APPENDIX H – GEOTECHNICAL STUDIES

This appendix includes the text and Appendices A and B from Findling's Geotechnical Study for Harbor Sites Feasibility Study - Masonville Marine Terminal -Baltimore, Maryland (Findling 2006). Not included in this appendix are the Findling 2006 Masonville Geotechnical Study appendices containing boring logs, lab test data (for the boring samples and strength tests of dried dredged material at Cox Creek Dredged Material Containment Facility), and slope stability analyses. The full Findling 2006 Masonville Geotechnical Study is available on request from the United States Army Corps of Engineers - Baltimore District.

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation conducted in association with the design of Dredged Material Containment Facility (DMCF) at the Masonville Marine Terminal, Baltimore, Maryland. This investigation was conducted for Gahagan & Bryant Associates, Inc., in general accordance with Findling, Inc.'s proposal dated February 24, 2004.

2.0 SITE LOCATIONS AND DESCRIPTION

Masonville Marine Terminal is located on the south shore of Patapsco River, north west of the Harbor Tunnel (I-695), and immediately west of Fairfield Marine Terminal, in Baltimore, as shown on Figure 1 - Site Vicinity Map, and on Figure 2 - Site Location Map, in Appendix A.

Currently, the proposed project area is overwater. The depth of water ranges from 12 feet to 20 feet and is generally about 15 feet.

The area immediately south of the project area varies in elevation from about El. +6 to about El. +30, and is currently being developed for marine terminal activities. There is an existing ship slip to the east of the project area, adjacent to Pier 5.

Ferry Barge Channel, dredged to 42 foot depth, lies to the north of the site, and Kurt Iron lies to the south of the site, as shown on Figure 2.

3.0 PROJECT BACKGROUND AND DESCRIPTION

The Maryland Port Administration (MPA) is responsible for providing facilities for the disposal of material dredged from the harbors and channels of the Port of Baltimore. At present, the major disposal locations are Hart-Miller Island (HMI) and Poplar Island. A new facility, Cox Creek Dredged Material Containment Facility (CCDMCF) is under construction and should come into service sometime in the next year. However, Cox Creek has a relatively small capacity; HMI will be closing by 2009 and Poplar Island has been reserved to receive material from the Bay's channels and not from the channels within the Inner Harbor area.

The MPA has therefore been evaluating additional disposal sites for placement of material dredged from the inner harbor areas. One of the options under evaluation is an area adjacent to the existing Masonville Marine terminal. See Figure 2 – Site Location Map. The existing Masonville terminal is also undergoing an expansion. The proposed Masonville DMCF will connect to the existing MPA Masonville "fastland" and expand into the Patapsco River. The facility will encompass approximately 120 acres and will have a design capacity of approximately 16.0 million cubic yards of dredged material. It is estimated that the useful life of the facility will be 20 years. Upon completion of the filling operation, MPA plans to develop the site into a port facility.

The MPA, with the assistance of its consultants, has been evaluating the engineering and environmental issues associated with the design and construction of the Masonville DMCF. Initially, five dike alignments were evaluated, and in the fall of 2004, Findling completed an Interim Feasibility geotechnical report documenting its findings regarding the geotechnical aspects of designing the containment dike along each alignment to an ultimate height of El.+42. Other members of the team evaluated other engineering and environmental issues associated with each alignment.

Based on these studies, the MPA selected one alignment that was considered the best option for developing the DMCF. The MPA commissioned their consultants to prepare feasibility level studies for this alignment. The selected alignment is a combination of alignments 2 and 5 and is shown in Figure 3 in Appendix A.

The alignment begins at the Northwest corner of the existing Masonville Facility and extends North into the Patapsco River about 1600 feet at which point it turns East parallel to the Ferry Barge Channel for about 2000 feet. At that point the earth embankment bends to the Southwest and continues another 1400± feet where it will tie into a cellar cofferdam structure to be built. Overall, the earth embankment portion is about 5000 feet long. The cofferdam will parallel the existing piers 2 and 3 and tie into the marine terminal. The current design includes an initial earth embankment to be constructed to El.+10. As the facility is filled in with dredge material, the earth embankment will be raised in stages to an ultimate height of El.+42. Existing land along the Kurt Iron area of the marine terminal is at about El. +8, thus, a two-foot high earth dike will be constructed along the water edge around Kurt Iron and tie into the Masonville Terminal.

It is intended that the dike be constructed from the sand and clay that is available within the area contained by the dike. It is envisioned the soil will be dredged and placed using hydraulic dredging method, however, the contractors will have the option to use mechanical methods. Previous containment embankments constructed in open water, HMI and PI, have been built using granular material, sand and gravels, containing less than 30 percent fines in place. Due to the high variability, both vertically and laterally, of the soil within the containment area there may not be adequate sand available to construct the initial dike section to El.+10. To have a stable containment dike, the outside portion of the initial dike to El.+10 will be constructed of granular material, as shown on Figure 4. The section facing Patapsco River will be constructed from sand, and the section facing the containment area will be constructed from a combination of sand and/or clay.

After the completion of the dike to El. +10 and the construction of the ancillary facilities, such as spillways, unloading facilities, etc., the DMCF will be operational and can receive dredged material. After a period of time, the dikes will be raised to a higher elevation – currently the next planned raising will be El. +28 MLLW. Current plans are to construct the 28-foot dike with off-site borrow. The raised portion will be stepped back 20 feet from the outside edge of the El. +10 dike. The dike will be partially supported on the initial El.+10 dike, and partially over previously placed dredge material on the inside of the cell. See Figure 5. It is anticipated that the facility operators will have dried out the dredged material through a process called "crust management", which requires dewatering the dredged material using drainage ditches and natural drying. This results in consolidation of the placed material and increases in the shear strength.

The dike will then be raised in stages to El. +42 to achieve the design capacity of the facility. The dike raising above El. +28 may be accomplished using dried reclaimed dredged material to construct the embankment as it has been done at other DMCFs. The use of this material rather than off-site borrow, results in increased capacity.

The raising of the dike from El. +28 to El. +42 will be done in 4 feet to 6 feet dike height increments, rather than in one step. The incremental dike will likely be founded partially on the crust and partially on previously placed controlled dike fill. Raising the dike in small increments will minimize displacement or the risk failure of the inside slope.

Figure 6 shows the proposed raising of the dike to El. +42.

There are two utilities that cross, approximately perpendicular, to the dike alignment about Sta. 45±. See Figure 2. These include a reportedly abandoned electric line and a 48-inch diameter water main, which is in service. It is our understanding that the electric cable will remain in-place and the water main will be relocated to within the cofferdam portion of the structure prior to embankment construction.

4.0 PURPOSE AND SCOPE

The purpose of this geotechnical study is to focus on one alignment (the selected alignment) and associated borrow area and to provide geotechnical analysis and recommendations to the engineering team that is preparing the feasibility level project report.

The scope of work of the current geotechnical study includes:

1) Collection of additional subsurface data along the preferred alignment. To meet this objective, Findling:

    a. completed six (6) additional borings along the selected dike alignment;

    b. conducted in-situ vane shear tests at various locations along the alignment to supplement information on the strength of the soft soils; and

    c. conducted additional laboratory tests on selected samples

2) Evaluate the probe data obtained by GBA, to identify areas where additional subsurface information is required, and compare the probe data with the test boring information. This data will then be used to refine the limits of undercut of Stratum I soil under the dike.

3) Evaluating the quality of sand available in the potential borrow area, based on the available boring data and on the additional borings drilled in the borrow area. The location of the potential borrow area is shown on Figure 7.

5.0 GEOTECHNICAL CONSIDERATIONS/RELEVANT AVAILABLE DATA

Several subsurface investigations, test borings, have been undertaken at the Masonville site in the area of this dredge material disposal facility over the past 25+ years. These include test borings by Sprague & Henwood in 1980 (W series), borings conducted by Findling in 2003 – WB series for the proposed cofferdam, F series drilled by Findling in April and May 2004 to evaluate alignment options and test borings along the selected alignment and borrow area for this study – 2F and VS series of borings. A test boring location plan and logs of all test borings are included in Geotechnical Study – Selected Alignment – Interim Feasibility Study Volume II.

5.1 <u>Summary of pertinent information from previous geotechnical study</u>

The prior geotechnical study dated September 28, 2004 contains information that is relevant to the study of the selected alignment. This section summarizes the information from the prior study that was utilized in the current study.

5.1.1 Field Investigation

The field investigation was conducted in April, May and October 2004. A total of 42 borings (F-1 through F-17, F-19 through F-26, and F-32 to F-43, F-44 to F-48 and F-50) were drilled during this investigation at the location shown on Figure 8 – Boring Location Plan, in Appendix A.

Of the borings that were completed earlier, the following are pertinent to the alignment under study and to the borrow areas being evaluated for the project. The general locations of the borings drilled at the site were as follows:

Boring	General Location
F-8, F-46 to F-48 & F-50	Potential Borrow Area
F-24 to F-20, F-27, F-29, F-30 F-7, F-32, WB-103, WB-104	Selected Alignment

The boring logs relevant to the selected alignment are found in Appendix C of the full geotechnical report.

In-situ (field) vane shear tests were conducted in accordance with ASTM D-2573 in cohesive soils at selected locations along the dike alignment.

Laboratory tests were conducted on selected samples to define physical and strength characteristics of soils. The results of all field vane shear tests and laboratory tests from the previous study are included on Findling's September 04 report, and Volume II of this study.

5.1.2 Subsurface Conditions

The subsurface conditions were divided into three basic strata, as follows:

Stratum I: Dark Gray to Black Clayey SILT with standard penetration resistance (N value) of WOR to WOH.

Stratum II: Gray-Brown medium dense to very dense Silty SAND with little gravel and cobbles and pockets of Silty Clay. N Values vary from 10 to 50 blows per foot.

Stratum III: Gray-Purple-Tan – stiff to hard Silty CLAY with N values of 11 to over 50 blows per foot.

5.2 The Current Geotechnical Study

This section of the report presents information used for dike design along the selected alignment.

5.2.1 Field Investigation

The field investigation was conducted in June and July 2005, when a total of 13 borings (2F-1 through 2F-11 and 2WB103 and 2WB104) were drilled. In addition, separate holes were drilled to allow for conducting in-situ vane shear tests and for obtaining undisturbed shelby tube samples. The locations of the borings drilled during this investigation are shown on Figure 9 – Boring Location Plan, in Appendix A, and boring logs included in Appendix C of the full geotechnical report.

The general locations of the borings drilled are shown in the following table:

Boring	General Location
2F-1 through 2F-4, 2F-7, 2F-8	Potential Borrow Area
2F-5 & 2F-6	Along Dike Alignment
2F-9, 2F-10, 2F-11	Along alignment of cofferdam
2WB-103 & 2WB-04	Along Dike alignment and proposed alignment of relocated watermain
VS-1 and VS-2	Along Dike Alignment

All borings drilled during this investigation were drilled using a truck mounted CME 75 drill rig equipped with an automatic hammer, that was mounted on a steel barge. The barge was held in-place by spuds. The borings were advanced using hollow stem augers. Standard penetration tests were conducted and split spoon samples were obtained in every boring, at depth intervals of 2.5 ft. and 5 ft., as required. Representative portion of each sample was placed in a glass jar and was appropriately marked. Three inch diameter Shelby tube samples were obtained in cohesive soils at the following locations.

Boring	Depth (ft)
VS-1	38 – 40 42.5 – 44.5 52.5 – 54.5
VS-2	34 – 36 44 – 46 54 – 56

The Shelby tube samples were sealed in the field, and were appropriately marked. All samples were transported to Findling's laboratory for further analysis and testing.

In-situ field vane shear tests were conducted in accordance with ASTM D-2573 in cohesive soil at the following locations.

Boring	Depth (ft)
VS-1	36 40 42 45 47 48 49 52
VS-2	38 48 57

A 6-inch long vane with a vane diameter of 2 3/8 inches was used. The torque was measured using a calibrated torque wrench with an arm length of 12-inches. The results of the field vane shear tests are summarized in Table 3 in Appendix B.

In addition to the vane shear tests completed in borings VS-1 and VS-2, Findling completed fourteen (14) vane shear tests at 8 locations (VS-4, VS-6 thru VS-12) by pushing the vane attached to AW drill rods by hand over the side of the barge. The purpose of these tests were to evaluate the shear strength of the soft Stratum I soil with depth. These vane shear tests were performed using a 6-inch long vane with a vane diameter of 2 3/8 inches. The vanes were pushed to the desired depth and the torque was applied. The torque was measured using a calibrated torque wrench with an arm length of 12-inches. The results of the tests are included in Table 1.

5.2.2 Probe Data

A critical issue in the dike design and construction for the Masonville containment facility is ensuring that the dike is constructed on a foundation that has sufficient shear strength to support the weight of the dike. Borings drilled in the Interim Feasibility Study identified that a layer of very soft silt (Stratum I) covered the site.

The geotechnical analysis concluded that this material should be removed (Stratum I) from beneath the dike to provide a suitable foundation. A generalized subsurface profile was developed from the initial borings indicating the thickness of Stratum I layer. The removal and disposal of this material through dredging is a significant expense. It was considered highly desirable to better define the lateral and vertical extent of this soft layer which required removal. To accomplish this objective, GBA undertook a program of probings to determine the thickness of the very soft silt (Stratum I), over the area of both the borrow source and the proposed dike alignment.

The result of the probing effort is contained in the report prepared by GBA entitled "Masonville Marine Terminal Feasibility Study Probing Analysis" dated August 2005.

The probings were completed in April and June of 2005. The probings were performed by lowering a pipe through which water is jetted below the mudline. The pipe probe was lowered until refusal to hand pushing on the rod. The depth of the water, the length of the probe below the water were measured and the thickness of the muck calculated. Each probe location was determined by a GPS system. The probings were made at each nodal point on a 100 to 200 foot grid pattern.

The data regarding the location, depth of water, and depth to probe refusal were collected and stored for each probe location. The probe data was used by GBA to develop a contour plan of firm bottom over the Masonville DMCF. The plan is included in the GBA Probing Analysis Report and shows a highly irregular depth to probe refusal. The depth to refusal data was compared to the test boring data. In general, there is relatively good correlation between the bottom of Stratum I from test boring data and probe resistance. However, the probe data identified a deep soft area between Sta. 25 and Sta. 29 along the dike alignment which had not been detected by the initial test boring program. The results of the probe data are shown on Figure 10.

6.0 LABORATORY TESTING

All samples were visually examined in the laboratory by a geologist or geotechnical engineer to corroborate and/or modify the field classifications. Selected samples were tested for their natural water content, Atterberg limits, sieve analysis, percent fines, and shear strength (from unconfined compression tests). All tests were conducted in accordance with applicable ASTM procedures. A total of sixteen (16) sieve analyses, eleven (11) Atterberg Limits, and six (6) unconfined compressive strength tests were conducted. The graphical plots of the test results are included in the Appendix of the full geotechnical report. The results of the tests are summarized on Table 2.

7.0 SUBSURFACE CONDITIONS

The borings from this and the previous investigations, and the probes provided information on the soils underlying the selected dike alignment as follows:

Stratum I: Stratum I consists of a Black Dark Gray clayey silt (muck) and was found along the entire dike alignment below the mudline. The thickness of this stratum varies from about 5 feet to in excess of the 40 feet and standard penetration resistance (N value) varies from WOR (weight of rods) to WOH (weight of hammer). The probings identified a very thick layer of this material on the northern section of the alignment in vicinity of Sta. 25 to Sta. 29. The in-situ vane shear tests provided information that the shear strength of the soil in Stratum I increases with depth, from about 100 psf near the mudline to about 600 psf at about El. -50 (or about 35 feet below the mudline). Laboratory unconfined compression tests conducted on samples recovered from two of the borings provided additional shear strength data and the strengths from laboratory tests were less than the field vane shear tests. It appears that this stratum is normally consolidated. Its Liquid Limit is between 70 and 90, the Plasticity Index is between 15 and 21, and its natural water content varies from 130% to 140%. The water content is generally greater than the Liquid Limit.

These index properties are very similar to the ones reported in the September 28, 2004 report.

Stratum II: Stratum II consists of medium dense to very dense gray-brown-tan-red silty sand with pockets of silty clay. The lateral and vertical extent of the clay could not be established, since its location is sporadic and the borings were spaced up to 600 feet apart. The N values in Stratum II vary from about 10 blows/foot to 50 blows/foot. The fines content (i.e.% passing US Standard Sieve No. 200) in the sand portion is generally less than 30%. In the clay pockets, the liquid limits vary from 29 to 44, and the plasticity index varies from 9 to 26. The water content in the clay portion varies from about 36% to 67%.

This layer consist primarily of Silty Sand with pockets of Silty Clay, thus, the stability analysis was conducted for both conditions - Sand and Clay. Strength parameters for Stratum II soil used in this study are the same as those used in our previous report dated September 28, 2004. The thickness of Stratum II varies considerably from 5 feet (Boring W-12) to over 50 feet (Borings W-6 and W-10).

It should be noted that this stratum is of alluvial origin and contains gravel, cobble, and even occasional boulders. These larger particles could have an impact on dredging activities. The sand in the stratum is angular to semi-angular.

Stratum III: This layer consists of stiff to hard silty clay and underlies Stratum II. It extended to the bottom of the soil borings. Standard penetration resistance varies from about 11 blows/foot to 50 blows/foot. Laboratory tests and data from the boring logs indicates that the clay varies in strength from 700 to 1,000 psf near the interface with Stratum II and increasing to over 2,000 psf. Its index properties are as follows:

Liquid Limit	=	50 to 75
Plasticity Index	=	30 to 45
Water Content	=	18 to 22

Based on the information from the borings, the probings and the laboratory analyses, generalized subsurface profiles were developed along the selected alignment. These profiles are shown on Figure 11 and Figure 12.

8.0 EVALUATION AND ANALYSIS

8.1 General

The two major issues concerning the geotechnical evaluation of a dredged material placement site are:

a) Availability of Borrow Materials, and
b) Foundation Conditions.

These issues are discussed below:

a) Borrow Material – Availability of borrow material within the enclosed area: Historically, dredge material containment facilities constructed in open water in the Baltimore area have been built with granular material (sand and gravel) excavated within the containment area or near the site. The Sand typically contained less than 30 percent fines in place in the dike section. However, the borrow area identified within the proposed Masonville containment facility does not appear to have sufficient quantity of granular material to construct the entire section of the initial dike to El.+10. Similar containment facilities have been constructed in other areas using clay and/or mixture of sand and clay and are stable. Therefore, the dike design for this site is based on a portion of the embankment containing both granular material and clay.

b) Foundation Conditions – Foundation conditions under the perimeter dike:
Stiff clays and sands are the preferred foundation conditions. Soft clays in the foundation soils would require flatter slopes for the dike, or steeper slopes and stabilizing berms. Flatter slopes or berms would increase the cost. Additionally, areas that have very soft clays may require the total or partial removal of theses very soft soils by undercutting. The undercut soil will need to be disposed of, and the undercut area will need to be backfilled with sand.

In evaluating the stability of a slope, four variables have to be considered:

i) The analytical method used.
ii) Shear strength of the foundation soil and the embankment soil.
iii) Cross-section of the containment dike and the side slopes.
iv) Factor of safety, acceptable and computed.

8.2 Borrow Material: Quantity and Quality of Sand and Clay

It is proposed to build a dike from the sand of Stratum II and the clay of Stratum II and III.

In evaluating the borrow area, two variables have to be evaluated: 1) quantity of sand and clay, and 2) quality of sand and clay.

8.2.1 Quantity of Sand and Clay:

Subsurface information from previous investigation, completed as part of this study and probe data were used to evaluate the quantity of the various types of borrow. This analysis was conducted by another consultant.

8.2.2 Quality of Sand (Stratum II) and Clay (Stratum III):

The sand of Stratum II appears to be angular to semi-angular. The percent of fines in the sand portion of Stratum II varies considerably, but is generally less than 30%. The sand appears to be suitable for building the dike using hydraulic or mechanical dredging.

It should be noted that the sand (Stratum II) does contain layers/pockets of silty clay. It will not be practical to segregate this clay from the sand. The clay would probably get incorporated in the dike, as balls or chunks depending on the construction material and methods. It is also possible that portions of the dike could consist mostly of clay, rather than sand, from Stratum II and/or Stratum III. The initial dike design to EL. +10 is based on the exterior portion of the dike to contain sand with up to 30 percent fines, however the interior portion of the dike could be either sand or clay. The stability analysis was conducted for both types of material on the inside portion of the dike.

The clay in Stratum III is stiff to hard. It is anticipated that this clay will form balls during hydraulic dredging and placement or relatively large chunks if mechanical methods are used. The balls or chunks will form a fairly steep slope above and below water.

8.3 Slope Stability

8.3.1 Analytical Method:

Slope stability analyses were conducted using typical cases based on the subsurface profile along the dike alignment. Purdue University PC STABL-6H program was used in analyzing the stability of the slopes. This program incorporates several different analytical methods, such as circular failure and wedge failure. Also, the failures can be analyzed using different approaches, such as the Modified Bishop Method, the Modified Janbu Method and the Spencer Method. The Janbu Method results in factor of safety, which is generally considered to be too conservative, and is about 15% less than the Bishop's Method. For this study the Modified Bishop method, which is accepted by the U.S. Army Corps of Engineers (USACE), was used.

8.3.2. Design Parameters:

a) Foundation Soils: The generalized subsurface profiles indicate that, at the selected alignment, there is about 5 feet to 40 feet of very soft clayey silt (Stratum I). This is underlain by sand (or clay) of Stratum II. Stratum III is very dense or hard and will have minimal impact on the stability of the dike.

Elevation	Stratum	Type of soil	Unit Weight γ (pcf)	Cohesion, C (psf)	Angle of Internal Friction, ϕ (Degree)
El.-15 to El.-30	I	Soft Silt (Muck Undercut)	100	100	0
El.-30 to El.–45	II	Sand	115	0	30
El.-45 to El.-55	III	Clay	120	1000	-
Below El.-55	III	Clay	120	2000	0

b) Embankment Soils:

The initial dike to El.+10 will be constructed with Silty Sand and/or Clay of stratum II and Clay of Stratum III. Stratum II is predominantly Silty Sand with pockets of Clay. In general, the fines content in the granular portion of this layer is less than 30 percent in place in the borrow area. Previous containment embankment design using silty sand containing 20 to 30 percent fines in place have used an angle of internal friction (ϕ) of 28^{o} below the water level and 30^{o}

above water level. This design also uses these design parameters for the predominantly granular portion of the containment dike.

Very little if any data is available locally regarding the shear strength of hydraulically or mechanically dredged and placed clay fills. Limited laboratory tests were conducted by E2Si in the early 1990's as part of the Poplar Island facility design. These tests were conducted by placing nominal 1 inch size clay balls, cut a molded form large chunks obtained from the Poplar Island site, into a tank of water, obtaining tube samples of the clay balls and conducting unconfined compression test. The results yield shear strength of 150 to 200 psf. It is our opinion that due to the size of the clay balls and test sample, 3 inches diameter; the strength test results are low and do not accurately represent shear strength in a clay dike.

In attempt to more accurately model field condition, Findling conducted a large scale field test on clay backfill in water at the Cox Creek DMCF in the fall of 2005. The clay material used for the field test was dark red-gray-tan stiff to hard clay, which was encountered at the Cox Creek Site. This material is the Arundel Clay of the Potomac Group, which also underlies the Masonville Site and is identified as Stratum III. The Liquid Limit and Plastic Index of the Cox Creek material (clay) were 52 to 34 respectively. The plate bearing test was conducted by excavating a pit about 6 feet wide by 10 feet long by 6 feet deep, filling the pit with water then filling the pit to 2 feet above the water level with clay balls. The clay size chunks were generally 4 to 8 inches size and they were allowed to set in the pit for 2 weeks before testing. A 2-foot diameter steel plate was placed on top of the clay and a wood frame to support a drill rig was constructed over the pit. The steel plate was loaded to failure by jacking against the drill rig. Failure was defined as the maximum load that could be developed and held on the plate. The load on the plate was computed from recording the pressure on the hydraulic jack and conversion charts provided by the jack supplier. See Figure 13. The shear strength of the clay was back calculated using the bearing capacity equations. Several methods have been developed to calculate the soil bearing capacity based on strength parameter of the soil, i.e. cohesion and angle of internal friction. For our analysis we assumed a cohesive soil with the angle of internal friction of 0. The most common correlation for bearing capacity was developed by Terzaghi, which utilizes bearing capacity factor (Nc, Nq and Nj), depth of footing, and size of footing. Since we have assumed a friction angle of zero we only use the bearing capacity factor related to cohesion and cohesion is our analysis. Terzaghi developed bearing capacity factor for two conditions, local shear and general shear, and Meyerhof developed similar correlations. Based on the various methods and test data the cohesion of the clay mass was estimated to vary from about 300 psf to 425 psf. A plot of load verses settlement is shown in Figure 17 in Appendix A.

In addition to the plate load test, an attempt was made to measure the shear strength in the clay pit using a hand vane test apparatus, Geonor H-60 model.

This method was not successful because the vane could not be pushed into or thru the clay balls and there was little to no torque resistance on the vane in the space between the clay balls.

In addition to the plate load test, some of the clay material was brought to Findling's laboratory and placed in a trough filled with water. The trough was about 30 inches wide by 6 feet long by 3 feet deep. The clay chunks were allowed to soak in the water trough for 2 months and the shear strength tested using a hand vane shear apparatus, Geonor H-60 Model. After 2 months, there were no sings of clay balls or chunks, but only a clay mass. The result of the hand vane test indicate the shear strengths varied from about 600 to 950 psf. Thus, as the clay is allowed to consolidate under its own weight, the shear strength increases from that measured in the plate load test.

For design purposes, it was assumed that the shear strength of underwater clay fill is 300 psf, and that of above water clay fill is 400 psf immediately after placement. The density of the clay fill was assumed to be 115 pcf below water and 120 pcf above water.

c) Incremental Dikes:

Incremental dike from El. +10 to El. +28 will be constructed from off-site borrow. Its design parameters were assumed to be as follows:

γ = 120 pcf	C = 1500 psf	ϕ = 0	cohesive material
γ = 120 pcf	C = 0 psf	ϕ = 34^o	granular material

Incremental dike from El. +28 to El. +42 will be constructed from crust material. Its design parameters were assumed to be as follows:

γ = 120 pcf C = 1200 psf ϕ = 0

The containment dike at the Cox Creek DMCF is currently being raised from El.+24 to El.+28 using previously placed dredge material – Silt and Clay size particles, which has been reclaimed from the facility and dried. The dried dredge material is being placed in lifts and compacted to at least 90 percent of the maximum density per ASTM D 1557, Modified Proctor. To evaluate the shear strength of dredge material, which has been used to construct the Cox Creek Dike, Findling obtained undisturbed tube samples of the fill and conducted unconfined compression test. The results indicate shear strength (cohesion) ranged from about 2100 to 3500 psf. The test results are included in the Appendix of the full geotechnical report.

The design parameters for the dredged material under the incremental dike (i.e., the foundation material for the incremental dike) were assumed to be as follows:

γ = 90 pcf C = 300 psf ϕ = 0

It is envisioned the dredge material under the incremental dike will be displaced or will consolidate to the design strengths. The design strengths or displacement should be confirmed by field test prior to construction of the dike above El. +10.

It is difficult to accurately predict the amount of displacement of dredge material that will occur when constructing the raised dike over previously placed dredged material. Several factors will effect the amount of displacement such as type of material, effectiveness of crust management, schedule for dike construction, etc. In attempt to evaluate the amount of displacement of dredge material, we conducted a stability analysis of the interior slope with various thicknesses of dike material over crust and dredge material. In our analysis we increased the thickness of dike material required for a factor of safety of 1 or more. The analysis included a 2 foot thick layer of dried crust with a shear strength of 300 psf over the previously placed dredge material with a shear strength of 100 psf. The results indicate that 2 to 4 feet of material may be displaced. The results of the stability analysis are included in the Appendix of the full geotechnical report and are identified as Crust Analysis. For estimating purposes, we recommend four feet of displacement be included for quantity estimates, i.e. when estimating the quantity of material for dike raising, include a four foot thick zone of dike material for the portion of the dike constructed over previously placed dredge material to account for displacement.

8.4 Slope of Dike / Dike Geometry

The dike will be constructed with the outer portion (facing Patapsco River) being silty sand, and the inner portion (facing the cell) being clay or sand, thus, the stability analysis considered both cases.

During construction, the slope of the dike can vary considerably, depending upon the type of soil, placement methodology, and whether the soil is placed above or below the water. The design is based on the outside slope of the dike to be 3H to 1V and the inside slope of sand portion to be 2.5H to 1V and the inside slope, the sand/clay portion to be 4H to 1V. These slopes can be obtained by mechanically shaping and/or shaping and various construction placement techniques. The crest of the dike will be about 70 feet wide, of which about 20 feet will be sand and 50 feet will be clay and/or sand. The wide berm will act as the partial foundation for raising the dike to El. +28. The dike geometry for the dike constructed to El. +10 is shown on Figure 4.

8.5 Acceptable Factor of Safety

The acceptable factor of safety was assumed to be 1.3, for the end of construction of the containment dike's outer slope. This was also based on the experience at the Hart Miller Island Dredged Material Containment Facility and the Poplar Island Environmental Restoration Project, and was considered to be acceptable to the reviewing agencies and MPA. This Factor of Safety of 1.3 was to be used for the shallow and deep-seated failure of the outside slope for the initial dike to El. +10, and the later raising of the dike to El. +28 and to El. +42.

The Factor of Safety for the containment cell side slope is not considered to be as critical as the outside slope. It was recognized that the inside slope of the dike raised to El. +28 or to El. +42 could be founded on the crust of the dredged material, and some material will be displaced, during construction. This would not result in any release of the dredged material into Patapsco River. Therefore, a factor of safety greater than 1 for the condition immediately after construction, with the raised portion not retaining any dredge material, was considered acceptable.

8.6 Computed Factor of Safety - Selected Alignment

The Factors of Safety computed and discussed below are based on the design section and geotechnical properties discussed above.

The generalized subsurface profiles indicate that the subsurface conditions under the dike vary significantly. Generally, there are three different types of subsurface conditions:

Type 1: Sta. 6 to Sta. 21 (15± feet of Stratum I over Stratum II)
Type 2: Sta. 23 to Sta. 29 (40± feet of Stratum I over Stratum III)
Type 3: Sta. 31 to Sta. 44 (15 feet of Stratum I over 5 feet of Stratum II).

These are shown on Figure 14, Figure 15 and Figure 16.

The stability of the dike for each type of subsurface condition was analyzed for different dike elevations for shallow and deep failures. The computed Factor of Safety for each case is summarized on Table 4 and the results of the slope stability analyses are included in Appendix D of the full geotechnical report. Figures 18, 19, 20, 21, 22 show the failure planes for the limiting factors of safety for the outer slopes of the +10, +28, and +42 ft dikes and the inner slopes of the +28 and +42 ft dikes.

The analyses indicate the following:

- The outside slope of the dike at El. +10, has a Factor of Safety greater than 1.3 for each of the subsurface conditions evaluated.
- The outside slope of the dike at El. +28 has a Factor of Safety greater than 1.3 for each of the subsurface conditions evaluated.
- The outside slope of the dike at El. +42 has a Factor of Safety of about 1.3. Therefore, before the dike is raised to El. +42, the shear strength of the dredged material and the crust should be tested to confirm the design shear strength.
- The inside slope of the dike, when raised to El. +28 or to El. +42 will have a Factor of Safety of about 1. Thus, some shallow sloughing and/or material displaced could occur. This could be prevented by increasing the strength of the dredged material by crust

management. In any case, the minor sloughs of the inside slope will not result in any discharge of the dredged material into Patapsco River, and is considered to be acceptable.

8.7 Undercut

The very soft soils of Stratum I will need to be undercut from under the dike, for the dike to be stable. The undercutting, disposal of this soil is costly, therefore, it would be highly desirable to minimize the volume of undercut and maintain a stable dike.

Two alternatives were evaluated:

Alternative A: Undercut the soft soil from under the sand portion of the dike only, but not from under the interior clay/sand dike, as shown on Figure 19. This will result in a smaller volume of undercut. However, the inside slope will likely slough during construction. The soft soil will be displaced by the fill, until a stable section is obtained. This approach will result in placement of some additional material over the template quantities.

Alternative B: Undercut the soft soil under the entire dike section (dike to El. +10). This will minimize sloughing of the inside of the dike slope. However, the volume of undercut will be large.

Undercut alternative A was used for the analysis of the selected alignment and is recommended for construction.

Table 5 summarizes the vertical and lateral extent of the soft soils that need to be undercut for Alternative A. It should be noted that the width of undercut is based on the following geometry:

Top of Dike	at	El. +10
Outside Slope	at	3H:1V
Crest Width	=	20 feet
Inside Slope	at	2.5H:1V
Water Depth	:	12 feet to 15 feet (depending upon Station)
Thickness of Undercut:		12 feet to 36 feet (depending upon Station)

8.8 Water Main

A 48-inch diameter water main intersects the dike alignment at approximately Sta. 45+00 at about a 75-degree angle. The line is currently active and owned by Baltimore City. Based on available information, the water main section from south of the Ferry Bar Channel to the Marine Terminal is supported on piling. However, information regarding the type of pile, design load, and pile tip elevations is not available. The invert of the water main in the area of the containment dike is about El. –35.

The water main will be relocated prior to dike construction so that it will not cross beneath the Masonville DMCF dike. The proposed realignment will parallel the containment dike about 250 feet north of the dike to the northeast corner of the containment facility, where the utility turns and continues south behind the proposed cofferdam structure. Final design details are currently ongoing.

Subsurface explorations have been limited in the area of the water main due to lack of a precise field location and subsequent concern for damaging the water main. The closest borings are about 200 feet away from the utility. Efforts are currently underway to accurately locate the water main in the field and drill two test borings, one about 50 feet from each side of the water main along the dike alignment. This work will be conducted at the same time test borings are drilled along the water main for thrust restraint design.

Since the water main is supported on piling, we expect the soil from the mud line to about 2 feet below the pipe invert, i.e. El. –37±, will be soft soil similar to Stratum I material. After the water main has been relocated this soft soil will need to be undercut and backfilled as recommended in this report. Additionally, we recommend the water pipe be removed in the area of the proposed containment dike.

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APPENDIX A

FIGURES

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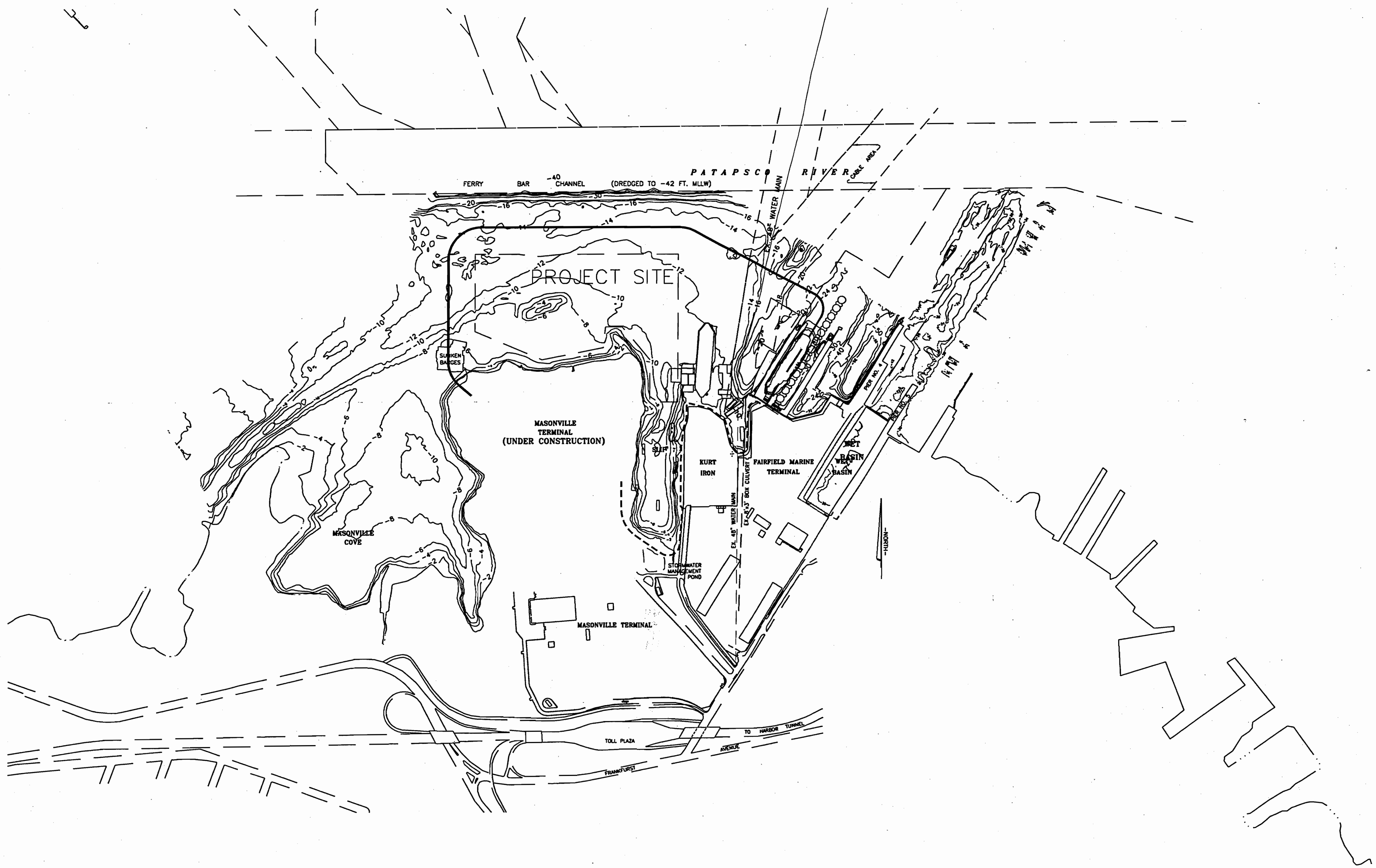
FINDLING, INC.

Site Vicinity Map

Masonville Marine Terminal

Baltimore, Maryland

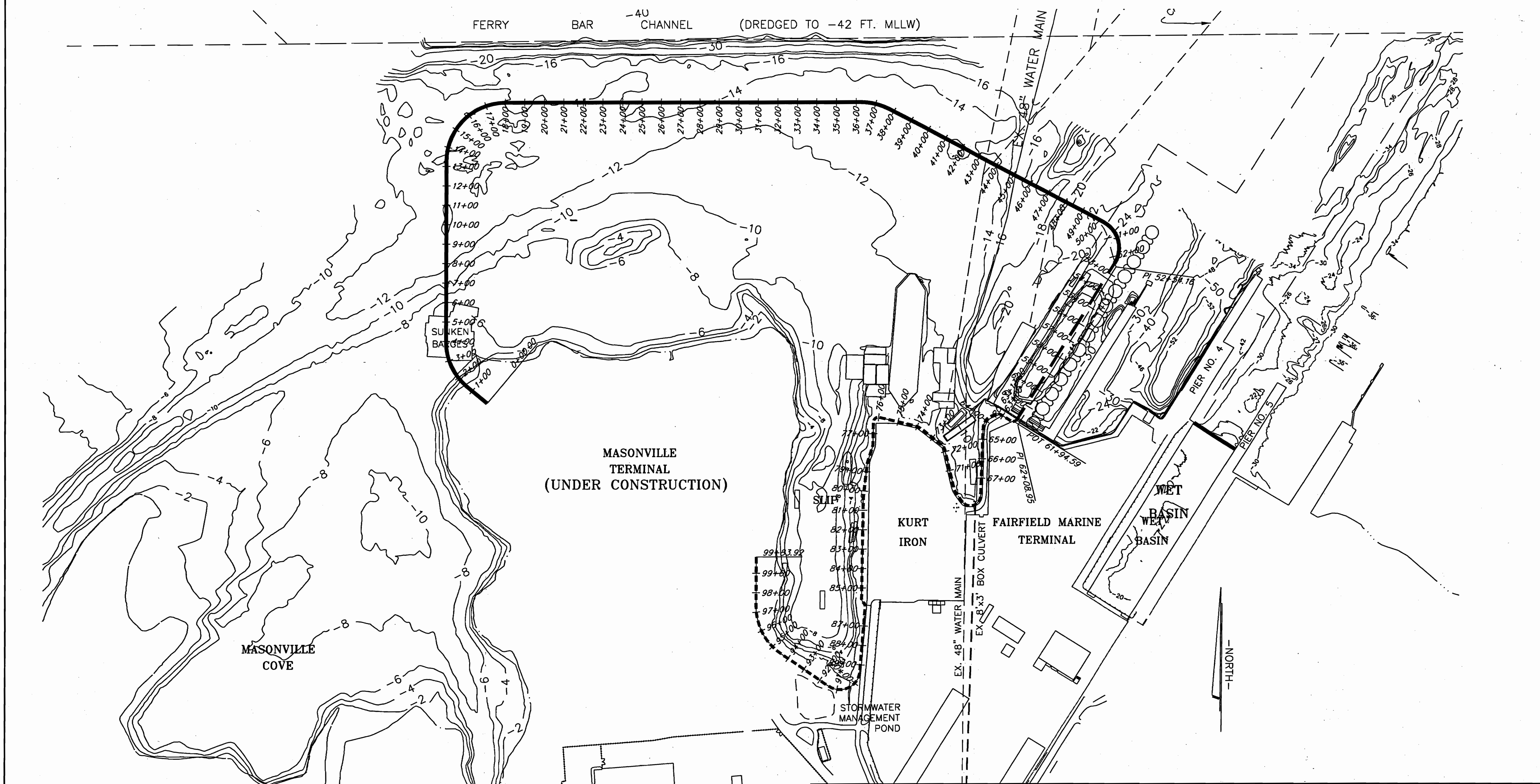
FIGURE:	DRAWN BY:	CHECKED BY:
1	JG	EHD
DATE:	**JOB NO.:**	**SCALE:**
Sep-04	04-1009	As Shown



Findling, Inc.

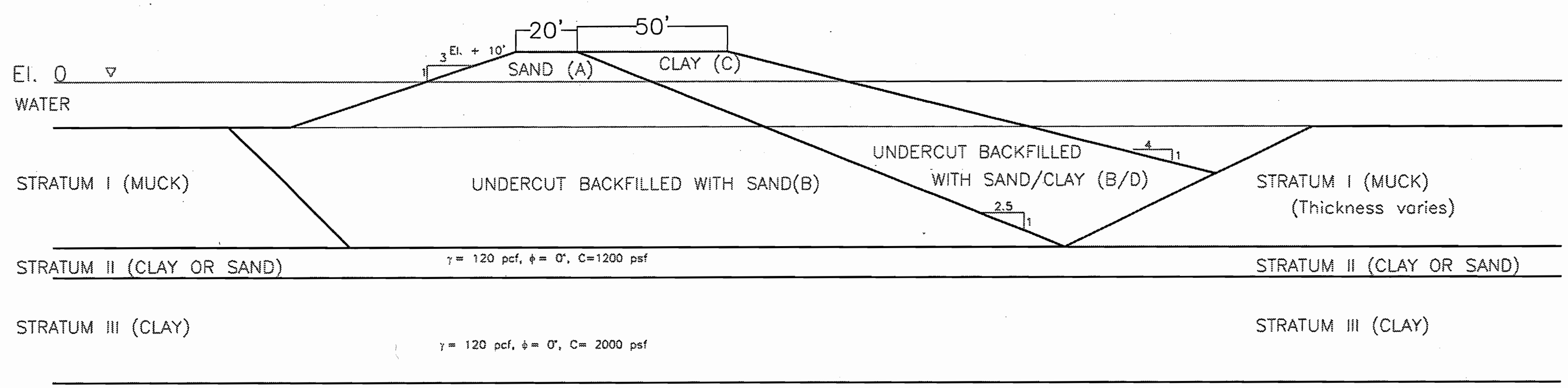
Project Location

DRAWING NO.: Figure 2	DRAWN BY:	CHECKED BY:
DATE:	JOB NO.:	SCALE: 1" = 1000'



Findling, Inc.

DIKE ALIGNMENT		
DRAWING NO.: FIGURE 3	DRAWN BY:	CHECKED BY:
DATE:	JOB NO.:	SCALE: 1" = 500'



TYPICAL SECTION -- CREST OF DIKE AT El. +10 MLLW

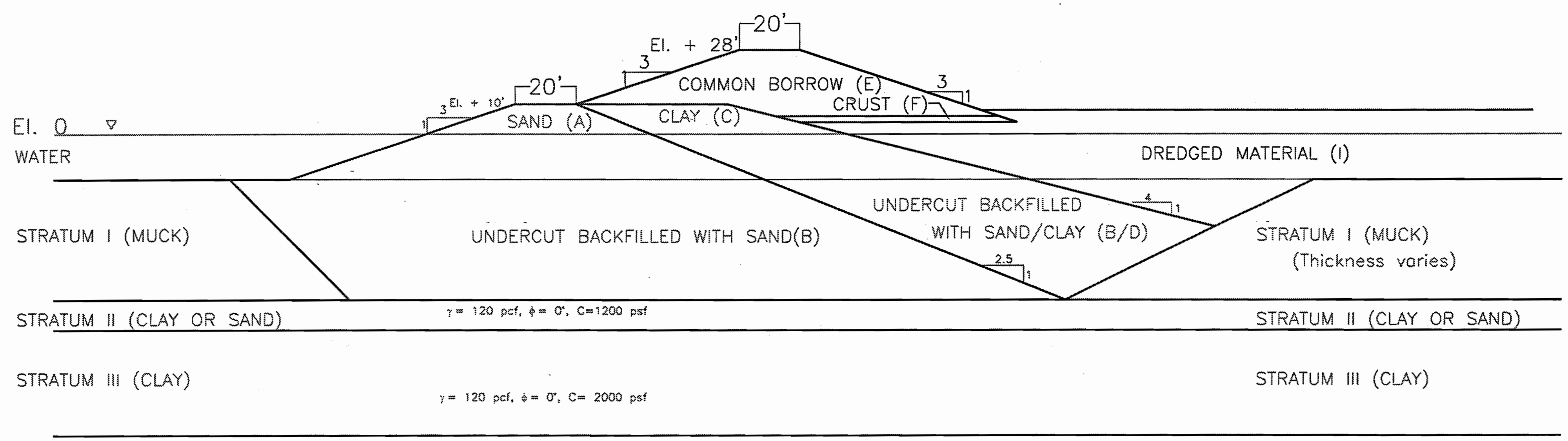
DIKE MATERIALS:	γ	ϕ	C
A. Sand above water	120	30	0
B. Sand Below water	115	28	0
C. Clay above water	120	0	500
D. Clay below water	120	0	500
E. Common Borrow	120	0	1.500
F. New Crust	120	0	300
G. Old Crust	120	0	400
H. Dried Dredge Material	120	0	1,200
I. Dredge Material	90	0	100

Scale 1" = 50'

Findling, Inc.

MASONVILLE DMCF TYPICAL SECTION

DRAWING NO.: Fig. 4	DRAWN BY:	CHECKED BY:
DATE: Revised Feb. 24, 2006	JOB NO.:	SCALE: As Shown



TYPICAL SECTION -- CREST OF DIKE AT El. +28 MLLW

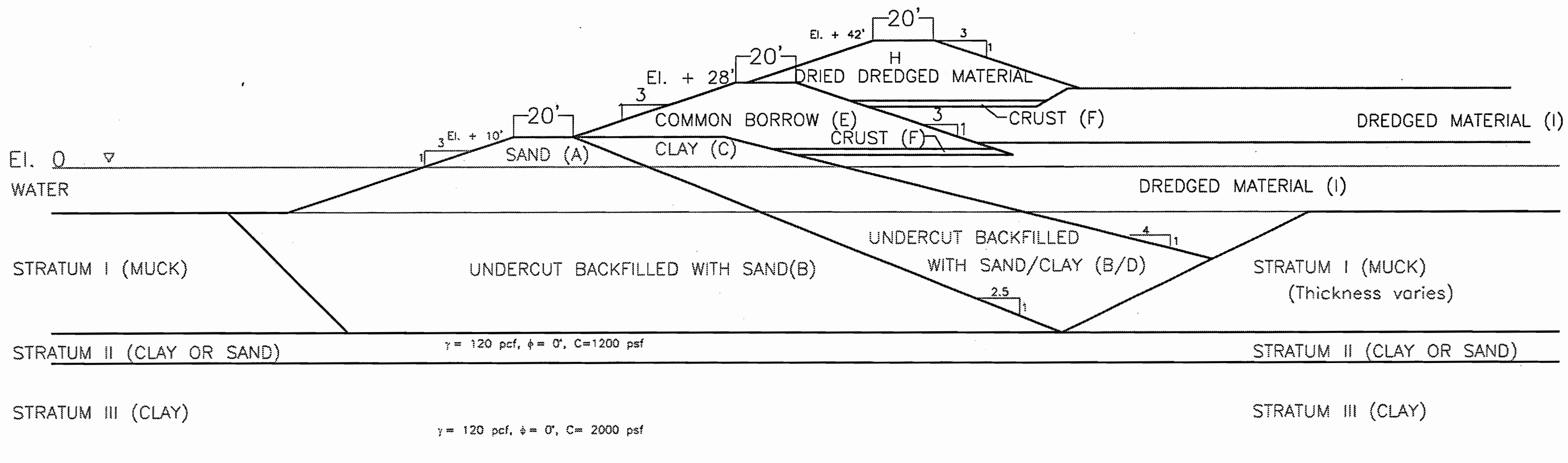
DIKE MATERIALS:	γ	φ	C
A. Sand above water	120	30	0
B. Sand Below water	115	28	0
C. Clay above water	120	0	500
D. Clay below water	120	0	500
E. Common Borrow	120	0	1.500
F. New Crust	120	0	300
G. Old Crust	120	0	400
H. Dried Dredge Material	120	0	1,200
I. Dredge Material	90	0	100

Scale 1" = 50'

Findling, Inc.

MASONVILLE DMCF
TYPICAL SECTION

DRAWING NO.: Fig. 5	DRAWN BY:	CHECKED BY:
DATE: Revised Feb. 24, 2006	JOB NO.:	SCALE: As Shown



TYPICAL SECTION -- CREST OF DIKE AT El. +42 MLLW

TYPICAL SECTION -- CREST OF DIKE AT El. +10 MLLW

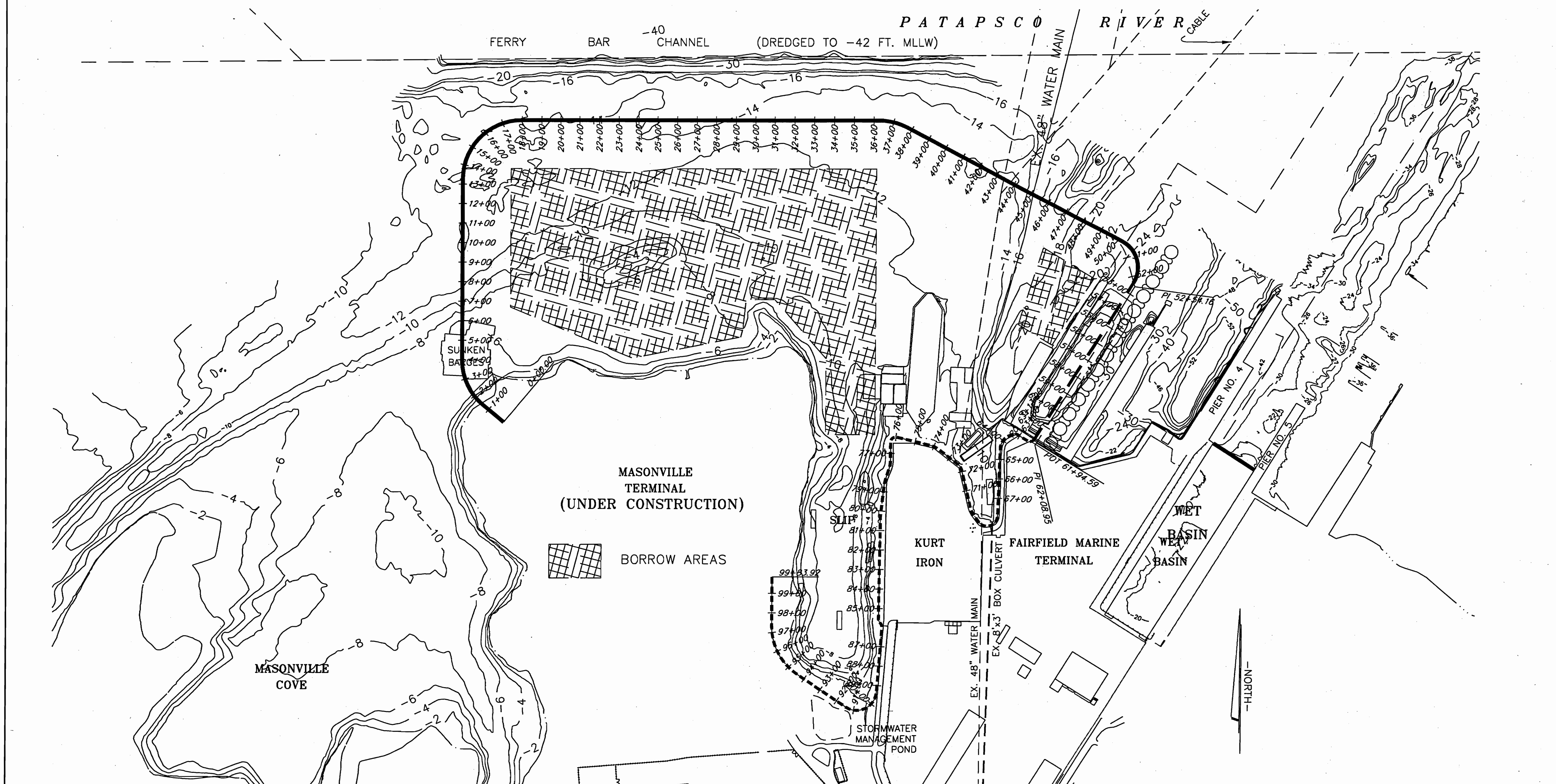
Scale 1" = 50'

DIKE MATERIALS:	γ	ϕ	C
A. Sand above water	120	30	0
B. Sand Below water	115	28	0
C. Clay above water	120	0	500
D. Clay below water	120	0	500
E. Common Borrow	120	0	1.500
F. New Crust	120	0	300
G. Old Crust	120	0	400
H. Dried Dredge Material	120	0	1,200
I. Dredge Material	90	0	100

Findling, Inc.

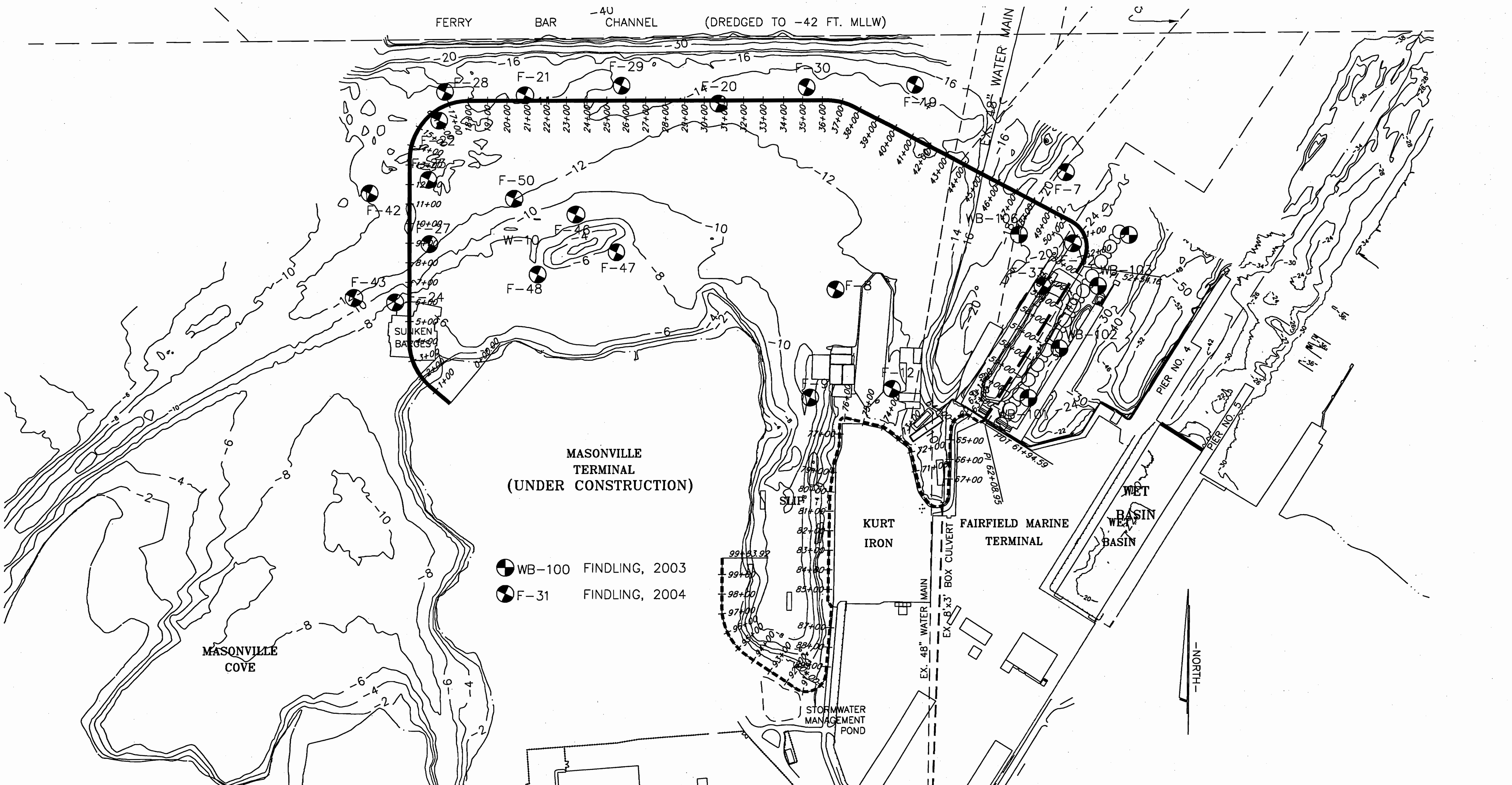
MASONVILLE DMCF
TYPICAL SECTION

DRAWING NO.: Fig. 6	DRAWN BY:	CHECKED BY:
DATE: Revised Feb. 24, 2006	JOB NO.:	SCALE: As Shown



Findling, Inc.

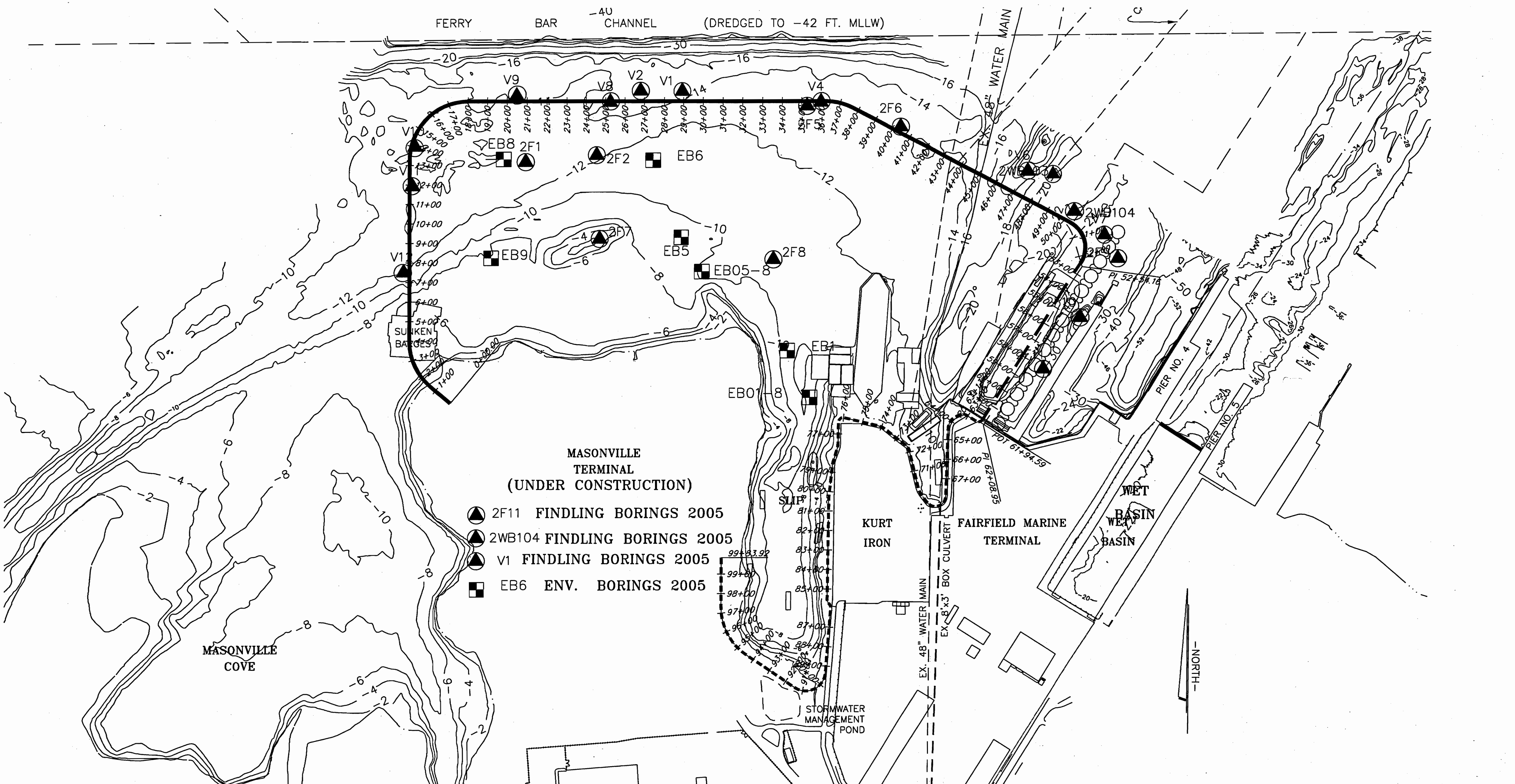
BORROW AREAS		
DRAWING NO.: FIGURE 7	DRAWN BY:	CHECKED BY:
DATE:	JOB NO.:	SCALE: 1" = 500'



Findling, Inc.

BORING LOCATION PLAN
2004 BORINGS

DRAWING NO.: FIGURE 8	DRAWN BY:	CHECKED BY:
DATE:	JOB NO.:	SCALE: 1" =500'



Findling, Inc.

BORING LOCATION PLAN
2005 BORINGS

DRAWING NO.: FIGURE 9	DRAWN BY:	CHECKED BY:
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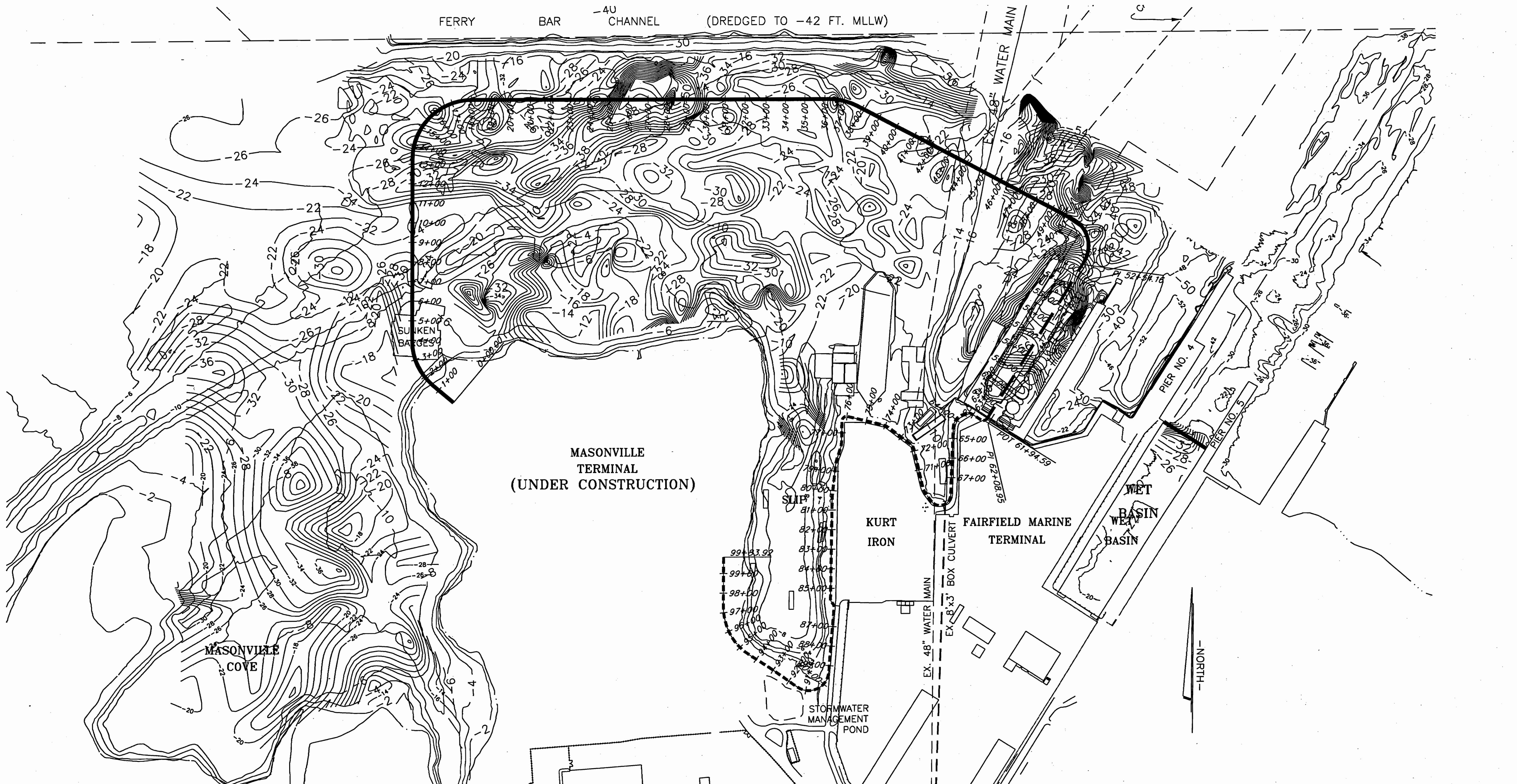

Findling, Inc.

PROBE DATA CONTOURS

DRAWING NO.: FIGURE 10	DRAWN BY:	CHECKED BY:
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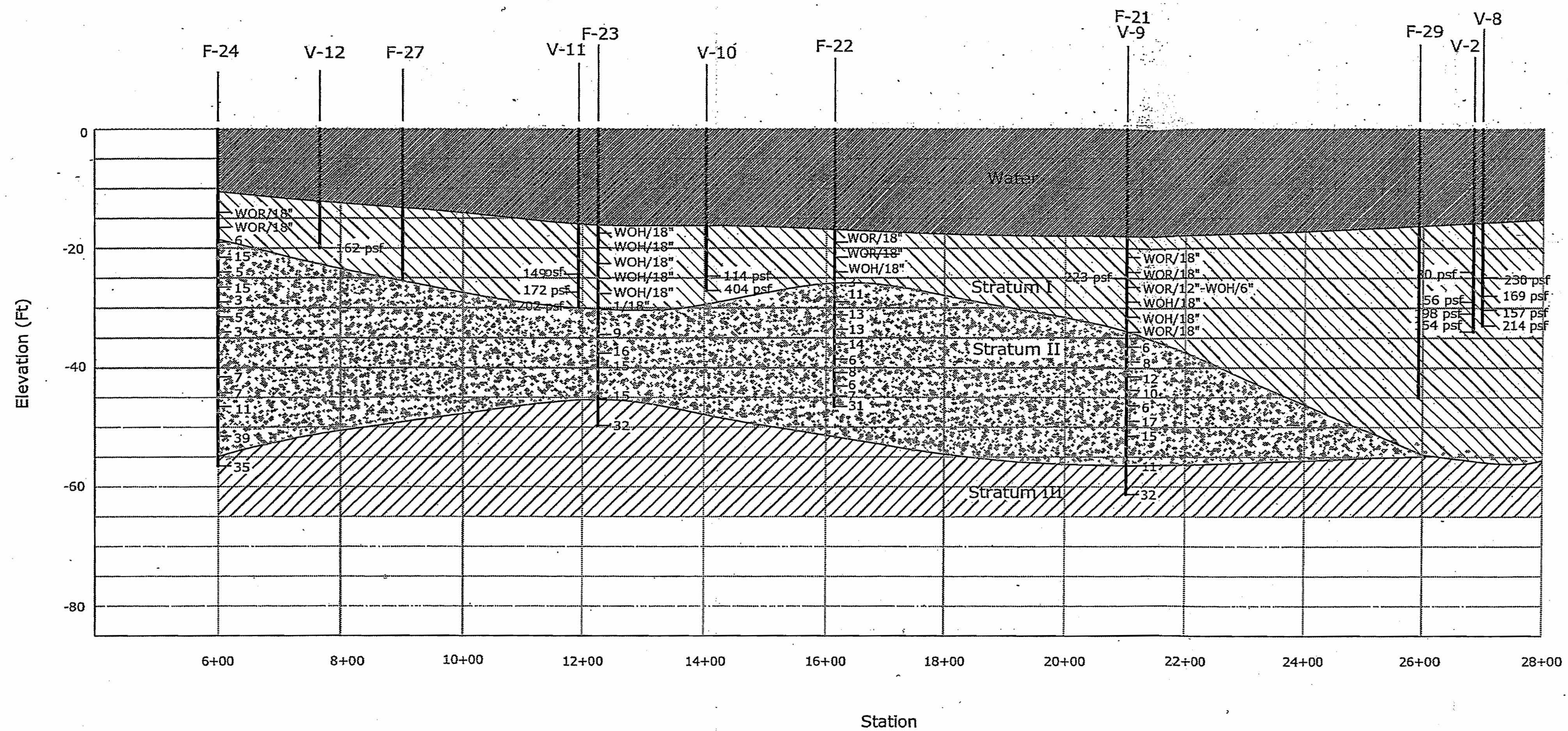

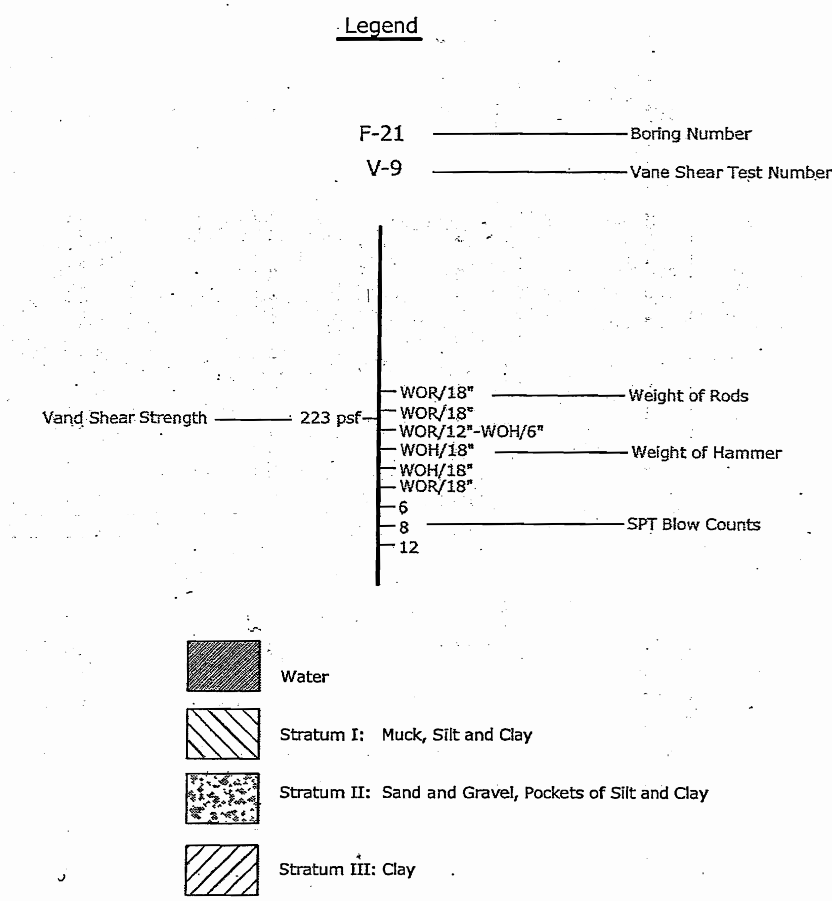

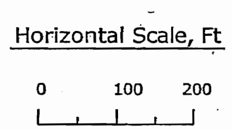

Findling, Inc.

GENERALIZED SUBSURFACE PROFILE

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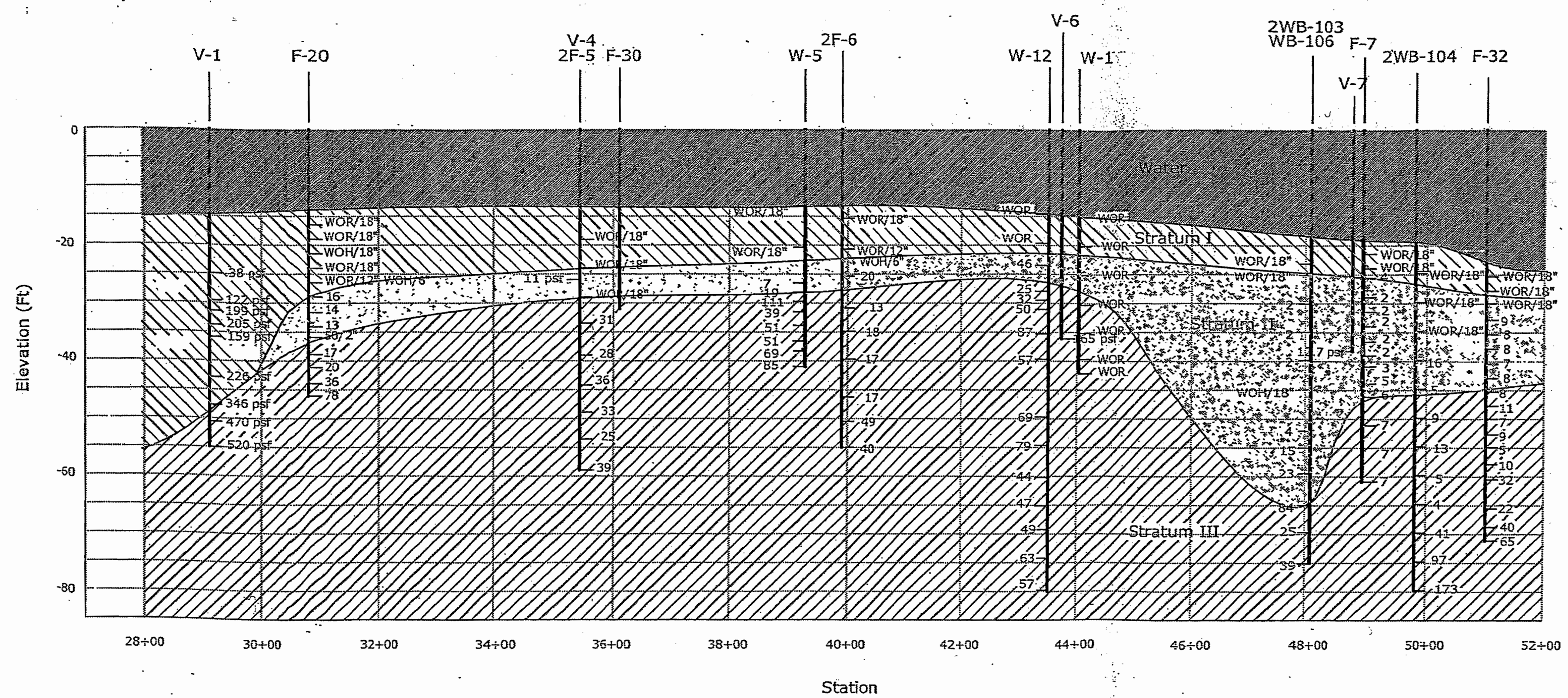
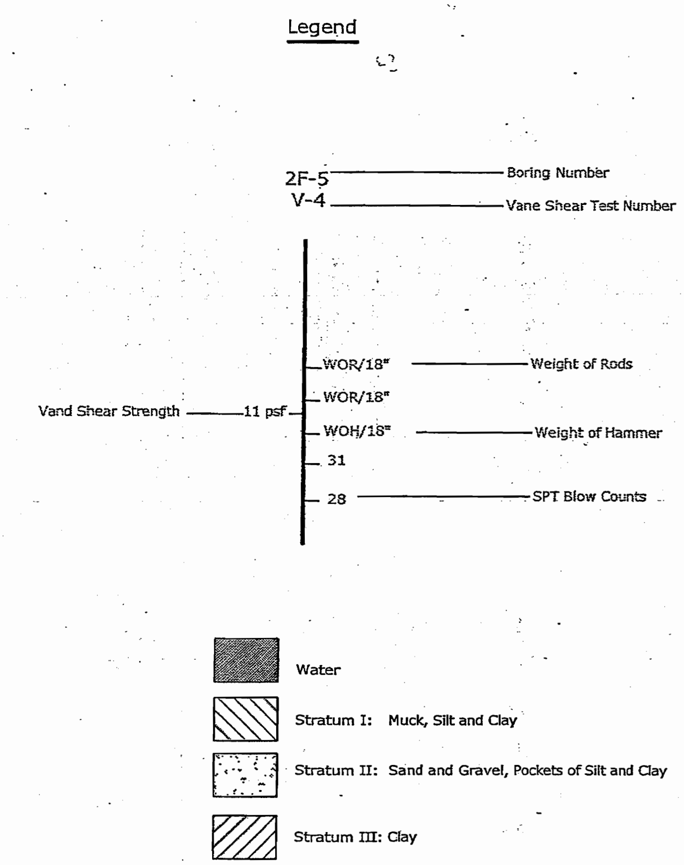
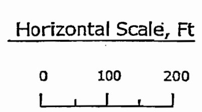
DATE:	JOB NO.: 04-1009	SCALE: As Shown







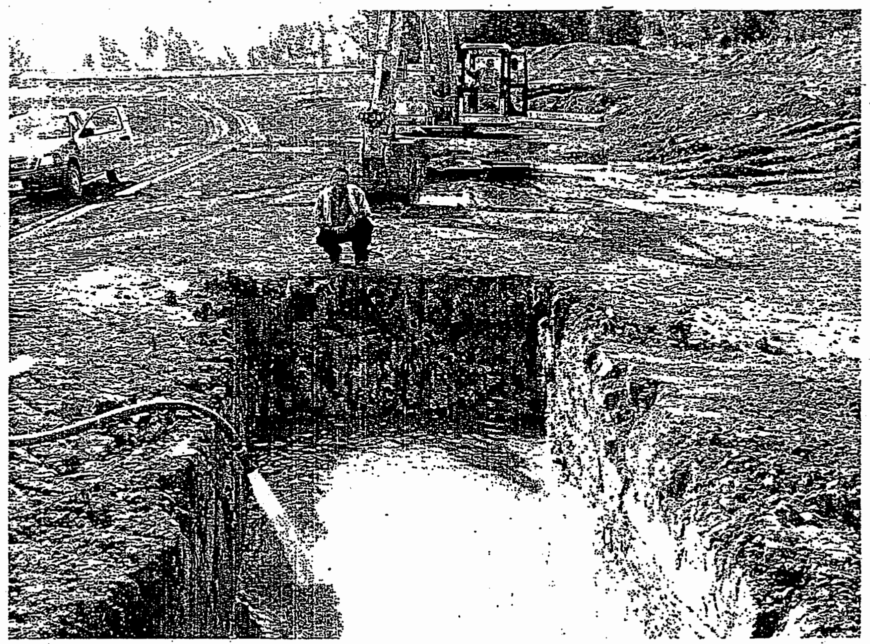
Findling, Inc.

GENERALIZED SUBSURFACE PROFILE

DRAWING NO.: 12	DRAWN BY:	CHECKED BY:
DATE:	JOB NO.: 04-1009	SCALE: As Shown

FIGURE 13: TESTING CLAY FILL UNDERWATER

Test Pit Being Filled With Water



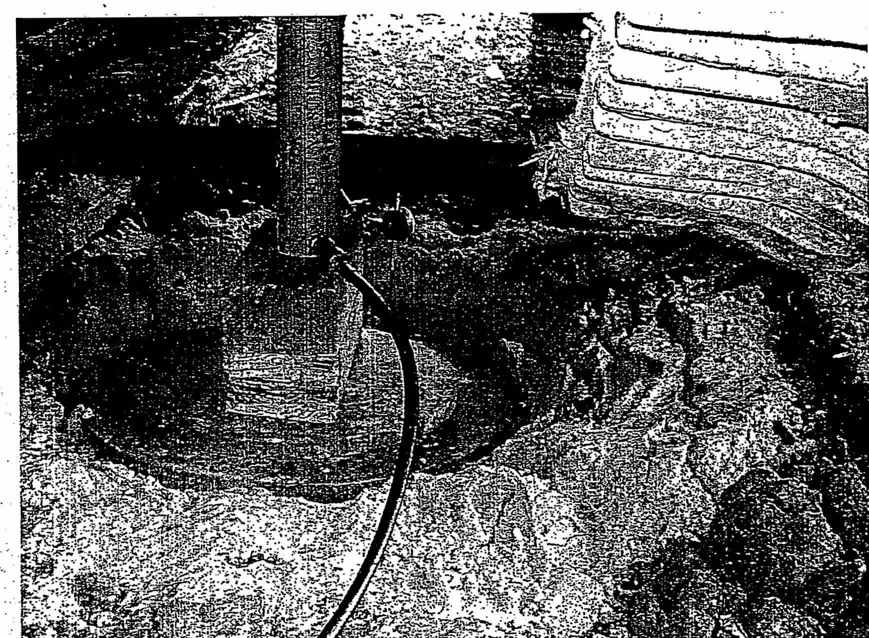
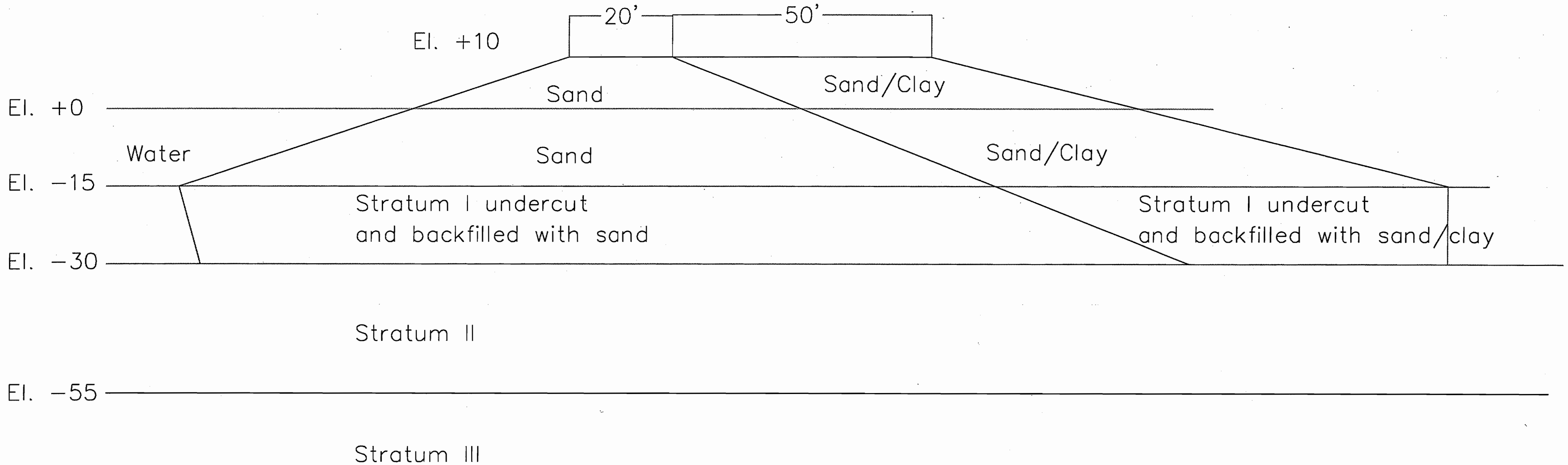
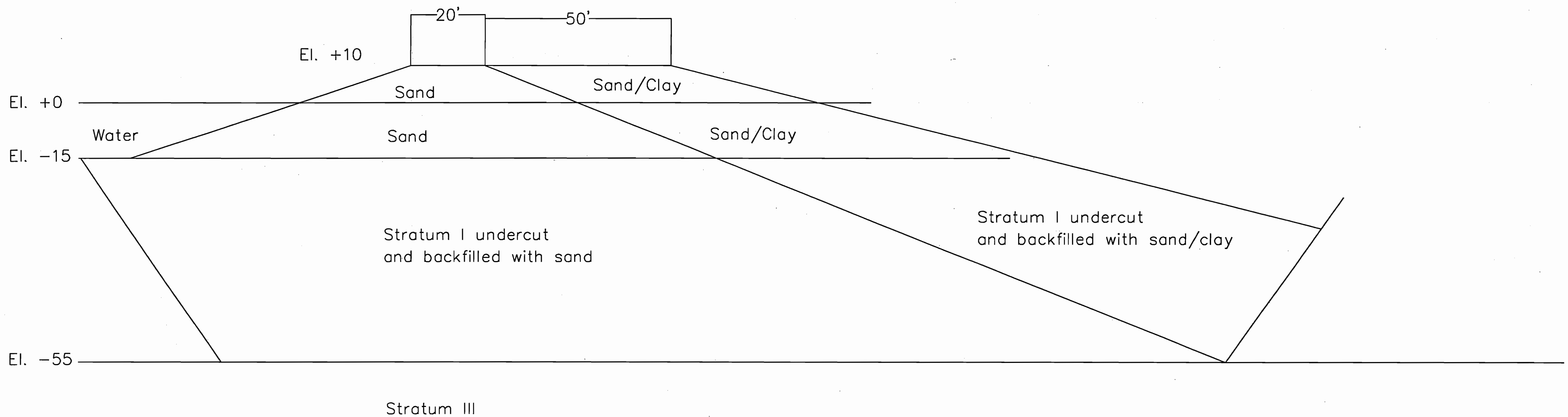


Plate Load Test on Clay Fill



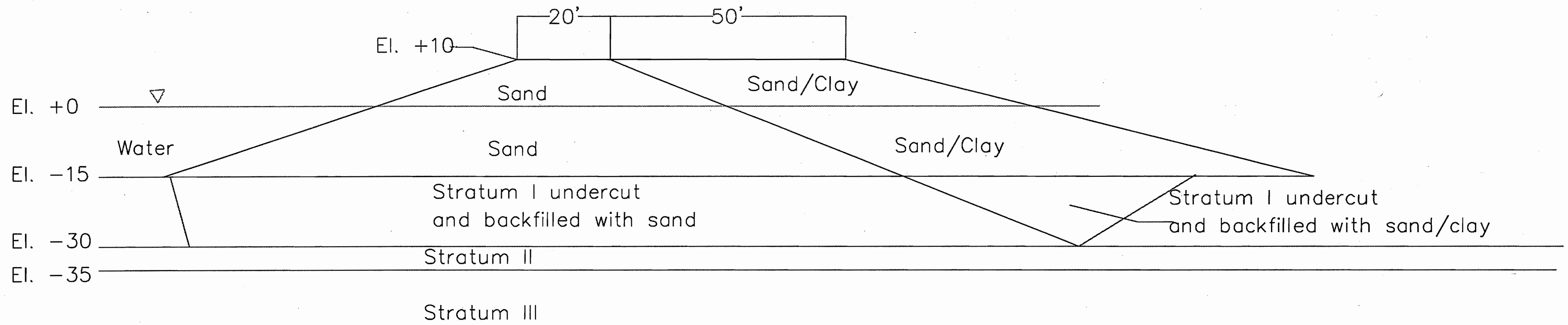
SUBSURFACE CONDITIONS -- SECTION A
STATION 6 TO STATION 21

Findling, Inc.	MASONVILLE DMCF		
	DRAWING NO.: Fig. 14	DRAWN BY:	CHECKED BY:
	DATE:	JOB NO.:	SCALE: NTS



SURFACE CONDITIONS -- SECTION B
STATION 23 TO STATION 29

Findling, Inc.	MASONVILLE DMCF	
DRAWING NO.: Figure 15	DRAWN BY:	CHECKED BY:
DATE:	JOB NO.:	SCALE: NTS



SUBSURFACE CONDITIONS -- SECTION C
STATION 31 TO STATION 44

Findling, Inc.	MASONVILLE DMCF		
	DRAWING NO.: Fig. 16	DRAWN BY:	CHECKED BY:
	DATE: Jan. 31, 2006	JOB NO.:	SCALE: NTS

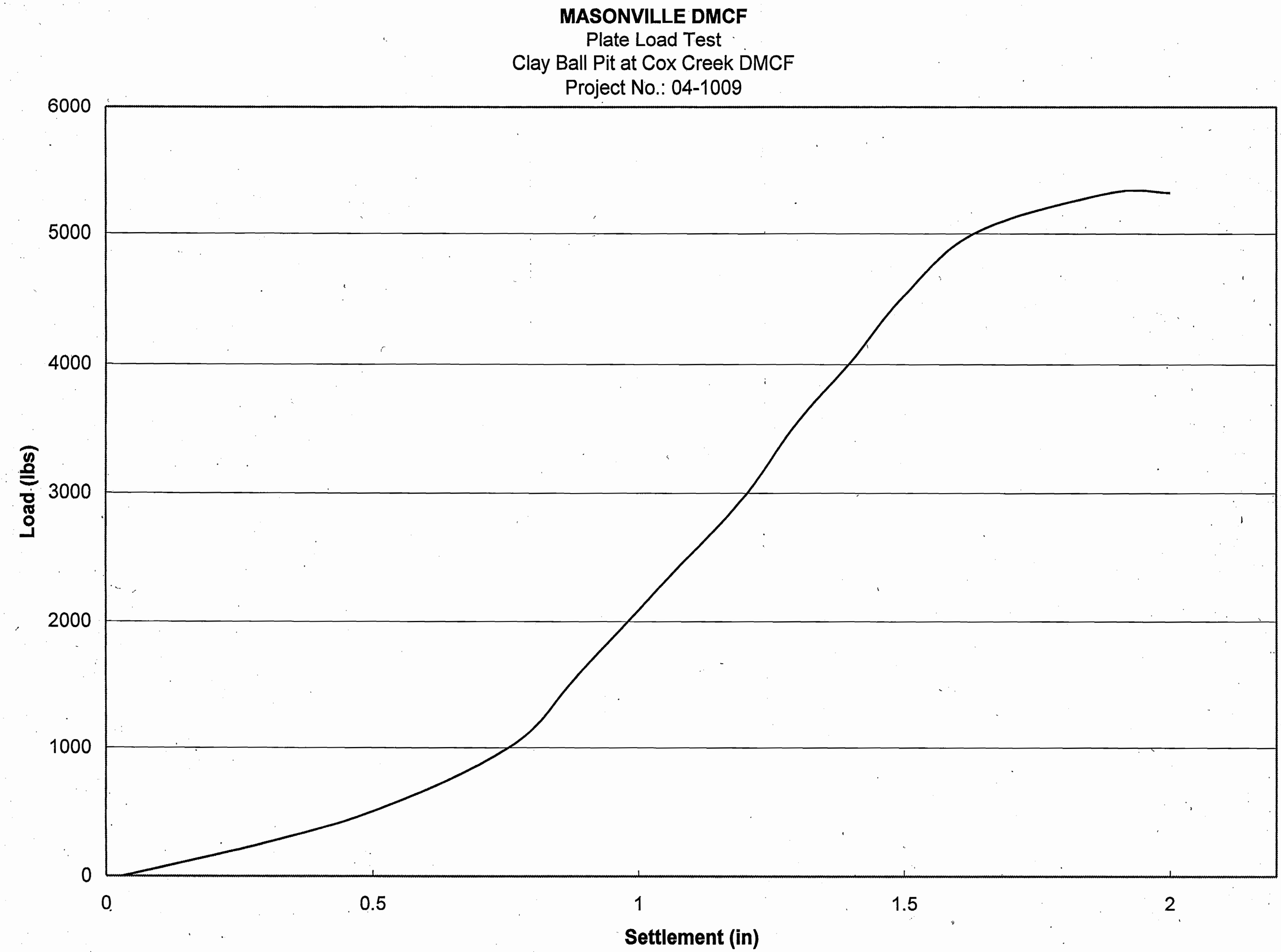

MASONVILLE DMCF
Plate Load Test
Clay Ball Pit at Cox Creek DMCF
Project No.: 04-1009
Load (lbs)
6000
5000
4000
3000
2000
1000
0
0
0.5
1
1.5
2
Settlement (in)

Figure 17

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APPENDIX B

TABLES

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LIST OF TABLES

TABLE 1: SUMMARY OF IN-SITU VANE SHEAR DATA

Masonville Marine Terminal
Findling Project No. 04-1009

LOCATION	SAMPLE	DEPTH OF WATER (ft.)	DEPTH BELOW MUDLINE (ft.)	SHEAR STRENGTH (psf)		SENSITIVITY
				Undisturbed	Remolded	
VS-1	V-1	14	22	140	70	2
	V-2	14	26	390	260	1.5
	V-3	14	28	390	260	1.5
	V-4	14	31	430	-	-
	V-5	14	33	520	415	1.25
	V-6	14	34	520	-	-
	V-7	14	35	650	450	1.5
	V-8	14	38	430	-	-
VS-2	V-1	14	24	350	300	1.2
	V-2	14	34	480	390	1.2
	V-3	14	44	519	475	1.1
VS-4	V-1	15	11	10		
VS-6	V-1	22	13	165		
VS-7	V-1	27	13	10		
VS-8	V-1	15	9	230		
	V-2	15	12	170		
	V-3	15	15	160		
	V-4	15	16	210		

LOCATION	SAMPLE	DEPTH OF WATER (ft.)	DEPTH BELOW MUDLINE (ft.)	SHEAR STRENGTH (psf)		SENSITIVITY
				Undisturbed	Remolded	
VS-9	V-1	15	10	220		
VS-10	V-1	14	11	115		
	V-2	14	12	405		
VS-11	V-1	14	10	150		
	V-2	14	13	170		
	V-3	14	16	200		
VS-12	V-1	13	7	160		

TABLE 2: SUMMARY OF LABORATORY TEST DATA

Masonville Marine Terminal

Findling Project No.: 04-1009

Boring No.	Sample No.	Depth (ft.)		Water Content %	Atterberg Limits		Cohesion (psf)	% Fines	USC	Stratum
		Below Water	Below Mudline		LL	PI				
VS-1	U-1	39	25	142.5	71	21	180		OH	I
	U-2	44	30	132.4	77	26	40		OH	I
	U-3	54	40	129.5	67	15	40		OH	I
VS-2	U-1	35	21	149.8	72	20	160		OH	I
	U-2	44	30	141.7	71	12	290		OH	I
	U-3	54	40	137.1	86	18	230		OH	I
2F-1	7	45	30					17.5	SM	II
	8	50	35					78.6	CL	III
2F-2	7	46	32					5.2	GW	II
2F-3	4	26	14					5.3	GW	II
	5	30	18					48.4	SM	II
	6	36	24					87.9	ML	II
	7	40	28					51.0	SM	II
2F-4	3	20	8					48.2	SM	II
	6	35	23					26.8	SM	II

Boring No.	Sample No.	Depth		Water Content %	Atterberg Limits		Cohesion (psf)	% Fines	USC	Stratum
		Below Water	Below Mudline		LL	PI				
2F-5	2	20	8		44	20				I
	3	25	13					41	SM	II
	4	29	17		39	20			CL	III
	8	50	38		71	44			CH	III
2F-6	3	25	12					9.8	SM-SP	II
	5	35	22		55	31				III
	9	55	42		74	47				III
2F-7	4	26	16					65	ML	II
	6	36	26					23.5	SM	II
2F-8	7	40	29					40	SM	II
	9	51	40					22.5	GM	II
	10	55	44					23.4	GM	II

TABLE 3: BORROW AREA CHARACTERISTICS

Masonville Marine Terminal
Findling Project No. 04-1009

BORING	WATER DEPTH	THICKNESS OF STRATUM I	THICKNESS OF SAND	REMARKS	% FINES		
					Sample No.	Elevation	% Fines
F-46	11	17	24	Gravel, cobble, clay pockets	S-7	-28	60
					S-8	-29	25
					S-10	-35	6
					S-11	-40	80
					S-12	-45	20
					S-13	-50	34
F-47	9.5	6.5	15	Gravel, cobble, Clay pockets	S-7	-25	52
					S-8	-27	56
					S-9	-30	45
					S-10	-35	84
					S-11	-40	57
F-48	6	0	53	Gravel, cobble, boulders, clay pockets	S-3	-13	52
					S-4	-15	27
					S-5	-17	27
					S-6	-20	11
					S-7	-23	7
					S-12	-43	73
					S-14	-53	4
					S-16	-63	93

BORING	WATER DEPTH	THICKNESS OF STRATUM I	THICKNESS OF SAND	REMARKS	% FINES		
					Sample No.	Elevation	% Fines
F-50	14	17	36	Gravel, cobbles, Pockets of clay	S-5	-24	84
					S-6	-26	16
					S-7	-30	79
					S-8	-32	33
					S-9	-35	42
					S-10	-40	8
					S-12	-50	4
					S-13	-55	25
					S-14	-60	3
					S-16	-70	80
2F-1	15	21	14	Clay pockets	S-5	-30	96
					S-7	-47	18
					S-8		79
2F-2	14	31	15	Gravel, cobbles	S-7	-45	5
2F-3	12	4	36	Pockets of clay	S-4	25	5
					S-5	-30	48
					S-6	-35	88
					S-7	-40	51

BORING	WATER DEPTH	THICKNESS OF STRATUM I	THICKNESS OF SAND	REMARKS	% FINES		
					Sample No.	Elevation	% Fines
2F-4	12	4	18	Gravel, cobble, Clay pockets	S-3	-20	48
					S-5	-30	27
2F-7	9	16	40	Pockets of clay	S-4	-25	65
					S-6	-35	24
2F-8	11	24	25	Gravel, cobbles, pockets of clay	S-7	-40	40
					S-9	-50	23
					S-10	-55	23

TABLE 4: FACTOR OF SAFETY

Masonville Marine Terminal
Findling Project No. 04-1009

SECTION/ TYPE	INITIAL DIKE		RAISED DIKE		RAISED DIKE		FAILURE MODE	FACTOR OF SAFETY
	Top Elevation	Width (ft.)	Top Elevation	Width	Top Elevation	Width		
A / I	+10	20 + 50	-	-	-	-	Outside	1.4
	+10	20 + 50	-	-	-	-	Inside	1.3
	+10	20 + 50	28	20	-	-	Outside	1.6
	+10	20 + 50	28	20	-	-	Inside	0.99
	+10	20 + 50	28	20	42	20	Outside	1.3
	+10	20 + 50	28	20	42	20	Inside	1.1
B / 2	+10	20 + 50	-	-	-	-	Outside	1.3
	+10	20 + 50	-	-	-	-	Inside	1.1
	+10	20 + 50	28	20	-	-	Outside	1.6
	+10	20 + 50	28	20	-	-	Inside	1.0
	+10	20 + 50	28	20	42	20	Outside	1.3
	+10	20 + 50	28	20	42	20	Inside	1.1
C / 3	+10	20 + 50	-	-	-	-	Outside	1.4
	+10	20 + 50	-	-	-	-	Inside	1.30
	+10	20 + 50	28	20	-	-	Outside	1.6
	+10	20 + 50	28	20	-	-	Inside	1.1
	+10	20 + 50	28	20	42	20	Outside, shallow	1.4
	+10	20 + 5	28	20	42	20	Outside, shallow	1.0

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