GEOTECHNICAL ENGINEERING REPORT
NEXTEL - OLYMPIA/ROBERTS SITE NO. WA0221-2
7601 RAINIER ROAD S.E.
OLYMPIA, WASHINGTON

Submitted to:

Nextel Communications, Inc.
c/o D. Garvey Corporation
1700 Westlake Avenue North, Suite 400
Seattle, Washington 98109

Submitted by:

AGRA Earth & Environmental, Inc.
11335 N.E. 122nd Way, Suite 100
Kirkland, Washington 98034-6918

6 November 1997

7-91M-11932-0

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

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7-91M-11932-0

NEXTEL Communications, Inc.
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1700 Westlake Avenue North, Suite 400
Seattle, Washington 98109

Attention: Mr. David Denning

Subject: **Geotechnical Engineering Report**
NEXTEL - Olympia/Roberts Site No. WA0221-2
7601 Rainier Road S.E.
Olympia, Washington

Dear David:

AGRA Earth & Environmental, Inc. (AEE) is pleased to submit this report providing the results of our recent geotechnical engineering evaluation conducted for the above-referenced project. The purpose of our evaluation was to derive conclusions and recommendations concerning site preparation, tower and equipment shelter foundations, structural fill, and other considerations. In accordance with NEXTEL Communications requirements, our scope of work included a site reconnaissance, a limited exploration, limited laboratory testing, geotechnical research, engineering analyses, and preparation of this report.

We received authorization to proceed with our evaluation on 17 October 1997. This report has been prepared for the exclusive use of NEXTEL Communications and their consultants, for specific application to this project, in accordance with generally accepted geotechnical engineering practice. We appreciate the opportunity to be of service on this project and would be happy to answer any questions you may have.

Respectfully submitted,
AGRA Earth & Environmental, Inc.

John R. Zeman
Senior Staff Engineer

Distribution: NEXTEL Communications, Inc. (3) Attn: Mr. David Schnebele
D. Garvey Corporation (1) Attn: Mr. David Denning

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Boring Log B-1

1.0 SITE AND PROJECT DESCRIPTION

The project site is addressed 7601 Rainier Road S.E. in Olympia, Washington, as shown on the enclosed *Location Map* (Figure 1). The leased parcel, which measures 45 feet-square, is part of a larger residential parcel, which is roughly rectangular and measures about 430 feet (north-south) by 660 feet (east-west), with an existing house situated towards the northeast corner. The proposed tower lease area is located at the southwest corner of the property, or about 10 feet east and about 6 feet north of the southwest property boundary. The enclosed *Site & Exploration Plan* (Figure 2) illustrates these site boundaries and adjacent existing features. Currently, the tower site is inaccessible to vehicular traffic due to densely forested land.

Development plans call for construction of a new 150-foot-tall, self-supporting, lattice tower and prefabricated equipment shelter on the site. We anticipate that site grades will remain virtually unchanged after clearing, stripping, and grubbing operations have been completed.

The conclusions and recommendations contained in this report are based on our understanding of the currently proposed project, as derived from information provided to us. If any changes are made in the project, we should be allowed to review such changes to determine whether any modifications to our report recommendations are needed.

2.0 EXPLORATORY METHODS

We explored surface and subsurface conditions at the project site on 30 October 1997. Our exploration and testing program comprised the following elements:

- A visual surface reconnaissance of the site;
- One soil boring (designated B-1) advanced at a strategic location on the site, to a depth of about 33 feet.
- A review of published geologic maps and seismologic literature.

The location and depth of our exploration was selected in relation to the proposed site features, under the constraints of budget and site access. We determined the location of our exploration by measuring from existing site features with a hand-held measuring tape and scaling these measurements onto a layout plan provided to us. Exploration elevations were estimated by interpolating between contour lines shown on this same plan.

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Consequently, the location and elevation data presented in this report should be considered accurate only to the degree permitted by our data sources and implied by our measuring methods.

It should be realized that our exploration revealed subsurface conditions only at a discrete location near the project site and that actual conditions could vary at the actual tower location. Furthermore, the nature and extent of any such variations will not become evident until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations to reflect actual conditions.

2.1 Soil Boring Procedures

Our exploratory boring was advanced with a hollow-stem auger, using a truck-mounted drill rig operated by an independent drilling company working under subcontract to AEE. A geotechnical engineer from our firm continuously observed the boring, logged the subsurface conditions, and collected representative soil samples. All samples were stored in watertight containers and later transported to our laboratory for further visual examination and testing, as deemed necessary. After each boring was completed, the borehole was backfilled with a mixture of bentonite chips and soil cuttings, and the surface was patched with asphalt or concrete (where appropriate).

Throughout the drilling operation, soil samples were obtained at 2½- to 5-foot depth intervals by means of the Standard Penetration Test (SPT) per ASTM:D-1586. This testing and sampling procedure consists of driving a standard 2-inch-diameter steel split-spoon sampler 18 inches into the soil with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "SPT blow count." If a total of 50 blows is struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed *Boring Log* describes the vertical sequence of soils and materials encountered in each boring, based primarily on our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradational, our log indicates the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our log also graphically indicates the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the borings, as well as any laboratory tests performed on these soil samples. If any groundwater was encountered in a borehole, the approximate groundwater depth is depicted on the boring log. Groundwater depth estimates are

typically based on the moisture content of soil samples, the wetted height on the drilling rods, and the water level measured in the borehole after the auger has been extracted.

3.0 SITE CONDITIONS

The following sections of text present our observations, measurements, findings, and interpretations regarding surface, soil, groundwater, and seismic conditions at the project site. Interpretive logs of our subsurface explorations and graphic results of our laboratory tests are enclosed with this report.

3.1 Surface Conditions

The project site is located in a region characterized by relatively flat terrain with scattered residential development. The proposed tower lease area is situated on a knoll which extends about 40 to 50 feet above the surrounding area. Topography at the site slopes gently to moderately downward in all directions. Vegetation consists of moderately dense coniferous and deciduous tree growth and a typical understory. No ponding or significant amounts surface water were observed at the project site.

3.2 Soil Conditions

According to published geologic maps, soil conditions in the site vicinity are characterized by glacially deposited soils, which our on-site exploration confirmed. Specifically, the soil profile observed in boring B-1 consisted of a medium dense, wet, silty sand with some gravel extending to a depth of 2½ feet. Below this depth, our boring encountered a very dense, moist, gravelly, silty sand, which extended to the maximum depth explored (approximately 33 feet). Although not encountered in our boring, large boulders (sometimes called glacial erratics) are commonly associated with glacially derived soils and may be encountered on a random basis.

3.3 Groundwater Conditions

At the time of exploration (November 1997), groundwater was not encountered within our boring; however, it should be noted that groundwater levels fluctuate and may vary in response to season, precipitation patterns, on- or off-site construction activities, site utilization, and other factors.

3.4 Seismic Conditions

According to the *Seismic Zone Map of the United States* contained in the 1994 *Uniform Building Code*, the project site lies within seismic risk zone 3. Based on soil conditions encountered at the site, we interpret the subsurface site conditions to correspond to a seismic soil profile type S-2, as defined by Table 16-J of the 1994 *Uniform Building Code*. Soil profile type S-2 applies to a profile consisting of predominantly medium-dense to dense or medium-stiff to stiff soil which exceeds 200 feet in depth.

4.0 CONCLUSIONS AND RECOMMENDATIONS

The proposed construction is feasible from a geotechnical perspective, given the subsurface conditions disclosed in our field exploration. Our specific recommendations concerning site preparation, tower and equipment shelter foundations, and structural fill placement are presented in the following sections, along with preliminary conclusions regarding environmental conditions.

4.1 Site Preparation

Site preparation will involve temporary drainage, clearing and grubbing, preparing subgrades, and other construction activities. The following comments and recommendations apply to site preparation.

Temporary Drainage: We recommend that initial site preparation involve intercepting and diverting any potential sources of surface or near-surface water within the construction zones. Because the selection of an appropriate drainage system will depend on the water quantity, season, weather conditions, construction sequence, and contractor's methods, final decisions regarding drainage systems are best made in the field at the time of construction. Nonetheless, we anticipate that berms or ditches placed along the uphill side of the work areas will adequately intercept surface water runoff.

Clearing and Grubbing: Once surface runoff is controlled, site preparation should include clearing, grubbing, and removal of all vegetation, sod, topsoil, any organic-rich soils from foundation and fill areas. Based on the observed soil conditions, we estimate the stripping depths needed to remove vegetation and topsoil to range from approximately 1 to 3 inches, with localized deeper stripping required for removal of root balls.

Subgrade Protection: Because of the high silt content of the near-surface site soils, these soils are moisture-sensitive and prone to disturbance when wet. To reduce site disturbance, the contractor should minimize traffic above prepared subgrades. 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 then be recompacted, if necessary, prior to placing subsequent lifts of structural fill or foundation components.

4.2 Equipment Shelter Foundations

We anticipate that the prefabricated equipment shelter will be placed on shallow concrete footings and that bearing pressures will be relatively low. Our specific conclusions and recommendations are presented below.

Subgrade Conditions: Footing subgrades should consist of firm, unyielding structural fill or granular native soils. Footings should never be cast atop soft, loose, organic, or frozen soils, nor atop subgrades covered by standing water. An AEE representative should be allowed to observe all subgrades before any concrete is poured and to verify that they have been adequately prepared.

Footing Depths: For frost and erosion protection, all exterior footings should be embedded at least 18 inches below exterior finish grades.

Bearing Capacities: We recommend utilizing a maximum allowable bearing capacity of 1,500 pounds per square foot (psf) for equipment building foundations supported on the medium-dense to very dense, silty sands. This allowable bearing capacity may be increased by one-third to resist short-term transient loads such as wind and seismic forces.

Lateral Resistance: Lateral forces 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 value of 0.35 would be appropriate, in our opinion. An allowable passive earth pressure of 250 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: With the footing subgrade areas prepared as described above, we would not expect settlements greater than 1 inch. Settlements may be greater than this if loose subgrade soils are left within footing areas. On the other hand, we would expect less settlement if the actual foundation loads are less than the allowable maximum soil bearing pressure given above.

4.3 Tower Foundations

The foundation support for a self-supporting communications tower typically consists of either drilled piers or a mat footing, in order to provide adequate resistance to vertical, horizontal and overturning forces. In our opinion, drilled piers and mat footings are both feasible foundation options for this site. Our specific recommendations concerning design and construction of drilled piers and mat footings are presented in the following subsections.

4.3.1 Drilled Piers

For the purposes of design, we have extrapolated geotechnical design information beyond the refusal depth in our boring (approximately 33 feet), based on our experience with similar soils and our knowledge of soil conditions in the vicinity of the site. We

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recommend that AEE be present in the field during construction to confirm the extrapolated soil conditions. The following comments and recommendations are provided for drilled pier design and construction purposes.

Design Values: Table 1, below, summarizes the soil values utilized in our geotechnical engineering analysis for the soil layers encountered at this site. We derived these values from our explorations and field testing, as well as from laboratory testing performed on similar materials and established correlations for soil properties. These values are presented for background information only; the use of these soil values in cursory or supplemental analyses should be by knowledgeable geotechnical engineers only and should incorporate physical factors that may influence the allowable soil reaction for known loading conditions.

TABLE 1
SUMMARY OF SOIL VALUES FOR DESIGN OF DRILLED PIERS

Depth Interval (feet)	In-Place Density (pcf)	Internal Friction Angle (degrees)	Cohesion (psf)
0 to 3	128	32	0
3 to 18	134	35	0
18 to 40	138	38	0

Compressive Capacity: We recommend that the drilled pier penetrate at least 10 feet below the ground surface. Table 2 presents our recommended allowable compressive capacities for drilled piers of varying diameter and depth. These values incorporate a safety factor of 1.5 and include reductions for the dead weight of the pier; as such, they represent groundline compressive capacities. A linear interpolation may be used between each depth interval to obtain the allowable capacity for that specific depth.

TABLE 2 RECOMMENDED ALLOWABLE COMPRESSIVE CAPACITIES (TONS)					
Tip Depth (feet)	Drilled Pier Diameter (feet)				
	4	5	6	7	8
10	85	132	489	256	333
15	185	286	410	555	723
20	324	502	718	974	1,268
25	411	636	909	1,232	1,603
30	500	772	1,103	1,493	1,941
35	591	911	1,300	1,757	2,283
40	684	1,052	1,498	2,024	2,628

Uplift Capacity: Our recommended uplift capacities are presented in Table 3, below. These values incorporate a safety factor of 1.5 and include the dead weight of the pier; as such, they represent groundline uplift capacities. A linear interpolation may be used between each depth interval to obtain the allowable capacity for specific depths not shown.

TABLE 3 RECOMMENDED ALLOWABLE UPLIFT CAPACITIES (TONS)					
Tip Depth (feet)	Drilled Pier Diameter (feet)				
	4	5	6	7	8
10	7	10	14	18	22
15	13	18	24	31	38
20	21	29	37	47	57
25	30	41	53	65	79
30	41	55	70	87	105
35	54	72	91	112	134
40	68	90	114	139	167

Lateral Resistance: Drilled pier foundations for communication towers are typically rigid and act as a pole which rotates around a fixed point at depth. Although more complex and detailed analyses are available, either the simplified passive earth pressure method or the subgrade reaction method is typically used to determine the pier diameter and depth required to resist groundline reaction forces and moments. These methods are described below.

- *Passive Earth Pressure Method*: The passive earth pressure method is slightly conservative by neglecting the redistribution of shear forces that develop near the bottom of the pier. Our recommended passive earth pressures for the soil layers encountered at this site are presented in Table 4. These values are expressed as an equivalent fluid unit weight to reflect the linear increase with depth and may be assumed to act over an area measuring two pier diameters wide by eight pier diameters deep. A lateral deflection at the ground surface equal to about 0.002 times the pier length would be required to mobilize the full allowable passive pressure presented below. The values listed in Table 4 incorporate a safety factor of at least 1.5, which is commonly applied to transient or seismic loading conditions.

TABLE 4
RECOMMENDED ALLOWABLE PASSIVE EARTH PRESSURES

Depth Interval (feet)	Passive Pressure (pcf)
0 to 3	0
3 to 18	275
10 to 18	350
18 to 40	400

- *Subgrade Reaction Method*: The subgrade reaction method is slightly more complex than the passive pressure method and is typically used to compute lateral design loads based on allowable lateral deflections. Using this method, the soil reaction pressure (p) on the face of the pier is related to the lateral displacement (y) of the pier and the coefficient of horizontal subgrade reaction (k_h); this relationship is expressed as $p = k_h y$. The calculation of k_h depends primarily on the constant of horizontal subgrade reaction, the pier diameter, and the soil type as detailed below:

  - SAND and Soft CLAY: For soil layers consisting of sand or soft clay deposits, the coefficient of horizontal subgrade reaction (k_h) increases linearly with depth and is derived from the constant of horizontal subgrade reaction (n_h) multiplied by the depth (z) below the ground surface and divided by the pier diameter (B); expressed as $k_h = n_h(z/B)$.

  - Stiff CLAY: For soil layers consisting of stiff clay deposits, the coefficient of horizontal subgrade reaction (k_s) remains uniform with

depth and is considered directly proportional to the coefficient of vertical subgrade reaction (k_s) divided by 1½ pier diameters (B); expressed as $k_h = k_s/(1.5B)$.

Our recommended constants of horizontal subgrade reaction (n_h and k_s) for the soil layers encountered at this site are presented in Table 5, below. These values do not include a factor of safety since they model the relationship between contact pressure and displacement. Therefore, the structural engineer or tower manufacturer should select an appropriate allowable displacement for design. In any case, we recommend that the maximum allowable displacement not exceed 2 inches at the groundline.

TABLE 5
RECOMMENDED HORIZONTAL SUBGRADE REACTION VALUES

Depth Interval (feet)	n_h (pci)	k_s (pci)
0 to 3	90	N/A
3 to 18	95	N/A
18 to 40	100	N/A
Coefficient of Horizontal Subgrade Reaction (pci)	**$k_h = n_h(z/B)$ (Sand & Soft Clay)**	**$k_h = k_s/(1.5B)$ (Stiff Clay)**

Settlements: For the given pier diameters, we estimate that total post-construction settlements of properly designed and constructed drilled piers could approach 1 inch. Differential settlements between foundation elements may approach approximately half of the total settlement.

Construction Considerations: Construction activities for drilled piers could encounter caving sidewalls, groundwater, and other physical difficulties. We offer the following comments and recommendations regarding these issues.

Excavation Stability: The foundation drilling contractor should be prepared to case all or part of the excavation to prevent caving of the sidewalls. If alternative methods of stabilizing the sidewalls are proposed, these should be reviewed and accepted by the owner, or their representatives, prior to installation.

- Borehole Preparation: The foundation drilling contractor should be prepared to clean out the bottom of the borehole if loose soil is observed or suspected. We recommend that the drilling contractor have a cleanout bucket on site to remove loose soils and/or mud from the bottom of the borehole. Personnel entry into the borehole is not recommended.

- Wet Excavation: Coagulant admixtures used during drilling saturated or wet soils should consist of materials that will not deposit deleterious coatings on reinforcing steel. Specifically, we recommend synthetic-based coagulants.

- Concreting: Concrete should be placed by tremie methods if more than 3 inches of water has accumulated in the excavation. For concrete placement under water, we recommend that the foundation concrete be tremied from the bottom of the hole to displace accumulated water and reduce the risk of adversely impacting the concrete mix.

4.3.2 Mat Footings

A reinforced concrete mat footing could be used for tower foundation support. The following recommendations and comments are provided for mat footing design and construction purposes.

Subgrade Conditions: Footing subgrades should be prepared according to the *Site Preparation* section of this report. Footings should never be cast atop soft, loose, organic, or frozen soils, nor atop subgrades covered by standing water. An AEE representative should be retained to observe all subgrades before any concrete is poured and to verify that they have been adequately prepared.

Footing Embedments: Based on our exploration, we anticipate that suitable bearing soils would be encountered at a depth of 5 feet below the existing ground surface, which would provide adequate embedment for frost protection.

Bearing Capacities: Footings bearing directly on the native, very dense, gravelly, silty sands and embedded a minimum depth of 5 feet may be designed for a maximum allowable bearing capacity of 5,000 pounds per square foot (psf). This bearing capacity incorporates a static safety factor of at least 2.0 and can be increased by one-third for transient loads.

Uplift Resistance: We anticipate that uplift loads will be resisted by the dead weight of the mat foundation, as well as the soil materials covering the mat. Native soils used to cover the mat, if compacted to a minimum of 90 percent of the Modified Proctor maximum dry density, may be assumed to have a unit density of 120 pcf.

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 value of 0.45 and an allowable passive earth pressure of 250 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 will not exceed 1 inch, with differential settlements approaching one-half of the total.

4.4 Structural Fill

The following comments, recommendations, and conclusions regarding the use of structural fill are provided for design and construction purposes.

Materials: "Structural fill" includes any fill materials placed under footings, pavements, driveways, and other permanent 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 derived from crushed parent material is also useful as structural fill.

Soil Composition: 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. As the "fines" content (that portion 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 use of "clean" soil is necessary for fill placement during wet-weather site work. 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. 4 Sieve.

On-Site Soils: The existing gravelly, silty sands with varying amounts of gravel encountered throughout the entire depth explored at the site, appear suitable for reuse as structural fill, provided that they are free of organics or other deleterious material and that the moisture content is near optimum moisture.

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, on-site soils, and recycled materials should be placed in horizontal lifts not

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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 minimum densities presented in Table 6.

TABLE 6
RECOMMENDED COMPACTION REQUIREMENTS

Fill Application	Minimum Compaction (ASTM:D-1557)
Wall and footing backfill	90 percent
Footing subgrade	90 percent
Gravel driveway subgrade (upper 1 foot)	95 percent
Gravel driveway subgrade (below 1 foot depth)	90 percent

Subgrades and Testing: Regardless of location or material type, all structural fill should be placed over firm, unyielding subgrades. We recommend that an AEE representative be allowed to observe all subgrades before fill placement begins, and to perform a series of in-place density tests during structural fill placement. In this way, the adequacy of soil compaction efforts may be evaluated as earthwork progresses, to minimize construction delays.

5.0 CLOSURE

The conclusions and recommendations presented in this report are based, in part, on the explorations that we performed for this study; therefore, if variations in the subgrade conditions are observed at a later time, we may need to modify this report to reflect those changes. AEE is available to provide geotechnical monitoring, soils and concrete testing, steel and masonry inspection, and other services throughout construction.

We appreciate the opportunity to be of service on this project. If you have any questions regarding this report or any aspects of the project, please feel free to contact our office.

Respectfully submitted,

AGRA Earth & Environmental, Inc.

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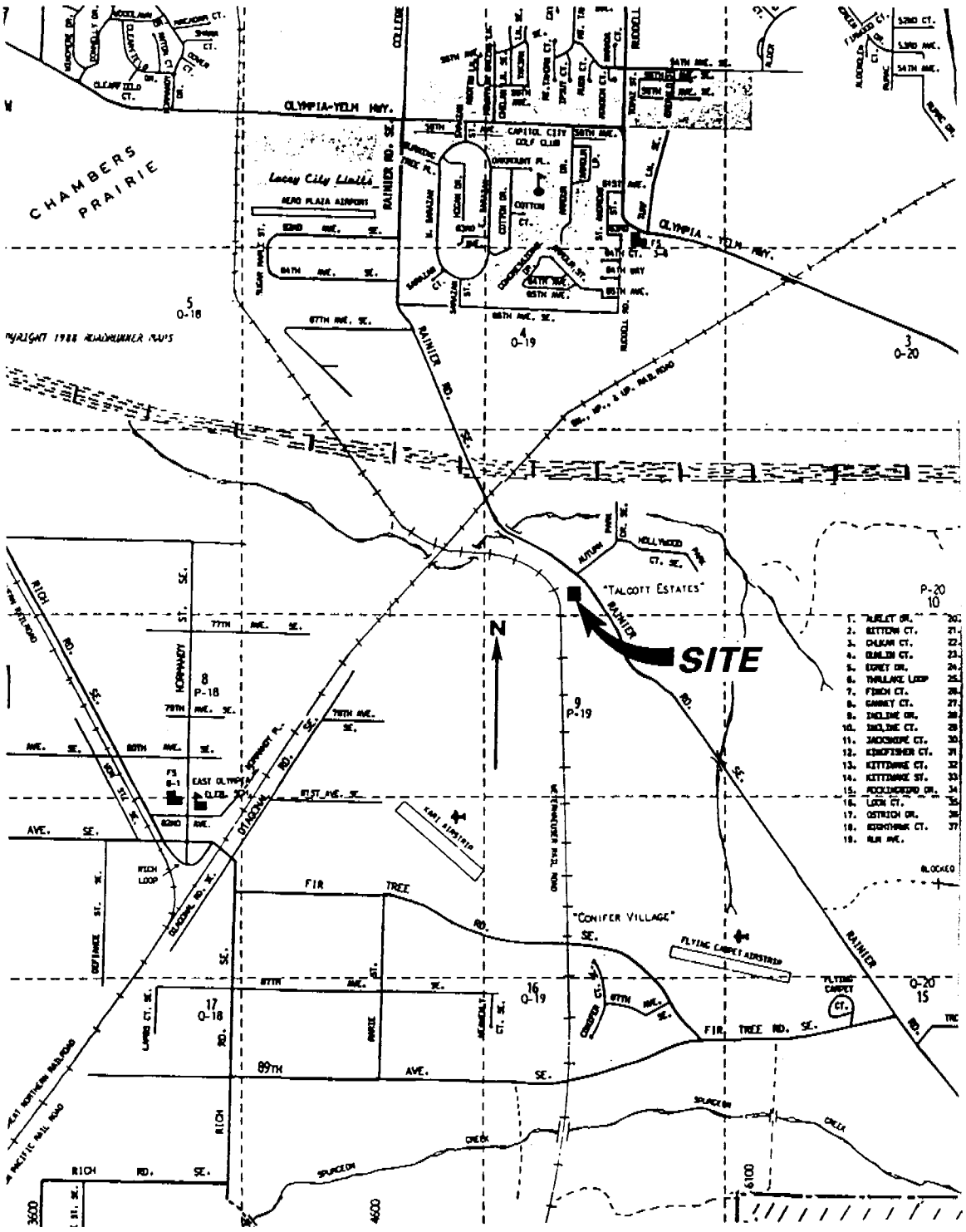

N.T.S.

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LOCATION MAP

NEXTEL-EAST OLYMPIA/ROBERTS - WA0221-2

7601 RAINIER ROAD SE
OLYMPIA, WASHINGTON

FIGURE
1

JOB NO.: 7-91W-11932-0 | DWG DATE: 11-04-97 | SCALE: N.T.S. | DESIGN BY: JRZ | FILE NAME: LOCATION.DWG

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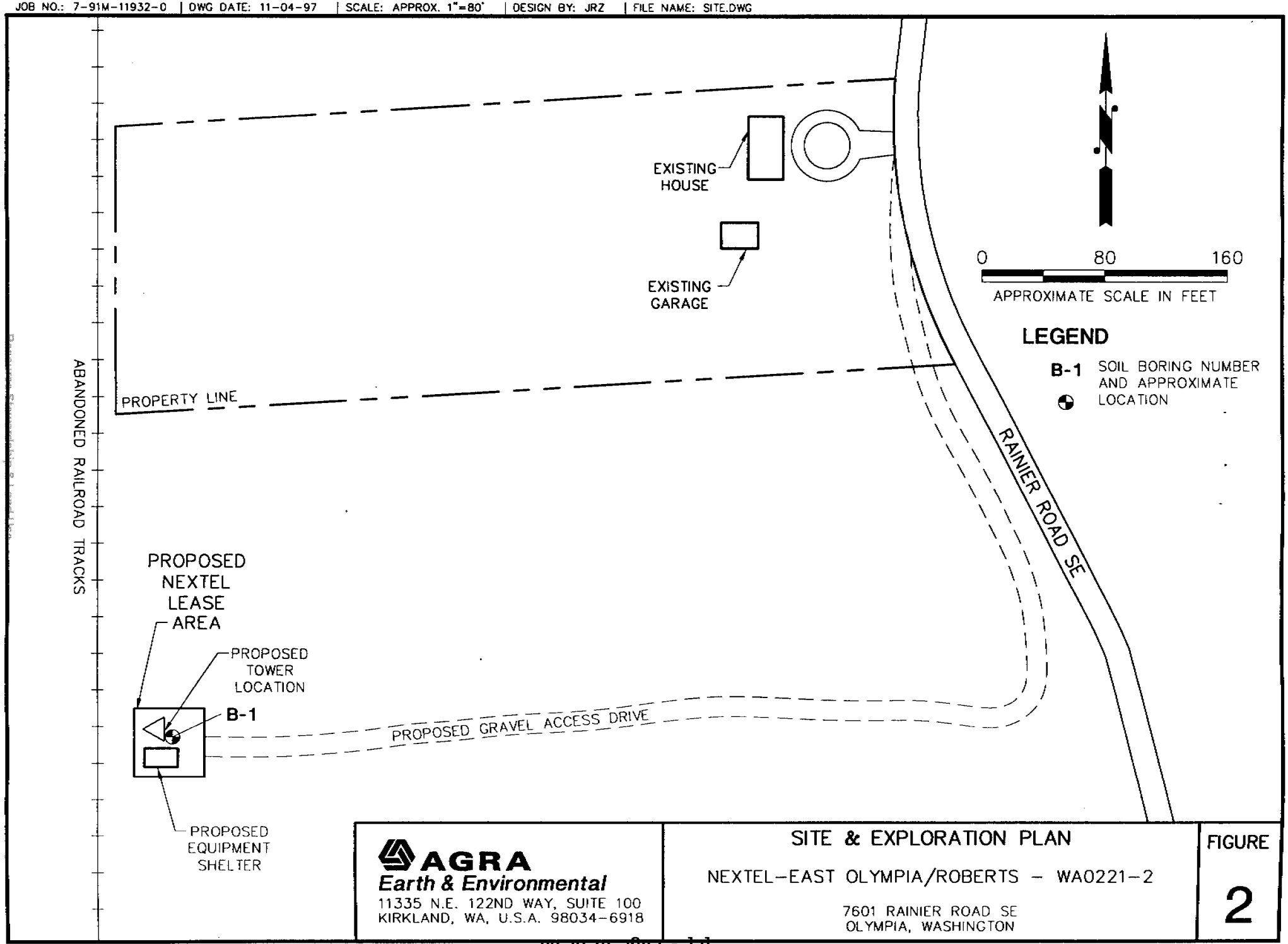
JOB NO.: 7-91M-11932-0 | DWG DATE: 11-04-97 | SCALE: APPROX. 1"=80' | DESIGN BY: JRZ | FILE NAME: SITE.DWG



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SITE & EXPLORATION PLAN

NEXTEL-EAST OLYMPIA/ROBERTS - WA0221-2

7601 RAINIER ROAD SE
OLYMPIA, WASHINGTON

FIGURE 2

PROJECT: NEXTEL - East Olympia/Roberts - WA0221-2 W.O. 7-91M-11932-0 BORING NO. B-1

DEPTH (feet)	SOIL DESCRIPTION Location: Forested area Approximate ground surface elevation: Unknown	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	PENETRATION RESISTANCE (▲ Standard, △ Other; Blows per foot)	TESTING
0	Medium dense, saturated, gray, silty SAND with some gravel			N/E		
	Very dense. moist, gray, silty SAND with some gravel		S-1		Blowcount Overstated 50/4"	
5	Very dense, moist, gray, gravelly, silty SAND (Glacial Till)					
			S-2		50/3"	
10						
			S-3		50/4"	
15						
			S-4		50/6"	
20						
			S-5		50/5.5"	
25						
			S-6		50/3"	
30	(continued)					

Penetration resistance scale: 0, 10, 20, 30, 40, 50

MOISTURE CONTENT: 0, 20, 40, 60, 80, 100 — Plastic limit / Natural / Liquid limit

LEGEND

I 2.00-inch O.D. split spoon sample

N/E No groundwater encountered

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Drilling method: HSA Hammer type: Cathead Date drilled: 30 October 1997 Logged by: JRZ

PROJECT: NEXTEL - East Olympia/Roberts - WA0221-2 W.O. 7-91M-11932-0 BORING NO. B-1

DEPTH (feet)	SOIL DESCRIPTION Location: Forested area Approximate ground surface elevation: Unknown	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	PENETRATION RESISTANCE ▲ Standard △ Other Blows per foot 0 10 20 30 40 50	Page 2 of 2 TESTING
30	Gravelly, silty SAND (Glacial Till) as above					
			S-7		50/3"	
	Boring terminated at approximately 33 feet					
35						
40						
45						
50						
55						
60	(continued)					

LEGEND

I 2.00-inch O.D. split spoon sample

N/E No groundwater encountered

MOISTURE CONTENT

0 20 40 60 80 100

Plastic limit Natural Liquid limit

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Drilling method: HSA Hammer type: Cathead Date drilled: 30 October 1997 Logged by: JRZ

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