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4741 GEOTECHNICAL REPORT
(ISSUED 6-12-08)

Rivergate Scrap Metals, Inc.
P.O. Box 83169
Portland, OR 97283

Attention: Steve Lovin

SUBJECT: Geotechnical Investigation
Rivergate Scrap Metals Shredder
9707 **~~9645~~ N. Columbia Boulevard**
Portland, Oregon

1924

2N1W36CB 00100

At your request, GRI has conducted a geotechnical investigation for the above-referenced project. The general location of the site is shown on the Vicinity Map, Figure 1. The site is located between N. Columbia Boulevard and a rail line that runs along the southern bank of the Columbia Slough in the Rivergate Industrial Area of Portland, Oregon. Background information and a summary of our previous preliminary recommendations for the design and construction of the project are provided in the following:

Final Report "Subsurface Investigation, Elkem Metals Facility, Portland, Oregon," dated August 12, 1982; prepared by Foundation Sciences, Inc.

Final Report, "Foundation Investigation, Taulman/Weiss Composting Tank System, Columbia Boulevard Sewage Treatment Plant, Portland, Oregon," dated October 1, 1982; prepared by Foundation Sciences, Inc.

Memorandum, "Geotechnical Consultation, Rivergate Scrap Metals Shredder, Preliminary Foundation Design Considerations, N. Columbia Boulevard Property, Portland, Oregon," dated September 12, 2007; prepared by GRI.

Memorandum, "Summary of Chemical Analysis, Rivergate Scrap Metals, Inc., Shredder Site, 9707 N. Columbia Boulevard, Portland, Oregon," dated January 4, 2008; prepared by GRI.

"Progress Memorandum, Development of Foundation Design Criteria, Rivergate Scrap Metals Shredder, N. Columbia Boulevard Property, Portland, Oregon," dated January 11, 2008 (issued 2-5-08); prepared by GRI.

This report documents the subsurface materials and conditions previously disclosed by our field explorations at the site and provides additional recommendations for the design and construction of earthwork and foundations for the new facility.

SITE AND PROJECT DESCRIPTION

The Site Plan, Figure 2, shows the proposed layout of the new shredder system on a property that was most recently used for a composting business. The existing Rivergate Scrap Metals facility operates on the adjoining property to the east. Two ponds once occupied these properties and contained a very soft and highly compressible sludge that resembles a fine-grained soil such as silt. The sludge was estimated to be about 20 ft thick and was subsequently capped in place with a cover of fill soils. The original outlines of the ponds are shown on Figure 2. The ponds are underlain by medium dense sand which is underlain by dense sand and gravel. Details regarding the final closure of the ponds are presently being negotiated between Elkem Metals, Union Carbide, and the Oregon Department of Environmental Quality (DEQ). The potential for migration of contamination from the ponds to the nearby Columbia Slough has been a concern of the DEQ.

As planned, deeply driven piles will support the heaviest elements of the new shredder system, which consist of the shredder box, drive motor, and control building. The footprint of the project area will be overexcavated to the bottom of the sludge and backfilled with compacted structural fill to reduce concerns for future contaminant migration downward along the sides of the piles. Shallow spread footings or mat foundations will support the lighter elements of the system, which consist of various conveyors and other equipment and a small, metal-framed maintenance building. The site will be paved with concrete and asphaltic pavements with the concrete provided in the heavy-use areas.

SUBSURFACE CONDITIONS

General

Subsurface conditions at the site were evaluated with three borings, designated B-1 through B-3, made at the approximate locations indicated on Figure 2. The borings were completed between October 24 and November 1, 2007, and were advanced to depths of about 82 to 101 ft. Boring B-1 was located farthest from the adjacent Columbia Slough, borings B-1 and B-2 were drilled through the capped sludge pond, and boring B-3 was completed closest to the slough and through the containment dike that separated the sludge pond from the railroad and slough. A detailed discussion of the field exploration and laboratory testing programs completed for this investigation is provided in Appendix A. Logs of the borings are shown on Figures 1A through 3A. The terms used to describe the materials encountered in the borings are defined in Table 1A.

Soils

For the purpose of discussion, the materials and soils disclosed by the borings have been grouped into the following categories:

1. **FILL**
2. **SLUDGE**
3. **SAND and SILT**
4. **GRAVEL**

1. FILL. Borings B-1 and B-2 were drilled through the capped sludge pond, and the fill cap in these locations appears to extend to depths of 7.5 and 10 ft, respectively. Some fragments of construction debris, such as pieces of broken brick, concrete, and asphalt pavement, are present at the ground surface in the area. The cap material encountered in boring B-1 appears to mostly consist of gray, gravelly silt. A Standard Penetration Test N-value of 10 blows/ft indicates the relative consistency of the silt fill is stiff. Boring B-2 encountered approximately 4-in. of asphaltic concrete (AC) pavement over 6-in. of crushed rock at the ground surface. The pavement is underlain by cap material that appears to consist mostly of brown, silty sand. An N-value of 12 blows/ft indicates the relative density of the sand fill is medium dense. Boring B-3 was drilled through the containment dike that separated the sludge pond from the railroad and the slough and encountered approximately 5.5 ft of gravel fill at the ground surface.

2. SLUDGE. Sludge resembling black silt was encountered beneath the cap fill in borings B-1 and B-2, and extended to depths of about 20 and 17 ft, respectively. N-values of 1 to 2 blows/ft and Torvane shear strength values of 0.30 to 0.33 tsf indicate the relative consistency of the sludge is very soft to medium stiff. Natural moisture contents range from about 28 to 153%.

3. SAND and SILT. Alluvial floodplain deposits consisting of interbedded layers of sand and silt were encountered in borings B-1 through B-3 beneath the fill and sludge. The silt layers contain varying percentages of sand and clay, ranging from a trace to sandy or clayey, and scattered wood fragments. N-values of 4 to 14 blows/ft indicate the relative consistency of the silt is generally medium stiff to stiff. Natural moisture contents range from about 30 to 95%. The sand layers contain varying percentages of silt, ranging from a trace to silty, and scattered gravel. N-values of 0 to 56 blows/ft indicate the relative density of the sand varies widely from very loose to very dense and typically increases with depth. Natural moisture contents range from about 20 to 40%.

4. GRAVEL. Gravel was encountered below depths of 74, 77, and 77.5 ft in borings B-1 through B-3, respectively. The gravel appears generally fine to medium grained and is contained within a matrix of sand. N-values of 42 blows/ft to refusal (50 blows for less than 6 in. of sampler penetration) indicate the relative density of the gravel is dense to very dense. However, the Standard Penetration Test tends to overestimate the relative density of coarsely graded, granular soils.

Groundwater

Groundwater levels at the site will fluctuate in response to precipitation and the water level in the Columbia Slough. A network of piezometers and a groundwater observation well were installed in the borings, as discussed in Appendix A. The ground surface elevations shown on the boring logs were determined using direct optical survey. The elevations are referenced to an arbitrarily assumed elevation of 100 ft for the top of the monument that protects a previously installed monitoring well, designated MWP1-4, at the east end of the site, which was used as a temporary benchmark. A graph of groundwater levels versus time measured by the piezometers installed in the borings is provided on Figure 3. These data indicate the gradient across the site slopes downward toward the adjacent slough. We anticipate groundwater will periodically perch near the ground surface at the site as the result of heavy or prolonged precipitation and the relatively low permeability of the fill used to cap the sludge.

Soil Resistivity and pH

Two electrical resistivity survey lines, designated R-1 and R-2, were completed in accordance with section 7.2.4 of IEEE Standard 81-1983 (The IEEE Guide for Measuring Earth Resistivity, Ground Impedance, and Earth Surface Potentials of a Ground System). The survey lines were completed using electrode spacing ranging from 2 to 80 ft. Resistivity line R-1 was oriented north-to-south, line R-2 was oriented east-to-west, and both were centered near the main portion of the proposed location of the shredder. The results of the soil resistivity testing are tabulated below.

SOIL RESISTIVITY MEASUREMENTS

	Apparent Resistivity, ohm-ft	
Electrode Spacing, ft	Line R-1 (North-South)	Line R-2 (East-West)
2		402
4	189	229
6	134	114
8	104	90
10	97	90
12	96	85
16	75	64
20	72	61
25	75	68
30	72	61
35	72	64
40	79	63
50	85	73

In addition to the soil resistivity surveys, samples of the capping soils were collected from borings B-1 and B-2 and submitted to an independent testing laboratory for pH determinations. The test results indicate soil pH values of 7.6 and 11.0 for the samples from borings B-1 and B-2, respectively.

CONCLUSIONS AND RECOMMENDATIONS

General

The borings indicate between 7.5 and 10 ft of capping material was placed over the sludge, which appears to consist mostly of mixed silt and sand fill with some gravel and scattered construction debris. The sludge, which has the relative consistency of soft to medium stiff silt soil, extends to depths of 17 to 20 ft at the locations of borings B-2 and B-1, respectively. The underlying sludge is considered to be relatively weak and of high compressibility. The sludge is underlain by alluvial deposits of silt and sand that grade to dense sand below a depth of about 60 ft. Gravel underlies the sand below a depth of 74 ft and extends to the maximum depth explored. The measured groundwater gradient at the site is presently directed downward and towards the Columbia Slough. Groundwater levels at the site will fluctuate in response to precipitation and the water level in the Columbia Slough. Groundwater may also periodically perch locally within or above relatively impermeable fill materials that cap the site.

As previously indicated, the heaviest elements of the new shredder system, which consist of the shredder box, drive motor, and control building, will be supported on driven piles. In concurrence with the review of our preliminary design recommendations by the DEQ, the area within and surrounding the tops of the piles and pile cap will be overexcavated to the bottom of the sludge and backfilled with compacted structural fill to reduce concerns for future contaminant migration downward along the sides of the piles. The lighter elements of the system, which consist of various conveyors and other equipment and a small, metal-framed maintenance building, will be supported on shallow spread footings or mat foundations. The site will be paved with both concrete and asphaltic pavements with the concrete provided in the heavy-use areas.

In our opinion, the primary geotechnical issues associated with the new shredder are resisting operational vibrations, determining tolerable post-construction settlements of shallow foundations, and addressing the possible risks of liquefaction-induced settlement or lateral spreading at the site due to proximity to the Columbia Slough. The moisture sensitive nature of the fine-grained soils that cap the sludge and mantle the site will be an important construction consideration during site preparation and earthwork. The following sections of this report provide our additional conclusions and recommendations for the design and construction of the facility.

Earthwork

General. The relatively fine-grained, moisture-sensitive soils that appear to compose most of the capping material will likely have a moisture content generally above optimum for proper compaction during the majority of the year. Water levels at the site may approach the ground surface during the wet, winter season then fall slowly throughout the dry season. When fine-grained soils are in excess of their optimum moisture content, they tend to soften and become unstable when subjected to construction traffic. We anticipate the moisture content of the upper few feet of the soils that mantle the site will decrease relatively slowly during warm, dry weather. However, below this depth, the moisture content of the soils tends to remain relatively unchanged and is expected to remain well above optimum throughout the year.

Subgrade Preparation. Portions of the existing ground surface at the site are paved with asphalt, and other areas consist of exposed capping soils. The site is largely devoid of vegetation and stripping requirements will be minimal. However, the exposed soils at the site do not appear to have been compacted to the standards of structural fill. Consequently, we recommend moisture-conditioning the exposed ground surface, if necessary, and compacting it during dry weather with several passes with a segmented-pad roller. This subgrade preparation should be evaluated by the contractor and the geotechnical engineer to detect any loose, soft, or disturbed areas that may require additional overexcavation and replacement with structural fill. A working blanket of granular structural fill should then be provided to protect the underlying subgrade and to support construction activities and traffic, especially during wet weather.

Generally, a minimum 12- to 24-in.-thickness of relatively clean, fragmental rock placed over a geotextile fabric is required to protect the subgrade during wet weather; the appropriate thickness depends on the intensity of the construction traffic. A geotextile fabric, such as AMOCO 2006 or similar product, used between the granular fill and underlying subgrade soils serves as a separation layer to limit the movement of subgrade fines into the cleaner crushed rock and will tend to reduce maintenance of the section during construction.

Structural Fill

General. All structural fill materials should be compacted to at least 95% of the maximum dry density at a moisture content within about 3% of optimum, as determined by ASTM D 698, or until well keyed. The contractor's compactive effort should be evaluated on the basis of field observations and density testing at the time of construction, moisture contents should be adjusted accordingly by either wetting or drying, and lift thicknesses should be proportionate to the type of compaction equipment used to meet compaction specifications. Additional information regarding specific types of fill is provided below.

Granular Fill. Imported granular fill materials should consist of reasonably well-graded sand and gravel, or crushed rock with a maximum size on the order of 4 in. and not more than 5% passing the No. 200 sieve (washed analysis). Material particles should be relatively hard and durable. Material satisfying these requirements can be usually placed during periods of wet weather. Lift thicknesses should be based on the type of compaction equipment used. For example, we recommend limiting lift thicknesses to about 12 in. when using medium- to heavy-weight vibratory rollers, and to 6 in. when using hand-operated vibratory plates within more limited working areas.

Trench Backfill. Utility trench backfill should consist of granular fill limited to a maximum size of about 4 in., with a maximum size of about 3/4 in. for material used in the pipe zone. The granular trench backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 in the upper 4 ft of the trench and to at least 90% of this density below this depth. The use of hoe-mounted vibratory plate compactors is usually most efficient for this purpose. Lift thicknesses should be evaluated on the basis of field observations or density tests. Particular caution should be taken when operating hoe-mounted compactors to prevent damage to the newly placed conduits. Flooding or jetting to compact the trench backfill should not be permitted.

Excavations

The method of excavation and design of sidewall support are the responsibility of the contractor and subject to applicable local, state, and federal safety regulations, including the current OSHA excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. The information provided below is for use by the owner and engineer and should not be interpreted to mean that GRI is assuming responsibility for the contractor's actions or site safety.

We recommend that the capping soils and underlying sludge be classified as Type C soil according to the most recent OSHA regulations. In our opinion, all excavations should be safely sloped or shored. Special precautions should be taken to protect any adjacent, existing foundations or utilities and may require the use of temporary shoring or underpinning to safely accomplish the necessary excavations. The contractor should be aware that all excavation and shoring should conform to the requirements specified in the applicable local, state, and federal safety regulations, such as OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. We understand that such regulations are being strictly enforced, and if not followed, the contractor may be liable for substantial penalties.

Based on the groundwater levels measured in the piezometers and observation well, the top of the permanent groundwater table at the site appears to exist below the bottom of the sludge. However, as previously indicated, we anticipate that groundwater will periodically perch near the ground surface at the site as the result of heavy or prolonged precipitation and the relatively low permeability of the fill used to cap the sludge. Consequently, any groundwater coincident with excavation would likely be perched water that could be controlled by sump pumping. The control of groundwater will be minimized if the work is scheduled for the dry season.

As currently planned, all of the required overexcavation within the limits of the pile-founded structures will be accomplished during the dry season, which typically extends from mid-June through mid-October. If the earthwork is accomplished during this time, we are of the opinion that temporary excavation slopes should be made no steeper than 1H:1V. We also recommend that surcharge loads due to construction traffic, material laydown, excavation spoils, etc., should not be allowed within a distance equal to H/2 from the top of the cut, where H is height of the excavation. In this regard, we recommend using fencing or barricades along the top of the slope to protect this area from being subjected to any significant surcharge loads. It must be emphasized that following these recommendations will not guarantee that failure of the temporary cut slopes will not occur; however, the recommendations should reduce the risk of a major slope failure to an acceptable level. In this regard, the short-term slope stability of the temporary excavation slopes in the sludge is admittedly difficult to quantify, and it would be prudent for the contractor to closely monitor the temporary excavation slopes until they are backfilled.

Shallow Foundations

A system of conveyors will lead to and from the shredder box to other equipment such as separators or stackers that will be relatively lightly loaded. Provided the ends of the conveyors are simply supported, we anticipate they will be capable of tolerating large differential settlements. We further anticipate that lightly loaded equipment will also be tolerant of relatively large settlements or could be periodically re-leveled as necessary to allow these units to be supported on footings or shallow mat foundations. This would prevent the need for additional piles or overexcavation and replacement of the capping material and sludge beneath these lighter foundations.

Based on the borings completed between October 24 and November 1, 2007, the underlying sludge is relatively soft and compressible and appears to have been provided with a minimum 7.5-ft-thick cap. We recommend new mat foundations be embedded 24 in. and underlain by a minimum of 6 in. of compacted crushed rock, which should leave a minimum of about 5 ft of capping material in place beneath these new foundations. Settlement will occur beneath the dead load and any sustained or frequently applied live loads placed on shallow foundations due to the consolidation of the capping material and underlying sludge. In this regard, the allowable bearing pressure for shallow foundations will be controlled by settlement rather than ultimate bearing capacity considerations. The underlying sludge is considered to be highly compressible. Consequently, we recommend assuming an allowable soil bearing pressure of 1,000 psf for the design of shallow foundations and the support of normally applied live and dead loads. This value may be increased by one-third for the support of all loads such as infrequently applied wind or seismic forces. The dimensions of shallow foundations should be proportioned such that the resultant of the applied forces lies within the middle third of the overall foundation dimensions.

Estimates of primary settlement are provided on Figure 5 for various footing loads up to 200 kips for normally applied bearing pressures of up to 1,000 psf. These estimates assume that the underlying sludge is normally rather than over-consolidated and has a coefficient of compression on the order of about 0.20. These estimates may be used to proportion various shallow foundation sizes to minimize potential differential settlements between more lightly or heavily loaded units. However, it should be realized that some additional component of differential or total settlement may occur due to the inherent variations that likely exist in the compressibility of the capping materials and underlying sludge. In addition, some additional settlement should be expected due to long-term secondary consolidation and the vibrational forces which will be associated with the operating equipment. Some seismically induced settlement at the site resulting from a strong earthquake would be in addition to foundation settlements. Seismic considerations are further discussed in a following section of this report.

Horizontal forces may be resisted by friction developed beneath the base of a spread footing or mat and the underlying subgrade. We recommend that footings be underlain by a minimum 6-in. thickness of clean, compacted 3/4-in.-minus crushed rock, and an ultimate value of 0.35 may be assumed for the coefficient of friction beneath footings. If additional lateral resistance is required, passive earth pressure against the embedded sides of foundations can be computed using an equivalent fluid with a unit weight of 250 pcf, which assumes that lateral deflections will be relatively small and in the range of about 1/4 in. or less. This design passive earth pressure would be effective only if compacted granular structural fill is used for the backfill. This value also assumes the ground surface in front of the foundation is nearly horizontal, i.e., does not slope significantly downward away from the face of the footing. Foundation backfill and any other structural fill should be compacted to a minimum of 95% of the maximum dry density at moisture contents within about 3% of optimum as determined by ASTM D 698. Any uplift forces acting on the foundations would be best resisted by the weight of the foundation and the weight of any backfill placed over the foundation. The weight of any backfill placed over a foundation as structural fill may be assumed to be about 125 pcf.

All foundation excavations should be made using a smooth-edged bucket and observed by a geotechnical engineer. The subgrade soils will likely be disturbed during foundation excavation. The bottom of excavations should be moistened, if necessary, and compacted with a vibratory plate compactor. It should be anticipated that some overexcavation and replacement with structural fill might be necessary in the event that very soft soils are encountered at a shallow depth in the bottom of a foundation excavation.

For the design of slabs-on-grade to support floor, traffic, or equipment loads, we recommend these slabs be underlain by a minimum 12-in. thickness of 3/4-in.-minus crushed rock compacted to a minimum of 95% of the maximum dry density as determined by ASTM D 698, at a moisture content within about 3% of optimum. A coefficient of subgrade reaction of 250 pci may then be assumed for the design of slabs-on-grade.

Pile Foundations

General. In concurrence with the review of our preliminary design recommendations by the DEQ, the area within and surrounding the tops of the piles and pile cap will be overexcavated to the bottom of the sludge and backfilled with compacted structural fill to reduce concerns for future contaminant migration downward along the sides of the piles. Figure 4 illustrates this concept. In our opinion, compacted dredged sand would be better suited for use as structural fill than crushed rock for this purpose and subsequently driving the piles through the backfill. Our additional recommendations pertaining to the design and construction of pile foundations are provided below.

Axial Capacity. For a previous shredder project in the local area, PP24x0.500 steel pipe piles were installed. We anticipate these piles would also be well-suited to support the shredder box, drive motor, and control building for this project. We recommend installing these piles closed-ended and driving to practical refusal within the gravel that underlies the site below a depth of about 75 ft, using a relatively large impact hammer to drive the piles. Actual pile tip penetration into the gravel is difficult to predict; however, we would expect the pile tips to penetrate less than 20 ft into the very dense gravel deposit. The piles would develop their compressive capacity in a combination of outer friction and end bearing and their tensile capacity in outer skin friction. We estimate an ultimate compressive capacity for these piles of 440 tons, which includes about 125 tons of skin friction and 315 tons of end bearing. For design, we recommend assuming an allowable compressive capacity of 220 tons, which includes a factor of safety of 2based on normally applied dead and live loads and static soil support considerations. A minimum center-to-center spacing of three diameters should be provided to avoid potential group effects for the static axial loading of the piles.

The axial stiffness of the pile will depend somewhat on the load distribution and transfer to skin friction and end bearing, and will be progressively mobilized from the ground surface downwards. However, for preliminary purposes, a reasonable assumption could be based on an effective pile length of about 75 ft. The axial stiffness of the piles could also be increased by adding a heavy, reinforcing steel cage and filling the closed-ended piles with concrete.

For seismic loading conditions, some downdrag will act along the upper portion of the pile shafts due to seismically induced settlement at the site, and these anticipated vertical ground displacements will temporarily reduce the available ultimate compressive capacity of the piles. However, based on the relatively high contribution of end bearing to the total pile capacity, we do not anticipate appreciable resulting pile settlement due to the seismically induced downdrag loads.

Lateral Capacity. Lateral loads can be resisted by piles in bending and the passive resistance of the soil adjacent to the pile cap. As previously indicated, passive earth pressure against the embedded sides of foundations can be computed using an equivalent fluid with a unit weight of 250 pcf, which assumes that lateral deflections will be relatively small and in the range of about 1/4 in. or less, and that compacted granular structural fill is used for backfill against the sides of the pile cap. Any friction beneath the pile cap should be neglected. We have used the computer program L-Pile Plus by Ensoft, Inc. to model cyclic lateral loading conditions on the piles. Assuming a fixed connection between the top of a pile and the cap, we estimate a 65-kip lateral load will produce a horizontal pile top deflection of about 1/4 in. These results are applicable to the case of single, isolated piles. This assumes the piles will be provided with a minimum center-to-center spacing of eight pile diameters in the direction of loading. For a pile spacing less than eight diameters, a lateral load reduction factor should be applied. This reduction factor may be estimated by straight-line interpolation between 100% at eight diameters and 25% at three diameters, with the full capacity assumed for one pile in the group and a reduced capacity assigned to the others. As previously indicated, a minimum center-to-center spacing of three diameters should be provided to avoid potential group effects for the static axial loading of the piles.

Pile Installation. We recommend driving the steel pipe piles to practical refusal, defined as 8 to 10 blows/in. with an impact hammer having a minimum rated energy of about 100,000 ft-lbs. A description of the proposed pile driving equipment and accessories should be provided to the geotechnical engineer for review prior to mobilizing the equipment to the site. We also recommend that qualified personnel maintain a continuous record of driving resistance (blows/ft or blows/in.) for each pile at the time of installation. The driving record should be maintained for the full depth of pile penetration.

Machine Vibrations

For the structural analysis of the response of pile foundations to the applied machine vibrations, we recommend Poisson's ratio be assumed as 0.33 for the underlying soils. We also recommend assuming a total unit weight of 125 pcf and an effective shear modulus of about 12 ksi for the soil profile to a depth of about 75 ft for the pile support.

Stockpile Settlements

Settlements will occur beneath the weight of the pre- and post-process scrap stockpiles. We anticipate the stockpiles will be 40 ft wide with 1H:1V slopes, and the maximum imposed bearing pressure beneath the stockpiles would be about 3,000 psf. We assume the majority of settlement will occur during and shortly after initial placement of stockpiles to full height. Based on these considerations, we have roughly estimated about 12 in. of settlement may occur beneath the center of the stockpiles, and the settlement at the perimeter of the piles will be 3 to 6 in. We recommend surcharging designated stockpile areas to cause these settlements prior to yard paving. The surcharge fill can consist of soil rather than steel scrap. Subsequent future settlements beneath the weight of stockpiles would then be expected to be relatively elastic and mostly recoverable.

We understand the need for new utilities at the site is minimal, and the depths of installation are anticipated to be within the capping material and above the sludge. However, we recommend that the layout of utilities avoid designated heavy scrap stockpile areas, regardless of prior surcharging.

Pavements

The majority of the owner's adjacent property is presently being used for metal recycling and is underlain by a second similar sludge pond. We understand that this area was previously surfaced with 12-in.-thickness of Portland cement concrete (PCC) pavement, and has performed well under the recycling yard traffic. We also understand that other smaller areas of the adjacent property that are subject to less intense traffic were paved with 6-in. of AC over 12-in. of crushed rock base (CRB) and that these pavement sections have also performed well. We anticipate the majority of the area surrounding the new shredder will be similarly paved. In our opinion, this paving will provide a relatively impermeable surface that will supplement the existing cap over the sludge and will further minimize the infiltration of surface water at the site.

The construction of new pavements over an established granular construction working blanket could be accomplished by regrading the existing rock section to remove surficial contamination. Typically, a new 4-in.-thickness of fresh 3/4-in.-minus crushed rock is added to the section to serve as a leveling course beneath new pavement. Prior to paving, the base course should be proof rolled with a fully loaded dump truck and observed by a qualified geotechnical engineer. Soft areas identified by proof rolling should be overexcavated and backfilled with structural fill.

Seismic Considerations

General. The 2007 Oregon Structural Specialty Code is based on the 2006 International Building Code (IBC). Due to the thickness of the relatively soft sludge at the site, we recommend using IBC Site Class E to characterize the soil profile. Based on our review of the maps provided with the most recent IBC, the spectral response accelerations for the site, S_S and S_1, which correspond to periods of 0.2 and 1.0 second, are approximately 0.97 and 0.35 g, respectively. However, based on our experience in the Rivergate area and the proximity of this site to the Columbia Slough, there is some risk that seismically induced liquefaction will result in settlement and lateral spreading. We have used the results of our recent subsurface exploration to further characterize these risks.

Liquefaction. The liquefaction potential at the site has been evaluated using the soil profile disclosed in the recent borings and assuming an ordinary range of slough and groundwater levels. Using LiquefyPro, a seismically induced liquefaction and settlement analysis software developed by CivilTech Corporation, estimates were made of the extent and depth of liquefaction within the subsurface profile at the site for varying input values of peak ground surface acceleration and earthquake magnitudes consistent with a strong local event. The results of this analysis indicate that partial to complete liquefaction of saturated and looser zones within the underlying soil profile are likely for values of peak ground surface acceleration at this site equal to or less than those stipulated for design based on the IBC. We estimate the possibility of seismically induced settlements of about 1 to 3 in. at borings B-1 and B-2, close to the location proposed for the shredder, and up to about 15 in. of settlement at boring B-3 near the top of the bank of the slough.

Lateral Spreading. Since liquefaction of the underlying soils is likely during the design earthquake, there is a potential for lateral spreading along the bank of the Columbia Slough. The well-recognized methods of Bartlett and Youd can be used to estimate lateral spreading for both free-face and continuous slopes in free-field conditions. For free-face conditions, a lateral spreading on the order of several inches near boring B-3 and along the top of the slough bank is predicted by these methods. Significantly less movement will occur farther inland at borings B-1 and B-2, which are much closer to the location proposed for the shredder. It should be noted that these analytical approaches to evaluate ground displacements due to liquefaction are largely based on empirical methods and depend on a large number of variables that are difficult to quantify precisely. Consequently, any of these estimates should be considered approximate and are likely conservative.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that a representative from GRI should observe all construction operations dealing with pile installation. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to assist the owner and engineer in the design of this project. The scope is limited to the specific project and location described herein. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the new shredder system. In the event that any changes in the design and location of the modifications as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The analyses and recommendations submitted in this report are based on the data obtained from the subsurface explorations made at the locations shown on Figure 2 and from the other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations and that groundwater levels will fluctuate with time. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations may not become evident until construction. If, during construction, subsurface conditions different from those encountered in this report are observed or encountered, or appear to be present beneath or beyond foundations, we should be advised at once so that we can observe these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
8045
OREGON
JULY 22, 1974
H. STANLEY KELSAY
Exp. 6/09

H. Stanley Kelsay, PE, GE
Principal

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This document has been submitted electronically.
The original sealed document is on file in this office.

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LINNTON, OREG. (1aa) 2004

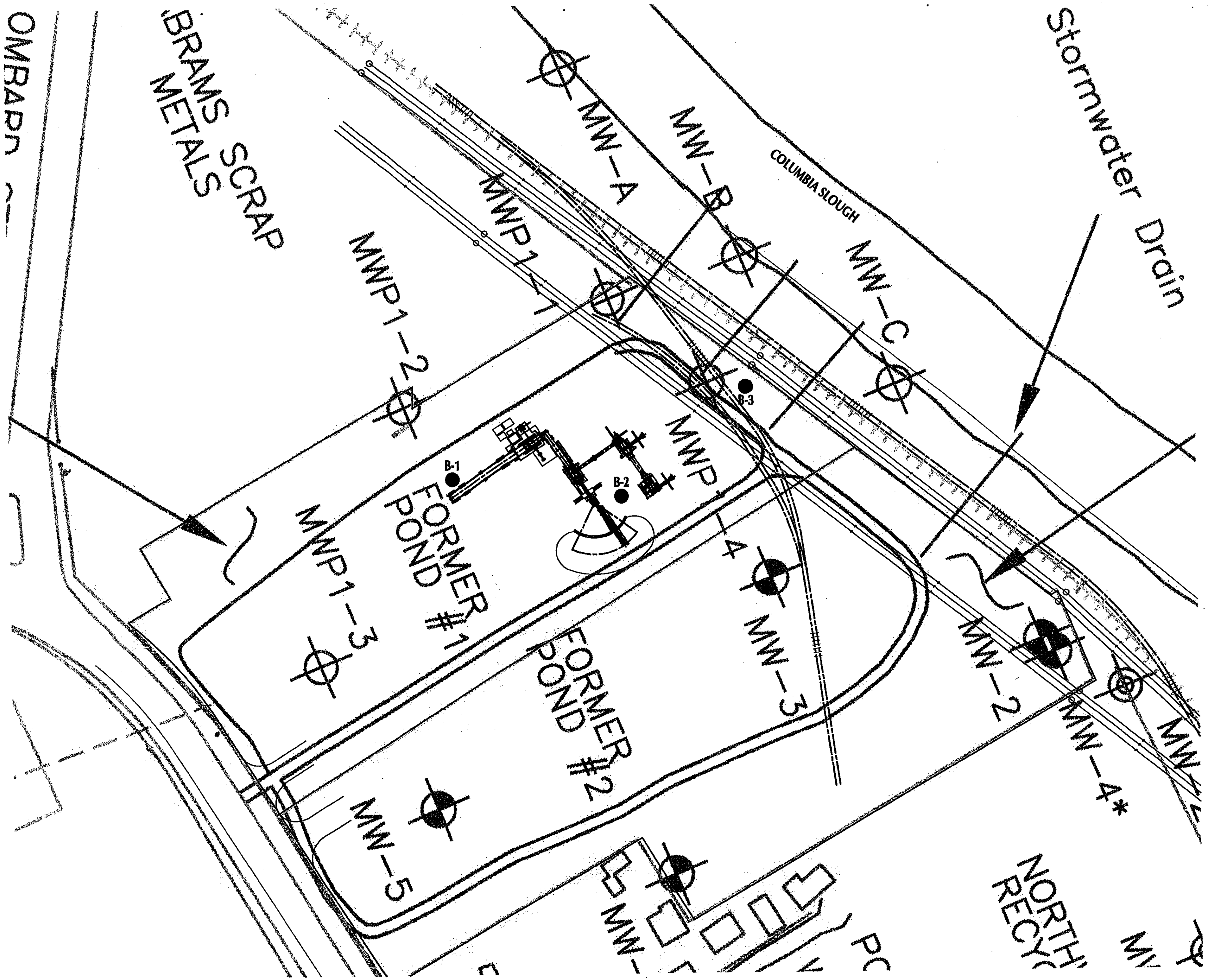
North

0 1/2 1 MILE

GRI RIVERGATE SCRAP METALS, INC.
RIVERGATE SCRAP METALS SHREDDER

VICINITY MAP

MAY 2008 JOB NO. 4741 FIG. 1



CONFIDENTIAL

● BORING MADE BY GRI
(OCTOBER 24 - NOVEMBER 1, 2007)

SITE PLAN FROM FILE BY HARRIS GROUP INC., DATED OCTOBER 4, 2007

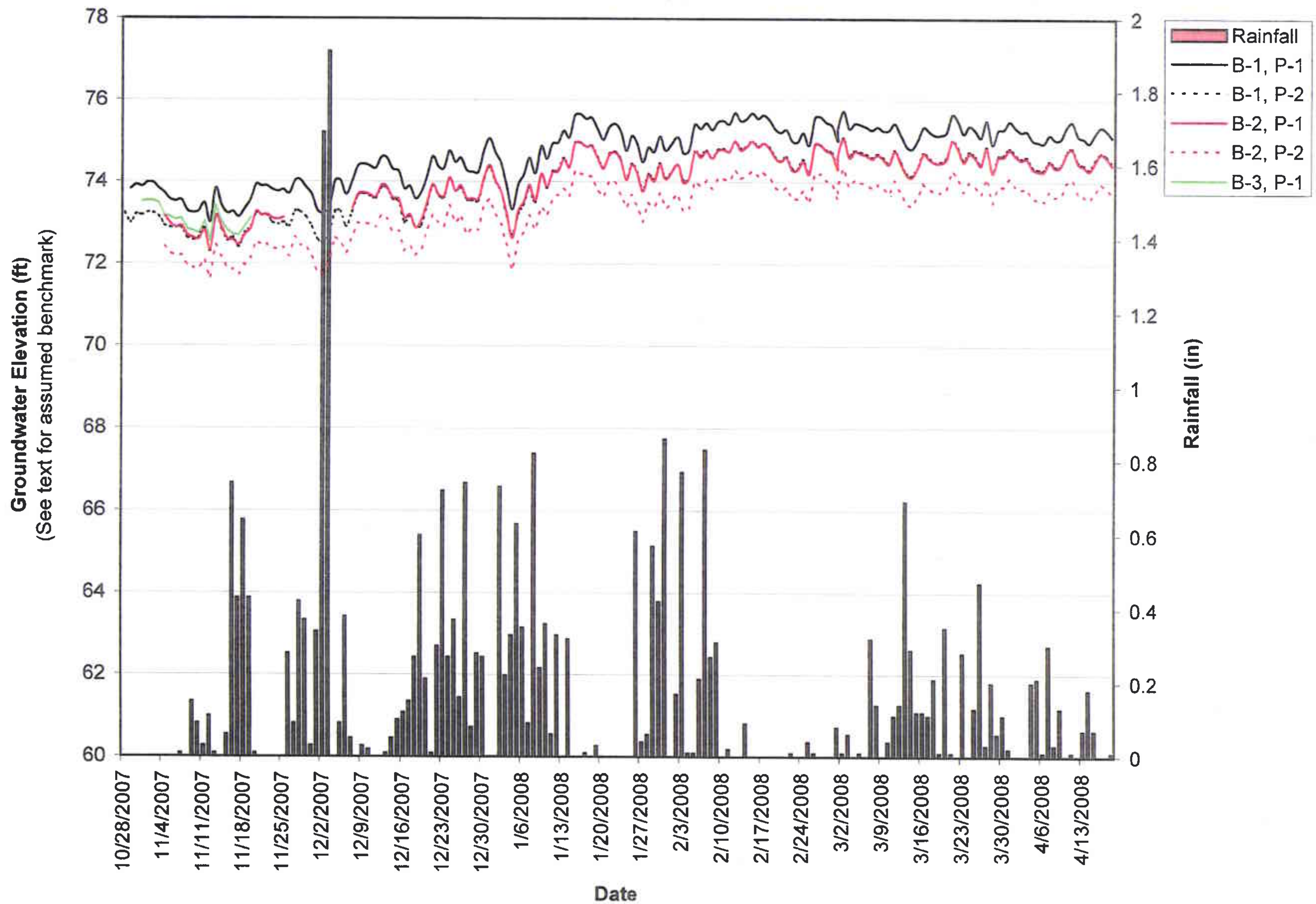
North

0 100 200 FT

GRI RIVERGATE SCRAP METALS, INC.
RIVERGATE SCRAP METALS SHREDDER

SITE PLAN

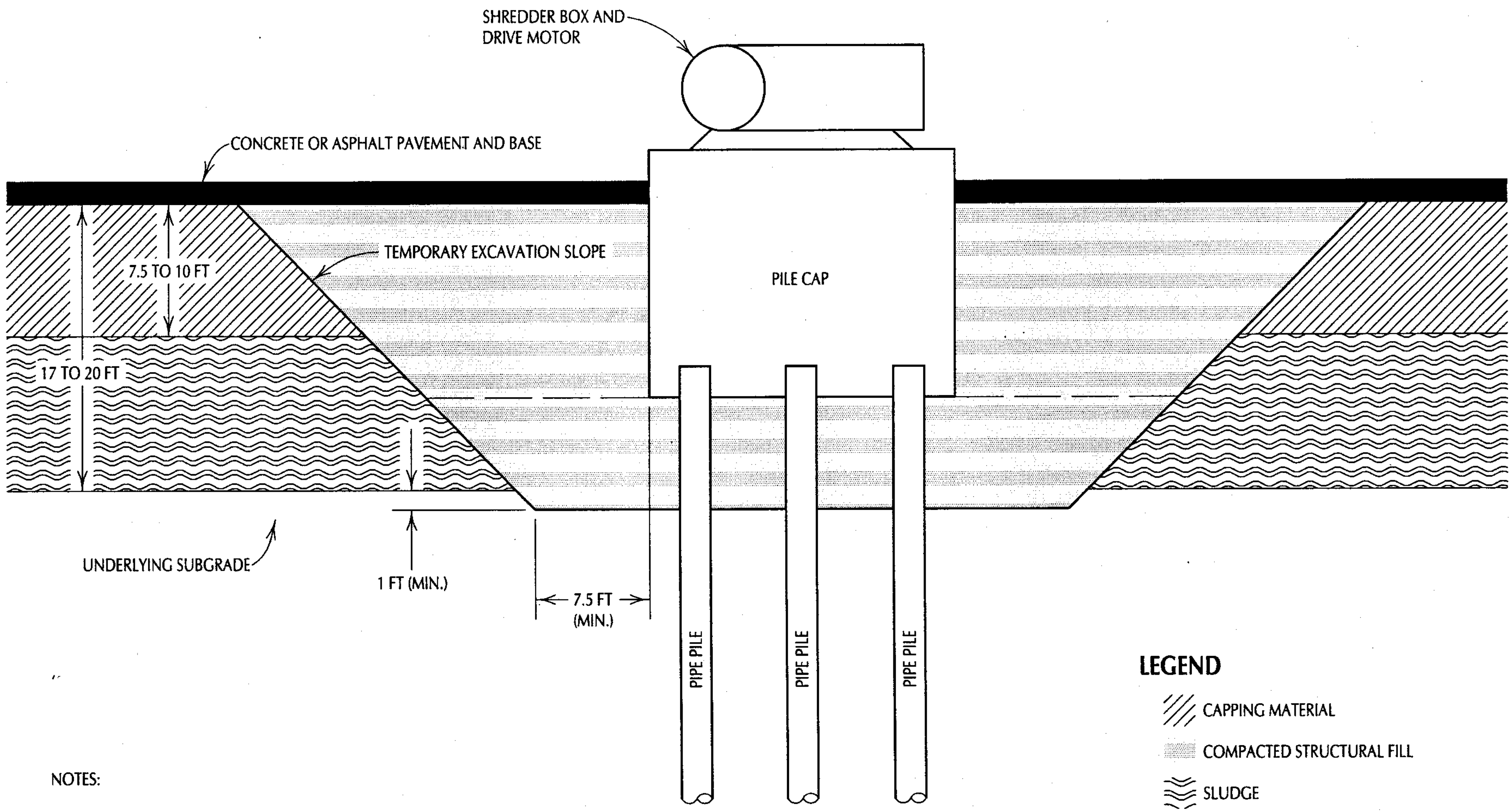
MAY 2008 | JOB NO. 4741 | FIG. 2



GRI RIVERGATE SCRAP METALS, INC.
RIVERGATE SCRAP METALS SHREDDER

GROUNDWATER LEVELS

MAY 2008 | JOB NO. 4741 | FIG. 3



NOT TO SCALE

NOTES:

1) OVEREXCAVATE TO A MINIMUM OF 1 FT BELOW THE BOTTOM OF THE SLUDGE AND 7.5 FT BEYOND THE EDGES OF THE PILE CAP.

2) COMPLETE COMPACTED STRUCTURAL FILL TO THE BOTTOM OF THE PILE CAP.

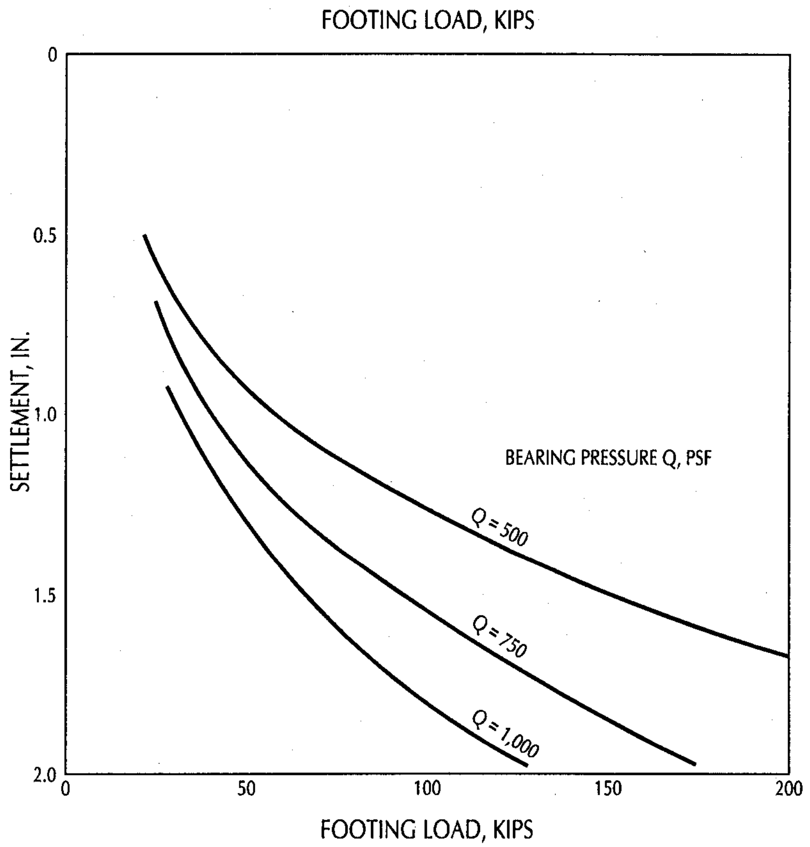
3) DRIVE PILES AND COMPLETE PILE CAP.

4) BACKFILL PERIMETER OF CAP WITH COMPACTED STRUCTURAL FILL.

GRI RIVERGATE SCRAP METALS, INC.
RIVERGATE SCRAP METALS SHREDDER

SCHEMATIC SHREDDER FOUNDATION CONCEPT

MAY 2008 JOB NO. 4741 FIG. 4



GRI RIVERGATE SCRAP METALS, INC.
RIVERGATE SCRAP METALS SHREDDER

ESTIMATED FOOTING SETTLEMENTS

MAY 2008 | JOB NO. 4741 | FIG. 5

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions at the site were investigated between October 24 and November 1, 2007, with three borings, designated B-1 through B-3. The approximate locations of the explorations are shown on the Site Plan, Figure 2.

Borings

The borings were drilled with mud-rotary techniques using a truck-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Aurora, Oregon. The borings were advanced to depths of about 82 to 101 ft. Disturbed and undisturbed samples were obtained from the boring at varying intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. All field operations were observed by a geologist from GRI, who maintained a detailed log of the materials and conditions encountered in each boring, directed the sampling operations, and collected representative samples for further examination and laboratory testing.

Relatively undisturbed 3.0-in.-O.D. Shelby tube samples of the sludge material were obtained by pushing the tubes using the hydraulic rams on the drill rig. The material exposed in the ends of the Shelby tube were examined and classified in the field. The ends of the tube were then sealed with rubber caps for further examination and laboratory testing.

Groundwater Monitoring Instrumentation

Groundwater monitoring instrumentation was installed in each of the borings to directly measure the groundwater gradient at the site and allow for future groundwater quality sampling. Two vibrating-wire piezometers were installed at different depths in borings B-1 and B-2, which were drilled through the capped sludge pond. One vibrating-wire piezometer and a groundwater monitoring well, to permit collection of water quality samples, were installed in boring B-3. Boring B-3 was drilled through the original containment dike that separated the pond from the railroad and slough. Automated data loggers, installed within monuments set in concrete at the ground surface of each boring, are used to monitor varying piezometric levels.

Soil Resistivity and pH

Two in situ electrical resistivity surveys were completed on April 18, 2008, using the Wenner electrode arrangement in substantial conformance with section 7.2.4 of IEEE Standard 81-1983 (The IEEE Guide for Measuring Earth Resistivity, Ground Impedance, and Earth Surface Potentials of a Ground System). Survey R-1 was oriented north-to-south with electrode spacing ranging from 2 to 50 ft. Survey R-2 was oriented east-to-west with similar electrode spacing. Both survey lines were centered near the main portion of the shredder. In conjunction with the resistivity surveys, a composite sample of near-surface soils from the borings was submitted to an independent testing laboratory for pH determination.

Boring Logs

Logs of the borings are provided on Figures 1A through 3A. Each log provides a descriptive summary of the various types of materials encountered in the borings and notes the depths where the materials and characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples collected during the drilling operation, and the depths at which the piezometers and the observation well were installed, are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture content values. The terms used to describe the materials encountered in the borings are defined in Table 1A.

The ground surface elevations shown on the boring logs were determined in the field by GRI using direct optical survey. The elevations are referenced to an arbitrarily assumed elevation of 100 ft for the top of the monument that protects the existing monitoring well, designated MWP1-4, at the east end of the site, which was used as a temporary benchmark.

LABORATORY TESTING

General

All samples obtained from the field were returned to our laboratory for examination and testing. The physical characteristics were noted, and the field classifications were modified where necessary. Our laboratory program included determinations of natural moisture content and Torvane shear strengths. The physical testing program is described in more detail below.

In addition to the physical testing, soil and groundwater samples were also submitted by chain-of-custody protocol to TestAmerica of Beaverton, Oregon, for chemical analysis. The test results were provided in our memorandum entitled, "Summary of Chemical Analysis, Rivergate Scrap Metals, Inc. Shredder Site, 9707 N. Columbia Boulevard, Portland, Oregon," dated January 4, 2008.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are shown on Figures 1A through 3A.

Torvane Shear Strength

The approximate undrained shear strength of the sludge or fine-grained soils retained in the Shelby tube samples was determined using the Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear strength testing are shown on Figures 1A through 3A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
Loose	4 - 10
medium dense	10 - 30
Dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials that exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

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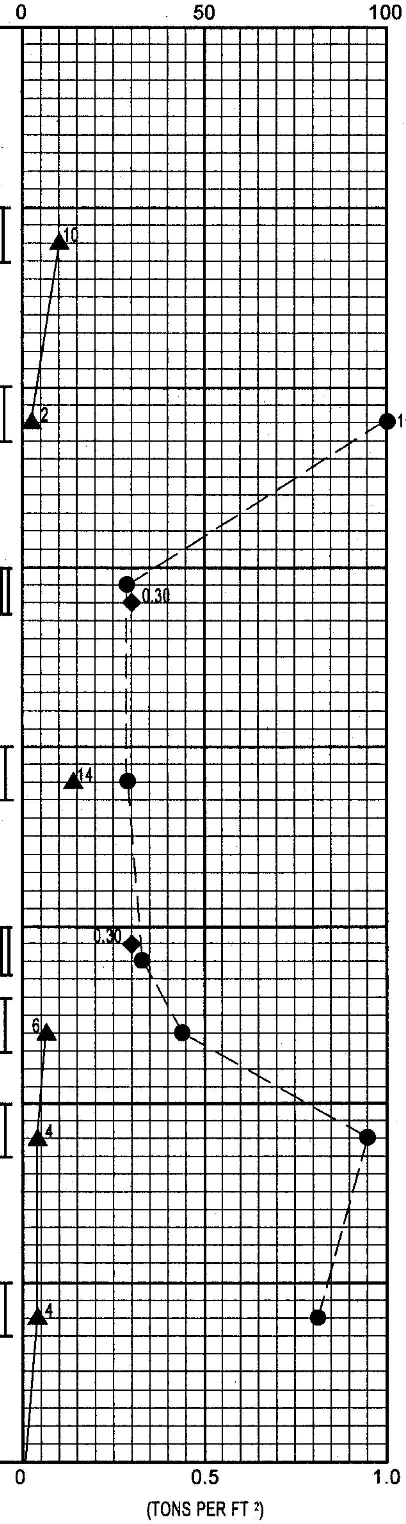
BORING B-1

SURFACE ELEVATION 101.35 ft (ASSUMED SEE TEXT)

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
0	FILL: Stiff, gray gravelly SILT			
5				S-1
7.5	SLUDGE: Soft to medium stiff black silt	7.5		
10				S-2
15				S-3
20	Medium dense, gray, sandy SILT; scattered 1-in. brown silty sand lenses	20.0		S-4
25	Medium stiff, gray SILT; some clay, trace sand	25.0		S-5
				S-6
30	—medium stiff, grayish brown sandy silt, scattered wood fragments			S-7
35	—increased wood content at 35 ft			S-8
40				

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
(P) VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
▼ Water Level (date)

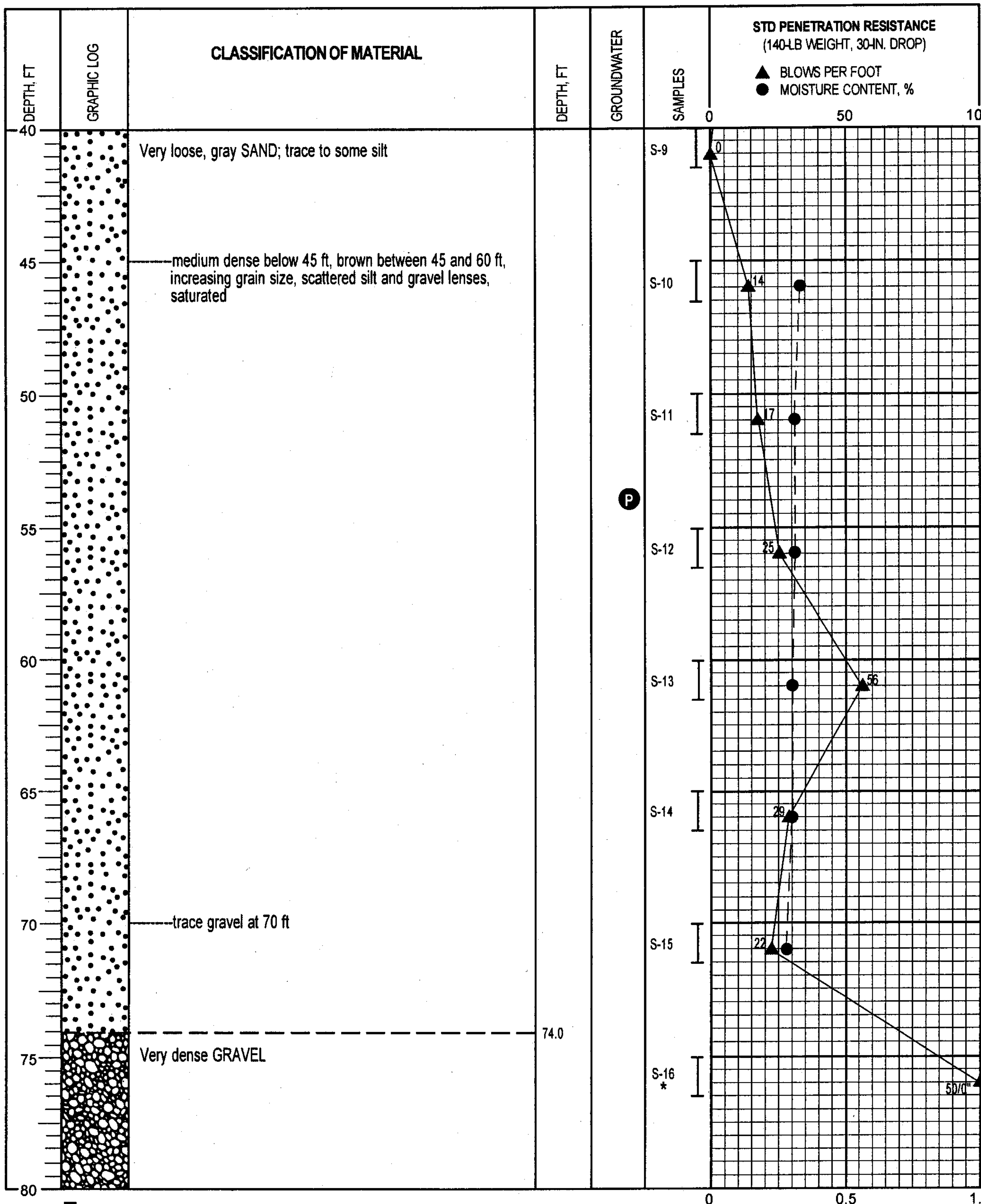
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1

MAY 2008 JOB NO. 4741 FIG. 1A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

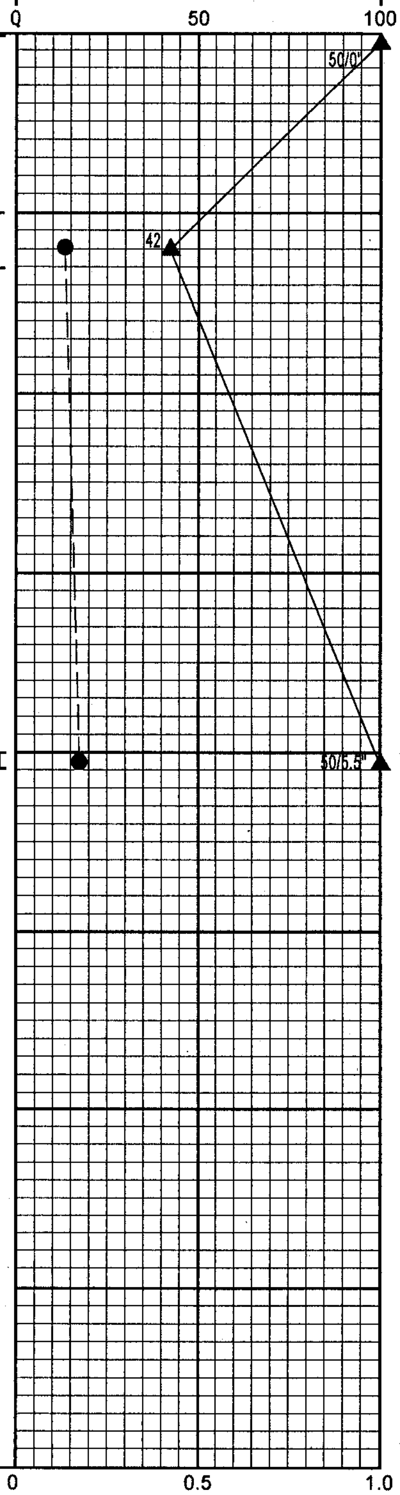
BORING B-1 (cont.)

MAY 2008 | JOB NO. 4741 | FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
80		Very dense GRAVEL			S-17
85					S-18
90				P	
95					
100		Very dense GRAVEL; fine grained in matrix of sand	100.5		S-19
		(10/24/2007)			
105					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



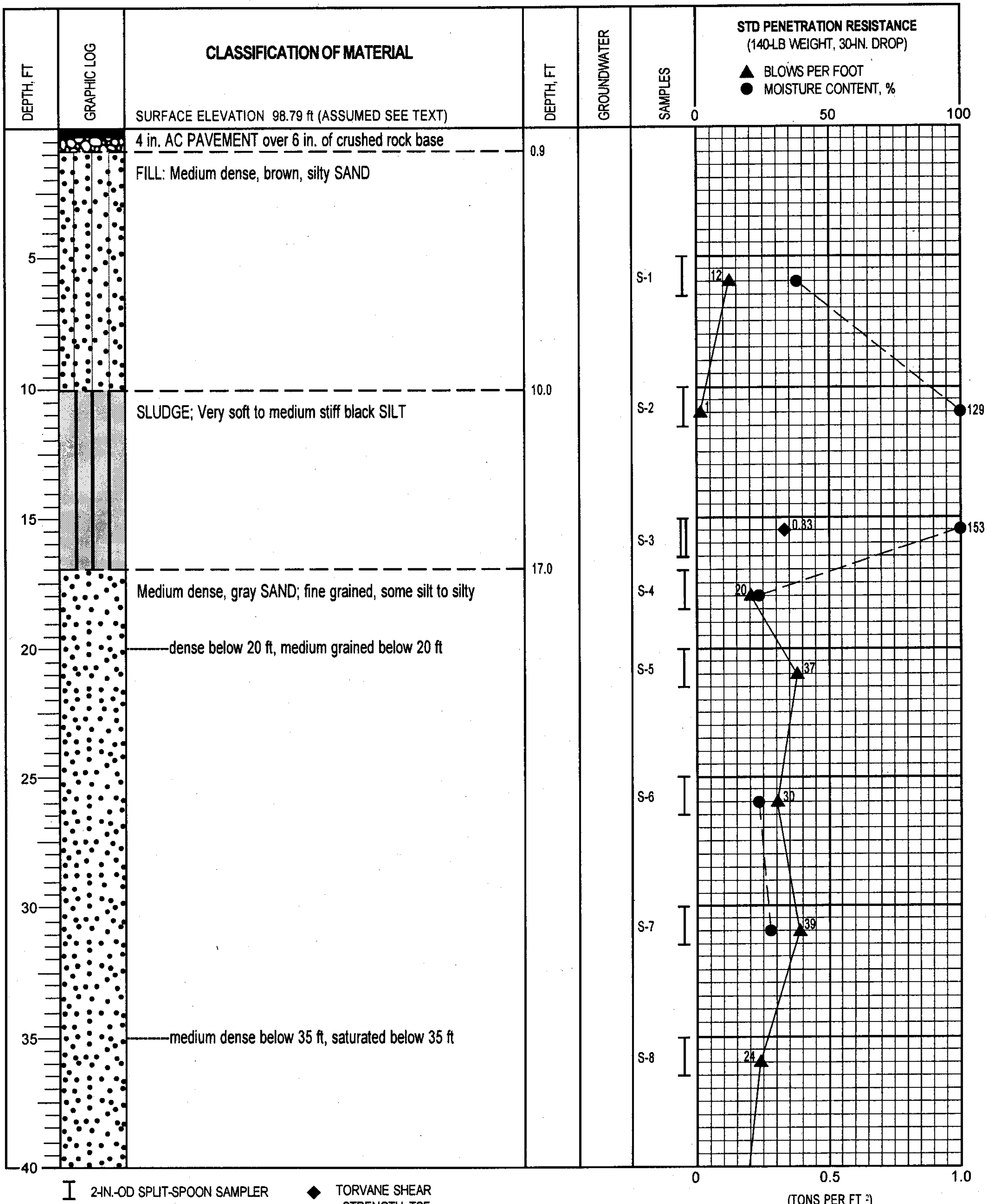
(TONS PER FT ²)

- Ⅰ 2-IN.-OD SPLIT-SPOON SAMPLER
- Ⅱ 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- P VIBRATING-WIRE PIEZOMETER
- SLOTTED PVC PIPE
- ▼ Water Level (date)
- ◆ TORVANE SHEAR STRENGTH, TSF
- ■ UNDRAINED SHEAR STRENGTH, TSF
- * NO RECOVERY
- Liquid Limit, Moisture Content, Plastic Limit

GRI

BORING B-1 (cont.)

MAY 2008 | JOB NO. 4741 | FIG. 1A



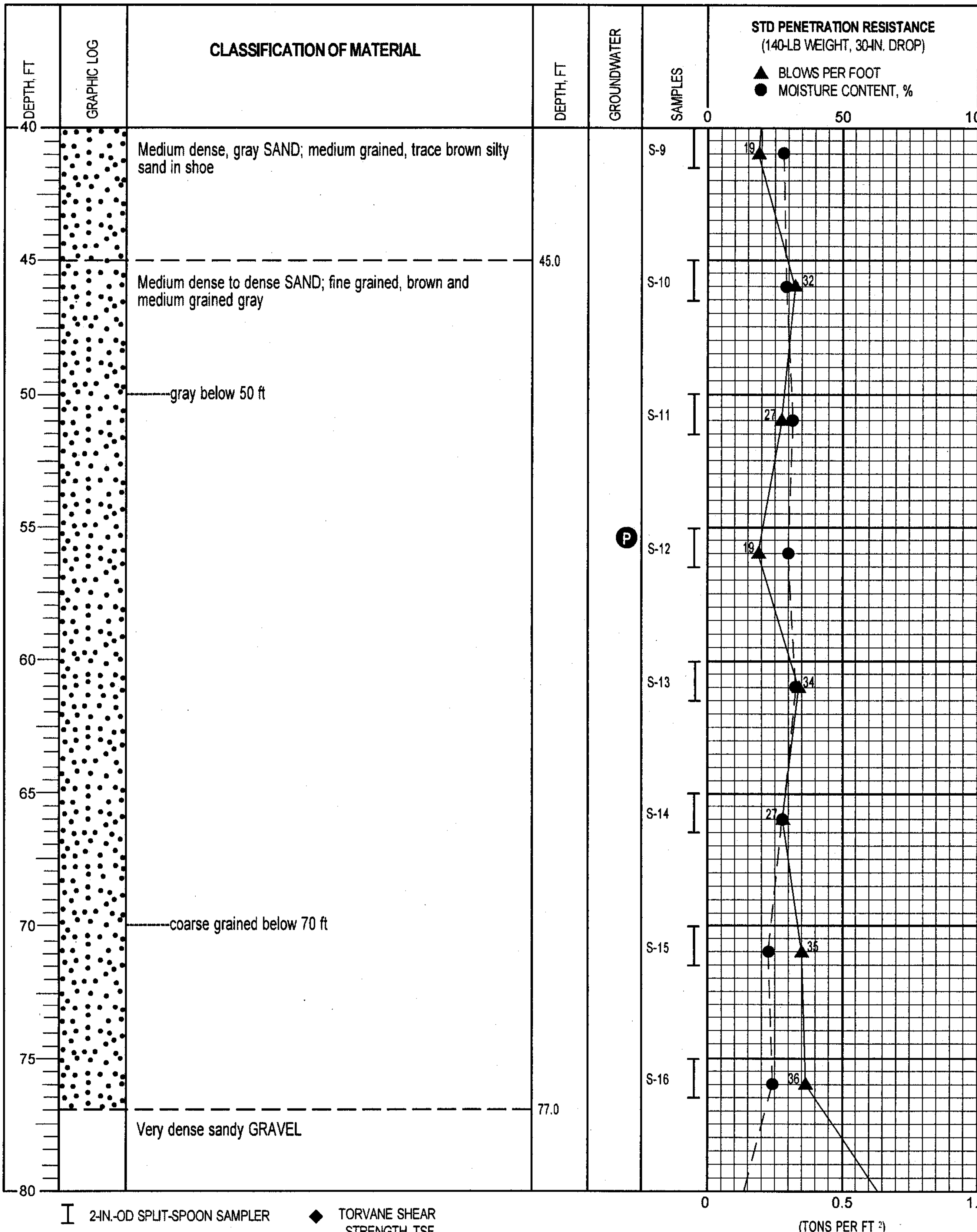
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-2

MAY 2008 JOB NO. 4741 FIG. 2A



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

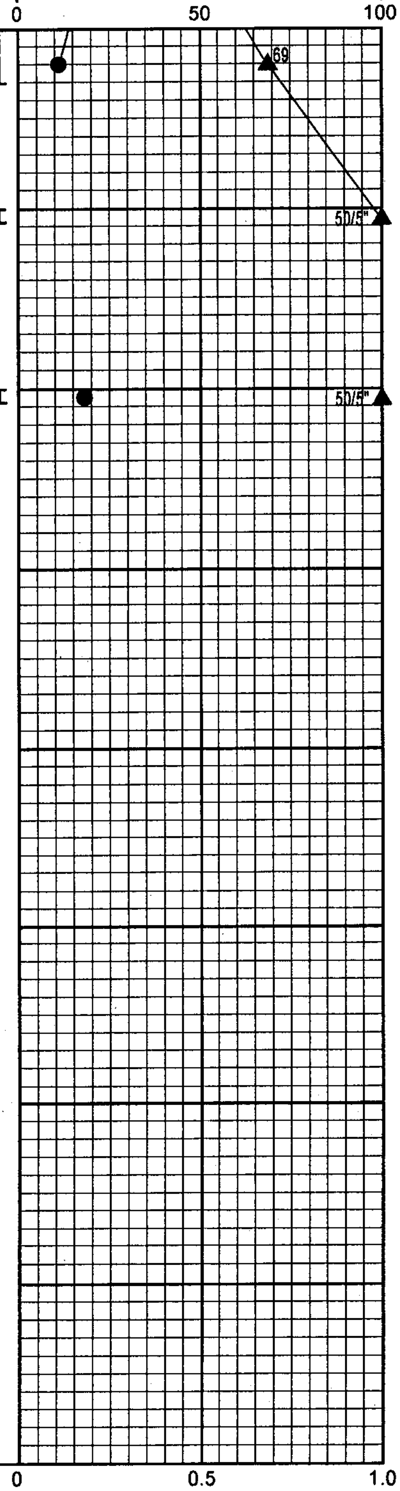
BORING B-2 (cont.)

MAY 2008 JOB NO. 4741 FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
80–90.5		Very dense sandy GRAVEL; decreasing sand content with depth		P	S-17, S-18, S-19
		(11/1/2007)	90.5		

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



(TONS PER FT ²)

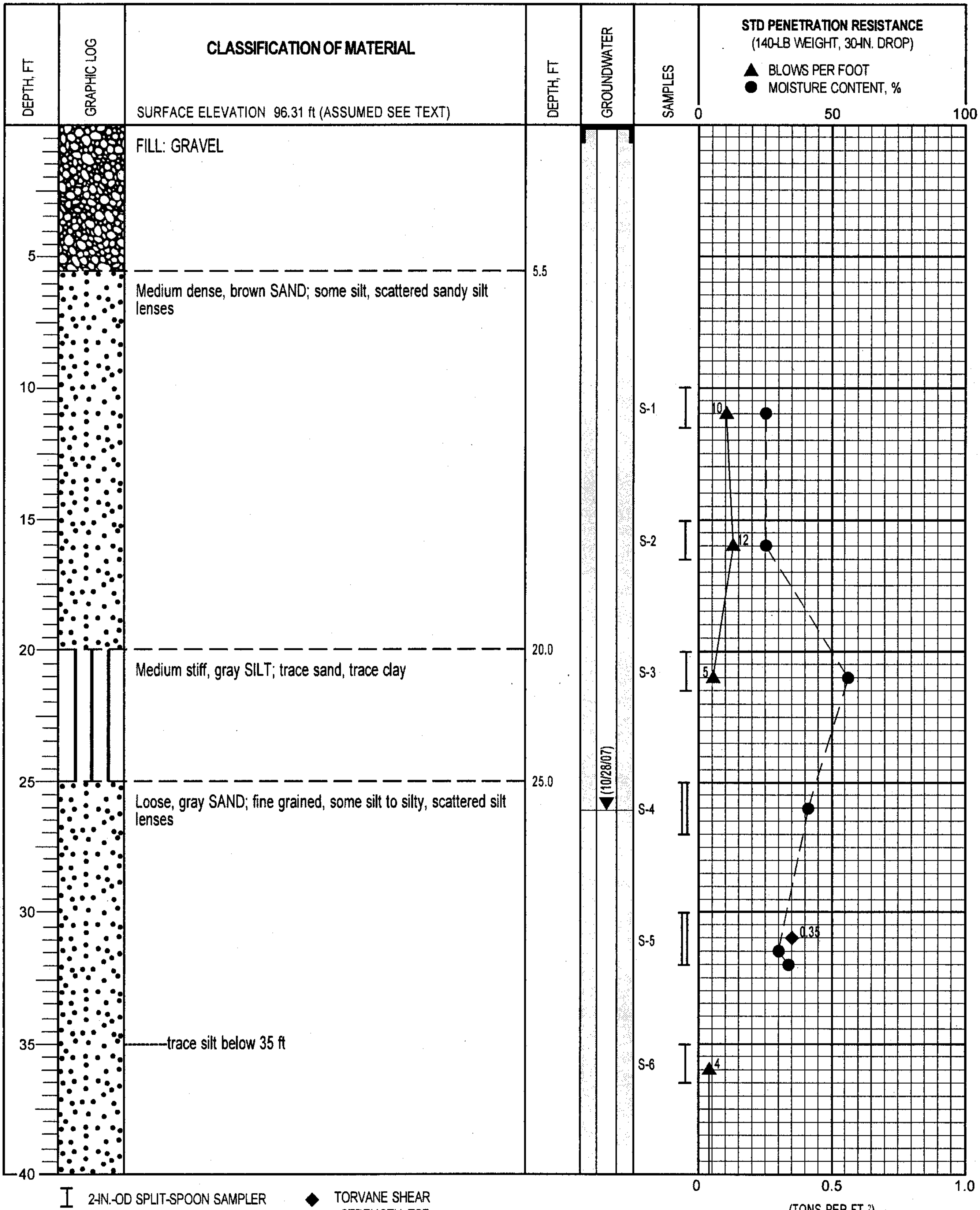
I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-2 (cont.)

MAY 2008 | JOB NO. 4741 | FIG. 2A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

P VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

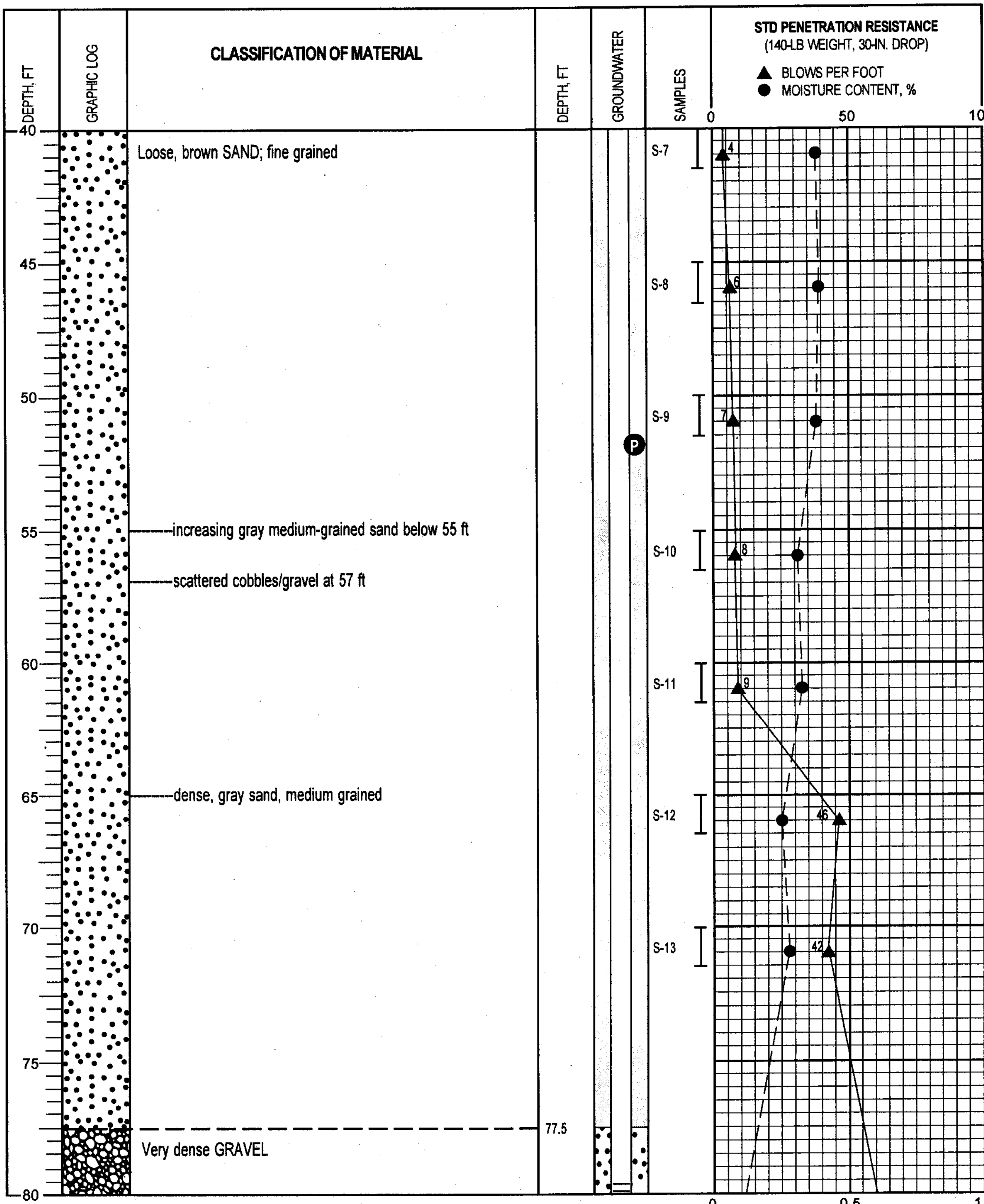
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-3

MAY 2008 | JOB NO. 4741 | FIG. 3A



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
Water Level (date)

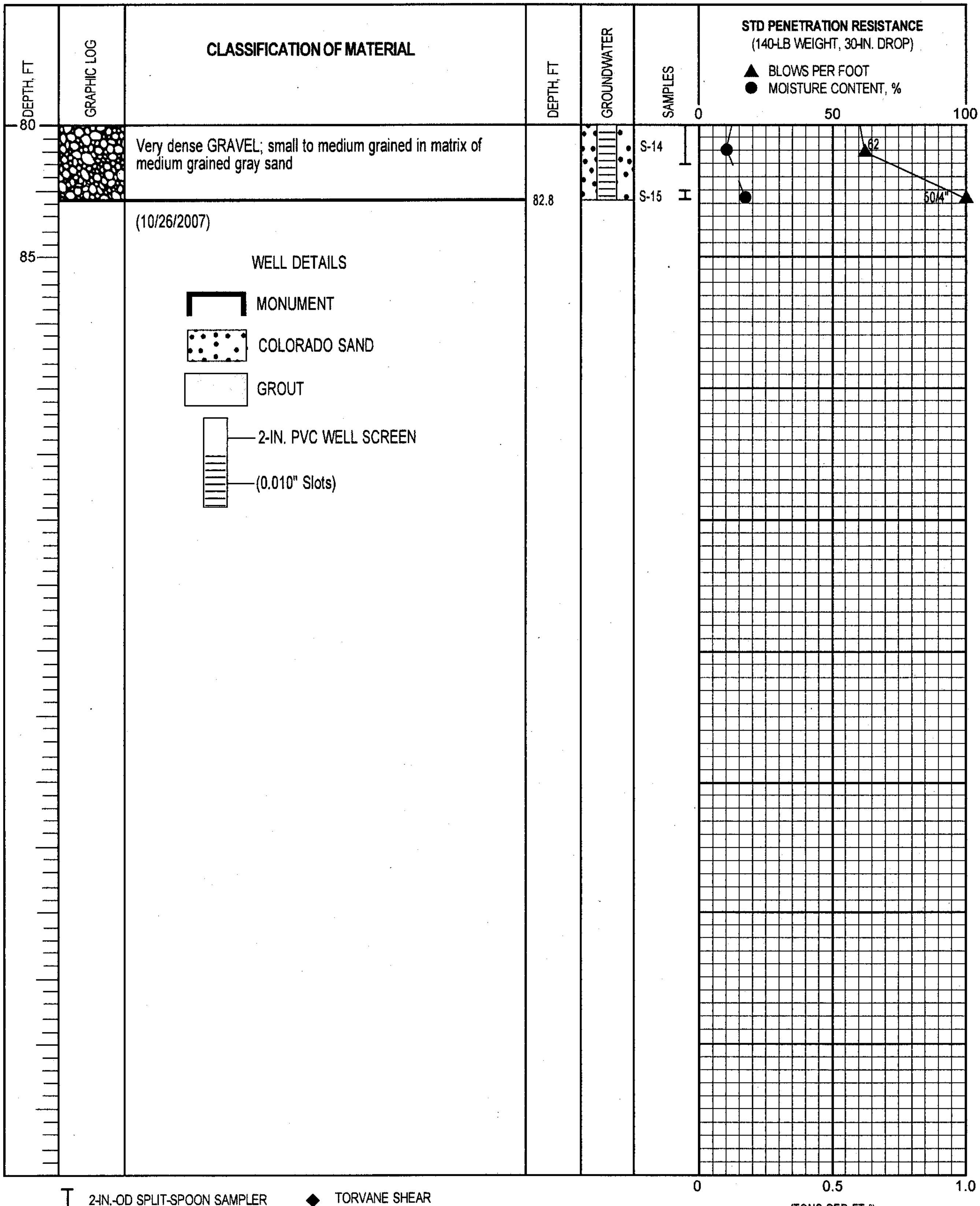
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-3 (cont.)

MAY 2008 JOB NO. 4741 FIG. 3A



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
P VIBRATING-WIRE PIEZOMETER
SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-3 (cont.)

MAY 2008 JOB NO. 4741 FIG. 3A