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April 26, 2001

LSI ADAPT Project No. OR01-5758

FILE

Nextel Communications
8705 SW Nimbus Road
Beaverton, Oregon 97008

Attention: Mr. Dann Detwiler

Subject: Geotechnical Engineering Evaluation
Proposed Tower Facility
Eastport/Daisy – OR0205-2
2330 SE 82nd Avenue
Portland, Oregon 97216
1S2E04CC 07800

01-197261-CO

Dear Mr. Detwiler:

Pursuant to your request, LSI ADAPT Inc. (ADAPT) is pleased to submit this report describing our recent geotechnical engineering evaluation for the above-referenced site. The purpose of this study was to interpret general surface and subsurface site conditions, from which we could evaluate the feasibility of the project and formulate design recommendations concerning site preparation, equipment pad and tower foundations, structural fill, and other considerations. Our scope of services consisted of a surface reconnaissance, a subsurface exploration, geotechnical analyses, and report preparation. Authorization to proceed with our study was given by Nextel prior to our performing the work.

This geotechnical engineering evaluation has been conducted in accordance with generally accepted geotechnical engineering practices. This report has been prepared for the exclusive use of Nextel and their agents for specific application to the project site. Use or reliance upon this report by a third party is at their own risk. ADAPT does not make any representation or warranty, expressed or implied, to such other parties as to the accuracy or completeness of this report or the suitability of its use by such other parties for any purpose whatever, known or unknown, to ADAPT.

ADAPT appreciates the opportunity to be of service to you on this project. Should you have any questions concerning this report, or if we can assist you further, please contact us at (503) 639-3413.

Respectfully submitted,

LSI ADAPT Inc.

Kirk L. Warner, R.G.
Senior Geologist

REGISTERED PROFESSIONAL ENGINEER
51399PE
OREGON
FEB. 8, 2000
DANIEL H. WATKINS
Expires: 30 June 2002

Daniel H. Watkins, P.E.
Geotechnical Engineer

FOR
K. V. Lew, P. Eng.
Senior Geotechnical Engineer

Attachments:	Figure 1	Location/Topographic Map
	Figure 2	Site and Exploration Plan
	Appendix A	Boring Log
	Appendix B	Soil Resistivity

PD/klw

LSI ADAPT

NEXTEL COMMUNICATIONS
Geotechnical Engineering Evaluation

EASTPORT
OR0205-2
Portland, Oregon

OR01-5758
April 26, 2001

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Suite100
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LSI ADAPT Inc.

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PROJECT DESCRIPTION

The host parcel is located at 2330 SE 82nd Avenue (abutting an existing auto body shop) in Portland, Multnomah County, Oregon. The proposed lease area is an approximately 50-foot by 50-foot parcel located in a paved parking area near the southeast corner of the host property. Tower installation would not require the removal of any trees and there were no drainage issues apparent at the time of the site visit and test drilling. We understand that site development would include the construction of a monopole tower and associated cellular support equipment building or cabinets. Site access is through a paved parking lot off of 83rd Avenue and is viable in all weather conditions. The project site and surrounding area are shown on the attached site vicinity map (Location/Topographic Map, Figure 1). The attached Site and Exploration Plan (Figure 2) shows the approximate location of the proposed cellular tower lease area and soil boring location relative to other site features.

It should be emphasized that the conclusions and recommendations contained in this report are based on our understanding of the currently proposed utilization of the project site, as derived from written information supplied to us by Nextel. Consequently, if any changes are made to the project, we recommend that we review the changes and modify our recommendations, if appropriate, to reflect those changes.

EXPLORATORY METHODS

ADAPT explored surface and subsurface conditions at the project site on April 20, 2001. Our surface exploration consisted of a visual site reconnaissance. Our subsurface exploration consisted of advancing one soil test boring to a depth of approximately 36.5-feet below ground surface (bgs). The approximate location of the boring, designated B-1, is shown on the attached Site and Exploration Plan (Figure 2).

The specific location and depth of the exploration performed was selected in relation to the proposed site features, under the constraints of budget and site access. The location of the boring and other features shown on Figure 2 were obtained by hand taping from existing site features. As such, the exploration location shown on Figure 2 should be considered accurate to the degree implied by the measuring methods used.

Boring Methods

Boring B-1 was advanced on April 20, 2001 using a truck mounted hollow-stem auger drill rig. The drill rig was operated by an independent company working under subcontract to ADAPT. A field geologist from our firm continuously observed the boring, obtained representative soil samples, and logged the subsurface conditions. After the boring was completed, the borehole was backfilled with bentonite chips and soil cuttings.

During drilling, soil samples were obtained at 5-foot depth intervals using the Standard Penetration Test (SPT) procedure (ASTM: D 1586). This test and sampling method consists of driving a standard 2-inch outside diameter (OD) split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer, free-falling a distance of 30 inches. The number of blows required to drive the sampler through each of the three 6-inch intervals is noted. The total number of blows struck during the final 12 inches of penetration is considered the Standard Penetration Resistance, or "blow count". If 50 or more blows are struck within one 6-inch interval, the driving is ceased and the blow count is recorded as 50 blows for the actual number of inches of penetration. The resulting Standard Penetration Resistance values provide a measure of the relative density of granular soils or the relative consistency of cohesive soils.

The boring log appended to this report describes the various types of soils encountered in the boring, based primarily on visual interpretations made in the field. The log also indicates the approximate depths of the contacts between different soil types, although these contacts may be gradational or undulating. Where a change in soil type occurred between sampling intervals, we inferred the depth of contact based on driller comments and/or field experience. In addition, the log indicates the depth of any groundwater observed in the boring, the Standard Penetration Resistance at each sample location, and any laboratory tests performed on the soil samples.

SITE CONDITIONS

The following sections describe our observations, measurements, and interpretations concerning surface, soil, groundwater, and seismic conditions at the project site.

Surface Conditions

The host parcel is located at 2330 SE 82nd Avenue (abutting an existing auto body shop) in Portland, Multnomah County, Oregon. The proposed lease area is an approximately 50-foot by 50-foot parcel located in a paved parking area near the southeast corner of the host property. Tower installation would not require the removal of any trees and there were no drainage issues apparent at the time of the site visit and test drilling. We understand that site development would include the construction of a monopole tower and associated cellular support equipment building or cabinets. Site access is through a paved parking lot off of 83rd Avenue and is viable in all weather conditions. The project site and surrounding area are shown on the attached site vicinity map (Location/Topographic Map, Figure 1). The attached Site and Exploration Plan (Figure 2) shows the approximate location of the proposed cellular tower lease area and soil boring location relative to other site features.

Soil Resistivity Testing

Field-testing of soil resistivity was performed at the site on April 24, 2001. Two surveys were completed at the approximate locations and orientations described in Appendix B. ADAPT personnel completed the resistivity testing using a Nilsson Model 400 Soil Resistance Meter, with the Wenner Four-Electrode

Method, in general accordance with ASTM method G-57-95a (IEEE Standard 81). Resistivity test results are tabulated in Appendix B.

Subsurface Conditions

On April 20, 2001, an exploratory test boring was drilled to a depth of approximately 36.5 feet below the existing ground surface (bgs). The location of the boring, designated as B-1, is shown on Figure 2. Subsurface conditions encountered in boring B-1 consisted of approximately 4.5 feet of medium stiff, dark brown silt overlying medium dense, gray, fine to coarse sand with some gravel that extended to a depth of about 9 feet. Below approximately 9 feet, the boring encountered medium dense, gray, fine to coarse sand, extending to the maximum depth of boring B-1 at 36.5 feet. No olfactory indications of potential contaminants were detected with the soil samples recovered during the test drilling.

Groundwater was encountered at an approximate depth of 15 feet (bgs) in boring B-1 at the time of drilling. However, groundwater levels can fluctuate due to factors such as seasonal variations in precipitation, changes in site utilization, or other factors.

Seismic Conditions

According to the Seismic Zone Map of the United States contained in Figure 16-2 of the 1997 Uniform Building Code (UBC), the project site lies within Seismic Risk Zone 3. Based on our subsurface exploration, we interpret the site conditions to closely correspond to a seismic Soil Profile type S_D, for stiff Soil, as defined by Table 16-J of the 1997 Uniform Building Code. This classification is based on the observed range of Standard Penetration Test (SPT) blow counts for the soil types encountered in boring B-1. The shallow soil conditions were assumed to be representative of site conditions beyond the depths explored. In addition, according to the USGS, a 10% probability of exceeding a peak ground acceleration of 0.175*g can be expected in the next 50 years.

CONCLUSIONS AND RECOMMENDATIONS

The proposed project would consist of the construction of a monopole tower and equipment building or cabinets. Based on the subsurface conditions encountered in our boring, we recommend the proposed tower be supported on either a mat or a drilled pier foundation. Due to the fact that the host property abuts an existing auto body shop, the potential exists for current and future environmental groundwater impacts related to the adjacent site use. Considering the limited tower height of 65-feet, in order to limit potential environmental liability, we recommend that foundations penetrate no deeper than 15 feet bgs, due to the possibility of encountering the impacted groundwater during construction. Design criteria for compressive, uplift and lateral support of mat and drilled pier foundations are presented below. Our specific recommendations concerning site preparation, equipment cabinets or building platform, tower foundations, access driveway and structural fill are presented in the following sections.

Site Preparation

Site preparation will involve removal of existing asphalt pavement and near surface soils, and preparation of subgrades. The following comments and recommendations apply to site preparation:

Clearing and Grubbing: We do not anticipate that significant grade changes will be required to achieve proposed site grades. At this location, site preparation will consist of removal of existing asphalt pavement and near surface soils, limited grading, followed by foundation preparation for the equipment cabinet/building and monopole foundation. Backfill materials, where required, should be placed and compacted according to the recommendations presented in the Structural Fill section of this report. A representative from ADAPT should be retained to observe the site preparation and installation process.

Wet Conditions: Because of the high fines content of the existing near surface silt soils, they should be considered highly moisture-sensitive and prone to disturbance when wet. The contractor should minimize traffic above prepared subgrade areas to minimize disturbance and softening which would require removal of the unstable soils. During wet conditions, the use of a working surface of quarry spalls or clean sand and gravel may be required to protect the subgrade, especially from vehicular traffic.

Frozen Subgrades: If earthwork takes place during freezing conditions, we recommend that all exposed subgrades be allowed to thaw and be recompacted prior to placing subsequent lifts of structural fill.

Liquefaction: We do not anticipate any liquefaction concerns at this site as the sands encountered below the groundwater table appear to be medium dense and not prone to liquefaction.

Equipment Building or Cabinet Foundation

It is our understanding that the foundation for the proposed equipment building or cabinets will consist of a poured in place concrete slab-on-grade with thickened edges. We anticipate that the pad bearing pressure will be relatively light. However, we recommend that the thickened slab edges be designed as spread footings. The following recommendations and comments are provided for purposes of footing design and construction:

Subgrade Conditions: We anticipate the subsoil encountered at the proposed foundation grade will likely consist of medium stiff, dark brown silt. To prepare the subgrade, the near surface soil should be stripped and excavated to a depth of at least 18-inches below the lowest adjacent grade (or deeper if local code dictates for frost protection). It should be confirmed, at the time of construction, that the material at the bearing elevation is as anticipated in the design. The subgrade should then be compacted in place, if the moisture content allows, resulting in a firm and unyielding subgrade condition. Overexcavation may be necessary to remove excessively soft or wet soils if encountered at the foundation design grade levels. If excessively soft soils are encountered, they should be removed to a depth of no deeper than 2-feet below the thickened slab bearing elevation, in light of the light loads imposed by the equipment cabinets. The resulting over-excavation should be backfilled with granular structural fill and compacted per the structural fill recommendations. A layer of geotextile may be required to separate the structural fill soils from the underlying subgrade materials. Footings should never be cast atop soft, loose, organic, or frozen soils, nor atop subgrades covered by standing water. A representative from ADAPT should be retained to observe the condition of footing subgrades before concrete is poured to verify that they have been adequately prepared.

Footing Dimensions: We recommend that the thickened edge of the slab be designed as a spread footing and be constructed to have a minimum width of 12 inches. For frost protection, the footings should penetrate at least 18-inches below the lowest adjacent exterior grades (or deeper if local code dictates). Footings may also be supported on structural fill placed on prepared soil subgrade. The horizontal limits of the fill pad below the building or cabinet foundation may be established by extending a line outward from the base of the thickened slab at an angle of 1 Horizontal: 1 Vertical (1H: 1V) down to the upper surface of the bearing horizon.

Bearing Pressure: A maximum allowable soil bearing pressure of 2,000 pounds per-square-foot can be used for static footing loads. This bearing pressure can be increased by one-third to accommodate transient wind or seismic loads. An allowable base friction coefficient of 0.31 and an allowable passive earth pressure of 280 pounds per cubic foot (pcf), expressed as an equivalent fluid unit weight, may be used for that portion of the foundation embedded more than 1 foot below finished exterior subgrade elevation.

Settlements: We estimate that total post-construction settlements of properly designed footings bearing on properly prepared subgrades from static loads could approach 1-inch, with differential settlements approaching one-half of the total.

Access Driveway

Access to the site is through a paved parking lot off of 83rd Avenue and is viable in all weather conditions. Hence, at this time we do not anticipate construction of a separate access driveway.

Tower Mat Foundations

In order to provide adequate resistance to horizontal, axial and overturning loads, a reinforced concrete mat footing could be used for tower foundation support. The following recommendations and comments are provided for purposes of mat footing design and construction:

Subgrade Conditions: Footing subgrades should consist of firm, unyielding native soils. The mat foundation should never be cast atop soft, loose, organic, or frozen soils; nor atop subgrades covered by standing water. A representative from ADAPT should be retained to observe the condition of mat subgrade soils before concrete is poured to verify that they have been adequately prepared.

Embedment Depths: We recommend that the mat foundation be embedded at least 5-feet below the existing ground surface bearing on the medium dense, sand underlying the site, but not more than 15-feet below existing ground due to the possibility of encountering the impacted groundwater during construction. We anticipate the medium dense, sand could be excavated using conventional excavating equipment such as a large trackhoe. After excavation to the design footing grade, the surface of the bearing horizon should be cleaned of material loosened by excavation, and if possible, compacted in place resulting in a firm and unyielding subgrade condition prior to placement of rebar and concrete.

Bearing Pressures: A maximum allowable soil bearing pressure of 4,000 pounds per square foot can be used for static loads to minimize foundation settlement. For shallow footings, the amount of settlement that will take place is directly related to the size of the footing for a given bearing pressure. Depending on the allowable settlement that can be tolerated, higher bearing pressure may be recommended. We can be contacted to provide recommendations for higher bearing pressures, if desired, for this site. This bearing pressure incorporates a factor of safety of 1.5 or more and can be increased by one-third to accommodate transient wind or seismic loads. We expect that uplift loads will be resisted by the dead load of the mat foundation, as well as the weight of soils covering the mat. Native soils used to cover the mat and compacted to a minimum of 90 percent of the modified Proctor maximum dry density could be assumed to have a unit density of 105 pcf in place.

Lateral Resistance: Lateral loads on the foundation caused by seismic or transient loading conditions may be resisted by a combination of passive soil pressure against the side of the foundation and shear friction resistance along the base. An allowable base friction coefficient of 0.36 for the medium dense sand-concrete interface and an allowable passive earth pressure of 350 pounds per-cubic-foot (pcf), expressed as an equivalent fluid unit weight, may be applied against that portion of the foundation embedded at least 2 feet in native soils. We recommend that the passive contribution of the upper 2-feet of embedment be fully discounted.

Settlements: We estimate that total post-construction settlements of properly designed mat foundation bearing on properly prepared subgrades could approach 1 to 2 inches, with differential settlements approaching one-half of the total.

Construction Considerations: Side slopes for the footing excavation should not be steeper than 1.5H:1V based on soil type and consistency. We do not anticipate the need for shoring or other special considerations if the excavation is constructed per these recommendations. In addition, should wet weather be anticipated, the slopes should be covered with plastic to prevent saturation and possible collapse. Also spoils should be kept back from the edge of the excavation so as to not add additional surcharge to the construction slopes.

Tower Drilled Pier Foundations

As an alternative to the mat foundation, the tower could also be supported on a drilled pier foundation. The following recommendations and comments are provided for purposes of drilled pier design and construction:

Compressive Capacities: We recommend that the drilled pier penetrate at least 5-feet below the existing ground surface bearing on the medium dense, gray, fine sand underlying the site, but not more than 15-feet below existing ground due to the possibility of encountering impacted groundwater during construction. For vertical compressive soil bearing capacity, we recommend using the unit end bearing capacity presented in Table 1 below. The allowable end bearing capacity, presented in Table 1, includes a safety factor of 1.5 or more. We anticipate that adequate pier embedment for end bearing; uplift and lateral resistance can generally be obtained within the limits stated above. If Nextel requires a deeper foundation for the tower, ADAPT can be contacted for those design parameters.

Table 1
Allowable End Bearing Capacity

Depth (feet)	Allowable Bearing Capacity (tsf)	Limiting Point Resistance (tsf)
5-15	2.2 D/B	10 TSF

Notes: D = the embedment depth (in feet) into the bearing layer.
B = pier diameter (feet).

Frictional Capacities: For frictional resistance of the drilled piers, acting both downward and in uplift, we recommend using the allowable skin friction value listed in Table 2. We recommend that frictional resistance be neglected in the uppermost 2 feet below the ground surface. The allowable skin friction value presented includes a safety factor of 1.5. The recommended friction resistance values for the soils above the bottom of the potentially liquefiable deposits have been fully discounted.

Table 2 Allowable Skin Friction Capacities	
Depth (feet)	**Allowable Skin Friction (tsf)**
0-2	0.0
2-5	0.10
5-15	0.30

Lateral Capacities: For design against lateral forces acting against the drilled pier, two methods are typically used. The parameter used to select the appropriate design method is the length to pier stiffness ratio L/T, where L is the pier length in inches, and T is the relative stiffness factor. The relative stiffness factor (T) should be computed by:

$$T = \left(\frac{EI}{n_h}\right)^{0.2}$$

where E = modulus of elasticity (psi)
I = moment of inertia (in^4)
n_h = constant of horizontal subgrade reaction (pci)

The factors E and I are governed by the internal material strength characteristics of the pier. A representative value of n_h for the soil types encountered at this site is presented below in Table 4. Piers with a L/T ratio of less than 2 may be assumed to be relatively rigid and acting as a pole. The passive pressure approach may be used for this condition. For piers with a L/T ratio greater than 2, the modulus of subgrade reaction method is typically used. Both of these methods are discussed below:

Passive Pressure Method: The passive pressure approach is conservative by neglecting the redistribution of vertical stresses and shear forces that develop near the bottom of the pier and contribute to resisting lateral loads. We recommend using the allowable passive earth pressure (expressed as equivalent fluid unit weights) listed in Table 3.

Table 3 Allowable Passive Pressures	
Depth (feet)	**Allowable Passive Pressure (pcf)**
0-2	0
2-5	280
5-15	350

The allowable passive earth pressure presented in Table 3 may be assumed to be acting over an area measuring 2 pier diameters in width by 8 pier diameters in depth, neglecting the uppermost 2 feet of embedment below the ground surface. According to the NAVFAC Design Manual 7.02 (1986), a lateral deflection equal to about 0.01 times the pier length would be required to mobilize the allowable passive pressure presented above. Higher deflections would mobilize higher passive pressures. When developing the allowable passive pressure listed in Table 3, we have incorporated a safety factor of at least 1.5, which is commonly applied to transient or seismic loading conditions.

Modulus of Subgrade Reaction Method: Using this method, the pier is designed to resist lateral loads based on acceptable lateral deflection limits. For granular soils, the coefficient of horizontal subgrade reaction (k_h) is considered to be directly proportional to the depth along the pier. The formula to determine k_h is $k_h = n_h x$, where x is the depth below the ground surface in inches. We recommend using the value for the constant of horizontal subgrade reaction (n_h) for the various soil types presented in Table 4 below.

Table 4
Constant of Horizontal Subgrade Reaction (n_h)

Depth (feet)	n_h (pci)
0-2	0
2-5	10
5-15	30

Construction Considerations: At this site, the near surface soil consisted of approximately 4.5 feet of medium stiff, dark brown silt overlying medium dense, gray, fine to coarse sand that extended to the maximum depth of boring B-1 at 36.5 feet. Groundwater was encountered at approximately 15 feet. Due to the sand encountered within the likely drilling depths for a pier foundation, the drilling contractor should be prepared to case the drilled pier excavation to prevent caving or raveling of pier shaft sidewalls. Drilling action did not indicate the presence of cobbles or boulders, but the contractor should be prepared in the event boulders/cobbles and/or hard drilling conditions are encountered. Since we recommend the drilled pier be installed above groundwater, we do not anticipate any significant groundwater seepage into the shaft. However, groundwater levels can fluctuate due to factors such as seasonal variations in precipitation, changes in site utilization, or other factors. The contractor should be prepared to pump any accumulated groundwater prior to pier concrete placement. Alternatively, the use of bentonite slurry could be utilized to stabilize the drilled pier excavation. However, this is not a preferred method due to the reduction of skin friction by the bentonite.

The drilling contractor should be prepared to clean out the bottom of the pier excavation if loose soil is observed or suspected, with or without the presence of slurry or groundwater. As a minimum, we recommend that the drilling contractor have a cleanout bucket on site to remove loose soils and/or mud from the bottom of the pier. If groundwater is present and abundant within the pier hole, we recommend that the foundation concrete be tremied from the bottom of the hole to displace the water and minimize the risk of contaminating the concrete mix. The Drilled Shaft Manual published by the Federal Highway

Administration recommends that concrete be placed by tremie methods if more than 3 inches of water has accumulated in the excavation.

Structural Fill

The following comments, recommendations, and conclusions regarding structural fill are provided for design and construction purposes:

Materials: Structural fill includes any fill materials placed under footings, pavements, driveways, and other such structures. Typical materials used for structural fill include: clean, well-graded sand and gravel (pit-run); clean sand; crushed rock; controlled-density fill (CDF); lean-mix concrete; and various soil mixtures of silt, sand, and gravel. Recycled concrete, asphalt, and glass, derived from pulverized parent materials may also be used as structural fill.

Placement and Compaction: Generally, CDF, and lean-mix concrete do not require special placement and compaction procedures. In contrast, pit-run, sand, crushed rock, soil mixtures, and recycled materials should be placed in horizontal lifts not exceeding 8 inches in loose thickness, and each lift should be thoroughly compacted with a mechanical compactor. Using the modified Proctor maximum dry density (ASTM: D-1557) as a standard, we recommend that structural fill used for various on-site applications be compacted to the following minimum densities:

Fill Application	Minimum Compaction
Slab/Footing subgrade	90 percent
Gravel drive subgrade (upper 1 foot)	95 percent
Gravel drive subgrade (below 1 foot)	90 percent

Subgrades and Testing: Regardless of location or material, all structural fill should be placed over firm, unyielding subgrade soils. We recommend that a representative from ADAPT be retained to observe the condition of subgrade soils before fill placement begins, and to perform a series of in-place density tests during soil fill placement. In this way, the adequacy of soil compaction efforts may be evaluated as earthwork progresses.

Fill Content: Soils used for structural fill should not contain individual particles greater than about 6 inches in diameter and should be free of organics, debris, and other deleterious materials. Given these prerequisites, the suitability of soils used for structural fill depends primarily on the grain-size distribution and moisture content of the soils when they are placed. When the "fines" content (that soil fraction passing the U.S. No. 200 Sieve) increases, soils become more sensitive to small changes in moisture content. Soils containing more than about 5 percent fines (by weight) cannot be consistently compacted to a firm, unyielding condition when the moisture content is more than about 2 percentage points above optimum. The existing near surface soils at this site consists of medium stiff silt, and should be considered moisture sensitive. The use of "clean" soil is necessary for fill placement during wet-weather site work, or if the in-situ moisture content of the silty site soils is too high to allow adequate compaction. Clean soils are defined as granular soils that have a fines content of less than 5 percent (by weight) based on the soil fraction passing the U.S. No.3/4-inch Sieve.

CLOSURE

The conclusions and recommendations presented in this report are based, in part, on the explorations that we performed for this study. If variations in subsurface conditions are discovered during earthwork, we may need to modify this report. The future performance and integrity of the tower foundations will depend largely on proper initial site preparation, drainage, and construction procedures. Monitoring by experienced geotechnical personnel should be considered an integral part of the construction process. We are available to provide geotechnical engineering services during the earthwork and foundation construction phases of the project. If variations in the subgrade conditions are observed at that time, we would be able to provide additional geotechnical recommendations, thus minimizing delays as the project develops. We are also available to review preliminary plans and specifications before construction begins, and to provide geotechnical inspection and testing services during construction.

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FIGURE 1 - Location/Topographic Map

Location : Eastport/Daisy (OR0205-2)
2330 SE 82nd Ave
Portland, Oregon 97216

Client : Nextel Communications

Date : 4/26/01 **Job # :** OR01-5758

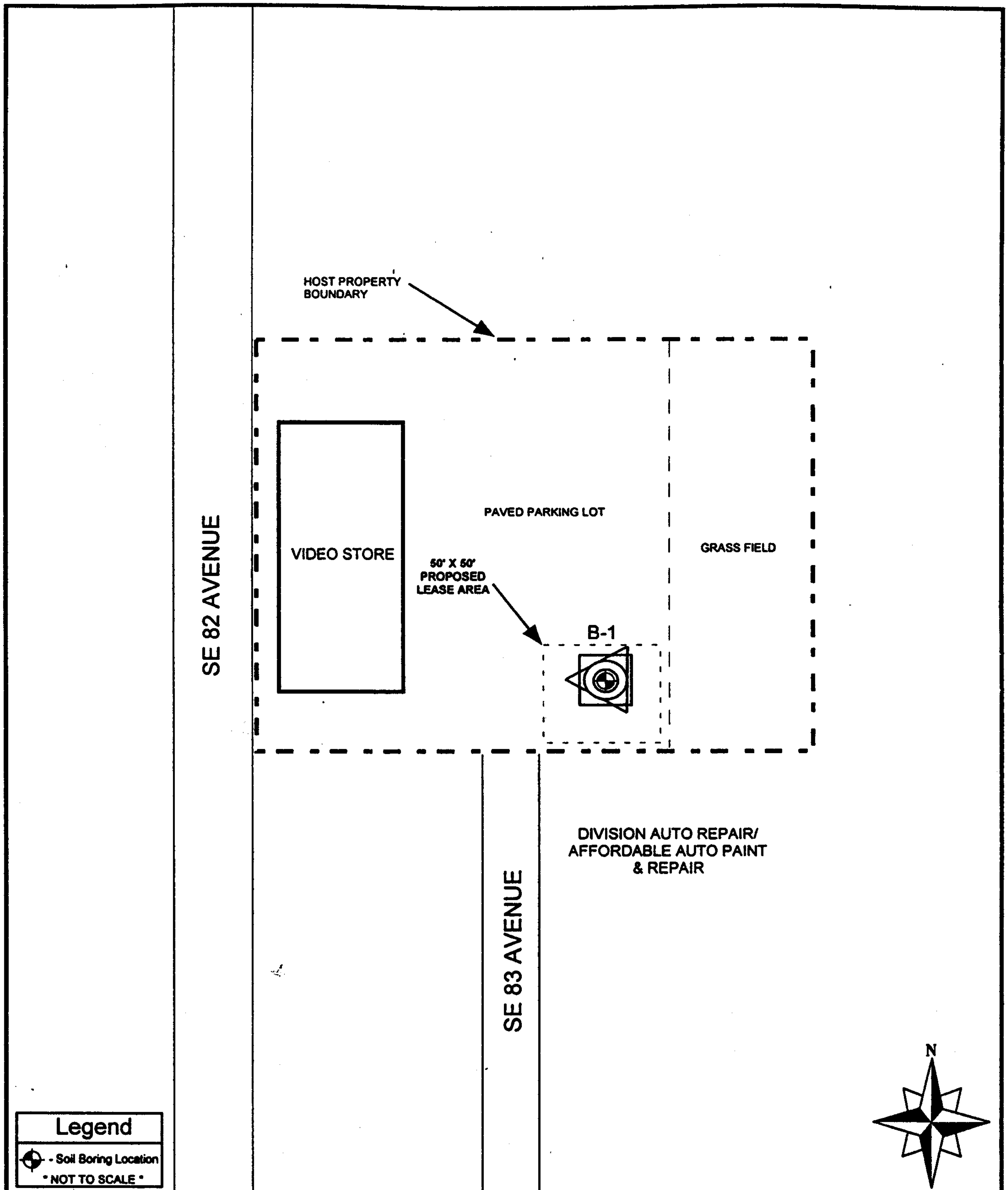

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FIGURE 2 - Site/Exploration Plan

Location : Eastport Daisy Site
2330 SE 82nd Avenue
Portland, Multnomah County, Oregon

Client : Nextel Communications, Inc.

Date : 4/26/01 **Job # :** OR01-5758

APPENDIX A

BORING LOG

BORING LOG

LSI ADAPT Inc.
17700 SW Upper Boones Ferry Rd., Suite 100
Portland, Oregon 97224
Tel: (503) 639-3413 Fax: (503) 639-4214

PROJECT : Eastport/Daisy
2330 SE 82nd Avenue
Portland, Oregon 97216

Job Number: OR01-5758
Nextel Site No.: OR0205-2

Boring No.: **B-1**

Elevation Reference : N.A.
Ground Surface Elevation : N.A.
Well Completed : N.A.
Casing Elevation : N.A.

DEPTH (feet)		SAMPLE TYPE	SAMPLE NUMBER	BLOW COUNT	OVM READING	GROUND WATER	AS-BUILT DESIGN	TESTING
0	Medium stiff, dark brown, SILT, damp							
		I		2 3 4				
5	Medium dense, gray, fine to coarse SAND, with some gravel, damp	I		6 11 11				
10	Medium dense, gray, fine to coarse SAND, damp	I		8 11 12				
15	becomes wet at 15 feet	I		4 9 9		▼		
20		I		5 12 13				
25		I		9 15 13				
30		I		11 13 14				

LEGEND

I	2-inch O.D. Split-Spoon Sample
II	1" Geoprobe
X	Sample not Recovered
▼ DATE	Static Water Level at Drilling
▽ DATE	Static Water Level
▼ (perched)	Perched Groundwater
▨	Grab Sample
(box)	Type of Analytical Testing Used
NR	No Recovery
ATD	At Time of Drilling

Page:

Drilling Start Date: 4/20/01
Drilling Completion Date: 4/20/01
Logged By: KLW

BORING LOG

LSI ADAPT Inc.
17700 SW Upper Boones Ferry Rd., Suite 100
Portland, Oregon 97224
Tel: (503) 639-3413 Fax: (503) 639-4214

PROJECT : Eastport/Daisy
2330 SE 82nd Avenue
Portland, Oregon 97216

Job Number: OR01-5758

Nextel Site No.: OR0205-2

Boring No.: **B-1**

Elevation Reference : N.A.
Ground Surface Elevation : N.A.
Well Completed : N.A.
Casing Elevation : N.A.

DEPTH (feet)		SAMPLE TYPE	SAMPLE NUMBER	BLOW COUNT	OVM READING	GROUND WATER	AS-BUILT DESIGN	TESTING
35			14					
			16					
			14					
	Boring terminated at 36.5 feet.							
40	Note: Groundwater was encountered at about 15 feet during drilling.							
45								
50								
55								
60								

LEGEND

- 2-inch O.D. Split-Spoon Sample
- 1" Geoprobe
- Sample not Recovered
- DATE — Static Water Level at Drilling
- DATE — Static Water Level
- Perched Groundwater
- Grab Sample
- Type of Analytical Testing Used
- NR — No Recovery
- ATD — At Time of Drilling

Page:

Drilling Start Date: 4/20/01 Drilling Completion Date: 4/20/01 Logged By: KLW

APPENDIX B

SOIL RESISTIVITY TEST RESULTS

Soil Resistivity Test Results

Project Name: Eastport/Daisy (OR0205-2)
ADAPT Project No.: OR01-5758
Project Address: 2330 SE 82nd Avenue in Portland, Oregon
Date: April 24, 2001
Weather: Cloudy, dry

Resistivity Survey #1

Orientation: N-S
Soil Conditions: Damp clayey silt

Distance Between Electrodes (feet)	Ohmic Meter Readings (ohms)	Calculated Resistance (ohm-cm)
2.5	150	71,810
5	29	27,770
10	16	30,640
15	13	37,340
20	10	38,300

Resistivity Survey #2

Orientation: E-W
Soil Conditions: Damp clayey silt

Distance Between Electrodes (feet)	Ohmic Meter Readings (ohms)	Calculated Resistance (ohm-cm)
2.5	51	24,420
5	20	19,150
10	9	17,240
15	5.7	16,370
20	4.2	16,090

The resistivity survey was completed using the Wenner Four-Electrode Method in general accordance with ASTM method G-57-95a (IEEE Standard 81), using a Nilsson Model 400 Soil Resistance Meter.