GEOPACIFIC ENGINEERING, INC.

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

1/22/14

1409 SW Dickinson Ln

1S1E33CA00902

01-14072 RS

GT_003655

01-140172-RS

June 20, 2001

Project No. 01-7333

Matt Crino
65225 E. Timberline Drive
Rhododendron, Oregon 97049
Fax: 503-622-2638

Subject: Geotechnical Investigation
Meadowview II Development
1409 S.W. Dickinson Lane
Portland, Oregon

This report presents the results of a geotechnical investigation conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above referenced Lot. The purpose of our investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site grading, foundation design, and construction of one single-family residence. This geotechnical study was performed in accordance with your verbal authorization, dated June 12, 2001.

BACKGROUND INFORMATION

Previous Reports

The following reports and investigations were reviewed for information to assist in preparation of this Geotechnical Investigation Report:

1) Subsurface Investigation for Meadowview Subdivision, Portland, Oregon, Land Development Consultants, Inc., May 6, 1991.
2) Site Reconnaissance for Meadowview Subsivision II, Portland, Oregon, Land Development Consultants, Inc, November 12, 1991.
3) Meadowview Subdivision, Permit #91-103766, Squier Associates, May 27, 1993.
4) Site Geologic Hazards Study, Proposed Meadowview Subdivision Phase II, Squier Associates, August 16, 1996.

Project Information

Location:	North side of S.W. Dickinson Lane, just east of S.W. 17th Avenue, off Stephenson Road in Portland, Oregon (see Figure 1).
Developer:	Crino Custom Homes – Fax: 503-622-2638 (see above address)
Jurisdictional Agency:	Portland, Oregon

Site Description And Proposed Development

The site is located in Meadowview II development, which is immediately west of Meadowview Subdivision. It consists of Tax Lot 902, approximately 7,761 ft^2 in area. The lot is located on the north side of S.W. Dickinson Lane at address 1409. Existing homes are located north and east of the site. A relatively small Tract D is adjacent to the site on the west. We understand that Tract D is to remain undisturbed, with natural vegetation.

The site topography rises steeply from S.W. Dickinson Lane for a distance of approximately 30 feet, with an estimated grade of approximately 30 percent. Northward from that point, the topography becomes more gently with a grade that averages between 5 to 10 percent. Most of the site, with the exception of the southern portion along the street, is covered by a dense growth of blackberry bushes. A small cherry tree is located in the northeast portion of the site. A loosely constructed rockery wall is present along the southern site boundary, adjacent to the sidewalk on the north side of S.W. Dickinson Lane. This wall has a height of about 5.5 feet and a slope of about 7 feet. The boulders used to construct the wall appear to be residual boulders of Boring Lava that were likely encountered in cuts during construction of S.W. Dickinson Lane.

REGIONAL AND LOCAL GEOLOGIC SETTING

The subject site is located on the eastern flank of Mt. Sylvania, a Boring Lava volcanic vent that was active beginning in Late Pliocene time and continuing into Pleistocene time, perhaps between some 1 to 3 million years ago. Subsequently, catastrophic flood deposits and eolian silt (loess) were deposited in the site region. These thin deposits appear to be largely absent from the site as a result of erosion.

SUBSURFACE CONDITIONS

Our site-specific exploration for this report was conducted on June 12, 2001. Two hand auger borings were drilled and logged to depths ranging between 36 and 94 inches. Hand auger drilling was required because of site access difficulties. No access was possible through adjacent properties, and access from the street was not possible without extensive ramping through or over the rockery wall. The subsurface conditions obtained from the hand auger borings, and from previous reports, will be verified by additional field observations during early stages of on-site construction. Exploration details and logs of hand auger borings are presented in Appendix A. The following report sections summarize subsurface conditions anticipated at the site, based on our exploration program and review of available engineering reports.

SUBSURFACE EXPLORATIONS

On June 12, 2001, two hand auger borings were drilled and logged on the subject property by a GeoPacific Geologist to depths of 36 and 94 inches at the approximate locations shown on Figure 2. The borings were logged with regard to soil type, moisture content, relative strength, and groundwater, and are presented in report.

Soils

On-site native soils consist of soil units as described below.

Topsoil: Between 6 and 9 inches of topsoil was encountered in the hand auger borings. It typically consisted of brown silt with a trace of clay and little organic debris.

Loess: Although this unit is mapped on higher elevations in the site region, it was found at the site only in thin surface deposits that appear to be influenced by erosion and re-deposition. At depths of generally less than 2.5 feet, the loess changes to colluvial soil.

Colluvial Soil: The soils immediately below topsoil and loess consist primarily of clayey silt colluvium that weathered from nearby slopes and was transported from its source by runoff of precipitation and gravity. These soils typically contain numerous inclusions of completely weathered basalt fragments that increase in size and abundance with depth. These soils are generally very stiff below about 3 feet depth. Colluvial soil is typically mottled brown and rust clayey silt with some black mineral stains. It was not possible to measure colluvial soil densities during drilling of the hand auger borings. However, observations in a cut bank in the eastern portion of the site, adjacent to the adjacent residence, indicated the following soil consistencies for colluvium shown in Table 1:

Table 1 – Soil Strength Measurements by Pocket Penetrometer

Depth in Feet	Relative Strength (tons/ft^2)
1	1.5
2	3.5
3	3.75
4	>4.5

Residual Soil – Weathered Rock: Residual soil is the result of in situ weathering of a formerly massive rock body. This change from rock to soil occurs very slowly and without lateral movement. At the site, the parent rock mass was Boring Lava. Features of the rock mass such as fractures, gaseous voids, and mineral stains on fracture surfaces are often preserved as residual features in the soil. These soils are generally very stiff, yet can be readily excavated.

Rock

Rock was encountered at 94 inches depth in Hand Auger Boring #1 and at 36 inches in Hand Auger Boring #2. A previous report for the Meadowview Subdivision contained a log for test pit TP-5 located in S.W. Dickinson Lane just east of the Meadowview II development. This test pit indicated hard basalt rock at a depth of 9 feet. Since the street was subsequently constructed on a cut, we assume that the top of rock may likely be near street grade. Observations presented in a geologic hazard report (August 16, 1996) indicated that a proposed 2H:1V cut on the north side of S.W. Dickinson Lane will most likely contain boulders. These previous observations appear to verify that the few boulders observed at the ground surface above the rockery wall appear to be natural exposures of rock, and are not related to the wall construction.

Soil Moisture and Groundwater

Groundwater was not encountered in the hand auger borings; however, soil moisture did increase slightly with depth.

SEISMIC SETTING

At least three major fault zones capable of generating damaging earthquakes are known to exist in the region. These include the Portland Hills Fault Zone, Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone, as discussed below.

Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The fault zone extends along the eastern margin of the Portland Hills for a distance of 25 miles, and lies about 2 miles east of the subject site. Geomorphic lineaments suggestive of Pleistocene deformation have been identified within the fault zone, but none of the fault segments have been shown to cut Holocene (last 10,000 years) deposits (Balsillie and Benson, 1971: Cornforth and Geomatrix Consultants, 1992). No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is judged to be potentially active (Geomatrix Consultants, 1995).

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies about 18 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A recent geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek or Newberg Faults; however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner, et al. 1992; Geomatrix Consultants, 1995).

Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). Very little seismicity has occurred on the plate interface in historic time, and as a result, the seismic potential of the Cascadia Subduction Zone is a subject of scientific controversy. The lack of seismicity may be interpreted as a period of quiescent stress buildup between large magnitude earthquakes or as being characteristic of the long-term behavior of the subduction zone. A growing body of geologic evidence, however, strongly suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred siesmogenic portion of the plate interface lies roughly 50 miles west of the Oregon coast and 20 to 40 miles below the ocean surface.

SLOPE STABILITY

Site grades appear to range between about 5 and 30 percent , with only about the southern 30 to 35 feet of the site having a grade of 30 percent. Description of the Meadowview II site in November of 1991, indicated a surface zone of perched groundwater in the general area but no seeps or springs were recognized, and no evidence of slope failure was presented in the previous reconnaissance of the area. GeoPacific also noted no evidence of slope failures on the subject site or on adjacent properties during our investigation. In our opinion, no evidence for large-scale sliding at this site was found. The potential for natural slope instability appears very unlikely due to high strength colluvial and residual soils that underlie the area.

CONCLUSIONS AND RECOMMENDATIONS

Our investigation indicates that construction of the proposed residence is geotechnically feasible provided that the following recommendations are incorporated in the design and construction phases of the project. The following report sections present conclusions and recommendations regarding site preparation, grading, foundations, drainage, excavating conditions and trench backfill, seismic design, and erosion control considerations.

Site Preparation

All areas to be graded should first be cleared of debris, trees, stumps, vegetation, etc., and all debris from clearing and organic-rich topsoil should be removed from construction areas of the site. We anticipate that a stripping depth of 6 inches will be necessary to remove organic topsoil. Deeper stripping, or tilling and root-picking, to depths of 1 to 2 feet may be necessary to remove any large tree roots.

Grading

Based on the site topography and planned construction of a single-family home, we anticipate that maximum cuts of 10 to 12 feet or more may be required for the foundation excavations, driveway, and garage space under the proposed house. Most of the site grading will involve cutting. Excess soils should be hauled off site. Fills will include minimal backfill around the house foundations and garage/ basement retaining walls and in utility trenching. Significant grading should be performed as engineered grading in accordance with Appendix Chapter 33 of the 1997 Uniform Building Code (UBC) with the exceptions and additions noted herein. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Engineered fill may consist of suitable on-site soils or imported material. Imported fill material must be approved by the geotechnical engineer prior to its arrival on site.

Engineered fill should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95% of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. Engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor or other selected owners representative be held contractually responsible for test scheduling, frequency, and for all distinct fill areas.

Earthwork is usually performed in the summer months, generally mid-June to mid-October, when warm dry weather facilitates proper moisture conditioning of soils. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact fill to the recommended engineering specifications.

Foundations

The subject site is suitable for foundations bearing on stiff, native soil or engineered fill. Foundation design, construction, and setback requirements should conform to Chapter 18 of the UBC and Oregon

Structural Specialty Code (OSSC). For protection against frost heave, spread footings should be embedded at a minimum depth of 18 inches below exterior grade. The recommended minimum widths for continuous wall footings are presented in Table 2.

Table 2 - Recommended Minimum Width of Continuous Spread Footings

Number of Stories	Minimum Width of Continuous Spread Footings
1-Story	12 inches
2-Story	15 inches
3-Story	18 inches

The recommended allowable soil bearing pressure is 2,000 lbs/ft^2 for footings on stiff, native soil and engineered fill. A maximum chimney and column load of 50 kips is recommended for the site. For heavier loads, the geotechnical engineer should be consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.5, including a factor of safety of 1.5. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ½ inch over a span of 20 feet, respectively.

The above recommendations apply to foundations constructed under dry weather conditions. Due to the moisture sensitivity of on-site native colluvial soils, foundations constructed during the wet weather season may require overexcavation of footings and backfill with up to 12 inches of compacted, crushed aggregate.

Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings. Footings located on slopes or near retaining walls will require to be embedded deeper or lowered. These footings should be reviewed by a geotechnical engineer or engineering geologist.

Retaining Walls

We understand the daylight basement and garage walls will be designed as retaining structures to support cuts into the sloping hillside. The following discussion is appropriate for the proposed daylight basement walls. In the event rockery or concrete block walls are planned along the driveway or other portions of the site, we will be happy to prepare a suitable addendum letter to this report.

The average allowable bearing pressure for retaining walls may be taken as 2,000 lbs/ft^2 with a maximum allowable toe pressure of 2,500 lbs/ft^2. The coefficient of friction between native soil or engineered granular fill and poured-in-place concrete may be taken as 0.5 including a factor-of-safety of 1.5.

Recommended lateral soil pressures for design of permanent retaining structures with adequate drainage can be calculated using the equivalent fluid unit weights provided in Table 1. The effect of surcharges or live loads on lateral pressures has not been included. Drainage should be such that no hydrostatic pressures are realized behind the walls. The unit weights in Table 3 are for backfill consisting of free-draining granular material (crushed aggregate or sand); on-site soils are not recommended for retaining wall backfill behind living spaces or supporting slabs-on-grade. Wall backfill should be compacted to at least 95% of the maximum dry density determined by ASTM D698 or equivalent.

Table 3 - Recommended Equivalent Fluid Unit Weights for Calculating Lateral Earth Pressures

	Unrestrained Wall		Restrained Wall	
Type	Level Profile	2H:1V Upslope	Level Profile	2H:1V Upslope
Active Pressure ($lbs/ft^2/ft$)	32	45	-	-
At-Rest Pressure ($lbs/ft^2/ft$)	-	-	50	65
Passive Pressure * ($lbs/ft^2/ft$)	280	280	130	130

* Passive pressure values are allowable and include a factor of safety of 1.5. For passive pressure calculations, the upper 6 inches of embedment should be ignored.

Subdrains should be installed behind all retaining walls to retard the build-up of adverse hydrostatic pressure. Subdrains should consist of a minimum 3-inch diameter ADS Highway Grade (or equivalent), perforated, plastic pipe enveloped in a minimum of 3 ft^3 per lineal foot of 2"- ½", open, graded gravel (drain rock) wrapped with geofabric filter (Amoco 4545, Trevia 1120, or equivalent). A minimum one-half percent fall should be maintained throughout the drain and non-perforated pipe outlet. For concrete retaining walls, waterproofing and a geocomposite wall drain such as Tuff-N-Dry, CONTECH C-DRAIN 11K, or equivalent are recommended to minimize the potential for interior moisture problems.

Concrete Slabs-On-Grade

If slab-on-grade foundations are used for living spaces, we recommend that underslab base rock consist of ¾"-0 crushed aggregate containing no more than 5% fine-grained material passing the No. 200 (0.75 mm) sieve. The minimum recommended base rock section for capillary break is 8 inches for dry-weather construction and 12 inches for wet-weather. Soil subgrade should be sloped away from the center of the slab at an approximate gradient of 1% in order to promote underslab drainage. Underslab aggregate should be compacted to at least 90% of its maximum dry density as determined by ASTM D1557 or equivalent.

Underslab moisture protection should be considered. Care should be taken during construction to avoid puncturing the barrier. Moisture barrier products should be installed in accordance with manufacturer recommendations. For wet-weather construction, we recommend that moisture sensitive flooring (such as vinyl tiles) be installed after the building is complete and the HVAC system operating for a period of time long enough to allow the vapor gradient within and below the building to stabilize and obtain acceptable slab moistures.

Drainage

Footing drains are recommended on the upgradient sides of the building foundations. These drains should consist of a minimum 3-inch diameter ADS Highway Grade (or equivalent), perforated, plastic pipe enveloped in a minimum of 1 ft^3 per lineal foot of 2"- ½", open, graded gravel (drain rock) wrapped with geofabric filter (Mirafi 140N or equivalent). A minimum 0.5% fall should be maintained throughout all subdrains and non-perforated pipe outlets.

Roof drain and surface run off should be directed away from structures and directed to a suitable discharge point in the street.

Excavating Conditions and Trench Backfill

Site observations and a review of previous site reports indicate that residual soils containing large basalt boulders will very likely be encountered during excavations for the lower foundations and garage portions of the proposed house. With exception of the boulders, overlying colluvial and residual soils are readily excavated and stable on temporary vertical cuts with minimal sloughing to a depth of 4 feet.

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Heath Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only.

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that structural trench backfill be compacted to at least 90% of the maximum dry density obtained by Modified Proctor ASTM D1557 or equivalent. Initial backfill lift thick nesses for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing in structural areas should be performed during construction to verify that the recommended relative compaction is achieved. Typically, one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Seismic Design

Probabilistic assessments of the seismic shaking hazard in Oregon predict that in the next 50 years bedrock underlying the subject site has a 10% probability of experiencing a peak ground acceleration (PGA) of 0.12 g, a 5% probability of experiencing a PGA of 0.26 g, and a 2% probability of experiencing a PGA of 0.36 g (Geomatrix, 1995).

The project site lies within Seismic Zone 3, as defined in Chapter 16, Division IV of the 1997 Uniform Building Code (UBC). Seismic Zone 3 includes the western portion of Oregon, and represents an area of relatively high seismic risk. For comparison, much of California and southern Alaska are defined as Seismic Zone 4, which is an area of highest seismic risk. Consequently, moderate levels of earthquake shaking should be anticipated during the design life of the proposed improvements, and the structures should be designed to resist earthquake loading in accordance with the methodology described in the 1997 UBC. Based on the subsurface conditions we observed during our exploration program, UBC Soil

Type S_C may be assumed for the site. The corresponding seismic factors may be used in developing a normalized response spectra for the assumed UBC Soil Type.

In our opinion, the potential for liquefaction or liquefaction-related ground failure at the subject site is very low, and no special mitigating measures are recommended.

Erosion Control Considerations

Since the site has moderate to locally steep slopes, the erosion potential is considered to be moderate to high. In our opinion, the primary concern regarding erosion potential will occur during construction, in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan. If used, these erosion control devices should be in place and remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly covering or re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and their consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, GeoPacific attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

JAMES E. PYNE
G1051

14743
JAN. 23, 1990
JAMES D. IMBRIE
Exp. 6-30-03

James E. Pyne, R.G.
Senior Geologist

James D. Imbrie, P.E.
Principal Geotechnical Engineer

Attachments: References
Figure 1 – Site Location Map
Figure 2 – Site Plan
Hand Auger Log – HA-1
Hand Auger Log – HA -2
Appendix A – Field Exploration and Laboratory Testing

REFERENCES

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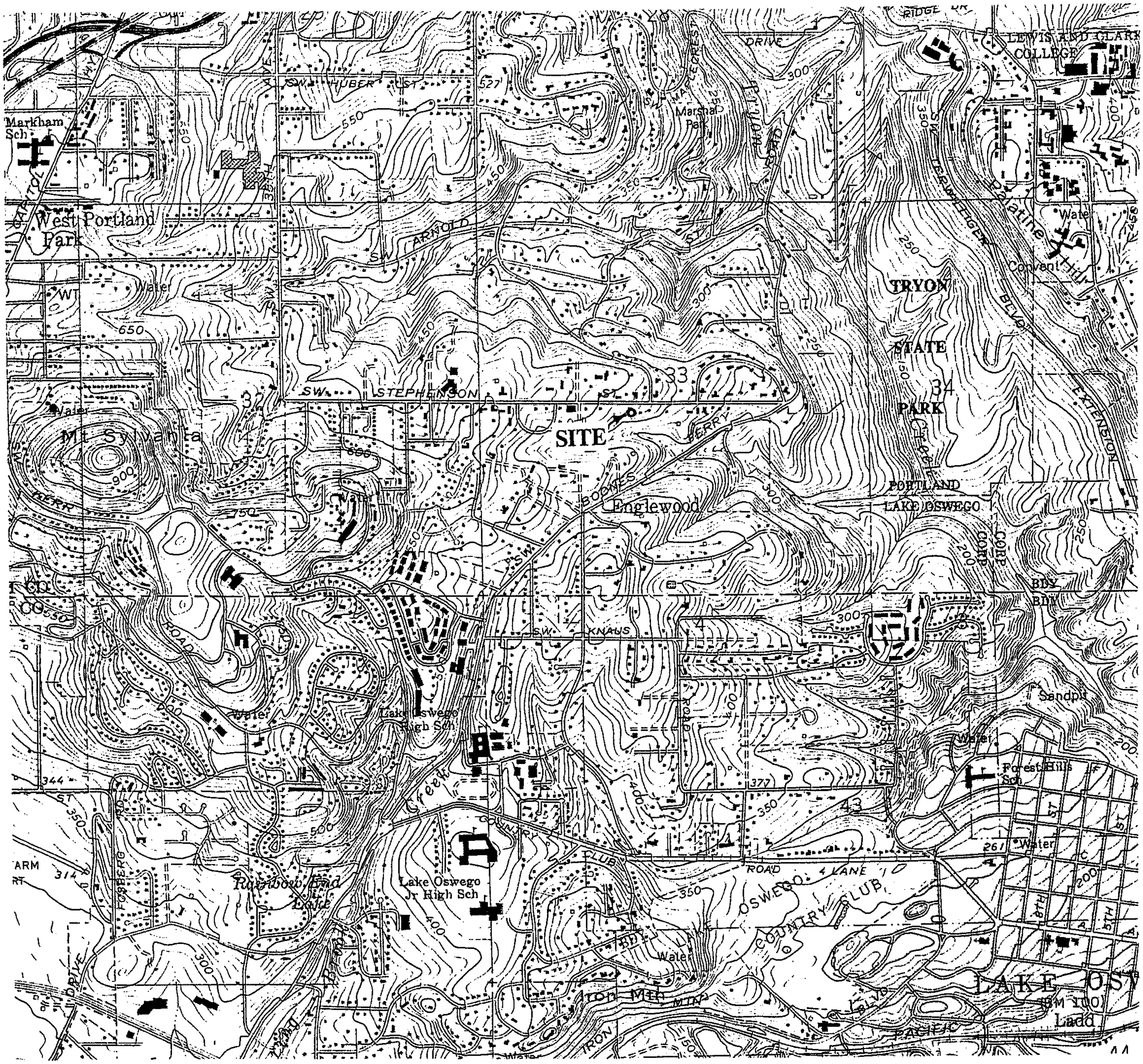


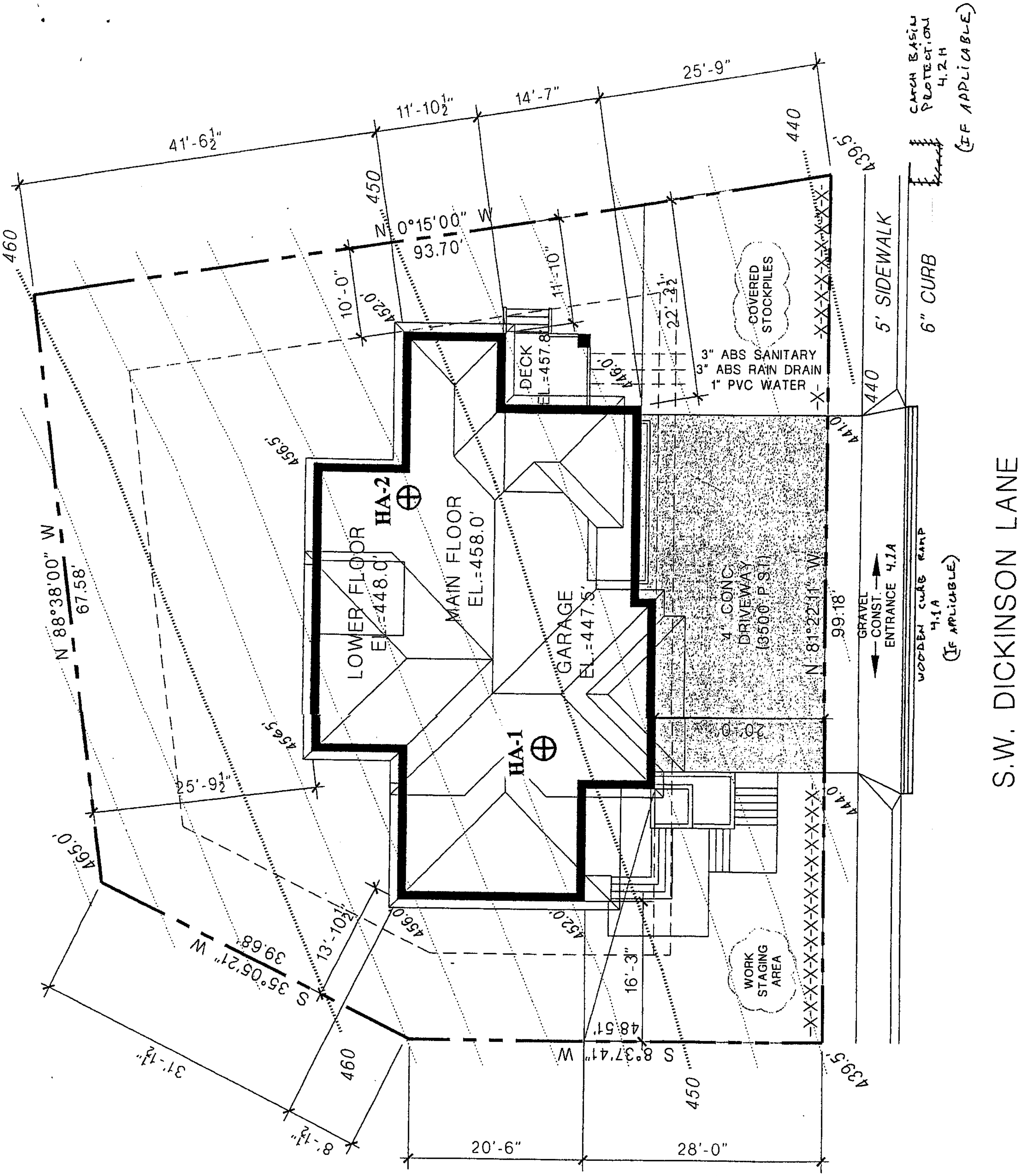
Portion of USGS Lake Oswego, Oregon
7.5 Minute Series (topographic), 1961,
Revised 1984.

Crino Custom Homes
Meadowview II Development

SITE LOCATION MAP

Scale 1"= 2000'

GeoPacific	June, 2001	Figure 1



From Alan Mascord Design Associates, Inc.

Crino Custom Homes
Meadowview II Development

SITE PLAN

Scale 1"= 18'

GeoPacific	June, 2001	Figure 2

GEOPACIFIC ENGINEERING, INC.
17700 SW Upper Boones Ferry Road, Suite 100
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 598-8705

HAND AUGER LOG

Project: Meadowview II
Portland, Oregon

Job No. 01-7333

Hand Auger No. **HA-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	**Material Description**
						Brown silt, trace of clay, little organic debris, some roots, moist, soft (Topsoil)
1						
2						Light brown silt, some clay, medium stiff, damp (Loess)
3						Mottled light brown and grey silt, fragmented, numerous small inclusions of completely weathered basalt (Colluvial Soil)
4						
5						
6						
7						Occasional fragment of basalt, highly weathered; in matrix of brown silt with some clay; more weathered rock at 82"
8						
9						Hand auger terminated at 94" on weathered rock, No groundwater encountered.
10						
11						
12						
13						
14						
15						
16						
17						

LEGEND

- 100 to 1,000 g Bag Sample
- 5 Gal. Bucket Bucket Sample
- ▯ Shelby Tube Sample
- 💧 Seepage
- ▨ Water Bearing Zone
- ▽ Water Level at Abandonment

Date Augered: June 12, 01
Logged By: JEP
Surface Elevation:

GEOPACIFIC ENGINEERING, INC.
17700 SW Upper Boones Ferry Road, Suite 100
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 598-8705

HAND AUGER LOG

Project: Meadowview II
Portland, Oregon

Job No. 01-7333

Hand Auger No. **HA- 2**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	**Material Description**
						Dark brown silt, trace of clay, little organic debris, soft, moist (Topsoil)
1						Brown silt, trace of clay, soft, fragmented, moist (Possibly Fill)
2						Mottled light and dark brown silt, some clay, stiff (Colluvial Soil) Rock fragments encountered at 36" depth
3						
4						Test pit terminated at 36 inches on top of rock fragments, No groundwater encountered.
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17						

LEGEND

100 to 1,000 g	5 Gal. Bucket			▨	
Bag Sample	Bucket Sample	Shelby Tube Sample	Seepage	Water Bearing Zone	Water Level at Abandonment

Date Augered: June 12, 01
Logged By: JEP
Surface Elevation: