GEOTECHNICAL ENGINEERING
INVESTIGATION REPORT
COPPER CANYON DEVELOPMENT
RIDGETOP BOULEVARD NW
SILVERDALE, WASHINGTON

JOB NUMBER 102-03046
JUNE 10, 2003

Prepared for:

Crescent Investments, LLC
P. O. Box 5
Tracyton, WA 98393

Prepared by:

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Krazan & ASSOCIATES, INC.
SITE DEVELOPMENT ENGINEERS

Permit Number: 20-01254

Krazan & ASSOCIATES, INC.

GEOTECHNICAL ENGINEERING • ENVIRONMENTAL ENGINEERING
CONSTRUCTION TESTING & INSPECTION

June 10, 2003

KA Project No. 102-03046

Mr. Hal Fergusson
Crescent Investments, LLC
P. O. Box 5
Tracyton, WA 98393

RE: GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
COPPER CANYON DEVELOPMENT
RIDGETOP BOULEVARD NW
SILVERDALE, WASHINGTON

In accordance with your request, we have completed a Geotechnical Engineering Investigation for the referenced project. The results of our investigation are presented in the attached report. This report presents the results of our field exploration, laboratory tests, and engineering analyses including design recommendations for 1H:1V Mechanically Stabilized Earth slope.

If you have any questions or if we can be of further assistance, please do not hesitate to contact our office.

Respectfully submitted,

KRAZAN AND ASSOCIATES, INC.

Todd S. Parkington

Todd S. Parkington, P.E.
Senior Geotechnical Engineer

TSP

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Krazan & ASSOCIATES, INC.
GEOTECHNICAL ENGINEERING • ENVIRONMENTAL ENGINEERING
CONSTRUCTION TESTING & INSPECTION

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KA Project No. 102-03046

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RIDGETOP BOULEVARD NW
SILVERDALE, WASHINGTON

INTRODUCTION

This report contains the results of a site investigation performed by Krazan & Associates for the above referenced project.

SITE LOCATION

The proposed Copper Canyon Development is located east of Ridgetop Boulevard NW between NW Timber Shadow Court and NW Thornwood Circle in Silverdale, Washington. According to the United States Geological Survey (USGS), 7.5 minute Poulsbo, Washington topographic quadrangle map, the site is located in the southeast quarter of the northwest quarter of Section 10, Township 25 North, Range 1 East, W.M. and at approximately Latitude 47.675 degrees, Longitude 122.663 degree. The site location is shown on the Site Vicinity Map, Figure 1.

PROPOSED CONSTRUCTION

We understand that the development will consist of 147 residential lots with associated streets, utilities and common areas. The subject property consists of two parcels, hereinafter referred to as the western parcel and the eastern parcel. The eastern parcel will have 82 lots located in a relatively flat area at the base of a hill. The remaining 65 lots will be adjacent to the access road, which will descend the hill from Ridgetop Boulevard.

We assume that the residences will be one to two stories in height and of relatively light wood frame construction. We further assume that traffic on the access road and other streets within the development will consist primarily of passenger vehicles with occasional service vehicles.

We understand the plan is to mass grade the site with an essentially balanced cut & fill operation. Based on the preliminary site plan provided to us, we estimate cuts of up to 30 feet and fills of up to 50 feet. Due to space constraints, 1H:1V (horizontal:vertical) fill slopes are planned for two areas. The westernmost of the two fill slopes will be approximately 450 feet long with a maximum height of about 50 feet. The easternmost of the two fill slopes will be approximately 350 feet long with a maximum height of about 25 feet. The location of these fill slopes are indicated on the Site Plan, Figure 2.

Note that the Site Plan does not include the 82 lots on the eastern parcel, as we did not perform any fieldwork on the eastern parcel.

In the event the proposed construction information detailed in this report is inconsistent with the final design, we should be notified so that we may update this writing as applicable.

PURPOSE & SCOPE

The purpose of this project is to provide geotechnical engineering recommendations for construction of the referenced development. Our scope of work includes the following items:

- Investigation of the soil and groundwater conditions in the project area by drilling 3 borings. The borings ranged in depth from 41 to 61 feet below the ground surface (bgs). Groundwater measurements were taken during drilling.
- Laboratory testing appropriate to the soil conditions encountered and the planned construction was conducted. Tests for moisture content, grain size distribution and direct shear strength were performed.
- Slope stability analyses of the proposed 1H:1V fill slopes and the native slope in the ravine on the north side of the property were performed.
- Preparation of this report detailing our findings and conclusions including recommendations for setbacks from the native slopes in the ravine to the north, reinforcement for the 1H:1V fill slopes, structural fill requirements, drainage, pavement design, soil compaction criteria, and the suitability of the on-site soil for reuse as fill.

SITE INVESTIGATION

SITE DESCRIPTION

The site is bordered to the north and south by residential developments, to the west by Ridgetop Boulevard NW and to the east by undeveloped land.

The western parcel slopes down to the east with gradients ranging from about 15 to 50 percent and a total elevation change from the west to the east of about 140 feet. There is a steep sided (50 percent slopes) ravine along the north property edge of this parcel, with two short ravines leading into the property off of the main ravine. The proposed grading plan for the project includes filling of the two short ravines. Please see the Site Plan, Figure 2, for more information on the topography of the site.

The eastern parcel also slopes to the east with gradients ranging from 10 to 15 percent and a total elevation change of about 30 feet. Note that the site investigation focused on the western parcel, as the slopes on the eastern parcel are too gentle to constitute a slope hazard.

At the time of our investigation, the site was forested with second growth timber and moderately heavy underbrush.

GEOLOGIC SETTING

The subject site lies within the central Puget Lowland. The lowland is part of a regional north-south trending trough that extends from southwestern British Columbia to near Eugene, Oregon. North of Olympia, Washington, this lowland is glacially carved with a depositional and erosional history including at least four separate glacial advance/retreats. The Puget Lowland is bounded on the west by the Olympic Mountains and on the east by the Cascade Range. The lowland is filled with glacial and nonglacial sediments consisting of interbedded gravel, sand, silt, till, and peat lenses.

The Geologic Map of Washington – Northwest Quadrant published by the Washington State Division of Geology and Earth Resources, 2002, indicates the site is underlain by glacial till. Till consists of an unsorted, unstratified, highly-compacted mixture of clay, silt, sand, gravel, and boulders deposited by glacial ice. Till may contain interbedded stratified sand, silt, and gravel. The till generally overlies advanced outwash deposits.

SUBSURFACE EXPLORATION

The field investigation consisted of drilling three borings to depths ranging from 41 to 61 feet below the ground surface. Groundwater measurements were taken during drilling. The boring locations are indicated on the Site Plan, Figure 2.

Soil

The soils encountered in the borings consisted of silty sands and poorly graded sands to the maximum depth explored. The sands were loose in the upper 2 to 4 feet of the borings grading to dense to very dense below 4 feet. Please refer to the boring logs in Appendix A for more information.

Groundwater

Groundwater was encountered at approximately elevation 274 in borings B-1 and B-2. Groundwater was not encountered in boring B-3. Note that the lowest elevation reached by boring B-3 was approximately elevation 299. Water table elevations fluctuate with time, being dependent upon seasonal precipitation, irrigation, land use, and climatic conditions, as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered during the construction phase of the project. The evaluation of such factors is beyond the scope of this report.

Laboratory Testing

Soil samples were obtained from the borings for visual classification and laboratory testing for engineering properties. Tests were performed for moisture content, fines content, grain size distribution and direct shear strength. Please see Appendix A for more information.

SEISMIC ZONE

According to the *Seismic Zone Map of the United States* contained in the 1997 *Uniform Building Code,* the project site lies within Seismic Risk Zone 3. The overall soil profile generally corresponds to seismic soil profiles S_C as defined by Table 16-J of the 1997 *Uniform Building Code (UBC)*. Soil profile S_C applies to a profile consisting primarily of very dense soils within the upper 100 feet of the profile. The United States Geologic Survey, Earthquake Hazards Program, National Seismic Hazard Mapping Project website indicates that the peak ground acceleration for the site with a probability of exceedence of 10 percent in 50 years is 0.32 g.

Due to the relatively dense nature of the soils encountered during our field exploration, it is our opinion that the risk for liquefaction of the soils at the site is minimal.

SITE RECONNAISSANCE

The slopes and adjacent properties were examined for indications of slope failures or instability. Indications of slope failure and/or instability include head scarps, hummocky terrain, inconsistent patterns of vegetation, tension cracks, seepage zones and course grain material overlaying silt and clay soils. We did not observe any indication of previous slope failures or instability.

CONCLUSIONS AND RECOMMENDATIONS

The results of our analysis indicate that the proposed grading for the site, including the proposed 1H:1V fill slopes, is feasible. The 1H:1V fill slopes will require geogrid reinforcement commonly referred to as mechanically stabilized earth slopes or MSE slopes. Setbacks for buildings will be 12 feet from the top of 1H:1V reinforced fill slopes and 8 feet from 2H:1V native slopes. Please refer to the sections below for additional details.

MECHANICALLY STABILIZED EARTH SLOPE (MSE SLOPE)

As described above, 1H:1V fill slopes are proposed for the project in two areas. In this section we provide recommendations for constructing a mechanically stabilized earth fill slope at 1H:1V. Two types of reinforcement will be used; primary and secondary.

Slope Preparation

The area beneath the reinforced section of the MSE slopes will need to be prepared to ensure that adequately dense soil underlies it. Up to 4 feet of loose sand was encountered in our borings. The loose sand must be removed and recompacted from beneath the reinforced section of the MSE slope (i.e. there should be no loose sand beneath the bottom layer of reinforcement). Any of the loose sand relatively free of organics and otherwise conforming to the recommendations given below under Structural Fill may be reused as such. In areas where the primary reinforcement would extend into the existing slope, a bench will need to be cut into the existing slope. The back wall of the bench should slope up at 1H:1V.

Construction of the taller of the two MSE slopes will involve filling two ravines. After the loose sand has been removed as described above, we recommend that a layer of washed rock be placed in the base of each of the ravines. The washed rock should be at least 2 feet thick and 4 feet wide and wrapped in filter fabric. The washed rock should consist of washed gravel with no sand or fines. The upper end of the rock drain should end at about 4 feet below the top of the fill. The exit from the rock drain at the face of the MSE slope should be wrapped in filter fabric and covered with a 1-foot thick layer of 2 to 4 inch quarry spalls.

In addition, a drain should be placed at the back edge of the deepest reinforcement on both MSE slopes. These drains should consist of a round section of washed gravel wrapped in filter fabric with a diameter of at least 18 inches. These drains should connect to the ravine drains on the taller MSE slope and should connect to 2 evenly spaced drains constructed in a similar manner to the ravine drains on the smaller MSE slope.

Primary Reinforcement

The primary reinforcement will consist of Tensar Earth Technologies, Inc. Structural Geogrid UX1400HS or an equivalent geogrid with a long-term allowable load of at least 2,000 lb/ft. Any proposed substitute geogrid should be submitted to us for approval. In general, the vertical spacing of the primary reinforcement within the MSE slope will be 4 feet to a depth of 42 feet below the top of the slope and 2 feet below a depth of 42 feet. In the tables below we provide the depth and length of each layer of primary reinforcement. The first table is for the portion of the slopes that are less than 34 feet high. The second table is for the portion of the slopes that are more than 34 high up to a maximum height of 54 feet. In reading the tables below, please note that any layers that would be deeper than the toe of the MSE slope may be omitted (i.e. at a section of MSE slope that is 20 feet high, layers 5, 6 & 7 from Table 1 may be omitted).

Uniaxial grids (UX) are always unrolled perpendicular to the slope (i.e. start unrolling at the slope face and roll into the fill. Follow the manufacturers recommendations for connecting geogrid rolls together.

Table 1

Primary Reinforcement for MSE Slopes less than 34 feet high

Layer #	Depth below top of slope (ft)	Length (ft)
1	6	10
2	10	12
3	14	22
4	18	24
5	22	26
6	26	28
7	30	28

Table 2

Primary Reinforcement for MSE Slopes more than 34 feet up to 54 feet high

Layer #	Depth below top of slope (ft)	Length (ft)
1	6	24
2	10	28
3	14	32
4	18	34
5	22	34
6	26	36
7	30	38
8	34	38
9	38	40
10	42	40
11	44	40
12	46	42
13	48	42
14	50	42
15	52	42

Tension must be maintained in the reinforcement as fill is placed over it. Tracked vehicles should never be allowed to drive directly on the reinforcement.

The design life of geogrid reinforcement is estimated to be 75 years, which is similar to the intended design life of most structures. As geogrid is a relatively new product, the design life can only be estimated.

Secondary Reinforcement

The secondary reinforcement will consist of Tensar Earth Technologies, Inc. Structural Geogrid BX1400 or an equivalent geogrid with a true tensile strength at 5 percent strain of 900 lb/ft in the direction perpendicular to the slope. Any proposed substitute geogrid should be submitted to us for approval. The secondary reinforcement will be placed at 1 foot vertical intervals between the primary reinforcement, beginning at 1 foot below the ground surface. The secondary reinforcement should extend at least 5 feet into the slope. Biaxial geogrids can be unrolled either perpendicular or parallel to the slope. We have assumed for our design that the BX1400 will be unrolled parallel to the slope.

Fill

The fill used for construction should conform to the recommendations given under Structural Fill below. Fill placed along the slope face should be overbuilt by at least 6 inches. The slope face should be compacted every 2 to 3 feet using a hoe pack in addition to the standard rolling of the top of the lift.

SLOPE STABILITY

Slope stability analyses were performed on three cross-sections; two within the taller MSE slope and one on a ravine slope that will not be re-graded. The locations of the cross-sections are indicated on the Site Plan, Figure 2. Topography used in the analysis was based on the Copper Canyon Site Plan, dated March 31, 2003 prepared by Team 4 Engineering. The slope stability computer program Slope/W by GeoSlope International was used to evaluate the stability of the existing slopes and proposed MSE slopes under static and seismic conditions. Soil strength parameters used in our analysis were based on in-situ penetration tests, laboratory shear strength tests and published values. The engineering properties of the soil used in our analysis are presented on Figure 3, Cross Sections A-A', Figure 4, Cross Section B-B', and Figure 5 Cross Section C-C'. Cross Section A-A' represents the maximum height of MSE slope on the site, Cross Section B-B' represents the closest approach of the access road to the MSE slope, and Cross Section C-C' represents a relatively steep section of native slope. Water levels used in the stability analyses were conservatively assumed to be higher than the water level encountered in our borings.

The psuedostatic method was used for our slope stability analyses to estimate the factor of safety under seismic conditions. The United States Geologic Survey, Earthquake Hazards Program – National Seismic Hazard Mapping Project, indicates that a peak ground acceleration (PGA) of 0.32 g has a 10 percent probability of exceedence in 50 years (500 year return period). The seismic coefficient is typically taken to be ½ of the PGA. A seismic coefficient of 0.16 was used in our analyses.

The results of slope stability analyses are expressed as factors-of-safety against rotational failure. The factor-of-safety is the ratio of driving forces to resisting forces. A factor-of-safety of 1.0 is equilibrium; a factor-of-safety of less than 1.0 indicates failure. Typically, a factor-of-safety of 1.5 for static conditions and 1.1 for seismic conditions is considered adequate. Factors of safety greater than 1 but less than 1.5 (or 1.1) are not adequate due to the uncertainties inherent in the modeling process. A lower safety factor for seismic conditions is considered adequate, as the probability of occurrence of the seismic conditions analyzed is relatively low. The slope stability analyses used on cross sections A-A' and B-B' were used to design the lengths and geogrid spacing for the MSE slopes. The slope stability analyses performed on cross section C-C' indicate a static factor of safety of 1.70 and a seismic factor of safety of 1.21. The results of our slope stability analyses are also presented graphically in Appendix B.

In our opinion the existing steep slopes and the proposed MSE slopes will have an adequate factor of safety against slope failure. For more information concerning the slope stability results see Appendix B.

Based on our slope reconnaissance, and slope stability analysis the slopes are relatively stable in their present condition. In order to enhance the long-term stability of the slopes, surface runoff from the development will need to be collected and directed away from slopes. Do not allow additional surcharge loads, soil stockpiles, standing water or loosened soil conditions to occur between the residences and the top of slopes. Irrigation utilized in landscaping should be monitored closely to insure that it is functioning correctly. Malfunctioning irrigation systems or ruptured irrigation lines may flood slope areas causing slope failures.

Setbacks

In order to protect structures from slope migration and future instability, structures should be setback at least 12 feet from the top edge of the proposed MSE slopes. Note that the primary reinforcement will likely extend beneath structures on Lot Numbers 20, 21 and 22. Setbacks from native slopes steeper than 3H:1V should conform to UBC requirements except that setbacks need not exceed 8 feet. Setbacks are to be measured from the furthest projection of the footing element. The setback distance assumes a standard footing embedment depth of 1.5 feet, re-vegetation of graded slope areas and that site grades are roughly the same as analyzed in our two cross sections.

Note that the setback described above is intended for buildings constructed on the lots as presented on the Plan presented to us and is **not** intended to apply to the roads as laid out on the Site Plan, Figure 2. If the road alignments are altered with respect to the top of slopes we should be notified so that we may review the stability of slopes with respect to road locations.

Note that the UBC requirements provide for measuring the setback from the base of the footing to the slope at the elevation of the footing. This effectively allows a setback to be met by increasing the depth of the footing. This method is acceptable at this site for setbacks from native slopes but does **not** apply to setbacks from the MSE slopes. The setback from the MSE slope is to be measured from the top of the slope to the furthest projection of the footing element regardless of footing depth.

EARTHWORK CONSIDERATIONS

During wet weather conditions, typically October through April, subgrade stability problems and grading difficulties may develop due to high moisture content in the soil, disturbance of sensitive soils and/or the presence of perched groundwater. Therefore, we recommend that grading activities be limited to the dry season (May through September). Note that this is a recommendation to avoid additional costs associated with earthwork activities performed during wet weather. Earthwork activities may occur during the wet season provided the owner and contractor are prepared to accept additional costs associated with wet weather earthwork construction.

Note when installing utilities on the site, care must be taken not to damage the geogrid reinforcement.

Site Preparation

General site clearing should include removal of vegetation, trees and associated root systems, wood, pavement, retaining walls, rubble, and rubbish. Site stripping must extend to a minimum depth of 4 inches, or until all organics in excess of 3 percent by volume are removed. Deeper stripping may be required in localized areas. These materials will not be suitable for use as fill for parking or building areas. However, stripped topsoil may be stockpiled and reused in landscape or non-structural areas.

Any buried structures encountered during construction will likely need to be removed. Specific recommendations should be obtained from the geotechnical engineer regarding any buried structures encountered. In general, any septic tanks, underground storage tanks, debris pits, cesspools, or similar structures should be entirely removed. Concrete footings should be removed to an equivalent depth of at least 3 feet below proposed footing elevations or as recommended by the Geotechnical engineer. Excavations, depressions, or soft and pliant areas extending below planned finish subgrade level should be cleaned to firm undisturbed soil, and backfilled with general fill.

Groundwater Concerns

Groundwater was encountered in 2 of our 3 borings at approximately elevation 276. As we did not observe any evidence of groundwater seepage on the slopes at or above the existing storm water ponds, we do not anticipate construction on the western parcel to be significantly impacted by groundwater. We did not perform a reconnaissance on the eastern parcel and the eastern parcel is relatively close to the outlet stream for Island Lake. It is possible that shallow groundwater may impact construction on the eastern parcel. However, a more detailed analysis of the effect of groundwater on construction on the eastern parcel is outside the scope of this report.

Excavations

In our opinion the soils encountered in our subsurface investigation are a Type B material as defined by the Washington Industrial Safety and Health Act's (WISHA) regulations on excavation, trenching and shoring. Temporary slopes excavated in Type B material should be inclined no steeper than 1H:1V. Permanent cut and fill slopes (non-reinforced) should be inclined no steeper than 2H:1V. Please see the MSE Slopes section above for more information on permanent slopes steeper than 2H:1V. A representative of our firm should evaluate temporary and permanent slopes to insure that they are appropriate for the soils encountered during construction.

Temporary slope areas should be covered with plastic visqueen to help minimize erosion and raveling and reduce sediment loading in surface runoff. During construction, any signs of instability along temporary slopes should be brought to our attention. All permanent slopes should be replanted with fast-growing, deep-rooted grass, shrubs and other ground cover as soon after final grading as practical. If the vegetation is not fully established prior to the on set of wet weather, the slopes should be covered with clear visqueen to aid in preventing excessive erosion and water infiltration.

In areas where it is not possible to maintain the recommended slopes due to space constraints, temporary shoring will be required. Please contact us for more information if temporary shoring will be required.

Structural Fill

The on site soils may be used as structural fill. Structural fill should be placed in loose lifts no more than 12-inches thick, moisture-conditioned as necessary, (moisture content of soil should be within ±2 percent of optimum moisture) and compacted to 95 percent of the maximum density based on ASTM Test Method D-1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. Note that, although density testing of fill is frequently used as the primary criteria for acceptance of fill, it should not be the only criteria. If, in the judgment of the geotechnical engineer or his representative, placed fill is not suitable it should be rejected regardless of density test results. As an example, fill that is compacted wet of the optimum moisture content may exhibit "pumpy" behavior even if density test results indicate better than 95 percent compaction has been achieved. In such a situation, the fill should be removed and replaced with drier material.

Imported structural fill material should consist of well-graded gravel or a sand and gravel mixture with a maximum grain size of 1½ inches and less than 5 percent fines. All imported structural fill material should be submitted for approval to the Geotechnical Engineer at least 48 hours prior to delivery to the site.

Note that the on site soils typically have a high silt content and will therefore be difficult or impossible to compact if they are well over the optimum moisture content.

Utility Trench Backfill

Utility trenches should be excavated according to accepted engineering practice following WISHA standards by a contractor experienced in such work. The responsibility for the safety of open trenches should be borne by the contractor. Traffic and vibration adjacent to trench walls should be minimized. Cyclic wetting and drying of excavation side slopes should also be avoided.

Utility trench backfill should be structural fill. Pipe bedding should be in accordance with pipe manufacturer's recommendations. The contractor should use appropriate equipment and methods to avoid damage to the utilities and/or structures during fill placement and compaction.

DRAINAGE

The ground surface should slope away from building pad and pavement areas toward appropriate drop inlets or other surface drainage devices. We recommend that adjacent exterior grades be sloped a minimum of 2 percent for a minimum distance of 5 feet away from structures. Roof drains should be tightlined away from foundations. Subgrade soils in pavement areas should be sloped a minimum of 1 percent and drainage gradients maintained to carry all surface water to collection facilities. These grades should be maintained for the life of the project. Footing drains should be placed around the perimeter of the building.

EROSION CONTROL

Erosion and sediment control (ESC) is used to minimize the transportation of sediment to wetlands, streams, lakes, drainage systems, and adjacent properties. As the site is not directly adjacent to surface waters, we anticipate that standard erosion and sediment control measures (such as silt fences at the perimeter of the construction area, and protection for any existing storm sewer inlets that may be affected by the construction) for this site will be sufficient. Note that water should not be allowed to flow over the top of the steep slope.

PAVEMENT DESIGN

The soils underlying the topsoil consist of silty sand and poorly graded sand with silt. We rate this soil as fair subgrade material. We estimate that this subgrade will have a California Bearing Ratio (CBR) value of 10 to 20, provided the subgrade is prepared in general accordance with our recommendations.

We recommend that all topsoil be removed and a minimum 12 inches of the existing subgrade material be moisture conditioned (as necessary) and re-compacted to prepare for the construction of pavement sections. The parking and pavement subgrade areas should be proof-rolled with a fully loaded dump truck. The proof rolling will identify loose and pliant areas. Such areas should either be compacted or over-excavated and backfilled.

Traffic loads were not provided. However, based on our knowledge of the proposed development, we expect the majority of the traffic loads to be for light traffic and occasional service vehicles. Provided below are recommendations for light and heavy-duty pavement areas. Heavy-duty pavement areas are intended primarily for the unloading of large delivery trucks and/or the movement of garbage trucks and as such are required for all streets within the development. Light duty pavement areas are intended for any areas that are likely to see only automobile traffic, such as parking areas. The following tables show the recommended pavement sections for light duty and heavy-duty areas.

ASPHALTIC CONCRETE PAVEMENT
LIGHT DUTY

Traffic Level	Asphaltic Concrete	Aggregate Base*
Low	2.0 inches	8.0 inches

ASPHALTIC CONCRETE PAVEMENT
HEAVY DUTY

Traffic Level	Asphaltic Concrete	Aggregate Base*
Low	4.0 inches	8.0 inches

** Aggregate base should conform to the specifications for Crushed Surfacing – Base Course provided in Section 9-03.9(3) of the Washington State Department of Transportation Standard Specifications Manual dated 2000. Aggregate base and subgrade should be compacted to 95% based on ASTM Test Method D1557.*

The provided heavy and light duty pavement sections are based on flexible pavement design procedures for low volume roads presented in the AASHTO Guide for Design of Pavement Structures. Based on the subsurface exploration for the pavement design, we assumed a fair subgrade with inherent reliability of 50 percent. The structural number used in the pavement design was based on the climatic region II and assumed traffic volume. Pavement design recommendations assume proper drainage and construction observation and are based on AASHTO design parameters for a 15 to 25 year design period. However, pavement maintenance after about 8 to 10 years should be expected to obtain the desired service life.

FOUNDATIONS & SUBSURFACE WALLS

Recommendations for design of individual residence foundations and retaining walls were not part of our scope of work. Recommendations for these elements can be provided upon request for an additional fee.

TESTING AND INSPECTION

A representative of Krazan & Associates, Inc. should be present at the site during the earthwork activities to confirm that actual subsurface conditions are consistent with the exploratory fieldwork. This activity is an integral part of our services. Specifically, Krazan and Associates should be present to observe placement of structural fill (to verify soil type and minimum compaction requirements were met) and placement of geogrid reinforcement within the MSE slopes. Note that monitoring of fill and geogrid placement within the MSE slopes must be done on a full-time basis (i.e. the representative of the geotechnical engineer is on-site whenever the contractor is placing fill or geogrid in the MSE slope area). We should also observe cut and fill slopes to ensure that the soils encountered during construction match the soils encountered during the exploration.

LIMITATIONS

Geotechnical engineering is one of the newest divisions of Civil Engineering. This branch of Civil Engineering is constantly improving as new technologies and understanding of earth sciences improves. Although your site was analyzed using the most appropriate current techniques and methods, undoubtedly there will be substantial future improvements in this branch of engineering. In addition to improvements in the field of Geotechnical engineering, physical changes in the site either due to excavation or fill placement, new agency regulations or possible changes in the proposed project after the time of completion of the soils report may require the soils report to be professionally reviewed. In light of this, the Owner should be aware that there is a practical limit to the usefulness of this report without critical review.

Earthwork construction is characterized by the presence of a calculated risk that soil and groundwater conditions have been fully revealed by the original field investigation. This risk is derived from the practical necessity of basing interpretations and design conclusions on limited sampling of the earth. The recommendations made in this report are based on the assumption that soil conditions do not vary significantly from those disclosed during our field investigation. If any variations or undesirable conditions are encountered during construction, the Geotechnical engineer should be notified so that supplemental recommendations can be made.

The conclusions of this report are based on the information provided regarding the proposed construction. If the proposed construction is relocated or redesigned, the conclusions in this report may not be valid. The Geotechnical engineer should be notified of any changes so the recommendations can be reviewed and reevaluated.

This report is a geotechnical engineering investigation with the purpose of evaluating the soil conditions in terms of foundation design. The scope of our services did not include any environmental site assessment for the presence or absence of hazardous and/or toxic materials in the soil, groundwater or atmosphere, or the presence of wetlands. Any statements, or absence of statements, in this report or on any logs regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous materials.

The geotechnical information presented herein is based upon professional interpretation utilizing standard engineering practices and a degree of conservatism deemed proper for this project. It is not warranted that such information and interpretation cannot be superseded by future geotechnical developments. We emphasize that this report is valid for this project as outlined above, and should not be used for any other site.

Respectfully submitted,

KRAZAN & ASSOCIATES, INC.

Todd S. Parkington

Todd S. Parkington, P.E.
Senior Geotechnical Engineer

TSP:wrj/sew

TODD S. PARKINGTON
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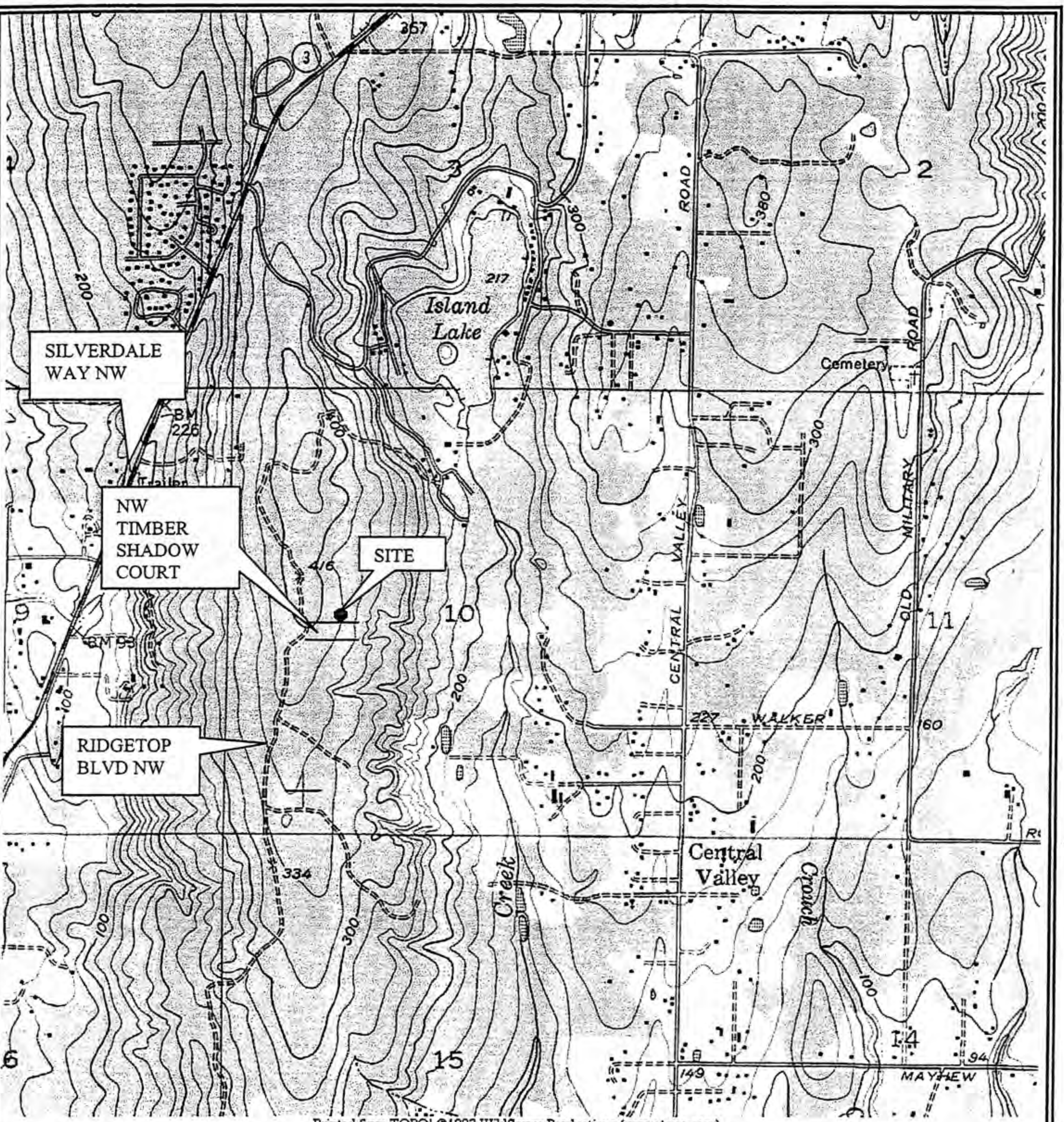

Note: Map adapted from TOPO! © 1997, Wildflower Productions.

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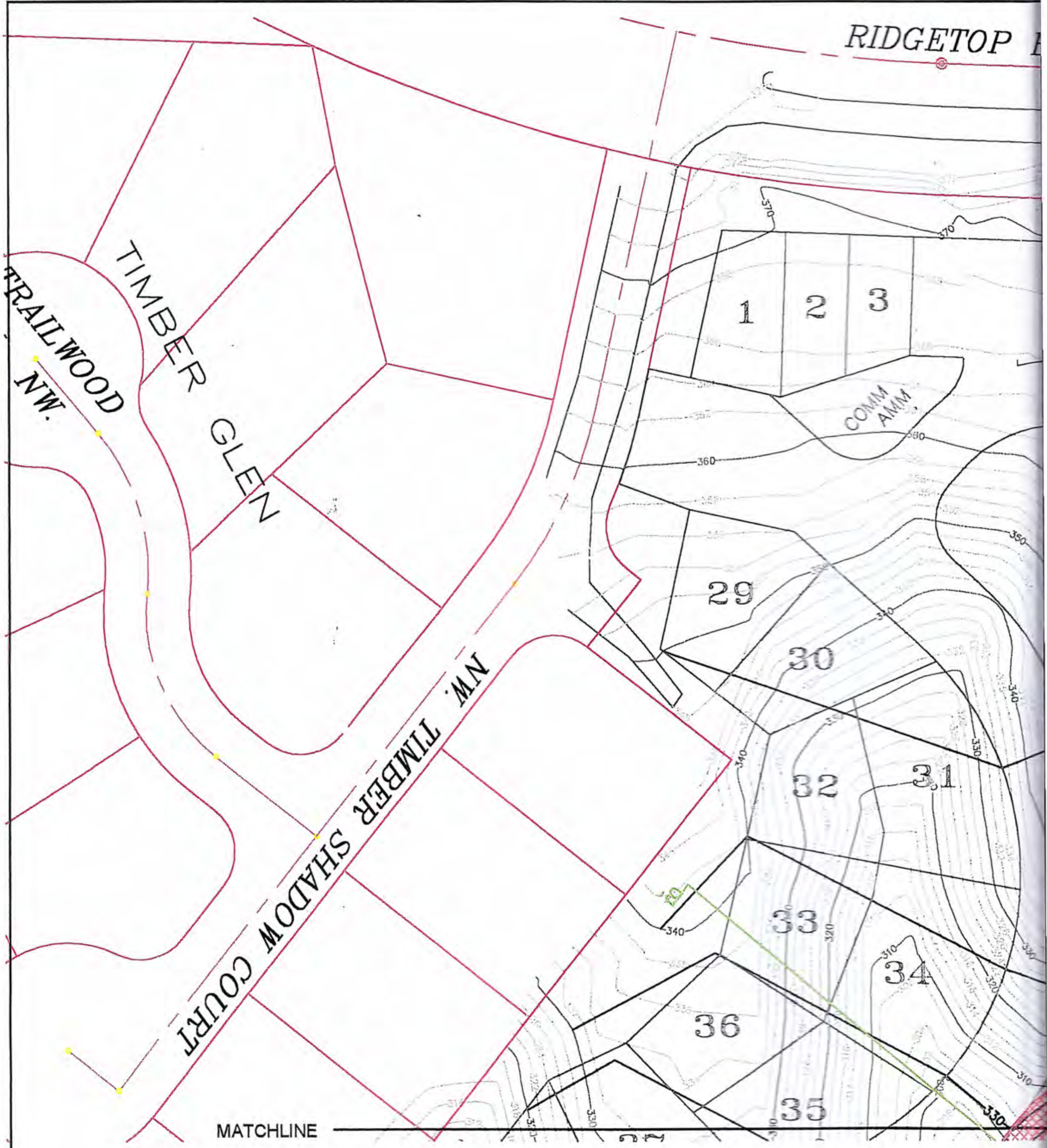
FIGURE 1 – SITE VICINITY MAP

Job name: Copper Canyon
Location: Kitsap County, Washington
Job No. : 102-03046
Client: Crescent Investments, LLC
Date: 5/27/03

N

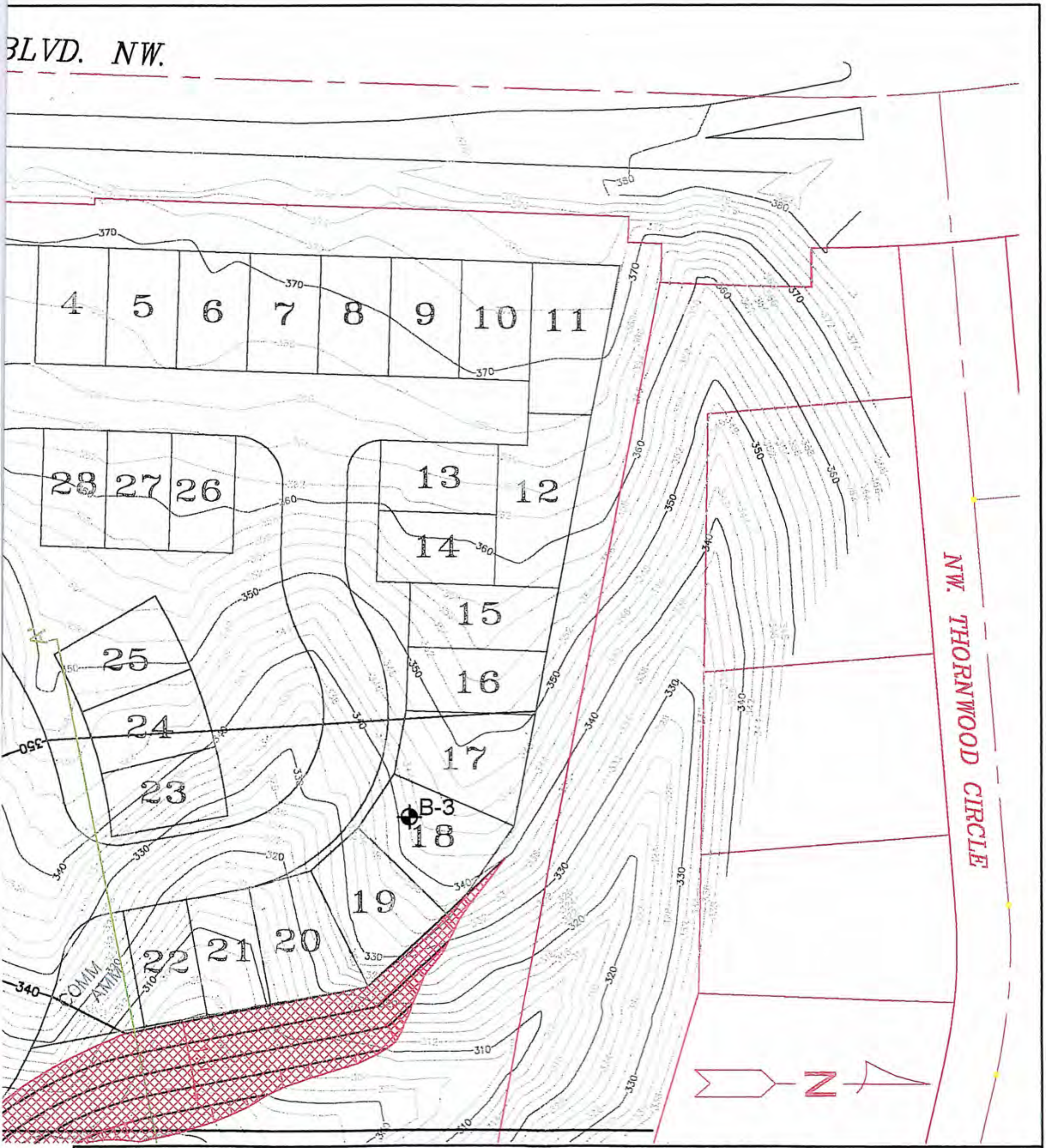
Permit Number: 20-01254



Note Legend information on Figure 2B

Site Plan developed from plans provided by Team 4 Engineering, Copper Canyon Site Plan, Dated 3/31/03

Permit Number: 20-01254



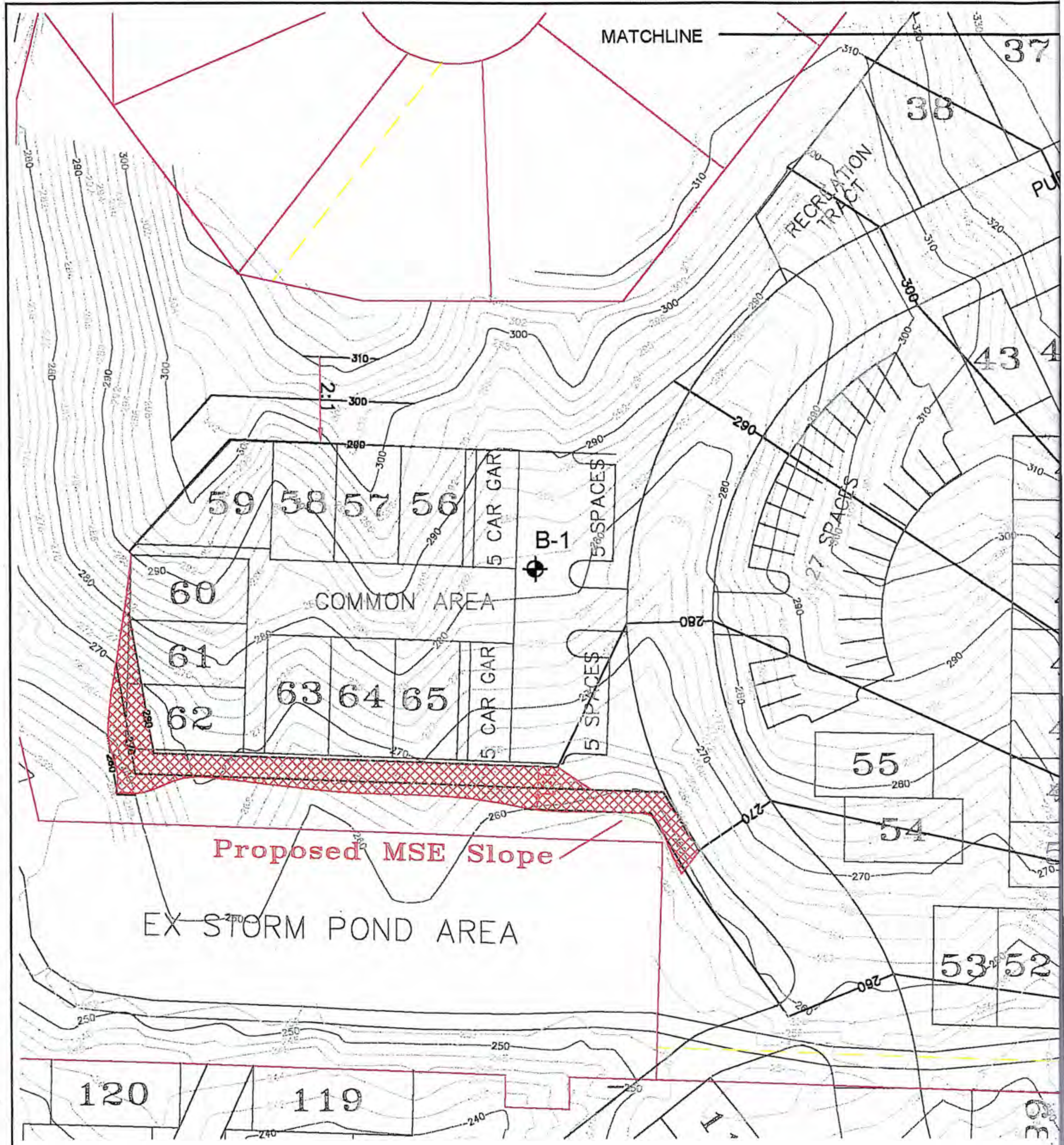
Site Plan - West Section

Copper Canyon

Figure 2a

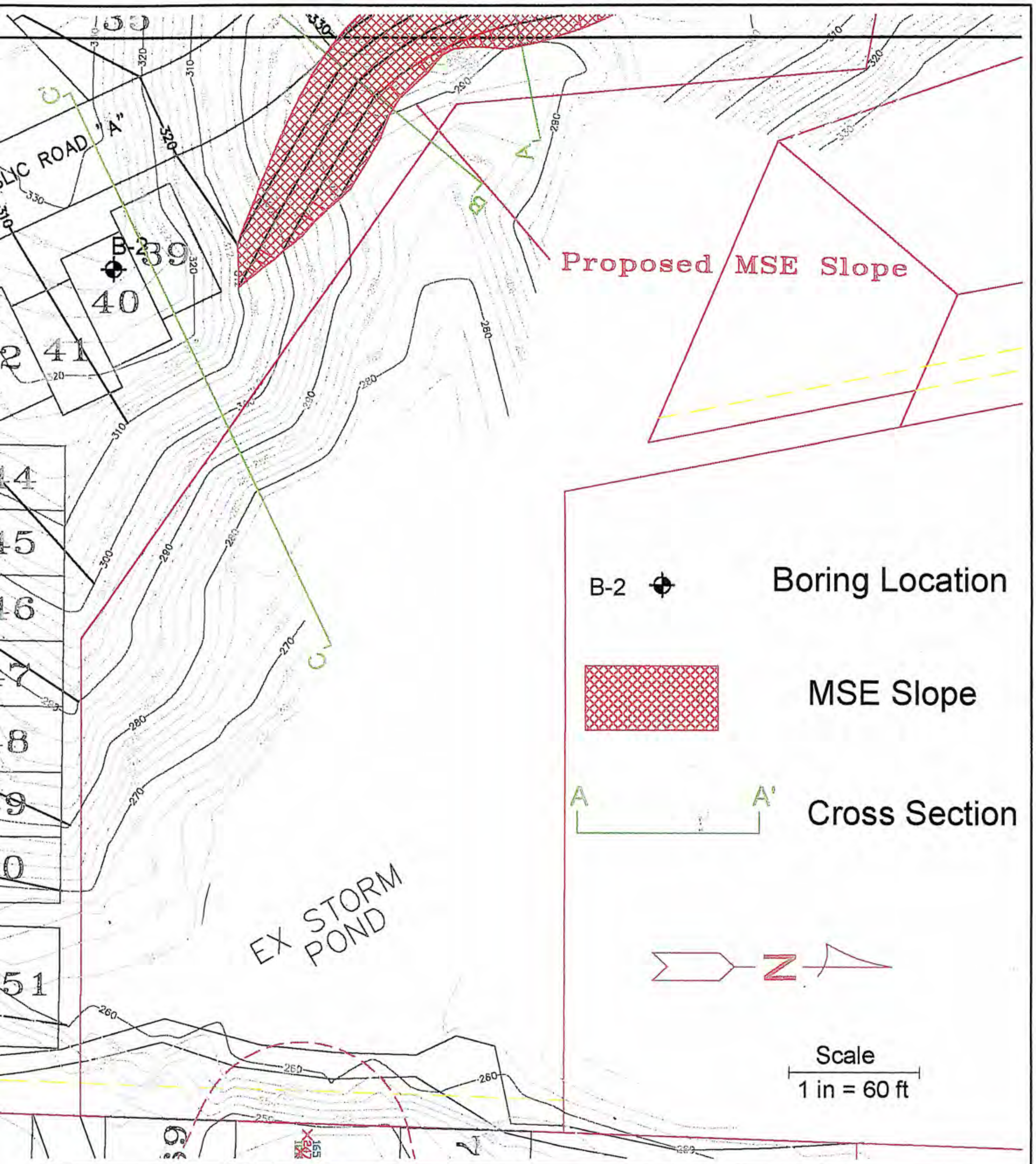
Scale: 1 in = 60 ft	Job Number: 102-03046	Drawn By:	JLM
		Revised By:	
Date: June 9, 2003	Krazan & Associates, Inc.	Drawing Number:	1
		Drawing Type:	Site Map

Permit Number: 20-01254



Site Plan developed from plans provided by Team 4 Engineering, Copper Canyon Site Plan, Dated 3/31/03

Permit Number: 20-01254



Site Plan - East Section

Copper Canyon

Figure 2b

Scale: 1 in = 60 ft

Job Number: 102-03046

Drawn By: JLM

Revised By:

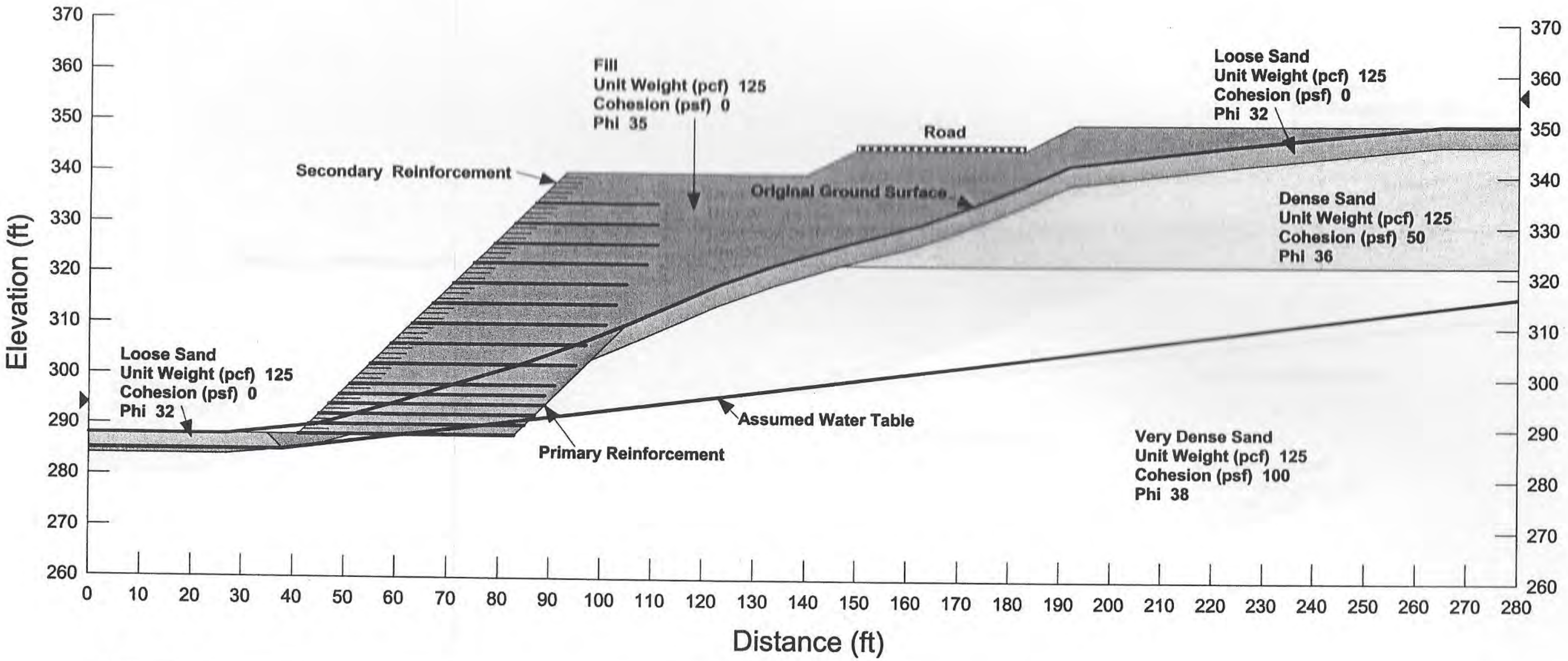
Date: June 9, 2003

Krazan & Associates, Inc.

Drawing Number: 1

Drawing Type: Site Map

Permit Number: 20-01254



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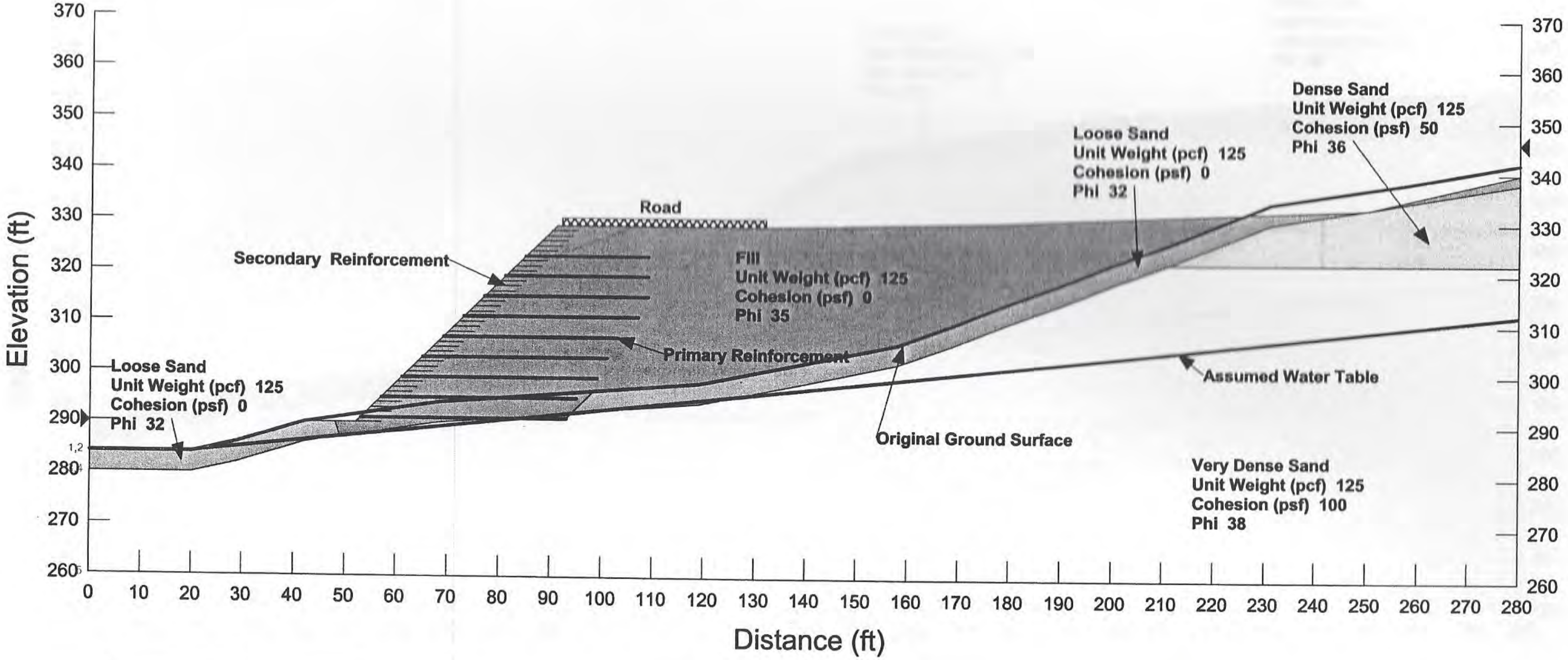
360-598-2126

FIGURE 3 – CROSS SECTION A-A'

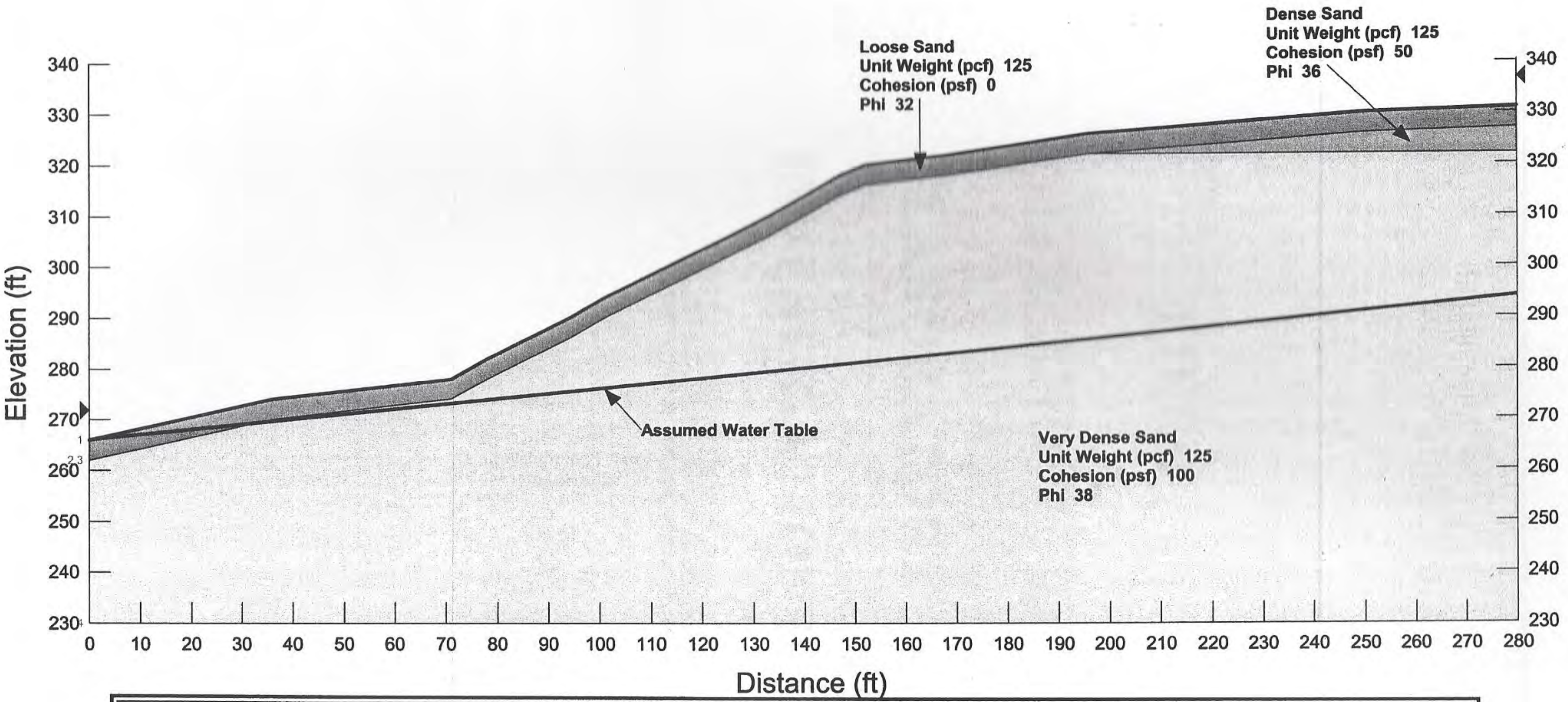
Job name:	Copper Canyon
Location:	Silverdale, Washington
Job No. :	102-03046
Client:	Crescent Investments, LLC
Date:	June 12, 2003

Permit Number: 20-01254



KRAZAN & ASSOCIATES, INC. **20714 State Highway 305 NE, Suite 3C** **POULSBO, WA 98370** **360-598-2126**	**FIGURE 4 – CROSS SECTION B-B'** **Job name:** Copper Canyon **Location:** Silverdale, Washington **Job No. :** 102-03046 **Client:** Crescent Investments, LLC **Date:** June 12, 2003

Permit Number: 20-01254



KRAZAN & ASSOCIATES, INC. 20714 State Highway 305 NE, Suite 3C POULSBO, WA 98370 360-598-2126	**FIGURE 5 – CROSS SECTION C-C'** Job name: Copper Canyon Location: Silverdale, Washington Job No. : 102-03046 Client: Crescent Investments, LLC Date: June 12, 2003

FIELD

AND

LABORATORY

INVESTIGATIONS

Appendix A

APPENDIX A

FIELD AND LABORATORY INVESTIGATIONS

Field Investigation

The field investigation consisted of a surface reconnaissance and a subsurface exploratory program with three borings. The boring locations are shown on the Site Plan, Figure 2. The depths shown on our boring logs are established from the existing ground surface at the time of the subsurface exploration.

The borings were advanced using a limited access track-mounted drill rig. Disturbed soil samples were obtained by using the Standard Penetration Test (SPT) as described in ASTM: D-1586 and relatively undisturbed soil samples were obtained using a California ring-lined sampler as described in ASTM: D-3550. The Standard Penetration Test and sampling method consists of driving a standard 2-inch outside-diameter, split barrel sampler into the subsoil with a 140-pound hammer free falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the Standard Penetration Resistance, or N-value. The resistance, or "N" value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. The California sampling method consists of the same driving methods as the Standard Penetration Test, and is used to obtain relatively undisturbed samples. The blow counts obtained from a California sampler are multiplied by 0.63 to obtain an N-value that is nominally equivalent to the N-value obtained from the Standard Penetration Test. Note that the value presented on the log for California samples is the uncorrected blow count.

The soils encountered were logged in the field during the exploration and, with supplementary laboratory test data, are described in accordance with the Unified Soil Classification System.

All samples were returned to our Poulsbo laboratory for evaluation.

Laboratory Investigation

The laboratory investigation was used to estimate the physical and mechanical properties of the foundation soil underlying the site. In-situ moisture content, fines content, grain size distribution, and direct shear strength tests were performed for samples representative of the subsurface material. These tests, supplemented by visual observation, comprised the basis for our evaluation of the site soil.

The results of the moisture content and the fines content tests are presented on the boring logs. The results of the grain size distribution and direct shear strength tests are presented on individual sheets following the logs.

WinLoG Symbol Legend

USCS

- ⁘ Well Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
- ⁘ Poorly Graded Gravels, Gravel-Sand Mixtures, Little or No Fines
- ▥ Silty Gravels, Gravel-Sand-Silt Mixtures
- ▨ Clayey Gravels, Gravel-Sand-Clay Mixtures
- ⁘ Well Graded Sands, Gravelly Sands, Little or No Fines
- ⁘ Poorly Graded Sands, Gravelly Sands, Little or No Fines
- ▥ Silty Sands, Sand-Silt Mixtures
- ▨ Clayey Sands, Sand-Clay Mixtures
- ▥ Inorganic Silts and Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands
- ▨ Inorganic Clays of Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty
- ▥ Organic Silts and Organic Silty Clays of Low Plasticity
- ▥ Inorganic Silts, Micaceous or Diatomaceous Fine Sandy or Silty Soils, Elastic
- ▨ Inorganic Clays of High Plasticity, Fat Clays
- ▩ Organic Clays of Medium to High Plasticity, Organic Silts
- ▤ Peat, Humus, Swamp and Other Highly Organic Soils
- ☐
- ☐
- ☐

Well Symbols

Pipes and Screens

- NONE None
- Pipe
- Double Walled Pipe
- Sealed Pipe
- Fine Screen
- Coarse Screen
- Screen 1
- Screen 2

Top Fittings

- NONE None
- Cap
- Flush-mount Cap
- Above-ground Cap
- Connector
- Reducer
- Pipe Break
- Packer

Bottom Fittings

- NONE None
- Cap
- Cone
- Screw-on Cap
- Connector
- Enlarger
- Pipe Break
- Packer

Packing and Backfill

- NONE None
- ▧ Bentonite
- ▧ Clay
- ▤ Silt
- ■ Cement
- ⁘ Sand
- ⁘ Sand and Gravel
- ⁘ Gravel

Sample Symbols

- Split Spoon
- Auger
- Core
- G Grab
- Shelby Tube
- ▩ Excavation
- ■ Undisturbed
- ◢ No Recovery

Log of Boring B-1

Project: Copper Canyon

Client: Crescent Investments LLC

Location: Kitsap County, WA

Depth to Water: ~15.5' during drilling, ~7' on removal

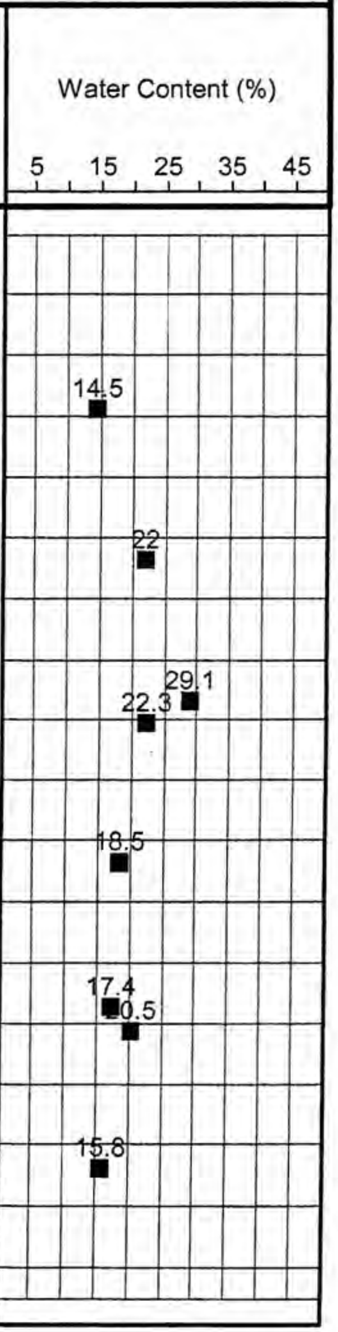
Project No: 102-03046

Figure No.: A-1

Logged By: D.H.

Elevation: ~280 Feet.

SUBSURFACE PROFILE			SAMPLE				
Depth (ft)	Symbol	Description	Sample Number	Dry Density (pcf)	Fines (%)	Type	N-Value (Blows/Ft.)
0		Ground Surface					
		POORLY GRADED SAND WITH SILT (SP-SM) Loose, reddish brown, moist. (TOPSOIL/DISTURBED AREA FOR DRILLING PAD)					
5		***POORLY GRADED SAND WITH SILT (SP-SM)*** Medium dense, tan to gray, very moist to wet.	S-1			SS	16
10		***SILTY SAND (SM)*** Dense, fine grained sand, gray, moist to wet.	S-2		39	SS	35
15		***POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)*** Dense, gray, wet. Contains silt lenses.	S-3A S-3B			SS SS	46 46
20		Becomes very dense at 20 feet.	S-4			SS	52
25			S-5A S-5B		6 45	SS SS	19\50:5.5" 19\50:5.5"
30			S-6		6	SS	17\50:4"
35							



Method: Track HSA

Driller: Davies

Operator: Jeff Davies

Krazan and Associates
20714 State Highway 305 N.E.
Suite 3C
Poulsbo, Washington 98370

Drill Date: 3/22/03

Sample Method: SPT, California

Sheet: 1 of 2

Permit Number: 20-01254

Log of Boring B-1

Project: Copper Canyon

Client: Crescent Investments LLC

Location: Kitsap County, WA

Depth to Water: ~15.5' during drilling, ~7' on removal

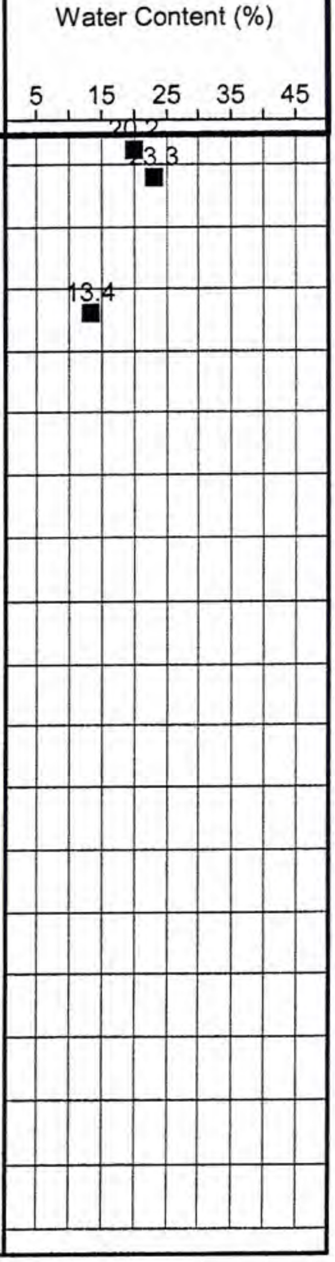
Project No: 102-03046

Figure No.: A-1

Logged By: D.H.

Elevation: ~280 Feet.

SUBSURFACE PROFILE			SAMPLE				
Depth (ft)	Symbol	Description	Sample Number	Dry Density (pcf)	Fines (%)	Type	N-Value (Blows/Ft.)
	▨		S-7A			SS	23\50:4"
	▨		S-7B		95	SS	23\50:4"
40	▨		S-8		7	SS	39\50:3"
		End of Boring Boring collapsed at 14 feet on removal of auger.					
45							
50							
55							
60							
65							
70							



Method: Track HSA

Driller: Davies

Operator: Jeff Davies

Krazan and Associates
20714 State Highway 305 N.E.
Suite 3C
Poulsbo, Washington 98370

Drill Date: 3/22/03

Sample Method: SPT, California

Sheet: 2 of 2

Permit Number: 20-01254

Log of Boring B-2

Project: Copper Canyon

Client: Crescent Investments LLC

Location: Kitsap County, WA

Depth to Water: ~52.5' durring drilling

Project No: 102-03046

Figure No.: A-2

Logged By: D.H.

Elevation: ~327 Feet.

SUBSURFACE PROFILE			SAMPLE					
Depth (ft)	Symbol	Description	Sample Number	Dry Density (pcf)	Fines (%)	Type	N-Value (Blows/Ft.)	Water Content (%)
		Ground Surface						
0		***POORLY GRADED SAND WITH SILT (SP-SM)*** Loose, reddish brown, moist. (TOPSOIL/DISTURBED AREA FOR DRILLING PAD)	S-0			Grab		14.7
		SILTY SAND (SM) Very dense, fine grained sand, gray, moist. Contains gravel.						
5			S-1			SS	60	10
10			S-2		30	SS	57	8.5
15			S-3			SS	75	9
20			S-4			SS	76	6
		POORLY GRADED SAND WITH SILT (SP-SM) Very dense, fine grained sand, brown, moist to very moist.						
25			S-5			SS	51	11.8
30			S-6		6	SS	36\50:5"	7.3
35								

Water Content (%): 5 15 25 35 45

Method: Track HSA

Driller: Davies

Operator: Jeff Davies

Krazan and Associates
20714 State Highway 305 N.E.
Suite 3C
Poulsbo, Washington 98370

Drill Date: 3/22/03 - 3/23/03

Sample Method: SPT, California

Sheet: 1 of 2

Permit Number: 20-01254

Log of Boring B-2

Project: Copper Canyon

Client: Crescent Investments LLC

Location: Kitsap County, WA

Depth to Water: ~52.5' durring drilling

Project No: 102-03046

Figure No.: A-2

Logged By: D.H.

Elevation: ~327 Feet.

SUBSURFACE PROFILE			SAMPLE					
Depth (ft)	Symbol	Description	Sample Number	Dry Density (pcf)	Fines (%)	Type	N-Value (Blows/Ft.)	Water Content (%) 5 15 25 35 45
			S-7			Calif.	74\110:5"	9.9
			S-8			SS	69	8.8
40			S-9			SS	82	8.1
45		*SILTY SAND (SM)* Very dense, brown, wet.	S-10		20	SS	81	14.6
50			S-11			SS	85	16.7
55			S-12			SS	38\50:5"	24
60			S-13		56	SS	67	25.8
		End of Boring						
65		Boring collapsed at 35 feet on removal of auger.						
70								

Method: Track HSA

Driller: Davies

Operator: Jeff Davies

Krazan and Associates
20714 State Highway 305 N.E.
Suite 3C
Poulsbo, Washington 98370

Drill Date: 3/22/03 - 3/23/03

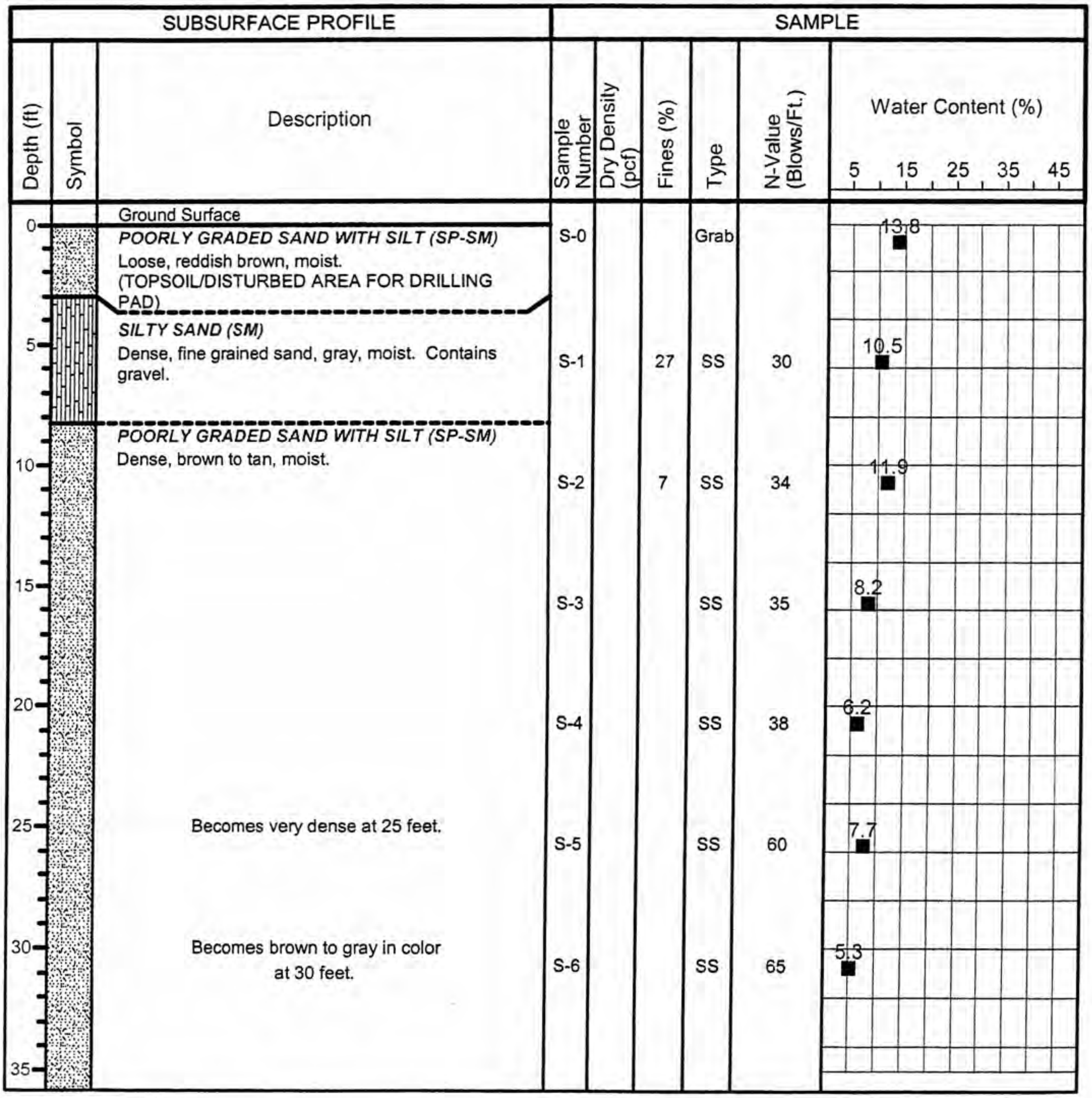
Sample Method: SPT, California

Sheet: 2 of 2

Log of Boring B-3

Project: Copper Canyon
Client: Crescent Investments LLC
Location: Kitsap County, WA
Depth to Water: Not encountered

Project No: 102-03046
Figure No.: A-3
Logged By: D.H.
Elevation: ~340 Feet.



Description	Sample Number	Dry Density (pcf)	Fines (%)	Type	N-Value (Blows/Ft.)	Water Content (%)
Ground Surface						
POORLY GRADED SAND WITH SILT (SP-SM) Loose, reddish brown, moist. (TOPSOIL/DISTURBED AREA FOR DRILLING PAD)	S-0			Grab		13.8
SILTY SAND (SM) Dense, fine grained sand, gray, moist. Contains gravel.	S-1		27	SS	30	10.5
POORLY GRADED SAND WITH SILT (SP-SM) Dense, brown to tan, moist.	S-2		7	SS	34	11.9
	S-3			SS	35	8.2
	S-4			SS	38	6.2
Becomes very dense at 25 feet.	S-5			SS	60	7.7
Becomes brown to gray in color at 30 feet.	S-6			SS	65	5.3

Method: Track HSA
Driller: Davies
Operator: Jeff Davies

Krazan and Associates
20714 State Highway 305 N.E.
Suite 3C
Poulsbo, Washington 98370

Drill Date: 3/23/03
Sample Method: SPT
Sheet: 1 of 2

Permit Number: 20-01254

Log of Boring B-3

Project: Copper Canyon

Client: Crescent Investments LLC

Location: Kitsap County, WA

Depth to Water: Not encountered

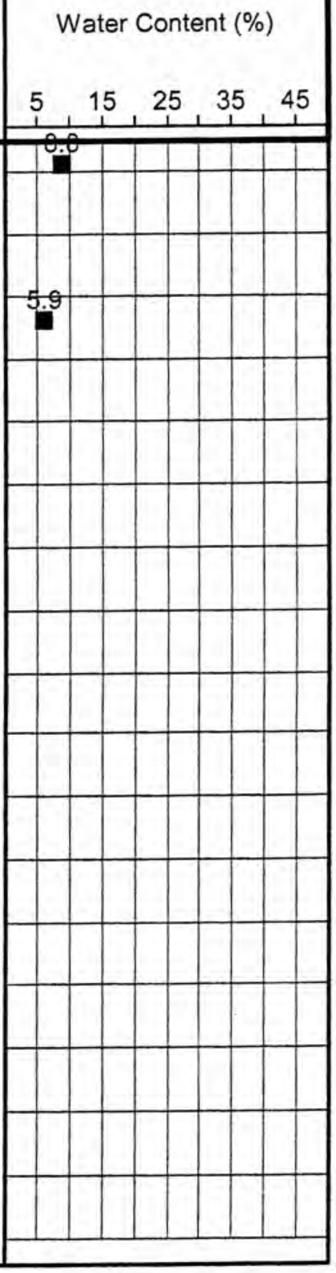
Project No: 102-03046

Figure No.: A-3

Logged By: D.H.

Elevation: ~340 Feet.

SUBSURFACE PROFILE			SAMPLE				
Depth (ft)	Symbol	Description	Sample Number	Dry Density (pcf)	Fines (%)	Type	N-Value (Blows/Ft.)
	▨		S-7			SS	53
40	▨		S-8			SS	69
		End of Boring					
45							
50							
55							
60							
65							
70							



Method: Track HSA

Driller: Davies

Operator: Jeff Davies

Krazan and Associates
20714 State Highway 305 N.E.
Suite 3C
Poulsbo, Washington 98370

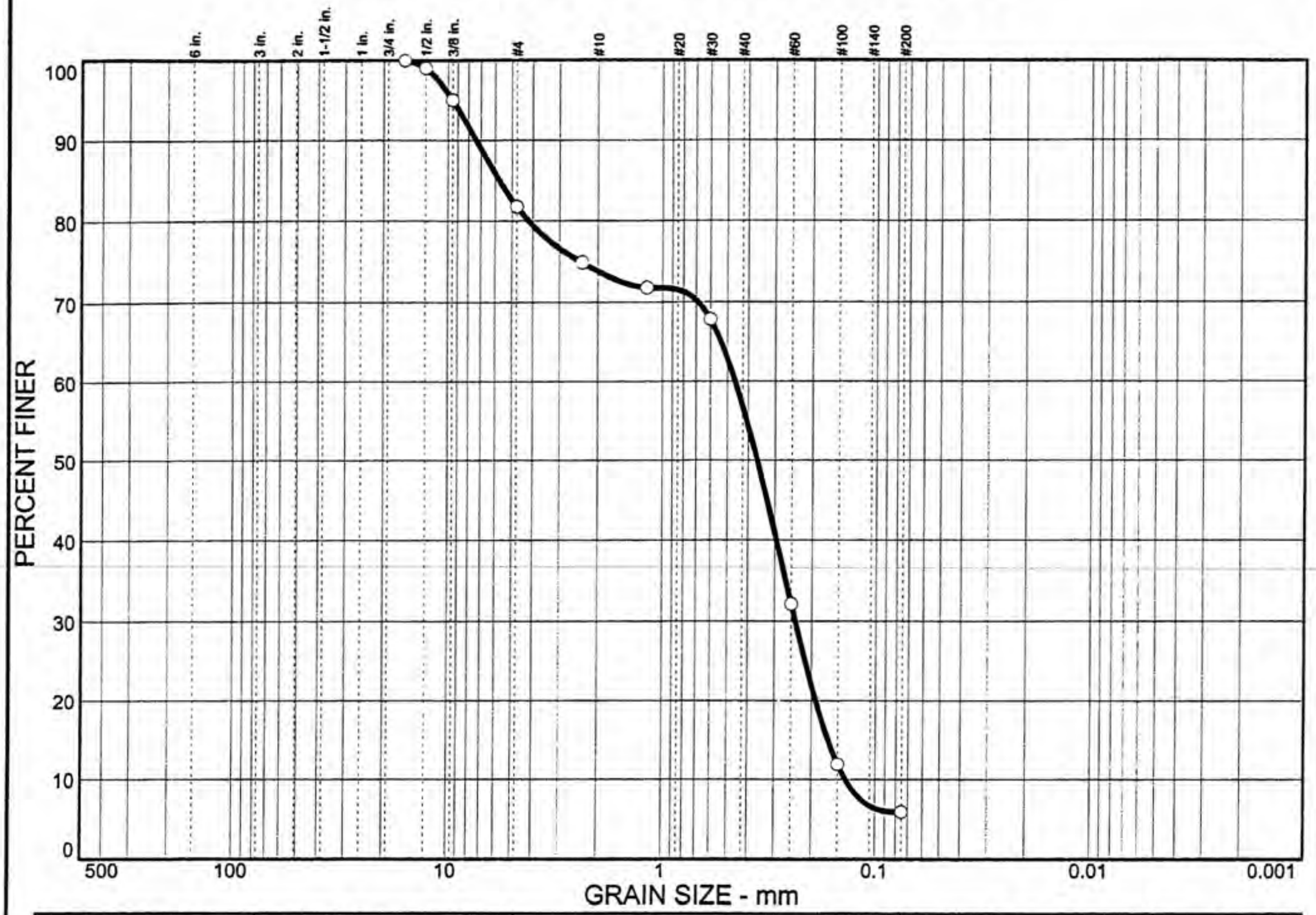
Drill Date: 3/23/03

Sample Method: SPT

Sheet: **2 of 2**

Permit Number: **20-01254**

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	18.2	75.9	5.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.625 in.	100.0		
0.5 in.	99.0		
0.375 in.	95.0		
#4	81.8		
#8	74.9		
#16	71.8		
#30	67.8		
#60	32.1		
#100	11.9		
#200	5.9		

* (no specification provided)

Soil Description

USCS: POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 5.71 D_{60}= 0.461 D_{50}= 0.363
D_{30}= 0.239 D_{15}= 0.167 D_{10}= 0.138
C_u= 3.34 C_c= 0.90

Classification

USCS= SP-SM AASHTO=

Remarks

SAMPLE #; P4832
REPORT #: 10072
DATE: 5/27/2003

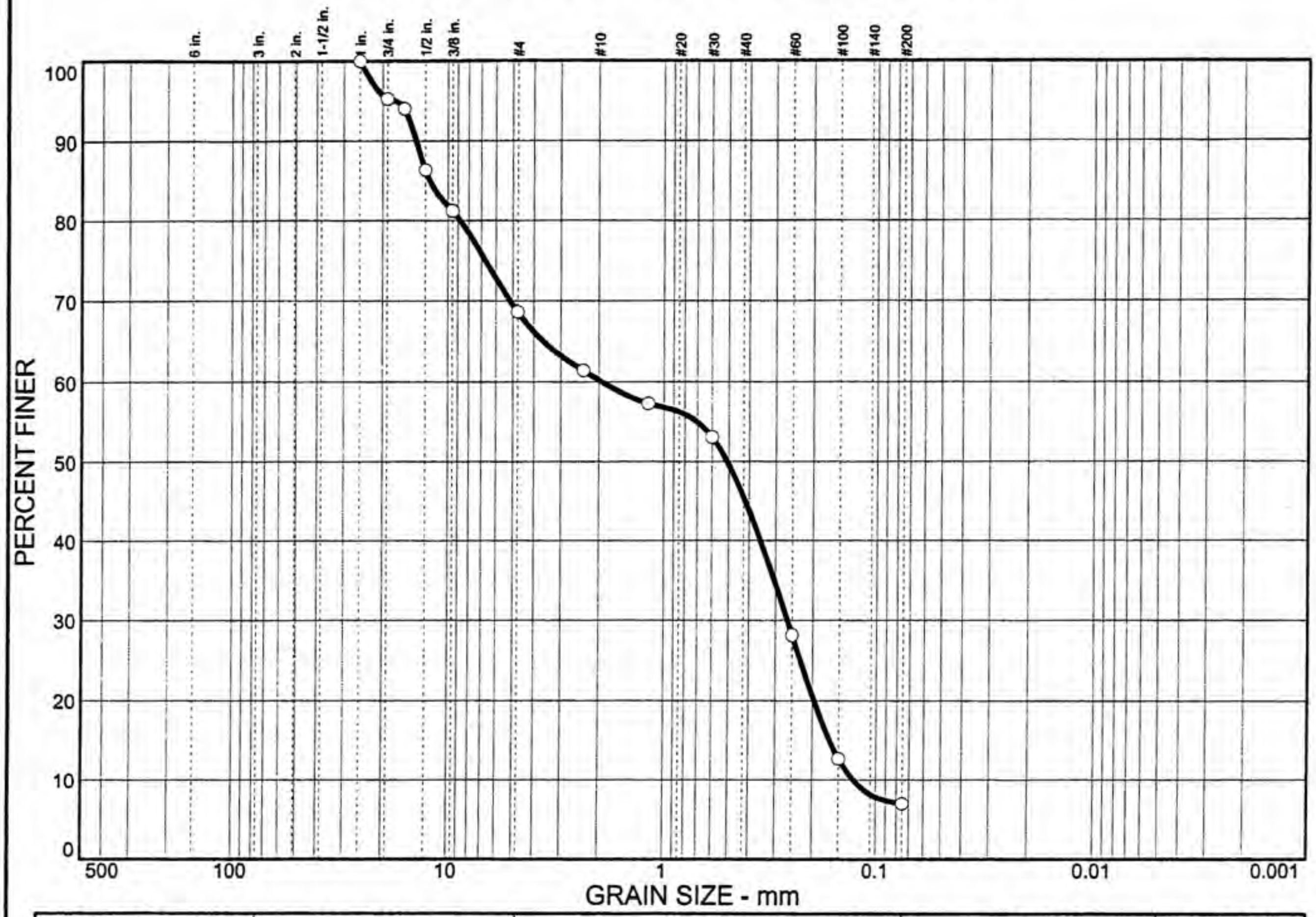
Sample No.: P4832 **Source of Sample:** BORING 1 **Date:** 5/27/2003
Location: B-1,S-6 **Elev./Depth:**

KRAZAN & ASSOCIATES, INC.

Client: CRESCENT INVESTMENTS, LLC
Project: COPPER CANYON
Project No: 102-03046 **FIGURE** A-4

Permit Number: 20-01254

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	31.4	61.6	7.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0 in.	100.0		
0.75 in.	95.2		
0.625 in.	94.0		
0.5 in.	86.4		
0.375 in.	81.2		
#4	68.6		
#8	61.4		
#16	57.3		
#30	53.1		
#60	28.2		
#100	12.7		
#200	7.0		

* (no specification provided)

Soil Description

USCS: POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 12.1 D_{60}= 1.94 D_{50}= 0.509
D_{30}= 0.263 D_{15}= 0.165 D_{10}= 0.128
C_u= 15.16 C_c= 0.28

Classification

USCS= SP-SM AASHTO=

Remarks

SAMPLE #: P4832
REPORT #: 10072
DATE: 5/27/2003

Sample No.: P4832, B-1
Location: B-1,S-8
Source of Sample: BORING 1
Date: 5/27/2003
Elev./Depth:

KRAZAN & ASSOCIATES, INC.

Client: CRESCENT INVESTMENTS, LLC
Project: COPPER CANYON
Project No: 102-03046 **FIGURE** A-5

Particle Size Distribution Report

% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	8.4	61.9	29.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.625 in.	100.0		
0.5 in.	98.7		
0.375 in.	96.0		
#4	91.6		
#8	87.8		
#16	84.5		
#30	80.3		
#60	58.9		
#100	40.0		
#200	29.7		

* (no specification provided)

Soil Description

USCS: SILTY SAND (SM)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.33	D_{60}= 0.258	D_{50}= 0.199
D_{30}= 0.0779	D_{15}=	D_{10}=
C_u=	C_c=	

Classification

USCS= SM AASHTO=

Remarks

SAMPLE #; P4833
REPORT #; 10072
DATE: 5/27/2003

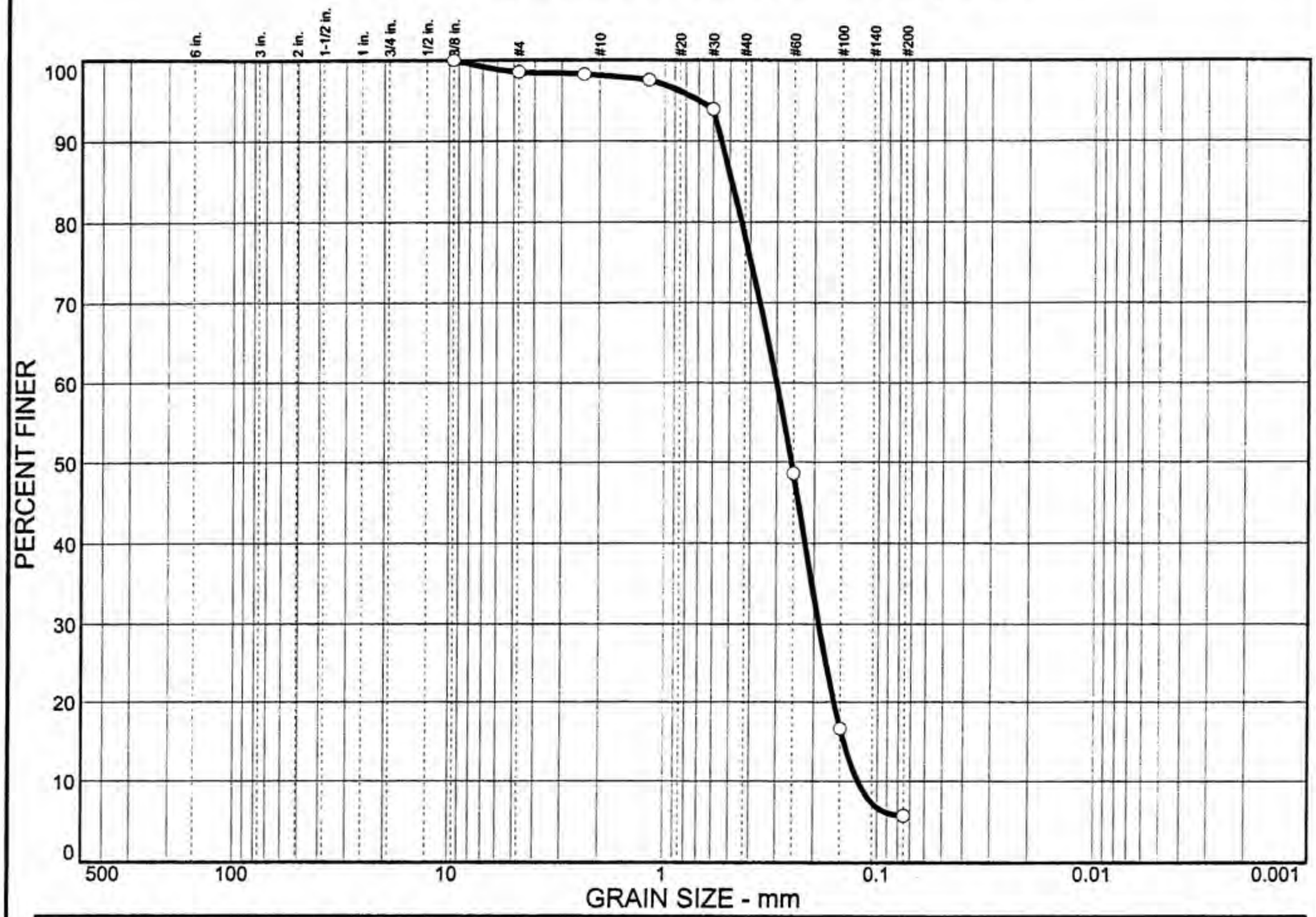
Sample No.: P4833 **Source of Sample:** BORING 2 **Date:** 5/27/2003
Location: B-2,S-2 **Elev./Depth:**

KRAZAN & ASSOCIATES, INC.

Client: CRESCENT INVESTMENTS, LLC
Project: COPPER CANYON
Project No: 102-03046 **FIGURE** A-6

Permit Number: 20-01254

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.5	92.7	5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.5		
#8	98.2		
#16	97.5		
#30	93.9		
#60	48.6		
#100	16.6		
#200	5.8		

* (no specification provided)

Soil Description

USCS:POORLY GRADED SAND WITH SILT (SP-SM)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.491 D_{60}= 0.301 D_{50}= 0.255
D_{30}= 0.190 D_{15}= 0.144 D_{10}= 0.123
C_u= 2.44 C_c= 0.97

Classification

USCS= SP-SM AASHTO=

Remarks

SAMPLE #; P4833
REPORT #; 10072
DATE: 5/27/2003

Sample No.: P4833,B-2 **Source of Sample:** BORING 2 **Date:** 5/27/2003
Location: B-2,S-6 **Elev./Depth:**

KRAZAN & ASSOCIATES, INC.

Client: CRESCENT INVESTMENTS, LLC
Project: COPPER CANYON

Project No: 102-03046 **FIGURE** A-7

Permit Number: 20-01254

Direct Shear of Consolidated, Drained Soils
ASTM D - 3080 / AASHTO T - 236

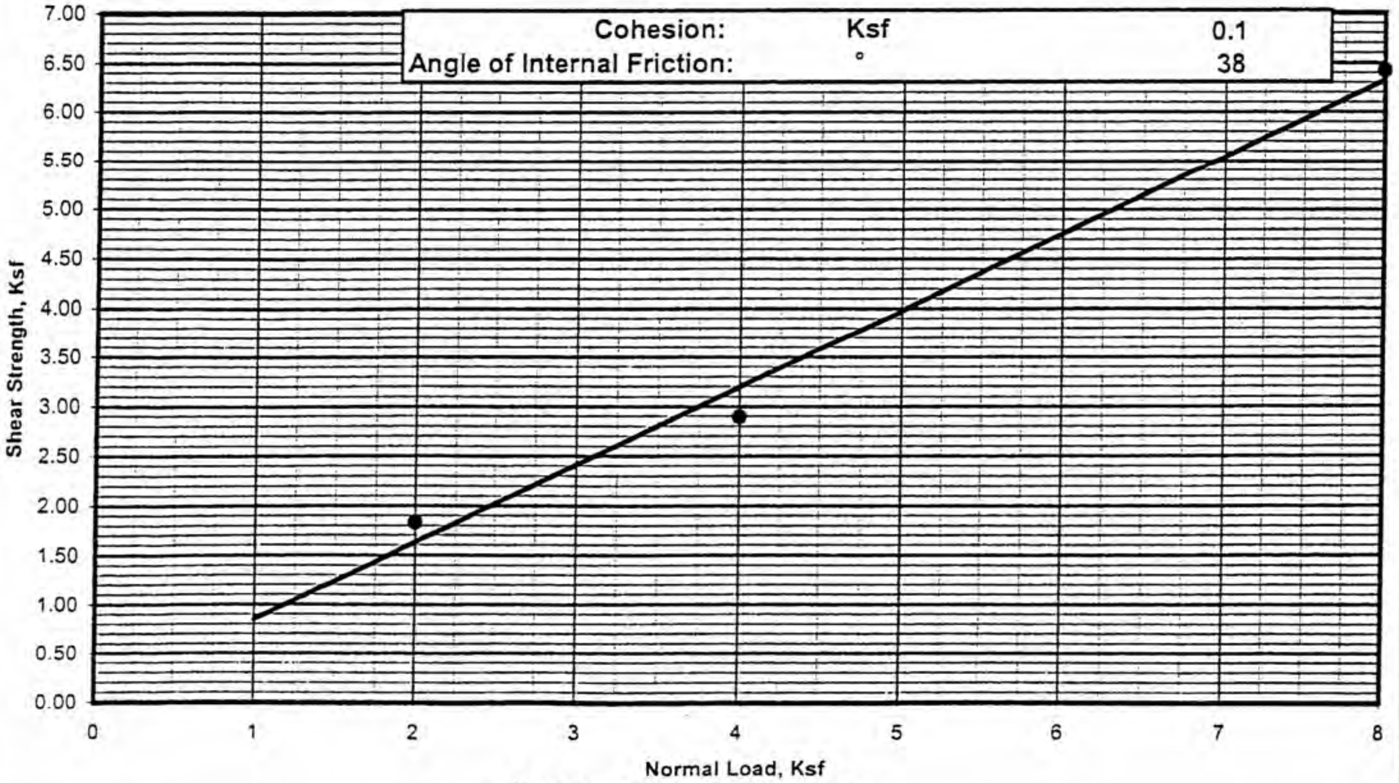
Project Number	: 10203046
Project Name	: Copper Canyon
Date	: 5/23/03
Sample Location	: B-2/S-7
Soil Classification	: SP
Sample Surface Area	: 0.03168

STRESS DISPLACEMENT DATA

Lat. Disp. (in.)	Normal Load		
	2000	4000	8000
0	0	0	0
0.030	100	184	338
0.060	152	228	430
0.090	180	261	464
0.120		284	479
0.150		289	482
0.180			
0.210			
0.240			
0.270			
0.300			
0.330			
0.360			

Normal Load psf	Shear force lbs	Shear Stress psf
2	58.0	1831
4	91.7	2894
8	203.5	6425

Specimen Information		
	Initial	Final
Diameter (in):	2.5	2.5
Thickness (in):	1	1
Moisture Content	9.88%	26.00%
Wet Density (pcf):	133.85	133.85
Dry Density (pcf):	106.23	121.82



Krazan and Associates, Inc.
19501 144th Ave. NE #F-300
Woodinville, Washington
98072

Figure A-8

Permit Number: 20-01254

SLOPE

STABILITY

ANALYSES

Appendix B

APPENDIX B

SLOPE STABILITY ANALYSES

Slope stability analyses were performed on three cross-sections; two within the taller MSE slope and one on a ravine slope that will not be re-graded. The locations of the cross-sections are indicated on the Site Plan, Figure 2. Topography used in the analysis was based on the Copper Canyon Site Plan, dated March 31, 2003 prepared by Team 4 Engineering. The slope stability computer program Slope/W by GeoSlope International was used to evaluate the stability of the existing slopes and proposed MSE slopes under static and seismic conditions. Soil strength parameters used in our analysis were based on in-situ penetration tests, laboratory shear strength tests and published values. The engineering properties of the soil used in our analysis are presented on Figure 3, Cross Sections A-A', Figure 4, Cross Section B-B', and Figure 5 Cross Section C-C'. Cross Section A-A' represents the maximum height of MSE slope on the site, Cross Section B-B' represents the closest approach of the access road to the MSE slope, and Cross Section C-C' represents a relatively steep section of native slope. Water levels used in the stability analyses were conservatively assumed to be higher than the water level encountered in our borings.

The psuedostatic method was used for our slope stability analyses to estimate the factor of safety under seismic conditions. The United States Geologic Survey, Earthquake Hazards Program – National Seismic Hazard Mapping Project, indicates that a peak ground acceleration (PGA) of 0.32 g has a 10 percent probability of exceedence in 50 years (500 year return period). The seismic coefficient is typically taken to be ½ of the PGA. A seismic coefficient of 0.16 was used in our analyses.

The results of slope stability analyses are expressed as factors-of-safety against rotational failure. The factor-of-safety is the ratio of driving forces to resisting forces. A factor-of-safety of 1.0 is equilibrium; a factor-of-safety of less than 1.0 indicates failure. Typically, a factor-of-safety of 1.5 for static conditions and 1.1 for seismic conditions is considered adequate. Factors of safety greater than 1 but less than 1.5 (or 1.1) are not adequate due to the uncertainties inherent in the modeling process. A lower safety factor for seismic conditions is considered adequate as the probability of occurrence of the seismic conditions analyzed is relatively low. The calculated minimum factor of safety for each slope is presented on the figures in this appendix.

Two figures are provided for each cross section; one for static conditions and one for seismic conditions.

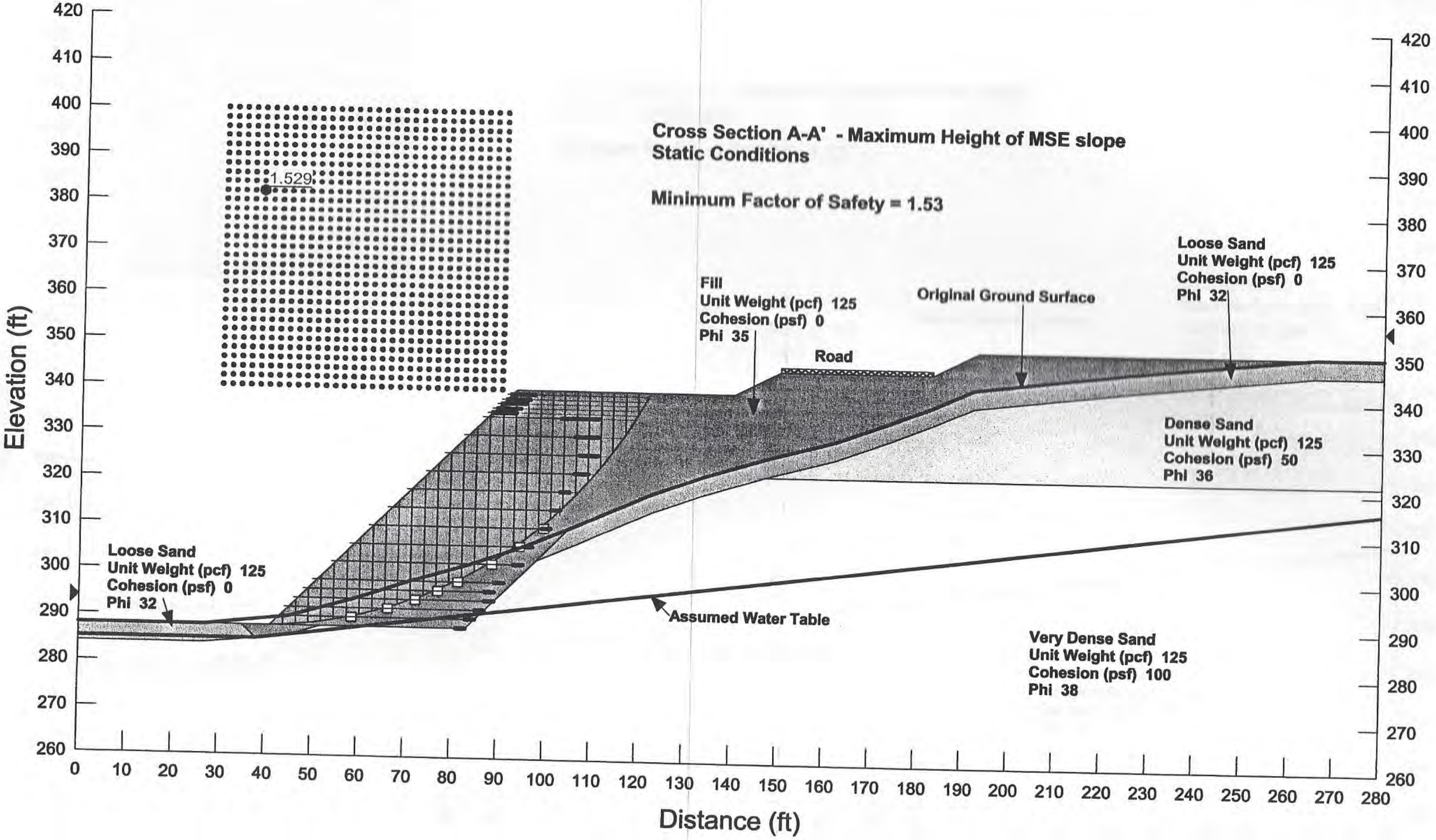


Figure B-1

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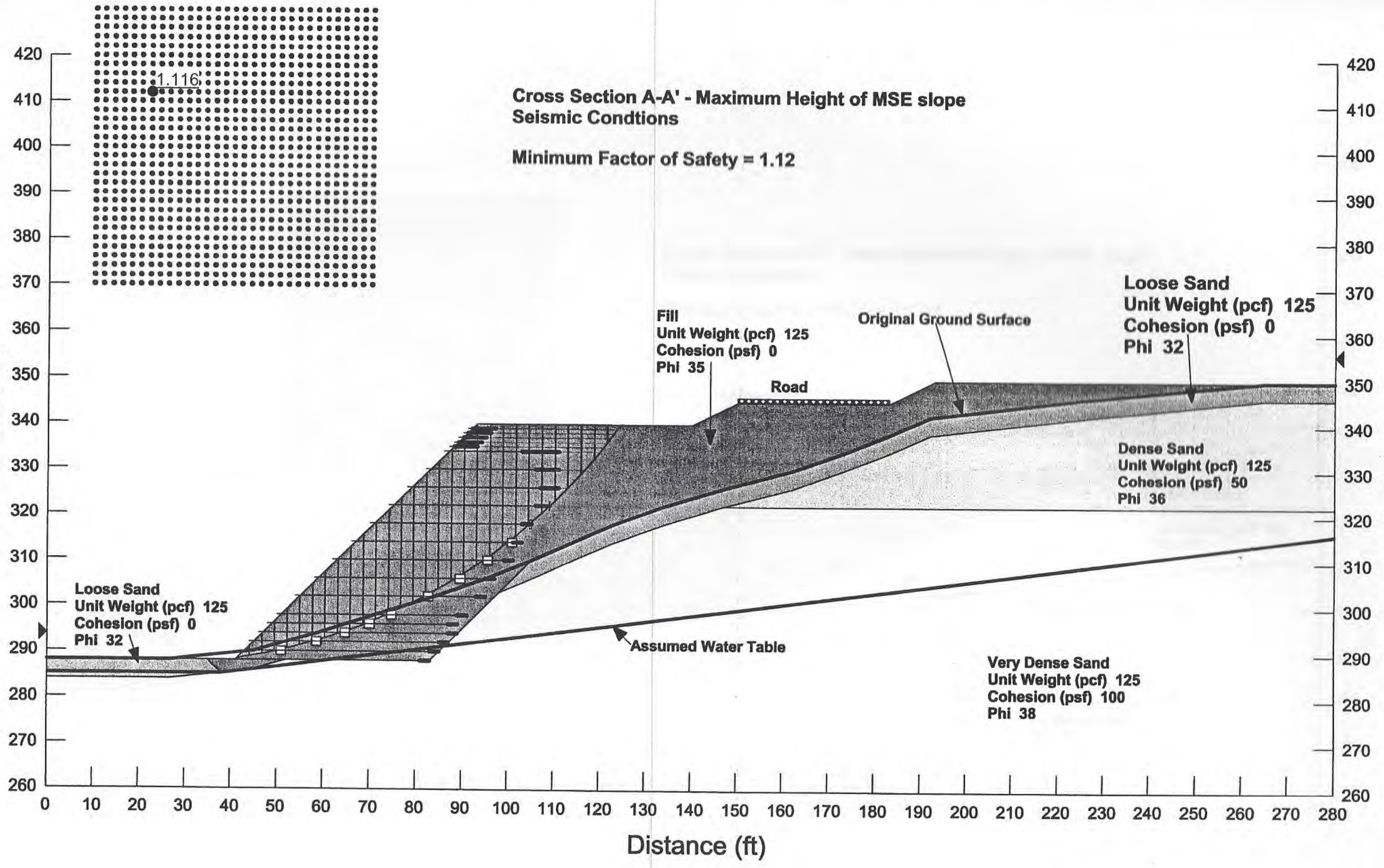


Figure B-2

Permit Number: 20-01254

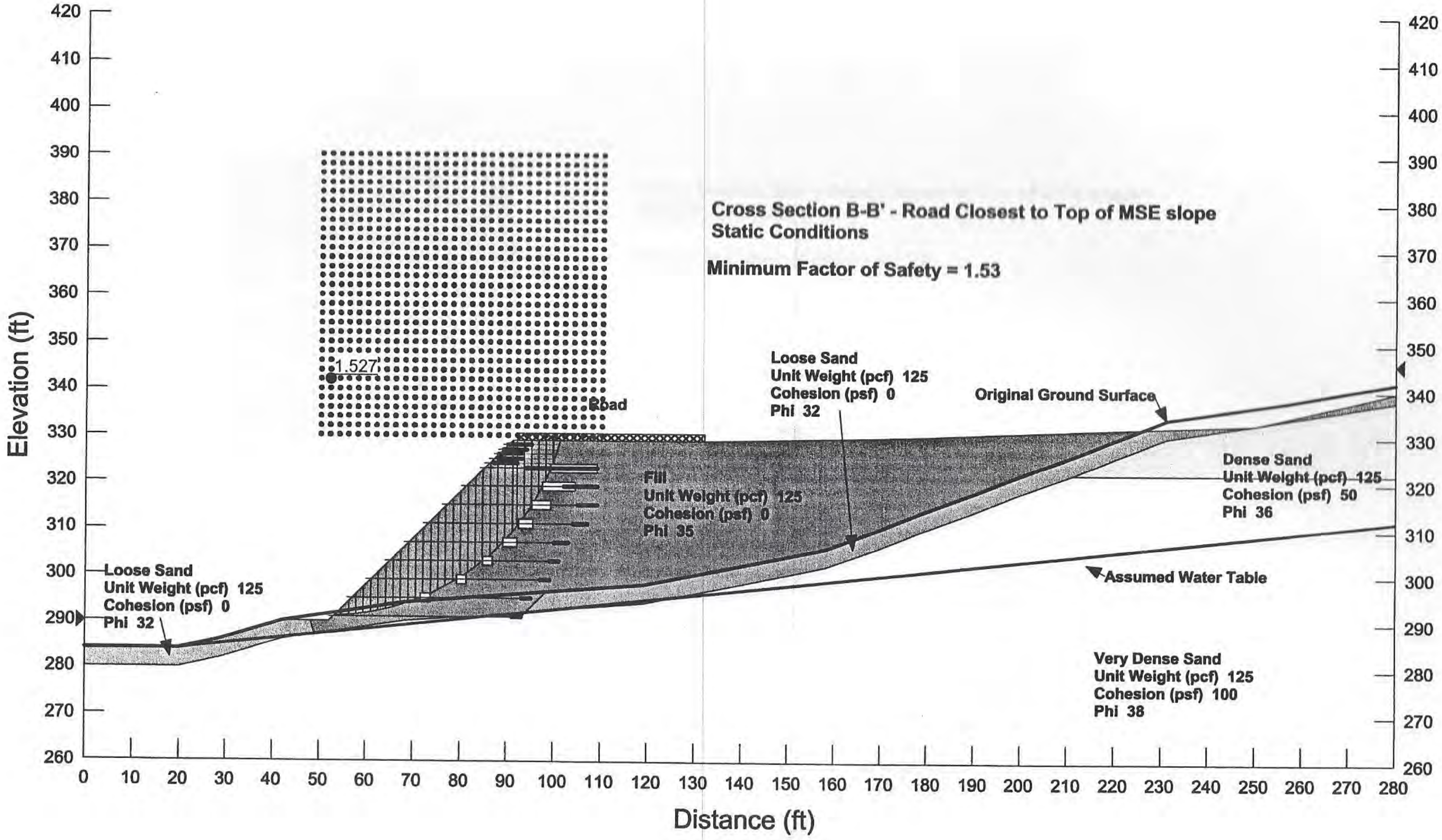

Cross Section B-B' - Road Closest to Top of MSE slope
Static Conditions
Minimum Factor of Safety = 1.53
1.527
Road
Loose Sand
Unit Weight (pcf) 125
Cohesion (psf) 0
Phi 32
Original Ground Surface
Fill
Unit Weight (pcf) 125
Cohesion (psf) 0
Phi 35
Dense Sand
Unit Weight (pcf) 125
Cohesion (psf) 50
Phi 36
Assumed Water Table
Loose Sand
Unit Weight (pcf) 125
Cohesion (psf) 0
Phi 32
Very Dense Sand
Unit Weight (pcf) 125
Cohesion (psf) 100
Phi 38
Elevation (ft)
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
270
260
Distance (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
260
270
280

Figure B-3

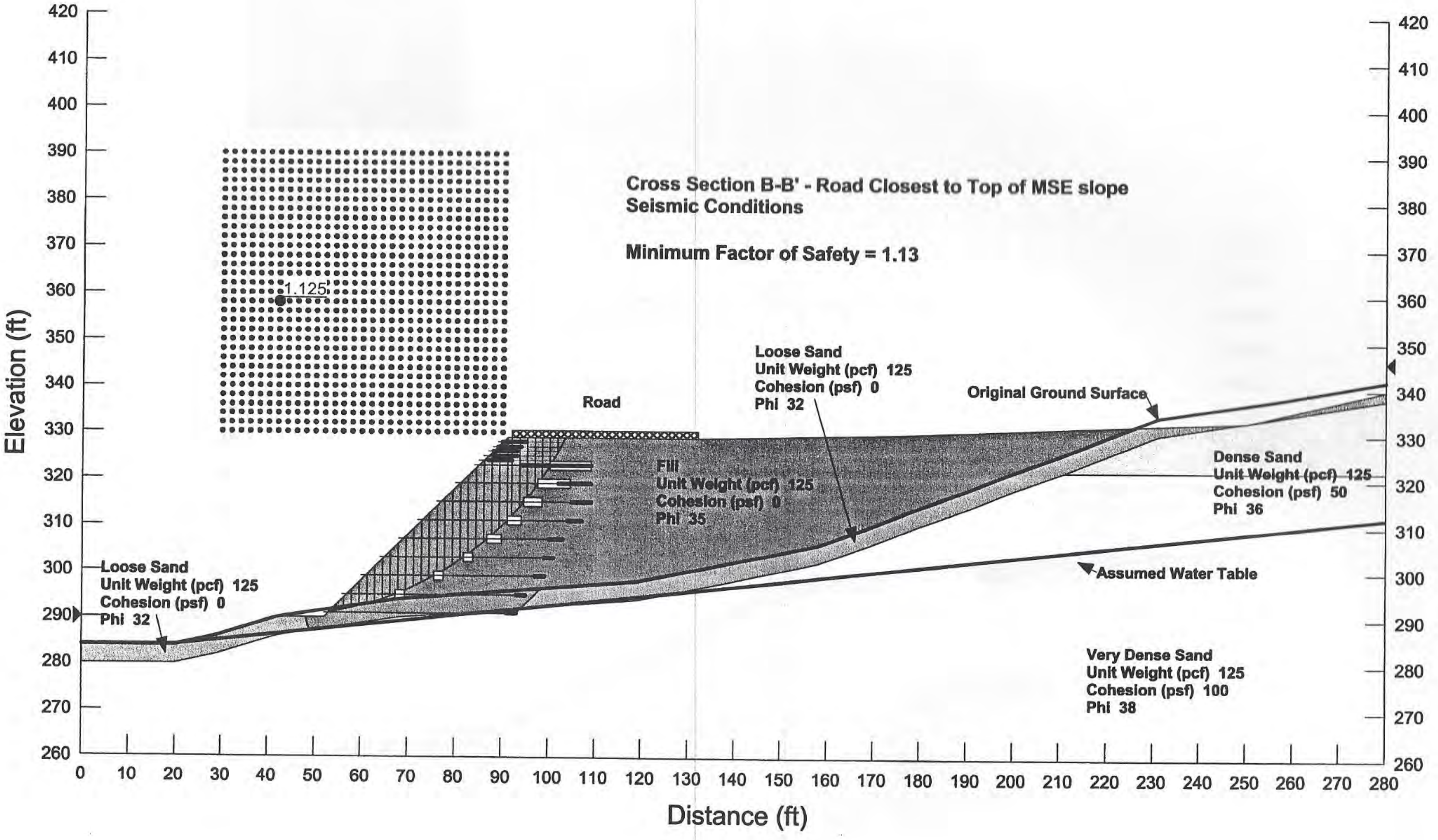
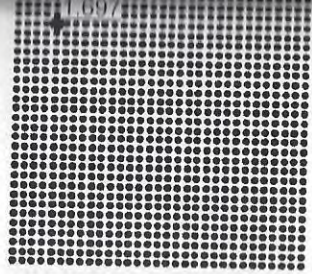
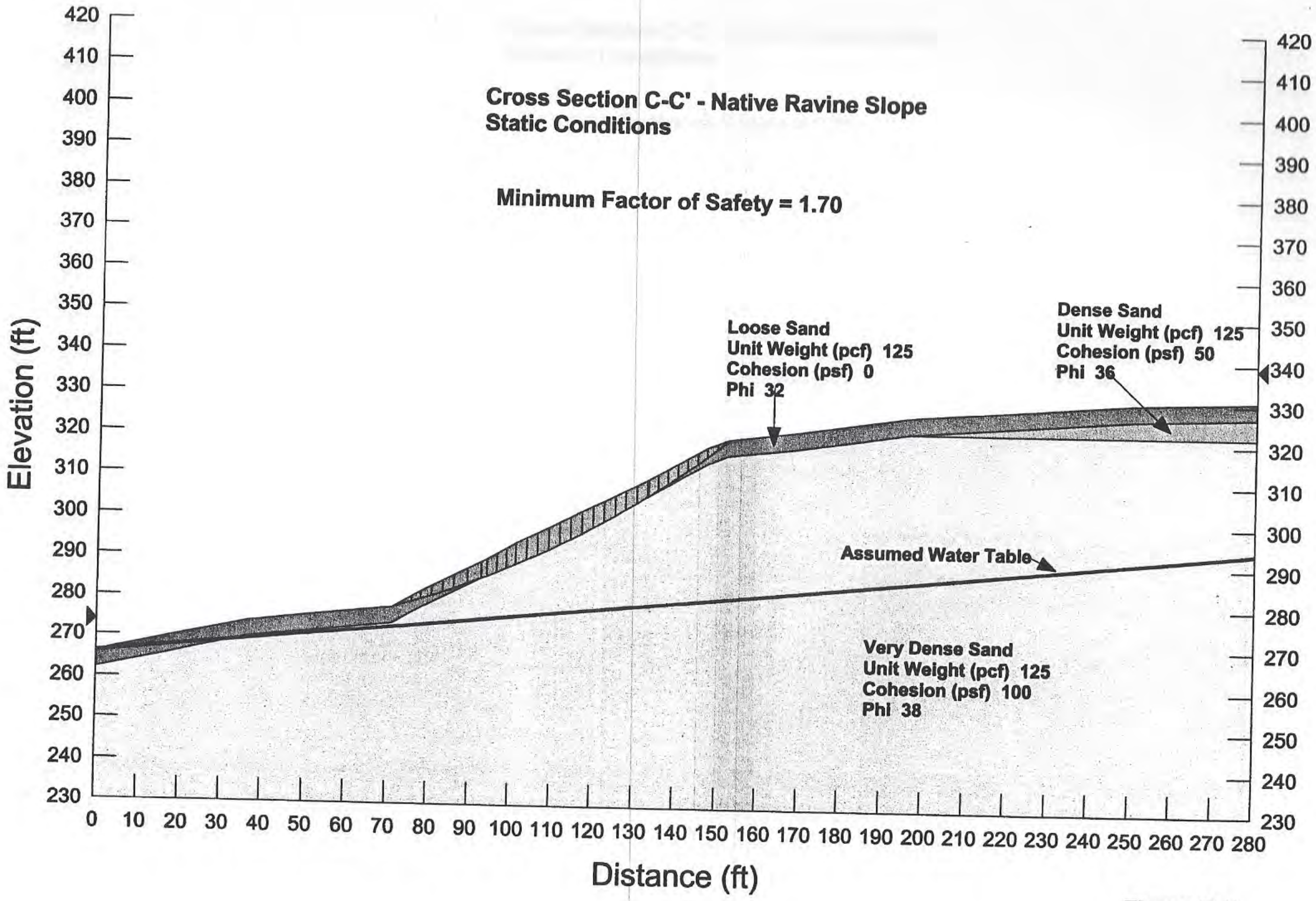


Figure B-4

Permit Number: 20-01254

Permit Number: 20-01254

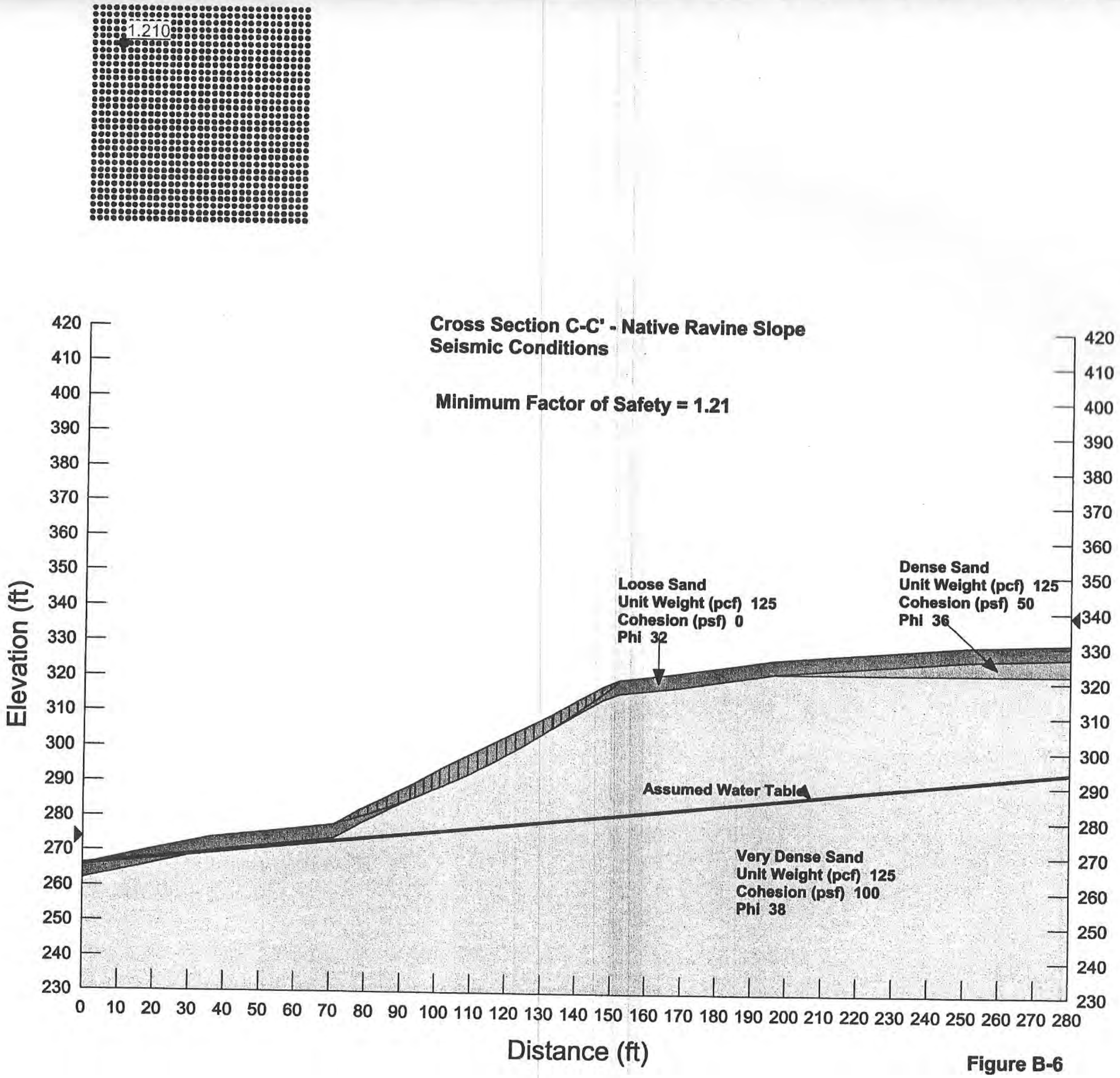




Cross Section C-C' - Native Ravine Slope
Static Conditions

Minimum Factor of Safety = 1.70

Figure B-5



Cross Section C-C' - Native Ravine Slope
Seismic Conditions

Minimum Factor of Safety = 1.21

Figure B-6

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