mr. Tim Martin
Golder Associates, Inc.
Project No. 4147-90-104
January 22, 1990
Page 2

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

Westinghouse

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	460.56	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

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

WHC-SD-W025-FDR-001, Rev. 1

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°
Adhesion: 188 psf

Dry Interface: Angle of Interface Friction: 28°
Adhesion: 180 psf

Mr. Tim Martin
Golder Associates, Inc.
Project No. 4147-90-104
January 22, 1990
Page 2

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

Goldier 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	0.53	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		0.35	180.26	0.64	320.46
0.500	0.32	160.23		0.36	180.26	0.64	320.46

A.2-11
 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	0.32	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	395.57
1.025	0.42	210.30	0.47	235.34	0.79	395.57
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	445.64
1.675	0.56	280.40	0.61	305.44	0.88	440.63
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

Golden 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
 GOLDR ASSOCIATES

A.2-18

Westinghouse

PROJ NO 4147-90-104 FIG NO. 3

WHC-SD-W025-FDR-001, Rev. 1

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:

<u>Normal</u> <u>Stress (psf)</u>	<u>Dry</u> <u>Density (pcf)</u>	<u>Moisture</u> <u>Content (%)</u>
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.

Mr. Tim Martin
Golder Associates, Inc.
Project No. 4147-90-104
January 22, 1990
Page 2

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
 GOLDR ASSOCIATES

Westinghouse

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	Volts	Shear Stress (psf)	Volts	Volts	Shear Stress (psf)	Volts
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.			Volts	Shear Stress (psf)		Volts	Shear Stress (psf)		Volts	Shear Stress (psf)	
(in)											
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

PROJ

E

H-2

MIXED WASTE (RMW) LAND DISPOSAL FACILITY NON-DAG-OFF

PROJECT W-025

WASTE MANAGEMENT, INC., REDMOND (SEATTLE), WASHINGTON

LIST OF DRAWINGS

131573 COVER SHEET	H-2-131584 TRUCK STAGING AREA AND ACCESS RAMP
131574 SITE PLAN AND EXPLORATIONS	H-2-131585 LEACHATE COLLECTION TANK AND PIPING (2 SHEETS)
131575 GRADING PLAN	H-2-131586 SUMP PUMP DETAILS
131576 SECONDARY ADMIX CONTOURS	H-2-131587 ELECTRICAL (4 SHEETS)
131577 OPERATIONS LAYER CONTOURS	H-2-131588 CONCEPTUAL FILL PLAN
131578 CROSS-SECTIONS	H-2-131589 SUBGRADE GRID POINTS
131579 LINER SYSTEM DETAILS	H-2-131590 SECONDARY ADMIX GRID POINTS
131580 SUMP LAYOUT	H-2-131591 SECONDARY DRAINAGE GRAVEL GRID POINTS
131581 SUMP CROSS-SECTIONS	H-2-131592 PRIMARY DRAINAGE GRAVEL GRID POINTS
131582 SUMP LEACHATE COLLECTION PIPES	H-2-131593 OPERATIONS LAYER GRID POINTS
131583 SIDE SLOPE AND VERTICAL RISER PIPES	H-2-131594 PRIMARY ADMIX GRID POINTS

DESIGNED T. MARTIN 04/02/90		U. S. DEPARTMENT OF ENERGY ROMAN OPERATIONS OFFICE	
DRAWN J. SNOW 04/02/90		GOLDER ASSOCIATES INC.	
CHECKED F. SHULER 04/02/90		COVER SHEET	
REVIEWED P. CORSER 04/02/90		RMW LAND DISPOSAL FACILITY	
BY M. J. OTTERSON 04/02/90		NON-DAG-OFF	
PROJECT W-025		PAGE 1 OF 1	
SIZE 11x17 IN. (MAX. NO. 1)		H-2-131573	
F 200-0-00000		3	
SCALE AS SHOWN		JOB NO. 003-1106.035 SHEET 1 OF 1	

REV NO	DESCRIPTION	DATE	APPROVALS
3	REVISION FOR ECH W-025-001, ECH W-025-002	8/26/94	SGH F. SHULER
2	REVISION FOR ECH W-025-001, ECH W-025-002	5/21/93	JPP F. SHULER
1	ORIGINAL	5/1/92	KGH F. SHULER

DATE FILED	FILE NO.	PROJECT NO.	JOB NO.	SCALE	SHEET NO.
B131573A	2W:IBM-ACD2:10:SS				1

WHC-SD-W025-FDR-001, Rev. 1

DAYTON AVE

NOTES:

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

3-8427
3-8427
KMR
DRAW

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

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3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

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3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

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3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

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3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

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3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

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3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

DATE		BY		APPROVALS		REVISIONS		DESCRIPTION		DATE		APPROVALS		DATE		APPROVALS		DATE		APPROVALS	
3	8/26/94	F. SHUR																			
2	5/21/93	F. SHUR																			
1	5/1/92	F. SHUR																			

WHC-SD-W025-FDR-001, Rev. 1

DAYTON AVE

E

D

C

B

A

685.6 685.6 685.71
2W-14
N44,501.09
W78,194.99
EL=685.71

LIMIT OF SUMP

SEE DETAIL: 1
H-2-131500

TANK AREA
SEE NOTE 2

FOR V-DITCH
SEE DETAIL:
17
H-2-131504

E2,216,500
N450,500

W79,000

W-025 LOBE 1 SUBMITTAL/CM #22385 CROSS INDEX

SUB#	SUP#	SPEC REF.	DESCRIPTION
301		01500, 1.4, 1.2, 1	6" Hot Top Items
304		01050, 1.5, 5	As-Built Survey of Subgrade

NOTES:

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

DATE	
BY	
CHECKED	
APPROVED	
NAME	
DATE

CONTROL POINT SUBGRADE COORDINATES

POINT NO.	NORTHING	EASTING	ELEVATION
CP1	44201.68	78687.74	671.98
CP2	44200.85	78453.00	663.12
CP3	44200.85	78359.12	694.42
CP4	44121.29	78585.41	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	44307.28	78356.20	696.00
CP10	44337.15	79391.44	696.00
CP11	44341.99	78467.25	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	78793.66	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	44085.07	78289.19	698.43
CP22	43952.04	78285.92	697.40
CP23	44065.07	78323.34	699.84
CP24	43972.36	78323.41	699.75
CP25	43972.88	78427.30	699.50

EXPLANATION

544.500	HANFORD PLANT COORDINATE
502.210 000	WASHINGTON STATE PLANE COORDINATE
EXISTING FENCE
1.02	SURVEY MONUMENT
5.02	
1.56	
693.2 •	SPOT ELEVATION
UTILITY POLE
— C —	CONTROL LINE
CP1	CONTROL POINT

DESIGNATION

1

H-2-131580

SHEET NUMBER

		AS-BUILT FOR ECH CHANGE CORRESPONDING POINTS FOR ECH W-252-013 AND CH CROSS INCHES AND SIZE S FOR ECH W-21-020 ADJUST W/OUT BEARING AREA CORRESPONDING POINTS FOR ECH W-252-013 MOVE ACCESS ROAD 10 FT NORTH FOR ECH W-210-012 CORRESPONDING POINTS FOR ECH W-252-013 REWORK SPECIFIC AREA FOR ECH W-252-013 FOR ECH W-252-013 PLEASE CONSIDER CORRECTING WEST OF LINEAL ADJUST LAMBERT, COLUMBIA AND CORRESPONDING POINTS		DESIGNED T. MARTIN 04/02/90 DRAWN J. S. NOW 04/02/90 CHECKED F. SHURI 04/02/90 REVISIONS P. CORSER 04/02/90 OR M. J. OTTERSON 04/02/90		U. S. DEPARTMENT OF ENERGY REGIONAL OPERATIONS OFFICE GO DER ASSOCIATES INC. GRADING PLAN RMW LAND DISPOSAL FACILITY NON-DRAG-OFF	
3		SGH F. SHURI 8/26/94	APPROVED FOR IMPLEMENTATION		PROJECT W-025		
2		LPP F. SHURI 5/21/93	BY		SIZE (SHEET NO.) (SHEET NO.) (SHEET NO.) F 250-4-4 0401 H-2-131575 3		
1		NGH F. SHURI 5/1/92					
REV NO		DESCRIPTION	DATE	APPROVALS			
		REVISIONS 00 FILE 0131575A 100 CDR 20000101ACD2-10-CC					
ECHW 100 LVL BLK BGRN (CHS 2.1 AND 7)		NUMBER		TITLE		REFERENCES	
TRACEABILITY LIST		NEXT USED ON					

WHC-SD-W025-FDR-001, Rev. 1

DAYTON AVE

E

D

C

B

A

NOTES:

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

B-4

WHC-SD-W025-FDR-001, Rev. 1

B-5

NOTES:

1. Vertical riser pipe not shown.
2. Topographic maps prepared by Degross Aerial Mapping based on July 18, 1988, aerial photography.
3. Topography west of landfill revised January 21, 1992 per ECH 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.

WHC-SD-W025-FDR-001, Rev. 1

COMPACT
GEOSYNTHETIC
SYSTEM
SEE DETAIL

KEY:

DESIGNATION
C
No. 2-131577
SHEET NUMBER

B-6

DESIGNED T. MARTIN 04/02/90			U. S. DEPARTMENT OF ENERGY		
DRAWN J. SHAW 04/02/90			GOLDER ASSOCIATES INC.		
CHECKED F. SHAW 04/02/90			CROSS-SECTIONS		
REVIEWED P. CORSEY 04/02/90			RMW LAND DISPOSAL FACILITY		
BY M. J. OTTERSON 04/02/90			NON-DRAW-OFF		
PROJECT W-025			H-2-131578		
SIZE 11x17 (40x28) (1/4"=1'-0")			3		
SCALE AS SHOWN			1		

NO.	DESCRIPTION	DATE	APPROVALS
3	AS-BUILT FOR CONSTRUCTION FOR RMW H-2-131578, RMW H-2-131579	8/26/94	SGH F. SHAW
2	ADDITIONAL	5/21/93	JPP F. SHAW
1	INCREASE THICKNESS OF LINING SYSTEM	5/1/92	NGH F. SHAW

FILE	REVISIONS
B131578A	2W:IGM/ACD2:10:SS

FILE	REVISIONS
B131578A	2W:IGM/ACD2:10:SS

ANCHOR TRENCH DETAIL

5
H-2-131578

0 2 4
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			
SUB#	SUP#	SPEC. REF.	DESCRIPTION
25	02275	1.7A	Geomembrane Installer's QA Manual
25A	02275	1.7A	Geomembrane Crystallinity
25B	02275	1.7A	Geomembrane/Admix Friction Angle
25.1	02275	1.7A	Geomembrane QA/QC Data
25.2	02275	1.7A	Extrudate Certification
25.3	02275	1.7A	Geomembrane QA/QC Data
25.3A	02275	1.7A	Geomembrane Resin Certification
25.4	02275	1.7A	Geomembrane QA/QC Data
25.4B	02275	1.7A	Geomembrane Env. Stress Crack Certification
25.4C	02275	1.7A	Geomembrane Env. Stress Crack Certification
25.5	02275	1.7A	Geomembrane Low Temp. Welding Certification
25.6	02275	1.7A	Geomembrane QA/QC Documentation
25.6A	02275	1.7A	Geomembrane Direct Shear Test Report
25.7	02275	1.7A	Geomembrane Resin Documentation
25.8	02275	1.7A	Geomembrane Resistance to Soil Burial Cert.
25.9	02275	1.7A	Geomembrane Resistance to Soil Burial Cert.
25	02275	1.7B	Geosynthetic Samples

NOTES:

- Operations layer shall consist of soils excavated from on site, meeting the requirements of Section 0222B of the Technical Specifications.
- Compacted admix shall meet the composition, compaction, and permeability requirements as described in Section 02224 of the Technical Specifications.
- Primary and secondary base liner drainage gravel shall meet the requirements of Section 0222B of the Technical Specifications.
- Geosynthetic materials shall meet the requirements of Section 02275 of the Technical Specifications.
- Geosynthetic layers not drawn to scale.
- Grade shall be minimum of 1 foot above underlying layer.
- Grade shall be minimum of 3 feet above underlying layer.
- Overlap geomembrane as appropriate for welding method (see Specifications Section 02275).
- Tolerances based on design grade.
- Geotextile to be seen (see Specifications Section 02275).
- Grade shall be minimum of 1.5 feet above underlying layer.
- For as-built conditions, see "Secondary HDPE Geomembrane Liner As-Built Drawing", and "Primary HDPE Geomembrane Liner As-Built Drawing" in the CDA Record, WHC-SD-W025-RPT-001.

KEY:

5
H-2-131578
DESIGNATION
SHEET NUMBER

GRADING TOLERANCE DETAIL

0 2 4
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

[illegible]

0 5 10
SCALE IN FEET

0 10 20
SCALE IN FEET

SUMP SUBGRADE 1
H-2-131580

0 10 20
SCALE IN FEET

P SECONDARY GRAVEL 3
H-2-131580

0 10 20
SCALE IN FEET

SUMP PRIMARY ADMIX

0 10 20
SCALE IN FEET

		3. APPROVED FOR ECH ADD OR CROSS HIDE AND NOTE 2 FOR ECH W-022-003 COTTON, POLYESTER-003-003, ECH W-022-003		SCHI F. SHURI 8/26/94 JPP F. SHURI 5/21/93 KGH F. SHURI 5/5/92		Drawn J. SHOW 04/02/90 CHECKED F. SHURI 04/02/90 REVISED P. CORSER 04/02/90 OR M. J. OTTERSON 04/02/90		GOLDER ASSOCIATES INC. SUMP LAYOUT RMW LAND DISPOSAL FACILITY NON-DRAW-OFF	
		1. INCREASE COTTONS LAYER TO 2" FOR 3" FILL FOR IMPROVED STABILITY AND DRAINAGE						PROJECT W-025 SIZE (BLDG NO. POLE NO. CHL NO.) 238-0-0 0401 H-2-131580 SCALE AS SHOWN JOB NO. 903-1184-0301 SHEET 1 OF 3	
		IF NO. DESCRIPTION DATE APPROVALS		REVISIONS		APPROVED FOR IMPROVISED		BY DATE	
TITLE WING "RACABILITY LIST"		NUMBER NOT USED ON		TITLE REFERENCES		DO NO. B131580A QAO CODE 2V/IBM-ACD2-10:SS			

WHC-SD-W025-FDR-001, Rev. 1

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 slump area.
4. Round off trench corners as required.

KEY:

DESIGNATION
H-2-131580
SHEET NUMBER

SECTION

F
H-2-131580

0 2 4
SCALE IN FEET

SECTION

H
H-2-131580

0 2 4
SCALE IN FEET

DESIGNED: T. MARTIN 04/02/90	WASTE LEVEL	151814
DRAWN: J. SNOW 04/02/90	U. S. DEPARTMENT OF ENERGY BIDING OFFICE	
CHECKED: F. SHURT 04/02/90	GOLDER ASSOCIATES INC.	
REVIEWED: P. CORSEY 04/02/90	SUMP CROSS-SECTIONS	
BY: M. J. OTTERSON 04/02/90	RMW LAND DISPOSAL FACILITY	
	NON-DRAIN-OFF	
PROJECT: W-025	SIZE: 1/8" X 1/4" X 1/4" X 1/4"	REV. NO. 3
APPROVED FOR IMPLEMENTATION	F 1218-4-4 0401	H-2-131581

NO.	DESCRIPTION	DATE	APPROVALS
3	RE-DESIGN FOR CDM	8/25/94	SCHE F. SHURT
2	WASTE CONTROL	5/21/93	JPP F. SHURT
1	INCREASE OPERATIONS LAYER THICKNESS TO 3 FT.	5/1/92	KGH F. SHURT

WHC-SD-W025-FDR-001, Rev. 1

PRIMARY LEACHATE COLLECTION SUMP

10

W-2-131582

0 2 4
SCALE IN FEET

LEACHATE COLLECTION WELL HOLE DETAIL

12

W-2-131582

0 1 2
SCALE IN FEET

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

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

NOTES:

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

KEY:

G
W-2-131582
DESIGNATION
SHEET NUMBER

B-10

SECTION

SECONDARY SLO

N.T.S.

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

SECTION

REINFORCED PIPE DETAIL

15
H-2-131583

2 4
IN FEET

ELEVATION

RISER PIPE

ELEVATION

PLAN

SECONDARY SLOPE RISER PIPE

21
H-2-131583

N.T.S.

SIDESLOPE CLEANOUT RISER DETAIL

40
H-2-131583

N.T.S.

REVISIONS				APPROVALS			
NO.	DESCRIPTION	DATE	BY	NO.	DESCRIPTION	DATE	BY
3	REVISOR FOR 8 IN	10/6/94	SCH. F. SHURI	1	ADD PRIMARY AND LIND	5/1/92	HCHI F. SHURI
2	REVISOR FOR 8 IN	5/21/93	LPP F. SHURI	2	REVISOR FOR 8 IN	5/21/93	LPP F. SHURI
1	REVISOR FOR 8 IN	5/21/93	LPP F. SHURI	3	REVISOR FOR 8 IN	5/21/93	LPP F. SHURI

DESIGN	M. BROWN	04/02/90
DRAWN	J. SNOW	04/02/90
CHECKED	F. SHURI	04/02/90
REVIEWED	P. CORSER	04/02/90
DATE	M. J. OTTERSON	04/02/90

PROJECT	W-025
SCALE	AS SHOWN
DATE	04/02/90
BY	F. SHURI
FOR	DATE

U. S. DEPARTMENT OF ENERGY	151914
OFFICE OF ENVIRONMENTAL MANAGEMENT	
GOLDER ASSOCIATES INC.	
SIDE SLOPE AND VERTICAL RISER PIPES	
RMW LAND DISPOSAL FACILITY	
NON-DRAW-OFF	
PROJECT	W-025
SCALE	AS SHOWN
DATE	04/02/90
BY	F. SHURI
FOR	DATE

FILE	B131583A
CODE	2W:IBM:ACD2:10:SS
NO. USED ON	3
NO. USED ON	3

DAYTON AVENUE

R = 50 FT (197)

GRADING PLAN OF TRUCK STAGING AND UNLOADING AREAS

16
H-2-131584

0 20 40
SCALE IN FEET

PAVED CROSS SECTION OF TRUCK STAGING AND UNLOADING AREAS

0 20 40
SCALE IN FEET

W-025 LOBE 1 SUBMITTAL/CN #22585 CROSS INDEX			
SUB	CV	SPEC. REF.	DESCRIPTION
31	02511, 1.5		Asphalt Mix Design
31.1	02511, 1.5		Asphalt Seal Coat

NOTES:

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

KEY:

B-12

PROJECT: RMW LAND DISPOSAL FACILITY TRUCK STAGING AREA & ACCESS RAMP GOLDEN ASSOCIATES INC. 151914		APPROVED FOR CONSTRUCTION DATE: 04/02/90 BY: J. SHULTZ PROJECT: W-025 NON-DRAIN-OFF		REVISIONS NO. DATE DESCRIPTION 1 5/21/93 F. SHULTZ 2 8/26/94 F. SHULTZ 3 04/02/90 J. SHULTZ		REFERENCES TITLE NUMBER		TBL NO. RACIABILITY LIST	
U.S. DEPARTMENT OF ENERGY BUREAU OF RECLAMATION 151914		APPROVED FOR CONSTRUCTION DATE: 04/02/90 BY: J. SHULTZ PROJECT: W-025 NON-DRAIN-OFF		REVISIONS NO. DATE DESCRIPTION 1 5/21/93 F. SHULTZ 2 8/26/94 F. SHULTZ 3 04/02/90 J. SHULTZ		REFERENCES TITLE NUMBER		TBL NO. RACIABILITY LIST

LEACHATE TEMPORARY STORAGE TANK & PIPING PLAN

NOTES:

1. Tank shall be ASME Code stamped for 25 psi design pressure.
2. All welds shall be continuous and ground smooth.
3. Tank point 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 water-tight liner on floor and sides of concrete containment areas. Details of liner around tank access 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.

0 5 10
SCALE IN FEET

N

KEY:

DESIGNATION
SHEET NUMBER

WS7 VALVE AND PUMP IDENTIFICATION

B-13

WALL REINF. WITH #408 IN VERTICAL
#4012 IN HORIZONTAL

TAIL

DETAIL 23
H-2-131585

SECTION K
H-2-131585

SECTION N
H-2-131585

NOT TO SCALE
SLOPE EXAGGERATED FOR CLARITY

DR BOLT DETAIL

NOT TO SCALE

LEACHATE TEMPORARY STORAGE TANK & PIPING
SECTION J
H-2-131585

WING TRACEABILITY LIST		NEXT USED ON		4-4433 FOR ECH REVISE FLIGHTPLAN FOR ECH 9-425-423 ADD RUC SHED SCHEM FOR ECH 9-425-423 DATE 9-423-77 3 REVISE FPMG LIGHT FOR ECH 9-425-423 ADD INSPECTION FOR ECH TRUCK AND INSULATOR VALUE NOT FOR ECH 9-425-423 ADD RAMP COORDINATE FPMG LEACHATE REGULATION LINE FOR ECH 9-425-411 EXTERNAL FOR ECH 9-425-423 DATE 9-423-423		SJS F. SHURI 10/6/94		WING LEVEL OF 151814	
				2 MINOR CORRECTIONS JPP F. SHURI 5/21/93		U. S. DEPARTMENT OF ENERGY REGIONAL OPERATIONS OFFICE GOLDER ASSOCIATES INC.		LEACHATE COLLECTION TANK & PIPING RMW LAND DISPOSAL FACILITY NON-DRAG-OFF	
				1 ADVISE OPERATIONS UNDER PERMITS BY 2 RUC AND ADD RUC 8-122-77 TO EXISTING RUC JPP F. SHURI 5/1/92		4 M. J. OTTERSON 04/02/90			
TITLE		NUMBER		TITLE		REVISED		REV NO	
REFERENCES				DESCRIPTION		DATE		APPROVALS	
				REVISIONS		APPROVED FOR IMPROVEMENT		BY	
				CDO FILE B131585A		CDO CODE 2W:IBM:ACD2:10:SS		FOR DATE	
								PROJECT W-025 SIZE 1800 NO. 0401 H-2-131585 3 SCALE AS SHOWN 1:500 NO. 003-1166.030 SHEET 1 OF 2	

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SECTION P

H-2-131585

SECTION Q

H-2-131585

N

CONTAINMENT BASIN ACCESS STAIR PLAN

36
H-2-131585

W-025 LOBE 1 SUBMITTAL/CN #22583 CROSS INDEX			
SUB#	CN	SPEC REF.	DESCRIPTION
39	03210, 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 - Letter
53.4	13205, 1.3.A		Leachate Tank Fabrication Drawings - ASME Cert.
53.5	13205, 1.3.A		Leachate Tank Fabrication Drawings
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		Same Cocks
65	15100, 1.3.A		Check Valves
66	15100, 1.3.A		Automatic Drain Valve - Manuf. Information
67	15104, 1.2.A		Butterfly Valves
68A	15104, 1.2.A		Butterfly Valves
69	15150, 1.3.A		Flowmeter
70	15250, 1.3.A		Leachate Tank and Pipe Insulation - Manuf. Info.
71	15250, 1.3.B		Leachate Tank and Pipe Insulation - Certification

B-14

CREST WALL CABLE LAYOUT 39
H-2-131585
NOT TO SCALE

QUICK CONNECT SECTION 0
H-2-131585
NOT TO SCALE
NOTE: INSULATE EXTENSION PIPES, CHECK VALVE, BALL VALVES, AND QUICK CONNECT.

HOSE CONNECTION DETAIL 37
H-2-131585
0 5
SCALE IN FEET

DISCHARGE PIPING SCHEMATIC 38
H-2-131585
NOT TO SCALE

REVISIONS			APPROVALS		
NO	REV	DESCRIPTION	DATE	BY	FOR
0	AS-BUILT FOR ECH		10/06/84	F. SHURI	
PROJECT: W-025			PROJECT: W-025		
SHEET: 0401			SHEET: 0401		
SCALE: AS SHOWN			SCALE: AS SHOWN		

ISSUED: M. BROWN 04/02/90	U. S. DEPARTMENT OF ENERGY
DRAWN: J. SNOW 04/02/90	GOLDER ASSOCIATES INC.
CHECKED: F. SHURI 04/02/90	LEACHATE COLLECTION TANK & PILING
REMOVED: P. CORSEY 04/02/90	RMW LAND DISPOSAL FACILITY
ON: M. J. OTTERSON 04/02/90	NON-DRAW-OFF
APPROVED FOR MODIFICATION	
DATE

PRIMARY LEACHATE SELF PRIMING PUMP (W5P3) SECTION

SEE DETAIL: 25, H-2-131583

0 10 20
SCALE IN INCHES

LEVEL SENSOR TRANSDUCER DETAIL

25, H-2-131583

NOT TO SCALE

NOTES:

- Requirements for drain down valve and associated piping are described in Section 15100 of the Technical Specifications.
- Requirements for float switch housing are described in Section 12340 of the Technical Specifications.
- 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	SPEC. REF.	DESCRIPTION
44	05500, 1.2		Vertical Riser Lid and Grating Fab. Drawings
50	11210, 1.2.A		Pumps, Motors, Materials Information
50.1	11210, 1.2.A		Pumps, Motors, Materials Information
50.1A	11210, 1.2.A		Pumps, Motors, Materials Information
	11210		Transfer Pump - Installation, Operation, and Maintenance Instructions; Parts and Disassembly Instructions
	11210		Submersible Pump - Installation and Operation Instructions
	11210		Submersible Pump - Performance Curves
	11210	1-2, A; 2-1	Self-Priming Pump - Installation, Operation, and Maintenance Instructions; Spare Parts List; Maintenance Manual
	11210	1-2	Volume-Depth Curve; Primary Pump
300	11210		Final Self-Priming Pump Piping-Box Arrangement

DESIGNATION		3 H-2-131586		SHEET NUMBER		DESIGNED M. BROWN 04/02/90 DRAWN J. SHOUR 04/02/90 CHECKED F. SHOUR 04/02/90 REVISIONS P. CORSEY 04/02/90 OK M. J. OTTERSON 04/02/90		WASTE LEVEL 151014 U. S. DEPARTMENT OF ENERGY RADIATION PHYSICS DIV. GOLDER ASSOCIATES INC. SUMP PUMP DETAILS RMW LAND DISPOSAL FACILITY NON-DRAG-OFF	
TITLE		MAPING		SCALE		PROJECT W-025		SIZE (BASE NO.) (MOCK NO.) (DWC NO.) F 300-0 0401 H-2-131586 SCALE AS SHOWN JOB NO. 203-1186.0330 SHEET 1 OF 3	
DRAWING "RACABILITY LIST"		NOT USED ON		REFERENCES		REVISIONS 1. 04/02/90 2. 04/02/90 3. 04/02/90 4. 04/02/90 5. 04/02/90 6. 04/02/90 7. 04/02/90 8. 04/02/90 9. 04/02/90 10. 04/02/90 11. 04/02/90 12. 04/02/90 13. 04/02/90 14. 04/02/90 15. 04/02/90 16. 04/02/90 17. 04/02/90 18. 04/02/90 19. 04/02/90 20. 04/02/90 21. 04/02/90 22. 04/02/90 23. 04/02/90 24. 04/02/90 25. 04/02/90 26. 04/02/90 27. 04/02/90 28. 04/02/90 29. 04/02/90 30. 04/02/90 31. 04/02/90 32. 04/02/90 33. 04/02/90 34. 04/02/90 35. 04/02/90 36. 04/02/90 37. 04/02/90 38. 04/02/90 39. 04/02/90 40. 04/02/90 41. 04/02/90 42. 04/02/90 43. 04/02/90 44. 04/02/90 45. 04/02/90 46. 04/02/90 47. 04/02/90 48. 04/02/90 49. 04/02/90 50. 04/02/90 51. 04/02/90 52. 04/02/90 53. 04/02/90 54. 04/02/90 55. 04/02/90 56. 04/02/90 57. 04/02/90 58. 04/02/90 59. 04/02/90 60. 04/02/90 61. 04/02/90 62. 04/02/90 63. 04/02/90 64. 04/02/90 65. 04/02/90 66. 04/02/90 67. 04/02/90 68. 04/02/90 69. 04/02/90 70. 04/02/90 71. 04/02/90 72. 04/02/90 73. 04/02/90 74. 04/02/90 75. 04/02/90 76. 04/02/90 77. 04/02/90 78. 04/02/90 79. 04/02/90 80. 04/02/90 81. 04/02/90 82. 04/02/90 83. 04/02/90 84. 04/02/90 85. 04/02/90 86. 04/02/90 87. 04/02/90 88. 04/02/90 89. 04/02/90 90. 04/02/90 91. 04/02/90 92. 04/02/90 93. 04/02/90 94. 04/02/90 95. 04/02/90 96. 04/02/90 97. 04/02/90 98. 04/02/90 99. 04/02/90 100. 04/02/90 101. 04/02/90 102. 04/02/90 103. 04/02/90 104. 04/02/90 105. 04/02/90 106. 04/02/90 107. 04/02/90 108. 04/02/90 109. 04/02/90 110. 04/02/90 111. 04/02/90 112. 04/02/90 113. 04/02/90 114. 04/02/90 115. 04/02/90 116. 04/02/90 117. 04/02/90 118. 04/02/90 119. 04/02/90 120. 04/02/90 121. 04/02/90 122. 04/02/90 123. 04/02/90 124. 04/02/90 125. 04/02/90 126. 04/02/90 127. 04/02/90 128. 04/02/90 129. 04/02/90 130. 04/02/90 131. 04/02/90 132. 04/02/90 133. 04/02/90 134. 04/02/90 135. 04/02/90 136. 04/02/90 137. 04/02/90 138. 04/02/90 139. 04/02/90 140. 04/02/90 141. 04/02/90 142. 04/02/90 143. 04/02/90 144. 04/02/90 145. 04/02/90 146. 04/02/90 147. 04/02/90 148. 04/02/90 149. 04/02/90 150. 04/02/90 151. 04/02/90 152. 04/02/90 153. 04/02/90 154. 04/02/90 155. 04/02/90 156. 04/02/90 157. 04/02/90 158. 04/02/90 159. 04/02/90 160. 04/02/90 161. 04/02/90 162. 04/02/90 163. 04/02/90 164. 04/02/90 165. 04/02/90 166. 04/02/90 167. 04/02/90 168. 04/02/90 169. 04/02/90 170. 04/02/90 171. 04/02/90 172. 04/02/90 173. 04/02/90 174. 04/02/90 175. 04/02/90 176. 04/02/90 177. 04/02/90 178. 04/02/90 179. 04/02/90 180. 04/02/90 181. 04/02/90 182. 04/02/90 183. 04/02/90 184. 04/02/90 185. 04/02/90 186. 04/02/90 187. 04/02/90 188. 04/02/90 189. 04/02/90 190. 04/02/90 191. 04/02/90 192. 04/02/90 193. 04/02/90 194. 04/02/90 195. 04/02/90 196. 04/02/90 197. 04/02/90 198. 04/02/90 199. 04/02/90 200. 04/02/90 201. 04/02/90 202. 04/02/90 203. 04/02/90 204. 04/02/90 205. 04/02/90 206. 04/02/90 207. 04/02/90 208. 04/02/90 209. 0			

WHC-SD-W025-FDR-001, Rev. 1

NOTES:

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

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

LEACHATE TEMPOR SCHEMATIC C

W-025 LOBE 1 SUBMITAL/CN #22563 CROSS INDEX		
CN	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:

B-17

0
SCALE

N

[illegible]

WHC-SD-W025-FDR-001, Rev. 1

DAYTON AV

E2,217,000

N450.50

W78,500

E2,216,590

N450.500

W79,000

E2,216,000

N450.500

N44,500

LIGHT POLE #4

TANK

CONDUIT & SEE SCHEDULE

POLE SCHEDULE

POLE NO.	COORDINATES		LUMINAIRES	NOTES
	N	W		
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	

B-18

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

Sparling & Associates, Inc. 1920 Eastlake Avenue East Seattle, Washington 98102										PROJECT LEVEL										DATE 12/18/14									
										U. S. DEPARTMENT OF ENERGY EOLAND OPERATIONS OFFICE										GOLDER ASSOCIATES INC.									
										ELECTRICAL SITE PLAN										RMW LAND DISPOSAL FACILITY									
										NON-DRAIN-OFF																			
										PROJECT W-025																			
										SIZE (BLDG NO. 1) (BLDG NO. 2) (BLDG NO. 3)										REV. NO. 3									
										BY 7401										H-2-131587									
										FOR DATE										AS SHOWN									
										DRAWING NO. 8131587C										JOB NO. 2W-IBM-ACD2-10:SS

29
4-2-131587-3

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

NOTE: GROUND TRANSFORMER

N.T.S.

		  	
--	--	--	--

DETAIL BARRIER BLOCK WALL

34
H-2-131588

0 4 8
SCALE IN FEET

SECTION A

A
H-2-131588

0 30 60
SCALE IN FEET

SECTION B

B
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. Fleet 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. Filing 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.

BACKFILL COVER

REMOTE HANDLED WASTE

OPERATIONS LAYER OR INTERMEDIATE COVER

750

700 ELEVATION (FEET)

650

TOP OF OPERATIONS LAYER CONTOURS

TOP OF WASTE CONTOURS

CONCEPTUAL FILL PLAN TOP OF WASTE CONTOURS

0 50 100
SCALE IN FEET

750

700 ELEVATION (FEET)

650

KEY:

DESIGNATION

C

SHEET NUMBER

LEGEND

- REMOTE HANDLED WASTE PACKAGE
- CONCRETE BLOCK RADIATION BARRIER WALL

DRAWING TRACEABILITY LIST			REVISIONS			APPROVALS			PROJECT		
FILE	NUMBER	FILE	NO	DESCRIPTION	DATE	BY	DATE	FOR	PROJECT	W-025	151014
DRAWING *TRACEABILITY LIST			3	AS-BUILT FOR CONCEPTUAL FILL PLAN	8/28/94	SGH	F. SHURI		U. S. DEPARTMENT OF ENERGY ROUND OPERATIONS OFFICE		
			2	UNION CIRCULAR	5/21/93	JPP	F. SHURI		GO/DER ASSOCIATES INC.		
			1	REVISE LINE GEOMETRY	5/1/92	JPP	F. SHURI		CONCEPTUAL FILL PLAN		
									RMW LAND DISPOSAL FACILITY		
									NON-DRAG-OFF		
									PROJECT W-025		
									SCALE AS SHOWN		
									SHEET 1 OF 1

NOTES:

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

EXPLANATION

N44,500 HANFORD PLANT COORDINATE
 E2,216,000 WASHINGTON STATE PLANE COORDINATE

--- TOE OF SLOPE
 --- C --- CONTROL LINE
 CP1▲ CONTROL POINT

GRID POINT — POINT NUMBER
 — POINT ELEVATION

WIND TRACEABILITY LIST		NOT USED ON		REFERENCES		3 AS-BUILT FOR EIR 2 LUMP CROWN 1 VERIFY LUMP/CL, GEOMETRY AND CONTROL POINTS		3 F. SHURE 8/26/94 2 F. SHURE 5/21/93 1 F. SHURE 5/5/92		APPROVED FOR EIR/DOE DATE		PROJECT W-025 F 20-4-4 0401 H-2-131589 3	
8131589A		2W:IBM:ACD2:10:SS

NOTES:

1. Complete truck staging area grid points shown on Sheet H-2-131584.
2. Topographic maps prepared by Degress 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

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.

EXPLANATION	
144,500	HANFORD PLANT COORDINATE
E2,216,000	WASHINGTON STATE PLANE COORDINATE
---	TOE OF SLOPE
— C —	CONTROL LINE
CP1▲	CONTROL POINT
GRID POINT	POINT NUMBER
	POINT ELEVATION

DESIGNED T. MARTIN 04/02/90		PROJECTED 151014	
DRAWN J. SHAW 04/02/90		U. S. DEPARTMENT OF ENERGY	
CHECKED F. SHURT 04/02/90		ROSLAND OPERATIONS OFFICE	
REVIEWED P. CORSEY 04/02/90		GOLDER ASSOCIATES, INC.	
BY M. J. OTTERSON 04/02/90		SECONDARY GRAVEL GRID POINTS	
		RMW LAND DISPOSAL FACILITY	
		NON-DRAW-OFF	
PROJECT W-025		SHEET 1 OF 1	
DATE 04/01/90		H-2-131591	
SCALE AS SHOWN		JOB NO. 903-1106.036	

NO.	DESCRIPTION	DATE	APPROVALS
1	10-BEAR FOR CEN	8/26/94	SCH. F. SHURT
2	HOW ACCESS ROAD IS 14 FT WIDE FOR 10-BEAR FOR CEN	5/21/93	JPP/F. SHURT
3	NEED EXTENSION	5/5/92	JPP/F. SHURT
4	NEEDY LANDFILL SECURITY AND CONTROL POINTS		

FILE	NUMBER	FILE
ING TRACEABILITY LIST	NEED USED ON	REFERENCES

CD FILE	B131591A	CD CODE	2W:IRM:ACD2:10:SS
---------	----------	---------	-------------------

B-24

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

EXPLANATION

N44,500 HANFORD PLANT COORDINATE
 E2,215,000 WASHINGTON STATE PLANE COORDINATE

--- TOE OF SLOPE
 --- C --- CONTROL LINE
 CP▲ CONTROL POINT

GRID POINT — POINT NUMBER
 — POINT ELEVATION

						DESIGNED T. MARTIN 04/02/90		U. S. DEPARTMENT OF ENERGY SCIENCE AND OPERATIONS OFFICE GOLDER ASSOCIATES INC PRIMARY GRAVEL GRID POINTS RMW LAND DISPOSAL FACILITY NON-DRAG-OFF	
						DRAWN J. SHAW 04/02/90			
						CHECKED F. SHAW 04/02/90			
						REVIEWED P. CORNER 04/02/90			
								BY M. J. OTTERSON 04/02/90	

WHC-SD-W025-FDR-001, Rev. 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. Indicated elevations do not include geosynthetic thickness and grading tolerances. Maintain required layer thickness at all locations.

0 20 40
FEET

EXPLANATION

1144,500	HANFORD PLANT COORDINATE
22,216,000	WASHINGTON STATE PLANE COORDINATE
-----	TOE OF SLOPE
— C —	CONTROL LINE
CP1 ▲	CONTROL POINT

GRID POINT — X — POINT NUMBER
 079.08
 POINT ELEVATION

DRAWING NO.		REV		131014	
DESIGNED T. MARTIN 04/02/90		CHECKED J. SNOW 04/02/90		U. S. DEPARTMENT OF ENERGY EDWARDS GORDON OFFICE	
DESIGNED F. SHURI 04/02/90		CHECKED P. CORSEY 04/02/90		GOLDER ASSOCIATES INC.	
DESIGNED M. J. OTTERSON 04/02/90		CHECKED F. SHURI 04/02/90		OPERATIONS LAYER GRID POINTS RMW LAND DISPOSAL FACILITY NON-DRAW-OFF	
PROJECT W-025		APPROVED FOR REPLICATION		BY	
SCALE AS SHOWN		DATE		REV NO	
JOB NO. 003-1186-035		SHEET 1 OF 1		REV NO

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 K. HASKELL 05/01/92 DRAWN K. HASKELL 05/01/92 CHECKED F. SHURE 05/05/92 REVIEWED G. MAJOR 05/06/92 OR J. GORDMAN 05/07/92		U. S. DEPARTMENT OF ENERGY RICHLAND OPERATIONS OFFICE GOLDER ASSOCIATES INC. PRIMARY ADMIX GRID POINTS RMW LAND DISPOSAL FACILITY NON-DRAW-OFF	
PROJECT W-025 SHEET NO. 0401 SCALE AS SHOWN		H-2-131594 SHEET 1 OF 1	

NO.	REV.	DESCRIPTION	DATE	APPROVALS
2		AD-BAIT FOR BOM	8/28/94	SCH F. SHURE
1		AD-BAIT FOR BOM	5/21/93	LPD F. SHURE

FILE	REFERENCES
NO TRACEABILITY LIST	NOT USED ON

CO FILE B131594A
 CO CODE 2W:IBM:ACD2:10:SS

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

11-17-94
Date

Project Manager

11-17-94
Date

QA Manager

11-17-94
Date

Design Engineer

11-17-94
Date

Westinghouse Hanford Company

11-17-94
Date

APPENDIX C
DESIGN CALCULATIONS

VOLUME II

TABLE OF CONTENTS

- 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
- C.16 Elastic Settlement of Foundation Sand
- C.17 Consolidation Settlement of Compacted Admix
- C.18 Settlement Summary
- C.19 Subsidence And Sinkhole Potential
- C.20 Potential for Uplift
- C.21 Earth Loading Primary Slope Riser Pipe
- C.22 Earth Loading Primary Drainage Pipe
- C.23 Down Drag Analysis
- C.24 Earth Load Leachate Collection Well
- C.25 Geotextile Selection
- C.26 Estimated Permeability of Drainage Gravel
- C.27 Puncture Resistance of HDPE Liner
- C.28 Vertical Riser Pipe Seismic Design Analysis

Calculation Cover Sheet

Sheet 1 of 75Project Title: WHC- Non Drag OffProject No. 893- 1186.032Calculation Title: Surface Water Hydrology + High Runs -
Appendices B.21 and B.11

Name

Date

Prepared By: MICHAEL BROWN Dec. 1989Checked By: JOHN VERMESIS JANUARY 19, 1990Approved By: [Signature] 8 FEB 90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

APPENDIX B.21 SURFACE WATER HYDROLOGY

WHC-SD-W025-FDR-001, Rev. 1

**Golder
Associates**

SUBJECT **NON DRAG OFF / SURFACE WATER**

Job No. 893-1186.032

Made by **MUP**

Date **11/13/89**

Ref.

Checked **JSU**

Sheet **2** of **25**

Reviewed **RL**

OBJECTIVES:

The waste pit is to be surrounded by a 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 site and off site to discharge point.

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

**Golder
Associates**

SUBJECT Non Drzg Off

V-DITCH

Job No. 873-11862
Ref.

Made by myg
Checked JSD
Reviewed RL

Date 11/13/89
Sheet 24 of 44 75
5

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)

R (YEARS)	AMOUNT (INCHES)							INTENSITY (INCHES PER HOUR)						
	TIME PERIOD							TIME PERIOD						
	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.

C.1-7

X1-8

WHC-SD-W025-FDR-001, Rev. 1

pg 7 of 75

**Golder
Associates**

SUBJECT NONDRAG OFF

V-DITCH

Job No. 843-1186.032

Made by

Checked

Reviewed

Date 11/13/89

Sheet 3 of 1475

NATURAL GROUND SLOPE / MAX RAINFALL INTENSITY

Approx Slope of natural ground from DWG H-2-99275

from EL. 720 to EL. 710 C.N. 44300 = $\frac{10'}{350'} = 2.9\%$ ✓

Assume overland flow velocity ≈ 2 fps ^{OK} _{NEARLY BARE GROUND}
 $3\% \rightarrow 1.7$ fps _{USDA} _{W/ SLOPE} _{W/ SLOPE} _{W/ SLOPE}

Distance from Western Edge of Basin ditch discharge ≈ 2000 ft.

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

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

from Climatological Summary for Hanford (Table 61)

the 20min precip (25year) = 0.47 in = 1.4 in/hr. ✓

∴ Use 1.5 in/hr. as design intensity ^{OK}

Revised low
if the future
catch to 110 mm

SUBJECT NON DRAG OFF - V DITCH DESIGN			
Job No. 883-1186	Made by [Signature]	Date 11/13/89	
Ref. .032	Checked JSU	Sheet 4 of 147-	
	Reviewed [Signature]	9	

Compute Query

RATIONAL FORMULA

$Q = CIA = \text{Rational formula}$ [applicable for $A < 200$ acres]

$$I = 1.5 \text{ in/hr}$$

$A = 17$ zeros

C = coefficient dependent on basin characteristics.

for 1st lawn^(2nd) on sandy soil $C = 0.05$ to 0.10

Assume sandy soil — slope is $\sim 29^\circ$

Assume C double since there is no lawn

$C = 0.20$ — OTHER LIMITATION

$$Q = CIA = (0.2)(1.5)(17) = 5.1 \text{ cfs}$$

$T_{A-SS} \approx$
 $CN = 80$
 $t_c = .16$ sec
 $Q_{P2} = 6$ cc ✓

0 Puow As 324 TE
00 Free Bot 1
QIN

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

W = 5.10 cfs
Bottom width = .10 ft
Side slope = 3.00:1
Channel bed slope = .02000
Mannings n = .0250
Depth - .67 ft
Velocity - 3.56 ft/s
Froude number - 1.07

Critical depth - .69 ft

Critical Slope = .0088

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

Critical depth - .69 ft

Critical Slope = .0138

Assumes:
1) Gravel Ditch System $\rightarrow n = .025$
2) CLEW $\rightarrow$ max. velocity - 4.0 FPS
3) Excessive Flow will not be
substantially reduced in F.D. Reduction

Velocity - 5.37 fps $\leftarrow$ too fast, keep slope less than 2.5% without
erosion protection or drops.

Critical depth - .69 ft

Critical Slope = .0088

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

Critical depth - .69 ft

Critical Slope = .0138

minimum slope.

maximum depth!

Check:

1) Suburban Flow
 $\rightarrow$ max 2.5%

OK

**Golder
Associates**

SUBJECT NON-DRAIN OFF

HYDROLOGY-Flood

Job No. 883-1186.032
Ref. 893Made by ML. Brown
Checked JSV
Reviewed JKLDate 11/22/89
Sheet 12 of 127
75

REGRESSION EQN's

REFERENCE 1 - Hydrology Studies within the COLUMBIA PLATEAU, WASH.
An Integration of Current Knowledge, Oct 1979
Rockwell International

PAGE III - 41

Ref 1 References Cummins et al 1975, USAS

COMPLETE 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 $A = 1.0$

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

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

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

**Golder
Associates**

SUBJECT NON DRAG OFF		FLOOD HYDROLOGY	
Job No. BB3-1186.032	Made by M.L. BROWN	Date 11/22/89	
Ref. 893	Checked JSU	Sheet 12 of 14	
	Reviewed REL	13 of 14 75	

$Q = 60 A^{0.87}$			
$\frac{A}{\text{ACRES}}$	$\frac{A}{\text{SQ MI}}$	$\frac{Q}{\text{CFS}}$	$\frac{\text{CFS}}{\text{mi}^2}$
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 ($< 1.8 \text{ mi}^2$)

IT DOES APPEAR TO BE A REASONABLE APPROXIMATION
[THIS INCLUDES SNOWMELT]

Using the rational method again —

$$\text{time of concentration} = \frac{L}{V}$$

L = basin hydraulic length
 V = average velocity

$$\text{estimate } L = 1200', V = 2 \text{ fps}$$

$$\frac{1200}{2} = 600 \text{ sec} = 10 \text{ min}$$

Max rainfall intensity for 10 min interval (from Fig 36, *Handbook*) $\approx 2.3 \text{ in/hr}$

$$\text{Estimate } C = 0.2 \quad \downarrow \text{10 acres unit? (as one unit?)}$$

$$Q = CIA = (0.2)(2.3)(10) = 4.6 \text{ cfs.} ✓$$

Using HEC1 with $CN = 65$ $Q = \phi$ because $CN65$ produces
no runoff with 1.56" precip storm. ✓

$$C_{10} = 85 \quad Q_{10} = 9 \text{ cfs.} \quad \text{QUERY CONSIDERATION?} ✓$$

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

172CHS

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

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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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

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

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IT

HYDROGRAPH TIME DATA

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NMHIN      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

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pg 15 of 75

ENGLISH UNITS

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*****
●          ●
●  25_year  ●
●          ●
●          ●
*****

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

```

7 BA SUBBASIN CHARACTERISTICS
TAREA .03 SUBBASIN AREA

9 PB STORM 1.56 BASIN TOTAL PRECIPITATION

[illegible]

35 UD SCS DIMENSIONLESS UNITGRAPH
TLAG .15 LAG

9572

UNIT HYDROGRAPH
10 END-OF-PERIOD ORDINATES
8. 4. 2.

TO END-OF-PERIOD COORDINATES

30. 65. 44. 18. 8. 4. 2. 1. 0. 0.

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	.00	15 NOV 0300	151	.00	.00	.00	.00
14 NOV 1205	2	.00	.00	.00	.00	15 NOV 0305	152	.00	.00	.00	.00
14 NOV 1210	3	.00	.00	.00	.00	15 NOV 0310	153	.00	.00	.00	.00
14 NOV 1215	4	.00	.00	.00	.00	15 NOV 0315	154	.00	.00	.00	.00
14 NOV 1220	5	.00	.00	.00	.00	15 NOV 0320	155	.00	.00	.00	.00
14 NOV 1225	6	.00	.00	.00	.00	15 NOV 0325	156	.00	.00	.00	.00
14 NOV 1230	7	.00	.00	.00	.00	15 NOV 0330	157	.00	.00	.00	.00
14 NOV 1235	8	.00	.00	.00	.00	15 NOV 0335	158	.00	.00	.00	.00
14 NOV 1240	9	.00	.00	.00	.00	15 NOV 0340	159	.00	.00	.00	.00
14 NOV 1245	10	.00	.00	.00	.00	15 NOV 0345	160	.00	.00	.00	.00
14 NOV 1250	11	.00	.00	.00	.00	15 NOV 0350	161	.00	.00	.00	.00
14 NOV 1300	12	.00	.00	.00	.00	15 NOV 0400	162	.00	.00	.00	.00
14 NOV 1305	13	.00	.00	.00	.00	15 NOV 0405	163	.00	.00	.00	.00
14 NOV 1310	14	.00	.00	.00	.00	15 NOV 0410	164	.00	.00	.00	.00
14 NOV 1315	15	.00	.00	.00	.00	15 NOV 0415	165	.00	.00	.00	.00
14 NOV 1320	16	.00	.00	.00	.00	15 NOV 0420	166	.00	.00	.00	.00
14 NOV 1325	17	.00	.00	.00	.00	15 NOV 0425	167	.00	.00	.00	.00
14 NOV 1330	18	.00	.00	.00	.00	15 NOV 0430	168	.00	.00	.00	.00
14 NOV 1335	19	.00	.00	.00	.00	15 NOV 0435	169	.00	.00	.00	.00
14 NOV 1340	20	.00	.00	.00	.00	15 NOV 0440	170	.00	.00	.00	.00
14 NOV 1345	21	.00	.00	.00	.00	15 NOV 0445	171	.00	.00	.00	.00
14 NOV 1350	22	.00	.00	.00	.00	15 NOV 0450	172	.00	.00	.00	.00
14 NOV 1355	23	.00	.00	.00	.00	15 NOV 0455	173	.00	.00	.00	.00
14 NOV 1400	24	.00	.00	.00	.00	15 NOV 0500	174	.00	.00	.00	.00
14 NOV 1405	25	.00	.00	.00	.00	15 NOV 0505	175	.00	.00	.00	.00
14 NOV 1410	26	.00	.00	.00	.00	15 NOV 0510	176	.00	.00	.00	.00
14 NOV 1415	27	.00	.00	.00	.00	15 NOV 0515	177	.00	.00	.00	.00
14 NOV 1420	28	.00	.00	.00	.00	15 NOV 0520	178	.00	.00	.00	.00
14 NOV 1425	29	.00	.00	.00	.00	15 NOV 0525	179	.00	.00	.00	.00
14 NOV 1430	30	.00	.00	.00	.00	15 NOV 0530	180	.00	.00	.00	.00
14 NOV 1435	31	.00	.00	.00	.00	15 NOV 0535	181	.00	.00	.00	.00
14 NOV 1440	32	.00	.00	.00	.00	15 NOV 0540	182	.00	.00	.00	.00
14 NOV 1445	33	.00	.00	.00	.00	15 NOV 0545	183	.00	.00	.00	.00
14 NOV 1450	34	.00	.00	.00	.00	15 NOV 0550	184	.00	.00	.00	.00
14 NOV 1455	35	.00	.00	.00	.00	15 NOV 0555	185	.00	.00	.00	.00
14 NOV 1500	36	.00	.00	.00	.00	15 NOV 0600	186	.00	.00	.00	.00
14 NOV 1505	37	.00	.00	.00	.00	15 NOV 0605	187	.00	.00	.00	.00
14 NOV 1510	38	.00	.00	.00	.00	15 NOV 0610	188	.00	.00	.00	.00
14 NOV 1515	39	.00	.00	.00	.00	15 NOV 0615	189	.00	.00	.00	.00
14 NOV 1520	40	.00	.00	.00	.00	15 NOV 0620	190	.00	.00	.00	.00
14 NOV 1525	41	.00	.00	.00	.00	15 NOV 0625	191	.00	.00	.00	.00
14 NOV 1530	42	.00	.00	.00	.00	15 NOV 0630	192	.00	.00	.00	.00
14 NOV 1535	43	.00	.00	.00	.00	15 NOV 0635	193	.00	.00	.00	.00
14 NOV 1540	44	.00	.00	.00	.00	15 NOV 0640	194	.00	.00	.00	.00
14 NOV 1545	45	.00	.00	.00	.00	15 NOV 0645	195	.00	.00	.00	.00
14 NOV 1550	46	.00	.00	.00	.00	15 NOV 0650	196	.00	.00	.00	.00
14 NOV 1555	47	.00	.00	.00	.00	15 NOV 0655	197	.00	.00	.00	.00
14 NOV 1600	48	.00	.00	.00	.00	15 NOV 0700	198	.00	.00	.00	.00
14 NOV 1605	49	.00	.00	.00	.00	15 NOV 0705	199	.00	.00	.00	.00
14 NOV 1610	50	.00	.00	.00	.00	15 NOV 0710	200	.00	.00	.00	.00
14 NOV 1615	51	.00	.00	.00	.00	15 NOV 0715	201	.00	.00	.00	.00
14 NOV 1620	52	.00	.00	.00	.00	15 NOV 0720	202	.00	.00	.00	.00
14 NOV 1625	53	.00	.00	.00	.00	15 NOV 0725	203	.00	.00	.00	.00
14 NOV 1630	54	.00	.00	.00	.00	15 NOV 0730	204	.00	.00	.00	.00
14 NOV 1635	55	.00	.00	.00	.00	15 NOV 0735	205	.00	.00	.00	.00
14 NOV 1640	56	.00	.00	.00	.00	15 NOV 0740	206	.00	.00	.00	.00
14 NOV 1645	57	.00	.00	.00	.00	15 NOV 0745	207	.00	.00	.00	.00
14 NOV 1650	58	.00	.00	.00	.00	15 NOV 0750	208	.00	.00	.00	.00
14 NOV 1655	59	.00	.00	.00	.00	15 NOV 0755	209	.00	.00	.00	.00
14 NOV 1700	60	.00	.00	.00	.00	15 NOV 0800	210	.00	.00	.00	.00
14 NOV 1705	61	.00	.00	.00	.00	15 NOV 0805	211	.00	.00	.00	.00
14 NOV 1710	62	.00	.00	.00	.00	15 NOV 0810	212	.00	.00	.00	.00
14 NOV 1715	63	.00	.00	.00	.00	15 NOV 0815	213	.00	.00	.00	.00
14 NOV 1720	64	.00	.00	.00	.00	15 NOV 0820	214	.00	.00	.00	.00
14 NOV 1725	65	.00	.00	.00	.00	15 NOV 0825	215	.00	.00	.00	.00
14 NOV 1730	66	.00	.00	.00	.00	15 NOV 0830	216	.00	.00	.00	.00
14 NOV 1735	67	.00	.00	.00	.00	15 NOV 0835	217	.00	.00	.00	.00
14 NOV 1740	68	.00	.00	.00	.00	15 NOV 0840	218	.00	.00	.00	.00
14 NOV 1745	69	.00	.00	.00	.00	15 NOV 0845	219	.00	.00	.00	.00
14 NOV 1750	70	.00	.00	.00	.00	15 NOV 0850	220	.00	.00	.00	.00
14 NOV 1755	71	.00	.00	.00	.00	15 NOV 0855	221	.00	.00	.00	.00
14 NOV 1800	72	.00	.00	.00	.00	15 NOV 0900	222	.00	.00	.00	.00
14 NOV 1805	73	.00	.00	.00	.00	15 NOV 0905	223	.00	.00	.00	.00
14 NOV 1810	74	.00	.00	.00	.00	15 NOV 0910	224	.00	.00	.00	.00
14 NOV 1815	75	.00	.00	.00	.00	15 NOV 0915	225	.00	.00	.00	.00
14 NOV 1820	76	.00	.00	.00	.00	15 NOV 0920	226	.00	.00	.00	.00
14 NOV 1825	77	.00	.00	.00	.00	15 NOV 0925	227	.00	.00	.00	.00
14 NOV 1830	78	.00	.00	.00	.00	15 NOV 0930	228	.00	.00	.00	.00

TIME	PEAK FLOW	(CFS)	(HR)	6-HR	24-HR	72-HR	29.90-HR	MAXIMUM AVERAGE FLOW	TOTAL RAINFALL = 1.56, TOTAL LOSS = 1.52, TOTAL EXCESS = .04
15 NOV 1948	79	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 1954	80	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2000	81	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2006	82	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2012	83	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2018	84	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2024	85	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2030	86	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2036	87	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2042	88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2048	89	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2054	90	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2100	91	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2106	92	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2112	93	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2118	94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2124	95	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2130	96	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2136	97	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2142	98	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2148	99	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2154	100	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2200	101	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2206	102	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2212	103	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2218	104	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2224	105	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2230	106	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2236	107	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2242	108	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2248	109	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2254	110	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2300	111	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2306	112	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2312	113	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2318	114	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2324	115	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2330	116	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2336	117	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2342	118	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2348	119	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 2354	120	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0000	121	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0006	122	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0012	123	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0018	124	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0024	125	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0030	126	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0036	127	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0042	128	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0048	129	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0054	130	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0100	131	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0106	132	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0112	133	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0118	134	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0124	135	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0130	136	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0136	137	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0142	138	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0148	139	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0154	140	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0200	141	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0206	142	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0212	143	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0218	144	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0224	145	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0230	146	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0236	147	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0242	148	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0248	149	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15 NOV 0254	150	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

CUMULATIVE AREA = .03 SQ MI

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

C.1-18

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)	6-HR (CFS)	24-HR (CFS)	72-HR (CFS)	29.90-HR (CFS)
9.	12.20	1.	0.	0.	0.
		(INCHES)	.400	.490	.490
		(AC-FT)	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	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
				6-HOUR	24-HOUR	72-HOUR	
HYDROGRAPH AT	25_year	9.	12.20	1.	0.	0.	.03

NORMAL END OF HEC-1 ***

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 zeros

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 85
 35 UD 0.150
 36 ZZ

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
 NDMTIME 1754 ENDING TIME

COMPUTATION INTERVAL .10 HOURS
 TOTAL TIME BASE 29.90 HOURS

C.1-20

ENGLISH UNITS

24-hour storm, $P = 1.56''$

TIME DATA FOR INPUT TIME SERIES

JXXIN	6	TIME INTERVAL IN MINUTES
JXDATE	14NOV89	STARTING DATE
JXTIME	1206	STARTING TIME

7 BA

TAREA .03 SUBBASIN AREA

9 PB

STORM 1.56 BASIN TOTAL PRECIPITATION

INCREMENTAL PRECIPITATION PATTERN

[illegible]

SCS LOSS RATE

STRTL	.35	INITIAL ABSTRACTION
CRVNR	25.00	CURVE NUMBER
RTIMP	.00	PERCENT IMPERVIOUS AREA

SDS DIMENSIONLESS UNITGRAPH

TLAG .15 LAG

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

**Golder
Associates**

SUBJECT <u>NON-DRAG-OFF / SNOW MELT</u>		
Job No. <u>89-1186</u>	Made by <u>M. BROWN</u>	Date <u>JAN 8 90</u>
Ref. <u>893-486.032</u>	Checked <u>JSV</u>	Sheet <u>23</u> of <u>25*</u>
	Reviewed <u>PRC</u>	

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% $\Rightarrow$ 1.2" WATER
Assume 32°F, frozen.

During peak 20 minutes precip rate = 1.5 in/hr $\Rightarrow$ 0.5 in total

Assume temp = 50°F.

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

$\therefore$ 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.

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

25 year $\Rightarrow$ 4% prob.

C.1-24

IV-14

**Golder
Associates**

SUBJECT NON-DRAIN-OFF

LEACHATE PRODUCTION

Job No. 893-1186.032

Made by M. BROWN

Date JAN 11 1990

Ref.

Checked JSV

Sheet 25 of 75

Reviewed

DL

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

Hamford Town Site Prep

Yehime 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 gal.

↳ 15057 gal

PEAK DAY - FROM RUN PEAK 5-2

Peak day 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 losses 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 losses 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 10² 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	EBASCO		GOLDER				
	NORMAL STORM		EBASCO- EBASCO- EBASCO- EBASCO- EBASCO-				
	PRECIP	PRECIP					
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. Conductivity (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

FILE HIST10-1

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

*****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
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29.60	37.60	46.80	53.30	61.90	69.20
75.30	75.10	64.70	53.30	39.30	29.90

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	0.940	0.565	0.194	0.386	0.323	0.256
(INCHES)	0.090	0.160	0.215	0.217	0.590	0.451
LATERAL DRAINAGE FROM	0.3620	0.1417	0.0436	0.0233	0.0164	0.0120
LAYER 2 (INCHES)	0.0098	0.0081	0.0067	0.0060	0.1069	0.5023
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	0.01	0.00	0.00	0.00	0.00	0.00
LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	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	0.00	0.00

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

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

STD. 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	0.00	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

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

MONTHLY TOTALS FOR YEAR 81

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.56 0.19	0.60 0.03	0.70 0.60	0.02 0.39	0.99 1.08	0.43 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 0.269	1.175 0.247	0.315 0.126	0.405 0.217	0.205 0.577	0.579 0.475
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6925 0.0870	0.1097 0.0738	0.0409 0.0388	0.0224 0.0230	0.0160 0.3858	0.0199 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 0.01	0.00 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 0.01	0.00 0.00

ANNUAL TOTALS FOR YEAR 81

(INCHES)	(CU. FT.)	PERCENT
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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

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
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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

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

MONTHLY TOTALS FOR YEAR 83

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

LATERAL DRAINAGE FROM	0.6846	0.3034	0.1557	0.0463	0.0254	0.0165
LAYER 2 (INCHES)	0.0127	0.0100	0.0080	0.0071	0.7179	1.2227
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	0.01	0.00	0.00	0.00	0.00	0.00
LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.01	0.02
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

ANNUAL TOTALS FOR YEAR 83

	(INCHES)	(CU. FT.)	PERCENT
PRECIPITATION	11.07	136530.	100.00
UNOFF	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

MAY ANNUAL.

MONTHLY TOTALS FOR YEAR 84

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

PRECIPITATION (INCHES)	0.23 0.06	0.94 0.00	1.01 0.42	0.60 0.07	0.55 1.83	0.99 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 0.296	1.351 0.258	1.022 0.252	0.497 0.228	0.441 0.400	0.564 0.665
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6849 0.0097	0.1184 0.0081	0.0414 0.0070	0.0226 0.0064	0.0161 0.0142	0.0118 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

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

MONTHLY TOTALS FOR YEAR 85

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.34 0.12	0.82 0.01	0.36 0.63	0.01 0.46	0.12 1.24	0.15 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 0.011	0.958 0.119	0.331 0.208	0.038 0.213	0.120 0.221	0.150 0.697
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.1440 0.0091	0.0589 0.0076	0.0330 0.0063	0.0196 0.0057	0.0145 0.1731	0.0108 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 0.01	0.00 0.01

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

MONTHLY TOTALS FOR YEAR 86

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

PRECIPITATION (INCHES)	1.76 0.21	1.21 0.02	0.76 0.96	0.00 0.29	0.30 0.65	0.00 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 0.210	1.695 0.020	0.854 0.152	0.071 0.215	0.300 0.201	0.000 0.469
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.2029 0.0120	0.6176 0.0095	0.1470 0.0077	0.0384 0.0072	0.0228 0.0196	0.0152 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 0.00	0.01 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

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

MONTHLY TOTALS FOR YEAR 87

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

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

MONTHLY TOTALS FOR YEAR 88

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

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

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	<u>12222.</u>	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

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 0.18	0.75 0.09	0.66 0.51	0.44 0.41	0.50 1.09	0.42 1.25
STD. DEVIATIONS	0.53 0.14	0.46 0.12	0.29 0.28	0.40 0.40	0.41 0.57	0.40 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 0.272	1.051 0.184	0.735 0.178	0.319 0.224	0.460 0.486	0.331 0.473
STD. DEVIATIONS	0.113 0.161	0.396 0.089	0.515 0.071	0.170 0.105	0.439 0.367	0.202 0.155
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.4710 0.0258	0.2212 0.0200	0.0711 0.0142	0.0281 0.0179	0.0221 0.1923	0.0255 0.5671
STD. DEVIATIONS	0.2258 0.0310	0.1836 0.0222	0.0483 0.0116	0.0091 0.0262	0.0118 0.2476	0.0346 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

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

↳ = 154 620 gallons

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 = 15063 gallons
PERCOLATION FROM LAYER 3	0.0000	0.0
HEAD ON LAYER 3	0.1	
SNOW WATER	0.76	9431.8
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1800	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0630	

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

FILE: PEAK5-2

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

*****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

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 0.458	1.380 0.091	0.667 0.219	0.410 0.279	1.637 0.247	0.629 0.517
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	1.1470 0.0176	0.2496 0.0130	0.0509 0.0099	0.0254 0.0109	0.0181 0.0261	0.0256 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

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

MONTHLY TOTALS FOR YEAR 81

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.56 0.19	0.60 0.03	0.70 0.60	0.02 0.39	0.99 1.08	0.43 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 0.273	1.398 0.250	0.325 0.106	0.394 0.219	0.210 0.648	0.671 0.398
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.5017 0.0455	0.1228 0.0525	0.0473 0.0317	0.0244 0.0202	0.0170 0.3824	0.0155 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 0.01	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 0.01	0.00 0.00

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

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)	0.916 0.482	0.720 0.182	0.242 0.263	0.559 0.323	0.331 0.513	0.753 0.405
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.6428 0.0096	0.1052 0.0079	0.0390 0.0065	0.0218 0.0062	0.0157 0.4399	0.0115 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 0.01	0.00 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

ANNUAL TOTALS FOR YEAR 82

(INCHES)	(CU. FT.)	PERCENT
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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

MONTHLY TOTALS FOR YEAR 83

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	1.44 0.31	1.36 0.12	1.00 0.46	0.42 0.52	0.52 2.12	0.68 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 0.307	1.260 0.279	1.650 0.236	0.450 0.255	0.376 0.838	0.349 0.428
LATERAL DRAINAGE FROM LAYER 2 (INCHES)	0.7502 0.0131	0.3449 0.0103	0.1805 0.0082	0.0508 0.0072	0.0268 0.6822	0.0171 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 0.00	0.01 0.00	0.00 0.00	0.00 0.00	0.00 0.01	0.00 0.07
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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

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.	
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

MONTHLY TOTALS FOR YEAR 84

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION (INCHES)	0.23 0.06	0.94 0.00	1.01 0.42	0.60 0.07	0.55 1.83	0.99 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)	0.788 0.389	1.186 0.000	0.974 0.171	0.547 0.312	0.444 0.314	0.869 0.244

LATERAL DRAINAGE FROM	1.0859	0.1051	0.0379	0.0214	0.0155	0.0114
LAYER 2 (INCHES)	0.0095	0.0079	0.0065	0.0058	0.0073	0.5297
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	0.02	0.00	0.00	0.00	0.00	0.00
LAYER 3 (INCHES)	0.00	0.00	0.00	0.00	0.00	0.01
STD. DEV. OF DAILY HEAD	0.03	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

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 0.16	0.95 0.07	0.66 0.58	0.53 0.54	0.75 1.28	0.76 1.88
STD. DEVIATIONS	0.56 0.13	0.37 0.08	0.35 0.17	0.33 0.49	0.46 0.69	0.23 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 0.382	1.189 0.161	0.772 0.199	0.472 0.277	0.599 0.512	0.654 0.398
STD. DEVIATIONS	0.152 0.091	0.276 0.115	0.571 0.062	0.077 0.043	0.586 0.242	0.194 0.098
LATERAL DRAINAGE FROM LAYER 2						

TOTALS	0.8255 0.0191	0.1855 0.0183	0.0711 0.0126	0.0288 0.0101	0.0186 0.3076	0.0162 1.0968
STD. DEVIATIONS	0.2807 0.0152	0.1077 0.0192	0.0614 0.0108	0.0125 0.0060	0.0047 0.2884	0.0058 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

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. ²⁴⁰⁸⁰⁰ 29.19

PERCOLATION FROM LAYER 3 0.0000 (0.0000) 0. 0.00

CHANGE IN WATER STORAGE -0.042 (0.612) -522. -0.47

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	

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	

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

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

pg 57 of 75

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

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 0.16	0.95 0.07	0.66 0.58	0.53 0.54	0.75 1.28	0.76 1.56
STD. DEVIATIONS	0.56 0.13	0.37 0.08	0.35 0.17	0.33 0.49	0.46 0.69	0.23 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 0.468	1.197 0.186	0.797 0.193	0.480 0.303	0.609 0.659	0.650 0.434
STD. DEVIATIONS	0.119 0.188	0.192 0.171	0.583 0.089	0.114 0.088	0.595 0.422	0.321 0.136
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.7553 0.0413	0.3269 0.0352	0.1588 0.0277	0.0827 0.0245	0.0575 0.0684	0.0435 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

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

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	

FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)
-------	----------	-----------

3

16.20

0.4500

SNOW WATER

0.00

-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

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

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 0.16	0.95 0.07	0.66 0.58	0.53 0.66	0.75 1.28	0.76 1.56
STD. DEVIATIONS	0.56 0.13	0.37 0.08	0.35 0.17	0.33 0.77	0.46 0.69	0.23 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 0.468	1.197 0.186	0.797 0.193	0.480 0.303	0.609 0.659	0.649 0.434
STD. DEVIATIONS	0.119 0.188	0.192 0.171	0.583 0.089	0.114 0.088	0.595 0.422	0.321 0.136
LATERAL DRAINAGE FROM LAYER 2						
TOTALS	0.7619 0.0413	0.3270 0.0352	0.1588 0.0277	0.0827 0.0307	0.0575 0.1477	0.0435 0.4957
STD. DEVIATIONS	0.1709	0.1629	0.0651	0.0214	0.0104	0.0066

Fig 62 of T2

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

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

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	

FINAL WATER STORAGE AT END OF YEAR 84

LAYER	(INCHES)	(VOL/VOL)
-------	----------	-----------

3

16.20

0.4500

SNOW WATER

0.00

 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

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

pg 65 of 75

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

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 0.16	0.95 0.07	0.66 0.58	0.53 0.54	0.75 1.28	0.76 1.88
STD. DEVIATIONS	0.56 0.13	0.37 0.08	0.35 0.17	0.33 0.49	0.46 0.69	0.23 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

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

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

MAXIMUM VEG. SOIL WATER (VOL/VOL) 0.1927

MINIMUM VEG. SOIL WATER (VOL/VOL) 0.0630

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	

 EBASCO-4; WHC NON-DRAW 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

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

pg 69 of 75

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

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 0.16	0.95 0.07	0.66 0.58	0.53 0.54	0.75 1.28	0.76 1.88
STD. DEVIATIONS	0.56 0.13	0.37 0.08	0.35 0.17	0.33 0.49	0.46 0.69	0.23 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

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

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

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	

Pg 72 of 75

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

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

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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

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SOIL WATER CONTENT INITIALIZED BY PROGRAM.

CLIMATOLOGICAL DATA

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

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

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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

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 0.16	0.95 0.07	0.66 0.58	0.53 0.54	0.75 1.28	0.76 1.88
STD. DEVIATIONS	0.56 0.13	0.37 0.08	0.35 0.17	0.33 0.49	0.46 0.69	0.23 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

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

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 4Project Title: NON DRAG OFFProject No. 893-1186Calculation Title: LEACHATE TANK SECONDARY CONTAINMENT SYSTEM
B.24

Name

Date

Prepared By: M. BROWN 1/22/90Checked By: Frank S. Shari 2-7-90Approved By: [Signature] 2-8-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

**Golder
Associates**

SUBJECT NDO

Tank Sump

Job No. 843-1182
Ref.

Made by M. BRANN
Checked FS
Reviewed FS

Date 1/22/90
Sheet 1 of 4

OBJECTIVE:

SIZE SUMP

CRITERIA: 1.0 tank Volume + Avg Annual Precip.

From Spec. 2. Approx. Tank dimensions = 8'0" x 25' long $\Rightarrow$ Vol. = $\frac{\pi D^2}{4} \times L = 1256 \text{ ft}^3$

Tank Volume = 10 000 gal. = 1337 ft^3 ✓

Area of bottom of Sump [from dwg., attached]

Interior 12' wide x 35' long = 420 ft^2 ✓

depth to hold 1 tank volume = $\frac{1337 \text{ ft}^3}{420 \text{ ft}^2} = 3.18 \text{ ft.}$

Avg. 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. ✓

LATE TEMPORARY STORAGE TANK & PIPE PLAN

0 5 10
SCALE IN FEET

H-2-1315805

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	0.44	6.41
1913	1.39	0.16	0.05	0.16	0.61	1.11	0	1	0.03	1.42	0.75	0.76	6.44
1914	1.12	1.15	1	0.54	0.60	0.29	0.73	0	0.32	1.75	0.58	0.74	7.82
1915	0.65	0.61	0.51	0.39	0.90	0.76	0.71	1	1	0.20	1.56	1.18	6.96
1916	0.69	1.54	0.55	0.77	0.01	---	---	0	0.02	0	1.70	0.87	6.46
1917	1.00	0.78	0.31	0.72	0.57	0.09	1	0	0.07	0	0.81	0.50	4.35
1918	0.63	0.50	0.72	0	0.34	0.48	0.28	0.36	0.15	0.54	0.69	0.49	4.90
1919	1.04	1.09	0.38	0.32	0.64	0	1	1	0.74	0.71	0.88	1.00	6.30
1920	1.44	1	1	0.78	0.05	0.69	1	1.18	0.75	0.51	0.99	1.72	7.63
1921	0.36	1.25	1	0.60	0.33	0.12	1	0.34	0.06	0.79	1.61	0.49	5.45
1922	0.37	0.45	0.76	0.14	1	0.65	0	0.36	0.76	0.72	0.28	1.05	4.42
1923	0.37	0.40	0.05	0.05	0.77	1.11	0.33	1	0.43	1.00	0.64	0.54	5.44
1924	1.02	0.78	0.10	0.01	1	0.14	0.37	0.17	0.45	0.77	1.34	1.16	6.71
1925	0.44	0.70	0.04	0.60	0.67	0.34	0	1	0.49	0.07	0.96	0.44	5.61
1926	0.93	0.57	0.08	0.04	0.76	0.00	0	0.10	0.30	0.40	1.05	0.78	7.31
1927	1.79	0.77	0.45	0.05	0.09	1	1	0.29	1.14	0.62	1.76	0.40	6.34
1928	1.34	0.09	0.63	0.71	0.17	0.08	0.15	0.05	0.01	0.50	0.49	1.33	5.58
1929	1.37	0.07	0.07	0.05	0.25	0.19	1	0.07	0.15	0.87	1	0.04	3.78
1930	0.54	0.78	0.14	0.34	0.43	0.09	1	0.04	0.40	0.04	0.32	0.30	3.34
1931	1.35	0.17	0.99	0.11	0	0.51	0.19	0	0.16	0.47	1.15	2.32	7.63
1932	1.52	0.15	0.63	0.96	0.29	0.06	0.03	0.10	1	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	1.70	3.96
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	0.06	0.08	0.13	0.37	0.45	0	0.05	0.39	0.50	0.87	3.42
1936	1.54	0.65	0	0.15	0.33	0.50	0.04	0.01	0.78	0.04	1	0.06	4.74
1937	0.00	0.63	0.76	0.73	0.01	1.09	0.74	0.06	0.16	0.39	1.04	1.08	7.79
1938	0.35	0.77	1.15	0.18	0.11	1.78	0.04	0.14	0.02	0.79	0.67	0.71	6.73
1939	0.59	0.00	0.57	1	0.07	0.07	0	0.25	0.11	0.25	1	1.04	4.36
1940	0.04	1.18	0.47	0.76	0.78	0.11	0.34	0	0.00	2.05	1.45	1.57	11.76
1941	1.82	0.65	0.16	0.31	0.83	1.16	0.04	0.59	0.57	0.41	0.46	0.40	7.44
1942	0.46	0.47	0	0.70	1.02	1.78	0.02	0	1	0.54	1.34	1.19	6.32
1943	0.40	0.37	0.33	---	---	---	---	---	---	---	---	0.73	11.33
1944	0.39	0.56	0.83	0.77	---	---	---	---	---	---	---	---	11.75
1945	---	---	---	---	---	---	---	---	---	---	---	---	---
1946	---	---	---	---	---	---	0.15	0.35	0.52	0.45	0.44	0.11	12.44
1947	0.32	0.77	0.42	0.70	0.02	1.02	0.71	0.46	1.34	2.70	0.81	0.75	9.79
1948	1.34	0.69	0.07	0.95	1.71	1.47	0.40	0.39	0.16	0.45	0.95	1.15	9.71
1949	0.13	0.68	1.12	0.02	0.16	0.01	0.01	0.03	0.23	0.10	1.47	0.16	4.12
1950	2.00	2.06	0.87	0.47	0.77	2.72	0.87	1	0.01	2.46	0.55	0.97	13.45
1951	0.04	0.51	0.46	0.53	0.43	1.38	0.37	0.15	0.10	0.71	0.82	0.70	7.00
1952	0.65	0.50	0.06	0.13	0.58	1.07	1	0.08	0.04	0.04	0.20	0.77	4.16
1953	2.16	0.25	0.17	0.77	0.78	0.55	1	0.96	0.13	0.70	0.96	0.49	6.92
1954	1.48	0.78	0.59	0.07	0.41	0.18	0.22	0.47	0.51	0.47	0.84	0.35	3.71
1955	0.56	0.27	0.17	0.40	0.59	0.78	0.57	0	0.77	0.40	1.54	2.00	7.53
1956	1.71	0.54	1.10	1	0.77	0.64	1	0.34	0.04	1.05	0.15	0.54	5.60
1957	0.48	0.73	1.15	0.34	0.87	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	1	0.05	0.19	0.77	1.04	8.74
1959	2.05	1.17	0.40	0.70	0.50	0.73	1	0.00	1.76	0.54	0.41	0.76	7.07
1960	0.51	0.58	0.67	0.53	0.71	0.14	1	0.76	0.23	0.23	0.92	0.44	5.42
1961	0.33	2.10	1.02	0.48	0.00	0.42	0.15	0.09	1	0.07	0.49	0.06	6.23
1962	0.13	0.90	0.14	0.34	1.35	0.12	1	0.50	0.36	0.95	0.62	0.60	6.03
1963	0.95	0.69	0.53	1.12	0.43	0.78	0.31	0.01	0.02	0.04	0.74	1.14	6.31
1964	0.37	0.04	0.05	0.11	0.04	0.90	0.04	0.34	0.09	0.78	0.94	2.34	5.39
1965	0.93	0.14	0.03	0.09	0.15	0.49	0.11	0.05	0.11	0.04	1.17	0.39	3.65
1966	0.68	0.05	0.39	0.03	0.05	0.43	0.11	1	0.77	0.39	2.25	0.60	5.93
1967	0.32	1	0.14	0.90	0.56	0.57	1	1	0.05	0.13	0.16	0.43	3.76
1968	0.88	0.58	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	0.54	0.10	1.22	0.51	0.75	1	1	0.48	0.10	0.13	1.79	6.36
1970	2.51	0.75	0.27	0.43	0.54	0.25	0.01	1	0.05	0.24	0.71	0.61	6.33
1971	0.78	0.10	1.02	0.07	0.56	0.71	0.13	0.09	1.13	0.18	0.46	1.07	6.20
1972	0.10	0.77	0.38	0.10	2.03	0.66	0.14	0.56	0.83	1	0.55	1.77	6.79
1973	0.90	0.71	0.08	1	0.74	0.01	1	0.62	0.43	1.72	2.44	2.02	8.77
1974	0.90	0.41	0.52	0.46	0.78	0.12	0.71	1	0.01	0.71	0.71	0.97	5.30
1975	1.43	0.98	0.33	0.42	0.36	0.74	0.32	1.14	0.02	0.57	0.60	0.70	7.46
1976	0.56	0.36	0.73	0.41	0.06	0.11	0.13	0.96	1	0.04	1	0.11	2.92
1977	0.08	0.57	0.41	1	0.45	0.37	0.06	1.34	0.44	0.15	0.63	1.47	6.41
1978	1.72	0.52	0.30	0.46	0.41	0.99	0.32	0.11	0.11	1	1.71	0.79	6.57
1979	0.54	0.17	0.54	0.52	0.10	1	0.09	0.34	0.70	0.67	1.36	0.99	5.56
1980	1.32	1.30	0.30	0.66	1.41	0.96	1	0.02	0.85	0.33	0.44	1.09	9.68
AVG	0.92	0.60	0.37	0.39	0.46	0.54	0.15	0.74	0.31	0.56	0.85	0.99	6.30
*NORMAL	0.97	0.56	0.40	0.39	0.53	0.45	0.16	0.79	0.29	0.46	0.80	0.95	6.75

THE HIGHEST, AND IF MEASURABLE, THE LOWEST TOTALS IN EACH COLUMN ARE UNDERSCORED

T DEMOTES A TRACE (0.005 INCH)

F? DEMOTES AN INCOMPLETE ANNUAL TOTAL

* NORMALS ARE AVERAGES FOR THE PERIOD 1951-1980

--- DEMOTES MISSING DATA

← Ave. Annual.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 20Project Title: WHC Non Drag OffProject No. 893-1186Calculation Title: Primary Leachate Collection System
B.12

Name

Date

Prepared By: M. Brown 1/22/90Checked By: Frank S. Sherris 2-7-90Approved By: Frank S. Sherris 2-8-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

CALCULATION B.12

Golder
AssociatesSUBJECT MOO PRIMARY LEACHATE COLLECTION SYSTEMJob No. 843-1186Made by M. BROWNDate 1/22/90

Ref.

Checked FSSSheet 2 of 20Reviewed RELOBJECTIVES

Select capacities of drainage elements

MethodTypical Section FIRST COMPUTE THE TIME OF CONCENTRATION T_c AT BOTH THE TOE OF SLOPE AND THE DRAIN PIPE.

Travel time through operation layer

$$Q = k_i A$$

$$\frac{Q}{A} = k_i = \text{apparent velocity}$$

$$\text{actual velocity} = \frac{k_i}{e} = \frac{10^{-2} \text{ cm/sec} \times 1}{0.38} = 0.0263 \text{ cm/sec}$$

effective porosity, see attached 1A

$$\text{Distance} = 24" \times 2.54 \text{ cm/in} = 60.96 \text{ cm}$$

$$\text{Time} = \frac{60.96}{0.0263 \text{ cm/sec}} = 38.6 \text{ minutes}$$

$$\text{Travel time in pond} \sim \frac{\text{distance}}{\text{velocity}} = \frac{75'}{0.6 \text{ fps}} = 125 \text{ sec} \sim 2 \text{ min.}$$

See attached sheet 1B

Author's First Draft 10/21/88

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. Sjoström
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 ~~1A~~ 4 of 2

Figure 6. General relation between soil water properties and soil texture (Rauis et al., 1982)

FROM "HELP" MODEL DOCUMENTATION

40

sheet 5 of 20

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**Golder
Associates**

SUBJECT <i>NDO Primary</i>		
Job No. <i>873-1186</i>	Made by <i>M. BROWN</i>	Date <i>1/28-70</i>
Ref.	Checked <i>FSS</i>	Sheet <i>18</i> of <i>20</i>
	Reviewed <i>RL</i>	<i>6</i>

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 (6×0.33)

$T = 2 \text{ gpm/ft}$ (Perimeter $T = 7 \text{ gpm/ft}$) $\checkmark$
for our calculation in feet + seconds: $T = 0.0045 \text{ cfs./ft}$

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 Leach**

Job No.

Made by

Date

Ref.

Checked **FSS**

Sheet

Reviewed **KL**

21 of **20**

7

Time of concentration at toe of slope = $38.6 + 2 = 40.6 \text{ min.}$

Max. Rainfall intensity (Fig 3.6 of Hanford Climatological Data)

[ATTACHED]

for $T = 40.6 \text{ min} \approx 41 \text{ min}$

$i = 0.3 \text{ in/hour}$

for 1 acre $Q = 0.3 \text{ cfs} = 358 \text{ gpm}$

for 1 1/2 mile strip, 75 ft long

$Q_{\text{TOE}} = \frac{75}{43560} \times 358 \text{ gpm} = 0.62 \text{ gpm}$

flow at center of facility $Q_c = \frac{75+30}{43560} \times 358 = 7.02 \text{ gpm}$

Actual Q is slightly lower due to increased T_c and resulting reduced rainfall intensity. conservatively use 358.

Check capacity of gutter

See attached page 2A.

From manufacturer info $\text{transm} \approx 69 \text{ gpm/ft} \quad (i = 1.0)$

for $i = 0.025$ $T = 0.15 \text{ gpm/ft} < 0.62 \text{ gpm}$ NOT OK on Floor

on slope

$i = 0.33$

$T = 6 \times 0.33 = 2 \text{ gpm/ft} > 0.62 \text{ gpm}$

$\therefore$ OK on slope

USE AN ADDITIONAL HDPE PERF PIPE AT TOE OF SLOPE. ✓

Golder
Associates

SUBJECT

NOO

PRIMARY

LOADING

JOB NO.

MADE BY M. FERNAN

DATE 1-30-90

REF.

CHECKED FSS

SHEET

24 OF 20

REVIEWED

24

The tex-net which we propose to use is

TN1001 or equal.

The transmissivity

for TN1001 = 0.0015 m/sec

See attached Igam fabric report.

However, this

will be specified as an "or equal"

Assume Transmissivity of final product is

one step lower - similar to TN3001, $T = 0.0012 \text{ m/sec}$

$$T = 0.0012 \frac{\text{m}}{\text{sec}} \times (3.28 \frac{\text{ft}}{\text{m}})^2 \times \frac{449 \text{ gpm}}{\text{ft}^3/\text{sec}} = 5.8 \text{ gpm/ft}$$

° Transmissivity of geonet $\approx 6.0 \text{ gpm/ft}$

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.

ICI Geotext range of 1 - 1.0
ICI Geotext of .25

1A) 48 Lbs - 6.9 psi
1B) Available on back order

Deck Drain	polypropylene	1.2/0.0 (4/10)	9.6 (C80)	958 (139)	Amoco 1196	1.5-4.8 (7-23) ^F	1.2-4.6 (6-22) ^F
Sheet Drain	polypropylene	1.2/0.0 (4/10)	9.6 (C80)	717 (104)	Amoco 9140 NP	1.2-3.9 (6-19) ^F	1.0-3.7 (5-18) ^F
Sheet Drain HS	polypropylene	1.2/0.0 (4/10)	5.2 (C10)	1199 (174)	Amoco 9140 NP	2-1.1 (1.2-5.2) ^F	2-1.0 (1.1-4.9) ^F

Greenstreak Inc.

TEX-NET T1001	polypropylene	(7.5/20)	5.7 (C25)	>900 (130)	Tierras 1120 ^F	.0015	
TEX-NET T1001	polypropylene	(7.5/20)	5.1 (C20)	>900 (130)	Tierras 1120 ^F	.0012	
TEX-NET T1001CN	polypropylene	(7.5/20)	5.1 (C20)	>900 (130)	Tierras 1120 ^F	.0010	
TEX-NET T14001	polypropylene	(7.5/20)	6.9 (C70)	>900 (130)	Tierras 1120 ^F	.0025	

Fluid Systems Inc.

LDC-4000	polypropylene	1.2/0.5 (4/10)	15.2 (400)	280 (40)	various	7.4 (25.3) ^F	6.6 (21.9) ^F
X34110	polypropylene	2.1/1.4 (8.9/200)	6.1 (C40)	>900 (130)	various	3.4 (11.4) ^F	3.0 (10.5) ^F
X34210	polypropylene	2.1/1.4 (8.9/200)	4.1 (160)	>900 (130)	various	1.9 (9.2) ^F	1.8 (8.7) ^F
X34310	polypropylene	2.1/1.4 (8.9/200)	4.8 (190)	>900 (130)	various	2.7 (13.0) ^F	2.6 (12.6) ^F
X34350N	polypropylene	2.1/1.4 (8.9/200)	5.1 (C20)	>900 (130)	various	2.6 (12.6) ^F	2.6 (12.6) ^F
X34410	polypropylene	2.1/1.4 (8.9/200)	7.6 (C60)	>900 (130)	various	4.3 (20.8) ^F	4.2 (20.8) ^F

Corwood Plastics

C-112	polypropylene	1.6/1.6 (3.0/5.0)	5.6 (C20)	no	Tierras 1112	no	no
C-114	polypropylene	1.6/1.6 (3.0/5.0)	5.6 (C20)	no	Tierras 1114	no	no
C-212	polypropylene	1.6/1.6 (3.0/5.0)	5.6 (C20)	no	Tierras 1112	no	no
C-214	polypropylene	1.6/1.6 (3.0/5.0)	5.6 (C20)	no	Tierras 1114	no	no
STURDRAIN 100	HDPE	20.44-61.78 (11.3-2.25)	25.4 (1000)	379 (55)	Tierras 1114	5.11 (24.7)	4.66 (22.5)
STURDRAIN 25	HDPE	55.1-195 (1.3-4.18)	19.1 (750)	276 (40)	Tierras 1114	5.2 (21)	4.1 (21)

Contech Construction Products Inc.

Product Name	Structure	Polymer Composition	Width Length (ft) - (m)	Thickness (mm) (in)	Crush Strength ASTM D1621 kPa (psi)	Geotextile Product	Permeability
						ASTM D7178-87M	
						10 kPa (1.45 psi)	100 kPa (14.5 psi)
						10 kPa (1.45 psi)	100 kPa (14.5 psi)

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

Goldier Associates

SUBJECT: N00		JOB NO. 893-112		MADE BY M. J. J. J.		DATE 1/28/90	
REVIEWED		CHECKED		REVIEWED		SHEET 11 OF 20	

CHECK DRAIN PIPE CAPACITY.

FIRST - ESTIMATE PEAK FLOW WITH FLOODING EVAPORATION

Estimate travel time to most distant point.

Flow South West corner. - Thin ops layer 38.6 min

140 ft on slope - velocity = 0.6 fps [page 18]

Travel time = 140 ft / 0.6 fps =

By experience assume velocity in pipe is about 5 fps.

Length of pipe at 1/2 slope = 320'

T = 4.3 min

Line of construction is 4.6 min = T.

Intensity = 0.8
from rainfall intensity graph pg. 26

Inflowing Area = See Attached plan view

Truck Staging

A
B
C
D
E

100' x 100' = 10000 sq ft

130' x 90' = 11700

25' x 25' = 625

110' x 210' = 23100

90' x 40' = 3600

1/2 110' x 110' = 6050

TOTAL

55075 sq ft = 1.26 MGAL

PEAK FLOW ESTIMATE FORMULA

Q = CIA where C = 1

no losses

$Q = (1)(0.8 \text{ in/hr})(176,000 \text{ sq ft}) = 140,800 \text{ cfs}$

454 gpm

12/F20

WHC-SD-W025-FDR-001
Rev. 0
Page 213 of 297

Scale 1" = 50'

SCALE 1"=50'

Tank Area

Control Building

Truck Unload Area

Sump

R 30 FT

(11P)

**Golder
Associates**

SUBJECT **NDO**

Job No. **897-1186**
Ref.

Made by **M. Brown**
Checked **TSD**
Reviewed **RA**

Date **1/28/10**
Sheet **4** of **20**
13

Check capacity of Pipe -

USE Either 6" SDR 11 or 4" SDR 11

Using Hazen Williams per Manufacturers info (attached)

$$C = 150$$

$$Q = 1.008 \text{ cfs} \times \frac{448 \text{ gpm}}{\text{cfs}} = 452$$

$$4" \text{ Id} = 3.68"$$

$$\Delta P = \frac{452 \times (1.48)^{1.85}}{150^{1.85} \times 3.68^{4.75}} = 6.1 \text{ psi/100'}$$

$$\frac{6.1 \text{ psi}}{100'} \times \frac{1}{0.4375} = 0.141 \text{ friction slope}$$

$$6" \text{ Id} = 5.42"$$

$$\Delta P = \frac{452 \times (1.48)^{1.85}}{150^{1.85} \times 5.42^{4.75}} = 0.93 \text{ psi/100'}$$

$$\Delta P = 0.0216 \text{ friction slope}$$

ACTUAL PIPE SLOPE IS 0.025

∴ 4" pipe will cause backwater effects, 6" 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.

$$Q_{fr} = 0.463$$

D = pipe ϕ

$$Q = \frac{0.463 (0.025)^{1/2} \left(\frac{3.68}{12} \right)^{4/3}}{0.009} = 0.3479 \text{ cfs} = 155 \text{ gpm}$$

There are 3 pipes flowing into primary sump - each with a capacity of 155 gpm. Sump contains ^{pumps with} combined discharge capacity of 155 gpm (150+5).

**Golder
Associates**

SUBJECT <u>NDO Primary Leachate Collection</u>		
Job No. <u>893-1186</u>	Made by <u>M. Brown</u>	Date <u>1/30/90</u>
Ref.	Checked <u>FSS</u>	Sheet <u>4A</u> of <u>20</u>
	Reviewed <u>DE</u>	<u>14</u>

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/op's layer) while it is flowing full. When pump is running the maximum avg discharge per pipe is $155/3 = 52 \text{ gpm}$.

Capacity from previous page is $155 \text{ gpm} > 52 \text{ gpm} \therefore \underline{\text{OK}}$

Backwater effects in pipe acceptable.

From USBR
Hydraulic + Excavation Tables
Eleventh Edition 1957

106 HYDRAULIC AND EXCAVATION TABLES 107

Table 21.—Uniform flow in circular sections flowing partly full

t —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

t/D	A/D^2	R/D	$Q_n/D^{5/3}$	$Q_n/D^{5/3}$	$Q_n/D^{5/3}$
0.00	0.0000	0.0000	0.0000	0.0000	0.0000
0.01	0.0001	0.0001	0.0001	0.0001	0.0001
0.02	0.0004	0.0004	0.0004	0.0004	0.0004
0.03	0.0009	0.0009	0.0009	0.0009	0.0009
0.04	0.0016	0.0016	0.0016	0.0016	0.0016
0.05	0.0025	0.0025	0.0025	0.0025	0.0025
0.06	0.0036	0.0036	0.0036	0.0036	0.0036
0.07	0.0049	0.0049	0.0049	0.0049	0.0049
0.08	0.0064	0.0064	0.0064	0.0064	0.0064
0.09	0.0081	0.0081	0.0081	0.0081	0.0081
0.10	0.0100	0.0100	0.0100	0.0100	0.0100
0.11	0.0121	0.0121	0.0121	0.0121	0.0121
0.12	0.0144	0.0144	0.0144	0.0144	0.0144
0.13	0.0169	0.0169	0.0169	0.0169	0.0169
0.14	0.0196	0.0196	0.0196	0.0196	0.0196
0.15	0.0225	0.0225	0.0225	0.0225	0.0225
0.16	0.0256	0.0256	0.0256	0.0256	0.0256
0.17	0.0289	0.0289	0.0289	0.0289	0.0289
0.18	0.0324	0.0324	0.0324	0.0324	0.0324
0.19	0.0361	0.0361	0.0361	0.0361	0.0361
0.20	0.0400	0.0400	0.0400	0.0400	0.0400
0.21	0.0441	0.0441	0.0441	0.0441	0.0441
0.22	0.0484	0.0484	0.0484	0.0484	0.0484
0.23	0.0529	0.0529	0.0529	0.0529	0.0529
0.24	0.0576	0.0576	0.0576	0.0576	0.0576
0.25	0.0625	0.0625	0.0625	0.0625	0.0625
0.26	0.0676	0.0676	0.0676	0.0676	0.0676
0.27	0.0729	0.0729	0.0729	0.0729	0.0729
0.28	0.0784	0.0784	0.0784	0.0784	0.0784
0.29	0.0841	0.0841	0.0841	0.0841	0.0841
0.30	0.0900	0.0900	0.0900	0.0900	0.0900
0.31	0.0961	0.0961	0.0961	0.0961	0.0961
0.32	0.1024	0.1024	0.1024	0.1024	0.1024
0.33	0.1089	0.1089	0.1089	0.1089	0.1089
0.34	0.1156	0.1156	0.1156	0.1156	0.1156
0.35	0.1225	0.1225	0.1225	0.1225	0.1225
0.36	0.1296	0.1296	0.1296	0.1296	0.1296
0.37	0.1369	0.1369	0.1369	0.1369	0.1369
0.38	0.1444	0.1444	0.1444	0.1444	0.1444
0.39	0.1521	0.1521	0.1521	0.1521	0.1521
0.40	0.1600	0.1600	0.1600	0.1600	0.1600
0.41	0.1681	0.1681	0.1681	0.1681	0.1681
0.42	0.1764	0.1764	0.1764	0.1764	0.1764
0.43	0.1849	0.1849	0.1849	0.1849	0.1849
0.44	0.1936	0.1936	0.1936	0.1936	0.1936
0.45	0.2025	0.2025	0.2025	0.2025	0.2025
0.46	0.2116	0.2116	0.2116	0.2116	0.2116
0.47	0.2209	0.2209	0.2209	0.2209	0.2209
0.48	0.2304	0.2304	0.2304	0.2304	0.2304
0.49	0.2401	0.2401	0.2401	0.2401	0.2401
0.50	0.2500	0.2500	0.2500	0.2500	0.2500
0.51	0.2601	0.2601	0.2601	0.2601	0.2601
0.52	0.2704	0.2704	0.2704	0.2704	0.2704
0.53	0.2809	0.2809	0.2809	0.2809	0.2809
0.54	0.2916	0.2916	0.2916	0.2916	0.2916
0.55	0.3025	0.3025	0.3025	0.3025	0.3025
0.56	0.3136	0.3136	0.3136	0.3136	0.3136
0.57	0.3249	0.3249	0.3249	0.3249	0.3249
0.58	0.3364	0.3364	0.3364	0.3364	0.3364
0.59	0.3481	0.3481	0.3481	0.3481	0.3481
0.60	0.3600	0.3600	0.3600	0.3600	0.3600
0.61	0.3721	0.3721	0.3721	0.3721	0.3721
0.62	0.3844	0.3844	0.3844	0.3844	0.3844
0.63	0.3969	0.3969	0.3969	0.3969	0.3969
0.64	0.4096	0.4096	0.4096	0.4096	0.4096
0.65	0.4225	0.4225	0.4225	0.4225	0.4225
0.66	0.4356	0.4356	0.4356	0.4356	0.4356
0.67	0.4489	0.4489	0.4489	0.4489	0.4489
0.68	0.4624	0.4624	0.4624	0.4624	0.4624
0.69	0.4761	0.4761	0.4761	0.4761	0.4761
0.70	0.4900	0.4900	0.4900	0.4900	0.4900
0.71	0.5041	0.5041	0.5041	0.5041	0.5041
0.72	0.5184	0.5184	0.5184	0.5184	0.5184
0.73	0.5329	0.5329	0.5329	0.5329	0.5329
0.74	0.5476	0.5476	0.5476	0.5476	0.5476
0.75	0.5625	0.5625	0.5625	0.5625	0.5625
0.76	0.5776	0.5776	0.5776	0.5776	0.5776
0.77	0.5929	0.5929	0.5929	0.5929	0.5929
0.78	0.6084	0.6084	0.6084	0.6084	0.6084
0.79	0.6241	0.6241	0.6241	0.6241	0.6241
0.80	0.6400	0.6400	0.6400	0.6400	0.6400
0.81	0.6561	0.6561	0.6561	0.6561	0.6561
0.82	0.6724	0.6724	0.6724	0.6724	0.6724
0.83	0.6889	0.6889	0.6889	0.6889	0.6889
0.84	0.7056	0.7056	0.7056	0.7056	0.7056
0.85	0.7225	0.7225	0.7225	0.7225	0.7225
0.86	0.7396	0.7396	0.7396	0.7396	0.7396
0.87	0.7569	0.7569	0.7569	0.7569	0.7569
0.88	0.7744	0.7744	0.7744	0.7744	0.7744
0.89	0.7921	0.7921	0.7921	0.7921	0.7921
0.90	0.8100	0.8100	0.8100	0.8100	0.8100
0.91	0.8281	0.8281	0.8281	0.8281	0.8281
0.92	0.8464	0.8464	0.8464	0.8464	0.8464
0.93	0.8649	0.8649	0.8649	0.8649	0.8649
0.94	0.8836	0.8836	0.8836	0.8836	0.8836
0.95	0.9025	0.9025	0.9025	0.9025	0.9025
0.96	0.9216	0.9216	0.9216	0.9216	0.9216
0.97	0.9409	0.9409	0.9409	0.9409	0.9409
0.98	0.9604	0.9604	0.9604	0.9604	0.9604
0.99	0.9801	0.9801	0.9801	0.9801	0.9801
1.00	1.0000	1.0000	1.0000	1.0000	1.0000

Table 22.—Velocity head and static pressures in circular conduits partly full

t/D	P/D	Q/D^2	A/D	R/D	$Q_n/D^{5/3}$
0.00	0.0000	0.0000	0.0000	0.0000	0.0000
0.01	0.0001	0.0001	0.0001	0.0001	0.0001
0.02	0.0004	0.0004	0.0004	0.0004	0.0004
0.03	0.0009	0.0009	0.0009	0.0009	0.0009
0.04	0.0016	0.0016	0.0016	0.0016	0.0016
0.05	0.0025	0.0025	0.0025	0.0025	0.0025
0.06	0.0036	0.0036	0.0036	0.0036	0.0036
0.07	0.0049	0.0049	0.0049	0.0049	0.0049
0.08	0.0064	0.0064	0.0064	0.0064	0.0064
0.09	0.0081	0.0081	0.0081	0.0081	0.0081
0.10	0.0100	0.0100	0.0100	0.0100	0.0100
0.11	0.0121	0.0121	0.0121	0.0121	0.0121
0.12	0.0144	0.0144	0.0144	0.0144	0.0144
0.13	0.0169	0.0169	0.0169	0.0169	0.0169
0.14	0.0196	0.0196	0.0196	0.0196	0.0196
0.15	0.0225	0.0225	0.0225	0.0225	0.0225
0.16	0.0256	0.0256	0.0256	0.0256	0.0256
0.17	0.0289	0.0289	0.0289	0.0289	0.0289
0.18	0.0324	0.0324	0.0324	0.0324	0.0324
0.19	0.0361	0.0361	0.0361	0.0361	0.0361
0.20	0.0400	0.0400	0.0400	0.0400	0.0400
0.21	0.0441	0.0441	0.0441	0.0441	0.0441
0.22	0.0484	0.0484	0.0484	0.0484	0.0484
0.23	0.0529	0.0529	0.0529	0.0529	0.0529
0.24	0.0576	0.0576	0.0576	0.0576	0.0576
0.25	0.0625	0.0625	0.0625	0.0625	0.0625
0.26	0.0676	0.0676	0.0676	0.0676	0.0676
0.27	0.0729	0.0729	0.0729	0.0729	0.0729
0.28	0.0784	0.0784	0.0784	0.0784	0.0784
0.29	0.0841	0.0841	0.0841	0.0841	0.0841
0.30	0.0900	0.0900	0.0900	0.0900	0.0900
0.31	0.0961	0.0961	0.0961	0.0961	0.0961
0.32	0.1024	0.1024	0.1024	0.1024	0.1024
0.33	0.1089	0.1089	0.1089	0.1089	0.1089
0.34	0.1156	0.1156	0.1156	0.1156	0.1156
0.35	0.1225	0.1225	0.1225	0.1225	0.1225
0.36	0.1296	0.1296	0.1296	0.1296	0.1296
0.37	0.1369	0.1369	0.1369	0.1369	0.1369
0.38	0.1444	0.1444	0.1444	0.1444	0.1444
0.39	0.1521	0.1521	0.1521	0.1521	0.1521
0.40	0.1600	0.1600	0.1600	0.1600	0.1600
0.41	0.1681	0.1681	0.1681	0.1681	0.1681
0.42	0.1764	0.1764	0.1764	0.1764	0.1764
0.43	0.1849	0.1849	0.1849	0.1849	0.1849
0.44	0.1936	0.1936	0.1936	0.1936	0.1936
0.45	0.2025	0.2025	0.2025	0.2025	0.2025
0.46	0.2116	0.2116	0.2116	0.2116	0.2116
0.47	0.2209	0.2209	0.2209	0.2209	0.2209
0.48	0.2304	0.2304	0.2304	0.2304	0.2304
0.49	0.2401	0.2401	0.2401	0.2401	0.2401
0.50	0.2500	0.2500	0.2500	0.2500	0.2500
0.51	0.2601	0.2601	0.2601	0.2601	0.2601
0.52	0.2704	0.2704	0.2704	0.2704	0.2704
0.53	0.2809	0.2809	0.2809	0.2809	0.2809
0.54	0.2916	0.2916	0.2916	0.2916	0.2916
0.55	0.3025	0.3025	0.3025	0.3025	0.3025
0.56	0.3136	0.3136	0.3136	0.3136	0.3136
0.57	0.3249	0.3249	0.3249	0.3249	0.3249
0.58	0.3364	0.3364	0.3364	0.3364	0.3364
0.59	0.3481	0.3481	0.3481	0.3481	0.3481
0.60	0.3600	0.3600	0.3600	0.3600	0.3600
0.61	0.3721	0.3721	0.3721	0.3721	0.3721
0.62	0.3844	0.3844	0.3844	0.3844	0.3844
0.63	0.3969	0.3969	0.3969	0.3969	0.3969
0.64	0.4096	0.4096	0.4096	0.4096	0.4096
0.65	0.4225	0.4225	0.4225	0.4225	0.4225
0.66	0.4356	0.4356	0.4356	0.4356	0.4356
0.67	0.4489	0.4489	0.4489	0.4489	0.4489
0.68	0.4624	0.4624	0.4624	0.4624	0.4624
0.69	0.4761	0.4761	0.4761	0.4761	0.4761
0.70	0.4900	0.4900	0.4900	0.4900	0.4900
0.71	0.5041	0.5041	0.5041	0.5041	0.5041
0.72	0.5184	0.5184	0.5184	0.5184	0.5184
0.73	0.5329	0.5329	0.5329	0.5329	0.5329
0.74	0.5476	0.5476	0.5476	0.5476	0.5476
0.75	0.5625	0.5625	0.5625	0.5625	0.5625
0.76	0.5776	0.5776	0.5776	0.5776	0.5776
0.77	0.5929	0.5929	0.5929	0.5929	0.5929
0.78	0.6084	0.6084	0.6084	0.6084	0.6084
0.79	0.6241	0.6241	0.6241	0.6241	0.6241
0.80	0.6400	0.6400	0.6400	0.6400	0.6400
0.81	0.6561	0.6561	0.6561	0.6561	0.6561
0.82	0.6724	0.6724	0.6724	0.6724	0.6724
0.83	0.6889	0.6889	0.6889	0.6889	0.6889
0.84	0.7056	0.7056	0.7056	0.7056	0.7056
0.85	0.7225	0.7225	0.7225	0.7225	0.7225
0.86	0.7396	0.7396	0.7396	0.7396	0.7396
0.87	0.7569	0.7569	0.7569	0.7569	0.7569
0.88	0.7744	0.7744	0.7744	0.7744	0.7744
0.89	0.7921	0.7921	0.7921	0.7921	0.7921
0.90	0.8100	0.8100	0.8100	0.8100	0.8100
0.91	0.8281	0.8281	0.8281	0.8281	0.8281
0.92	0.8464	0.8464	0.8464	0.8464	0.8464
0.93	0.8649	0.8649	0.8649	0.8649	0.8649
0.94	0.8836	0.8836	0.8836	0.8836	0.8836
0.95	0.9025	0.9025	0.9025	0.9025	0.9025
0.96	0.9216	0.9216	0.9216	0.9216	0.9216
0.97	0.9409	0.9409	0.9409	0.9409	0.9409
0.98	0.9604	0.9604	0.9604	0.9604	0.9604
0.99	0.9801	0.9801	0.9801	0.9801	0.9801
1.00	1.0000	1.0000	1.0000	1.0000	1.0000

DuraTuff®

polyethylene pipe

Pg 15 of 20

STRAIGHT LENGTHS

Polaris Pipe

Manufacturers of Plastic Pipe Co.

		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	.178	.53
3"	3.500	2.51	.479	1.97	2.70	.389	1.65	2.88	.318	1.38	2.97	.250	1.15
4"	4.500	3.23	.610	3.27	3.47	.500	2.73	3.68	.409	2.28	3.81	.333	1.89
5.375"	5.375	3.88	.738	4.72	4.18	.597	4.00	4.39	.480	3.26	4.55	.398	2.75
5"	5.563	3.99	.782	4.99	4.29	.618	4.16	4.55	.506	3.49	4.71	.412	2.89
6"	6.625	4.78	.908	7.08	5.11	.738	5.92	5.42	.602	4.95	5.61	.491	4.10
7.125"	7.125	5.11	.978	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	6.39	7.31	.639	6.96
10"	10.750	7.72	1.473	18.63	8.20	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	1.488	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	16.28	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.
Reference: ASTM D 3035

Calculation of the pressure-drop in a system pumping water and fluids of like viscosity are based on the Williams and Hazen formula.

$$\Delta P_f = \frac{453 \times Q^{1.05}}{C^{1.05} \times d^{4.06}}$$

POLYPIPE INDUSTRIES
DESIGN & ENG. 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

Poly Pipe
 Very smooth and straight, steel, glass
 Smooth wood and wood stave
 New rivited steel, cast iron
 Old cast iron
 Old pipes in bad condition
 Small pipes, badly corroded

"C" Value	Manning's n^*
150	
130-140	0.009-0.013
120	0.010-0.014
110	0.011-0.015
95	
60-80	
40-50

* From King's Handbook,
Pg. 182, . . .

Handbook of Hydraulics
 Horace King
 Third Edition.
 McGraw-Hill, 1939

**Golder
Associates**

SUBJECT NDO - Primary Leachate Collection System

Job No. 813-1156

Made by M. BROWN

Date 1/31/90

Ref.

Checked FSS

Sheet 6th of 20

Reviewed RW

COMPUTE NO. OF HOLES AND/OR SLOTS PER FOOT REQ'D
IN LEACHATE COLLECTION PIPE

$\frac{1}{4}" \phi$

PIPE WALL
THICKNESS

0.409 in (see previous hang table)

Hardness = $\frac{0.409}{0.25} = 1.64$

[assume coeff. from
Kings' still applies]

$C = 0.82$ from Kings Handbook

$Q = C a \sqrt{2gh}$ (see attached page)

$$a \text{ for } \frac{1}{4}" \phi \text{ hole} = \frac{\pi D^2}{4} = \frac{\pi \left(\frac{1}{4}\right)^2}{4} = 0.000341 \text{ ft}^2$$

if $h = 1 \text{ ft}$ ← reasonable assumption — op's layer slightly more than half saturated

$$Q = 0.82 (0.000341) (\sqrt{2 \times 32.2 \times 1}) = 0.0022 \text{ cfs/hole}$$

25yr inflow max. = 0.62 gpm/ft of width

$$\frac{0.62 \text{ gpm}}{\text{ft}} \cdot \frac{44.25 \text{ gpm}}{\text{cfs}} = 0.0014 \text{ cfs/ft}$$

Geomet. Capacity 2 gpm/ft → 0.0045 cfs/ft

Set criteria such that drain holes have capacity of geomet.

$$\text{No. Holes} = \frac{0.0045 \text{ cfs/ft}}{0.0022 \text{ cfs/hole}} = 2 \text{ holes/ft} \leftarrow \text{MIN. REQ'D } \frac{1}{4}" \phi \text{ HOLES}$$

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.
1939

$$Q = C a \sqrt{2gh}$$

$$C = 0.82$$

C.3-19

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HANDBOOK OF HYDRAULICS

its length, about $1\frac{1}{2}d$, such that the jet will not touch the sides of the tube. The following are average coefficients:

$$C = 0.51, \quad C_c = 0.98, \quad C_e = 0.52$$

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.

ORIFICES, GATES, AND TUBES

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Experiments for these tubes give conflicting results. Fair average values are given by Unwin¹ as follows:

Angle θ	0°	53°	113°	223°	45°
C , for Fig. 33.....	0.83	0.94	0.92	0.85	
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 Weisbach² for $d = 0.033$ foot. Other experiments indicate that these results hold approximately for larger orifices.

h in feet.....	0.068	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°.

¹Unwin, W. C., "Treatise on Hydraulics," p. 89.

²Weisbach, Julius, "Ingenieur und Maschinen-Mechanik," p. 969 (ed. 1875).

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 $2\frac{1}{2}$ 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 Fanning¹ from experiments by Michelotti, Bossut, Rennie, Castel, Lespinasse, and Ellis. Fanning's results to three decimal places are given in Table 23 (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 Bovey² from experiments on orifices of different shapes, having the same area as a circle $\frac{1}{2}$ 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 Smith³ from experiments by Lebron. 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. Table 30 suppressed contraction means that the side of the channel coincides with the edge of the orifice, and partially suppressed contraction means that the distance between the side of the channel and edge of the orifice is 0.066 foot.

Williams⁴ has determined coefficients for rectangular orifices 30 inches wide, with heights of opening of 2 to 4 inches under 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 contractions

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 Kansas 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

$$Q = Ca\sqrt{2gh} \quad (20)$$

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

Handbook of Hydraulics, Horace King, 3rd Ed., McGraw-Hill, 1939.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 7Project Title: NON DRAG OFFProject No. 893-1186.060
 Calculation Title: Tank Secondary Containment System B.25
Concrete Structural Design

Name

Date

Prepared By: M. Brown 1/16/90Checked By: Frank S. Shari 2-7-90Approved By: PAUL COOPER 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**

Made by **M. Brown**

Date **1/16/90**

Ref.

Checked **FSS**

Sheet

of **7**

Reviewed **gld**

OBJECTIVE: Design concrete floor slab and walls for TSCS concrete sump.

LOADINGS: Active earth pressure, tank weight

Equiv force
 $P = \frac{p \cdot h}{2}$

$M_{max} = \frac{P \times 4}{3} = 800 \text{ ft-lbs} = 0.8 \text{ k-ft}$

$P = \frac{300 \times 4}{2} = 600$

Treat all load as live load

$M_u = 0.8 \times 1.7 = 1.36 \text{ k-ft}$

UH moment, ultimate strength design (see code attached)

assume $a = 1"$

$M_u = A_s f_y (d - \frac{a}{2})$

$A_s = \frac{M_u}{f_y (d - \frac{a}{2})}$

use ultimate strength design see Winkler-Nelson attached

$\frac{1.36 \text{ k-ft} \times 12}{60 \text{ ksi} (3 - \frac{1}{2}) \text{ in}} = 0.109 \text{ in}^2/\text{ft}$

#4 bars @ 8" provide $0.29 \text{ in}^2/\text{ft}$

conservative practice

see attached table

**Golder
Associates**

SUBJECT

NDO Tank Secondary Containment System

Job No. 893-118.060

Made by

M. Brown

Date 1/16/90

Ref.

Checked

RSC

Sheet 2 of 7

Reviewed

RSC

checking assumed a : $\Sigma F_H = 0 \Rightarrow a \times 0.85 f_c' \times 12 \frac{1}{4} = F_y A_s$ ✓
 $a = \frac{F_y A_s}{0.85 f_c' \times 12} = \frac{60 \text{ ksi} \times 0.29 \text{ in}^2}{0.85 \times 3 \text{ ksi} \times 12} = 0.56" < 1" \therefore \text{OK}$

Provide Steel Reinforcement as follows:

Vertical Wall Steel #4's @ 8"
 Horizontal Wall Steel #4's @ 12"
 Floor Slab Steel #4's @ 18" EW. } at center of 6" THK slab.

Weight of $\frac{1}{4}$ " THK Steel Plate:

$\frac{0.25}{12} \times 490 \text{ lb/ft}^3 = 10.2 \text{ lb/ft}^2$
 ↑ Steel density

From dwg:
 tank diam. = 8'
 side shell length = 25'

Check load under tank footings -

Tank weight = Water + Steel
 Water = $1000 \text{ g} \times 8.5 \text{ ft}^3 = 85000 \text{ lbs.}$
 Steel = area $\times 10 \text{ lb/ft}^2 = \{ (\pi \times 25) + \pi \times 2 \} \times 10 = 7400 \text{ lbs.}$
 weight = 93 kips.

Area = 2 footings

Length = 10

Area per footing = $2 \times 10 = 20 \text{ ft}^2$
 TOTAL AREA = 40 ft²

foundation pressure = 2.32 ksf

↑ no problem under 2b. *

*SEE FURTHER DISCUSSION - NEXT PAGE

SHG Inspection
 Project Parameters
 150075 Reviewed
 RSC

**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 (expected occurrence). 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. 27.1a can be either the major or the minor principal stress, the ratio $K = p_h/p_v$ can assume any value between the limits,

$$K_A = \frac{p_h}{p_v} = \frac{1}{N_\phi} = \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \quad (27.2)$$

and

$$K_P = \frac{p_h}{p_v} = N_\phi = \tan^2 \left(45^\circ + \frac{\phi}{2} \right) \quad (27.3)$$

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

$$p_h = K_0 p_v \quad (27.4)$$

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 $p_h = K p_v$, 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

$$p_h = K_A p_v = K_A \gamma z = \gamma z \frac{1}{N_\phi} \quad (27.5)$$

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. 27.1b. 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. 27.1a. According to Eq. 16.3, such surfaces of sliding intersect the direction of the minor principal stress at the angle $45^\circ + \phi/2$. Since the minor principal stresses in the active Rankine state are horizontal, the shear planes rise at an angle of $45^\circ + \phi/2$ 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. 27.1c. 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

$$p_h = K_P p_v = K_P \gamma z = \gamma z N_\phi \quad (27.6)$$

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 $45^\circ - \phi/2$ with the horizontal, as shown in Fig. 27.1c.

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 Engg. Practice
Kerzagh & Pick, 2nd Ed, Wiley, 1968

- 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

$$U = 1.4D + 1.7L \quad (9-1)$$

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

$$U = 0.75(1.4D + 1.7L + 1.7W) \quad (9-2)$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W \quad (9-3)$$

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

$$U = 1.4D + 1.7L + 1.7H \quad (9-4)$$

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

$$U = 0.75(1.4D + 1.4T + 1.7L) \quad (9-5)$$

but required strength U shall not be less than

$$U = 1.4(D + T) \quad (9-6)$$

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_t 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 $0.10f_c' A_g$ to zero.

For other reinforced members, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10f_c' A_g$ or ϕP_{n1} , 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

$$C = 0.85f_c'ab \quad (2.30)$$

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:

$$a/c = k_1 = 0.85 \text{ for } f_c' \leq 4000 \text{ psi and decreases by 0.05 for every 1000 psi above 4000} \quad (2.31)$$

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, $\epsilon_c = \epsilon_s/(d - c) = 0.00138(4.35/18.65) = 0.00032$. 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, Nilson, McGraw-Hill 7th Ed. 1964

7.57

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	0.75	0.98	1.18	1.37	1.57	1.77	1.96	2.16	2.36	2.55	2.73	
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	7.21	7.82	8.42	
8	1.57	2.35	3.14	3.93	4.71	5.50	6.29	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	17.18	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	9.00	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	92.2	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	0.75	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	1.83	2.40	3.04	3.73		
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		

Design of Concrete Structures
Winter, Ungerhart, O'Rourke, Nilson
McGraw Hill, 7th Ed. 1964

appendix B

TABLES AND DIAGRAM

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	½ = 0.250	0.05	0.79	0.167
3	¾ = 0.375	0.11	1.18	0.376
4	½ = 0.500	0.20	1.57	0.683
5	¾ = 0.625	0.31	1.96	1.043
6	¾ = 0.750	0.44	2.36	1.502
7	¾ = 0.875	0.60	2.75	2.044
8	1 = 1.000	0.79	3.14	2.670
9	1½ = 1.125†	1.00	3.54	3.400
10	1½ = 1.270†	1.27	3.99	4.303
11	1½ = 1.410†	1.56	4.43	5.313
14S	1½ = 1.693†	2.25	5.32	7.650
18S	2½ = 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 ½ in.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 10Project Title: WHC NON DRAG:OFFProject No. 893-1186Calculation Title: Secondary Leachate Collection Sump - B.13
Leak Detection System.

Name

Date

Prepared By: M. Brown 1/22/90Checked By: Frank S. Shiri 2-7-90Approved By: PAUL CORSE 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 FB
Reviewed RC

Date 1/22/10
Sheet 1 of 10

Check Accuracy of Secondary Sump Monitoring

Minimum water depth = 6"

Area at depth = 6"

(min. depth = 6" because pump cannot remove more)

30 ft²

(from spreadsheet comp's by Porterfield et al) Attached.

Maximum accuracy of transducer = 0.5% of full scale
See attached manuf. info.
Full scale = 16 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

↑ Action Leakage Rate

6.3 < 15 gal/day

(actually 5 to 20)

OK

AREA OF THE
LINED WASTE
DISPOSAL FACILITY,
SEE HELP MODEL CAMP'S
CMCS B.11 and B.21.

Secondary Drainage Gravel										Primary Drainage Gravel										Primary Operations Layer										Free Water Pond														
Eff Porosity = 0.45										Eff Porosity = 0.50										Eff Porosity = 0.35										Eff Porosity = 1.00														
Increment					Accum					Increment					Accum					Increment					Accum					Increment					Accum									
Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore	Area	Volume	Eff Pore	Eff Pore	Eff Pore					
sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	cu ft	gallons	sq ft	cu ft	cu ft	gallons						
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00							
0.25	8.6	1.1	5.9	4.8	0.50	20.0	1.1	5.9	4.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00						
0.50	20.0	1.1	5.9	4.8	0.75	62.1	1.1	5.9	4.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00					
0.75	62.1	1.1	5.9	4.8	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0	1.00	107.3	21.2	38.6	33.0
1.00	107.3	21.2	38.6	33.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0	1.25	171.0	34.8	73.4	63.0
1.25	171.0	34.8	73.4	63.0	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3	1.50	245.3	52.0	125.4	107.3
1.50	245.3	52.0	125.4	107.3	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1	1.75	326.3	71.5	196.9	164.1
1.75	326.3	71.5	196.9	164.1	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7	2.00	456.2	97.8	294.7	241.7
2.00	456.2	97.8	294.7	241.7	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8	2.25	587.1	124.1	375.1	304.8
2.25	587.1	124.1	375.1	304.8	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9	2.50	729.8	152.1	427.2	341.9
2.50	729.8	152.1	427.2	341.9	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1	2.75	881.8	180.4	500.8	409.1
2.75	881.8	180.4	500.8	409.1	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3	3.00	1036.4	221.0	583.8	477.3
3.00	1036.4	221.0	583.8	477.3	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4	3.25	1204.8	261.0	686.6	556.4
3.25	1204.8	261.0	686.6	556.4	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6	3.50	1386.0	304.8	801.2	648.6
3.50	1386.0	304.8	801.2	648.6	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2	3.75	1586.0	341.9	937.7	755.2
 																		

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, 0/60, 0/130, 0/230, 0/340, 0/400, 0/600

PSI: 0/6, 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: $\pm 2\%$ output change for $\pm 25^\circ\text{C}$ temp. change within compensated range (for ranges over 30 psi)ACCURACY: $\pm 0.5\%$ O.S. (including linearity, hysteresis and repeatability, B.S.L.)

IMPEDANCE:

Typical Excitation
Input Impedance
Output Impedance10 $\pm 5\text{VDC}$
8K ohms typical
5K ohms typical

EXCITATION VOLTAGE: 16 VDC max; 10 VDC (regulated) typical, 2 VDC min. 571 motor provides 8 VDC excitation aid.

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: $\frac{1}{4}$ " 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	...	.	.	.	..	(Specify in feet)
57 SN	...	.	.	.	..	(Specify in feet)

MODEL
57SP Transducer ($\frac{1}{4}$ " NPT Process Conn.)
57SN Transducer (Non-Clogging)

RANGE	M.V. OUTPUT
016 = 0-6 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. St.

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., FEASTERTVILLE, PA 19047
TELEPHONE (215) 355-6900 TELEEX 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

P. R. Schroeder, B. M. McEnroe, R. L. Peyton, and J. W. Sjoström
US Army Engineer Waterways Experiment Station
Vicksburg, MS 39181-0631

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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

5.10

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

6.410

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Source.

56

Source:

7.410

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,

8-10

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)

Leakage Mechanism	FML Alone		
	Hydraulic Head, ft		
	0.1	1	10
Small Hole (1-2 mm)	30	100	300
Standard Hole (1 cm ²)	300	1,000	3,000

Leakage Mechanism	Composite Liner (good contact)		
	Hydraulic Head, ft		
	0.1	1	10
Small Hole (1-2 mm)	0.01	0.1	2
Standard Hole (1 cm ²)	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:

$$h = (Q_d/B)/(k_d \tan \beta) \quad (1)$$

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 discrete 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

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SUBJECT **NDO**

Job No. 893-1186
Ref.

Made by **M. BROWN**
Checked **FSS**
Reviewed

Date **2/7/90**
Sheet **04** 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)

$$\text{Uplift} = 7.5' \times 62.4 \text{ pcf} = 468 \text{ psf}$$

$$\text{Soil pressure down} = 6' \times 110 \text{ pcf} = 660 \text{ psf} > 468 \therefore \underline{\underline{OK}}$$

↑
est. moist density

C.5-11

TABLE III
LEACHATE STORAGE VOLUME AVAILABLE

10' x 10'

Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 16Project Title: WHC NON DRAG OFFProject No. 893-1186Calculation Title: Vertical Riser Pipe Foundation - Structural Design.

Name

Date

Prepared By: M. BROWN 1-24-90Checked By: Frank S. Shui 2-7-90Approved By: TRACY COBB 2-13-90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

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Associates**

SUBJECT WHC / NDO VERTICAL RISER (CONCRETE FD)		
Job No. F93-1186	Made by M. Prown	Date 1/24/90
Ref.	Checked FSS	Sheet 1 of 6
	Reviewed	

SELECT REINFORCEMENT FOR FOUNDATION OF VERTICAL RISER

Conservatively assume

$M_{\max} = (6.85 \text{ k/ft}) \left(\frac{1 \text{ ft}}{2} \right) \left(\frac{1 \text{ ft}}{2} \right) = 3.42 \text{ k-ft}$

Treat this load as mostly a deadload with a partial live load component.

Using Ultimate Strength Design $M_u = 1.5 M_{\max} = 5.14 \text{ k-ft}$

see attached code

- 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

$$U = 1.4D + 1.7L \quad (9-1)$$

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

$$U = 0.75(1.4D + 1.7L + 1.7W) \quad (9-2)$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W \quad (9-3)$$

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

$$U = 1.4D + 1.7L + 1.7H \quad (9-4)$$

except that where D or L reduce the effect of H , 0.9 D shall be substituted for 1.4 D 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 r is included in design, load combinations of Section 9.2.4 shall apply, except that 1.4 F shall be substituted for 1.7 H . 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

$$U = 0.75(1.4D + 1.4T + 1.7L) \quad (9-5)$$

but required strength U shall not be less than

$$U = 1.4(D + T) \quad (9-6)$$

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_{n1} , whichever is smaller, to zero.

- (d) Shear and torsion 0.85

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SUBJECT WHC/ND 0

SLAB.

Job No. 843-1186

Made by M. BROWN

Date

Ref.

Checked FSS

Sheet 4 of 6

PLAN SHOWING REINFORCEMENT

NOTE:

1. Provide $1\frac{1}{2}$ " 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 $1\frac{1}{2}$ 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	1 1/2

(c) Concrete not exposed to weather or in contact with ground:

Slabs, walls, joists:	
#14 and #18 bars	1 1/2
#11 bar and smaller	3/4
Beams, columns:	
Primary reinforcement, ties, stirrups, spirals	1 1/2
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	1 1/2
#11 bar and smaller	3/4
Other members:	
#14 and #18 bars	2
#6 through #11 bars	1 1/2
#5 bar, W31 or D31 wire, and smaller	1 1/4

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

$$C = 0.85f_c ab \quad (2.30)$$

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:

$$a/c = k_1 = 0.85 \text{ for } f_c \leq 4000 \text{ psi and decreases by } 0.05 \text{ for every } 1000 \text{ psi above } 4000 \quad (2.31)$$

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 $ab \times 0.85f_c = A_s f_s$, 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, $\epsilon_c = \epsilon_s/(d - c) = 0.00138(4.35/18.65) = 0.00032$. 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, Wile-Interscience, 1964, 7th Ed., Mc Graw Hill

Golder Associates Inc.

Calculation Cover Sheet

Sheet 0 of 10Project Title: WHC Non Drag OffProject No. 893-1186Calculation Title: B.14 Sump Design.

	Name	Date
Prepared By:	<u>M. Brown</u>	<u>1/22/90</u>

Checked By:	<u>Frank S. Shuri</u>	<u>2-7-90</u>
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Approved By:	<u>Patrick G. Golder</u>	<u>2-8-90</u>
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QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

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Associates**

SUBJECT NON DRAG OFF - PUMP DISCH. PIPE DIAM.

Job No. B93-1186
Ref.

Made by M. BROWN
Checked FSS
Reviewed RL

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

Minimum Pump Capacities 5gpm. = 0.011 cfs

Smallest pipe available Nominal 1 1/2" φ

In SDR 13.5 I.D. = 1.62" Velocity = $V = \frac{0.011}{\pi \left(\frac{1.62}{2}\right)^2} = 0.77 \text{ fps}$ ✓
OK.

Large pump 150gpm. = 0.333 cfs

Using SDR 17 ID = 3.08" $V = \frac{0.333}{\pi \left(\frac{3.08}{2}\right)^2} = 6.46 \text{ fps} < 8 \text{ OK}$ ✓
Size nominal,

* The selection of 8fps 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.

Occasional operation systems have higher velocities because of the importance of capital cost.

20410

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-

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Water Supply and Pollution Control

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SUBJECT WHC NON DRAG OFF

Job No. 893-1186.032
Ref.

Made by M. Brown
Checked JSS
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" HOPE piping SDR17

from pipe dimension chart I.D. = 3.97" $\Rightarrow V = 3.89 fps$

try 3" nominal SDR17 I.D. = 3.08" $V = 6.46 fps$ OK

Assumption
is used

Even if we use 3" ϕ SDR11 I.D. = 2.86" $V = 7.49 fps < 8$ still OK

Assume #1 primary & Secondary Submersible pumps

200 100 gpm = 0.022 cfs

Try nominal $1\frac{1}{2}$ " diam. SDR13.5 (lightest available)

I.D. = 1.62" $V = 1.55 fps$

for SDR7.3 I.D. = 1.38" $V = 2.15 fps$ ← USE THIS.

#2
for Primary
SDR17, 3" ϕ

$$A = \frac{\pi D^2}{4} = \frac{\pi (3.08)^2}{4} = 7.45 in^2 = 0.0517 ft^2$$

Length

20'

see plan view in calc. 8.12

$$L = 40 + 100 + 50 = 190'$$

(slope correction negligible)

$$\text{Volume} = 0.0517 ft^2 \times 190' = 9.82 ft^3 = 73.5 gallons$$

no vertical loop for submersible discharge line

for Submersibles

$$L = 100 + 50 = 150'$$

$$A = \frac{\pi D^2}{4} = \frac{\pi (1.38)^2}{4} = 1.5 in^2 = 0.0104 ft^2$$

$$V = 0.0104 \times 150 = 1.56 ft^3 = 12.9 gallons. OK$$

**Golder
Associates**

SUBJECT **NDO****PRIMING PUMP**Job No. **843-1186**Made by **M. BROWN**Date **1/28/10**

Ref.

Checked **PS**Sheet **4** of **10**

Reviewed

From attached Volume-elevation information:

Elevation
above bottom
of Priming
Pump.

Usable
Leachate Storage
Volume
gallons

0'
0.25'
0.5'
0.75'
1.00'

0
7
27
68
131

minimum water level, pump
diam = 4"

drain back volume from submersible pump will raise
water level

$$\frac{12 \text{ gal.}}{(68-27) \text{ gal.}} \times 3" = 0.88" \approx \underline{1"} \checkmark$$

from self priming pump

$$3" + \left[\frac{73.5 - (68-27)}{131-68} \times 3 \right] = \underline{4.5"} \checkmark$$

Float switches and
current sensitive relays on pump
controls should be set to reflect these
drain back levels.

NOX-DRAG OFF MIXED RADIOACTIVE WASTE LAX LL 831-1186.032

VOLUME - ELEVATION SUMP

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		Accum		Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increment		Accum		Eff Pore Volume cu ft	Eff Pore Volume gallons	Area sq ft	Increment		Accum		Eff Pore Volume cu ft	Eff Pore Volume gallons			
		Volume cu ft	Volume cu ft	Volume cu ft	Volume cu ft				Volume cu ft	Volume cu ft	Volume cu ft	Volume cu ft				Volume cu ft	Volume cu ft	Volume cu ft	Volume cu ft			Volume cu ft		Volume cu ft
0.00	0.00			0.00	0.00	0		0.00				0.00	0.0	0		0.00				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			0.00	0.0	0.0	0	0
0.50	20.0	4.8		5.9	2.7	20		0.0	0.0			0.0	0.0	0		0.0	0.0			0.00	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			0.00	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			0.00	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			0.00	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			0.00	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			0.00	0.0	0.0	0	7	
2.00	456.2	97.8		294.7	132.6	992		28.6	5.4	7.2		3.6	27		0.0	0.0			0.00	0.0	0.0	0	27	
2.25	187.1	80.4		375.1	168.8	1263		58.4	10.9	18.1		9.0	68		0.0	0.0			0.00	0.0	0.0	0	68	
2.50	229.8	52.1		427.2	192.2	1438		77.0	16.9	35.0		17.5	131		0.0	0.0			0.00	0.0	0.0	0	131	
2.75	281.8	64.0		491.2	221.0	1653		105.1	22.8	57.8		28.9	216		0.0	0.0			0.00	0.0	0.0	0	216	
3.00	336.4	77.3		568.4	255.8	1913		135.2	30.0	87.8		43.9	328		0.0	0.0			0.00	0.0	0.0	0	328	
3.25	304.8	80.2		648.6	291.9	2183		247.8	47.9	135.7		67.8	507		0.0	0.0			0.00	0.0	0.0	0	507	
3.50	226.0	66.4		714.9	321.7	2406		347.5	74.4	210.1		105.0	786		0.0	0.0			0.00	0.0	0.0	0	786	
3.75	168.0	49.3		764.2	343.9	2572		460.8	101.0	311.1		155.6	1164		0.0	0.0			0.00	0.0	0.0	0	1164	
4.00	0.0	21.0		785.2	353.3	2643		585.0	130.7	441.9		220.9	1653		0.0	0.0			0.00	0.0	0.0	0	1653	
4.25	0.0	0.0		785.2	353.3	2643		750.3	166.9	608.8		304.4	2277		0.0	0.0			0.00	0.0	0.0	0	2277	
4.50	0.0	0.0		785.2	353.3	2643		911.3	207.7	816.5		408.2	3054		0.0	0.0			0.00	0.0	0.0	0	3054	
4.75	0.0	0.0		785.2	353.3	2643		1073.1	248.0	1064.5		532.3	3981		0.0	0.0			0.00	0.0	0.0	0	3981	
5.00	0.0	0.0		785.2	353.3	2643		1240.8	289.2	1353.8		676.9	5063		0.0	0.0			0.00	0.0	0.0	0	5063	
5.25	0.0	0.0		785.2	353.3	2643		1405.4	330.8	1684.5		842.3	6300		0.0	0.0			0.00	0.0	0.0	0	6300	
5.50	0.0	0.0		785.2	353.3	2643		1900.0	413.2	2097.7		1048.9	7845		0.0	0.0			0.00	0.0	0.0	0	7845	
5.75	0.0	0.0		785.2	353.3	2643		2100.0	500.0	2597.7		1298.9	9715		0.0	0.0			0.00	0.0	0.0	0	9715	
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	11515	
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	13315	
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	15442	
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	17897	
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	20711	
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	23885	
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	27354	
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	31118	
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	35323	
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	7500.00	950.0	950.0	950.0	7106	48466
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	71795
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	33364	97970
9.00	0.0	0.0		785.2	353.3	2643		0.0	0.0	2860.2		1430.1	10697	16000.00	3787.5	22067.5	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

C.7-6

WMC-SD-M025-FDR-001, Rev. 1

50410

**Golder
Associates**

SUBJECT **WHC-NDO Leachate Collection Riser Pipes**
 Job No. **B93-1156** Made by **M. BROWN** Date **1/22/90**
 Ref. Checked **FS** Sheet **6** of **10**
 Reviewed

Calculate load necessary to bend slope riser pipes to required radius at toe of slope

PIPE = 8" DIAM. SDR=11 from T. Martin notes.

$$E = 136,000 \text{ psi}$$

from Pipe Manual
Attached

$$OD = 8.625"$$

$$ID = 7.05"$$

From AISC manual, attached

From AISC Steel Construction Manual

$$I = 0.049087 (d_o^4 - d_i^4)$$

$$I = 150 \text{ in}^4$$

$$M = EI \frac{d^2 y}{dx^2}$$

Required radius of curvature $\approx 20'$ from drawings.

from multi handbook, attached

$$R = \frac{EI}{M} = \frac{EI}{\frac{d^2 y}{dx^2}}$$

$$R = 20' = 240"$$

$$240' = \frac{136,000 \frac{\text{lb}}{\text{in}^2} \times 150 \text{ in}^4}{M}$$

$$\Rightarrow M = 85,000 \text{ in-lbs}$$

$$M = 7.1 \text{ k-ft}$$

Determine point load which would produce this moment, in order to get a sense of magnitude

$$\therefore M_{\text{max}} = 7.1 \text{ k-ft} = \frac{PL}{4} \Rightarrow P = \frac{7.1 \text{ k-ft} \times 4}{6.43 \text{ ft}} = 4.42 \text{ kips}$$

POLY PIPE INDUSTRIES DESIGN & ENGINEERING GUIDE

Fig. 1
NOMINAL PHYSICAL PROPERTIES
POLY PIPE PE 3400 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.

BEAM DIAGRAMMS 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 2-196 is noted.

BEAMS

Flexural stress at extreme fiber:

$$f = Mc/I = M/S$$

Flexural stress at any fiber:

$$f = My/I \quad y = \text{distance from neutral axis to fiber.}$$

Average vertical shear (for maximum see below):

$$v = V/A = V/dt \quad (\text{for beams and girders})$$

Horizontal shearing stress at any section A-A:

$$v = VQ/Ib \quad Q = \text{statical moment about the neutral axis of the entire section of that portion of the cross-section lying outside of section A-A,} \\ b = \text{width at section A-A}$$

(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:

$$EI \frac{d^2y}{dx^2} = M \quad x \text{ and } y \text{ are abscissa and ordinate respectively of a point on the neutral axis, referred to axes of rectangular coordinates through a selected point of support.}$$

(First integration gives slopes; second integration gives deflections. Constants of integration must be determined.)

CONTINUOUS BEAMS (THE THEOREM OF THREE MOMENTS)

$$\text{Uniform load: } M_a \frac{l_1}{l_1} + 2M_b \left(\frac{l_1}{l_1} + \frac{l_2}{l_2} \right) + M_c \frac{l_2}{l_2} = -\frac{1}{4} \left(\frac{w_1 l_1^3}{l_1} + \frac{w_2 l_2^3}{l_2} \right)$$

Concentrated loads:

$$M_a \frac{l_1}{l_1} + 2M_b \left(\frac{l_1}{l_1} + \frac{l_2}{l_2} \right) + M_c \frac{l_2}{l_2} = -\frac{P_1 a_1 b_1}{l_1} \left(1 + \frac{a_1}{l_1} \right) - \frac{P_2 a_2 b_2}{l_2} \left(1 + \frac{b_2}{l_2} \right)$$

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

6-27

PROPERTIES OF GEOMETRIC SECTIONS

90410

TRAPEZOID Axis of moments through center of gravity

$$\begin{aligned}
 A &= \frac{d(b + b_1)}{2} \\
 c &= \frac{d(2b + b_1)}{3(b + b_1)} \\
 I &= \frac{d^3(b^2 + 4bb_1 + b_1^2)}{36(b + b_1)} \\
 S &= \frac{d^2(b^2 + 4bb_1 + b_1^2)}{12(2b + b_1)} \\
 r &= \frac{d}{6(b + b_1)} \sqrt{2(b^2 + 4bb_1 + b_1^2)}
 \end{aligned}$$

CIRCLE Axis of moments through center

$$\begin{aligned}
 A &= \frac{\pi d^2}{4} = \pi R^2 = .785398 d^2 = 3.141593 R^2 \\
 c &= \frac{d}{2} = R \\
 I &= \frac{\pi d^4}{64} = \frac{\pi R^4}{4} = .049087 d^4 = .785398 R^4 \\
 S &= \frac{\pi d^3}{32} = \frac{\pi R^3}{4} = .098176 d^3 = .785398 R^3 \\
 r &= \frac{d}{4} = \frac{R}{2} \\
 Z &= \frac{d^3}{6}
 \end{aligned}$$

HOLLOW CIRCLE Axis of moments through center

$$\begin{aligned}
 A &= \frac{\pi(d^2 - d_1^2)}{4} = .785398 (d^2 - d_1^2) \\
 c &= \frac{d}{2} \\
 I &= \frac{\pi(d^4 - d_1^4)}{64} = .049087 (d^4 - d_1^4) \\
 S &= \frac{\pi(d^3 - d_1^3)}{32d} = .098176 \frac{d^4 - d_1^4}{d} \\
 r &= \frac{\sqrt{d^2 + d_1^2}}{4} \\
 Z &= \frac{d^3}{6} - \frac{d_1^3}{6}
 \end{aligned}$$

HALF CIRCLE Axis of moments through center of gravity

$$\begin{aligned}
 A &= \frac{\pi R^2}{2} = 1.570796 R^2 \\
 c &= R \left(1 - \frac{4}{3\pi}\right) = .57687 R \\
 I &= R^4 \left(\frac{\pi}{8} - \frac{8}{9\pi}\right) = .109757 R^4 \\
 S &= \frac{R^3}{24} \left(\frac{9\pi^2 - 64}{(8\pi - 4)}\right) = .190687 R^3 \\
 r &= R \frac{\sqrt{9\pi^2 - 64}}{6\pi} = .264336 R
 \end{aligned}$$

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 $y = r \sin \theta$
 $dx = \cos \theta dr - r \sin \theta d\theta$, $dy = \sin \theta dr + r \cos \theta d\theta$

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

$$f(b) = f(a) + (b - a)f'(x_1) \quad a < x_1 < b$$

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 $\lim_{x \rightarrow a} f(x) = 0$, $\lim_{x \rightarrow a} F(x) = 0$, and $\lim_{x \rightarrow a} F'(x) \neq 0$ [or if $\lim_{x \rightarrow a} f(x) = \lim_{x \rightarrow a} F(x) = \infty$], then

$$\lim_{x \rightarrow a} \frac{f(x)}{F(x)} = \lim_{x \rightarrow a} \frac{f'(x)}{F'(x)}$$

(b) If $\lim_{x \rightarrow a} f(x) = 0$ and $\lim_{x \rightarrow a} F(x) = \infty$ [that is, $F(x)$ becomes infinite as x approaches a as a limit], then $\lim_{x \rightarrow a} [f(x)F(x)]$ 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 $\lim_{x \rightarrow a} f(x) = \infty$ and $\lim_{x \rightarrow a} F(x) = \infty$, then $\lim_{x \rightarrow a} [f(x) - F(x)]$ 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 $\lim_{x \rightarrow a} [f(x)]^{F(x)}$ 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,
 Richard G. Barrington,
 McGraw-Hill, 1965.

for $dy/dx \neq 0$
 $R = \frac{1}{\frac{d^2y}{dx^2}}$

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 5Project Title: Non-Drag-OFF LandFill Design ProjectProject No. 893-1186.032Calculation Title: Subgrade Side Slope Stability

Name

Date

Prepared By: Timothy L. Martin 1-13-90Checked By: Frank S. Shum 2-5-90Approved By: [Signature] 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 FSS

Sheet 2 of 5

Reviewed Ed

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 ($18.4^\circ \approx 18^\circ$)
- 2) The slope will be fully drained.
- 3) Based on laboratory test data, $c = 0$, $\phi = 38^\circ$
- 4) Dynamic horizontal acceleration of $0.12g$ to be used in the analysis.

Analysis:

Infinite slope equation:

$$F.S. = \frac{(c/\gamma) \sec \alpha + [(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi'}{\sin \alpha + C_s \cos \alpha}$$

where c = effective cohesion

γ = unit weight of sliding mass

d = thickness (width) of sliding mass

α = slope angle

r_u = pore pressure ratio

C_s = horizontal acceleration

ϕ' = Effective Friction angle of slide mass

Since $c = 0$ and $r_u = 0$, and for a static analysis $C_s = 0$

Static Analysis:

$$F.S. = \frac{\tan \phi'}{\tan \alpha} = \frac{\tan 38^\circ}{\tan 18^\circ} = 2.4$$

For a dynamic Analysis, $C_s = 0.12g$

$$F.S. = \frac{[\cos \alpha - C_s \sin \alpha] \tan \phi'}{\sin \alpha + C_s \cos \alpha} = \frac{[\cos 18^\circ - (0.12) \sin 18^\circ] \tan 38^\circ}{\sin 18^\circ + (0.12) \cos 18^\circ}$$

$$F.S. = 1.69 \quad \checkmark$$

**Golder
Associates**

SUBJECT Subgrade Side Slope Stability

Job No. 893-1186.012

Made by TLM

Date 1-13-90

Ref.

Checked FJA

Sheet 3 of 5

Reviewed JH

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 .

$$FS = \frac{(cos \alpha - C_s \sin \alpha) \tan \phi'}{\sin \alpha + C_s \cos \alpha}$$

$$1.0 = \frac{(cos 18 - C_s \sin 18) \tan 38}{\sin 18 + C_s \cos 18}$$

$$1.0 = \frac{(0.951 - C_s 0.309) 0.781}{0.309 + C_s 0.951}$$

$$1.0 = \frac{0.743 - C_s 0.241}{0.309 + C_s 0.951}$$

$$0.309 + C_s 0.951 = 0.743 - C_s 0.241$$

$$C_s 1.192 = 0.434$$

$$C_s = 0.364 \approx \underline{0.36} \quad \checkmark \text{ critical acceleration}$$

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. $\checkmark$

4 of 5

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 underlain 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, CW . 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 $\bar{c}a \sec \alpha$ and the other due to friction and equal to $\bar{N} \tan \phi$, where $\bar{N}$ is the effective force normal to the failure plane. With a pore pressure ratio, r_u ,

$$\bar{N} = W [(1 - r_u) \cos \alpha - C_s \sin \alpha] \quad (6.1)$$

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

$$F = \frac{\bar{c}a \sec \alpha + W [(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{W(\sin \alpha + C_s \cos \alpha)} \quad (6.2)$$

Replacing W by γad , where γ is the unit weight of the sliding mass

$$F = \frac{(\bar{c}/\gamma d) \sec \alpha + [(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{\sin \alpha + C_s \cos \alpha} \quad (6.3)$$

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

$$F = \frac{[(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{\sin \alpha + C_s \cos \alpha} \quad (6.4)$$

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

$$F = (1 - r_u) \frac{\tan \phi}{\tan \alpha} \quad (6.5)$$

6.2 TRIANGULAR CROSS :

Figure 6.2 shows a triangular failure plane is along the bottom failure are the spoil banks of 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 infinite $H \csc \alpha$ instead of $a \sec \alpha$

$$F = \frac{\bar{c}H \csc \alpha + W}{W}$$

and

$$W = \frac{1}{2} \gamma H^2 \csc \beta$$

in which γ is the unit weight

Substituting W from Eq. 6.3

$$F = \frac{2 \sin \beta \csc (\beta - \alpha) (\bar{c}/\gamma d)}{W}$$

Note that $\bar{c}$, γ , and H can be determining the factor of safety, height, H ,

$$F = \frac{\bar{c}W_f \sin \beta / \sin (\beta - \alpha) + W \sin \alpha}{W \sin \alpha}$$

FIGURE 6.2

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Liner Side Slope Stability

Name

Date

Prepared By: Timothy L. Martin January 24, 1990Checked By: Frank S. Shum January 30, 1990Approved By: Bob Carr Jan 31, 1990

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

CR 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: $FS = (c/Gd) \sec a + (\cos a - Cs \sin a) \tan P / (\sin a + Cs \cos a)$

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
P = 30

Liner interfaces: Soil/geotextile

c = 0
P = 30

CRITICAL INTERFACE ✓

Note: c, P are residual strength parameters

Static Analysis: FS = 1.78 ✓

Dynamic Analysis: FS = 1.25 ✓

pg 3 of 11

893-1186

ER SIDE SLOPE STABILITY
Date: 30-Jan-90

Made by: TLM
Checked by: FEB
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 equations: $FS = (c/c_0) \sec \alpha + (\cos \alpha - C_0 \sin \alpha \tan P) / (\sin \alpha + C_0 \cos \alpha)$

Where: c = effective cohesion
 G = unit weight of sliding mass
 d = vertical thickness of sliding mass
 α = slope angle
 C_0 = horizontal acceleration
 P = friction angle

Based on current design:

$G = 120 \text{ pcf}$
 $d = 2 \text{ ft}$
 $\alpha = 18$
 $C_0 = 0.12 g$
Liner interface: HDPE/HDPE (LEX) $c = 320$
 $P = 36$

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: TLM
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: $FS = (c/G \sec a + (\cos a - C_s \sin a) \tan P) / (\sin a + C_s \cos a)$ Where: c = effective cohesion
G = weight of sliding mass
d = vertical thickness of sliding mass
a = slope angle
C_s = horizontal acceleration
P = friction angle

Based on current design:

G = 120 pcf
c = 2 ft
a = 18
C_s = 0.12 g
liner interface: Geocomp/HDP E (tex) = 188
F = 22Note: c, P_{peak} = residual, no distinct peak

Static Analysis: FS = 3.91 ✓

Dynamic Analysis: FS = 2.82 ✓

Pg 5 of 11

893-1186

WDM-DRAG-OFF LANDFILL DESIGN

Project No.: 893-1186

IER SIDE SLOPE STABILITY

Made by: TLM

Date: 24-Jan-90

Checked by: FSB

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 slopes will be fully drained.
3. Dynamic acceleration of 0.12g used for dynamic analysis.

Analysis:

Infinite slope equation: $FS = (c/\cos a + (\cos a - Cs \sin a) \tan P) / (\sin a + Cs \cos a)$

where: c = effective cohesion
G = unit weight of sliding mass
z = vertical thickness of sliding mass
a = slope angle
Cs = horizontal acceleration
P = friction angle

Based on current design:

G = 120 pcf
a = 2 ft
z = 18
Cs = 0.12 g
Liner interfaces: Geotextile/Soil (top) = 195
P = 12

Note: c, P are residual strengths

Static Analysis:	FS =	3.42	✓
Dynamic Analysis:	FS =	2.48	✓

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 underlain 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, $C_s W$. In contrast to the cylindrical failure, the effect of seismic force on the force

FIGURE 6.1. Analysis of infinite slope.

PART III / SIMPLIFIED METHODS OF STABILITY ANALYSIS

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 $\bar{c}a \sec \alpha$ and the other due to friction and equal to $\bar{N} \tan \phi$, where $\bar{N}$ is the effective force normal to the failure plane. With a pore pressure ratio, r_u ,

$$\bar{N} = W [(1 - r_u) \cos \alpha - C_s \sin \alpha] \quad (6.1)$$

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

$$F = \frac{\bar{c}a \sec \alpha + W [(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{W(\sin \alpha + C_s \cos \alpha)} \quad (6.2)$$

Replacing W by γad , where γ is the unit weight of the sliding mass

$$F = \frac{(\bar{c}/\gamma d) \sec \alpha + [(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{\sin \alpha + C_s \cos \alpha} \quad (6.3)$$

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

$$F = \frac{[(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{\sin \alpha + C_s \cos \alpha} \quad (6.4)$$

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

$$F = (1 - r_u) \frac{\tan \phi}{\tan \alpha} \quad (6.5)$$

SIMPLIFIED METHODS FOR PLANE FAILURE

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 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 $H \csc \alpha$ instead of $a \sec \alpha$, the factor of safety is

$$F = \frac{\bar{c}H \csc \alpha + W[(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{W \sin \alpha + C_s W \cos \alpha} \quad (6.6)$$

and

$$W = \frac{1}{2} \gamma H^2 \csc \beta \csc \alpha \sin (\beta - \alpha) \quad (6.7)$$

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

$$F = \frac{2 \sin \beta \csc (\beta - \alpha) (\bar{c}/\gamma H) + [(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{\sin \alpha + C_s \cos \alpha} \quad (6.8)$$

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 height, H ,

$$F = \frac{\bar{c}W_f \sin \beta / \sin (\beta - \alpha) + W[(1 - r_u) \cos \alpha - C_s \sin \alpha] \tan \phi}{W \sin \alpha + C_s W \cos \alpha} \quad (6.9)$$

FIGURE 6.2. Plane failure of triangular fill.

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

ADMIX vs HDT GEOMEMBRANE

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

GEOSYNTHETICS LABORATORY
 DIRECT SHEAR/FRICTION TEST
 GOLDR ASSOCIATES

Westinghouse

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

SANDY SOIL vs. TEXNET GEOCOMPOSITE

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

GEOSYNTHETICS LABORATORY
 DIRECT SHEAR/FRICTION TEST
 GOLDR ASSOCIATES

Westinghouse

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

GEOSYNTHETIC INTERFACE TEST

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 5Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Liner Tension Caused by Thermal Contraction

Name

Date

Prepared By: Timothy L. Martin December 14, 1989Checked By: Fay S. Shari 2-5-90Approved By: KATIA GIBER 2-5-90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval
_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____
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_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____

**Golder
Associates**

SUBJECT Liner tension caused by Thermal contraction

Job No. W93-1186.01A

Made by RLM

Date 12-14-89

Rel.

Checked FIS

Sheet 2

of 2 ⁵ ₉₆

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 $9 \times 10^{-5} \frac{\text{in}}{\text{in}^\circ\text{F}}$ $E = 113,000 \text{ psi}$

• Assume a temperature differential of 100 F.

• 60 mil HDPE (textured), S.G. = 0.945 - w/o carbon black, but Gerdau

• ϕ HDPE/Admix $\approx 20^\circ$ - assumed ϕ S.G. = 0.94 (typ.), so OK. ϕ _{assumed ϕ S.G.}

Geometry:

total panel slope length: 70 FT

$$(70 \text{ Ft})(12 \text{ in}) = 840 \text{ in}$$

$$\Delta L = (840 \text{ in})(9 \times 10^{-5} \frac{\text{in}}{\text{in}^\circ\text{F}})(100^\circ\text{F})$$

$$\frac{\Delta L}{L} = \frac{7.56}{840} = 0.009 < 10\%, 0-10\% \text{ strain is the elastic range,}$$

$$\Delta L = 7.56 \text{ inches}$$

See attached table!

ASTM D638

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:

$$\text{The weight of the liner, } W = \left(\frac{60 \text{ mils}}{1000 \text{ mils}} \right) \left(\frac{1 \text{ Ft}}{12 \text{ in}} \right) (0.945 \times 62.4) (70 \times 1) = 20.64 \text{ lbs of width}$$

$$\text{tangential force, } W_t = 20.64 \sin 18.4^\circ = 6.52 \text{ lbs}$$

Downslope force

$$\text{normal force, } W_n = 20.64 \cos 18.4^\circ = 19.58 \text{ lbs}$$

Acts like drag force

$$\text{Resisting force, } F_R = 19.58 \tan 20^\circ = 7.13 \text{ lbs}$$

$$\text{Total force: } W_t + F_R = 6.52 + 7.13 = 13.65 \text{ lbs / ft}$$

Assume
not anchored
at top.

**Golder
Associates**

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

Reviewed JKH

of 5
Mar

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 occurrence, what stress would the liner exhibit?

$$E = \frac{\sigma}{\epsilon} \quad \text{given that } E = 113,000 \text{ psi}$$

$$\sigma = (113,000)(0.009)$$

$$\sigma = 1,017 \text{ psi} \quad \checkmark$$

In terms of anchor trench capacity,

$$(0.060 \text{ in})(12) = 0.72 \text{ in}^2$$

$$(0.72 \text{ in}^2)(1,017 \frac{\text{lbs}}{\text{in}^2}) = \underline{\underline{732 \text{ lbs/Ft}}}$$

Conclusion: During construction it is not practical to construct an anchor trench with $\approx 700 \text{ lbs/Ft}$ 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. /

TABLE 5. MATERIAL PROPERTIES
HIGH DENSITY POLYETHYLENE (HDPE)

made by: TKM
checked: FSS
Reviewed: JLC
4/21/85

Property	Test Method	Unsupported(U)						
		30	36	40	50	60	80	100
Gauge (nominal)	—	30	36	40	50	60	80	100
Thickness, mils minimum $\pm 10\%$	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)		50	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*

Bonded Seam Strength (factory seam, breaking factor, ppl 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 ^a	FTB ^a	FTB ^a	FTB ^a	FTB ^a	FTB ^a	FTB ^a
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								
Peel Adhesion	ASTM D3083 (as modified in Appendix A)	FTB ^a	FTB ^a	FTB ^a	FTB ^a	FTB ^a	FTB ^a	FTB ^a
Bonded Seam Strength (percent change maximum in original value)		-10	-10	-10	-10	-10	-20	-20

*Factory bonded seam strength is the responsibility of the fabricator. Factory seams are further discussed in Item 4.2.

^aFTB - 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 regarding installation requirements, exposure conditions, performance expectations, and experience factor.

Sub No: 115-115000-001
 Made by: TLM
 Checked: FSS
 Reviewed: JDL

5/1/85

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.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 3Project Title: Non-Drag-OFF Landfill DesignProject No. 893-1186.032Calculation Title: Anchor Trench Pullout Resistance

Name

Date

Prepared By: Timothy L. Martin December 14, 15, 1989Checked By: Frank S. Shui 2-5-90Approved By: KATIE C. CORRE 2-5-90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

**Golder
Associates**

SUBJECT Calculation of Anchor Trench Pullout Resistance

Job No. 893-1186.035

Made by TLM

Date 12-14-89

Ref.

Checked RS see note

Sheet 2 of 3

Reviewed gld

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: 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° .

$$\gamma_s = 120 \text{ pcf}$$

$$F.S. = 1.5$$

Calculate Resisting Forces

$$RF_A = q \cdot L_c \cdot \tan \delta$$

$$= (432)(3.0)(\tan 20)$$

$$\text{where, } q = (120)(3.6) = 432 \text{ psf}$$

$$RF_A = 471.7 \text{ lbs/ft}$$

$$RF_B = q \cdot L_s \cdot \tan \delta$$

$$= (348)(5.2)(\tan 20)$$

$$\text{where, } q = (120)(2.9) = 348 \text{ psf}$$

$$RF_B = 791.6 \text{ lbs/ft}$$

$$RF_C = q \cdot L_c \cdot \tan \delta$$

$$= (240)(3.0)(\tan 20)$$

$$\text{where, } q = (120)(2) = 240 \text{ psf}$$

$$RF_C = 436.8$$

$$\text{Total Resisting Force} = \Sigma RF_{A+B+C} = 1700.1 \text{ lbs/ft}$$

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° , i.e., $\tan^{-1}(1/3) = 18.4^\circ$. The one possibility for a driving

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SUBJECT Calculations of Anchor Trench Pullout Resistance

Job No. 892-1186-035

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Checked FSS

Sheet 3 of 3

Reviewed JLD

(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 Contractions". 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 underlying soil does not move with it, the membrane/soil interaction is a double shear situation, and the anchorage capacity would be $2 \times 1700 = 3400 \text{ lbs/ft}$. 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 (110% strain - which may be unacceptably high) is 1440 lbs/ft for 60 mil HDPE (see spec sheet attached to thermal expansion calc.). 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 loaded. 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. Anchorages 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.

FSS

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11Project Title: Non-Drag-off Land Fill DesignProject No. 893-1186.032Calculation Title: Ramp Stability Analysis

Name

Date

Prepared By: Timothy L. Martin January 30, 1990Checked By: Frank S. Shuri January 31, 1990Approved By: Patrick G. Golder Jan 31, 1990

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

RAMP STABILITY ANALYSIS

Jan-90

PROJECT NUMBER: 893-1184

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:

 W_s = Weight of the road base W_v = Weight of the vehicle F_b = Vehicle braking force ($0.3 \times W_v$)

DEFINE RESISTING FORCES:

 F_r = Frictional resistance at base of roadway $= (W_v + W_s)(\cos \text{ slope angle})(\tan \Phi)$ F_a = Adhesion force $= \text{roadway width} \times \text{roadway length} \times \text{cohesion}$ FACTOR OF SAFETY = $(F_r + F_a) / ((W_s + W_v)(\sin \text{ slope angle}) + F_b)$

ANALYSIS DATA:

(Based on laboratory geosynthetic interface testing program)

Equipment weight: 200000 pounds (100 ton load; largest load estimate by WHC)

Friction/Adhesion:

 Φ

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

friction: 36 degrees

c: 0 pcf

ANALYSIS:

 W_s = 1950000 pounds W_v = 200000 pounds F_b = 60000 pounds F_a = 0 pounds F_r = 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
 SUBJECT NUMBER: 893-1186

Prepared by: TLM
 Checked by: FAS
 Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste
 Landfill Cells and Surface Impoundments, Richardson, G.M.
 and Koerner, R.M.

DEFINE DRIVING FORCES:

W_s = Weight of the road base
 W_v = Weight of the vehicle
 F_b = Vehicle braking force ($0.3 \times W_v$)

DEFINE RESISTING FORCES:

F_r = Frictional resistance at base of roadway
 $= (W_v + W_s)(\cos \text{ slope angle})(\tan \Phi)$
 F_a = Adhesion force
 $= \text{roadway width} \times \text{roadway length} \times \text{cohesion}$

$$\text{FACTOR OF SAFETY} = (F_r + F_a) / ((W_s + W_v)(\sin \text{ slope angle}) + F_b)$$

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: 136 pcf

Ramp Subbase:

density: 130 pcf
 Phi: 36 degrees
 c: 0 psf

ANALYSIS:

W_s = 1950000 pounds
 W_v = 200000 pounds
 F_b = 60000 pounds

F_a = 2400000 pounds
 F_r = 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
Jan-90
PROJECT NUMBER: 893-1186

Prepared by: TLM
Checked by: FLS
Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste
Landfill Cells and Surface Impoundments, Richardson, G.M.
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

$$\text{FACTOR OF SAFETY} = (Fr + Fa) / ((Ws + Wv) * (\sin \text{slope angle}) + Fb)$$

ANALYSIS DATA: (Based on laboratory geosynthetic interface testing program)
Equipment weights: 200000 pounds (100 ton load; largest load estimate by WHC)

Friction/Adhesion:	Fhi	C
HDPE/geotextile	12	195 Residual Strength Parameters

Rep Geometry:

width: 30 ft
length: 250 ft
thickness: 2 ft
slope: 4.57 degrees (8 percent)
density: 130 pcf

Rep 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
 SUBJECT NUMBER: B93-1186

Prepared by: TLM
 Checked by: RS
 Reviewed by:

Ref: "Geosynthetic Design Guidance for Hazardous Waste
 Landfill Cells and Surface Impoundments, Richardson, G.N.
 and Koerner, R.M.

DEFINE DRIVING FORCES:

W_s = Weight of the road base
 W_v = Weight of the vehicle
 F_b = Vehicle braking force ($0.3 \cdot W_v$)

DEFINE RESISTING FORCES:

F_r = Frictional resistance at base of roadway
 $= (W_v + W_s)(\cos \text{ slope angle})(\tan \Phi)$
 F_a = Adhesion force
 $= \text{roadway width} \cdot \text{roadway length} \cdot \text{cohesion}$

$$\text{FACTOR OF SAFETY} = (F_r + F_a) / ((W_s + W_v)(\sin \text{ slope angle}) + F_b)$$

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
HDFE/geocomposite	22	168 Residual $\approx$ 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:

W_s =	195000 pounds	
W_v =	200000 pounds	
F_b =	60000 pounds	
F_a =	1410000 pounds	
F_r =	865895 pounds	
FS =	13.29	static stability
FS =	9.64	dynamic stability (braking force)

Recommended minimum FS = 3 static analysis
 FS = 2 dynamic analysis

**Golder
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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?

$$FS = \frac{(W_1 + W_2) \cos \phi}{(W_1 + W_2) \sin \phi}$$

$$FS = \frac{1}{\tan \phi}$$

$$\phi = \text{ATAN}(FS \tan i)$$

$$= \text{ATAN}(3 \times \tan 4.57)$$

$$= 13.5^\circ$$

What ϕ gives $FS = 2$ for dynamic?

$$FS = \frac{(W_1 + W_2) \cos \phi}{(W_1 + W_2) \sin \phi + F_R}$$

$$\tan \phi = \frac{FS((W_1 + W_2) \sin \phi + F_R)}{(W_1 + W_2) \cos \phi}$$

$$= 12.2^\circ$$

So any system w/ $\phi > 13.5^\circ$ is OK.

Lab results are much higher than this, even allowing for uncertainty, so design is acceptable.

Cell Component: RAMP			
Consideration: <u>Sliding</u> - verify that ramp sub-base is stable under load.			
Required Material Properties	Range	Test	Standard
S _m - LCR friction angle, δ_m	20-35°	Direct Shear	ASTM D1586
S _{ml} - FHL friction angle, δ_{ml}	14-25°		
LCR - FHL friction angle, δ_{ml}	10-15°		
Data for Example			
Analysis Procedure: (1) <u>Define Driving Forces</u> W_1 = weight of roadway. W_2 = weight of vehicle. F_B = vehicle braking force x 0.5 W_2 . (2) <u>Define Resisting Forces</u> F_R = Friction force @ base of roadway $= (W_1 + W_2) \times \cos \beta \times \tan \delta_{min}$ where δ_{min} is the minimum friction interface angle. F_A = Adhesion Force $= W_2 \times C_{min}$ where C_{min} is the minimum interface adhesion. (3) <u>Define Design Ratio, DR</u> $DR = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$ $DR = \frac{F_A + F_R}{(W_1 + W_2) \times \sin \beta + F_B}$ (assuming W/F_B)			
Design Ratio:		References:	
$DR_{min} = 3.0$ with 5 mm LCR 2.0 with 10 mm LCR			

EPA IV-13

Example:

Given:

- Equipment = 55 Ton PAV
- Friction/Adhesion:

	S	C
Soil-LCR	31°	20 MP
Soil-FHL	15°	50 MP
FHL-LCR	12°	0

- Ramp Geometry: Width = 10' Thickness = 24" $\beta = 0^\circ$
- Ramp Slope: $\phi = 36^\circ$ $C = 140 MP$

(1) Define Driving Forces

- W_1 - Weight of Roadway = $55 \times 10 \times 150 = 82,500$ LBS
- W_2 - Vehicle Weight = 55,000 LBS
- F_B - Braking Force = $0.5 \times 55,000 = 27,500$ LBS

(2) Define Resisting Forces

$$S_{min} = 12^\circ \quad C_{min} = 0 \rightarrow F_R = 0$$

$$F_R = (82,500 + 55,000) \times \cos 0^\circ \times \tan 12^\circ$$

$$= 170,900 \text{ LBS}$$

(3) Define Design Ratio, DR

$$DR_{static} = \frac{170,900}{(82,500 + 55,000) \sin 0^\circ} = \frac{170,900}{0} = 1.51 \text{ MS}$$

$$DR_{dynamic} = \frac{170,900}{113 + 33} = \frac{170,900}{146} = 1.17 \text{ MS}$$

Recommend placing small layer of sand between LCR and FHL to increase friction angle S_{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

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

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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

GEOSYNTHETIC INTERFACE TEST

C.12-11

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 4Project Title: Non-Drag-off Landfill DesignProject No. 993-1186.032Calculation Title: Bearing Capacity of Subgrade soils

Name

Date

Prepared By: Timothy L. Martin 11-30-89Checked By: Frank S. Shuri 2-5-90Approved By: W. H. C. Golder 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 TSF

Sheet 2 of 4

Reviewed RL

Objective:	To determine maximum allowable bearing capacity of onsite native soils.		
Reference:	Principles of Foundation Engineering Braj M. Das page 112, Table 3.2 Foundations and Earth Structures Naval Facilities Engineering Command Design Manual 7.02 page 7.2-42, Table 1		
	$q_u = C N_c + 2 N_q + \frac{1}{2} \gamma B N_\gamma$ where: $C = 0$ GAI lab tests $\gamma = 120 \text{ pcf (moist)}$ $\phi = 38^\circ$		
	$N_c = 61.35$ $N_q = 48.93$ $N_\gamma = 78.03$		
	(From Prin. of Found. Engr.) } Eolian Sands		
Assumptions:	1). Embedment Depth of foundation = 1.5 Ft 2). $FS = 3$ 3). $B = 1 \text{ Ft}$		
	$q = (120)(1.5) = 180 \text{ psf}$		
	$q_u = (120)(1.5) + \frac{1}{2} (120)(1)(78.03)$ $= 8,807.4 + 4,681.8$ $q_u = 13,489 \text{ psf}$		
	$q_{net} = q_u - q$ $= 13,489 - 180$ $= 13,309 \text{ psf}$		
	$q_{allowable} = \frac{q_{net}(w)}{FS}$ $= \frac{13,309}{3}$ $= 4,436 \text{ psf}$		
	$q_{allowable} = 4,436 \text{ psf} \Rightarrow 4000 \text{ psf}$ For compacted structural fill in eolian sands, 95 percent of ASTM D 1557		
Other soils:	Dense native sands and gravels (Hanford Formation) $\Rightarrow q_{allowable} = 6000 \text{ psf}$ * Proof rolled Silty sands (Eolian sands) $\Rightarrow 5,000 \text{ psf}$ *		
	* after DM 7.02 page 7.2-142		

Table 3.2. Bearing Capacity Factors*

ϕ	N_c	N_q	N_γ	N_c/N_q	$\tan \phi$
0	5.14	1.00	0.00	0.20	0.00
1	5.34	1.00	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	6.40	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	5.23	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	10.60	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	25.90	0.63	0.60
32	35.40	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	42.82	66.19	0.77	0.75
38	61.35	48.63	78.03	0.80	0.78
39	67.87	55.00	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.80	73.00	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	115.01	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	762.80	1.20	1.19

*After Vesic (1973)

Principle of Foundation Engineering
 Braja M. Das

TABLE 1
Presumptive Values of Allowable Bearing Pressures for Spread Footings

Type of Bearing Material	Consistency In Place	Allowable Bearing Pressure Tons Per sq ft	
		Range	Recommended Value Used
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	6 to 10	7.0
	Medium to compact	4 to 7	5.0
	Loose	2 to 6	3.0
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	4 to 6	4.0
	Medium to compact	2 to 4	3.0
	Loose	1 to 3	1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact	3 to 5	3.0
	Medium to compact	2 to 4	2.5
	Loose	1 to 2	1.5

DM-7.02

7.2-142

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 7Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Bearing capacity of liner soils

Name

Date

Prepared By: Timothy L. Martin January 10, 1990Checked By: Frank S. Shiri 2-5-90Approved By: MARK COBB 2-5-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

Golden Associates

SUBJECT: Bearing Capacity of Liner Soils

Job No. 893-1146.03a

Made by TLM
Checked by JAL
Reviewed by JAL

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

$$q_{ult} = 8DN_2 + 8B N_2$$

(Figure 1)

assumed 8×120 pcf

D = 0 (depth of embedment)

B = 5.0 ft (from attached figure sketch)

$\phi = 58^\circ$ (from laboratory tests)

$N_2 = 80$ (from Fig. 1, attached, DM 7.2)

$$q_{ult} = (120)(5)(80) = 24,000 \text{ pcf}$$

B) Rectangular Footing

$$q_{ult} = 8DN_2 + 0.4(80)N_2$$

(Same conditions as per continuous footing)

$$q_{ult} = (0.4)(80)(80)$$

$$q_{ult} = 19,200 \text{ pcf}$$

Allowable Bearing Capacities for Factor of Safety = 3.0

Sand Layer

A) Continuous Footing

$$q_{allow} = \frac{24,000}{3} = 8,000 \text{ pcf}$$

B) Rectangular Footing

$$q_{allow} = \frac{19,200}{3} = 6,400 \text{ pcf}$$

**Golder
Associates**

SUBJECT Bearing Capacity of Liner Soils

Job No. 813-1186.032

Made by TLM

Date 1-10-90

Ref.

Checked FS

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: $C = 1000 \text{ psf}$ (Based on pocket pen. $\approx 2000 \text{ psf}$, $S_u = q_{un}/2$) ✓

$D_f = 0$ ✓

$B' = 9.6 \text{ ft}$ (From sketch below, Ref. DM-7.2 pg (45)) ✓

$N_c = 5.53$ (From Fig 1, attached, DM 7.2) ✓

Assume $B = L$ $B' = L'$

$B = 5 \text{ ft}$ $B' = B + 1.16H$

$B' = 5 + 1.16(4)$

$B' = 9.6 \text{ ft}$ ✓

$$q_{ult} = CN_c + \gamma D$$

$$= (1000)(5.53)$$

$$q_{ult} = 5,530 \text{ psf}$$
 ✓

B) Rectangular Footing

$$q_{ult} = CN_c (1 + 0.3 \frac{B}{L}) + \gamma D$$

$$q_{ult} = (1000)(5.53)(1 + 0.3)$$

$$= 7,189 \text{ psf}$$
 ✓

Allowable Bearing Capacities For Factor of Safety = 3.0

Class Layer

A) Continuous Footing

$$q_{allow} = \frac{5,530 \text{ psf}}{3} = 1,843 \text{ psf}$$
 ✓

B) Rectangular Footing

$$q_{allow} = \frac{7,189 \text{ psf}}{3} = 2,396 \text{ psf}$$
 ✓

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.)

$$(B' \cdot L') = 92.93 \text{ ft}^2$$
 ✓

If Q = Foundation load, then

$$\frac{Q \text{ pounds}}{92.93 \text{ ft}^2} \leq q_{allow} \text{ psf}$$
 ✓

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SUBJECT Bearing Capacity of Liner Soils

Job No. 893-1186.032

Made by TLM

Date 1-10-90

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Sheet 4 of 7

Reviewed RL

Calculate load due to concrete barrier blocks

Assume: $\gamma_{\text{concrete}} = 50 \text{ pcf}$ (Civil Engineering Reference Manual, Lindburg, M.R.)
Height of stacked blocks = 12.5 ft. (see attached sketch)

$$Q_{\text{concrete}} = (50 \text{ pcf})(12.5 \text{ ft}) = 1,875 \text{ psf}$$

The Q_{concrete} stress is transferred 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 $(\frac{B_1}{B_2})$.

Calculate the foundation pressure applied to the admix; (assume B.F.L.)

$$Q_{\text{actual on admix}} = (Q_{\text{concrete}})(\frac{B_1}{B_2}) = (1,875 \frac{\text{lbs}}{\text{ft}^2})(\frac{5/5}{9.64/9.64}) = 504 \text{ psf}$$

Summary of Bearing Capacities

Soil	<u>Put</u>		<u>Q_{allow} (FS=3.0)</u>		
	<u>continuous</u>	<u>Rectangular</u>	<u>continuous</u>	<u>Rectangular</u>	
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	✓
<u>Factor of Safety Summary</u>					
Barrier Block Load: 1,875 psf					
<u>Sand:</u>					
Continuous:	FS = $\frac{24000}{1,875} = 12.8$				✓
Rectangular:	FS = $\frac{19,200}{1,875} = 10.2$		(Note: all FS > 3.0)		✓
<u>Admix:</u>					
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

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FIGURE 1
Ultimate Bearing Capacity of Shallow Footings With Concentric Loads

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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.)

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DETAIL BARRIER BLOCK WALL

0 4 8
SCALE IN FEET

H-2-131587

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Calculation Cover Sheet

Sheet 1 of 11Project Title: Non-Drag-off Landfill Design ProjectProject No. 893-1186.032Calculation Title: Stability Checks on Barrier Block Wall

Name

Date

Prepared By: Timothy L. Martin January 10-13, 1990Checked By: Frank S. Shari 2-5-90Approved By: Mark G. Golder 2-5-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

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SUBJECT Stability Checks on Barrier Block Wall

Job No. 893-1186-032

Made by TLM

Date 1-10-90

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Sheet 2 of 11

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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: $\delta = 11.5^\circ$ (assuming loose condition)

$\phi = 33^\circ$ (assuming loose condition)

$H = 12.5$ ft (height of barrier wall)

$\gamma_{\text{soil}} = 150$ pcf

Rankine Active force per unit length of barrier wall

$$P_a = \frac{1}{2} \gamma H^2 K_a \quad \checkmark$$

K_a = active earth pressure coefficient

$$= \tan^2(45 - \phi/2)$$

$$= \tan^2(45 - 33/2)$$

$$= 0.295 \quad \checkmark$$

$$P_a = (\frac{1}{2} \times 115)(12.5)^2(0.295)$$

$$P_a = 2,648.6 \approx 2,650 \text{ lbs/ft of wall length} \quad \checkmark$$

Check for overturning

$$FS_{\text{overturning}} = \frac{\sum m_R}{\sum m_o} \quad \text{about Point A}$$

$$\sum m_o = \frac{\text{force} \times \text{moment arm}}{(P_a)(H/3)}$$

$$= (2,650)(12.5/3)$$

$$= 11,041.67 \text{ lbs} \quad \checkmark$$

$$\sum m_R = \frac{\text{force} \times \text{moment arm}}{(5)(12.5)(150)(2.5)}$$

$$= 23,437.5 \text{ lbs} \quad \checkmark$$

$$FS_{\text{overturning}} = \frac{23,437.5}{11,041.7} \approx 2.1 \quad \checkmark$$

Check for sliding

A) At base of the wall

$$FS_{\text{sliding}} = \frac{\sum F_R}{\sum F_d}$$

$$\sum F_d = P_a \quad (\text{Assuming horizontal backslope}) = 2,650 \text{ lbs/ft of wall length} \quad \checkmark$$

$$\sum F_R = \sum V \tan \phi + B \phi + P \phi$$

$$= (5)(12.5)(150) \tan 33^\circ$$

$$= 6,088 \text{ lbs/ft} \quad \checkmark$$

$$FS_{\text{sliding}} = \frac{6,088}{2,650} \approx 2.3 \quad \checkmark$$

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SUBJECT: Stability Checks on Barrier Block Wall
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B) at first geotextile/HDPE contact
Assum: geotextile/HDPE $\phi \approx 2^\circ$
(See detail R08 facing of Keller Hills)
geotextile

$$FS = \frac{\sum F_R}{\sum F_D}$$

$$\sum F_D = \frac{1}{2} \gamma H^2 K_a$$

$$\text{where } K_a = 0.295$$

$$\gamma = 115 \text{ pcf}$$

$$H = 12.5 + 3 = 15.5$$

$$P_a = \frac{1}{2} (115) (15.5^2) (0.295)$$

$$\sum F_R = \sum V \tan \phi + B R + P_r$$

$$= [(2.5)(150) + (3)(20)] \tan 8^\circ + 1830.6$$

$$= 2,144.7 \text{ lbs/ft}$$

$$\sum F_{sliding} = \frac{\sum F_R}{\sum F_D} = \frac{2144.7}{4,035.2}$$

$$= 0.53$$

Not acceptable

$$P_r = \frac{1}{2} \gamma D^2 K_p = \frac{1}{2} (120 \times 3^2) (3.39)$$

$$= 1,830.6 \text{ lbs/ft of wall length}$$

$$\text{where } K_p = \tan^2 (45 + \frac{\phi}{2})$$

$$= \tan^2 (45 + \frac{33}{2})$$

$$K_p = 3.39$$

Calculate Berm height to provide passive force to obtain $FS \geq 1.5$

$$\text{Try Berm } 2.5 \text{ ft high and } 10 \text{ ft wide}$$

$$\sum F_R = \sum V \tan \phi + B R + P_r$$

$$= [(2.5)(150) + (3)(120)] \tan 8^\circ + 6,152.9$$

$$\sum F_R = 6,467.1 \text{ lbs/ft}$$

$$\sum F_D = \frac{\sum F_R}{\sum F_D} = \frac{6,467.1}{4,075.2}$$

$$= 1.59$$

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% (assumed of 2.5% and 2.5% + 1)

Analyze stability by reducing the critical interface friction angle. $3.5\% \approx 2^\circ$, therefore analyze stability using $\delta - \alpha = 6^\circ$ for critical interface.

$$P_r = 6,152.9 \text{ lbs/ft of wall length as in part B above.}$$

$$\sum F_D = 4,075.2 \text{ lbs/ft of wall length as in part B above.}$$

$$\sum F_R = [(2.5)(150) + (3)(120)] \tan (8 - 2^\circ) + 6,152.9 = 6,327.8 \text{ lbs/ft of wall length}$$

$$FS_{sliding} = \frac{6,327.8}{4,075.2} = 1.57$$

$$OK$$

(includes berm as shown in attached sketch)

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SUBJECT Stability Checks on Barrier Block Wall

Job No. 893-1186.032

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Conclusions:

1) **Overturning Stability** - The barrier block wall as configured in the attached sketch has a calculated $F_{\text{overturning}} = 2.1$ which is above the usual minimum desirable Factor of safety with respect to overturning of 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 provided a factor of safety of 1.57 on the weakest interface, on a sloping base. This analysis was concerned with the first 1 ft of waste placed on the initial operations layer. The Factor of safety against sliding will increase as subsequent lifts of waste are placed.

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, $\frac{1}{2}K_a H'^2$ 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 estimating the active force on retaining walls supporting backfills. Engineering Practice, 2nd edition 1967 by John Wiley & Sons

Table 5.8 Types of Backfill for Retaining Walls^a

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.

^aFrom *Soil Mechanics in Engineering Practice*, 2nd ed., by K. Terzaghi and R. B. Peck. Copyright 1967 by John Wiley and Sons. Reprinted by permission.

$$P_p = \frac{1}{2} K_p \gamma_1 D^2 + 2c_2 \sqrt{K_p} D \quad (5.22)$$

where γ_1 = unit weight of soil in front of the heel and under the base slab

K_p = Rankine passive earth pressure coefficient

= $\tan^2(45 + \phi/2)$

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

$$FS_{\text{(overturning)}} = \frac{\sum M_R}{\sum M_O} \quad (5.50)$$

where $\sum M_O$ = sum of the moments of forces tending to overturn about point C

$\sum M_R$ = sum of the moments of forces tending to resist overturning about point C

The overturning moment can be given as

$$\sum M_O = R_h \left(\frac{H'}{3} \right) \quad (5.51)$$

where $R_h = P_a \cos \alpha$

For calculation of the resisting moment, $\sum M_R$ (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_p also contributes to the resisting moment. P_p is the vertical component of the active force P_a , or

$$P_p = P_a \sin \alpha \quad (5.52)$$

The moment of the force P_p about C is equal to

$$M_p = P_p B = P_a \sin \alpha \cdot B \quad (5.53)$$

where B = width of the base slab

Once $\sum M_R$ is known, the factor of safety can be calculated as

6 05/1

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permeable (clean sand or
silt size.
with conspicuous clay

way that a negligible
silt or heavy rains. If this
used as backfill material,
duration of water increases

B. Peck. Copyright 1967 by John

(5.22)

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(5.50)

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(5.51)

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, or

(5.52)

(5.53)

ted as

Figure 5.21
Check for
overturning;
assume Rankine
pressure is
valid

Table 5.9 Procedure for Calculation of ΣM_a

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	$W_2 = \gamma_1 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_1 \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_1 \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_1 \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_1 \times A_6$	X_6	M_6
		P_a	B	M_0
		ΣV		ΣM_a

Note: γ_1 = unit weight of backfill
 γ_c = unit weight of concrete

$$FS_{\text{(overturning)}} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_0}{P_a \cos \alpha \frac{H'}{3}} \quad (5.54)$$

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:

$$FS_{\text{(overturning)}} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6}{(P_a \cos \alpha) \frac{H'}{3} - M_0} \quad (5.55)$$

Check for Sliding Along the Base

The factor of safety against sliding may be expressed by the equation

$$FS_{\text{(sliding)}} = \frac{\Sigma F_R}{\Sigma F_d} \quad (5.56)$$

where ΣF_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

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Figure 5.22 Check for

$$B\sigma = \text{sum of}$$

So

$$R' = (\Sigma V)$$

As can be seen in 1 force. The expressi

$$\Sigma F_R = (\Sigma$$

The only horiz force) is the horizo

$$\Sigma F_d = P_a$$

Combining Eqs. (5

$$FS_{\text{(sliding)}} = \frac{R'}{P_a}$$

A minimum factor

In many cases of safety with respec instances. The red two-thirds of the w to the value of 0.5

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Figure 5.22 Check for sliding along the base

$B\sigma = \text{sum of the vertical force} = \Sigma V$ (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

$$\Sigma F_{R'} = (\Sigma V) \tan \phi_2 + Bc_2 + P_p \quad (5.57)$$

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

$$\Sigma F_d = P_a \cos \alpha \quad (5.58)$$

Combining Eqs. (5.56), (5.57), and (5.58)

$$FS_{(\text{sliding})} = \frac{(\Sigma V) \tan \phi_2 + Bc_2 + P_p}{P_a \cos \alpha} \quad (5.59)$$

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

$$FS_{\text{(sliding)}} = \frac{(\Sigma V) \tan(k_1 \phi_2) + B k_2 c_2 + P_p}{P_p \cos \alpha} \quad (5.60)$$

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

$$P_p = \frac{1}{2} \gamma_2 D^2 K_p + 2c_2 D \sqrt{K_p}$$

However, if a key is included, the passive force per unit length of the wall will be

$$P_p = \frac{1}{2} \gamma_2 D'^2 K_p + 2c_2 D' \sqrt{K_p}$$

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 $P_p \cos \alpha$. Let R be the resultant force, or

$$\overline{R} = \Sigma V + (P_p \cos \alpha) \quad (5.61)$$

The net moment of these forces about the point C (Figure 5.23) is equal to

$$M_{\text{net}} = \Sigma M_R - \Sigma M_O \quad (5.62)$$

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

$$\overline{CE} = \overline{X} = \frac{M_{\text{net}}}{\Sigma V} \quad (5.63)$$

Hence, the eccentricity of the resultant, R , can be expressed as

$$e = \frac{B}{2} - \overline{CE} \quad (5.64)$$

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Figure 5.23 Ch.

The pres
the simple pi

$$q = \frac{\Sigma}{I}$$

where $M_{\text{net}} =$
 $I =$
 $=$

For maximum
B/2. Substit

$$q_{\text{max}} =$$

Similarly

$$q_{\text{min}} =$$

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Calculation Cover Sheet

Sheet 1 of 6Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Elastic Settlement of Foundation Sand

Name

Date

Prepared By: Timothy L. Martin December 13, 1989Checked By: Frank S. Shuri 2-5-90Approved By: [Signature] 2-5-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

Golder Associates

SUBJECT Calculation of Elastic Settlement of Foundation

Job No. 893-1126.052

Made by TLM

Date 12-13-89

Ref. Non-Dray-off

Checked TSS

Sheet 2 of 6

Reviewed *JK*

Objective	To estimate the maximum differential settlement of the non-cohesive foundation soils of the Non-Dray-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.						
Reference:	Soil Mechanics Design Manual 7.1, DM-7.1 pg 7.1-220-223, 1982						
Assumptions:	Foundation width ≈ 100 ft Embedment: none						
	Load: 4200 psf (base), (35 ft @ 120 psf, includes lower section) 1800 psf (Top of Slope), (15 ft @ 120 psf, 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.C. sand, some silt					3	
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 695-45-78							
Side Slope	Thickness of Layer	SPT Blow Count	Soil modulus	Effective E_c layer	Influence Factor		
Layer	Δz (in)	N	E_s/N (Fig 7)	E_s (tsf)	E_c (in)	I_z (attach 2)	I_z/E_s (Δz) (in/tsf)
1	210	13	4	52	195	0.107	0.404 ✓
2	120	12	7	84	270	0.275	0.393 ✓
3	222	60	10	600	441	0.440	0.63 ✓
4	1844	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. 893-1116.032

Made by TLM

Date 12-13-89

Ref.

Checked TSS

Sheet 3 of 6

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(Side Slope settlement calculations continued)

$$\text{Total Settlement} = \Delta H = C_1 C_2 \Delta P \sum_{0}^{\Delta B} \left(\frac{I_z}{E_s} \right) (\Delta z)$$

$$\text{where } C_1 = 1 - 0.5(P_o / \Delta P), \quad C_1 \geq 0.5$$

$$C_2 = 1 + 0.2 \log(10t)$$

 P_o = overburden pressure at foundation level ΔP = net foundation pressure increase t = elapsed time in years

$$C_1 = 1.0 \quad \text{since } P_o = 0$$

$$C_2 = 1 + 0.2 \log(10 \times 30 \text{ years}) = 1.495 \quad (30 \text{ years is monitored life})$$

$$\Delta P = 1800 \text{ psf} = 0.90 \text{ tsf}$$

$$\Delta H = 1.0 (1.495) (0.90) (1.434)$$

$$\Delta H = 1.93 \text{ inch} \quad (\text{for top of slope settlement})$$

Base

Layer	Δz (in)	N SPT blowcount	E_s/N	E_s	Δz (in)	I_z	$I_z (\Delta z) / E_s$ (in/tsf)
1	20	12	7	84	60	0.065	0.093
2	222	60	10	600	231	0.230	0.085
3a	264	100	12	1800	474	0.470	0.103
3b	1794	100	12	1200	503	0.500	0.449
							0.730

$$C_1 = 1.0$$

$$C_2 = 1.495$$

$$\Delta P = 4200 \text{ psf} = 2.10 \text{ tsf}$$

$$\Delta H = (1.0) (1.495) (2.10) (0.730)$$

$$\Delta H = 2.29 \text{ inch} \quad (\text{for base settlement})$$

Conclusion:

$$\text{Maximum differential settlement} = 2.29 - 1.93$$

$$= 0.36 \approx 0.4 \text{ inch}$$

Differential settlement of the subgrade is not anticipated to impact the design of the land fill.

WHC-SD-W025-FDR-001, Rev. 1

Interim Settlement Factors for Calculating
Elastic Settlement of Foundation Sands
Project No. 813-1116.032
New-Dry-off Landfill
Made by: TLM, 12-13-89

Page 4 of 6
Checked by: FSC
Reviewed by: 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 proposed average 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 = $\Delta H = C_1 C_2 \Delta p \sum_0^{2B} \left(\frac{I_z}{E_s} \right) \Delta Z$,

where $C_1 = 1 - 0.5 (p_o / \Delta p)$; $C_1 \leq 0.5$ 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)		Soil Description
		Moist	Sat.	
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

$$\Sigma = 0.568$$

$$P_0 = (2.0 \text{ ft})(95 \text{ pcf}) = 190 \text{ psf} = 0.095 \text{ tsf}$$

$$\Delta P = 120 \text{ tons}/(6 \text{ ft.})(8 \text{ ft.}) = 2.50 \text{ tsf}$$

At $t = 1 \text{ yr.}$

$$C_1 = 1 - 0.5(.095/2.50) = 0.981$$

$$C_2 = 1 + 0.2 \log (10)(1) = 1.20$$

$$\Delta H = (0.981)(1.20)(2.50)(0.568) = \underline{1.67 \text{ in.}}$$

FIGURE 7 (continued)

Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

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 3Project Title: Non-Drag-off Land Fill DesignProject No. 893-1186.032Calculation Title: Consolidation Settlement of Compacted Admix

Name

Date

Prepared By: Timothy L. Martin December 19, 1989Checked By: Fred S. Shum 2-5-90Approved By: K. P. ACK. (GSR) 2-5-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____
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_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____

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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 JOL

Objective: To determine the consolidation settlement of the compacted admix at the base of the land fill and at the top of slope.

Approach: Utilize laboratory consolidation test curve to calculate consolidation settlements within the compacted admix.

Geometry

Primary only - ok ✓

- Assumptions:
- 1) Maximum vertical overburden stress is 35 ft from base of land fill ✓
 - 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., $H_v = 3.1 \text{ ft} = 37.20 \text{ inches}$ ✓

Base consolidation settlement

$$\Delta \sigma_v = (35)(120) = 4,200 \text{ psf} \quad \checkmark$$

From consolidation curve at 4,200 psf $\Rightarrow e = 0.04 \text{ in/in}$ ✓

$$\Delta H = (37.2 \text{ inches})(0.04 \frac{\text{in}}{\text{in}}) = 1.49 \text{ inches} \quad \checkmark$$

Top of slope consolidation settlement

$$\Delta \sigma_v = (15)(120) = 1,800 \text{ psf} \quad \checkmark$$

From consolidation curve at 1,800 psf $\Rightarrow e = 0.029 \text{ in/in}$ ✓

$$\Delta H = (37.2 \text{ inches})(0.029 \frac{\text{in}}{\text{in}})$$

$$\Delta H = 1.08 \text{ inches} \quad \checkmark$$

$$\text{Maximum Differential settlement} = 1.49 - 1.08 = 0.41 \text{ inches} \approx 0.5 \text{ inches} \quad \checkmark$$

Conclusion: Consolidation settlement does not appear to be a problem ✓

Boring	Boring	Soil Classification	Moisture Before	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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Calculation Cover Sheet

Sheet 1 of 2Project Title: Non-Drag-off Landfill DesignProject No. 893-1186-032Calculation Title: Settlement Summary - elastic and consolidation settlements.

Name

Date

Prepared By: Timothy L. Martin January 10, 1990Checked By: Frank S. Shum 2-5-90Approved By: MARK. LOER 2-5-90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

Reviewed *gd*

Conclusion: The combined total and differential settlements are not anticipated to adversely affect the design and operation of the land fill.

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 2Project Title: Non-Drag-OFF Landfill DesignProject No. 893-1186.032Calculation Title: Subsidence and Sinkhole Potential

Name

Date

Prepared By: Timothy L. Martin January 9, 1990Checked By: Frank S. Shari 2-5-90Approved By: Mark C. Bell 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: *FSS*
Reviewed by: *qut*

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 Interim 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 petroleum 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 occurred 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 2Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Potential for uplift from Gas or Hydraulic Pressures

Name

Date

Prepared By: Timothy L. Martin January 9, 1990Checked By: Frank S. Shui 2-5-90Approved By: Frank S. Shui 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: ~~TSM~~Reviewed by: ~~RM~~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 Interim 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 10Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Earth Load - Primary Slope Riser Pipe

Name

Date

Prepared By: Timothy L. Martin January 5, 1990Checked By: Frank S. Skuri 2-5-90Approved By: [Signature] 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

Made by TLM

Date 1-5-90

Ref.

Checked FES

Sheet 2 of 10

Reviewed JEL

Objective:	To calculate the earth load being applied to the side slope riser pipes and to select a diameter and SDR rating for the pipe.
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:	
$W = C_d \times w_s \times B_d \times D$	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, lb/ft ³
<u>Trial 1</u>	
Assumptions:	
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 operations layer 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 8 inch nominal diameter, SDR 11, i.e., $D = 8.625/12 = 0.719$ ft	
$H/B_d = 35/6 = 5.8 \approx 6 \Rightarrow$ From Fig. 16 $C_d \approx 3.1$	
$W = (3.1)(120 \frac{lb}{ft^3})(6 ft)(0.719 ft) = 1,604.25 \text{ lbs/ft} \approx 1,604 \text{ lbs/ft}$	
For Flexible HDPE Pipe, the deflection under earth loading is:	
$\Delta X = \frac{DI \times K \times W}{3E} + 0.061 E'$	where ΔX = vertical deflection of pipe in inches DI = 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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Associates**

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. M. M. M.

Sheet 3 of 10

Reviewed RL

$$\Delta X = \frac{(1.50)(0.10)(134 \text{ lbs/in})}{(2)(30,000) + (0.06)(300)} + \frac{(3)(11^3)}{33.3}$$

Note: $E = 30,000$ ✓
 $E' = 300$ psi assumed
 sand tamped to
 65% of max. density
 $W = (160 + 14 \frac{Y}{Ft}) \cdot 13.4 \frac{lb}{Ft}$

$$\Delta X = \frac{20.1}{33.3} = 0.603 \text{ in}$$

$$\frac{\Delta X}{X} = \frac{0.603 \times 100\%}{8} = 7.6\% > 5\% \quad \underline{\text{N.G.}}$$

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., $B_1 = 3 \text{ Ft.}$
 All other conditions and assumptions remain the same.

$$H/B_1 = \frac{35}{3} \approx 12 \Rightarrow \text{From Fig. 16 } C_d \approx 3.7$$

$$W = (3.7)(120)(3)(0.719)$$

$$W = 957.7 \approx 958 \text{ lbs/Ft} \approx 80 \text{ lbs/in}$$

$$\Delta X = \frac{(1.50)(0.10)(80)}{(2)(30,000) + (0.06)(300)} + \frac{(3)(11^3)}{33.3} = \frac{12}{33.3} = 0.36 \text{ inch}$$

$$\frac{\Delta X}{X} = \frac{0.36}{8} = 4.5\% < 5\% \quad \underline{\text{OK}}$$

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 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 but fused polyethylene system although a reduction
 in flow would be noted. It would also be difficult to get 100%
 conventional cleaning equipment with this severe deflection. For these
 reasons, Poly. Pipe recommends maximum deflections of 5 to 7%."
 Calculations in this guide have been based on 5%.

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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.

$$W = C_d \times w \times B_d \times D \quad (10)$$

- 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. Safest 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 1 1/2 inch in diameter. This material can best be described as Class I or II of ASTM D2321 "angular 1/4 to 1 1/2 inch graded stone, slag, clinders, 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.

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.

Where: Δx = defect
0 = nominal

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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:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3 (SDR)^3} + 0.061 E'} \quad (11)$$

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

$$\% \text{ def} = \frac{\Delta x}{D} \times 100 \quad (12)$$

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	5990	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			

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-

Classification of Backfill Material

per ASTM D2321*

Non-Dreg-off

893-1186

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Class

Comments

Class I — Angular graded stone, $\frac{1}{4}$ to $1\frac{1}{2}$ 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 $\frac{1}{2}$ to $\frac{3}{4}$ -inch for ease of placement.

Class II — Coarse sands and gravels with maximum particle size of $1\frac{1}{2}$ 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 $\frac{1}{2}$ to $\frac{3}{4}$ -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 $1\frac{1}{2}$ 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.

1/10/11
893-1186

Fig. 16

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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. Shari 2-5-90

Approved By: KEVIN. CORRE 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 FSO

Sheet 2 of 10

Reviewed RL

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 loadings.

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/ft³

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 (0.375 ft).

$$H/B_d = \frac{35}{3} = 11.67 \approx 12 \Rightarrow \text{From Fig 16, } C_d \approx 3.7$$

$$W = (3.7)(120)(3)(0.375) = 499.5 \text{ lbs/ft} \approx 500 \text{ lbs/ft} \approx 42 \text{ lbs/inch}$$

For flexible HDPE pipe, the deflection under Earth loading is:

$$\Delta x = \frac{DI \cdot K \cdot W}{2E} \approx 0.0615' = 0.738 \text{ inches}$$

where: Δx = vertical deflection of pipe in inches

DI = 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, p

E' = soil modulus, psc

$SDR = OD/t$

**Golder
Associates**

SUBJECT Calculation of Earth Load - Primary Drainage Pipe

Job No. 893-1186.032

Made by TLM

Date 1-11-90

Ref.

Checked TCD

Sheet 3 of 10

Reviewed RCL

(continued)

$$\Delta X = \frac{(1.50 \times 0.10)(42)}{(2)(30,000) + 0.061(300)} \times 3(11)$$

Note: $E = 30,000$ psi

$E' = 300$ psi (assumed - hand tamped to 65% of max. density.)

$$\Delta X = 0.189 \text{ inch}$$

$$\frac{\Delta X}{X} = \frac{0.189}{4} = 0.047 \times 100\% = 4.7\% < 5\% \therefore \text{OK}$$

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 much a deflection. For these reasons, Poly Pipe recommends maximum deflections of 5 to 7%." Calculations in this guide have been based on 5%."

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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.

$$W = C_d \times w \times B_d \times D \quad (10)$$

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 1 1/2 inch in diameter. This material can best be described as Class I or II of ASTM D2321 "angular 1/4 to 1 1/2 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.

Hand or mechanical tam-
ing the soil pressure on the pipe
support a greater portion of
adequate stiffness in the side-
extension in the horizontal.
upon the selection, placement
Under normal circumstances
deflection can be calculated

Where: Δx = deflection

Δx = deflection
factor, K
bedding, C
load on pipe, W
mean radius, R
modulus, E
moment, I
where T = modulus, E

This equation is known as 1
Modified for flexible pipe

Where: Δx = vertical deflection
 Δx = deflection
bedding, K
earth for modulus, E
soil modulus, E
OD, SOR

Note: Values of C = deflection
factor of Practice No. 9
to 65% of maximum density.

from the above formula
can then be calculated by

Where: Δx = deflection
 Δx = nominal

5 of 10

G-2

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 trench 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 load 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. 16 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 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:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3 (SDR)^3} + 0.061 E'} \quad (11)$$

- 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

$$\% \text{ def} = \frac{\Delta x}{D} \times 100 \quad (12)$$

- 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	5990	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			

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

Non-Drag-off
893-1186

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Classification of Backfill Material

per ASTM D2321*

Class**Comments**

Class I — Angular graded stone, $\frac{1}{4}$ to $1\frac{1}{2}$ 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 $\frac{1}{2}$ to $\frac{3}{4}$ -inch for ease of placement.

Class II — Coarse sands and gravels with maximum particle size of $1\frac{1}{2}$ 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 $\frac{1}{2}$ to $\frac{3}{4}$ -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 $1\frac{1}{2}$ 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 3Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Down-Drage Analysis For Vertical Buser Pipe

Name

Date

Prepared By: Timothy L. Martin December 8, 1989Checked By: Frank S. Shum 2-5-90Approved By: HAZEL COOPER 2-5-90

QA Approval: _____

Revisions:

Date

By

Checked By

Approved By

QA Approval

_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____
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**Golder
Associates**

SUBJECT Down-Drain Analysis for vertical riser pipe

Job No. 893-1146.032

Made by TLM

Date 12-8-89

Ref.

Checked FSS

Sheet 2 of 3

Reviewed VL

Objective	To determine the downward drag force on the vertical riser pipe.
Reference:	Principles of Foundation Engineering, Braja M. Das, pg. 408.
Assumptions:	<ul style="list-style-type: none"> outside diameter of the vertical riser pipe is 4.5 ft. Smooth HDPE will be placed against the riser pipe to reduce skirt friction. Assume a friction angle between the concrete and the HDPE of 18 degrees. (0.31) Assume 35 ft total fill height
$F_n = k' \sigma_v' \tan \delta$, where	F_n = downward skin stress $k' =$ earth pressure coefficient $= k_0 = 1 - \sin \phi$ σ_v' = vertical effective stress at any depth $= \gamma_f Z$ δ_f' = Effective unit weight of fill δ = HDPE - concrete friction angle
Total downward drag force, $Q_n = \frac{\pi d k' \delta_f' H_f^2 \tan \delta}{2}$	$H_f = 1.0 \times 35 = 35$ $\delta = 0.31$
where H_f = height of the fill	
$Q_n = \frac{\pi (4.5) (1 - \sin 38^\circ) (120) (35)^2 \tan 18^\circ}{2}$	
$Q_n = 129,760$ pounds	
Weight of concrete riser pipe section	
4 ft nominal diameter	$(4.7) \times (\frac{2.5}{2}) \times (4) (150) = 1,571$ pounds/section
2.5 inches thick	
36 ft length (9 sections)	Total of 9 sections = 14,139 pounds
Concrete = 150 pcf	Base = $(6' \times 6' \times 0.5') (150)$ = 2,700 pounds
Total Force = self weight + down drag	
= 16,839 + 129,760	
= 146,599 pounds	
Total stress on soil = $146,599 / 36 = 4,072$ psf	
Stress on clay layer assuming a 1:1 stress distribution, clay under 1.5' gravel	
$146,599 / (9) = 1,812$ psf	
Note: overburden stress of fill 35 ft thick = $4,200$ psf = $(35') (120 \text{ pcf})$	
Conclusion: Down drag plus self weight will not exceed overburden stress of fill.	

408 Chapter 8 Pile Foundations

Job No. 843-1186.032
Date 12-8-89
Made By: TLM
Checked By: FOS
Reviewed By: JCL

Sheet 3 of 3

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

$$f_s = K' \sigma'_v \tan \delta \quad (8.109)$$

where $K' =$ earth pressure coefficient $= K_o = 1 - \sin \phi$
 $\sigma'_v =$ vertical effective stress at any depth $z = \gamma'_f z$
 $\gamma'_f =$ effective unit weight of fill
 $\delta =$ soil-pile friction angle $\approx 0.5-0.7\phi$

Hence, the total downward drag force, Q_n , on a pile is

$$Q_n = \int_0^{H_f} (pK' \gamma'_f \tan \delta) z \, dz = \frac{pK' \gamma'_f H_f^2 \tan \delta}{2} \quad (8.110)$$

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)

$$L_1 = \frac{(L - H_f)}{L_1} \left[\frac{L - H_f}{2} + \frac{\gamma'_f H_f}{\gamma'} \right] - \frac{2\gamma'_f H_f}{\gamma'} \quad (8.111)$$

where γ'_f and $\gamma' =$ 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

$$f_s = K' \sigma'_v \tan \delta \quad (8.112)$$

where $K' = K_o = 1 - \sin \phi$
 $\sigma'_v = \gamma'_f H_f + \gamma' z$
 $\delta = 0.5-0.7\phi$

Hence, the total drag force

$$\begin{aligned} Q_n &= \int_0^{L_1} p f_s \, dz = \int_0^{L_1} p K' (\gamma'_f H_f + \gamma' z) \tan \delta \, dz \\ &= (p K' \gamma'_f H_f \tan \delta) L_1 + \frac{L_1^2 p K' \gamma' \tan \delta}{2} \end{aligned} \quad (8.113)$$

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.

8.18 Negative 101

Example 8.15

Refer to Figure diameter of 0.305 m and $\phi = 32^\circ$. Determine

Solution
From Eq. (8.110)

$$Q_n = \frac{p K' \gamma'_f}{2}$$

$$p = \pi (0.305)$$

$$K' = 1 - \sin \phi$$

$$\delta = (0.6)(32)$$

$$Q_n = \frac{(0.056)}{2}$$

Example 8.16

Refer to Fig. $\phi_{\text{clay}} = 34^\circ$, $\gamma_{\text{moist(clay)}}$ top of the clay layer

Solution

The depth to the n

$$L_1 = \frac{L - H_f}{1}$$

Note that γ'_f in Eq. table. So

$$L_1 = \frac{(20 - H_f)}{L}$$

$$L_1 = \frac{242.}{L_1}$$

or

$$L_1 = 11.7$$

Now, referring to

$$Q_n = (p K$$

$$p = \pi (0$$

$$K' = 1 -$$

$$Q_n = (0.1$$

$$+ 1$$

$$= 60.$$

Golder Associates Inc.

Calculation Cover Sheet

Sheet 1 of 11Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Earth load - Leachate Collection well

Name

Date

Prepared By: Timothy L. Martin January 19, 1990Checked By: Frank S. Shum 2-5-90Approved By: [Signature] 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 K_0 conditions, i.e., the pipe will not move (at least enough to develop K_0 conditions). Analysis will be conservative when based on K_0 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 $\approx 40^\circ$.

Geometry:

Analysis:

K_0 = lateral earth pressure coefficient at rest

$$K_0 = 1 - \sin \phi$$

$$K_0 = 1 - \sin 40^\circ$$

$$K_0 = 0.36$$

σ_v = vertical stress

σ_v = vertical stress = 4,072 psf (as per Assumption 2)

$$\sigma_h = K_0 \sigma_v$$

$$\sigma_h = (0.36)(4,072)$$

$$\sigma_h = 1,445.9 \approx 1,450 \text{ psf}$$

W = earthload in lbs/ft. of pipe

$$W = \left(\frac{1,450 \text{ lbs}}{\text{ft}^2} \right) \left(\frac{30 \text{ in}}{12 \text{ in/ft}} \right) = 3,625 \frac{\text{lbs}}{\text{ft}}$$

Golder Associates

SUBJECT <i>Validation of Earthhead - Leachate collection well</i>		Job No. <i>893-1186.032</i>	Rel.
Made by <i>TLM</i>		Checked <i>FS</i>	Reviewed <i>Qd</i>
Date <i>1-15-90</i>	Sheet <i>3</i>	of <i>11</i>	

For flexible HDPE pipe, the deflection under earth loading:

$$\Delta x = \frac{D^3}{12} \times K \times W$$

$$\frac{D^3}{12} = \frac{3(60)^3}{12} = 0.051$$

$$\Delta x = (1.5 \times 0.10) \left(\frac{3625}{12} \right) = 0.061(700)$$

$$\Delta x = \frac{4.5/31}{8.13 + 4.32} = 0.89 \text{ inches}$$

$$\frac{\Delta x}{X} = \frac{0.89 \times 1000}{30} = 29.7\% < 50\% \text{ OK}$$

Δx = vertical deflection of pipe in inches
 D = deflection lag factor ≈ 1.50
 K = bedding constant ≈ 0.10
 W = Earthload on pipe in lb/inch width
 E = modulus of elasticity of pipe in psi
 E_s = soil modulus (psi) (700 psi)
 SDR = $OD/t = 13.5$ (assumed OK)

Conclusion:
 Recommended using the 30 inch nominal HDPE pipe, SDR13.5 as per the design for the leachate collection well. The maximum acceptable deflection of 5% was based on the design manual pg. 65, attached.
 Deflection assuming $E_{HDPE} = 300,000 \text{ psi}$, $1.26'' = 4.57\%$, also OK.
 Note: The analysis is very conservative, as it assumes loading is essentially uniform, rather than uniform radial. As such, it is clear to a buckling analysis. #55

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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.

$$W = C_d \times w \times B_d \times D \quad (10)$$

- 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 1 1/2 inch in diameter. This material can best be described as Class I or II of ASTM D2321 "angular 1/4 to 1 1/2 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.24-5

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 - SOR 17 to SOR 32.5. The designer should consider heavier wall pipe - SOR 33 and SOR 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 tamers the soil pressure on the pipe the soil load. If the side-fill is support a greater portion of adequate stiffness in the side expansion in the horizontal, upon the selection, placement.

Under normal circumstances deflection can be calculated

Where: Δx = deflection
 D = lag factor
 K = bedding c
 W = load on c
 R = mean rad
 E = modulus
 I = moment
 where T
 E = modulus

This equation is known as
 Modified for flexible P

Where: Δx = vertical
 D = deflection
 K = bedding c
 W = earth lo
 E = modulus
 E = soil mo
 SOR = ODI

Note: Values of D = de
 Manual of Practice No. c
 to 65% of maximum der
 density.

From the above form
 can then be calculated t

Where: Δx = deflec
 D = nomie

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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:

$$\Delta x = \frac{DI \times K \times W}{\frac{2E}{3 (SDR)^3} + 0.061 E'} \quad (11)$$

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

$$\% \text{ def} = \frac{\Delta x}{D} \times 100 \quad (12)$$

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	5990	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.

Fig. 15

Classification of Backfill Material

per ASTM D2321*

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Class

Comments

Class I — Angular graded stone, $\frac{1}{4}$ to $1\frac{1}{2}$ 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 $\frac{1}{2}$ to $\frac{3}{4}$ -inch for ease of placement.

Class II — Coarse sands and gravels with maximum particle size of $1\frac{1}{2}$ 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 $\frac{1}{2}$ to $\frac{3}{4}$ -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 $1\frac{1}{2}$ 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.

Types of retaining wall

When designing retaining walls, one must know the basic soil properties, that is, the unit weight, angle of friction, and cohesion—for the soil behind the wall and the soil below the base slab. Knowing the properties of the soil behind the wall enables an engineer to determine the lateral earth pressure for which the design has to be made.

There are two phases in the design of a retaining wall. First, with the lateral earth pressure known, the structure as a whole is checked for stability. This includes checking for possible overturning, sliding, and bearing capacity failure. Then, each component of the structure is checked for adequate strength, and the steel reinforcement of each component is determined. The next part presents the procedures for determination of lateral earth pressure and retaining-wall stability. Checks for adequate strength of each component of the structures are covered in Appendix A.

5.2

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

$$\sigma_v = q + \gamma z \quad (5.1)$$

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

$$\sigma_h = K_o \sigma_v + u \quad (5.2)$$

where u = pore water pressure

K_o = coefficient of at-rest earth pressure

For granular soil, the relation for K_o is

$$K_o = 1 - \sin \phi \quad (5.3)$$

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

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Calculation Cover Sheet

Sheet 1 of 1Project Title: Non-Drag-off Landfill DesignProject No. 893-1186.032Calculation Title: Geotextile Selection

Name

Date

Prepared By: Steve Wright 1-16-90Checked By: Timothy L. Martin 1-18-90Approved By: Mike Cook 1-28-90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

**Golder
Associates**

SUBJECT GEOTEXTILE SELECTION

Job No. B93-1106.072
Rel. Non-Dwg-Off

Made by SRW
Checked TLM
Reviewed JOL

Date 1-16-90
Sheet 2 of 13

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
- $\gamma_{WASTE} = 120 \text{ PCF}$
- ASSUME MAX TILE PRESSURE = 85 PSI ($\approx$ CAT 637 SCRAPER)
- ASSUME NO SLIPPAGE BETWEEN GEOTEXTILE & DRAINAGE STONE

SOIL PROPERTIES

OPERATIONS LAYER
 $D_{85} = 0.49 \text{ mm}$
 $D_{60} = 0.18 \text{ mm}$
 $D_{10} = 0.061 \text{ mm}$
 $C_u = 2.95$

DRAINAGE LAYER*
 $d_{50} \approx 1.2 \text{ mm}$
 SPHERICITY = 0.7
 $d_{10} \approx 0.25 \text{ mm}$

BASE COURSE / RAMP**
 $d_{50} \approx 6 \text{ mm}$
 SPHERICITY = 0.4

* 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

**Golder
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SUBJECT GEOTEXTILE SELECTION

Job No. 893-1186.032
Ref. No. -Arab - DFD

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Date 1-16-90
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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,

$$B = 1$$

$$B = 0.5C_u$$

$$B = B/C_u$$

$$\text{For } C_u < 2$$

$$\text{For } 2 < C_u < 4$$

$$\text{For } 4 < C_u < 8$$

FOR PROPOSED OPERATIONS LAYER MATERIAL, $C_u = 2.95$

$$\therefore 2 < C_u < 4 \Rightarrow B = 0.5C_u = 1.48$$

$$AOS = BD_{85}$$

$$AOS = 1.48 (0.49 \text{ mm})$$

$$AOS = 0.72 \text{ mm} = 30 \text{ sieve} \quad \leftarrow \text{For Separation/Filter}$$

TLM FHWA Manual

Clayey: Porosity $\geq 30\%$

$$O_{95} \geq 3D_{15} \rightarrow D_{15} = 0.068 \rightarrow O_{95} = (3)(0.068) = 0.20 \Rightarrow 80 \text{ sieve}$$

B. PERMEABILITY

[Require AOS between 30 and 80]

ACCORDING TO FHWA MANUAL, FOR CRITICAL/SEVERE APPLICATIONS,

$$K_{\text{GEOTEXTILE}} \geq 10 (K_{\text{SOIL}})$$

OPERATIONS LAYER IS ASSUMED TO HAVE $K_{\text{sp}} \approx 10^{-4} \text{ cm/s}$

$$\therefore K_{\text{GEOTEXTILE}} \geq 10 (10^{-4} \text{ cm/s})$$

$$K_{\text{GEOTEXTILE}} \geq 10^{-3} \text{ cm/s} \quad \leftarrow \text{For Separation/Filter}$$

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SUBJECT GEOTEXTILE SELECTION

Job No. 813-1106.032

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C. CLOGGING (BETWEEN 1st 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

$$\checkmark GR \leq 3$$

D. BURST RESISTANCE (BETWEEN 1st 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		BUDGETONE	
	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 FACING GEOTEXTILE INTO VOID $\approx 0.75 P_a$
 $\approx 0.75 (85 \text{ PSI})$
 $\approx 63.75 \text{ PSI}$

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,

$\therefore$ REQUIRED BURST RESISTANCE = $3(17) = 51 \text{ PSI}$ $\leftarrow P_{\text{SCRAPER}}$

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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

Job No. 813-1186.032

Made by SRW

Date 1-16-90

Rel. Non-Drag-Off

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Sheet 6 of 13

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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 (E) IS 75% ASSUMING NO STONE-TO-FABRIC SLIPPAGE.

KOERNER SHOWS (Pg 69) THAT

$$\begin{aligned} \text{MOBILIZED TENSILE FORCE} &= \text{PRESSURE} \times E^2 \\ &= 0.75(85 \text{ PSI})(0.75)^2 \\ &= 36 \text{ LB} \end{aligned}$$

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

$$\text{REQUIRED GRAB STRENGTH} = 3(72 \text{ LB}) = 216 \text{ LB}$$

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 CARRIES A 1/2-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

Job No. 813-1186.032

Made by SRW

Date 1-16-90

Ref. No. - DMG-OFF

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$$\begin{aligned} \text{STRESS ON FABRIC} &= 0.75 P_a \\ &= 0.75 (85 \text{ PSI}) \\ &= 64 \text{ PSI} \end{aligned}$$

$$AOS = 0.72 \text{ mm} = 0.0284 \text{ in}$$

$$TREQ'D = (\pi d_i d_a) P' S'$$

WHERE

$$d_i = AOS = 0.0284 \text{ in}$$

$$d_a = 1/2 \text{ inch}$$

$$P' = 64 \text{ PSI}$$

$$S' = (1 - S) = (1 - 0.7) = 0.3$$

$$\begin{aligned} TREQ'D &= \pi (0.0284 \text{ in}) (0.5 \text{ in}) (64 \text{ PSI}) (0.3) \\ &= 0.86 \text{ LB} \end{aligned}$$

KOERNER USES A FS = 2 IN HIS EXAMPLE

$$\therefore \text{PUNCTURE/TEAR RESISTANCE} = 2(0.86 \text{ LB}) = 1.7 \text{ LB}$$

G. SUMMARY OF REQUIRED PROPERTIES FOR SEPARATION (PS LAYER/1" GRAVEL)

PROPERTY	REQUIRED (BASED ON CALC'S)	FINAL RECOM. MINIMUMS*
AOS	0.72 mm — 0.20 mm 30 sieve — 80 sieve	—
K _{TEXTILE}	$\geq 10^{-3} \text{ cm/s}$	—
STRENGTH/Burst	51 PSI	320 PSI
TRAPEZOIDAL TEAR	—	50 LB
GRAB STRENGTH	216 LB	200 LB
ELONG. @ MAX. ST	—	15%
PUNCTURE	1.7 LB	80 LB

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SUBJECT GEOTEXTILE SELECTION

Job No. 893-1186.032

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Date 1-16-90

Ref. NON-DWG-OFF

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Sheet 8 of 13

Reviewed Jd

LIST OF PRODUCTS MEETING ABOVE REQUIREMENTS:

MANUFACTURER	NAME	TYPE	AOS (Sieve)	mass/Area ² oz/yd ²
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 Quiline Inc.	Quiline 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 1st DRAINAGE LAYER

FIND: REQUIRED STRENGTH OF GEOTEXTILE TO PROVIDE ADEQUATE CUSHIONING OF THE UNDERLYING HDPE

SOLUTION:

$$TREQ'D = \pi d_i d_a P'$$

WHERE:

$$\begin{aligned} d_i &= \# 100 \text{ SIEVE} = 0.15 \text{ mm} = 0.006 \text{ in} \quad (\text{ASSUME TREVIRA 1125}) \\ d_a &\approx d_{50} \approx 6 \text{ mm} = 0.24 \text{ in} \\ P' &= 0.75 (85 \text{ PSI}) = 64 \text{ PSI} \end{aligned}$$

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SUBJECT GEOTEXTILE SELECTION

Job No. 813-1186.032
Rel. Non-Drag-Off

Made by SAW
Checked TMM
Reviewed JDC

Date 1-16-90
Sheet 9 of 13

$$T_{REQ'D} = \pi (0.006 \text{ in}) (0.24 \text{ in}) (64 \text{ PSI}) (0.6) \\ = 0.1737 \text{ LB}$$

PER MANUFACTURER, TRIVIRA 1155 HAS A PUNCTURE
STRENGTH = 200 LB

$$200 \text{ LB} > 0.17 \text{ LB}$$

TRIVIRA 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
CORRESPONDES TO A TIRE INFLATION OF 64 PSI
WITH A 5/8" MAX. STONE SIZE

$$\text{FOR } FS = 3,$$

$$\checkmark \text{ REQUIRED BURST STRENGTH} = 3(8 \text{ PSI}) = 24 \text{ PSI}$$

PER MANUFACTURER, TRIVIRA 1155 HAS A BURST
STRENGTH = 765 PSI

$$\checkmark 765 \text{ PSI} > 24 \text{ PSI} \therefore \text{TRIVIRA 1155 OK}$$

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

**Golder
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SUBJECT GEOTEXTILE SELECTION

Job No. 093-1186.032

Made by SLW

Date 1-16-90

Ref. NDN-Dmb-0FF

Checked TLM

Sheet 10 of 13

Reviewed RL

SOLUTION:

$$T_{REQ'D} = \pi d_i d_a P' S'$$

WHERE

$$d_i = AOS \approx 0.7 \text{ mm} = 0.0276 \text{ in}$$

$$d_a \approx d_{50} \approx 1.2 \text{ mm} = 0.05 \text{ in}$$

$$P' = 35F + (120 \text{ LB/FL}^2) = 4,200 \text{ LB/FL}^2 = 29.2 \text{ PSI} *$$

$$S' = (1-S) = (1-0.7) = 0.3$$

* SEE "GEOMETRY", PG 1

$$T_{REQ'D} = \pi (0.0276 \text{ in}) (0.05 \text{ in}) (29.2 \text{ PSI}) (0.3) \\ = 0.038 \text{ LB}$$

PER MANUFACTURER, TREVIRA 1155 HAS A PUNCTURE STRENGTH = 200 LB

$$200 \text{ LB} > 0.038 \text{ LB} \therefore \underline{\underline{TREVIRA 1155 \text{ OK}}}$$

D. PERMEABILITY BETWEEN DRAINAGE MATERIAL & GEOTEXTILE "LUSION"

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 COLLESSIONLESS 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. 843-1186.032

Made by SLW

Date 1-16-90

Ref. Num. DWG-0115

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Sheet 11 of 13

Reviewed *SLW*

FOR OUR DRAINAGE MATERIAL, $D_{10} \approx 0.25 \text{ mm}$

BASED UPON HAZEN'S FORMULA, K OF DRAINAGE LAYER RANGES FROM

$$K = 10 (0.25)^2 = 0.625 \text{ mm/SEC} = 6.25 \times 10^{-2} \text{ cm/SEC}$$

TO

WORST
CASE $\Rightarrow$

$$K = 15 (0.25)^2 = 0.94 \text{ mm/SEC} = 9.4 \times 10^{-2} \text{ cm/SEC}$$

$$\therefore K_{\text{GEOTEXTILE}} > 9.4 \times 10^{-2} \text{ cm/SEC}$$

PER MANUFACTURER, TREVIRA 1155 HAS A PERMITTIVITY
= 0.5 SEC^{-1} AND A THICKNESS = 5.1 mm

$$\Rightarrow K_{1155} = (0.5 \text{ SEC}^{-1})(5.1 \text{ mm}) = 2.55 \frac{\text{mm}}{\text{SEC}} = 0.255 \frac{\text{cm}}{\text{SEC}}$$

$$0.255 \frac{\text{cm}}{\text{SEC}} > 9.4 \times 10^{-2} \text{ cm/SEC}$$

$\therefore$ 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

3-29.

TABLE 3-3.

SUMMARY OF GEOTEXTILE DESIGN AND SELECTION CRITERIA FOR DRAINAGE, FILTRATION, AND EROSION CONTROL APPLICATIONS

GEOTEXTILE CALC

293-1186

Pg 12 of 13

I. SOIL RETENTION (PIPING RESISTANCE CRITERIA)¹Soils
≤ 50% Passing?
U.S. No. 200 sieveSteady State Flow
ACS -- $O_{95} \leq 2 D_{85}$ Dynamic, Pulsating,
and Cyclic Flow $O_{95} \leq D_{15}$ (if soil can move beneath fabric) $C_u \leq 2$ or ≥ 8 $B = 1$ $2 \leq C_u \leq 4$ $B = 0.5 C_u$ $4 \leq C_u \leq 8$ $B = \frac{B}{C_u}$ OR
 $O_{50} \leq 0.5 D_{85}$ ≥ 50% Passing:
U.S. No. 200 sieveWovens: $O_{95} \leq D_{85}$ $O_{50} \leq 0.5 D_{85}$ Nonwovens: $O_{95} \leq 1.8 D_{85}$

AOS No. (fabric) ≥ No. 50 sieve

- 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.
- Select fabric on the basis of largest opening value required (smallest AOS)

II. PERMEABILITY CRITERIA¹

A. Critical/Severe Applications

 $k(\text{fabric}) \geq 10 k(\text{soil})$

B. Less Critical/Less Severe and (with Clean Medium to Coarse Sands and Gravels)

 $k(\text{fabric}) \geq k(\text{soil})$

- 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 Applications¹

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

- Whenever possible, fabric with maximum opening size possible (lowest AOS No.) from retention criteria should be specified.
- Effective Open Area Qualifiers²: Woven fabrics: Percent Open Area: $\geq 4\%$.
Nonwoven fabrics: Porosity³ $\geq 30\%$
- Additional Qualifier (Optional): $O_{95} \geq 3 D_{15}$
- Additional Qualifier (Optional): $O_{15} \geq 3 D_{15}$

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. Notes: experience required to obtain reproducible results in gradient ratio test.

- Qualifiers in potential clogging condition situations (e.g. gap-graded soils and silty type soils) where filtration is of concern.

- Porosity requirement based on graded granular filter porosity.

Ref: FHWA "TS-86-203 Geotextile Engineering Manual" (1985).

US Standard Pipe Sizes

OP'S
LAYER

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Calculation Cover Sheet

Sheet 1 of 6Project Title: Non-Drag-OFF LandFill DesignProject No. 893-1186.032Calculation Title: Estimated Permeabilities of Drainage Gravels

Name

Date

Prepared By: Timothy L. Martin January 24, 1990Checked By: Frank S. Shari 2-5-90Approved By: ROCK CORR JAN 29, 1990

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

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Associates**

SUBJECT Estimated Permeabilities of Drainage Gravel

Job No. 993-1186.032

Made by TLM

Date 1-24-90

Ref.

Checked FSS

Sheet 2 of 6

Reviewed PR

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 $k \geq 10^{-2}$ cm/sec and D_{10} size 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: Hazen's Equation: $k = C D_{10}^2$ ✓ where k = permeability (cm/sec)
 C = constant which varies between 0.4 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 1, $D_{10} = 6.7$ mm ✓

$$k = (1)(6.7^2) \quad \checkmark$$

$$k = 4.5 \times 10^{-1} \text{ cm/sec} \geq 10^{-2} \text{ cm/sec} \quad \underline{\text{OK}} \quad \checkmark$$

* 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

Rel.

Checked FSS

Sheet 5 of 6

Reviewed RL

Primary and Secondary Drainage Gravel

From attached Figure, $D_{10} = 0.425$ mm.

$$k = (1)(0.425)^2$$

$$k = 1.8 \times 10^{-1} \text{ cm/sec} > 10^{-2} \text{ cm/sec}$$

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.

Boring No.	Elev. or Depth	W _n	W _L	W _p	I _p	Description
						Primary and secondary sump gravel
						WSDOT standard specification 9-03.12(4)

Project Non-Drop-Off
Project No. 893-1186.032

Date 1/23/90 Tested By ... Approved By ...

Gold Associates

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GRAIN SIZE DISTRIBUTION

FIGURE

M.L.T. GRAIN SIZE SCALE

Project No. _____

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Date _____

By _____

*Due to migration of fines, channels, and air in voids.

Fig. 7.6 Permeability, drainage, soil type, and methods to determine the coefficient of permeability (after A. Casagrande, 1936, with minor additions).

7.4 Measurement of Permeability

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

$$k = CD_{10}^2 \quad (7-10)$$

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

$$k_1:k_2 = \frac{C_1 e_1^3}{1 + e_1} : \frac{C_2 e_2^3}{1 + e_2} \quad (7-11)$$

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

$$k_1:k_2 = C_1' e_1^2 : C_2' e_2^2 \quad (7-12)$$

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 5Project Title: WHC NON DRAG OFFProject No. ~~893-1186~~ 893-1186Calculation Title: CHECK PUNCTURE RESISTANCE OF HDPE
C.27

Name

Date

Prepared By: P. Golder 19 Apr 90Checked By: John Fillion 6/7/90Approved By: Timothy J. Martin 6/8/90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

Golder
Associates

SUBJECT NDO - CHECK PUNCTURE RESISTANCE OF HDPE
 JOB NO. 893-1186 Made by PGC Date 19 APR 90
 Ref. Checked JPP Sheet 2 of 5
 Reviewed

OBJECTIVE: CHECK PUNCTURE RESISTANCE OF GEOMEMBRANE AT
 NDN - DRAG-OFF FACILITY DUE TO EXPECTED LOADINGS
 AND ASSUMING GEOMEMBRANE IS IN CONTACT
 WITH DRAINAGE GRAVEL (i.e. ASSUME GEOTEXTILE
 NOT PRESENT)

REFERENCE: GEOTEXTILES AND GEOMEMBRANES DEFINITIONS, PROPERTIES
 AND DESIGN BY JEAN-PIERRE GIBOUD

GEOMETRY:

Assumption: Typical grader/wagon tire pressure ≈ 80 psi (conservative)
 CALCULATIONS

PUNCTURE RESISTANCE OF HDPE = 90 lb

ALLOWABLE PUNCTURE RESISTANCE = $90 \times \frac{1}{3} = 30$ lb

DETERMINE MAXIMUM PARTICLE SIZE

$$R_p = p d^2$$

R_p = ALLOWABLE PUNCTURE RESIST (lb)
 p = STATIC OR DYNAMIC STRESS (psi)
 d = SIZE OF STONE (in.)

$$p_{\text{static}} = (35 \text{ CL}) (110 \text{ lb/ft}^2) = 3850 \text{ lb/ft}^2 = 26.74 \text{ psi}$$

$$p_{\text{grader/wagon}} = \left(\frac{3}{4}\right) (\text{TIRE PRESSURE}) = (.75) (80 \text{ psi}) = 60 \text{ psi}$$

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SUBJECT NDO - CHECK PUNCTURE RESISTANCE OF HDPE

JOB NO. 893-1186

MADE BY PGC

DATE 19 APR 90

REF.

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SHEET 3 OF 5

REVIEWED

$$d = \sqrt{\frac{R_p}{P}}$$

$R_p = 3011 \text{ lb} = \text{ALLOWABLE PUNCTURE RESIST}$

$P = 26.74 \text{ lb/in}^2 = \text{OVERBURDEN STRESS}$

$$d = \sqrt{\frac{3011}{26.74 \text{ lb/in}^2}}$$

$$d = 1.05 \text{ in}$$

$$d = \sqrt{\frac{3011}{6011 \text{ lb/in}^2}}$$

$$= 0.71 \text{ in}$$

• MAXIMUM PARTICLE SIZE = $\frac{3}{4}$ in. FOR DRAIN GRAVEL

SEE ATTACHED GRAIN SIZE SPECIFICATIONS FOR BOTH DRAINAGE GRAVEL AND SUMP GRAVEL. FOR DRAINAGE GRAVEL ALL MATERIAL LESS THAN $\frac{3}{4}$ in. FOR SUMP GRAVEL AT WORST 20% GREATER THAN $\frac{3}{4}$ 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.

WHC-S-045

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 1×10^{-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 1×10^{-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.

WHC NON-DRAG OFF
893-1186.036

02226-2
April 2, 1990

Pat C

Seattle

1 MAR 1985

TADIOS 46

TADIOS

AD1

Geotextiles and Geomembranes Definitions, Properties and Design

Selected Papers; Revisions and Comments

THIRD EDITION

Jean-Pierre Giroud

1985

TA
710
.64
1984

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Calculation Cover Sheet

Sheet 1 of 4Project Title: Non-Drag-off Landfill DesignProject No. 893-1186Calculation Title: Vertical Riser Concrete DesignSeismic Design Analysis C.28

Name

Date

Prepared By: Michael L. Brown 6-11-90Checked By: Timothy L. Martin 6-11-90Approved By: [Signature] 6-12-90

QA Approval: _____

Revisions:	Date	By	Checked By	Approved By	QA Approval

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SUBJECT WHC / NDO VERTIC Riser CONCRETE

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JOB No. 893-1186

Made by M. BROWN

Date 6/8/90

Ref.

Checked TLM

Sheet 2 of 4

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OBJECTIVE: Check adequacy of design for seismic loadings. ^{concrete foundation slab}

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

$$V = \left[\frac{\pi 4.6^2}{4} - \frac{\pi 4^2}{4} \right] 12$$

$$V = 58.6 \text{ ft}^3$$

Unit wt of concrete = 150 pcf

$$W = 58.6 \text{ ft}^3 \times 150 \text{ pcf} = 8795 \text{ lbs.} = 7.3 \text{ kips}$$

$$\sum F = 0$$

TOTAL UPWARD REACTION = 7.3 kips.

$$= \left(\frac{P_1 + P_2}{2} \right) (6' \times 6') = 18(P_1 + P_2) *$$

$$7.3 \text{ kips} = 18(P_1 + P_2) \quad \text{Eqn (A)}$$

* 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/ND0

JOB No. 893-1186

Made by M. BRANN

Date 6/8/90

Ref.

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Sheet 45 of 4

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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 $0.348 \text{ ksf} \ll 6.85 \text{ ksf}$ 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.