WCG West Coast Geotech, Inc.

GEOTECHNICAL CONSULTANTS

W-1536

March 16, 2001

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Winstead and Associates
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Attn: Mr. Stephen Winstead
Architect

GEOTECHNICAL INVESTIGATION
AIRPORT WAY WAREHOUSE
BERNARD COMMERCE CENTER (LOT 3)
PORTLAND, OREGON

Gentlemen:

In general accordance with our proposal of January 24, 2001, and your authorization of February 5, 2001, West Coast Geotech, Inc., has completed the geotechnical investigation for the proposed commercial lot which is generally located north of NE Airport Way and east of NE Mason Street in Portland. This report provides a summary of our field and laboratory programs and presents our recommendations for foundation and pavement designs and grading operations.

This report was prepared for your use in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only, i.e., test pit logs and samples, if any are taken. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the formal logs and/or discussion of subsurface conditions contained herein.

PROJECT AND SITE DESCRIPTIONS

The proposed building (which measures approximately 125 feet by 200 feet, in plan area) tentatively abuts up to the east property line setback. We understand that the building will be a tall, single story concrete tilt-up framed structure with two-story, mezzanine office floors planned at the northwest and southwest corners of the proposed building.

The west elevation of the building appears to contain a depressed loading dock, which we assume will be about 4 feet below finished floor elevation.

Based on our telephone conversation with Mr. Watson, P.E., Miller Consulting Engineers, we understand that the structure will have 25-foot tall perimeter walls. Column spacing is anticipated at an approximate spacing of 30 by 50 feet. We understand that preliminary calculations indicate typical column loads of 60 kips per column; whereas, preliminary perimeter wall loads should be on the order of 2.5 kips per lineal foot.

Concrete slab-on-grade floor will also be constructed for this project. At the present time, the finished floor elevation has not yet been determined. We understand that a typical floor loading of 250 pounds per square foot in the warehouse and 50 pounds per square foot in the office portions of the building.

The project also includes asphalt drive lanes and parking lot pavements. Based on our conversation with you on March 14, we understand that the design truck traffic is as follows:

- 2 two-axle trucks per day,
- an occasional 5-axle truck,
- a weekly 3-axle garbage truck

Later in the design process, if the structural information and design truck information are found to be significantly different from that information which is presented in the previous paragraphs, we recommend that we be allowed to review our recommendations and modify as needed.

Based on our review of a Site Topographic Plan dated February 7, 2001, as prepared by James Andrews Surveying that you provided for our use on March 13, the property slopes gently downhill in an east/southeasterly direction from an approximate high elevation of +33 feet, more or less, adjacent to the cul-de-sac to an approximate elevation of +28 feet, more or less, northwest of the southeast corner of the property at the edge of the existing fill.

From this point, the property slopes off more steeply to an approximate low elevation of +22.5 feet, more or less, at the southeast corner of the property.

The pavement elevation of the cul-de-sac (NE Mason Street) is approximate 4 to 5 feet, more or less, below the existing ground surface of the west end of the property. Along the south perimeter, the property appears to be about 5 to 6 feet higher, in elevation, more or less, than NE Airport Way.

West Coast Geotech, Inc., has also previously investigated the subsurface soils at this site which led to the preparation of a Preliminary Geotechnical Report dated October 16, 1997, for the original Developer. Since that Report, some additional grading occurred at this Commerce Center (Lots 1, 2 and 3). We were not retained to provide any testing/consulting services during this time when the grading was conducted. Hence, the extent and depth of that grading and/or regrading, if any, of the existing fills at Lot 3 are essentially unknown and subject to further investigation.

Based on our investigation for this Report, we believe that the existing fill soils on Lot 3 do not appear to have been recompacted. Hence, the grading/regrading operations that occurred since the 1997 Geotechnical Report appear to have been limited to the removal and spreading out of the large mound of fill that was originally present at Lot 2 in 1997.

A portion of this mound fill appears to have been spread out over the existing fill for the north half of Commercial Lot #3 (approximately 1-1/2 to 2 feet thick). The surficial fill is loose and saturated, thereby indicating that systematic compaction was not conducted on this recent fill material.

In addition, the southeast corner of the property (with current ground elevations of +22 and +23 feet, more or less) was not filled during the original grading nor during the 1997 regrading. Hence, the southeast corner of the property may be prone to differential settlement between that portion of the property where the weight of the existing fill has already consolidated the native soils over time and that portion at the southeast corner of the property where no existing fill is present.

The presence of existing fill (that has not yet received adequate compaction) and the effect of this fill upon foundations, slabs and pavements along with the lack of existing fill at the southeast corner of the property (as initially presented in the previous paragraph) will be addressed, in more detail, later in this report.

FIELD EXPLORATIONS AND LABORATORY TESTING

The field exploration program consisted of five test pits at the approximate general locations shown on the Test Pit Location Plan, Figure 1, which was taken, in part, from a Topographic Plan (dated 2/7/01) prepared by James Andrews Surveying that you provided for our use.

The test pits, designated TP-1 through TP-5, were generally excavated to 4 to 8.5-foot depths using a small trackhoe provided by a local contractor. A West Coast Geotech, Inc., Engineer was present throughout the exploration to observe the excavations and prepare descriptive logs of the test pits. The test pits were purposely located by our Engineer to provide subsurface information in the general vicinity of the proposed building site as based on Site Concept Plan (ie, an approximately 125 feet by 200 feet, in plan area, structure tentatively abutting up to the east property line setback).

After the test pits were logged, the backhoe operator backfilled the test pits with the excavated material. No compaction of the backfill was conducted.

Summary test pit logs are presented in Figures 2 through 4. Soil descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The soil conditions between test pits may also be different than what is shown on the test pit logs. The locations of the test pits are approximate and based on pacing measurements from nearby reference points. The elevations, as shown on the test pit logs, are based on our interpolation of the Topographic Plan (2/7/01) provided for our use.

Because of our previous investigation (Geotechnical Report dated October 17, 1997) and our current knowledge of the project, no sophisticated strength/consolidation tests were conducted for this project.

SUBSURFACE INTERPRETATION

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume the exploratory test pits are representative of the subsurface conditions throughout the site. If, during construction, subsurface conditions different from those encountered in the exploratory test pits are observed or appear to be present beneath excavations, we should be advised at once so that we may review these conditions and reconsider our recommendations where necessary.

Organic Soils. Based on our test pits, the upper 8 to 10 inches of the surface soils contain topsoil/roots/duff. The thickness of the topsoil/duff layer may vary in thickness from one area of the Commercial Lot to the other (especially, in the southeast corner of the Lot where the thickness of the surficial organic layer is anticipated to be greater than the 12 to 14 inches measured at the higher elevations of the site where the existing fill is located).

Surficial Fill. Based on our field exploration program, the property appears to contain 2 or 3 types of different fill probably placed onto the property at different times in the past. Overall, the surficial fill (which generally varied from 1.5 feet thick, more or less, at the middle of the proposed building site to 4 to 6 feet thick, more or less, at the periphery of the building site) was observed to be evident at all of the test pits.

The upper 1.5 to 2 feet of the uncompacted fill appears to consist of very loose to loose, wet, brown, slightly sandy to sandy silt fill containing varying quantities of embedded gravels/cobbles that vary from "nil" to scattered. We believe that this fill originated from the spreading of the mound of fill (present on Lot 2) that was conducted in 1997. No compaction appears to have been conducted.

The existing fill material beneath this fill layer varies from one area of the building site to the other. As evidenced by Test Pit TP-1 and TP-2 (in the north portion of the building site), the fill appears to consist of 2-1/2 feet of loose to dense sand/gravel to gravel with varying quantity of embedded cobbles and occasional small boulders to an approximate depth of 4 feet. At Test Pit TP-2, the gravel fill layer overlies a 2-foot thick layer of soft to medium stiff, moist, gray clayey silt fill to an approximate depth of 6 feet beneath current ground surface.

At the south end of the building site, as evidenced by Test Pits TP-3 and TP-4, the underyling fill layer varies from a loose brown, gravelly sand/silt with scattered cobbles/small boulders, in the west, down to an approximate depth of 6 feet; while, in the east, the fill consists more of a loose, dark brown to gray sand/silt with little, if any, gravel and scattered cobbles/small boulders to an approximate depth of 6 feet. As evidenced by Test Pit TP-4, the fill at the southeast corner of the building site appears to contain some wood debris.

The existing fill (prior to the 1997 regrading) were generally tested for compaction in our October, 1997, Report. In general, the compaction levels within the existing fill (beneath the recent 1997 regrading event that created the upper 1.5 to 2 feet of new, uncompacted fill) dropped off significantly with depth; with the highest levels occurring within the depths of 2 to 3 feet below current ground surface (99% to 100+%) and the lowest levels at depths of 4 to 6 feet below current ground surface (80% to 94%).

We also bring to your attention the fact that the presence of rock and cobbles mixed in with the silt fill soils in the upper 3 feet may also cause a false perception of the compaction levels because the rocks and cobbles are removed from the soil when conducting the laboratory compaction series testing. Actual compaction levels are probably less than what is reported (99% to 100+%), and maybe more like on the order of 90 to 93% depending upon how high the maximum dry density is which is influenced by the quantity of rock/cobbles contained therein.

Hence, in our professional opinion, the existing fills should be classified as non-engineered fills that are incapable of satisfactorily supporting footings. In general, the existing fill from 0 to 2 feet is very loose and wet; from 2 to 4 feet (except at the southeast corner of the property), the fill appears to more granular and slightly more dense; while, from 4 to 6 feet, the fill is generally more fine-grained and less dense, even soft.

At the southeast corner of the property, at the edge of the fill, in plan area; the fill is, generally, loose throughout its depth (as we would anticipate for the edge of a fill area where trucks end-dump their loads and dozers push a thick lift over the edge). The edge of the fill is a place where trucks seldom travel; hence, compaction beneath the truck tires usually does not occur as much at the edge.

Native Inorganic Soils. Based on our explorations, the native inorganic soil beneath the existing fill generally consists of a loose to medium dense brown sandy silt to depths that typically vary from 8 to 12 feet, or more, beneath current ground surface.

Sandy gravels and cemented gravels were generally observed to be present beneath the silt/sandy silt layer except at the southeast corner of the property where the sandy gravels and cemented gravels are deeper than what can be reached by the trackhoe used in our study.

Based on our observations of the test pits now and in 1997, we generally believe that all three commercial lots (prior to the placement of the existing fill) probably had a rolling ground surface from one "hilly knob" to the next with shallow swales in between. The existing fill probably served to level the commercial lots and more of the fill was placed in the low areas between the "hill-tops"/"mounds".

However, the gray silt fill layers that are present within some of the existing fills at the southeast corner of the property are believed to have been imported and end-dumped from trucks to supplement the volume of fill.

Boulders. Boulders were evident in the existing fill and should be anticipated to be embedded in other areas of the site as well (within the fill and, possibly, within the native soils). In addition, a few 4 to 6-foot diameter boulders were present on the surface of the property in 1997 and assumed to have been removed from the site after the 1997 filling operations were substantially completed.

Groundwater. Groundwater seepage was observed to be present at an approximate elevation of +11 feet, more or less, on September 10, 1997, at the southeast corner of the property. Perched water conditions above the cemented gravels are possible at elevations higher than +11 feet. We anticipate that the groundwater will fluctuate with time and should be anticipated to be at the highest level in late winter or spring and at the lowest level in late summer or fall when rainfall is less frequent.

GEOTECHNICAL DESIGN RECOMMENDATIONS

General

Existing fill (with typical thickness of 4 to 6 feet thick, more or less, at the periphery of the building site and about 1 to 2 feet thick, more or less, in the central area of the building site) is present throughout the site and is not suitable for the support of footings.

The existing fill may be able to be left in-place beneath reinforced concrete floor slabs depending upon the design floor elevation and the level of risk the Owner is willing to assume. The upper two feet of the fill (the recent 1997 fill especially in the north, central and west portions of the building site) is very loose and would need to be completely moisture-conditioned and recompacted and tested to satisfactorily levels. Or, the finished floor elevation could be determined in such a way that the upper 1-1/2 to 2 feet of the loose fill, or any portion thereof, is removed.

The existing fill that is present at depths of 2 to 4 feet appears to be more granular and more dense; however, the fill from 4 to 6 feet is generally fine-grained and more soft. Thus, for the purpose of support for the slab, we advise that the floor elevation be determined to keep as much of the better granular fill (that is currently present, 2 to 4 feet, more or less, beneath ground surface) in place as possible. The underlying fine-grained fill (that is currently present from 4 to 6 feet, more or less) would not provide as much support for the slab as the overlying fill layer. This underlying fine-grained fill is also more moisture sensitive to construction traffic than the overlying granular fill that is currently present at an approximate depth of 2 to 4 feet beneath current ground surface.

Additional discussion about the fill and level of risk beneath slab areas will be presented later in this report.

The end-dumped fill that was dozed over the edge of the fill area at the southeast corner of the property (as evidenced by Test Pit TP-3) is not sufficient for the support of concrete slabs. In addition, more fill will be required to be satisfactorily placed, compacted and tested to raise the site grade at the southeast corner. "Stepped down footing construction" (to generally follow the existing grades) is not a viable alternative for this situation because concrete tilt-up panels usually need to be all the same height to be economical.

Thus, the southeast corner of the existing fill pad will need to be completely regraded and recompacted beginning with the first lift upon the native subgrade. Any, uncovered dump truck loads of fill that appear to be organic or unsuitable because of debris should be completely removed from the site and replaced.

Just how far into the existing fill pad will the regrading occur may need to be determined in the field during regrading. From our observations of the ground and our review of the Topographic Plan, we suspect that the +28 ground contour elevation, as shown on the

Topographic Plan, may represent the boundary of the recommended regrading for budgeting purposes only.

The regrading of the existing fill pad at the southeast corner of the property can occur at the same time that the low elevations at the southeast corner are being raised. Recommendations for engineered fill will be presented later in this Report.

However, because the underlying soils at the southeast corner will be experiencing the weight of new fill, the underlying soils are likely to experience more settlement than those areas that have already "felt" the weight of the fill placed years ago. Hence, the southeast corner of the proposed building could experience some differential settlements that could lead to structural distress.

To reduce differential settlements, we recommend that surcharging the southeast corner of the property be considered. Surcharging is often used as a typical remedial measure in similar fill situations for projects on the northeast side of Portland. Surcharging will be discussed in more detail later in this Report.

Foundation Design

Footings. Based on our understanding of the project and the results of the field exploration program, it is our opinion that the proposed structure can be supported by spread footings located on at least 2 feet of engineered fill, minimum, that is satisfactorily placed, compacted and tested in lifts upon approved native, inorganic, brown sand/silt soil subgrade after all of the topsoil and existing fill has been completely removed from footing locations.

The purpose of the minimum 2-foot thick layer of engineered fill beneath all of the footings is to replace a portion of the loose native sand/silt that is compressible with an engineered fill that is suitably compacted so total settlement, at the footings, can be reduced to an acceptable level given the allowable bearing pressure.

Please bear in mind that the 2-foot thick layer of engineered fill is a minimum requirement. All of the existing fill will still need to be removed from each footing. Hence, depending upon the footing grade, the thickness of the engineered fill for many of the footings may likely be greater than 2 feet thick.

For footings constructed upon engineered fill as described in the previous paragraphs, we recommend an allowable bearing pressure of 2,500 pounds per square foot when incorporating the structural loads (including all Code dead and live loads) into the footing design.

When sizing footings for seismic considerations, the allowable bearing pressure may be increased by 30 percent. Based on our review of the 1998 Uniform Building Code, the building site is currently in Zone 3. The Site Coefficient should be assumed to be S_D.

Continuous wall footings should have a minimum width of 18 inches, and column footings should have a minimum width of 24 inches. All perimeter footings should be founded at least 24 inches below the lowest adjacent grade which should be taken as the finished floor elevation or exterior grade, whichever is lower. Interior footings may be founded at least 12 inches below finished floor grade.

Each footing excavation should be evaluated by a the Geotechnical Engineer to confirm suitable bearing conditions and to determine that all topsoil, loose materials, organics and unsuitable soil or fill have been removed. If such unsuitable materials are encountered at footing locations, we recommend that the unsuitable material be removed. If you desire to raise the footing grade after excavation, the engineered fill should be placed according to our recommendations shown by Figure 5.

Based on our knowledge of the project and our experience in the area, total footing settlement is estimated to be, approximately, 1 inch provided all the existing fill/surficial topsoil, if any, have been satisfactorily removed from the footing and at least 2 feet of engineered fill is satisfactorily placed, compacted and tested lifts upon an approved native sand/silt soil subgrade. Our settlement estimate assumes that no disturbance to the foundation soils will be permitted during excavation and construction. To minimize the potential for disturbance, it is recommended that excavations be made with a smooth bucket (no teeth) backhoe. As an alternative, the bottoms of the footing excavations (into the native sand/silt soil subgrades) may also be compacted to dry densities of at least 95 percent of the standard Proctor maximum dry density (ASTM D698).

Our footing settlement estimate also assumes that the site filling operations are substantially completed and/or successfully surcharged prior to footing construction so the weight of the new fill has already caused a significant portion of the settlement (due to the weight of the fill) to occur within the building site.

Floor Slabs. All floor slabs-on-grade should be reinforced and founded on a minimum 6-inch layer of free-draining, well-graded sand and gravel or crushed rock with a maximum particle size of 1-1/2 inches and containing not more than 5 percent passing the No. 200 sieve (based on a wet sieve analysis). All underslab granular materials should be compacted to a dry density of at least 98 percent of the standard Proctor maximum dry density (ASTM D698) or as approved by the Geotechnical Engineer. A moisture-vapor barrier is also recommended as additional protection beneath the slab.

Concrete slabs should be designed assuming an effective modulus of subgrade reaction, k, of 75 pounds per square inch per inch for fine-grained soils typical to the site. This recommendation also assumes that a 6-inch layer of compacted aggregate base, minimum, be placed beneath the concrete slab.

We recommend that you consider concrete slabs for trash bin areas, if any, or any truck parking area in the depressed loading dock where heavy and/or loaded trucks may park overnight or for long periods of time.

These recommendations also assume that the subgrade (after excavating to required elevations) has passed a proof-roll test, observed by the Geotechnical Engineer, and has been adequately compacted to a dry density of at least 95 percent of the standard Proctor maximum dry density (ASTM D698) for a 12-inch depth (which does not limit or act contrary to any other recommendation(s) concerning the existing fill, as provided in this Report) prior to the placement of the aggregate base.

We did not observe the placement of any of the existing fill onto the property. Based on our tests and our test pits, the existing fill should not be classified as engineered fill. Hence, there is a risk to the slab if the existing fill is allowed to remain in-place at slab areas. Previous in this Report, we discuss the loose nature of the upper two feet of the recent fill and how this layer should be re-compacted or the floor slab elevation should be chosen such that this unsuitable fill layer is removed.

We have also presented a case where the fill between the depths of 2 and 4 feet generally appears to be more granular and more dense (but, not adequately compacted to levels normally required of engineered fill within building areas), and how this fill layer should be allowed to remain in place as much as possible, in order to provide a base of support to bridge the more soft existing fill layer that underlies at an approximate depth of 4 to 6 feet.

In addition, we have discussed the situation of the existing fill at the southeast corner and how this area of fill (possibly, end-dumped fill) should be replaced and/or repaired as part of the process of finishing off the building pad at the southeast corner. Surcharging the new fill area was also introduced.

We also assume, based on our judgment from similar projects, that the existing fill has had adequate time to significantly consolidate the underlying native soils as well as that layer of fill that appears to be present between the depths of 4 to 6 feet. And, the finished floor elevation will be chosen to minimize the presence of any new substantial fill overall the building site.

All of these recommendations presume that the Owner is willing to accept some level of risk to the performance of the slab, if the existing fill is allowed to remain in place at slab areas. If the Owner decides that his tolerance of risk is "zero tolerance", then the existing fill should either be completely replaced with engineered fill that is suitably placed, If the grading plan calls for significant cut excavations at the building or, even for pavement areas, we recommend that we be allowed to consider whether special drainage trenches should be required.

Retaining Walls. Small depressed-dock retaining walls and any other cantilever retaining walls, if any, should be designed to resist lateral pressures. The lateral pressure will depend on the ability of the walls to yield. Small retaining walls should be designed using an equivalent fluid weighing 35 pounds per cubic foot for the active condition. Non-yielding walls should be designed using an equivalent fluid weighing 45 pounds per cubic foot. Where parking lot pavements on the uphill side of the walls are within an imaginary slope of 1H:1V from the bottom of the wall or elevated concrete floor slab subjected to loaded forklifts loads, then we recommend that an additional surcharge of 2 to 3 feet be assumed in the wall's design for traffic/forklift loading.

These recommendations assume that the walls are designed according to Figure 6. These values also assume that the wall is properly drained to prevent the buildup of hydrostatic pressures and a level backfill. Higher lateral pressures may be anticipated for up-slope backfill.

Sliding, overturning and maximum toe pressure should be checked for the walls. The lateral pressure may be resisted, in part, from the passive resistance of the soil in front of the wall footing. Ultimate passive resistance may be computed on the basis of 275 pounds per cubic foot equivalent fluid provided that the backfill in front of the footing is thoroughly compacted for the full depth of the wall footing's embedment below lowest adjacent grade which should be taken as the top of the pavement surface, and all of the existing topsoil/fill/buried topsoil, if any, has been completely removed and replaced with engineered fill from within 3 feet linear distance from the outside edge of a wall footing.

Additional ultimate resistance to lateral earth pressure may be obtained from sliding resistance of the base of the wall footing. We recommend a friction factor of 0.3 for fine-grained subgrades up to a maximum sliding resistance of 1,500 pounds per square foot to determine the sliding resistance at the base. The friction factor can be increased to 0.45

for foundations formed on at least 8 inches thick layer of crushed rock, engineered fill subgrade up to a maximum sliding resistance of 2,500 pounds per square foot. The minimum factor of safety to resist sliding and overturning should be 1.5, in our opinion.

Drainage is considered necessary to protect against saturation of the backfill due to leakage from broken water or sewer lines or shallow subsurface seepage. Recommendations concerning backfill and drainage requirements for small cantilever retaining/basement walls are also shown on Figure 6. The perimeter drain lines should be adequately sloped to allow the water to drain under gravity. Failure to adequately dispose of the water behind a wall could lead to significantly higher lateral pressures than anticipated.

Additional lateral pressure on walls can be caused from nearby footings or heavy floor loads. We recommend that we be allowed to evaluate this additional lateral pressure for these situations or any new below-grade walls especially if the closest edge of the footing to the bottom of the wall is on a slope steeper than 1H (Horizontal):1V (Vertical).

Pavement Design

Design Assumptions. A study was conducted for the pavement section for the main access drive and the parking lot. Our pavement design recommendations are based on the following design assumptions:

- Design truck traffic, based on our telephone conversation with you on March 14, 2001, will consist of 2 two-axle trucks per day, 1 five-axle truck per week and 1 three-axle truck per week,

- Asphalt pavements with a traffic design period of 20 years with no growth factor,

- Traffic in the new parking lot stalls to consist of automobiles and light pickups except in designated truck lane areas,

- o No annual traffic growth rate,

- o The top 12 inches of the existing subgrade is compacted to a minimum of 95 percent of the Standard Proctor maximum dry density (ASTM D698) and has sustained a proof-roll test, as well,

- o Asphalt pavement design is based on the 1986 AASHTO Design Method which is currently in use by the Oregon Department of Transportation,

- o An assumed resilient modulus of 3,000 psi,

If these assumptions are substantially incorrect, new pavement thicknesses should be determined.

Pavement Section Recommendations. Based on our design analysis and the assumptions outlined above, the recommended asphalt pavement section is presented in Table 1 as shown below:

Table 1

Recommended Asphalt Pavement Section
Truck Designated Lane

	Main Access Drive Lanes
Asphalt Layer (Type C)	3.5"
Leveling Course 3/4-inch minus	2"
Aggregate Base Course 1-1/2 inch minus	8"

For automobile parking lot stalls (where trucks do not traverse), the recommended pavement section is 2-1/2 inches of asphalt concrete over 2 inches of aggregate base (3/4-inch minus, leveling course) over 6 inches of aggregate base (1-1/2 inch minus). Where an occasional garbage truck traverses the automobile parking lot, we recommend that the asphalt thickness be increased to 3 inches.

Pavement Materials. We recommend that class C asphaltic concrete be used for the upper lift for a better appearance (for those areas traversed by trucks. Pavement sections should conform to the Standard Specifications for Highway Construction (Oregon State Department of Transportation - 1991).

The aggregate base material should consist of a clean, well-graded crushed rock conforming to the Oregon Department of Transportation Standard Specifications (Section 02630) except that not more than 5 percent should pass the No. 200 sieve (based on a wet sieve analysis). The base material should be graded from 1-1/2"-0" except for the top 2 inches which should be a leveling course graded from 3/4"-0". The CBR (California Bearing Ratio) value of the material should not be less than 50, and preferably greater, and have a sand equivalent not less than 30. The base material should be compacted to a dry density of at least 100 percent of the standard Proctor maximum dry density (ASTM D698).

Grading Operations

Subgrade Preparation. The subgrade preparation should include the stripping and removal of all surficial organic soil (sod, topsoil, duff) and unsuitable fill and pavement debris, if any is found, from the building and new pavement areas as determined by a qualified representative of the Client (preferably, the Geotechnical Engineer). After excavation to reasonably level, required subgrade elevation, the building and pavement areas should be proof-rolled with a loaded dump truck or similar vehicle in the presence of a qualified representative of the Client. Any soft or disturbed areas that are detected by the proof-rolling should be removed and backfilled with engineered fill. The actual amount of material to be excavated may need to be determined in the field, and we recommend that the specifications, if any are written, include a unit cost bid item for any over excavation beyond that excavation normally required by the Contract.

Construction operations may need to be modified to minimize site disturbance especially during wet weather conditions when soil moistures are above optimum moisture content such that pumping or rutting of the subgrade is observed by the Client's representative. Any disturbed soil shall either be compacted to acceptable standards or removed and replaced with engineered fill. Due to the nature of the underlying soils, we recommend that the site work be conducted during the normal summer/fall construction season when subgrade and fill moisture contents are typically at their lowest and extended periods of dry, warm weather are usually common.

The Contractor should be made responsible for providing construction access roads and staging areas. Based on our experience, we provide the following information about workpads for budgeting purposes only, if winter or wet season construction is planned for this project where the subgrade cannot be adequately dried and recompacted, and it is necessary to protect the subgrade.

The workpad should consist of a relatively uniform crushed rock with a maximum particle size of 4 to 6 inches to be placed and roller compacted over a woven geotextile such as a Exxon GTF-200 or equivalent. Along a major construction access road, the workpad thickness should be about 18 to 24 inches where construction traffic is concentrated. At major staging areas where the construction traffic is less concentrated, the workpad thickness may be reduced to 12 to 18 inches thick (depending upon the needs of the Contractor based on vehicle weights and trip frequency). At the building site where much of the construction traffic is light (backhoe/light truck) and well distributed over the site, the work pad thickness may be reduced to 12 inches.

These workpad thickness recommendations are for preliminary budgetary purposes only since we do not know for certain how heavy the heaviest construction vehicle that will be used by the Contractor for this project. We recommend that the Contractor be consulted about the thicknesses of the workpad.

If the subgrade, in the winter or wet season, is not adequately protected, then construction trucks may cause rutting/pumping of the soil subgrade (especially during wet periods). Disturbed soil/disturbed workpad will need to be removed and replaced or fixed/recompacted with engineered fill.

If construction cannot be conducted during the normal summer/fall construction season and/or if pumping/rutting due to construction traffic begins to occur as observed by your representative, the subgrade should be protected and additional costs should be anticipated.

The Contractor should be made aware of the possibility of difficult excavations and should select the appropriate excavation equipment and methods at no additional cost to the Owner (unless, as otherwise directed within the Specifications, if any) should any excavation be made into the lower gravel areas or anywhere where embedded boulders are encountered, in our opinion. In addition, the Contractor should also be made aware that large boulders may be encountered. Such boulders may need to be hauled off-site if the boulders cannot be utilized in the landscape design per the Owner's wishes.

Engineered Fill. Any reasonably graded, insitu or imported silty or sandy soil that is substantially free of clay or organic or other deleterious matter or oversized material (larger than 2 to 3 inches, for testing purposes) would be suitable as engineered backfill beneath the slab and pavement areas, if any, if the backfill is placed during dry warm weather on a dry subgrade surface and it is properly moisture-conditioned to within 2 percent of optimum moisture (i.e. aerated to lower the moisture content or moistened to raise the moisture content depending upon existing field moisture and optimum moisture content) before and during placement.

Any surficial organic strippings/organic clay or debris laden fill, if any is found, should not be used for engineered backfill purposes inside building and pavement areas, in our opinion. This unsuitable material may be used for landscape fill if desired. Otherwise, this material should be properly disposed off-site.

We recommend that a clean (not more than 5 percent passing the No. 200 sieve based on a wet sieve analysis) reasonably well-graded granular material such as a sand and gravel or crushed rock be used for engineered backfill for the following situations:

- during the wet periods when there is insufficient time or dry hot weather to dry the soil moisture to optimum moisture content,

- o when excess moisture that is present in the subgrade is observed to be migrating to the fill layer during compaction such that pumping is observed or specified compaction levels cannot be achieved using on-site soils.

The gradation of the any granular import material selected should be checked to determine its compatibility for use adjacent to on-site soils. The maximum particle size of the granular engineered backfill should not exceed 1-1/2 inches for testing purposes. We also recommend that samples of backfill material intended for use as engineered fill be submitted for approval prior to earthwork construction.

Engineered backfills should be placed in about 9 to 12-inch loose lifts for areas that are compacted with large self-propelled rollers, and should be compacted to a dry density of at least 98 percent of the standard Proctor maximum dry density (ASTM D698) or as approved by the Geotechnical Engineer within the proposed building and pavement areas, if any. Landscape fills outside of the building and pavement areas, if any, may be compacted to 90 percent. Lift sizes for small vibrating plates typically used in trenches vary from 6 to 8 inches. The size of the lifts and the number of passes of the compactor may need to be modified to achieve the desired results using the equipment selected. The engineered fill should be placed in horizontal lifts commencing on a relatively level subgrade surface.

Surcharging. The subject of surcharging the new fill area in the southeast corner of the property has already been introduced earlier in this Report. After raising the site with engineered fill to an elevation that is, at least, 1 foot higher than required subgrade elevation, the surcharge fill can then be placed over the new fill area and that portion of the edge/slope of the existing fill pad, at the southeast corner, that was repaired during the raising of the new fill area.

The surcharge fill should overlap the top of the repaired edge/slope of the existing fill pad area for an approximate horizontal distance of 5 feet before beginning to slope down to natural ground surface, if at all possible. We recommend a temporary slope of 1H (Horizontal):1V (Vertical) for the surcharge fill.

Sometimes, Contractors calculate the volume of the surcharge fill required and the volume of crushed rock needed for the project. If the volumes are comparable, the Contractor may choose to stockpile the crushed rock for the entire project at the surcharge area. Or, likewise, depending upon the grading plan, the Contractor may use the "spoils" to surcharge the required area.

Either way is acceptable as long as the surcharge is placed relatively quickly (say, within a few days).

The surcharge fill does not need to be compacted and should be placed as quickly as possible so the total weight of the surcharge fill can begin to consolidate the underlying soils. We currently recommend a surcharge fill height of 12 feet for the southeast corner of the property. We recommend that we be allowed to review this recommendation when a grading plan is available and a finished floor elevation has been determined.

We estimate that the surcharge fill will need to remain on site for about 1 to 1-1/2 months, more or less. The actual duration of the surcharge fill should be determined from an analysis of settlement plate data. Two settlement plates are recommended to be installed at the southeast corner in order to provide settlement information (See Figure 7 for a typical settlement plate).

The settlement plate should be installed in areas chosen by the Geotechnical Engineer after the site has been stripped of its organic topsoil and existing fill layers. The installation of the settlement plates should be conducted by your Contractor and in our presence.

Your Surveyor should read the settlement plates every day during surcharge fill placement and for a period of about one week after the surcharge fill is completed. For the next week, we recommend settlement plate readings every other day. Then, given our approval, based on our analysis of the settlement plate data to date, the reading interval may be increased to weekly readings.

The Surveyor should provide the settlement plate data to us in a timely manner, so that we may be able to evaluate the actual duration of the surcharge load and/or whether or not the surcharge load needs to be modified to correlate to time constraints. After successful completion of the surcharging, the surcharge load can then be removed down to required subgrade elevation. Recompaction of the building and pavement areas is advised before proceeding with the project.

We recommend that your Surveyor be experienced with surcharges and settlement plate readings. The Surveyor's benchmark(s) should be located in an undisturbed, native soil area far enough away from any existing and/or new fill so as to be free of settlement influences of the surcharge load and construction disturbance.

Dewatering. Groundwater, if any, or any surface water flow should be controlled in a manner that will not affect excavation or fill construction. The underlying soils will slough into any trench excavation especially when wet. The Excavator should excavate in such a manner that nearby footings, slabs and utilities designated to remain are not undermined by potential sloughing. Water should not be allowed to pond in the bottoms of the footings/slab/pavement areas. Exposed subgrade or fill softened by ponded water should be removed and replaced with engineered fill.

Seepage from the underlying sandy gravel layer may be too rapid, in our opinion, to be satisfactorily removed by sump pumps in excavations without causing heaving and localized areas of liquefaction of the subsurface soils. If such conditions are observed during construction, we recommend that we be allowed to evaluate the situation and determine if more sophisticated dewatering systems, such as well points, are necessary.

The Contractor should be made responsible for the satisfactory installation of any sophisticated dewatering systems and the removal of groundwater/surface water seepage without causing "cave-ins" or sloughing.

Cut and Fill Slopes. All permanent cut and fill slopes, if any, should be groomed to slopes no steeper than 2 Horizontal (H): 1 Vertical (V) for stability purposes. Flatter slopes may be necessary for ground cover and maintenance operations. We also recommend that engineered fill extend outwards from the edge of any footing for an approximate length of 5 feet before down sloping to lower grades.

Because of safety considerations and the nature of temporary excavations, the Excavator should be responsible for maintaining safe cut excavations and supports. We recommend that the Excavator incorporate all pertinent safety codes during construction including the latest edition of the OR-OSHA Standards for Construction Industry (Type C Soil). This classification should be verified during excavation by a "competent person" as defined by OR-OSHA.

Underground Fuel Tanks. Underground fuel tanks or contaminated soil, if any are known to be present or are found during excavation, should be removed in accordance with Oregon Department of Environmental Quality requirements and backfilled with engineered fill.

LIMITATIONS

It is recommended that close quality control be exercised during the preparation and construction of building foundations and pavement sections. Fills and new asphalt or concrete pavement and base sections should be monitored and tested by a qualified representative. In addition, we also advise that the subgrade preparation, Geopier installation, if any, and the footing excavations be inspected by the Geotechnical Engineer.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes of construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or excavating test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, a contingency fund is recommended to accommodate such potential extra cost.

Be advised that the Local Governing Agency may sometimes require additional geotechnical or other studies in order to approve the development as part of the planning approval process. Our Geotechnical Report(s) does not guarantee that the development will be approved by the Local Governing Agency without these additional studies, if required by the Local Governing Agency, being performed. Expenses incurred in reliance upon our Report(s) prior to final approval of the Local Governing Agency are the exclusive responsibility of the Developer. In no event shall West Coast Geotech, Inc., be responsible for any delays in approval which are not exclusively caused by West Coast Geotech, Inc..

Very truly yours,

WEST COAST GEOTECH, INC.

By M.F. Schrieber

Michael F. Schrieber, P.E.
Geotechnical Engineer
President

REGISTERED PROFESSIONAL ENGINEER
OREGON
Jan. 20, 1989
M.F. SCHRIEBER

cc: Mr. Eric Watson, P.E. (Miller Consulting Engineers, Inc.)

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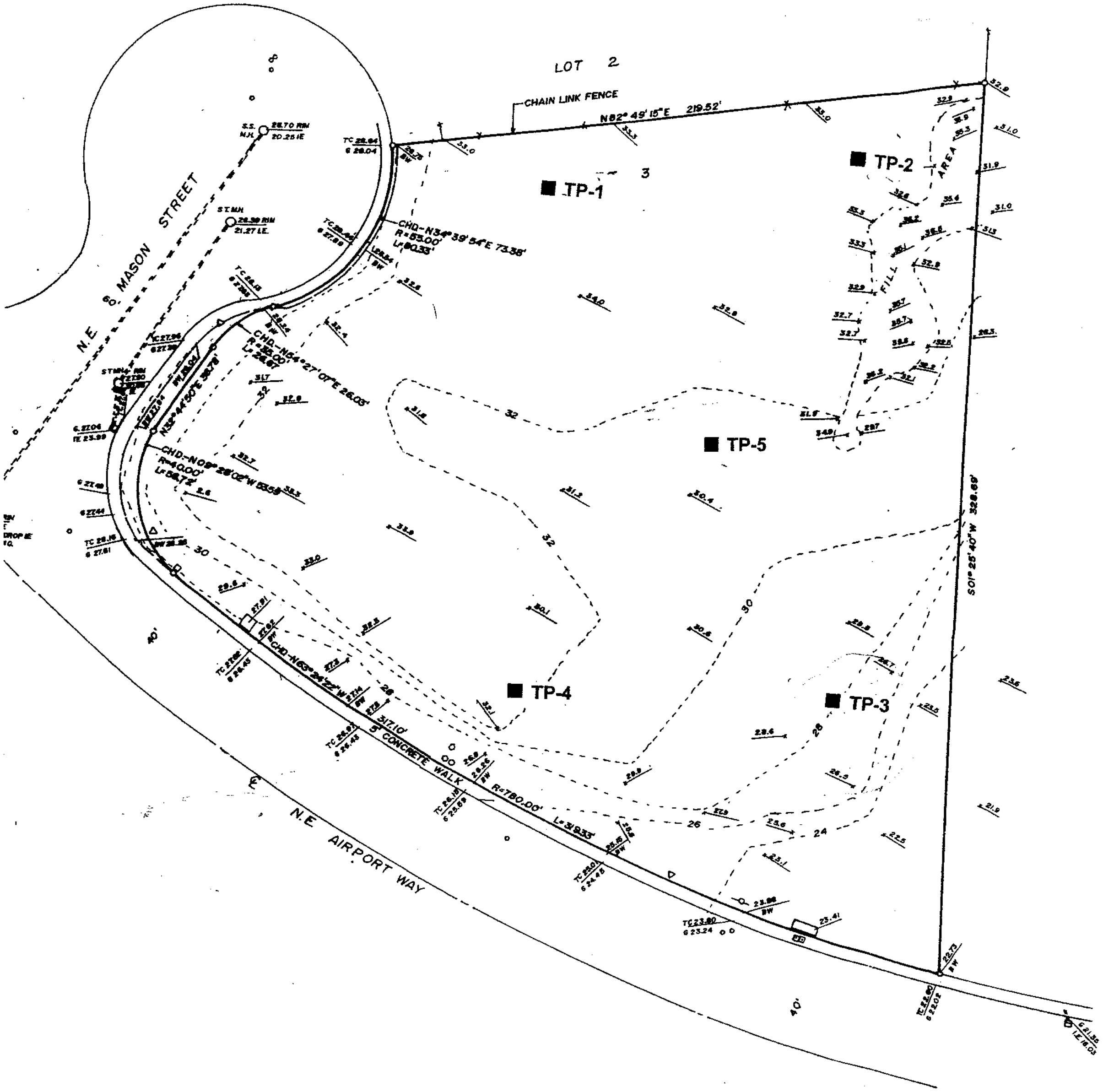

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Notes:
Test Pit Location Plan :
Topographic Plan, date
James Andrews Survey
use by Winstead & Ass

TP-1 ■

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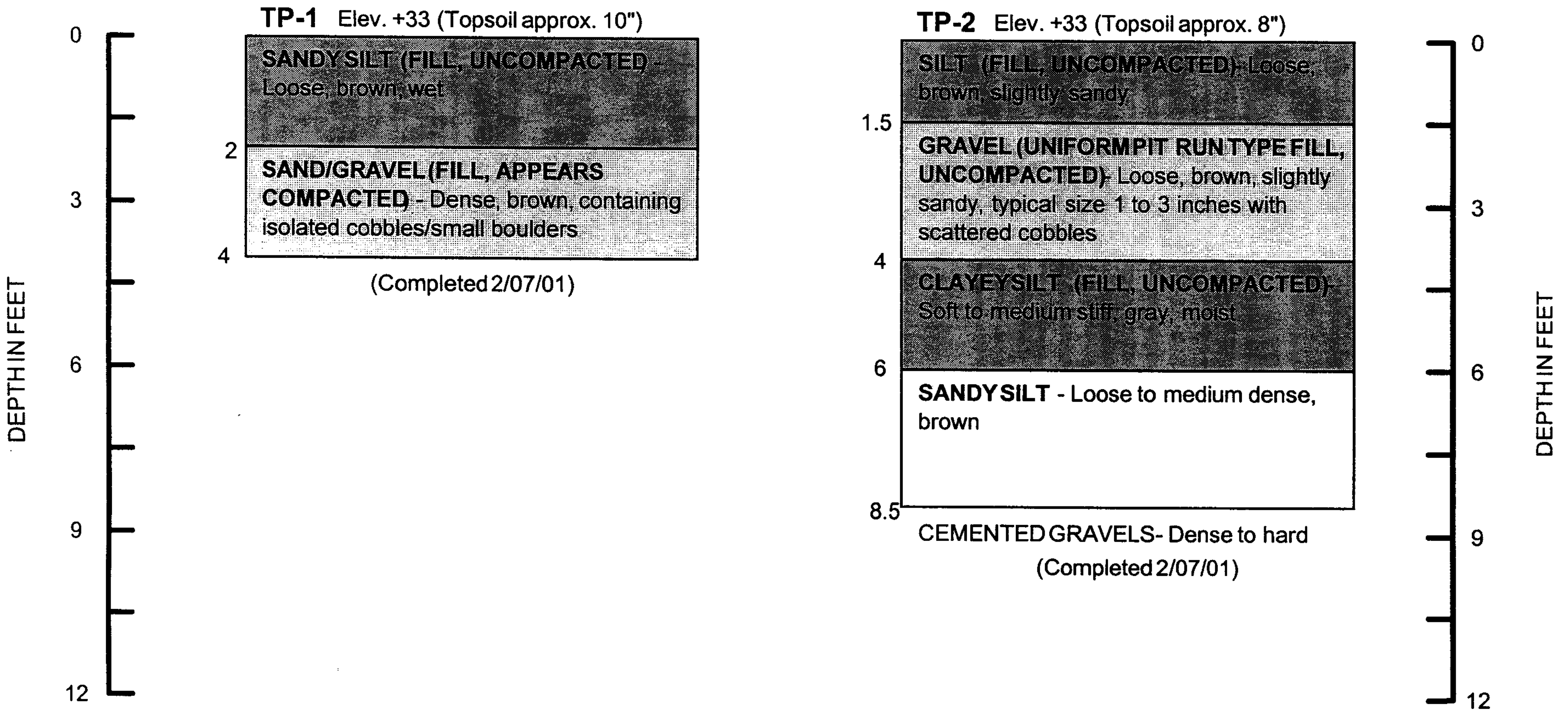


LEGEND

B BAG SAMPLE AND NUMBER

T TUBE SAMPLE AND NUMBER

NOTES

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

2. WATER LEVEL, IF ANY, IS FOR DATE SHOWN AND MAY VARY WITH TIME AND YEAR.

AIRPORT WAY WAREHOUSE **Portland, Oregon**	
LOGS OF TEST PITS **TP-1 TO TP-2**	
Mar., 2001	W-1536
WEST COAST GEOTECH, INC. Geotechnical Consultants West Linn, Oregon	**FIG. 2**

TP-3 Elev. +28 (Topsoil approx. 10")

DEPTH IN FEET

0 – 3: **SAND/SILT (FILL, UNCOMPACTED** - Loose, brown, gravelly, containing cobbles/small boulders

3 – 6: **SAND/SILT (FILL, UNCOMPACTED** - Loose, dark brown to gray, containing wood debris

6 – 8.5: **SANDY SILT** - Loose to medium dense, brown

(Completed 2/07/01)

TP-4 Elev. +32 (Topsoil approx. 8")

DEPTH IN FEET

0 – 1.5: **SILT (FILL, UNCOMPACTED)**- Loose, brown, slightly sandy

1.5 – 6: **SAND/SILT (FILL, UNCOMPACTED** - Loose, brown, gravelly, containing cobbles/small boulders

6 – 8.5: **SANDY SILT** - Loose to medium dense, brown

(Completed 2/07/01)

LEGEND

B BAG SAMPLE AND NUMBER

T TUBE SAMPLE AND NUMBER

NOTES

1. SOIL DESCRIPTIONSAND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

2. WATER LEVEL, IF ANY, IS FOR DATE SHOWN AND MAY VARY WITH TIME AND YEAR.

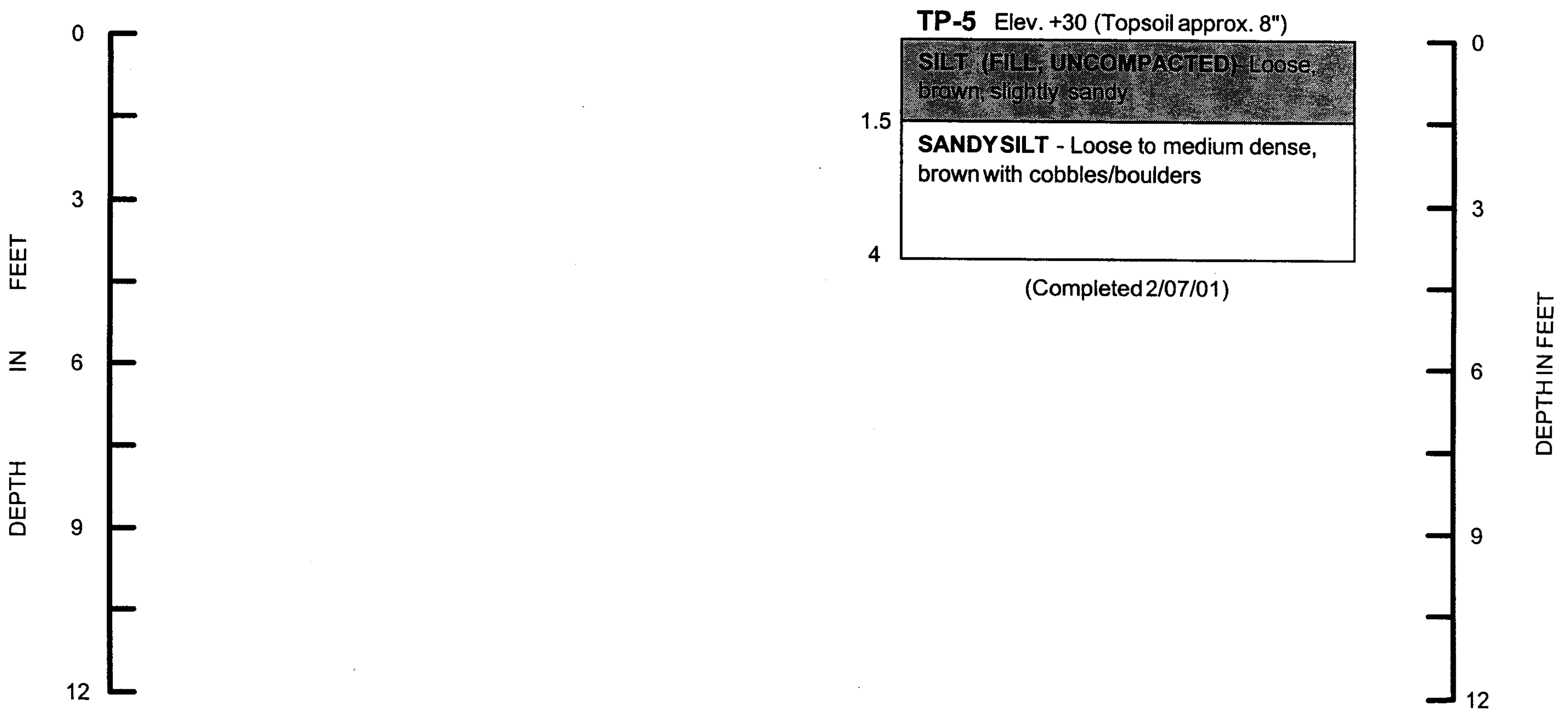
AIRPORT WAY WAREHOUSE
Portland, Oregon

LOGS OF TEST PITS
TP-3 TO TP-4

Mar., 2001 W-1536

WEST COAST GEOTECH, INC.
Geotechnical Consultants
West Linn, Oregon

FIG. 3



LEGEND

B BAG SAMPLE AND NUMBER

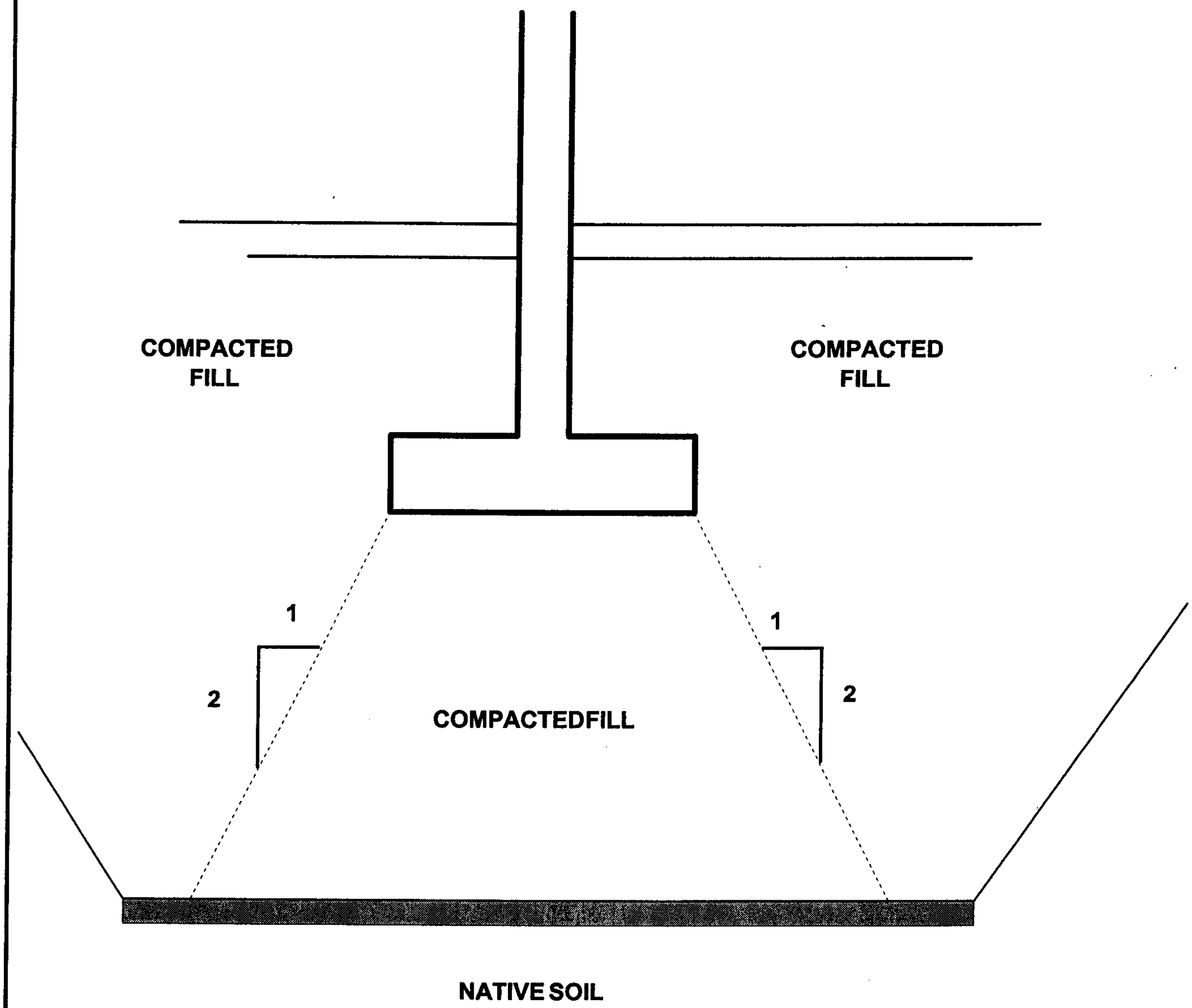
T TUBE SAMPLE AND NUMBER

NOTES

1. SOIL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

2. WATER LEVEL, IF ANY, IS FOR DATE SHOWN AND MAY VARY WITH TIME AND YEAR.

AIRPORT WAY WAREHOUSE **Portland, Oregon**	
LOG OF TEST PIT **TP-5**	
Mar., 2001	W-1536
WEST COAST GEOTECH, INC. Geotechnical Consultants West Linn, Oregon	**FIG. 4**



AIRPORT WAY WAREHOUSE
Portland, Oregon

LIMITS OF COMPACTED FILL UNDER FOOTINGS

Mar., 2001 **W-1536**

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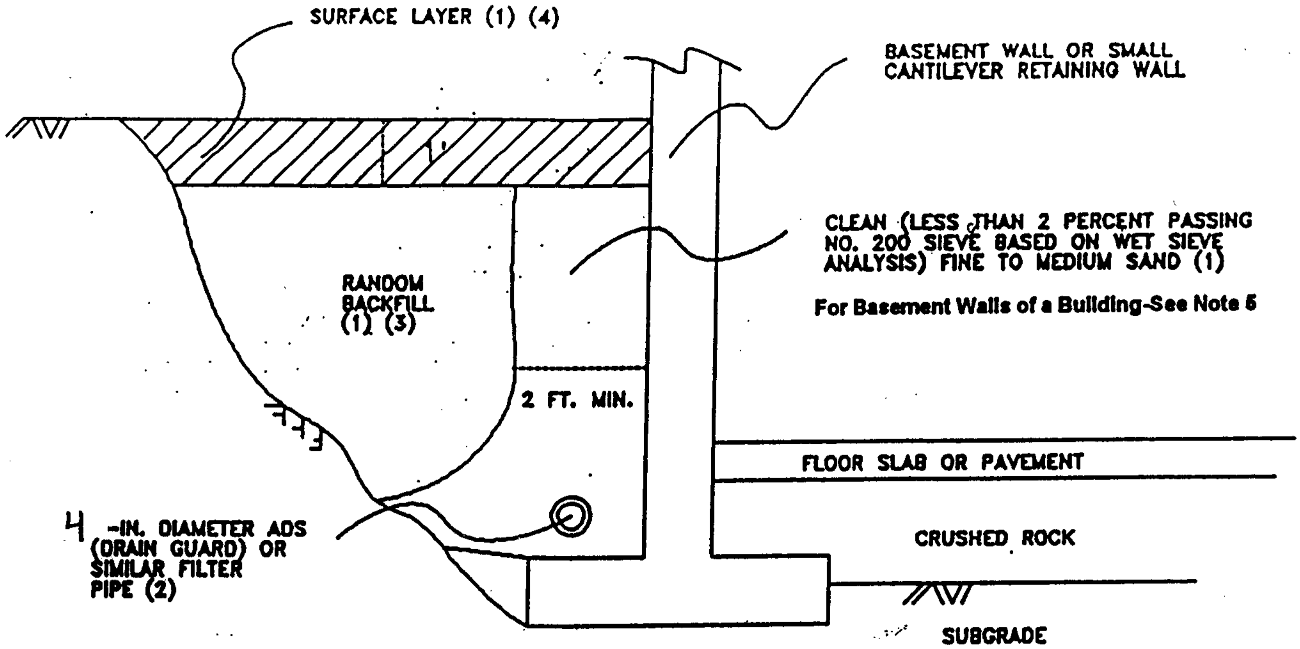
FIG. 5



NOTES:

1. COMPACT RANDOM BACKFILL, CLEAN SAND, AND SURFACE LAYER TO 92 PERCENT OF STANDARD PROCTOR MAXIMUM DRY DENSITY. ALL FILL MATERIALS SHALL BE PROPERLY MOISTURE CONDITIONED SUCH THAT MOISTURE CONTENT IS WITHIN 2 PERCENT OF OPTIMUM MOISTURE CONTENT.

2. PERFORATED PIPE SURROUNDED BY A 6-INCH ENVELOPE OF 1-1/2 INCH MINUS CRUSHED ROCK MAY BE USED IN LIEU OF FILTER PIPE.

3. RANDOM BACKFILL SHALL CONSIST OF:

   CLEAN (NOT MORE THAN 2 PERCENT PASSING NO. 200 SIEVE BASED ON WET SIEVE ANALYSIS) FINE TO MEDIUM SAND BENEATH CONCRETE SLABS

   EXCAVATED ON-SITE MATERIAL FREE OF ORGANIC AND DELETERIOUS MATERIAL, AND OVERSIZED FRACTION LARGER THAN 2 INCHES BENEATH LANDSCAPE AND PAVEMENT AREAS.

4. SURFACE LAYER SHALL CONSIST OF:

   CLEAN (NOT MORE THAN 2 PERCENT PASSING NO. 200 SIEVE BASED ON WET SIEVE ANALYSIS) FINE TO MEDIUM SAND BENEATH CONCRETE SLABS AND PAVEMENT AREAS.

   IMPERVIOUS FINE-GRAINED SILTS AND CLAYS FOR LANDSCAPE AREAS.

5. For basement walls of a building, if any, the drainage blanket should consist of clean, washed crushed rock with the following gradations (based on wash sieve):

Sieve Size Passing	Percentage (By Weight)
2	100
3/8"	40-80
No. 40	0-35
No. 100	0-10
No. 200	0-2

A non-woven geotextile is recommended to separate the drainage blanket from the random backfill in order to keep the drainage blanket clean.

AIRPORT WAY WAREHOUSE
Portland, Oregon

BACKFILL AND SUBDRAIN DETAILS

Mar., 2001 W-1536

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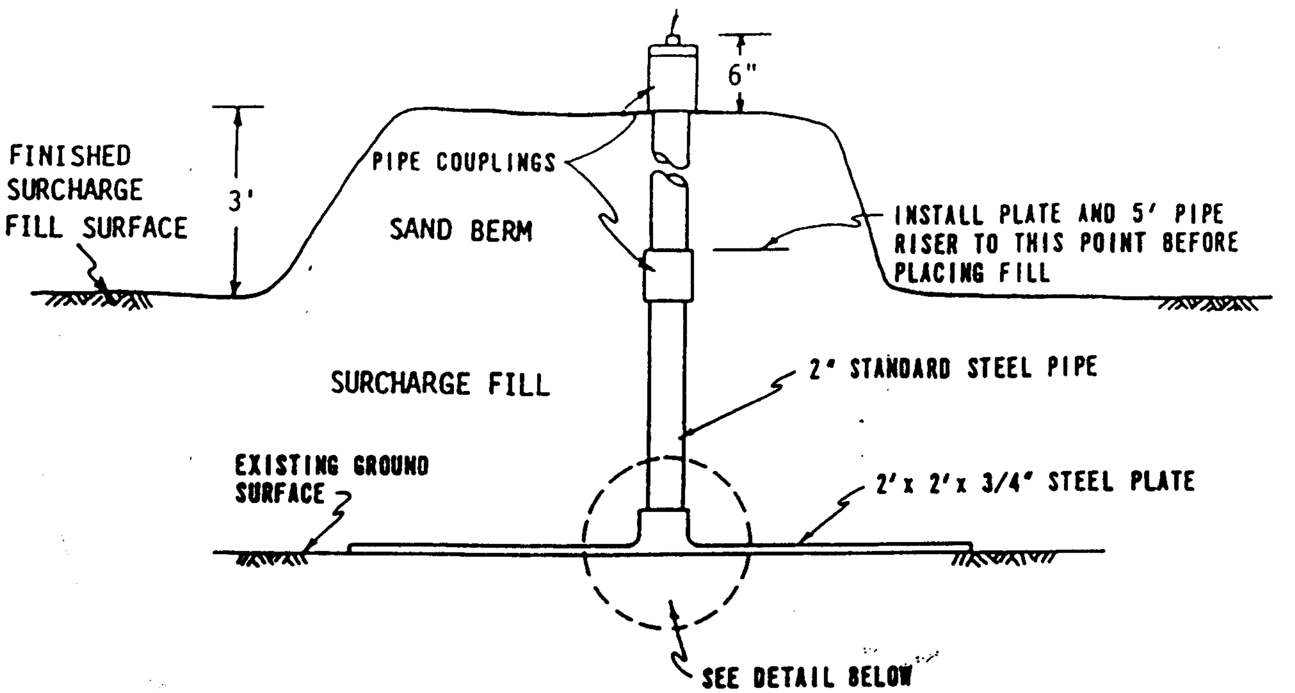
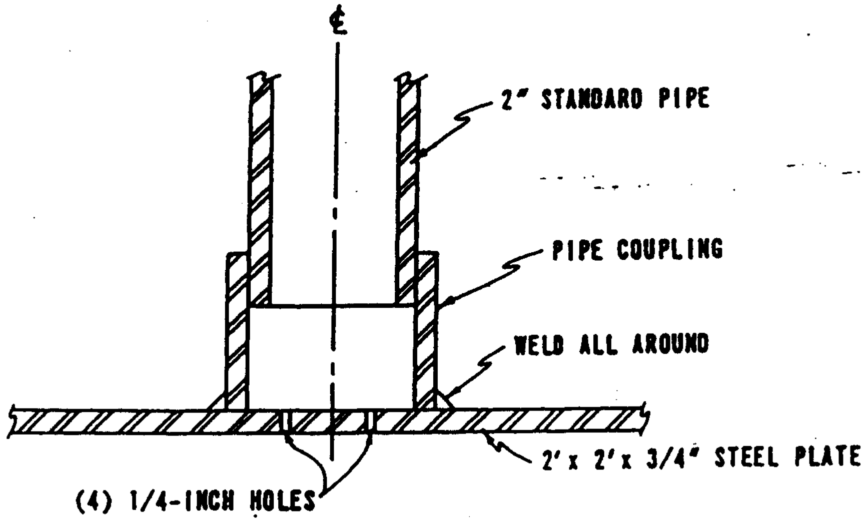
FIG. 6





AIRPORT WAY WAREHOUSE
Portland, Oregon

STANDARD STEEL SETTLEMENT PLATE DETAILS

Mar., 2001 W-1536

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FIG. 7