# GEOTECHNICAL ENGINEERING STUDY
# FINAL DESIGN REPORT
# TOLT BRIDGE NO. 1834A REPLACEMENT
# KING COUNTY, WASHINGTON

Prepared For
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Project No.: KC-300A
September 2005

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## GEOTECHNICAL ENGINEERING STUDY

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## INTRODUCTION

This final report presents the results of a final design geotechnical engineering study conducted for the Tolt Bridge No. 1834A Replacement project. This report supersedes our previous draft reports dated March 1997, September 2003 and April 2005. This report is structured as follows:



## EXECUTIVE SUMMARY

The subsurface conditions at the bridge site are typical of alluvial deposits, consisting of near-surface, soft, compressible soils and loose granular soils, underlain by stiffer and denser deposits. The alluvial nature of the onsite soils impacts the foundation design of Piers 2 through 6. In our opinion, the loose sand and soft compressible clay are unsuitable for design of shallow foundations to support the proposed structural loading. The use of deep foundations is recommended to obtain sufficient frictional and end-bearing support for the proposed loads. Design recommendations for drilled shafts are provided in subsequent report sections.

Due to the presence of existing fill soils and potential voids in the fill as encountered in boring B-3, a minimum foundation embedment of 15 feet below existing grade is recommended at Pier 1. We recommend that drilled shafts be used to support Pier 1. Retaining walls with embedded shallow foundations can be used to support the proposed 10-foot high approach fill to the west abutment.

The soft, compressible clay encountered in our explorations near Pier 6 will settle under the east approach fill loading. Settlements along the proposed retaining walls range from a few inches to one foot and are expected to occur over a 4 to 5 month period after the fill is in place. The placement of the proposed fill requires the use of stabilizing berms to support the fill while settlement occurs and the underlying, soft clays gain strength under the fill loading. Stabilizing berms are recommended on the south side of the approach fill from Stations 40+66 to 41+25 and wrapped around to the centerline at Station 40+66.

We understand that there is sufficient time in the project schedule to allow settlement of the fill to occur. Therefore, alternative subgrade improvement techniques or design modifications such as the use of lightweight fill, stone columns or GeoPier system are not currently being considered.

## PURPOSE AND SCOPE OF WORK

The purpose of this study is to provide geotechnical engineering final design and construction recommendations for the replacement bridge. The project scope of work consists of:

- Compiling and reviewing existing available site information;
- Conducting field explorations along the proposed bridge alignment;
- Coordinating and reviewing laboratory soil testing;
- Characterizing site subsurface conditions;
- Providing foundation design criteria;
- Identifying seismic considerations for foundation design components;
- Providing foundation construction and earthwork recommendations; and
- Preparing a geotechnical engineering report.

## SITE DESCRIPTION

The existing bridge is located southwest of the City of Carnation in King County, Washington, as shown in Figure 1, Vicinity Map. The two-lane bridge, built in 1922 is 696 feet long. The bridge consists of pile-supported approaches and a 200-foot long span across the Snoqualmie River. The bridge is oriented in an east-west direction and is located on NE Tolt Hill Road. The bridge provides a grade transition from Tolt Hill on the west to the Snoqualmie River valley bottom on the east. No detailed foundation drawings for the existing bridge were available for review.

The Snoqualmie River runs in a northerly direction. The low-lying area west of the river has ponded water in areas, soft, boggy ground conditions and has been classified as a wetland. The riverbank on the east side is well vegetated, but the area south of the existing bridge approach is limited to grass cover. The Snoqualmie River has been known to flood this project site on numerous occasions in the recent past.

While the east side is relatively flat in elevation, the west bridge approach area transitions from a flat wetland area adjacent to the Snoqualmie River to steeper roadway fills and natural valley slopes along the NE Tolt Hill Road and West Snoqualmie River Road NE. The fill slopes are generally 2 horizontal to 1 vertical (2H:1V) with steeper valley cut sections approaching 1H:1V. Active drainage courses exist to the north and west of NE Tolt Hill Road.

## PROJECT DESCRIPTION

The proposed replacement bridge will be located to the south of the existing bridge, as shown on Figure 2, Site and Exploration Plan. Six piers will be constructed to support the bridge. Two will be located on the west side of the river and the remaining four on the east side of the river. Pier locations have moved over the course of the project mainly because the bridge type changed from a steel plate girder bridge to a twin steel truss-precast concrete girder bridge. The final pier locations and loading provided by Lin & Associates, Inc. are presented in Table 1.

## TABLE 1: BRIDGE PIER LOCATION AND LOADING

| Pier Number | Pier Station | Design Load in kips | Number of Drilled Shafts Per Pier |
|---|---|---|---|
| 1 | 31+10 | 2776 | 2 |
| 2 | 34+13 | 4794 | 2 |
| 3 | 37+15 | 3505 | 2 |
| 4 | 38+28 | 2108 | 2 |
| 5 | 39+48 | 2108 | 2 |
| 6 | 40+66 | 1264 | 2 |

A maximum fill height on the order of 10 feet is proposed at the west bridge abutment. Retaining walls are proposed to support this fill on the existing slope. Fill slopes with an angle of 2H:1V are proposed for the approach fill west of the bridge.

The intersection of NE Tolt Hill Road and West Snoqualmie River Road NE will be modified to accommodate the west approach to the bridge. The alignment of the new intersection is presented on Figure 4. This alignment involves fill heights no greater than 10 feet.

A maximum fill height on the order of 10 feet will be placed for the east bridge approach. Fill heights are greater along the south side of the approach fill since filling on the north side involves filling over the existing roadway. Retaining walls are proposed to support the fill from Station 40+66 to 43+00 on both the north and south sides of the new roadway alignment. We understand that retaining walls instead of fill slopes will be constructed to reduce the amount of fill placed within the floodplain of the Snoqualmie River.

To the south of the bridge, a box culvert is proposed in an existing channel that discharges into the Snoqualmie River. The location of the proposed box culvert is indicated on Figure 5. The box culvert dimensions are 15' wide, 15' long and 10' deep. The bottom elevation of the box culvert will be 54 feet or about 2 feet below the bottom of the existing channel. A loading of 550 pounds per square foot (psf) was estimated by King County staff.

In the event that any changes in the nature, design or location of the project components change, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

## SUBSURFACE CONDITIONS

The subsurface conditions at the bridge site are typical of alluvial deposits, consisting of near-surface, soft, compressible soils and loose, granular soils, underlain by stiffer and denser deposits. However, river deposits are variable in nature. They are subjected to varying depositional environments over time, such as river velocity and channel characteristics. The subsurface conditions encountered in our field explorations reflect the variable depositional environments that occurred in the past. The near-surface soils are finer-grained and are typical of a more low-energy depositional environment than the deeper and more coarse-grained sand and gravel layers.

Because of the alluvial origin of these deposits, abrupt changes due to the presence of old, filled-in river channels can occur. These abrupt changes can result in more abrupt changes in soil type and stratigraphy. The impact of these changes would be reflected in the settlement response of the east abutment approach fill and possibly in the required tip elevation of the foundations.

Generalized subsurface profiles have been developed along the bridge alignment as indicated on Figure 2, Site and Exploration Plan. The profiles are shown on Figures 6 and 7. Standard Penetration Test (SPT) data or N-values are also presented on the profiles at the boring locations. The stratum lines, which are drawn on the profiles, are based upon interpolation between borings and represent an interpretation of the subsurface conditions based on currently available data. The profiles are intended to be illustrative in nature.

The locations of the field explorations completed during the field exploration program are also shown on Figures 2 through 5, Site and Exploration Plans. Exploration logs presenting more detailed subsurface conditions can be found in Appendix A.

On the west side of the river, at the proposed Pier 1 location, subsurface conditions consist of variable roadway fill over medium dense to dense granular soils. The road fill was encountered to a depth of about 15 feet. Although the fill was field-sampled in a dense condition, a void was encountered at the base of the fill. These conditions were observed in boring B-3. Below the void, dense, granular soils were encountered to 74 feet below existing ground surface.

Boring B-2 was completed near the proposed location of Pier 2. The subsurface conditions consist of loose to very loose, silty sand over layers of medium dense to dense, sand and gravel, and medium stiff to stiff clay. It should be noted that the upper layers of sand and gravel encountered were difficult to drill through due to their tendency to collapse into the hole, despite use of weighted drilling mud techniques. Casing had to be used to support the borehole. It should also be noted that although the sand and gravel was sampled as a dense soil deposit, the gravels could reflect higher densities than is actually present due to the sampling method used.

On the east side of the river, borings B-13, B-6 and B-8 were completed near the proposed locations of Piers 3 through 6, respectively. The subsurface conditions encountered included a near-surface layer of soft silt and clay over layers of sand and gravel ranging from very loose to very dense in density and medium stiff to very stiff, silt and clay.

Borings B-9 and B-11 were completed along the proposed north side of the east bridge approach fill. The subsurface conditions encountered included an existing roadway fill on the order of 4 to 7 feet thick overlying alluvial deposits similar to those encountered at the locations of Piers 3 through 6. The existing roadway fill varied from loose to medium dense sand with gravel.

**New Intersection of NE Tolt Hill Road and West Snoqualmie River Road NE**

Boring B-10 was completed near Station16+00 on the shoulder of West Snoqualmie River Road NE. Subsurface conditions consist of loose roadway fill over medium dense to dense, gravelly sand. Hand explorations were conducted near the toe of the existing roadway embankment near Station 16+00 and at the crest of the existing ridge near the previously proposed intersection of NE Tolt Hill Road and West Snoqualmie River Road NE. The new intersection will be located to the west of the existing intersection.

At the toe of the existing roadway embankment, near Station 16+00, a hand excavation encountered predominantly loose to medium dense granular soils under a ½ to 1-foot of organic duff or root mat. A hand excavation in the ridge to the north exposed ½ foot of duff over medium dense to very dense granular soil. This ridge appears to have been reworked as evidenced by a 4-inch piece of asphalt pavement encountered within ½ foot of the existing ground surface. In the existing roadway cut slope farther to the north of the proposed intersection, dense to very dense, gravelly sand and sandy gravel with silt are exposed below a ½-foot thick root zone.

### Groundwater Conditions

The use of mud rotary drilling techniques prevents the determination of groundwater levels at the time of drilling. However, the proximity of the bridge site to the Snoqualmie River will highly influence groundwater levels. In addition, the proximity of Tolt Hill to the west of the site may also influence groundwater conditions. In general, the groundwater will flow towards the river and may be under pressure due to the elevation drop from upper elevations down to the river valley.

One monitoring well located in Boring B-13 confirms the presence of groundwater under pressure. Preliminary readings measured water levels above existing ground surface. Monitoring well installation details and groundwater measurements are presented on Figure A-14 in Appendix A. Groundwater levels were measured by King County and are presented on Figure A-15. Fluctuations in groundwater levels may occur due to variations in river levels, rainfall and temperature.

Flooding is common in the low-lying areas adjacent to the river. Ponded water and saturated near-surface soils are common in the wetland area, west of the Snoqualmie River.

## GEOTECHNICAL ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

The recommendations presented in this geotechnical design report are based in part on the data obtained during the field exploration and laboratory testing programs conducted for this study. The nature and extent of variations between explorations may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

## GENERAL SITE AND FOUNDATION CONSIDERATIONS

This report section provides a discussion of the impacts of the onsite soils on the foundation design of the proposed bridge. Specific design criteria will be presented in subsequent report sections.

### Deep Foundation Design and Construction

Due to the presence of existing fill soils and potential voids in the fill as indicated in boring B-3, the foundation for Pier 1 should be embedded a minimum depth of 15 feet below existing grade. With the proposed 10 feet of new fill at Pier 1, an open excavation depth of 15 to 20 feet to allow construction of spread footings at Pier 1 is not reasonable. Drilled shafts supported in the competent soils below the existing fill can provide adequate foundation support to Pier 1.

The alluvial nature of the onsite soils impacts the foundation design of Piers 2 through 6. In our opinion, the loose sand and soft compressible silt and clay are unsuitable for support of shallow foundations. The more competent sand and gravel layers encountered at depths of 20 to 25 feet can support higher loads, but are underlain by medium stiff to stiff, silt and clay.

The use of deep foundations is recommended to obtain sufficient frictional and end-bearing support for the proposed loading. Preliminary designs considered the use of driven piling. However, the need to embed the pile cap to protect it from scour at Piers 2 and 3 made piling a less effective design. A scour depth of 18 and 24 feet is expected at Piers 2 and 3, respectively, according to a recent hydraulics and scour assessment conducted by West Consultants.

Because of the scour issue at Piers 2 and 3, the use of drilled shafts was considered a more appropriate deep foundation system for this project site. Although driven piling could be used at the other piers, it is more efficient to have one type of deep foundation system since the equipment required to install driven piling is different than that required to install drilled shafts,

Drilled shafts are machine-excavated, circular shafts that are filled with concrete and reinforcing steel. The excavation process allows visual observation of the soil conditions encountered. These observations can verify the presence of soil conditions anticipated for the design of the shaft. The excavation process is flexible, allowing the shafts to be extended should soil conditions warrant.

Drilled shafts are advantageous for this project in that single shafts can support large loads. Other advantages are that drilled shaft construction is less noisy and involves less vibration than pile driving. Construction equipment and personnel knowledgeable in the installation of drilled shafts are readily available in the Puget Sound area.

### Seismic Considerations

Portions of the onsite soils are susceptible to liquefaction under strong earthquake ground motions. The impact of the liquefaction will include ground surface settlement and potentially involve lateral spreading of the riverbank into the river. The results of our stability analyses are extremely sensitive to the assumed strength parameters in the liquefied deposit. However, assuming worst-case conditions, lateral spreading would occur at Piers 2 and 3. Loss of lateral support of the upper 15 and 20 feet, at Piers 2 and 3 respectively, should be assumed for design purposes. We anticipate that the amount of lateral movement does not significantly impact the design of Piers 2 and 3, because the soils susceptible to lateral spreading are also susceptible to scour which is factored into the design of the foundation.

## DEEP FOUNDATION DESIGN RECOMMENDATIONS

Design of drilled shafts for support of the proposed structural loading at Piers 1 through 6 is recommended. The subsurface conditions at this site are challenging for the installation of drilled shafts. High groundwater levels under pressure and the gravelly sand layers will likely require casing and slurry techniques to maintain the stability of the shaft. In addition, drilling obstructions such as cobbles, boulders and buried trees may be encountered.

Despite the potential construction difficulties, improvements in drilled shaft construction equipment have been made with the hydraulic casing oscillator. The hydraulic casing oscillator installs casing in segments by rotating the casing back and forth while at the same time pushing the casing downward. The casing is equipped with cutting teeth or a cutting shoe. Soil is removed from inside the casing with a grab hammer. Once the required tip elevation is achieved the reinforcing cage is installed and the concrete is placed to form the shaft. As the concrete is being placed, the casing is hydraulically extracted in sections, providing good control of casing removal while maintaining adequate concrete head during the concrete pour.

Because of the anticipated subsurface conditions, it is recommended that the hydraulic casing oscillator be used for this project.

### Vertical Shaft Capacity

The vertical capacity of the shaft would be derived from friction along the side of the shaft and end bearing at the tip. The capacities and recommended tip elevations are presented in Table 2, Summary of Deep Foundation Design Recommendations. These capacities were determined using soil parameters for the different conditions encountered in our field explorations. These soil parameters were estimated based on field and laboratory test results and our characterization of the site.

Given that the subsurface information was obtained near the proposed pier locations, a factor of safety of 2 was used in determining the allowable compressive and uplift capacities. The weight of the shafts below the ground surface was taken into account in determining the shaft capacities. For purposes of our analyses, we assumed that the oscillator shaft installation technique was used and therefore the temporary casing used during shaft installation was removed upon completion of the shaft construction.

At Pier 6, downdrag loads imposed on the shaft would result from adjacent ground settlement due to the abutment fill loading. These downdrag loads were taken into account in the design recommendations for Pier 6.

**Lateral Resistance**

To assist in the use of the LPILE computer program to generate p-y curves for the lateral resistance of the drilled shafts, the subsurface conditions and recommended input parameters have been summarized on Table 3 for each pier location.

**Settlement**

The foundations are expected to settle due to the elastic response of the foundation itself and due to the deformation of the bearing soil. The analysis for shaft settlement is based on procedures developed by Vesic (1977). Settlement estimates range from 1 to 1-1/2 inches. The majority (80%) of the settlement is expected to occur as the loads are applied.

## TABLE 2: SUMMARY OF DEEP FOUNDATION DESIGN RECOMMENDATIONS

| Pier Number | Pier Design Load (kips) | Foundation Type | Foundation Size | Number of Shafts Per Pier | Recommended Depth (Ft.) | Recommended Tip Elevation (Ft.) | Compressive Capacity (kips), FS=2 | Uplift Capacity (kips), FS=2 |
|---|---|---|---|---|---|---|---|---|
| 1 | 2776 | Drilled shaft | 6.5-ft diameter | 2 | 25 | 62 | 3000 | 250 |
| 2 | 4794 | Drilled shaft | 8-ft diameter | 2 | 100 | -35 | 5000 | 1000 |
| 3 | 3505 | Drilled shaft | 8-ft diameter | 2 | 110 | -42 | 5000 | 1000 |
| 4 | 2108 | Drilled shaft | 6.5-ft diameter | 2 | 110 | -42 | 3500 | 900 |
| 5 | 2108 | Drilled shaft | 6.5-ft diameter | 2 | 110 | -46 | 3500 | 900 |
| 6 | 1264 | Drilled shaft | 6.5-ft diameter | 2 | 110 | -46 | 3000 | 900 |

For transient loading, compressive and uplift design capacities may be increased by 1/3, resulting in a FS=1.5.

Drilled shaft weight below existing ground surface taken into account in compressive and uplift calculations.

Downdrag loads from abutment fill settlement at Pier 6 taken into account in the compressive capacity.

Downdrag loads from seismically-induced ground surface settlement at Piers 2, 3, 5 and 6 are taken into account in the compressive capacity.

**TABLE 3: SOIL PARAMETERS FOR LPILE ANALYSIS**

| Pier and Boring Number | Soil Layer | Top Elev. of Layer (Ft.) | Bottom Elev. of Layer (Ft.) | Soil Type | Effective Unit Weight (pcf) | Cohesion (psf) | Friction Angle, Static (degrees) | Friction Angle, Liquefied (degrees) | Static Modulus of Subgrade Reaction (pci) | Liquefied Modulus of Subgrade Reaction (pci) | Strain at 50% |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Pier 1: | 1 | 94.5 | 80 | Sand | 125 | 0 | 36 | 36 | 125 | 125 | |
| Boring | 2 | 80 | 72.5 | Sand | 135 | 0 | 40 | 40 | 150 | 150 | |
| B-3 | 3 | 72.5 | 65.5 | Sand | 70 | 0 | 38 | 38 | 125 | 125 | |
| | 4 | 65.5 | 375 | Sand | 75 | 0 | 40 | 40 | 150 | 150 | |
| | 5 | 37.5 | 20.5 | Sand | 75 | 0 | 40 | 40 | 150 | 150 | |
| Pier 2: | 1 Note1 | 65.6 | 39.6 | Sand | 50 | 0 | 30 | 15 | 20 | 2 | |
| Boring | 2 | 39.6 | 26.6 | Sand | 75 | 0 | 40 | 40 | 150 | 150 | |
| B-2 | 3 | 26.6 | 4.4 | Clay | 60 | 1000 | 0 | 0 | 500 | 200 | .007 |
| | 4 | 4.4 | 1.6 | Sand | 70 | 0 | 38 | 38 | 125 | 125 | |
| | 5 | 1.6 | -17.4 | Clay | 60 | 750 | 0 | 0 | 500 | 200 | .007 |
| | 6 | -17.4 | -49.7 | Sand | 70 | 0 | 38 | 38 | 125 | 125 | |
| Pier 3: | 1 Note 2 | 68 | 51 | Clay | 45 | 250 | 0 | 0 | 100 | 50 | .02 |
| Boring | 2 | 51 | 45 | Sand | 50 | 0 | 28 | 15 | 20 | 2 | |
| B-13 | 3 | 45 | 17 | Sand | 65 | 0 | 35 | 35 | 60 | 60 | |
| | 4 | 17 | 13 | Clay | 45 | 500 | 0 | 0 | 250 | 100 | .02 |
| | 5 | 13 | -22 | Clay | 50 | 900 | 0 | 0 | 500 | 200 | .007 |
| | 6 | -22 | -41 | Sand | 65 | 0 | 36 | 36 | 125 | 125 | |
| | 1 | 68 | 50 | Clay | 45 | 450 | 0 | 0 | 250 | 100 | .02 |
| Pier 4: | 2 | 50 | 22 | Sand | 65 | 0 | 35 | 35 | 60 | 60 | |
| Boring | 3 | 22 | 7 | Sand | 65 | 0 | 34 | 34 | 60 | 60 | |
| B-7 | 4 | 7 | -22 | Clay | 50 | 900 | 0 | 0 | 500 | 200 | .007 |
| | 5 | -22 | -31 | Sand | 50 | 0 | 27 | 27 | 20 | 20 | |
| | 6 | -31 | -61 | Sand | 65 | 0 | 36 | 36 | 125 | 125 | |

| Pier and Boring Number | Soil Layer | Top Elev. of Layer (Ft.) | Bottom Elev. of Layer (Ft.) | Soil Type | Effective Unit Weight (pcf) | Cohesion (psf) | Friction Angle, Static (degrees) | Friction Angle, Liquefied (degrees) | Static Modulus of Subgrade Reaction (pci) | Liquefied Modulus of Subgrade Reaction (pci) | Strain at 50% |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Pier 5: Boring B-6 | 1 | 64 | 46 | Clay | 45 | 450 | 0 | 0 | 250 | 100 | .02 |
| | 2 | 46 | 38 | Sand | 50 | 0 | 28 | 15 | 20 | 2 | |
| | 3 | 38 | 13 | Sand | 65 | 0 | 36 | 36 | 125 | 125 | |
| | 4 | 13 | -26 | Clay | 50 | 1400 | 0 | 0 | 500 | 200 | .007 |
| | 5 | -26 | -36 | Clay | 55 | 2500 | 0 | 0 | 1000 | 400 | .005 |
| | 6 | -36 | -65 | Sand | 65 | 0 | 36 | 36 | 125 | 125 | |
| Pier 6: Boring B-8 | 1 | 64 | 48 | Clay | 45 | 400 | 0 | 0 | 250 | 100 | .02 |
| | 2 | 48 | 16 | Sand | 65 | 0 | 35 | 35 | 60 | 60 | |
| | 3 | 16 | 4 | Sand | 65 | 0 | 33 | 33 | 60 | 60 | |
| | 4 | 4 | -26 | Clay | 50 | 2100 | 0 | 0 | 1000 | 400 | .005 |
| | 5 | -26 | -32 | Clay | 55 | 2100 | 0 | 0 | 1000 | 400 | .005 |
| | 6 | -32 | -65 | Sand | 65 | 0 | 34 | 34 | 60 | 60 | |

Note 1: At Pier 2, in layer 1, disregard upper 15 feet for post-earthquake loading condition, due to potential post-liquefaction lateral spreading.

Note 2: At Pier 3, disregard layers 1 and 2 for post-earthquake loading condition, due to potential post-liquefaction lateral spreading.

## DEEP FOUNDATION CONSTRUCTION RECOMMENDATIONS

### Drilled Shaft Construction

Drilled shaft construction involves drilling a hole to a specified depth and a required diameter, placing a reinforcing cage and concreting the hole. The construction equipment and procedures used to install a shaft significantly impact the performance of the completed shaft. Due to the difficult ground conditions anticipated on this project and the large diameter involved, the drilled shaft contractor selected for the construction contract should be required to demonstrate their past experience successfully installing similar large diameter shafts under similar ground conditions. Project specifications should also require that the contractor provide detailed procedures that will be used for casing withdrawal, tremie concrete and slurry displacement methods.

A combination of casing and wet (slurry) methods of drilled shaft construction will be required to successfully complete the shaft installation in the anticipated subsurface conditions. Casing will likely be required through the upper sand and gravel layers encountered in our borings. The casing would be pushed or vibrated into the underlying clay to form a seal. However, if high groundwater pressures are encountered in the sand and gravel, it may be difficult to obtain an adequate seal. In addition, once the shaft is drilled through the clay and into the underlying sand, there is a potential for bottom heave or blowout that would require use of a slurry to maintain stability of the shaft.

It should be noted that the presence of cobbles, boulders, buried trees or other debris, is typical of alluvial deposits. Wood and gravelly drilling conditions were encountered in our explorations. These conditions may also require the use of a casing to stabilize the shaft excavation once the obstruction is removed.

As previously discussed, the use of a hydraulic casing oscillator to install the temporary casing is recommended. Once the shaft is excavated to the required depth, the reinforcing steel is placed and the concrete is tremied into the hole. It is important that the slurry be processed to meet specifications prior to concrete placement. Processing may involve recirculating, desanding and replacing the slurry in order to meet the required slurry properties. Excess sediment suspended in the slurry could settle out before the concrete is placed, resulting in excessive drill cuttings at the base of the shaft.

Casing withdrawal shall be carefully coordinated with concrete placement. An adequate head of concrete shall be maintained to exceed outside soil and water pressure above the bottom of the casing at all times during casing removal. It will be important that the concrete be designed to prevent arching during casing withdrawal or setting of the concrete until after the casing is withdrawn. Concrete levels should be checked prior to, during and after casing withdrawal to confirm that separation of the shaft concrete has not occurred.

If the hydraulic casing oscillator is not used and conditions require that the casing be left in place, the shaft design and capacity should be modified appropriately. Provisions for filling voids outside the permanent casings with grout or concrete should be included in the project specifications.

### Drilled Shaft Installation Considerations

The drilled shaft installation should be monitored by qualified personnel to verify that the subsurface conditions assumed for the design are encountered in the shaft excavation. The drilled shaft report should document the excavation method used, steel reinforcement and concrete placement operations and casing removal procedures.

Piers 2 and 3 are located adjacent to the Snoqualmie River on the west and east sides respectively. The proposed shaft foundations for Pier 2 are a minimum of about 20 feet from the riverbank. Since the construction equipment will be supported by a temporary trestle at this distance from the river, it is our opinion that equipment loading and drilling activities will not adversely impact the riverbank stability.

At Pier 3, the northern shaft is closest to the river at a distance of about 11.5 feet from the edge of shaft to the river. The existing riverbank is steep and has no vegetation on the face of the slope. Exposed soils are fine-grained in nature. Signs of surficial sloughing are evident along the riverbank and tend to involve less than 10 feet landward of the bank edge. This is an active erosional process occurring along the river. Even without any additional erosion or loss of riverbank that may occur before the start of construction, the installation of the northern shaft presents challenges to maintain the slope given the marginally stable conditions that currently exist.

We understand that the drilling equipment can be supported on perimeter temporary piles to reduce reaction forces associated with casing withdrawal. Installation of the temporary piling will be closer to the edge of the bank since the drilling equipment extends 3.5 feet past the edge of the shaft. Even with non-displacement temporary piling, the vibrations of piling installations may be significant enough to result in surficial sloughing of the riverbank. Other construction activities, such as the rocking motion used to install the first few sections of casing and vibrations from the drilling action make it unrealistic to assure the County that construction activity this close to the river can meet a rigid standard of no soil sloughing into the river. If there is a zero tolerance to soil sloughing into the river, provisions such as use of a cofferdam or a retaining structure to support the riverbank before construction should be included in the project plans. It should be noted however that vibrations associated with the installation of a cofferdam or retaining structure may cause surficial sloughing of the riverbank.

## SEISMIC CONSIDERATIONS

For AASHTO seismic design procedures, we recommend the following parameters be used. The subsurface site conditions indicate that this site would be considered a soil profile type III. The corresponding site coefficient, S, for soil profile type III is 1.5. The acceleration coefficient, A, for use in determining the elastic seismic response coefficient is 0.26 (USGS, 1996).

The results of a probabilistic seismic analysis indicate a 90 percent probability of non-exceedance in 50 years for a peak acceleration in rock of 0.26g. Given the range in subsurface conditions encountered, results of a ground response analysis estimate peak ground accelerations at the site to range from 0.2 to 0.3g.

Under this range of ground motions, the onsite soils were assessed for their liquefaction potential based on empirical methods by Seed and Idriss. Liquefaction is the process by which loose, saturated granular soils lose strength due to the buildup of excess hydrostatic pressure due to the application of cyclic shear stresses induced by earthquake ground motions. The empirical method used in this study correlates values of the earthquake-induced cyclic shear stress ratios for sites that have or have not liquefied with site parameters such as relative density based on the Standard Penetration Test (SPT) results.

Based on this empirical analysis, the loose sand encountered in borings B-1, B-2, B-6, B-9 and B-13 is considered highly susceptible to liquefaction (factor of safety less than 1.0). On the west side of the river, in boring B-1, the thickness of the liquefiable layer is 15 feet and the thickness encountered in boring B-2 is 26 feet. On the east side, in boring B-6 the thickness of liquefiable layer is 5 feet, the thickness in boring B-9 is 25 feet and 6 feet in boring B-13.

The impact of these layers liquefying is anticipated to be ground surface settlement at Piers 2, 3, 5 and 6 and lateral spreading of the riverbank at Piers 2 and 3. The liquefaction–induced settlement at Piers 2, 3, 5 and 6 was estimated, using a method developed by Tokimatsu and Seed, to range from ½ to 1 foot. These post-earthquake ground surface settlements would induce downdrag loads on the foundations at Piers 2, 3, 5 and 6. These downdrag loads are relatively small compared to the allowable capacity of the shafts. These loads however have been taken into account in Table 2 at Piers 2, 3, 5 and 6.

Lateral spreading is anticipated at Piers 2 and 3 following liquefaction. With the loss of soil strength under earthquake loading, the soil will have a tendency to move laterally to a free face, i.e., the river channel at Piers 2 and 3. For purposes of the stability analyses of the riverbank under seismic conditions, a residual strength was estimated in the liquefied sand ranging from 200 to 400 psf. A sensitivity study was performed using this range of residual strengths. The results of the stability analyses indicate factors of safety against instability ranging from 0.9 to 1.8.

A conservative approach would assume a slope failure would occur following liquefaction of the loose sand that would result in a lateral displacement of approximately 15 feet of the upper sands encountered at the Pier 2 location and 20 feet at Pier 3.

The anticipated scour depth at Piers 2 and 3 is about this order of magnitude based on a recent hydraulics and scour assessment conducted by West Consultants. We understand that this upper layer would have been disregarded under static loading conditions. Therefore, assuming lateral spreading of this upper layer should not impact the design of Piers 2 or 3.

## EARTHWORK RECOMMENDATIONS

### General Site and Subgrade Preparation Recommendations

All surficial organic soils and vegetation should be stripped, grubbed and wasted from the site in areas to support roadway fill. Stripping depths should be determined by qualified geotechnical engineering personnel in the field at the time of construction. For estimating purposes, stripping depths of ½ to 1 foot can be used.

In currently paved areas to be filled, asphalt pavement should be removed and the underlying subgrade scarified prior to placement of new fill. These areas are located along West Snoqualmie River Road NE and east of the east bridge abutment along NE Tolt Hill Road.

Following stripping and grubbing of organics and prior to fill placement or any construction activities, the exposed surface should be prerolled to provide a degree of compaction to the near-surface soils and to delineate any soft or loose areas which may be present. Prerolling should be accomplished with a vibratory roller or other suitable equipment. Qualified geotechnical personnel should be present during prerolling to verify suitability of areas to receive fill.

In areas that appear too soft for prerolling, overexcavation of the soft soils and replacement with compacted structural fill or a stabilizing fill lift could be placed to provide a stable subgrade for subsequent fill lifts. Providing an 18-inch thick stabilizing lift consisting of 3-inch minus rock over a geotextile is recommended to facilitate placement of the east approach fill. Separation and/or reinforcing geotextiles should conform to WSDOT Standard Specifications 9-33.2 (Table 3).

Soft, saturated subgrade conditions will not support heavy construction equipment in the wetland area, west of the river. We understand that a contractor-designed and installed temporary trestle will be required to reduce wetland impacts.

*Existing Utility Considerations*

We understand that King County is unaware of active underground utilities in the areas to be filled. During our field exploration program, we encountered an abandoned, buried corrugated metal pipe at our boring B-9 location. The pipe is of unknown length, but may be located under the new fill east of Pier 6. We recommend that during construction, the contractor locate this pipe to verify that is not located under the fill. If this pipe and other utilities are discovered during the course of construction, they should be moved, abandoned in place and/or replaced. Utilities abandoned in place should be filled with grout to avoid potential pipe collapse and subsequent void below the roadway fill.

**Fill Placement and Compaction Recommendations**

All roadway fill should be placed and compacted as an engineered fill. The suitability of soils for use as fill will depend on the gradation and moisture content when it is placed, subgrade and weather conditions during placement. As the amount of fines (that portion passing the U.S. No 200 sieve) increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soils containing more than about 5 percent fines cannot be consistently compacted to a dense, non-yielding condition when the moisture content is significantly above or below optimum. Optimum moisture content is that moisture content which results in the greatest compacted dry density.

Fill should be placed after preparation of the subgrade soils is complete. Subgrade preparation activities should include draining standing water prior to fill placement, overexcavation of excessively soft, loose or wet soils, or prerolling an area to receive fill in order to provide a firm, non-yielding surface. Refer to report section **General Site and Subgrade Preparation Recommendations** for additional discussion.

All engineered fill should be compacted to a minimum of 92 percent of the modified Proctor maximum dry density as determined by ASTM D 1557 test procedure except for the upper two feet that should be compacted to a minimum of 95 percent.

All fill material should be placed in lifts not exceeding 12 inches in loose thickness and should be thoroughly compacted. The moisture content should be controlled during compaction to within two percent of the optimum moisture content. An independent testing firm should be present during placement of the fill to monitor filling and perform density tests.

Imported fill should consist of a select borrow in accordance with 2004 WSDOT Standard Specification Section 9-03.14(2). The fines content (that portion passing the U.S. No. 200 sieve) should be limited to five percent during periods of wet weather.

## FILL STABILITY AND SETTLEMENT

**West Side of Snoqualmie River**

The subgrade conditions for the proposed 10-foot fill at the west abutment and for the reconfiguration of the intersection of NE Tolt Hill Road and West Snoqualmie River Road NE are anticipated to consist of sand and gravel fill over native sand and gravel. These conditions are suitable for support of the proposed fill following site preparation recommendations, previously discussed. Following recommended placement and compaction procedures, side slopes of 2H:1V are reasonable to assume for slope configuration of the proposed fills on the west side of the replacement bridge.

Given the granular nature of the subgrade conditions at the west abutment and along the realignment of West Snoqualmie River Road NE, settlement of the proposed fills will be elastic, i.e. generally occurring as the fill is placed. For a10-foot high fill, an elastic settlement of less than 1 inch is anticipated.

**East Side of Snoqualmie River**

The presence of near-surface, low-strength soils encountered in our explorations on the east side of the Snoqualmie River, impacts the proposed abutment fill and retaining walls. We understand that retaining walls instead of standard fill slopes are proposed for the approach fills from Station 40+66 to 43+00 to reduce the amount of filling in the floodplain of the Snoqualmie River.

A maximum 10-foot high fill is anticipated on the south side of the east approach fill. Based on soils encountered on the east side of the river, post-construction settlements ranging from ½ to 1 foot are anticipated. On the north side of the proposed roadway alignment, fills heights are generally less than those on the south side due to the presence of the existing roadway fill supporting NE Tolt Hill Road. A maximum fill height of 5 feet is anticipated on the north side of the alignment. Post-construction settlements on the north side are estimated to range from less than an inch to a few inches. Post-construction settlement results from the consolidation of cohesive or fine-grained soil under loading. We expect the settlement to occur over a period of 4 to 5 months once the fill is in place.

The soft site soils also present a potential for a bearing capacity failure of the fill. Analyses indicate marginally stable (FS=1.1 to 1.2) conditions with a fill height greater than or equal to 9 feet. We recommend that temporary stabilizing berms be placed on the south side of the east approach fills from Station 40+66 to 41+25. The stabilizing berms should range from 6- to 4-foot high from Station 40+66 to Station 41+25, respectively. The stabilizing berms should be as wide as the approach fill is high. So if a 10-foot high fill is proposed, the stabilizing berm should be 10-feet wide. A temporary fill slope angle of 1.5H:1V can be used for planning purposes for the stabilizing berm. The stabilizing berm should be constructed as the approach fill is being placed.

The temporary stabilizing berms can be placed directly on the native subgrade and should consist of mineral aggregate or crushed recycled concrete. The fill should be placed in lifts, but requires no compaction other than that afforded by construction equipment spreading it into place.

If the stabilizing berm is left in place through a potential spring flood season, appropriate erosion control measures should be implemented to protect the fill berm.

*Staged Construction*

We understand that construction schedules and easements allow the proposed fill and stabilizing berms to be built and allowed to settle for the anticipated 4 to 5 months. The time required for settlement to occur is a conservative estimate based on the assumption that settlement does not occur until the entire fill is in place. In reality, settlement begins as the fill is being placed. The estimated time frame for the settlement to occur under the full height of the fill is on the order of about 4 to 5 months.

Settlement monitoring should include both magnitude and time rate of settlement. Settlement monitoring devices should be installed in the fill. These settlement plates should be continuously monitored as filling progresses and following completion of the fill placement. It is essential to monitor the progress of the settlement, as time required to allow consolidation to occur can only be estimated prior to construction. Continuous field time rate of settlement data would enable the project team to determine when the majority of the settlement has occurred.

A series of settlement plate monuments should be installed at 50-foot intervals along the south edge (15 to 20 feet south of the proposed centerline) of the fill. The plates should be installed prior to fill placement. Initial settlement plate readings should be obtained immediately after placement of the plates and prior to placement of any fill. Readings of the plates should be taken by standard differential leveling to the nearest 0.01 foot and should be taken at regular intervals. Additional data that should be monitored at the time of settlement plate readings include date and time of reading and current elevation and dimensions of the fill area. At least three readings a week should be taken during filling. Once the fill is in place and as time elapses, the frequency of readings may be decreased to a minimum of once a week following analysis of the data and approval by the project geotechnical engineer.

Following review of the settlement data and determination that the majority of the consolidation settlement is complete, the project geotechnical engineer can indicate the appropriate time for removal of the stabilizing berms and any surcharge fill.

*Utility Considerations*

We understand that new utilities are proposed within a few feet of final roadway grade. Settlement-sensitive utilities and utilities not designed to support potential surcharge fill loading should be installed after settlement of the new fill is complete and the surcharge fill is removed.

**Other Alternatives**

If current project schedules change and do not allow time for the fill to settle, the east approach fill area will require alternate subgrade treatments or design modifications.

To reduce the settlement and improve the stability of the approach fill, several other options were considered:

- Surcharge fill and installation of wick drains;
- Use of lightweight fill such as polystyrene (geofoam);
- Ground improvement such as the installation of stone columns or GeoPiers; and
- Use a pile supported wall.

*Surcharge Fill and Installation of Wick Drains*

Placement of a surcharge fill and installation of wick drains could be considered to accelerate settlements and reduce the time required for settlement to occur. However, if the fill plus surcharge is higher than 9 feet, the stabilizing berm would be increased from a minimum 4-foot high to a minimum 6-foot high and may need to be extended to the east of Station 41+25.

The design team would have to consider the cost of placing and removing increased fill quantities versus the potential settlement time reduction. For example, a 3-foot high surcharge placed over a 9-foot fill would result in an estimated settlement time of 2 months.

Installation of prefabricated vertical drains such as wick drains or sand drains could also reduce required settlement time. The drains would be installed at 3 to 5-foot spacing and would extend through the upper soft soils or depths up to 20 feet. A settlement time of 2 months is also estimated with the installation of vertical drains such as wick drains.

*Lightweight Fill*

Use of lightweight fill such as EPS (expanded polystyrene) geofoam, greatly reduces the loading and thereby reduces the settlement magnitudes. The geofoam backfilled wall could be constructed following subgrade stabilization for the wall facing and subgrade for the backfill. Subgrade stabilization would consist of excavation of 18 to 24 inches of soft surficial soils, placement of a separation and reinforcing geotextile, followed by placement of up to 2 feet of compacted sand and gravel or crushed rock. The design would have to take into account buoyancy issues of the geofoam due to the possibility of flooding in the area. The use of overburden or restraint devices such as geogrids, geomembranes or tiedowns would likely be required. Tiedown support may be difficult to obtain due to the low-strength nature of the subgrade soils.

This option was not considered feasible due to the flood-prone nature of the site and potential cost for tiedown supports.

*GeoPiers*

GeoPier technology is a patented soil reinforcement system that is designed and constructed in the Pacific Northwest area by the GeoPier Foundation Company Northwest. It involves the installation of 30-inch diameter GeoPiers in a grid pattern with a designed areal coverage and minimum depth. The GeoPiers are installed by drilling and removing onsite soils to a given depth followed by backfilling with compacted angular stone. The installation method increases the strength characteristics of the subgrade soils and reduces the settlement magnitudes. An MSE wall can then be constructed and backfilled with granular fill.

Based on our analyses, we recommend a minimum 30% areal coverage of 30-inch diameter GeoPiers installed to a minimum depth of 16 feet or through the surficial layer of soft compressible soils. With this design, stability of the proposed fill is improved (FS=1.5) and settlements are reduced to an estimated one to two inches. Settlements below the MSE wall facing would be less than one inch since the wall facing would be supported by a row of GeoPiers.

This option was not considered economically feasible.

*Pile-Supported Wall*

Another alternative would be to use piling to support the proposed approach fill walls. Although the walls would be pile supported, the fill would still settle relative to the wall. Any fill higher than 9 feet would also still have a potential to experience a bearing capacity failure. To reduce the potential for this bearing capacity failure, some form of subgrade improvement would be required or the entire fill mass greater than 9 feet high would have to be pile supported. We expect that this would be the highest cost option and therefore was not considered feasible.

It is our opinion that the use of GeoPiers is likely the most economical option if project schedules cannot allow sufficient time for the east approach fill and stabilizing berms to be placed and allowed to settle.

## RETAINING WALLS

Retaining and subgrade walls will be used at the bridge abutments and for the box culvert. Reinforced concrete walls will be used at these locations. Wing walls for the box culvert will consist of coir lift walls designed by King County. Mechanically stabilized earth walls are anticipated along the north and south sides of the east abutment approach fill.

### Lateral Earth Pressures

Lateral earth pressures against a wall will be a function of "active" or "at-rest" conditions, which in turn depend on the amount of lateral movement, permitted at the top of the wall during backfilling operations. If the top of the wall is free to yield at least 0.001 times the height of the wall, soil pressures would be of an active state, whereas, if the movement is limited by wall stiffness, or is structurally tied at the top, an at-rest condition should be assumed.

Estimated lateral earth pressures for yielding and non-yielding walls are based on horizontal backfill conditions and no hydrostatic pressures behind the wall. Walls not designed for hydrostatic pressures, should be backfilled with free-draining well-graded sand and gravel with a fines content less than 5 percent. A perforated pipe should be placed at the base of the drainage backfill and sloped to drain to a suitable discharge point.

The effects of surcharge loading, such as traffic loading should be included in the wall design. For uniform loads, apply a uniform distribution on the wall equal to 30 or 50 percent of the surcharge for yielding and non-yielding walls, respectively. Concentrated point loads should be considered to act against the wall as loads equal to 35 to 50 percent of the concentrated load for yielding and non-yielding walls, respectively. The location on the wall that the additional load can be considered to be acting may be approximated by projecting a line downward and outward at a 1H:1V slope from the applied vertical load to its intersection with the wall.

Lateral forces against the wall can be resisted by both passive resistance due to the soil in front of the wall and frictional resistance between the soil and the base of the wall. Passive resistance mobilized can be computed using the recommended ultimate equivalent fluid weight for the soil. Lateral movement can also be resisted by friction between the soil and concrete at the base of the structure. The frictional resistance is computed by multiplying the applied vertical load on the base of the wall by the frictional coefficient. Use a factor of safety of 1.5 in determining the allowable resistance to lateral movements.

We understand that the box culvert is designed as a rigid box and will be submerged.

The following design values are appropriate for the anticipated conditions at the bridge abutments and box culvert.

Lateral earth pressures:

Yielding wall: 30 pcf

Non-Yielding wall: 55 pcf

Non-Yielding wall (submerged conditions): 90 pcf (includes hydrostatic loading)

The earth load acts at H/3 above the base of the wall, where H is the height of the wall.

Ultimate passive resistance: 450 pcf

Ultimate passive resistance (submerged): 175 pcf

Coefficient of friction: 0.45

Seismic loading:

Yielding wall: 55 pcf, with the total load acting at H/2 above the base of the wall.

Use a factor of safety of 1.1 to 1.2 to determine the allowable resistance to lateral movements under seismic loading. It should be noted that non-yielding walls designed with appropriate static factors of safety are usually adequate to resist seismic loading.

**Shallow Foundation Design**

The wing walls at the west abutment will be located on an existing 1.5H:1V slope. The wing walls will support 8 feet of new fill. We recommend that the shallow foundations for the wing walls have a minimum embedment of 6 feet below existing ground surface or 8 feet behind the face of the slope to protect the foundations from surficial erosion of the existing slope. With this embedment and foundation soils consisting of competent native soils, an allowable soil bearing pressure of 3000 psf can be used for design.

The proposed box culvert is estimated to have a loading of 550 psf. The bottom of the culvert is proposed to be 2 feet below existing channel bottom. We anticipate soft clay will be exposed in the box culvert excavation. We recommend a minimum overexcavation of 18 inches of unsuitable native soils and replacement with 2-inch minus crushed rock over a separation and soil stabilizing geotextile conforming to 2004 WSDOT Standard Specifications 9-33.2 (Table 3). With this subgrade stabilization measure, an allowable soil bearing pressure of 1000 psf can be used to design the box culvert.

Continuous wall footings should be a minimum of 18 inches wide. The allowable bearing capacity can be increased by one-third to account for transient loads such as wind or dynamic loads.

Assuming proper subgrade preparation and anticipated structural loading, the estimated total settlement is less than an inch with a maximum differential settlement on the order of half of the total settlement. This settlement should occur relatively quickly as the loads are applied, given the nature of the anticipated subgrade conditions.

**Mechanically Stabilized Earth (MSE) Wall**

MSE walls use reinforcing elements between layers of fill. This reinforced soil mass acts as a unit and resists the lateral soil loads through the dead weight of the reinforced mass. MSE walls have reinforcement lengths that are typically 70 percent of the wall height.

The magnitude of settlement anticipated at the east approach fill, requires that a flexible wall system be chosen. An MSE wall such as a welded wire mesh wall system or Hilfiker wall is recommended. This wall system is flexible and can tolerate the anticipated total and differential settlements. Precast concrete wall facing panels can be installed after the settlement has occurred. Alternatively a shotcrete facing could be used.

The recommended design parameters for use in the design of MSE walls are presented in Table 4. The design values assume that the backfill soils and the retained soil are compacted to 95 percent of the modified Proctor maximum dry density as determined by ASTM D 1557 test procedure.

**TABLE 4: MSE Wall Design Parameters**

| Soil Properties | Backfill Soils | | Foundation Subgrade Soils | | |
|---|---|---|---|---|---|
| | Reinforced Zone | Retained Soil | Alluvial Silt and Clay Prior to Fill Placement | Alluvial Silt and Clay after Fill Settlement | Existing Roadway Fill over alluvial silt and clay |
| Unit Weight (pcf) | 125 | 125 | 110 pcf | 115 | 120 |
| Friction Angle (degrees) | 38 | 36 | 0 | 0 | 36 |
| Cohesion (psf) | 0 | 0 | 250 | 800 | 0 |
| Allowable Soil Bearing Pressure (psf) | N/A[Note1] | N/A[Note1] | 450[Note2] | 1500 | 2000 |

Note 1: Not applicable
Note 2: Stabilizing berms required to support fill as discussed under report section **FILL STABILITY AND SETTLEMENT**

MSE walls should also incorporate a drainage system to control water infiltration into the fill materials. MSE wall manufacturers typically provide recommendations for drainage systems for their walls. At a minimum a drainage zone should be provided behind the reinforced mass and in front of the existing or new fill embankment (or retained soils). This drainage zone should consist of a 6-inch perforated pipe surrounded by 12 inches in all directions with clean gravel suitable for drains. This drainage zone should be wrapped within a geotextile to prevent fines from migrating into the gravel drainage. The drain pipe should daylight to the wall face or tie into a drainage system at regular intervals.

MSE walls should be designed with a minimum static factor of safety of 1.5 against global instability. With the use of stabilizing berms, where recommended under report section, **FILL STABILITY AND SETTLEMENT,** a minimum static factor of safety during construction of 1.3 was estimated. After fill settlement and corresponding strength increase in the subgrade soils, a minimum static factor of safety greater than 1.5 was estimated.

MSE walls should be designed for a minimum factor of safety of 1.5 against sliding and pullout of reinforcing elements and 2.0 against overturning. The wall supplier of a proprietary system such as Hilfiker walls is responsible for evaluating their system for these design issues. The design should be reviewed by a qualified geotechnical engineer to verify that valid assumptions have been made relative to material properties and other factors.

The MSE wall design should include surcharge loading from items such as traffic loading which are located within a horizontal distance equal to the height of the wall. Traffic loading is typically modeled as a surcharge load equal to 2 additional feet of fill.

## RECOMMENDATIONS FOR ADDITIONAL SERVICES

It is recommended that Lorilla Engineering be involved in review of the final design drawings and specifications in order to verify that the recommendations presented herein have been properly interpreted and incorporated into the design. It should be noted that conditions of environmental permits may require modification of our recommendations.

It is recommended that Lorilla Engineering be involved during construction to provide geotechnical engineering services. This would include review of earthwork activities and fill test results, observation and review of drilled shaft foundation installation and review of settlement monitoring data. The purpose of these observations is to verify compliance with the design concepts and recommendations and to allow timely design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

## LIMITATIONS OF THIS STUDY

This study was completed in general accordance with our contracts dated August 17, 1995, March 18, 2003, March 3 and August 9, 2005. This work was performed for the exclusive use of King County and Lin & Associates, Inc., their design consultants, for specific application to this project and site. Lorilla Engineering performed this work in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed, or implied, is made.

**LORILLA ENGINEERING, INC., P.S.**

Michele Lorilla, P.E.
Geotechnical Engineering Consultant

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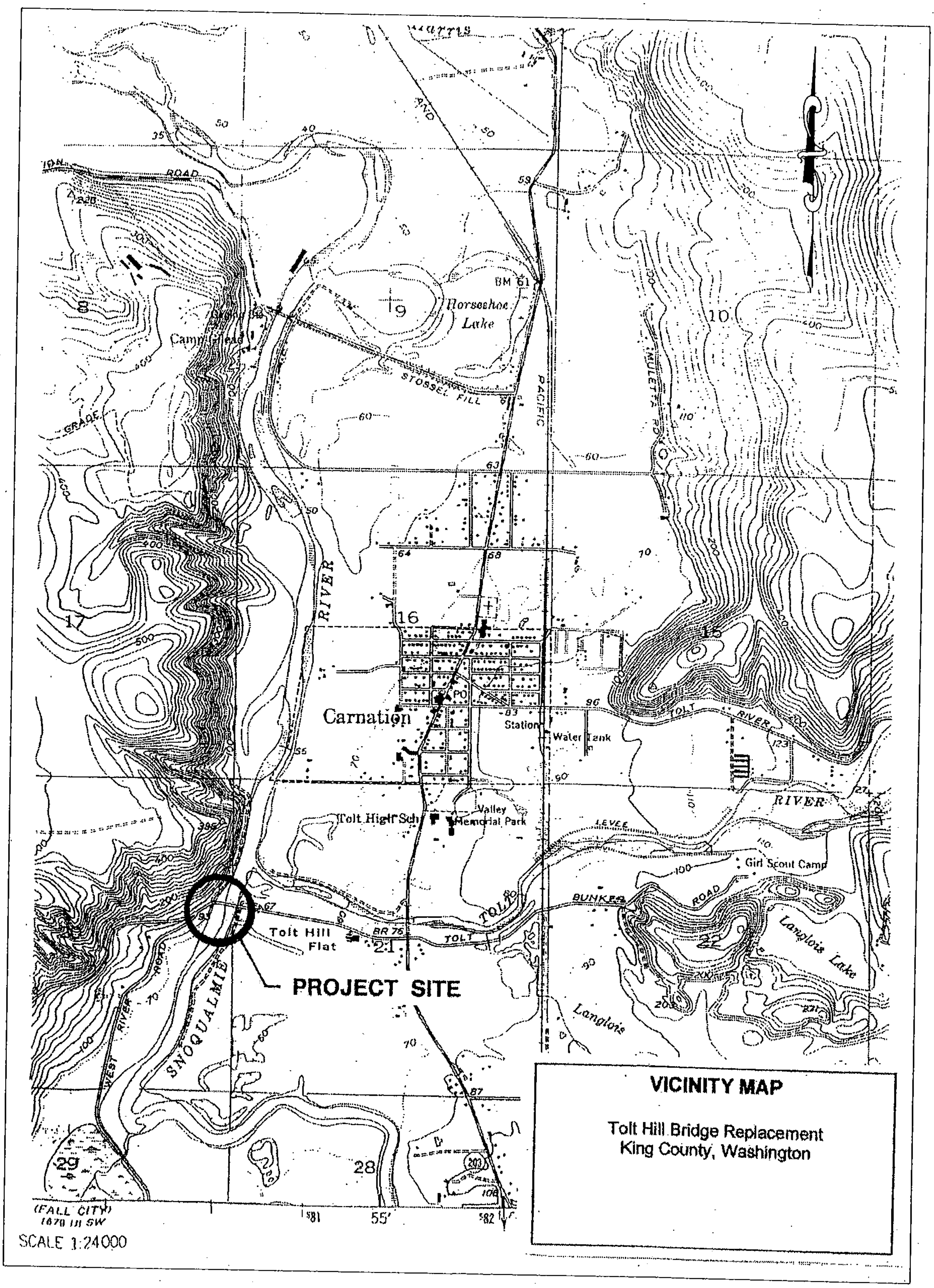



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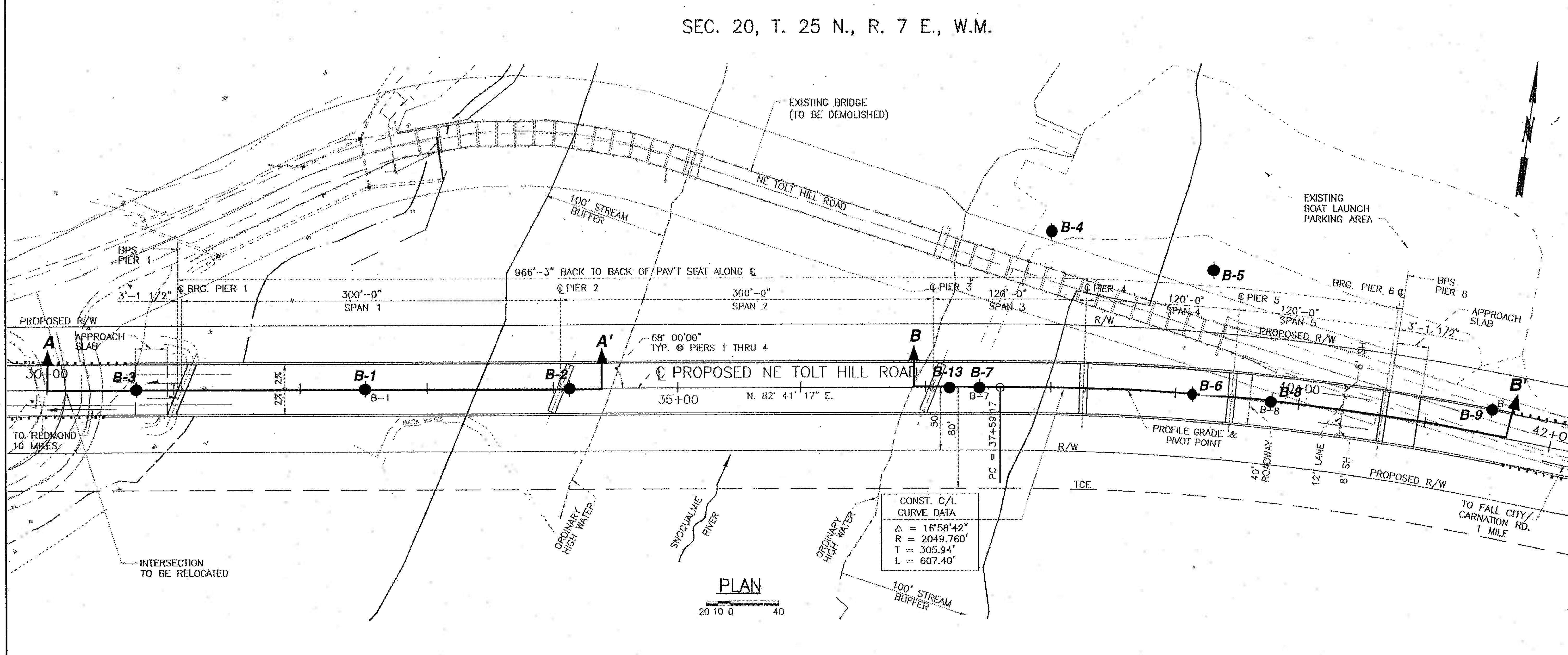
**TOLT BRIDGE REPLACEMENT**
**Bridge Number 1834A**
**King County, Washington**

| | |
|---|---|
| FIGURE NO | 1 |
| PROJECT NO | KC - 300 |
| DATE | April 2005 |




**NOTES:**

1. Site plan prepared from base map provided by Lin and Associates, Inc.
2. Boring locations are approximate.

**LEGEND:**

● B-1 Approximate Boring Location

A A' Profile Location and Designation

**Approximate Scale:**
**1" = 80 feet**

0 40 80

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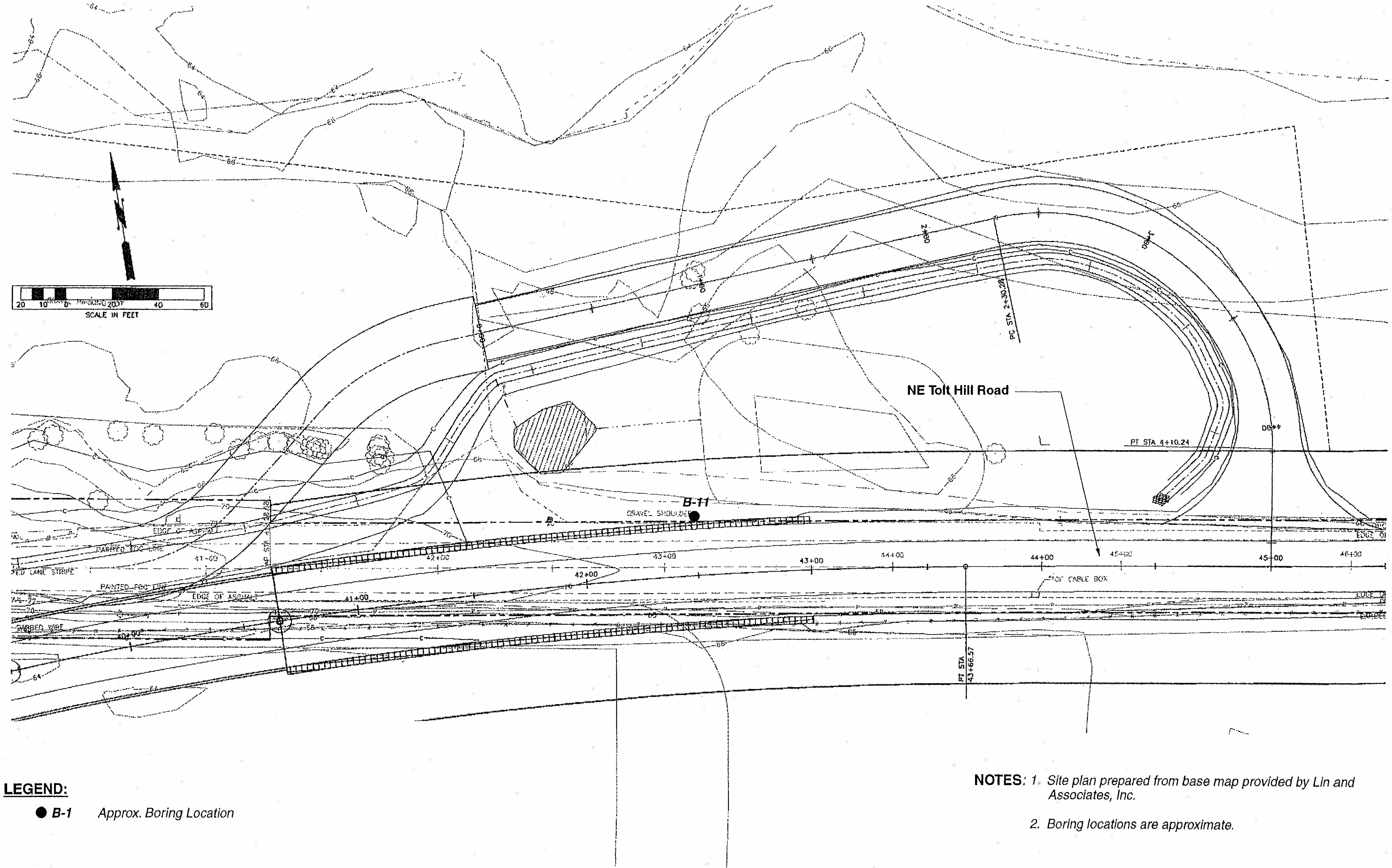
**TOLT BRIDGE NO. 1834A REPLACEMENT**
**NE Tolt Hill Road**
**King County, Washington**

**SITE and EXPLORATION PLAN**

FIGURE NO. 2
PROJECT NO. KC - 300
DATE April 2005




**LEGEND:**

● *B-1* *Approx. Boring Location*

**NOTES:** *1. Site plan prepared from base map provided by Lin and Associates, Inc.*

*2. Boring locations are approximate.*

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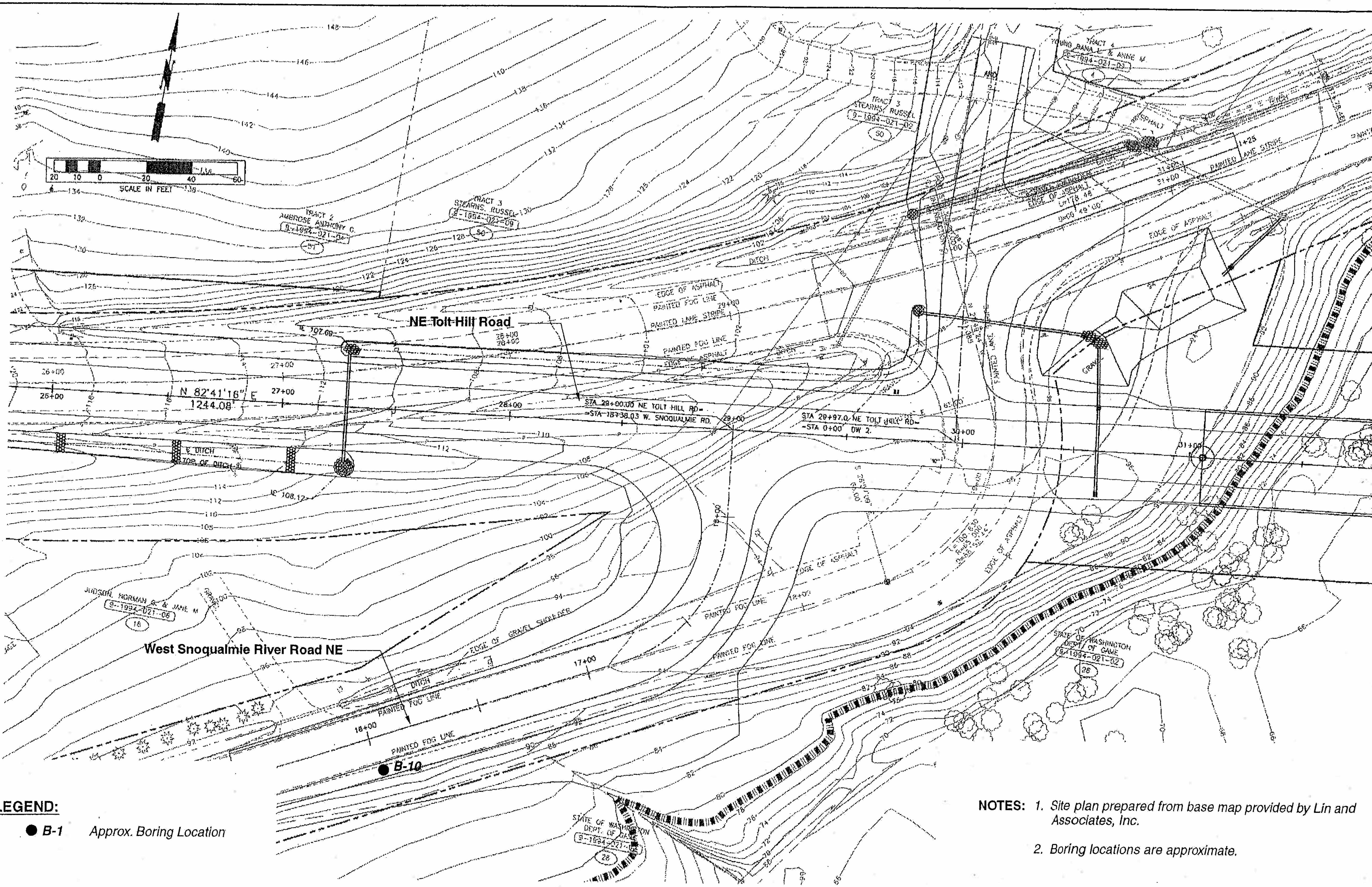
**TOLT BRIDGE NO. 1834A REPLACEMENT**
**NE Tolt Hill Road**
**King County, Washington**

**SITE and EXPLORATION PLAN**

FIGURE NO. 3
PROJECT NO. KC-300
DATE April 2005




LEGEND:

● B-1 Approx. Boring Location

NOTES:
1. Site plan prepared from base map provided by Lin and Associates, Inc.
2. Boring locations are approximate.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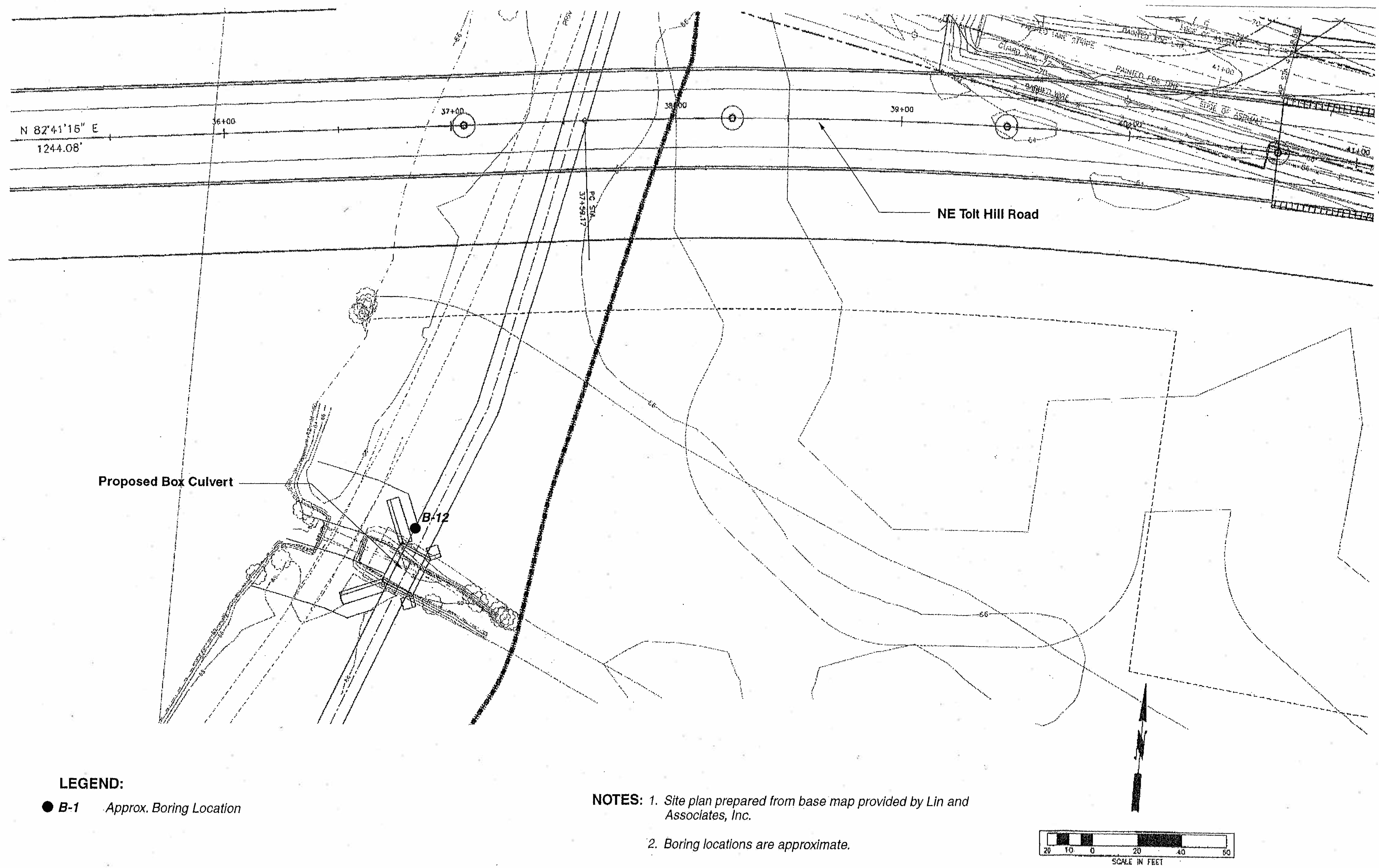
**TOLT BRIDGE NO. 1834 REPLACEMENT**
**West Snoqualmie River Road NE**
**King County, Washington**

**SITE and EXPLORATION PLAN**

FIGURE NO. 4
PROJECT NO. KC - 300
DATE April 2005




LEGEND:

● *B-1* *Approx. Boring Location*

NOTES: 1. *Site plan prepared from base map provided by Lin and Associates, Inc.*

2. *Boring locations are approximate.*

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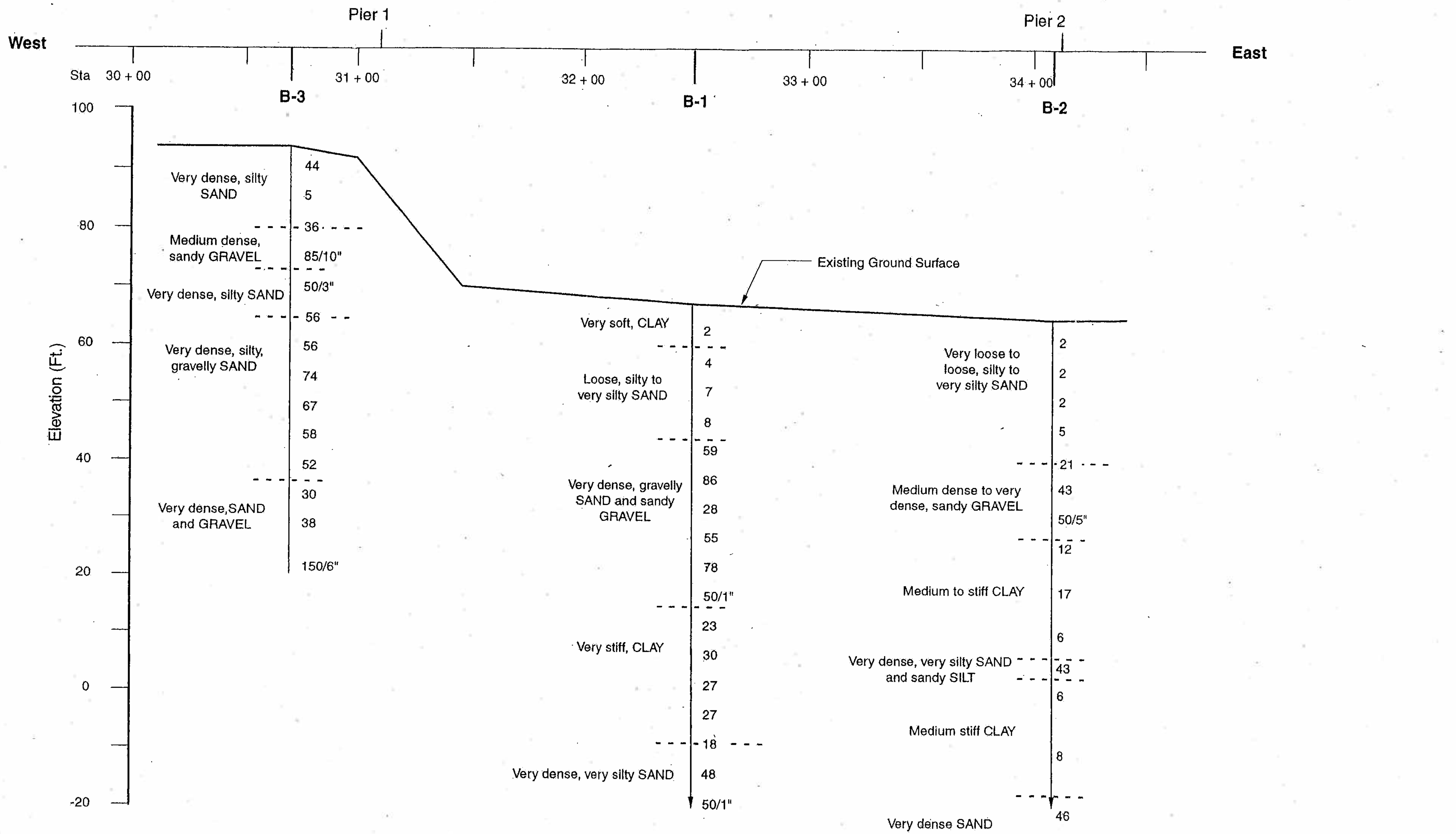
**TOLT BRIDGE NO. 1834A REPLACEMENT**
**NE Tolt Hill Road - Box Culvert Area**
**King County, Washington**

**SITE and EXPLORATION PLAN**

FIGURE NO. 5
PROJECT NO. KC - 300
DATE April 2005




NOTES: 1. *Reference Figure 2 for profile location along road section.*

2. *Contacts between soil units are based on interpolation between explorations and represent an interpretation of subsurface conditions based on available data.*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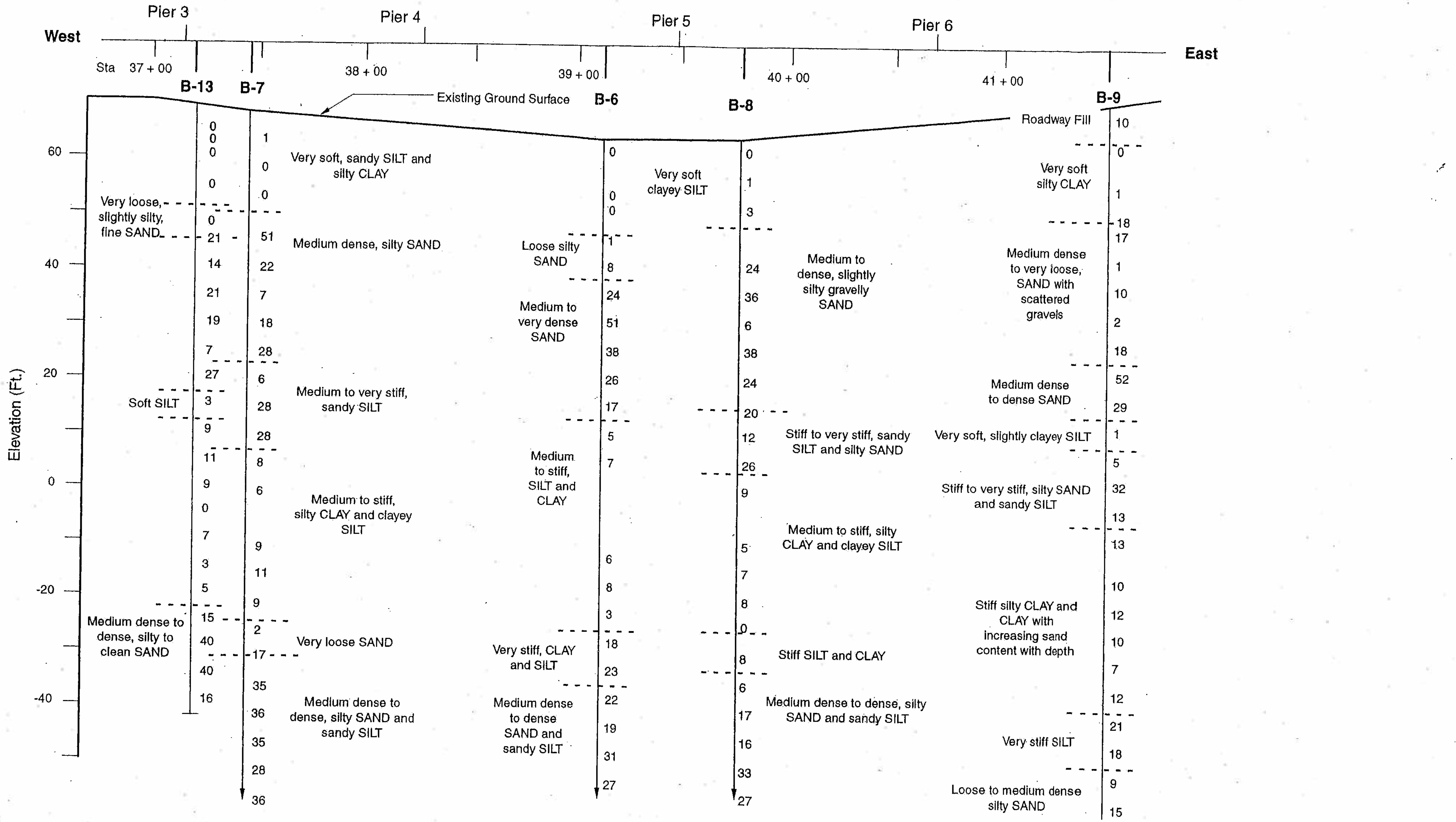
**TOLT BRIDGE NO. 1834A REPLACEMENT**
**NE Tolt Hill Road**
**King County, Washington**

**GENERALIZED SUBSURFACE PROFILE A-A'**

| | |
|---|---|
| FIGURE NO. | 6 |
| PROJECT NO. | KC - 300 |
| DATE | April 2005 |




NOTES: 1. *Reference Figure 2 for profile location along road section.*

2. *Contacts between soil units are based on interpolation between explorations and represent an interpretation of subsurface conditions based on available data.*

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**TOLT BRIDGE NO.1834A REPLACEMENT**
**NE Tolt Hill Road**
**King County, Washington**

**GENERALIZED SUBSURFACE PROFILE B-B'**

| | |
|---|---|
| FIGURE NO. | 7 |
| PROJECT NO. | KC - 300 |
| DATE | |

# APPENDIX A

## SUBSURFACE EXPLORATIONS

# APPENDIX A

## SUBSURFACE EXPLORATIONS

The field exploration program for this project included completion of thirteen soil borings. The results of the field exploration program are presented on the exploration logs within this Appendix. The exploration logs are a representation of the interpretation of the drilling, sampling and testing information. The depth where the soils or characteristics of the soils changed is noted. The change may be gradual. Soil samples obtained in the explorations were visually classified in the field in general accordance with geotechnical engineering practice.

The exploration locations are presented on Figures 2 through 5, Site and Exploration Plan. The explorations were located in the field by hand taping from surveyed locations provided by Lin & Associates, Inc. The approximate ground surface elevations at the exploration locations, as presented on the logs, are interpreted from elevations on a topographic site plan provided by Lin & Associates. The locations and elevations of the explorations should be considered accurate to the degree implied by the method used.

### Soil Borings

A total of thirteen soil borings, designated B-1 through B-13 were completed for this project. Borings B-1 through B-5 were completed from September 24 through October 14, 1996. Borings B-6 through B-8 were completed from September 4 through September 6, 2002. Borings B-9 and B-10 were completed June 18 and 19, 2003. Borings B-11 through B-13 were completed March 24 through March 26, 2004. The borings were drilled to depths ranging from 24 to 129 feet below ground surface.

Borings B-1 and B-2 were drilled with a track-mounted rig (LARS10T). A truck-mounted drill rig (Mobile B-59) was used to drill the remainder of the borings. Holt Testing, Inc. of Puyallup, Washington, under subcontract to Lorilla Engineering, Inc. advanced the borings using mud rotary for borings B-1 through B-4 and hollow-stem auger drilling techniques for the remainder of the borings.

The drilling was accomplished under continuous observation of an engineering geologist or geotechnical engineer. Detailed logs were prepared of each boring. Samples were obtained on 2-1/2- and 5-foot depth intervals using the Standard Penetration Test (SPT) procedure and thin-walled Shelby tubes.

The Standard Penetration Test procedure as described in ASTM D 1587 was used to obtain disturbed soil samples. A standard 2-inch outside diameter, split-spoon sampler is driven into the soil a distance of 18 inches using a 140-pound hammer, free-falling 30 inches. The number of blows required to drive the sampler the last 12 inches is the Standard Penetration Resistance. This resistance, or blow count, provides a measure of the relative density of granular soils and consistency of cohesive soils. The SPT procedure is a useful quantitative tool from which density/consistency is determined. The results must be used in conjunction with other tests and engineering judgment. Samples obtained from the split spoon sampler were field classified and place in watertight bags for further testing.

If high penetration resistance was encountered which precluded driving the total 18-inch sample interval, the penetration resistance for the partial penetration is entered on the logs. It should be noted that high penetration resistance was observed through soil deposits with high gravel content. The presence of the gravels tends to reflect a higher resistance due to the sampling technique used. The actual density of the deposit may be less than what is reflected from the sampling results.

In fine-grained soils, a 3-inch diameter, thin-walled steel (Shelby) tube sampler was pushed hydraulically below the auger to obtain a relatively undisturbed sample. Use of a piston sampler was necessary to improve sample recovery. The tubes were sealed in the field and taken to a testing laboratory for extrusion, classification and testing.

The boring logs are presented on Figures A-1 through A-13.

At the location of boring B-9, an 18-inch diameter, corrugated metal pipe (CMP) was observed after completion of the boring. Neither Lin & Associates, nor King County was aware of the existence or purpose of this CMP. This utility was not identified during the required utility-locate process prior to drilling. Intact bags of bentonite pellets were used to bridge the hole in the CMP during backfilling of the borehole.

**Monitoring Well**

A monitoring well was installed in boring B-13 in order to measure groundwater levels. Typical installation consists of providing a sand pack around a 2-inch-diameter, 10-foot-long, 20-slot screen; backfill with concrete grout to the surface. An above-grade surface monument was installed. Monitoring well details are presented on Figure A-14. Initial groundwater measurements are presented on Figure A-14. King County provided subsequent measurements and data reduction as indicated on Figure A-15.

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/3/96
Ground Surface Elevation: 67.2 ft (20.5m)



| Depth in Feet (in meters) | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 0 | | | Very soft, wet, gray CLAY with organics (wood). | |
| 5 | S-1 | 1<br>1<br>1 | | AL, WC: 86% |
| | | | Loose, wet, gray, silty to very silty, fine SAND. | |
| 10 | S-2 | 1<br>2<br>2 | - 1 inch wood fragment | |
| 15 (5m) | S-3 | 2<br>3<br>4 | - occasional wood fragments | |
| 20 | S-4 | 4<br>3<br>5 | | WC: 31%<br>-#200: 33% |
| | | | - gravelly drilling at 23 feet | |
| | | | Very dense, wet, brown, gravelly SAND and sandy GRAVEL. | |
| 25 | S-5 | 24<br>27<br>32 | | |
| 30 | S-6 | 36<br>50/5-1/2" | | |
| (10m) | | | | |
| 35 | S-7 | 20<br>18<br>10 | - medium dense, wet, brown, slightly gravelly, silty SAND. | GS |
| | | | - gravelly drilling at 38 feet | |
| 40 | S-8 | 14<br>32<br>23 | - no sample recovery | |
| 45 | S-9 | 28 | | |

| PROJECT NAME: Tolt Hill Bridge<br>PROJECT NO.: KC-300 | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 10/3/96<br>Ground Surface Elevation: 67.2 feet (20.5 m) | BORING LOG B-1<br>Page 2 of 3<br>Figure A-1 |
|---|---|---|

| Depth In Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 45 | | 50/6" | Very dense, wet, brown, sandy GRAVEL. | |
| | | | - Hole collapsed 9/24/96<br>- Redrill and case hole 10/1 to 10/3/96 | |
| (15m) 50 | S-10 | 50/1" | - no sample recovery | |
| | | | - casing drove easily at 52 feet | |
| | | | Very stiff, wet, gray CLAY. | |
| 55 | S-11A<br>S-11B | 12<br>13<br>10 | | S-11B: WC: 42% |
| 60 | S-12 | 7<br>11<br>19 | | WC: 39% |
| 65 (20m) | S-13 | 8<br>17<br>10 | - silty gray lenses | AL, WC: 44% |
| 70 | S-14 | 1<br>13<br>14 | - no sample recovery | |
| 75 | S-15 | 3<br>5<br>13 | - occasional sand partings | WC: 47% |
| | | | Very dense, wet, gray, very silty, fine SAND. | |
| | | | - gravelly drilling at 78 feet | |
| 80 | S-16A<br>S-16B | 6<br>20<br>28 | - brown, gravelly, medium to fine SAND | |
| (25m) | | | | |
| 85 | S-17 | 50/1" | - no sample recovery, sampler bouncing on rock | |
| 90 | S-18 | 27 | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/3/96
Ground Surface Elevation: 67.2 ft (20.5 m)



| Depth In Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 90 | (S-18) | 50/6" | Very dense, moist, tan, very silty, fine SAND (till-like). | |
| 95 | S-19 | 34<br>50/5" | | |
| (30m) | | | | |
| 100 | | | | |
| 105 | S-20 | 50/5" | | |
| | | | Bottom of boring at 105.5 feet (32.2 m) below existing ground surface. | |
| 110 | | | | |
| (35m) | | | | |
| 115 | | | | |
| 120 | | | | |
| 125 | | | | |
| 130 | | | | |
| (40m) | | | | |
| 135 | | | | |

**PROJECT NAME:** Tolt Hill Bridge
**PROJECT NO.:** KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/30/96
Ground Surface Elevation: 65.6 ft (20 m)

**BORING LOG B-2**
Page 1 of 3
Figure A-2

| Depth in Feet (in meters) | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 0 | | | Very loose to loose, wet, gray, silty to very silty, fine SAND and fine SAND. | |
| 5 | S-1 | 1<br>1<br>1 | - trace organics (wood) | |
| 10 | S-2 | 1/12"<br>1 | | |
| 15 (5m) | S-3 | 1<br>1<br>1 | | WC: 39%<br>- #200: 43% |
| 20 | S-4 | 2<br>2<br>3 | | |
| 25 | S-5 | 3<br>6<br>15 | Medium dense to very dense, wet, brown, sandy, GRAVEL.<br>- wood at 28 to 29 feet | |
| 30 (10m) | S-6 | 30<br>20<br>23 | - gravelly drilling at 34 feet | |
| 35 | S-7 | 50/5" | - no sample recovery<br>- easier drilling at 39 feet | |
| 40 | S-8 | 5<br>5<br>7 | Medium stiff to stiff, wet, gray CLAY.<br>- no sample recovery | WC: 52% |
| 45 | S-9 | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/30/96
Ground Surface Elevation: 65.5 ft (20 m)



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 45 | S-10 | | Medium stiff to stiff, wet, gray, CLAY.<br>- no sample recovery | |
| | | 11<br>9<br>8 | - very stiff CLAY | WC: 45% |
| (15m)<br>50 | S-11<br>S-12 | | | WC: 54%<br>CN, UU, AL<br>TV: 0.45 tsf, PP: 0.6 tsf<br>TV: 0.475 tsf, PP: 0.75 tsf<br>TV: 0.35 tsf, PP: 1.75 tsf<br>WC: 32% |
| 55 | S-13 | 3<br>3<br>3 | | WC: 57% |
| 60 | S-14 | 12<br>19<br>24 | | UU, AL<br>WC: 42%<br>TV: 0.55 tsf, PP: 0.5 tsf |
| | S-15 | | Very dense, wet, gray, very silty fine SAND and fine sandy SILT. | |
| 65<br>(20m) | S-16 | 3<br>2<br>4 | Medium stiff, wet, gray CLAY. | WC: 60% |
| 70 | S-17 | | - no sample recovery | |
| 75 | S-18 | 4<br>4<br>4 | | WC: 35% |
| 80<br>(25m) | S-19 | | - silt lenses | WC: 56%<br>TV: 0.5 tsf, PP: 0.75 tsf<br>WC: 50%<br>TV: 0.5 tsf, PP: 0.75 tsf |
| | | | Very dense, wet, gray, very silty, fine SAND. | |
| 85 | S-20A<br>S-20B | 8<br>12<br>34 | - 6 inch lense, very dense, moist, brown, SAND | |
| 90 | | | | |

**PROJECT NAME: Tolt Hill Bridge**
**PROJECT NO.: KC-300**

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/30/96
Ground Surface Elevation: 65.5 ft (20 m)



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 90 | S-21 | 16<br>6<br>18 | - upper 12 inches: very stiff, wet, tan, sandy SILT<br>Very dense, wet, gray, very silty, fine SAND. | |
| 95 | S-22 | 23<br>20<br>22 | | |
| (30m) | | | | |
| 100 | S-23 | 9<br>14<br>19 | | |
| | | | - gravelly drilling | |
| 105 | S-24 | 30<br>40<br>38 | - gravelly drilling | |
| 110 | S-25 | 50/3" | - broken rock recovered in sampler | |
| (35m) | | | | |
| 115 | S-26 | 50/3" | - no sample recovery<br>Bottom of boring at 115.3 feet (35.2 m) below existing ground surface. | |
| 120 | | | | |
| 125 | | | | |
| 130 | | | | |
| (40m) | | | | |
| 135 | | | | |

**PROJECT NAME: Tolt Hill Bridge**
**PROJECT NO.: KC-300**

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/9/96
Ground Surface Elevation: 94.5 ft (28.8 m)

**BORING LOG B-3**
Page 1 of 2
Figure A-3

| Depth in Feet (in meters) | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 0 | | | Very dense, moist, gray, slightly gravelly, silty SAND. | |
| 5 | S-1 | 26<br>19<br>25 | | |
| 10 | S-2 | 4<br>2<br>3 | - no sample recovery, gravelly drilling | |
| 15 | S-3 | 10<br>9<br>27 | - void? drilled to 13 -1/2 feet, 10-foot casing dropped, added 5 feet of casing dropped | |
| (5m) | | | Very dense, wet, brown, sandy GRAVEL. | |
| 20 | S-4 | 45<br>35<br>50/4" | - gravelly drilling from 19 to 22 feet | |
| 25 | S-5 | 50/3" | Very dense, moist to wet, tan, slightly gravelly, silty, medium to fine SAND. | |
| 30 | S-6 | 30<br>27<br>29 | - gravelly drilling at 29 feet | |
| | | | Very dense, wet, tan, silty, very gravelly SAND.<br>- cobbly drilling at 31 feet | |
| (10m) 35 | S-7 | 33<br>34<br>22 | | |
| 40 | S-8 | 54<br>43<br>31 | - gravelly drilling | |
| 45 | S-9 | 35<br>38<br>29 | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/9/96
Ground Surface Elevation: 94.5 ft (28.8 m)



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 45 | | | Very dense, wet, tan, silty, very gravelly SAND. | |
| | | | - gravelly drilling | |
| | S-10 | 30 35 23 | | |
| (15m) 50 | | | | |
| | S-11 | 38 25 27 | - brown, sandy GRAVEL | |
| 55 | | | | |
| | S-12 | 26 14 16 | Very dense, wet, gray, gravelly, medium SAND and sandy GRAVEL. | |
| 60 | | | | |
| | S-13 | 50 20 18 | | |
| 65 (20m) | | | | |
| 70 | | | - gravelly drilling at 70 feet | |
| | S-14 | 150/6" | | |
| 75 | | | Bottom of boring at 74 feet (22.6 m) below existing ground surface. | |
| 80 | | | | |
| (25m) | | | | |
| 85 | | | | |
| 90 | | | | |

**PROJECT NAME:** Tolt Hill Bridge
**PROJECT NO.:** KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/11/96
Ground Surface Elevation: 67.2 ft (20.5 m)

**BORING LOG B-4**

Figure A-4

| Depth in Feet (in meters) | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 0 | | | Very soft to soft, wet, gray and brown, fine sandy SILT and wet, gray with iron-like staining, CLAY. | |
| | S-1 | 1<br>1<br>1 | | WC: 47% |
| | S-2 | | | WC: 39%<br>TV: 0.35 tsf, PP: 0.5 tsf |
| | S-3 | | - no sample recovery | |
| | S-4 | 1<br>1<br>1 | | WC: 43%<br>- #200: 60% |
| | | | - gravelly drilling at 22 feet | |
| | | | Very dense, wet, brown, sandy GRAVEL. | |
| | S-5 | 41<br>32<br>24 | - gravelly drilling | |
| | S-6 | 100/3" | - no sample recovery, sampler bouncing on rock | |
| | | | - gravelly drilling at 30 feet | |
| | | | Very dense, wet, gray, medium to fine SAND with occasional gravel lenses. | |
| | S-7 | 46<br>24<br>27 | | |
| | S-8 | 10<br>18<br>17 | - occasional wood fragments | |
| | S-9 | 19<br>16<br>18 | - gravelly drilling at 42-1/2 to 43-1/2 feet | |

| | |
|---|---|
| **PROJECT NAME:** Tolt Hill Bridge<br>**PROJECT NO.:** KC-300 | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 10/11/96<br>Ground Surface Elevation: 67.2 ft (20.5 m) |

**BORING LOG B-4**

Figure A-4

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 45 | | | Very dense, wet, gray, medium to fine SAND with occasional gravel lenses. | |
| (15m) | S-10 | 26<br>12<br>6 | - no sample recovery | |
| 50 | | | Stiff to very stiff, wet, gray, CLAY. | |
| 55 | S-11 | 13<br>11<br>13 | | WC: 38% |
| 60 | S-12 | | | WC: 40%<br>TV: 0.75 tsf, PP: 1.75 tsf<br>UU, AL<br>WC: 42%<br>TV: 0.75 tsf, PP: 1.75 tsf |
| 65<br>(20m) | S-13 | 14<br>11<br>15 | | |
| 70 | S-14 | 9<br>7<br>9 | | WC: 56% |
| 75 | S-15 | 23<br>13<br>11 | | WC: 40% |
| | | | - gravelly drilling at 77 feet | |
| 80 | S-16A<br>S-16B | 11<br>8<br>8 | - gray, silty, fine sand | |
| (25m) | | | | |
| 85 | S-17 | 0<br>4<br>3 | - medium stiff, wet gray CLAY. | WC: 43% |
| | S-18 | 12<br>24<br>50/6" | - gravelly drilling at 89 feet | |
| 90 | | | - no sample recovery<br>Very dense, wet, gray, medium to fine SAND. | |

| PROJECT NAME: Tolt Hill Bridge<br>PROJECT NO.: KC-300 | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 10/11/96<br>Ground Surface Elevation: 67.2 ft (20.5 m) | BORING LOG B-4<br>Page 3 of 3<br>Figure A-4 |
|---|---|---|

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 90 | | | Very dense, wet, gray, medium to fine SAND. | |
| 95 | S-19 | 29<br>27<br>33 | | |
| (30m) | | | | |
| 100 | | | | |
| 105 | S-20 | 28<br>37<br>37 | | |
| 110 | | | | |
| (35m)<br>115 | S-21 | 6<br>13<br>39 | | |
| | | | Bottom of boring at 115 feet (35.1 m) below existing ground surface. | |
| 120 | | | | |
| 125 | | | | |
| 130 | | | | |
| (40m) | | | | |
| 135 | | | | |

**PROJECT NAME:** Tolt Hill Bridge
**PROJECT NO.:** KC-300

**Client:** Lin & Associates, Inc.
**Project Location:** King County, Washington
**Date Exploration Completed:** 10/14/96
**Ground Surface Elevation:** 65.6 ft (20 m)



| Depth in Feet (in meters) | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 0 | | | Very soft, moist to wet, gray CLAY. | |
| | S-1 | 2<br>1<br>1 | - with iron-like staining | WC: 52% |
| 5 | | | | |
| | S-2 | | | WC: 49%<br>TV: 0.2 tsf, PP: 0.25 tsf<br>UU, AL, CN<br>WC: 43%<br>TV: 0.15 tsf, PP: 0.25 tsf |
| 10 | | 1/12"<br>1 | | |
| | S-3 | | | WC: 46% |
| | S-4 | 1/18" | | WC: 44% |
| 15 | | | | |
| (5m) | | | | |
| | S-5 | | - medium stiff, moist, gray, clayey SILT and silty CLAY | WC: 26%<br>TV: 0.45 tsf, PP: 1.75 tsf<br>WC: 29%<br>TV: 0.3 tsf, PP: 0.75 tsf<br>WC: 37%<br>TV: 0.375 tsf, PP: 0.5 tsf |
| 20 | | | Medium dense, wet, brown, sandy GRAVEL and gravelly, medium to fine SAND. | |
| | S-6 | 8<br>7<br>5 | | |
| 25 | | | | |
| | S-7 | 3<br>10<br>16 | | |
| 30 | | | | |
| (10m) | S-8 | 4<br>13<br>24 | | |
| 35 | | | | |
| | S-9 | 6<br>7<br>7 | | |
| 40 | | | Medium dense, wet, brown and gray, fine SAND. | |
| | S-10 | 11<br>16<br>11 | - silty, fine SAND | |
| 45 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/14/96
Ground Surface Elevation: 65.6 ft (20 m)



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 45 | | | Medium dense, wet, brown and gray, fine SAND. | |
| | S-11 | 7 9 17 | | |
| (15m) 50 | | | | |
| | S-12 | 12 9 9 | | |
| 55 | | | | |
| | S-13A | 3 5 6 | | |
| | S-13B | | Medium stiff to stiff, moist to wet, gray CLAY. | |
| 60 | | | | |
| | S-14 | | - moist, gray, medium to fine SAND | WC: 19% |
| 65 (20m) | | | | |
| | S-15 | 2 4 3 | - brown and gray, CLAY | WC: 39% |
| 70 | | | | |
| | S-16 | 0 5 7 | - with very silty, fine sand interbeds | WC: 37% |
| 75 | | | | |
| | S-17 | 0 0 1 | - very soft, with very silty, fine sand interbeds, sampler fell one foot under weight of hammer | AL, WC: 44% |
| 80 | | | | |
| (25m) | S-18 | 0 4 5 | - Interlayered very soft, wet, gray, clay and loose, silty, fine SAND | WC: 42% |
| 85 | | | | |
| | S-19 | 5 7 7 | - with fine sand interbeds | WC: 37% |
| 90 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 10/14/96
Ground Surface Elevation: 65.6 ft (20 m)



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| 90 | | | Medium stiff to stiff, moist to wet, gray CLAY. | |
| | S-20 | 5<br>0<br>4 | - with fine sand interbeds | WC: 38% |
| 95 | | | | |
| (30m) | S-21 | 3<br>13<br>27 | Very dense, wet, gray, medium to fine SAND. | |
| 100 | | | | |
| | S-22 | 8<br>13<br>13 | - medium dense, silty, fine sand | |
| 105 | | | | |
| 110 | | | | |
| (35m) | S-23 | 14<br>23<br>26 | - silty, fine sand | |
| 115 | | | | |
| | | | Medium stiff to stiff, wet, gray, SILT. | |
| 120 | | | | |
| | S-24 | 2<br>3<br>6 | | AL, WC: 35% |
| 125 | | | | |
| | S-25 | 0<br>0<br>4 | | WC: 36% |
| 130 | | | Bottom of boring at 129 feet (39.3 m) below existing ground surface. Groundwater encountered at about 4-1/2 feet (1.4 m) below existing ground surface at time of drilling. | |
| (40m) | | | | |
| 135 | | | | |

| PROJECT NAME:Tolt Hill Bridge<br>PROJECT NO.: KC-300A | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 9/4/02<br>Ground Surface Elevation: 64 Feet |  |
|---|---|---|

| Depth In Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Very soft, moist to wet, brown to gray, silty CLAY and clayey SILT. | |
| | S-1 | 0 | | |
| | | 0 | | |
| | | 0 | -weight of hammer advances sampler | |
| 5 | | | | |
| | S-2 | | | AL, UU |
| | | | | WC: 30, 31% |
| | | 0 | | TV:0.1 to 0.5 tsf |
| 10 | | 0 | | PP: 0 to 0.25 tsf |
| | | 0 | -weight of hammer advances sampler | |
| | S-3<br>S-4 | 0 | | |
| | | 0 | | |
| | | 0 | -weight of hammer advances sampler | |
| 15 | | | | |
| | S-5 | | | |
| | | | | WC: 34% |
| | | | Very loose to loose, wet, gray, silty, medium to fine SAND with wood . | WC: 44% |
| 20 | | 1 | | |
| | | 0 | - no sample recovered | |
| | | 1 | | |
| | S-6 | 0 | | |
| | | 2 | -silty fine sand to fine sandy silt with occasional wood | -200:56% |
| 25 | S-7 | 6 | | WC: 39% |
| | S-8 | 5 | Medium dense to very dense, wet, gray-brown, coarse to medium SAND. | |
| | | 11 | | |
| | | 13 | -gravelly | |
| 30 | | | -5 feet of heave requires use of drilling mud | |
| | S-9 | 12 | | |
| | | 19 | | |
| | | 32 | | |
| 35 | | | | |
| | S-10 | 12 | | |
| | | 16 | | |
| | | 22 | | |
| 40 | | | | |
| | S-11 | 9 | | |
| | | 11 | | |
| | | 15 | - with silty sand interbeds | |
| 45 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/4/02
Ground Surface Elevation: 64 Feet



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | S-12 | 4<br>5<br>12 | Medium dense to very dense, wet, gray-brown, coarse to medium SAND.<br>-silty medium to fine sand | |
| 50 | | | | |
| | S-13 | 3<br>1<br>4 | Medium stiff to stiff, moist to wet, gray SILT and CLAY with occasional silty, fine sand seams. | |
| 55 | | | | |
| | | 2 | -no sample recovered | |
| 60 | S-14 | 3<br>4 | -fine sandy silt | |
| | | | -no sample recovered | |
| 65 | | | -no sample recovered | |
| | | | -no sample recovered | |
| 70 | | | | |
| | S-15 | | | AL, UU<br>WC: 38%<br>WC: 45%<br>PP: 0.5-0.6 tsf<br>TV:0.2 tsf |
| 75 | | | | |
| | S-16 | 3<br>3<br>3 | | |
| 80 | | | | |
| | S-17 | 0<br>5<br>3 | | AL<br>WC: 45% |
| 85 | | | | |
| | S-18 | 0<br>0<br>3 | -soft | |
| 90 | | | Very stiff, moist to wet, gray, interbedded CLAY and SILT with fine sand seams. | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/4/02
Ground Surface Elevation: 64 Feet


Figure A-6

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | 4 | Very stiff, moist to wet, gray, interbedded CLAY and SILT with fine sand seams. | |
| | | 8 | | |
| 95 | S-19 | 10 | | |
| | S-20 | 8 | | |
| | | 13 | | |
| | | 10 | | |
| 100 | | | Medium dense to dense, wet, gray, fine SAND and fine sandy SILT. | |
| | S-21 | 5 | | |
| | | 8 | | |
| | | 14 | | |
| 105 | | | | |
| | S-22 | 6 | | |
| | | 8 | | |
| | | 11 | | |
| 110 | | | | |
| | S-23 | 8 | | |
| | | 11 | | |
| | | 20 | | |
| 115 | | | | |
| | S-24 | 8 | | |
| | | 11 | | |
| | | 16 | | |
| 120 | | | | |
| | S-25 | 8 | | |
| | | 12 | | |
| | | 19 | | |
| 125 | | | | |
| | S-26 | 11 | | |
| | | 22 | | |
| | | 23 | | |
| 130 | | | Bottom of boring at 129 feet below existing ground surface. Groundwater encountered at about 9 feet below existing ground surface at the time of drilling. | |
| 135 | | | | |

| PROJECT NAME:Tolt Hill Bridge<br>PROJECT NO.: KC-300A | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 9/5/02<br>Ground Surface Elevation: 68 Feet | BORING LOG B-7<br>Page 1 of 3<br>Figure A-7 |
|---|---|---|

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Very soft, moist to wet, brown to gray, fine sandy SILT and silty CLAY. | |
| | S-1 | 1<br>0<br>1 | | |
| 5 | | | | |
| | S-2 | 0<br>0<br>0 | -weight of hammer advances sampler | |
| 10 | S-3 | | | WC: 30, 28%; AL<br>PP: 0 to 0.25 tsf<br>TV: 0.1 to 0.15 tsf |
| | | 0<br>0<br>0 | -weight of hammer advances sampler | |
| 15 | S-4 | | | |
| | S-5 | | | WC: 49%<br>WC: 39% |
| | | | Medium dense, wet, gray, silty, medium to fine SAND with scattered wood fragments. | WC: 37%<br>PP:0.6 to 1.25 tsf |
| 20 | | | | |
| | S-6 | 18<br>23<br>28 | - very dense, medium to fine sand with scattered gravels | |
| 25 | | | | |
| | S-7 | 6<br>8<br>14 | | |
| 30 | | | | |
| | S-8 | 1<br>1<br>6 | -heave requires use of drilling mud<br>- gray-brown, interbedded, fine, medium and coarse sand with silt lenses | -200: 25%<br>WC: 26% |
| 35 | | | | |
| | S-9 | 3<br>6<br>12 | - medium sand | |
| 40 | | | | |
| | S-10 | 10<br>12<br>16 | -coarse to medium sand with silty, fine sand lenses | |
| 45 | | | | |
| | | | Medium to very stiff, wet, gray, fine sandy SILT. | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/5/02
Ground Surface Elevation: 68 Feet



| Depth In Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | S-11 | 1, 1, 5 | Medium stiff to very stiff, wet, gray, fine sandy SILT. | |
| 50 | S-12 | 9, 11, 17 | - medium dense, slightly silty, gravelly SAND | |
| 55 | S-13 | 7, 11, 17 | | |
| 60 | S-14 | 4, 4, 4 | Medium stiff to stiff, wet, gray, silty CLAY and clayey SILT with fine sand seams. | WC: 36%<br>AL |
| 65 | S-15 | 3, 3, 3 | | |
| 70 | S-16 | | | AL, UU, CN<br>WC: 42, 52 and 50%<br>PP: 0-1.25 tsf<br>TV:0.3 to 0.5 tsf |
| 75 | S-17 | 2, 4, 5 | | |
| 80 | S-18 | 4, 5, 6 | | |
| 85 | S-19 | 2, 3, 6 | | |
| 90 | | | Very loose, wet, gray-brown, fine SAND with trace silt. | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/5/02
Ground Surface Elevation: 68 Feet



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | 1 | Very loose, wet, gray-brown, fine SAND with silt seams. | |
| | | 1 | | |
| | S-20 | 1 | -20 feet of heave | |
| 95 | | | | |
| | S-21 | 1 | | |
| | | 2 | -soft, gray, clay lense | |
| | | 15 | | |
| 100 | | | Medium dense to dense, wet, gray, silty, fine SAND and fine sandy SILT. | |
| | S-22 | 13 | | |
| | | 16 | | |
| | | 19 | | |
| 105 | | | | |
| | S-23 | 9 | | |
| | | 18 | | |
| | | 18 | | |
| 110 | | | | |
| | S-24 | 8 | | |
| | | 16 | | |
| | | 19 | | |
| 115 | | | | |
| | S-25 | 6 | | |
| | | 12 | | |
| | | 16 | | |
| 120 | | | | |
| | S-26 | 10 | | |
| | | 20 | | |
| | | 16 | | |
| 125 | | | | |
| | S-27 | 8 | | |
| | | 15 | | |
| | | 17 | | |
| 130 | | | Bottom of boring at 129 feet below existing ground surface. | |
| 135 | | | | |

| PROJECT NAME: Tolt Hill Bridge<br>PROJECT NO.: KC-300A | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 9/6/02<br>Ground Surface Elevation: 64 Feet | BORING LOG B-8<br>Page 1 of 3<br>Figure A-8 |
|---|---|---|

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Very soft to soft, wet, brown to gray, clayey SILT. | |
| | | 0 | | |
| | | 0 | -weight of hammer advances sampler | |
| | | 0 | -no sample recovered | |
| 5 | | | | |
| | S-1 | | | AL, UU, CN<br>WC: 45, 41, 39%<br>PP: 0 to 0.1 tsf<br>TV: 0.1 tsf |
| 10 | | 0 | | |
| | | 0 | | |
| | S-2 | 1 | | |
| | | 0 | | |
| | | 0 | | |
| 15 | S-3 | 3 | | |
| | S-4 | | Medium dense to dense, wet, brown-gray, slightly silty, gravelly, medium to fine SAND.<br>-silty fine sand | |
| 20 | | | | |
| | | 5 | | |
| | S-5 | 10 | | |
| | | 14 | | |
| 25 | | | - rough drilling between 25 to 30 feet, possible cobbles | |
| | S-6 | 15 | | |
| | | 20 | | |
| | | 16 | | |
| 30 | | | | |
| | S-7 | 7 | | |
| | | 5 | | |
| | | 1 | -loose zone | |
| 35 | | | | |
| | S-8 | 5 | | |
| | | 13 | | |
| | | 25 | - coarse to fine sand with scattered gravels | |
| 40 | | | | |
| | S-9 | 6 | | |
| | | 10 | | |
| | | 14 | -medium to fine sand with scattered gravels | |
| 45 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/6/02
Ground Surface Elevation: 64 Feet



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | S-10 | 6 | | |
| | | 7 | - medium to fine sand with scattered gravel | |
| | | 13 | Stiff to very stiff, wet, gray, fine sandy SILT and silty, fine SAND with scattered organics. | |
| 50 | | | | |
| | S-11 | 0 | | |
| | | 3 | | |
| | | 9 | | |
| 55 | | | | |
| | S-12 | 12 | | |
| | | 14 | | |
| | | 12 | | |
| 60 | | | Medium stiff to stiff, wet, gray, silty CLAY and clayey SILT. | |
| | S-13 | 3 | | |
| | | 5 | | |
| | | 4 | | |
| 65 | | | | |
| | S-14 | | | AL, UU<br>WC: 36,34,36%<br>PP: 0.75 to 2.0 tsf<br>TV:0.25 to 0.6 tsf |
| 70 | | | | |
| | S-15 | 1 | | |
| | | 2 | | |
| | | 3 | | |
| 75 | | | | |
| | S-16 | 0 | | |
| | | 4 | | |
| | | 3 | | |
| 80 | | | | |
| | S-17 | 0 | | |
| | | 3 | | |
| | | 5 | -fine sandy silt and silty fine sand | |
| 85 | | | | |
| | S-18 | 0 | | |
| | | 0 | - weight of rods advances sampler | AL |
| | | 0 | - very soft clay | WC: 34% |
| 90 | | | Stiff, wet, gray, interbedded SILT and CLAY with fine sand seams. | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 9/6/02
Ground Surface Elevation: 64 Feet



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | S-19 | 4, 4, 4 | Stiff, wet, gray, interbedded SILT and CLAY with fine sand seams. | |
| 95 | | | | |
| | S-20 | 0, 2, 4 | Medium dense to dense, wet, gray, silty, fine SAND and fine sandy SILT.<br>-loose zone<br>-20 feet of heave | |
| 100 | | | | |
| | S-21 | 4, 6, 11 | | |
| 105 | | | | |
| | S-22 | 5, 6, 10 | | |
| 110 | | | | |
| | S-23 | 10, 16, 17 | | |
| 115 | | | | |
| | S-24 | 6, 11, 16 | | |
| 120 | | | | |
| | S-25 | 2, 9, 11 | | |
| 125 | | | | |
| | S-26 | 13, 16, 18 | - gray, fine sand | |
| 130 | | | Bottom of boring at 129 feet below existing ground surface. Groundwater encountered at about 11 feet below existing ground surface at time of drilling. | |
| 135 | | | | |

PROJECT NAME:Tolt Hill Bridge
PROJECT NO.: KC-300A

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 6/18/03
Ground Surface Elevation: 70 Feet



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Six inches of asphalt over medium dense to loose, damp, | |
| | S-1 | 11 | brown, sandy GRAVEL. (Roadway fill) | |
| | | 7 | | |
| | | 3 | -loose, damp, brown, silty SAND with scattered gravel | |
| 5 | | | | |
| | | | -drilled through 18-inch diameter corrugated metal pipe at 5-1/2' depth | |
| | S-2 | 0 | | |
| | | 0 | Very soft to medium stiff, wet to moist, gray, silty CLAY. | |
| | | 0 | -weight of hammer advances sampler | MC: 49% |
| 10 | | | | |
| | S-3 | 0 | | |
| | | 0 | | |
| | | 1 | | MC: 45% |
| 15 | | | | |
| | SH-1 | | | |
| | | | | MC: 48, 50%, UU, AL |
| | | | | PP: 1 tsf, TV: 0.25 tsf |
| 20 | | 3 | Very soft to soft, moist, dark brown SILT with pockets of organics. | MC: 102% |
| | | 7 | Medium dense, wet, gray, silty, fine SAND with silt interbeds. | PP: 0.75 tsf, TV: 0 tsf |
| | | 11 | | |
| | S-4 | 6 | Medium dense to very loose, wet, gray and brown, coarse to | |
| | | 8 | medium SAND with scattered gravels. | |
| 25 | S-5 | 9 | -fine to medium sand | |
| | S-6 | 2 | | |
| | | 0 | | |
| | | 1 | -medium sand with silty, fine sand lenses | |
| 30 | | | | |
| | S-7 | 6 | | |
| | | 6 | | |
| | | 4 | -medium to coarse sand | |
| 35 | | | | |
| | S-8 | 3 | | |
| | | 1 | | |
| | | 1 | -slightly silty | |
| 40 | | | | |
| | S-9 | 5 | | |
| | | 9 | | |
| | | 9 | | |
| 45 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 6/18/03
Ground Surface Elevation: 70 Feet



| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | S-10 | 12 | Medium dense to dense, wet, brown and gray, medium to coarse SAND. | |
| | | 17 | | |
| | | 35 | -scattered gravel | |
| 50 | | | | |
| | S-11 | 9 | | |
| | | 13 | | |
| | | 16 | | |
| 55 | | | | |
| | S-12 | 1 | Very soft, wet, gray, slightly clayey SILT. | MC: 45% |
| | | 0 | | |
| | | 1 | | |
| 60 | | | | |
| | SH-2 | | | |
| | | | Stiff to very stiff, moist to wet, gray, silty, fine to medium SAND and fine sandy SILT. | MC: 23% |
| 65 | | 3 | | |
| | | 0 | - slightly silty to silty | |
| | S-13 | 5 | | |
| | | 14 | | |
| | | 15 | | |
| 70 | S-14 | 17 | | |
| | S-15 | 6 | | |
| | | 5 | | |
| | | 8 | | MC: 30% |
| 75 | | | | |
| | S-16 | 6 | Stiff, moist to wet, gray, silty CLAY and CLAY | |
| | | 6 | | |
| | | 7 | | MC: 26% |
| 80 | | | | |
| | SH-3 | | | |
| | | | -fine sand lenses | MC: 43, 37, 58% |
| 85 | | 1 | -varved clay | UU, AL, PP:.75-2.25 |
| | | 4 | | TV:0.5 tsf |
| | S-17 | 6 | | MC: 48% |
| | | SH-4 | | MC: 39, 51, 32% |
| | | | | UU, AL, PP:1, 2.25 |
| 90 | | 4 | | TV: 0.5 tsf |
| | | 4 | | |
| | S-18 | 8 | | MC: 43% |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 6/18/03
Ground Surface Elevation: 70 Feet


Figure A-9

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | SH-5 | Stiff, moist to wet, gray, silty CLAY and CLAY with increasing fine sand lenses with depth. | MC:28, 43, 53%<br>AL, CN, TV: 0.3, .5 tsf<br>PP: 2.25, 0.75 tsf |
| 95 | S-19 | 6<br>5<br>5 | | MC: 42 % |
| | | | -sampler dropped through sample interval, no driven sample taken | |
| 100 | S-21 | 3<br>3<br>4 | | MC: 29% |
| 105 | S-22 | 0<br>7<br>5 | | MC: 27% |
| 110 | S-23 | 6<br>6<br>15 | Very stiff, wet, gray SILT.<br>- fine sandy silt and silty fine sand | |
| 115 | S-24 | 3<br>6<br>12 | - slighty clayey silt lenses and occasional fine gravel | |
| 120 | S-25 | 1<br>1<br>8 | Loose to medium dense, wet, gray, silty, fine to medium SAND. | |
| 125 | S-26 | 6<br>7<br>8 | - 6-inch thick clayey silt lense | |
| | | | Bottom of boring at 129 feet below existing ground surface. | |
| 130 | | | | |
| 135 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 6/19/03
Ground Surface Elevation: 93 Feet

BORING LOG B-10
Page 1 of 1
Figure A-10

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Three inches of asphalt over dense, damp, brown, sandy GRAVEL. | |
| | S-1 | 4 | | |
| | | 2 | Loose, damp, reddish-brown, slightly silty, gravelly, medium to fine SAND with occasional charcoal fragments. (FILL) | |
| | | 2 | | |
| 5 | | | | |
| | S-2 | 3 | | |
| | | 4 | | |
| | | 11 | Medium dense to dense, damp, brown, slightly silty, gravelly SAND. | |
| 10 | | | -rough drilling at 9 feet | |
| | S-3 | 13 | | |
| | | 16 | -rough drilling from 12-1/2 to 17-1/2 feet | |
| | | 19 | -gravelly, medium to fine sand | |
| 15 | | | | |
| | S-4 | 11 | | |
| | | 10 | | |
| | | 9 | -moist, reddish-brown | |
| 20 | | | | |
| | S-5 | 8 | | |
| | | 7 | | |
| | | 12 | -reddish-brown, wet, slightly silty to silty, gravelly SAND | |
| 25 | | | Bottom of boring at 24 feet below existing ground surface. Groundwater encountered at about 21 feet below existing ground surface at time of drilling. | |
| 30 | | | | |
| 35 | | | | |
| 40 | | | | |
| 45 | | | | |

| PROJECT NAME: Tolt Hill Bridge<br>PROJECT NO.: KC-300A | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 3/24/04<br>Ground Surface Elevation: 69 Feet | BORING LOG B-11<br>Page 1 of 1<br>Figure A-11 |
|---|---|---|

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Loose, moist, gray-brown, fine to medium SAND with gravel. (FILL) | |
| | S-1 | 3/3/3 | - no sample recovery | |
| 5 | | 0/0/1 | Very soft, moist to wet, mottled gray-brown SILT grading to very soft, wet, gray silty CLAY and clayey SILT. (ML) | |
| | | 0/1/2 | - no sample recovery | |
| 10 | SH-1 | | | MC: 43, 50%, CN, AL<br>PP:3.25 tsf, TV: 0.55 tsf |
| | | 0/0/0 | - weight of hammer advances sampler | |
| 15 | S-2 | | | |
| | S-3 | 0/0/0 | - weight of hammer advances sampler | |
| 20 | SH-2 | | Medium dense, wet, gray, fine to medium SAND with some silt. | |
| | | 10/14/12 | | |
| 25 | S-4 | | Medium dense to dense, wet, gray-brown, gravelly, medium to coarse SAND with silt. | |
| | S-5 | 9/10/11 | | |
| 30 | | | | |
| | S-6 | 4/8/9 | | |
| 35 | S-7 | | | |
| | | 14/16/22 | | |
| 40 | | | Bottom of boring at 39 feet below existing ground surface. Groundwater encountered at about 8 feet below existing ground surface at time of drilling. | |
| 45 | | | | |

| PROJECT NAME: Tolt Hill Bridge<br>PROJECT NO.: KC-300A | Client: Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 3/24/04<br>Ground Surface Elevation: 67-1/2 Feet | BORING LOG B-12<br>Page 1 of 1<br>Figure A-12 |
|---|---|---|

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Loose, moist to wet, mottled gray-brown, silty, fine SAND. | |
| | S-1 | 1/1/2 | | |
| 5 | S-2 | 0/0/0 | - weight of hammer advances sampler | |
| | | | Very soft, wet, mottled gray-brown with iron staining, clayey SILT grading to silty CLAY. (ML) | |
| | S-3 | 0/0/0 | - weight of hammer advances sampler | MC: 42% |
| 10 | S-4 | 0/0/0 | -no iron staining, weight of hammer advances sampler | MC: 42%, AL |
| | | | Very soft, wet, gray, fine sandy SILT with clayey silt seams. | |
| | S-5 | 0/0/0 | - weight of hammer advances sampler | MC: 32% |
| 15 | | | | |
| | S-6 | 0/0/0 | - weight of hammer advances sampler | |
| 20 | | | Medium dense, wet, gray-brown, slightly silty, medium SAND with scattered fine gravel. | |
| | S-7 | 4/9/22 | | |
| 25 | | | Bottom of boring at 24 feet below existing ground surface. Groundwater encountered at about 7-1/2 feet below existing ground surface at time of drilling. | |
| 30 | | | | |
| 35 | | | | |
| 40 | | | | |
| 45 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 3/25/04
Ground Surface Elevation: 68 Feet

BORING LOG B-13

Figure A-13

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | S-1 | | Very soft, moist to wet, mottled gray-brown with iron staining, clayey SILT and fine sandy SILT. (ML) | |
| | | 0/0/0 | -weight of hammer advances sampler | MC: 36% |
| 5 | S-2 | 0/0/0 | -weight of hammer advances sampler | MC: 57% |
| | S-3 | 0/0/0 | -weight of hammer advances sampler | MC: 46% |
| 10 | SH-1 | | - no iron staining | MC: 41, 35%, UU, AL<br>PP: 0.25 to 0.5 tsf<br>TV: 0.15 to 0.2 tsf |
| | | 0/0/0 | -weight of hammer advances sampler | |
| | S-4 | | | |
| 15 | | S-5 | - no recovery with shelby sampler, pushed SPT sampler to obtain soil sample | |
| | | | Very loose, wet, gray, slightly silty, fine SAND. | |
| 20 | | 0/0/0 | -weight of hammer advances sampler | |
| | S-6 | | | |
| | | 5/7/14 | Medium dense to loose, wet, gray and brown, coarse to medium SAND with scattered gravels. | |
| 25 | S-7 | | | |
| | S-8 | | | |
| | | 5/8/6 | - 6 inches of heave | |
| 30 | | | | |
| | S-9 | | | |
| | | 5/12/9 | | |
| 35 | | | | |
| | S-10 | | | |
| | | 11/12/7 | | |
| 40 | | | | |
| | S-11 | | | |
| | | 4/3/4 | - interbedded silty fine sand and silt | |
| 45 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

Client:Lin & Associates, Inc.
Project Location: King County, Washington
Date Exploration Completed: 3/25/04
Ground Surface Elevation: 68 Feet

BORING LOG B-13

Figure A-13

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | | Medium dense to dense, wet, brown and gray, medium to coarse SAND. | |
| | | 15/17/10 | -no sample recovered | |
| 50 | | | | |
| | S-12 | | Soft, wet, gray SILT with silty fine sand seams. | |
| | | 0/0/3 | | |
| 55 | | | | |
| | S-13 | | Medium stiff, wet, gray, clayey SILT with fine sand and clay seams. | |
| | | 4/5/4 | | |
| 60 | | | | |
| | S-14 | | | |
| | | 4/5/6 | | |
| 65 | | | | |
| | S-15 | | | |
| | | 5/6/3 | | |
| 70 | | | | |
| | S-16 | | | |
| | | 1/0/0 | - very soft clay with fine sand seams | |
| 75 | | | | |
| | S-17 | | | |
| | | 1/4/3 | -varved clay | |
| 80 | | | | |
| | S-18 | | | |
| | | 0/1/2 | -soft varved clay | |
| 85 | | | | |
| | S-19 | | | |
| | | 0/2/3 | | |
| 90 | | | | |
| | | | Medium dense to dense, wet, gray and brown, silty to clean, fine to medium SAND. | |

| PROJECT NAME: Tolt Hill Bridge<br>PROJECT NO.: KC-300A | Client:Lin & Associates, Inc.<br>Project Location: King County, Washington<br>Date Exploration Completed: 3/25/04<br>Ground Surface Elevation: 68 Feet | BORING LOG B-13<br>Page 3 of 3<br>Figure A-13 |
|---|---|---|

| Depth in Feet | Sample | Blows Per Six Inches | SOIL DESCRIPTION | Laboratory Test Information |
|---|---|---|---|---|
| | | 1/5/10 | Medium dense to dense, wet, silty to clean, fine to medium SAND with silt and clay seams. | |
| 95 | S-20 | | | |
| | S-21 | | | |
| | | 15/18/22 | - 10 inches of heave in auger | |
| 100 | | | | |
| | S-22 | | | |
| | | 9/16/24 | - 3 feet of heave in auger | |
| 105 | | | | |
| | S-23 | | | |
| | | 6/6/10 | - 2 feet of heave in auger | GS |
| 110 | | | Bottom of boring at 109 feet below existing ground surface. Monitoring well installed. | |
| 115 | | | | |
| 120 | | | | |
| 125 | | | | |
| 130 | | | | |
| 135 | | | | |

PROJECT NAME: Tolt Hill Bridge
PROJECT NO.: KC-300A

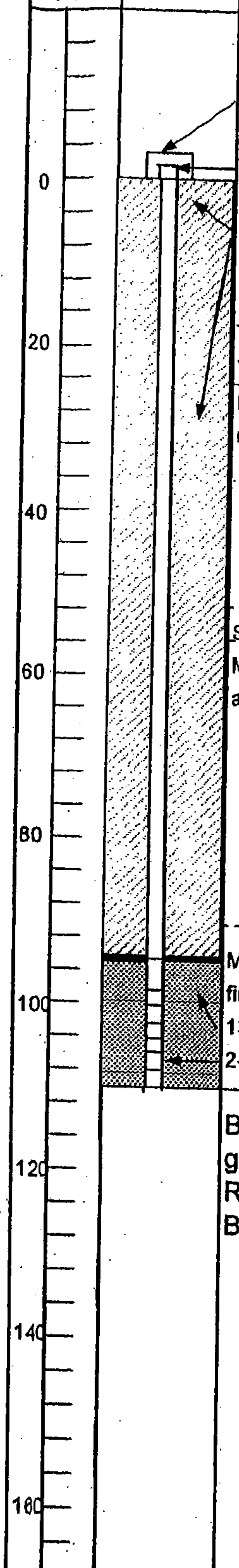
Client: Lin & Associates, Inc.
Project Location: King County, Washington
Date Monitoring Well Completed: 3/26/04
Ground Surface Elevation: 68 Feet


Figure A-14

Depth in Feet

## MONITORING WELL INSTALLATION DETAILS

Groundwater Level




Top of 6-inch diameter well monument is about 2.75' above ground surface.

Top of 2-inch diameter well casing is about 2.29' above ground surface

Concrete surface seal and grout to 94' below ground surface.

Very soft, moist to wet, mottled gray-brown with iron staining, clayey SILT and fine sandy SILT.

Very loose, wet, gray, slightly silty, fine SAND.

Medium dense to loose, wet, gray and brown, coarse to medium SAND with scattered gravels.

Soft, wet, gray SILT with silty fine sand seams.

Medium stiff, wet, gray, clayey SILT with fine sand and clay seams.

Medium dense to dense, wet, gray and brown, silty to clean, fine to medium SAND.

13 feet of 10-20 silica sand around screen

2-inch diameter, 10-foot long, 20 slot size screen

Bottom of monitoring well casing at 107 feet below existing ground surface.
Refer to Figure A-13 for detailed subsurface information for Boring B-13.

3/26/04: 1.65' above gs
3/27/04: 1.96' above gs
3/30/04: 2.00' above gs
4/21/04: 1.54' above gs

## Ground Water Level at the Tolt Bridge Replacement Pier 3

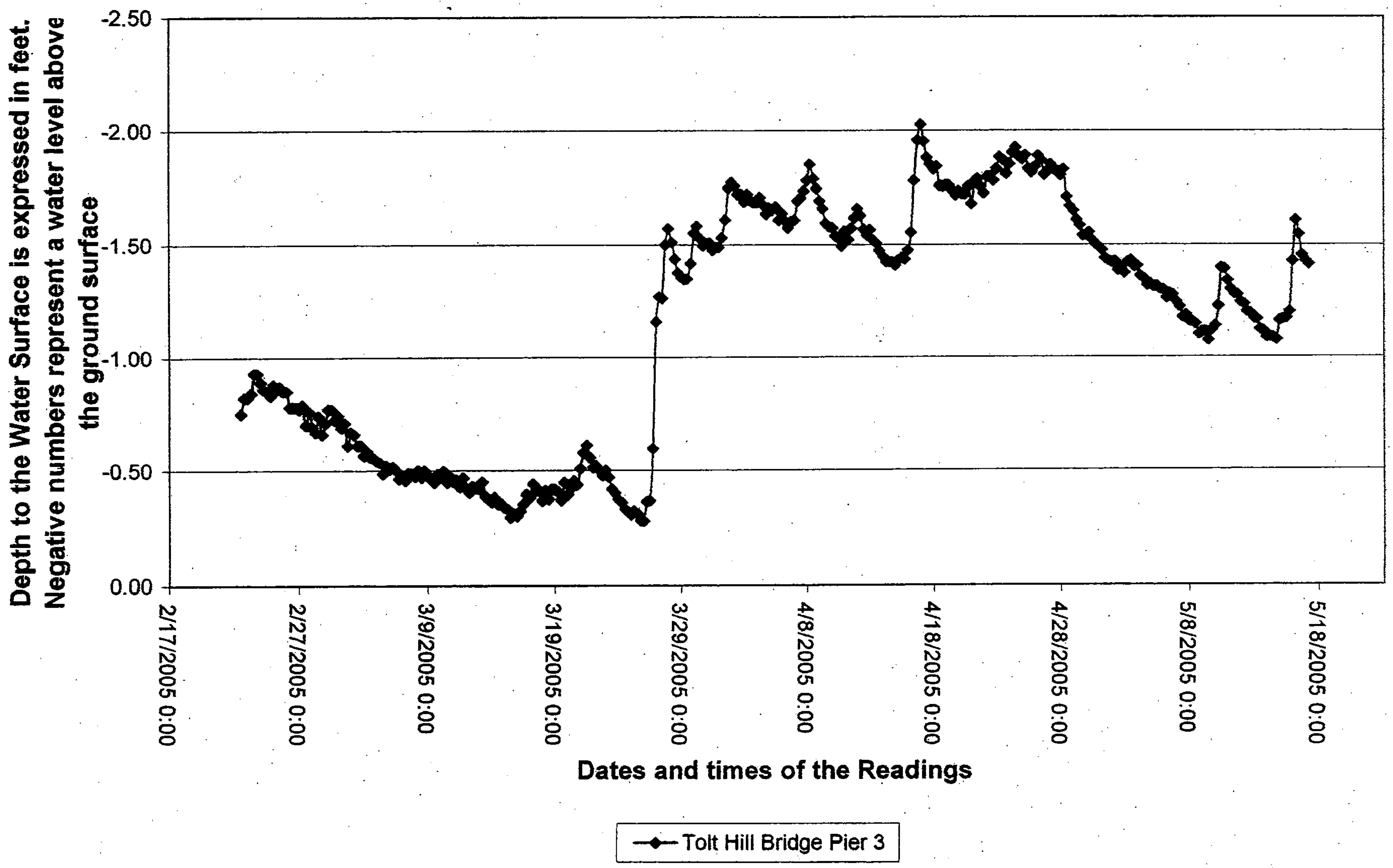



Figure A-15

# APPENDIX B

# LABORATORY TESTING PROGRAM

# APPENDIX B

## LABORATORY TESTING PROGRAM

A laboratory testing program was performed for this study to evaluate the basic index properties and geotechnical engineering properties of the site soils. Laboratory tests were performed on disturbed and relatively undisturbed samples. The laboratory tests performed and the procedures followed are outlined below. The laboratory testing was conducted by Soil Technology of Bainbridge, Washington in 1996. The testing in 2002 through 2004 was conducted by Rosa Environmental, later to become Analytical Resources, Inc. of Tukwila, Washington. The laboratory testing program was assigned by Lorilla Engineering, Inc.

### Soil Classification

Soil samples recovered in the explorations were visually classified in the field and then taken to our subcontract laboratory where the classifications were verified in a relatively controlled environment. Visual-manual field and laboratory observations include density/consistency, moisture condition, grain size and plasticity estimates.

The classifications of selected samples were checked by performing grain size analysis and Atterberg limits. Classifications were made in general accordance with the Unified Soil Classification (USC) system, ASTM D 2487.

### Water Content Determination (WC)

Water contents were determined for a portion of the samples recovered in the explorations in general accordance with ASTM D 2216. Water contents were not determined for very small samples or samples where large gravel contents would result in unrepresentative values. The results of the tests are presented on the exploration logs at the respective sample depth.

### Pocket Penetrometer and Torvane (PP and TV)

The pocket penetrometer and torvane procedures provide quick approximate tests of the consistency (undrained shear strength) of a cohesive soil sample. The pocket penetrometer device consists of a calibrated spring mechanism that measures penetration resistance of a ¼-inch diameter steel tip over a given distance. The penetration resistance is correlated to the unconfined compressive strength of the soil, which is typically twice the undrained shear strength of a saturated, cohesive soil.

The torvane device consists of a 1-inch diameter plate with eight equally spaced and radially arranged ¼-inch vanes. The vanes are pressed into the soil and the device is rotated. The vanes force a shear failure to take place over the area of the face of the plate, and the resistance at failure is measured by a calibrated spring. This measured resistance is correlated to the undrained shear strength of the sample tested. The results of the pocket penetrometer and torvane tests are presented on the exploration logs.

### Consolidation Test

The one-dimensional consolidation test provides data for developing settlement estimates. The test was performed in general accordance with ASTM D 2435. A relatively undisturbed, fine-grained sample was carefully trimmed and fit into a rigid ring with porous stones placed on the top and bottom of the sample to allow drainage. Vertical loads were then applied to the sample incrementally in such a way that the sample was allowed to consolidate under each load increment. Measurements were made of the compression of the sample with time under each load increment. In general, each load was left in place until the completion of 100% primary consolidation, as computed using Taylor's square root of time method. The next load was soon applied after attaining 100% primary consolidation. The test results plotted in terms of axial strain and the coefficient of consolidation versus applied load (stress) are presented on Figures B-1 through B-6.

**Triaxial Unconsolidated Undrained Compression Test (UU)**

The triaxial unconsolidated undrained compression test is a method to estimate the undrained shear strength of the soil. The test was performed in general accordance with ASTM D 2850. A relatively undisturbed fine-grained sample was trimmed to a length of 6 inches, encased in a rubber membrane and placed in the triaxial cell. An all-around confining pressure was applied hydraulically, but the sample was not allowed to consolidate, and no back pressure was applied. An axial load was then applied at a constant strain rate to the sample without allowing drainage from the sample. The stress-strain behavior was recorded until failure occurred. The failure stress was generally taken as the maximum load on the sample or the load recorded at 20 percent strain, whichever was greater. The test results are presented on Figures B-7 through B-19. The shear strength is considered to be one-half the maximum stress difference.

**Grain Size Analysis (GS)**

Grain size analyses were performed on selected samples in general accordance with ASTM D 422. The wet sieve analysis method was used for most samples and determines the grain size distribution greater than the U.S. No. 200 mesh sieve. The results of the tests are presented as curves on Figures B-20 and B-21 plotting percent finer by weight versus grain size.

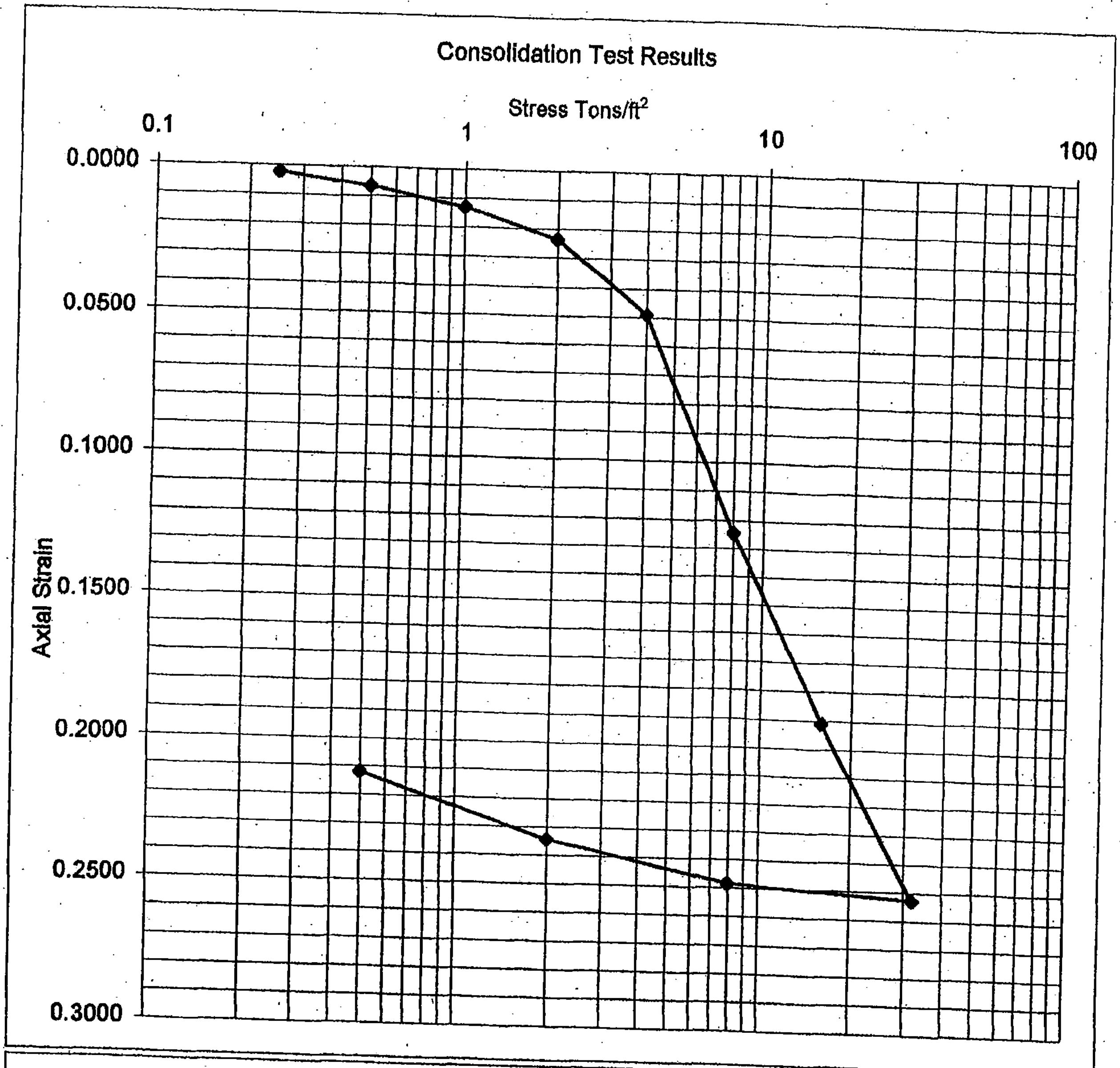
**Percent Passing the U.S. No. 200 Sieve (-200)**

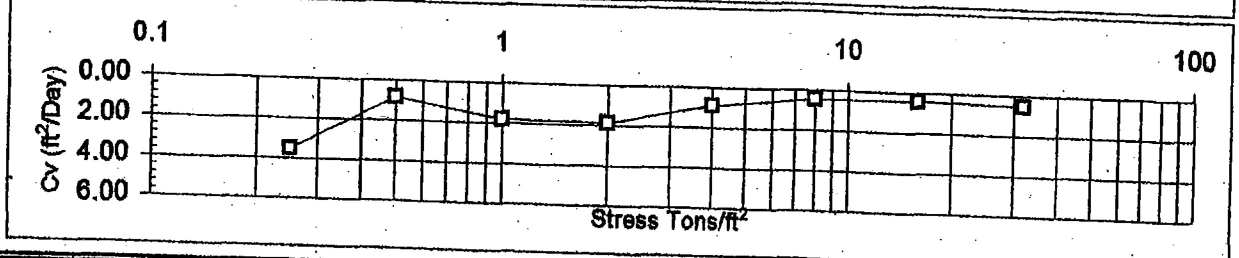
Selected samples were subjected to a modified grain size classification known as a 200 wash. The samples were washed through the No. 200 sieve to determine the relative percentages of the coarse and fine-grained material in the samples. The test was performed in accordance with ASTM D 1140. The percent passing value represents the percentage of the sample finer than the U.S. No. 200 sieve. The test results are presented on the exploration logs at the respective sample depths.

**Atterberg Limits (AL)**

Atterberg limit determinations were obtained for selected fine-grained soil samples. The liquid limit and plastic limit were determined in general accordance with ASTM D 4318. The results of the Atterberg limit tests and the plasticity characteristics are summarized on Plasticity Charts on Figures B-22 through B-27, which relates the plasticity index (liquid limit minus plastic limit) to the liquid limit.

Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300



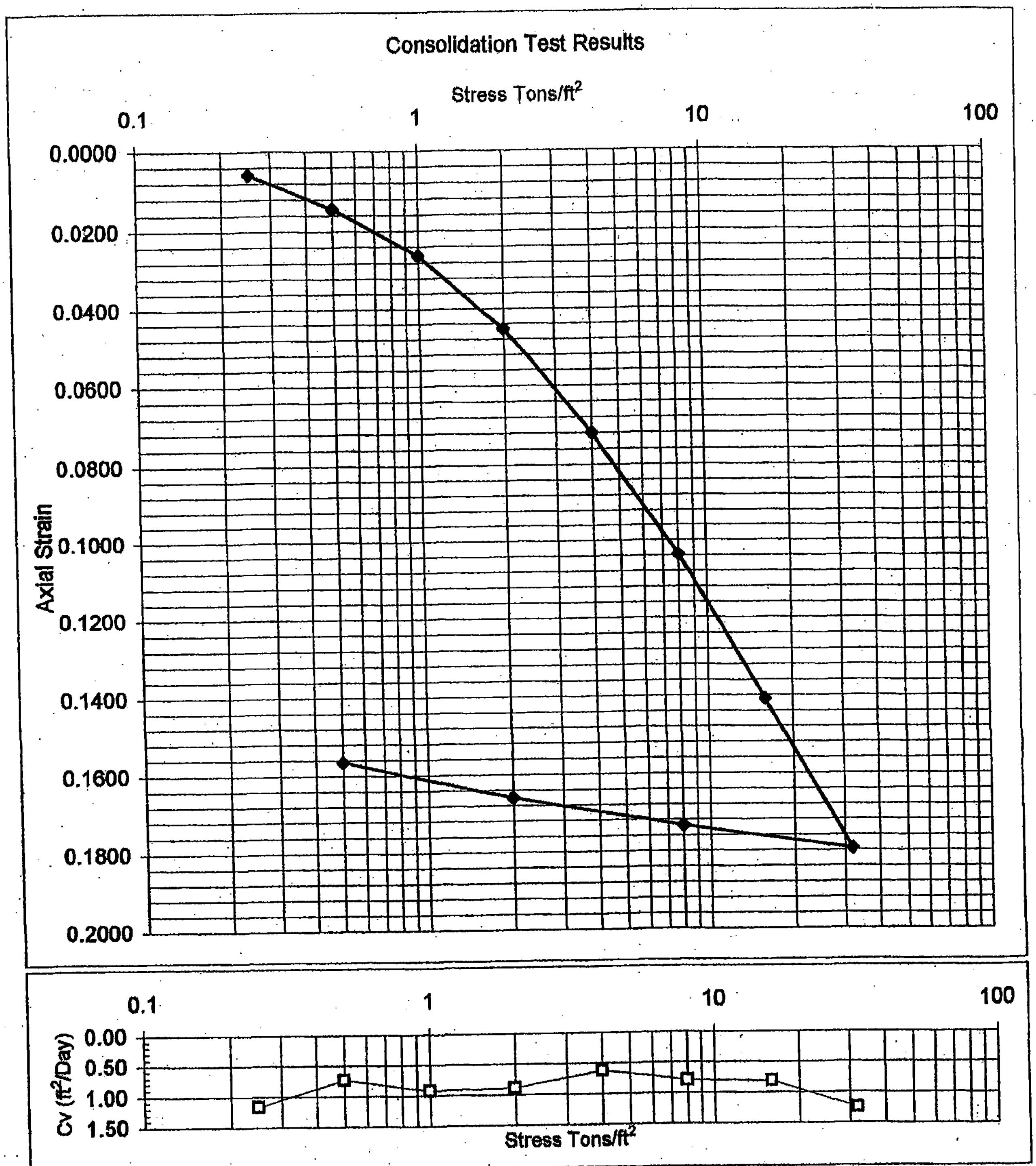




| Exploration Number | Sample Number | Depth ft | Moisture Content % | | Atterberg Limits | | | Wet Density pcf | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | Before | After | LL | PL | PI | | |
| B-2 | S-12 | 50.6-50.7 | 47 | 31 | 35 | 21 | 14 | 110 | CL |

Soil Technology, Inc.

**Figure B-1**

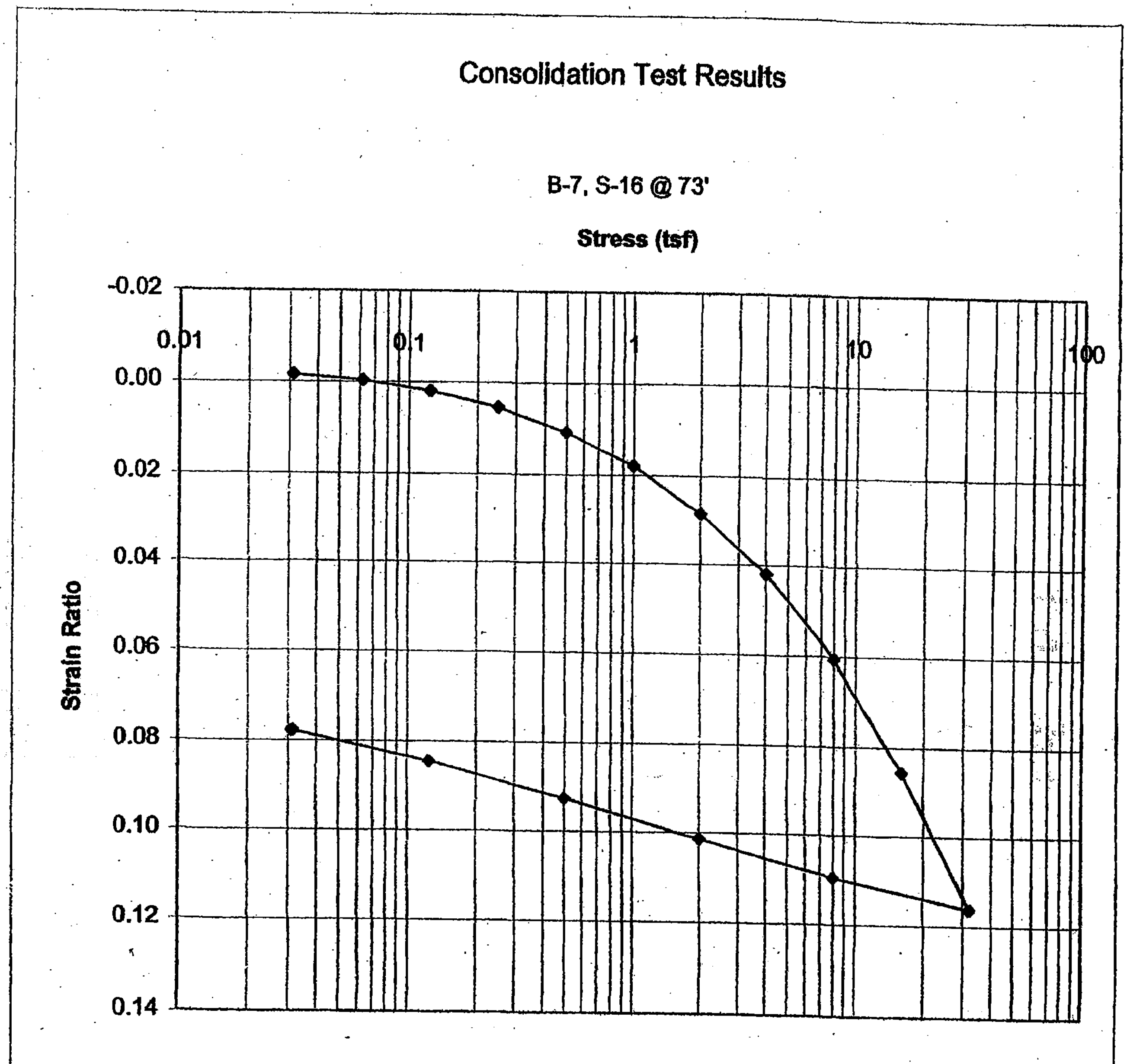
Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300



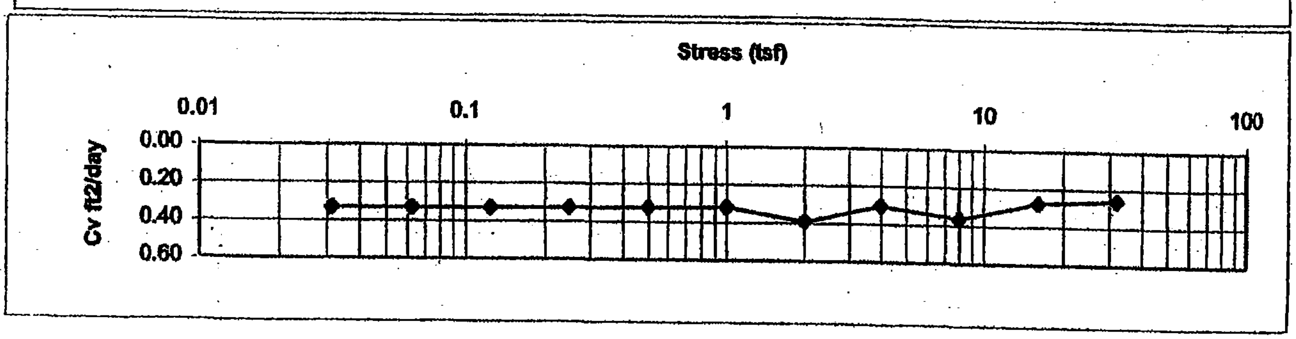

| Exploration Number | Sample Number | Depth ft | Moisture Content % | | Atterberg Limits | | | Wet Density pcf | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | Before | After | LL | PL | PI | | |
| B-5 | S-2 | 9.2-9.3 | 36 | 25 | 49 | 25 | 24 | 118 | CL |

Soil Technology, Inc.

**Figure B-2**



Consolidation Test Results
B-7, S-16 @ 73'
Stress (tsf)
Strain Ratio
-0.02
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.01
0.1
1
10
100

Stress (tsf)
Cv ft2/day
0.00
0.20
0.40
0.60
0.01
0.1
1
10
100


Figure B-3

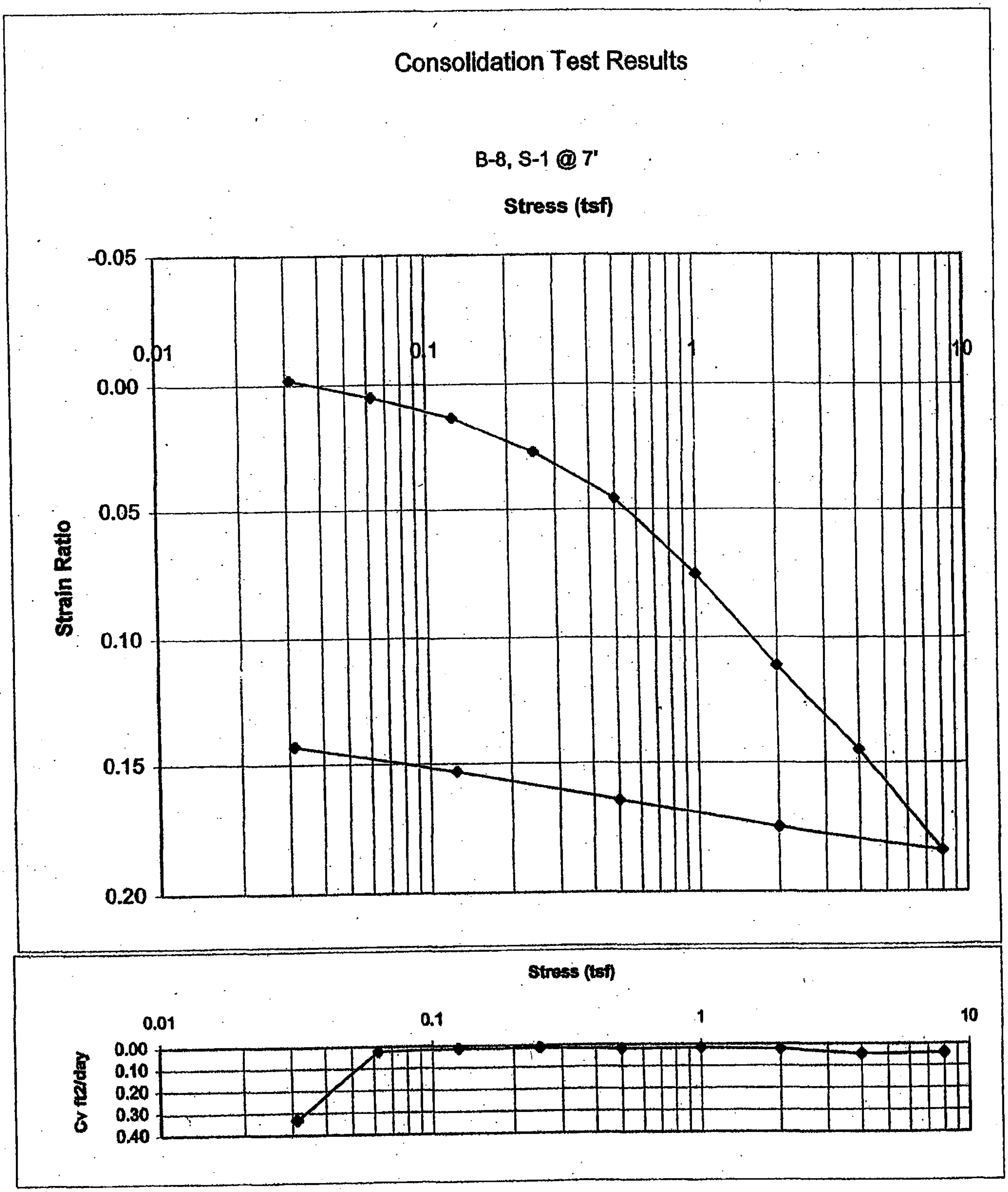
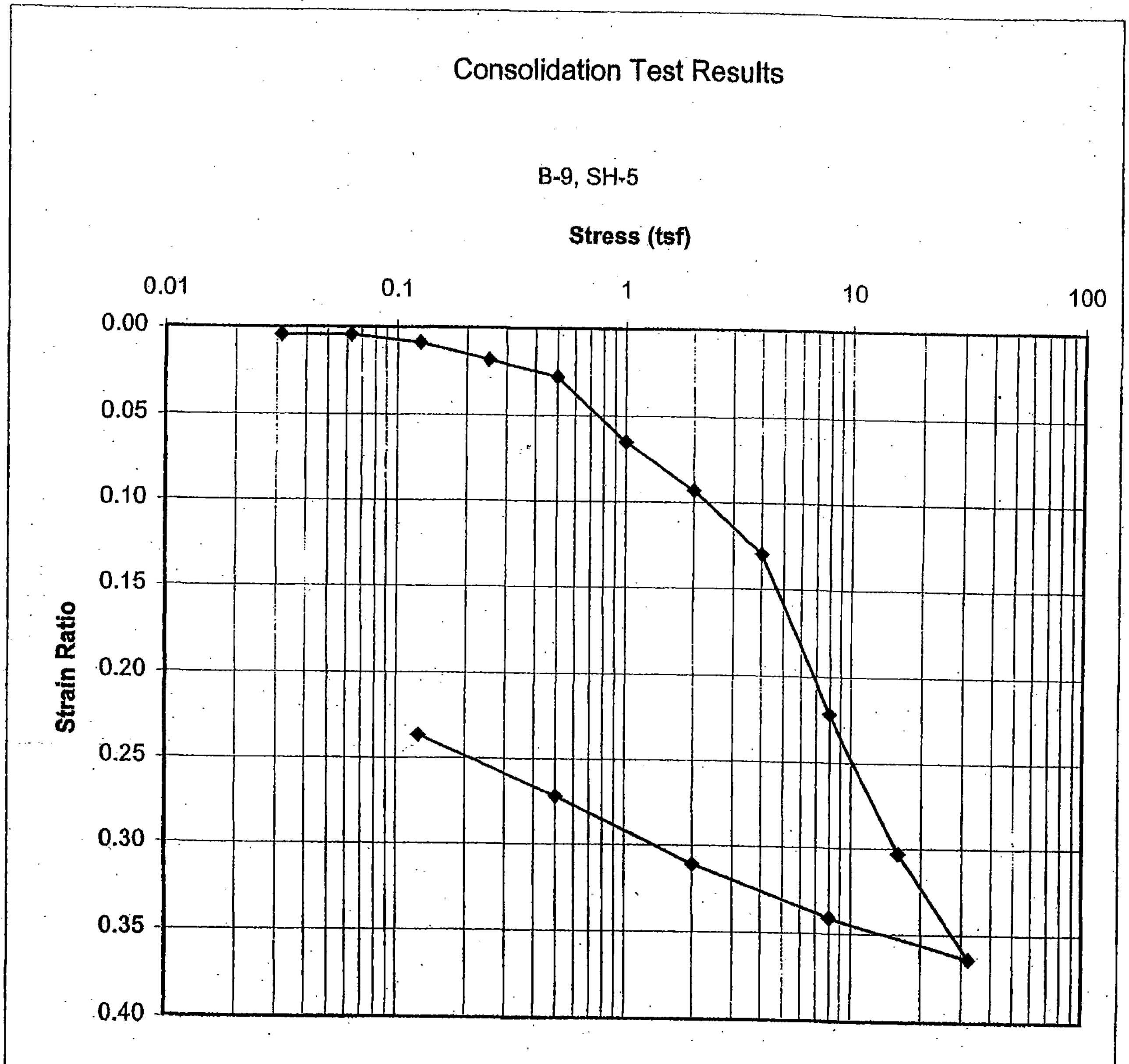

Consolidation Test Results
B-8, S-1 @ 7'
Stress (tsf)
Strain Ratio
-0.05
0.00
0.05
0.10
0.15
0.20
0.01
0.1
1
10
Stress (tsf)
Cv ft2/day
0.00
0.10
0.20
0.30
0.40
0.01
0.1
1
10

Figure B-4

ANALYTICAL RESOURCES INCORPORATED



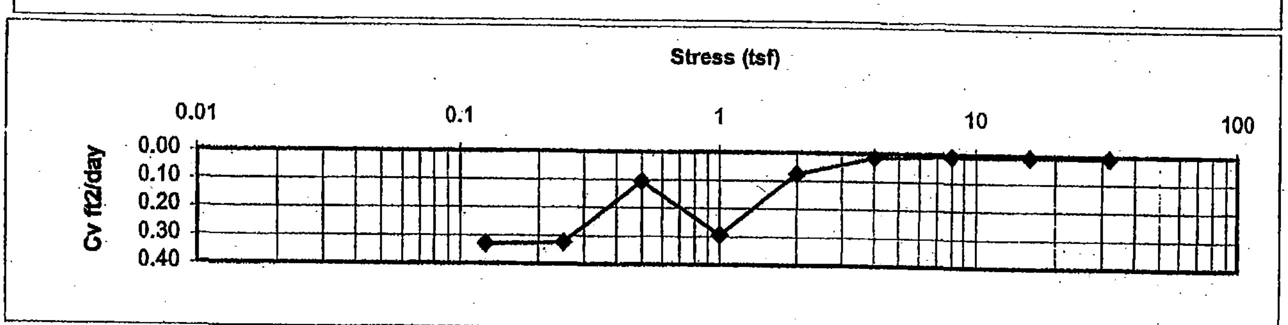
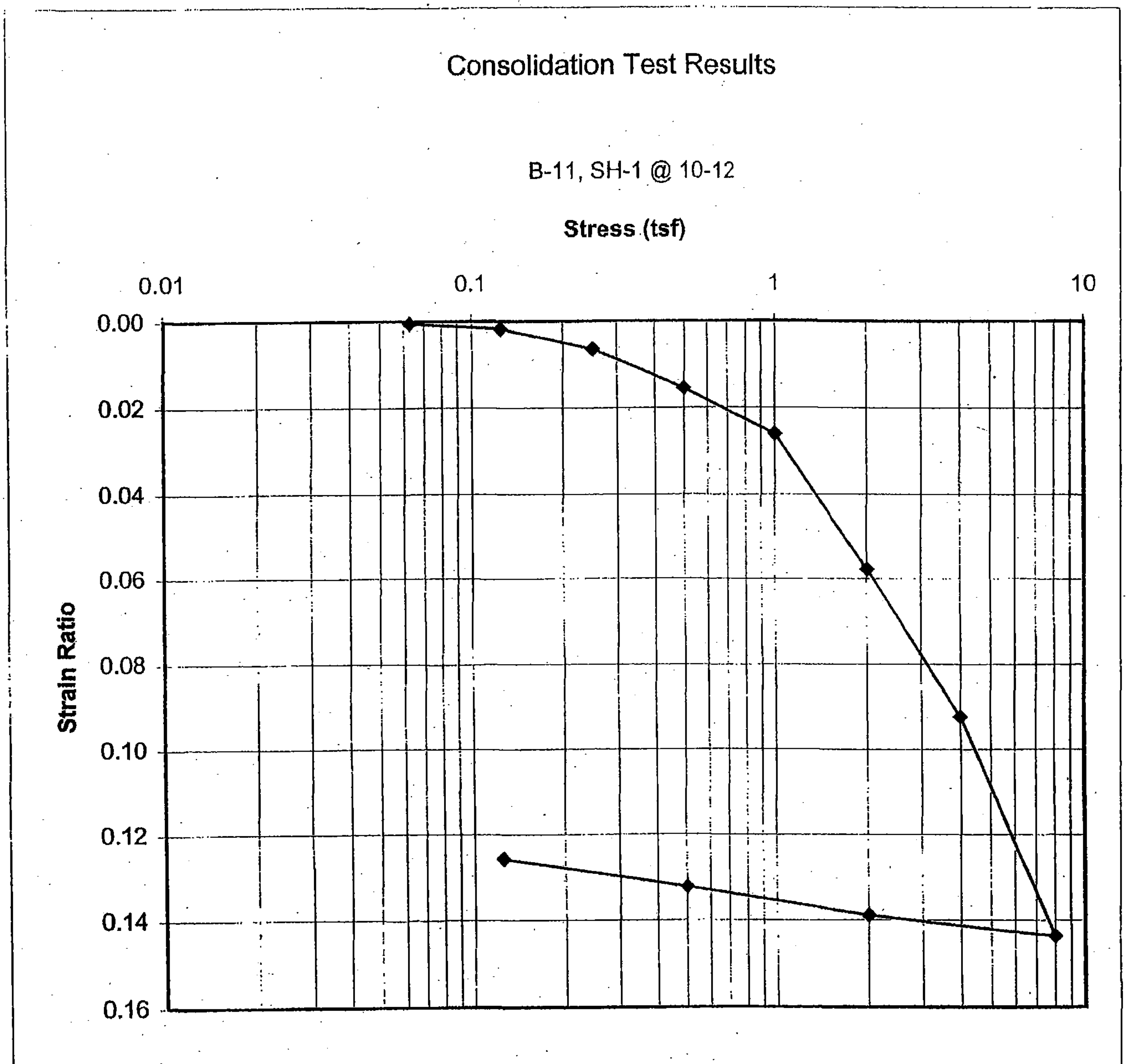




**Figure B-5**


Consolidation Test Results
B-11, SH-1 @ 10-12
Stress (tsf)
0.01
0.1
1
10
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
Strain Ratio

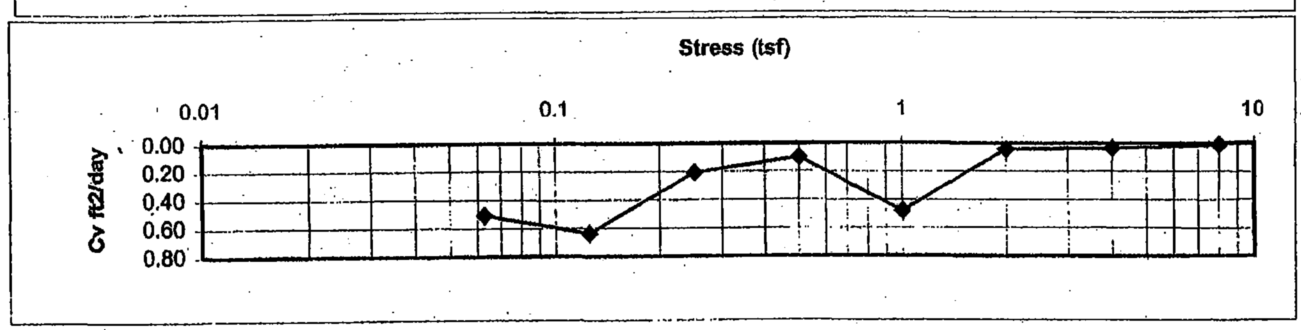
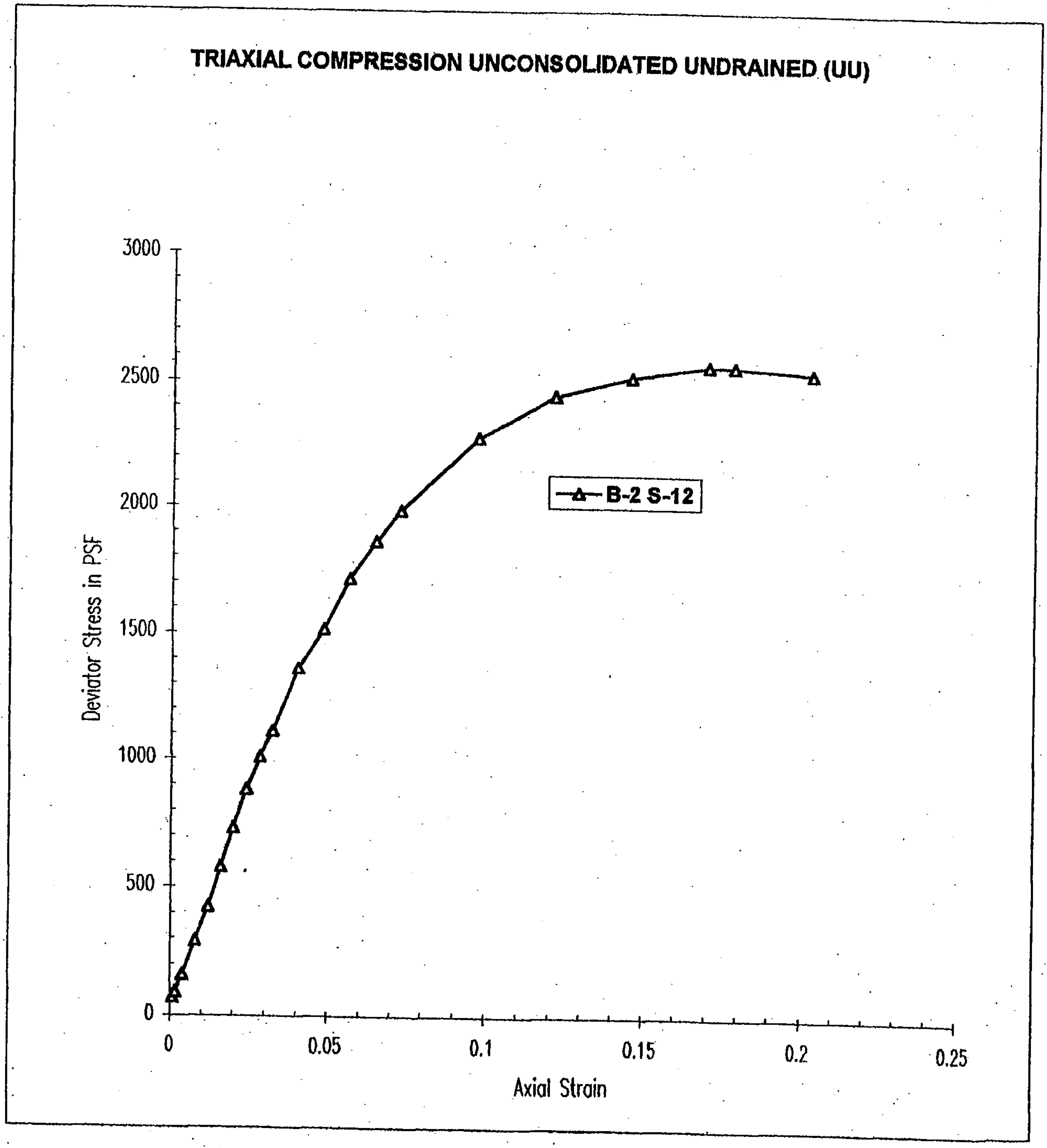

Stress (tsf)
0.01
0.1
1
10
0.00
0.20
0.40
0.60
0.80
Cv ft2/day

**Figure B-6**

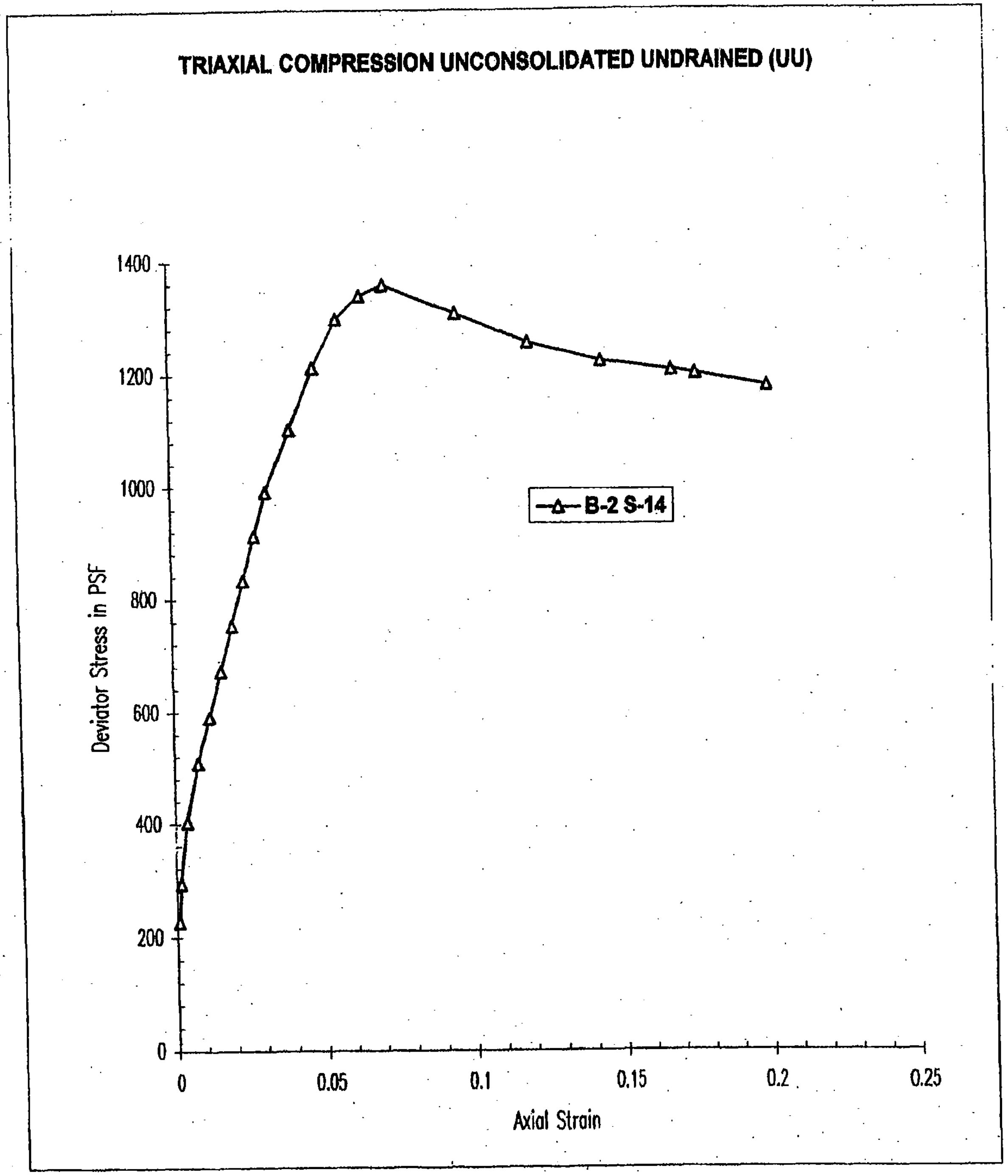
Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300




| Symbol | Boring Number | Sample Number | Depth ft. | Water Content in Percent | | | | Unit Weight | | Cell Pressure psi |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Natural | Liquid Limit | Plastic Limit | Plasticity Index | Wet pcf | Dry pcf | |
| Δ | B-2 | S-12 | 51.4-51.9 | 33 | 35 | 21 | 14 | 125 | 94 | 40 |

Figure B-7
Soil Technology, Inc.

Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300

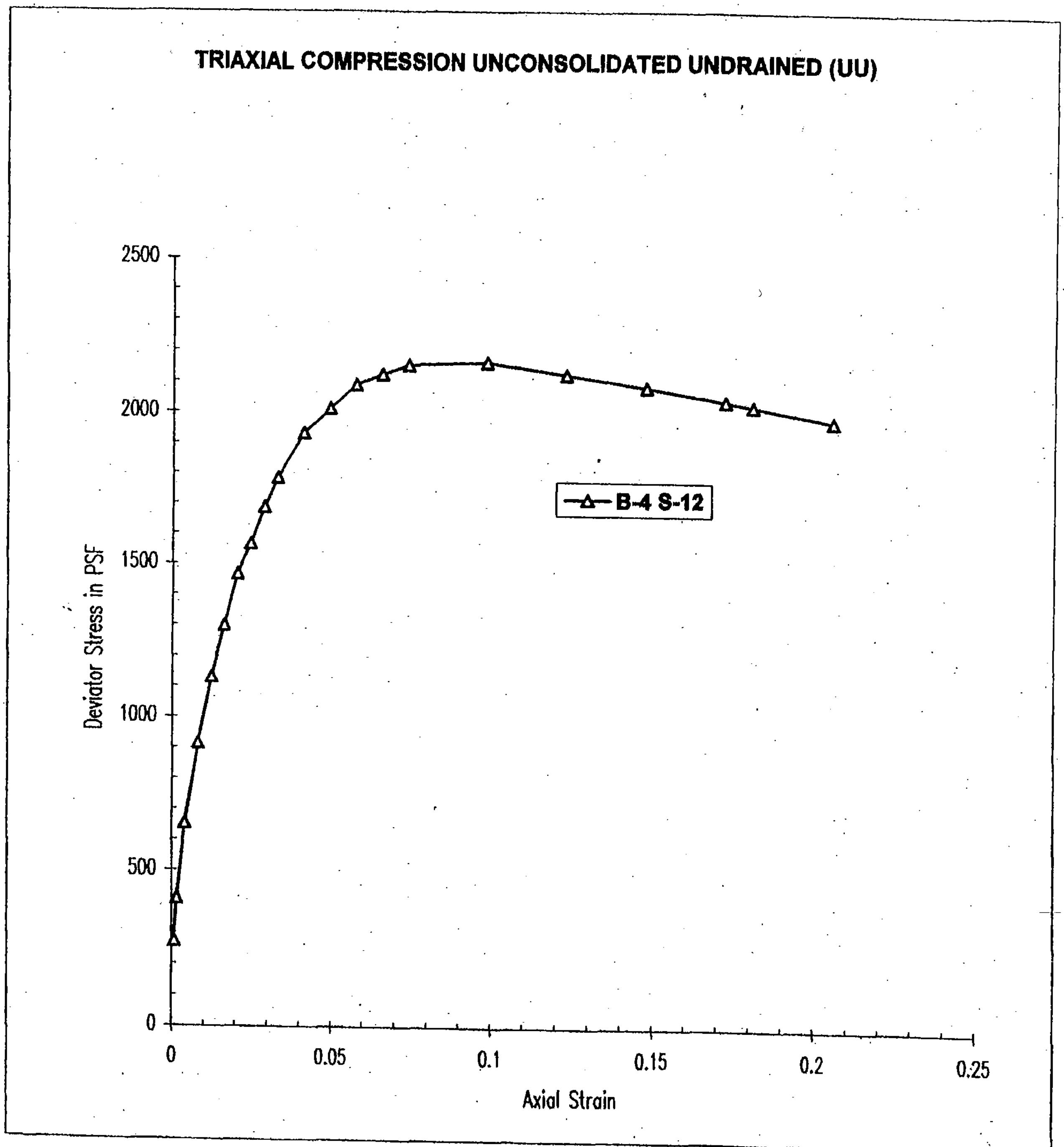



| Symbol | Boring Number | Sample Number | Depth ft | Water Content in Percent | | | | Unit Weight | | Cell Pressure psi |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Natural | Liquid Limit | Plastic Limit | Plasticity Index | Wet pcf | Dry pcf | |
| Δ | B-2 | S-14 | 60.0-61.3 | 42 | 49 | 23 | 26 | 115 | 81 | 50 |

**Figure B-8**

Soil Technology, Inc.

Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300

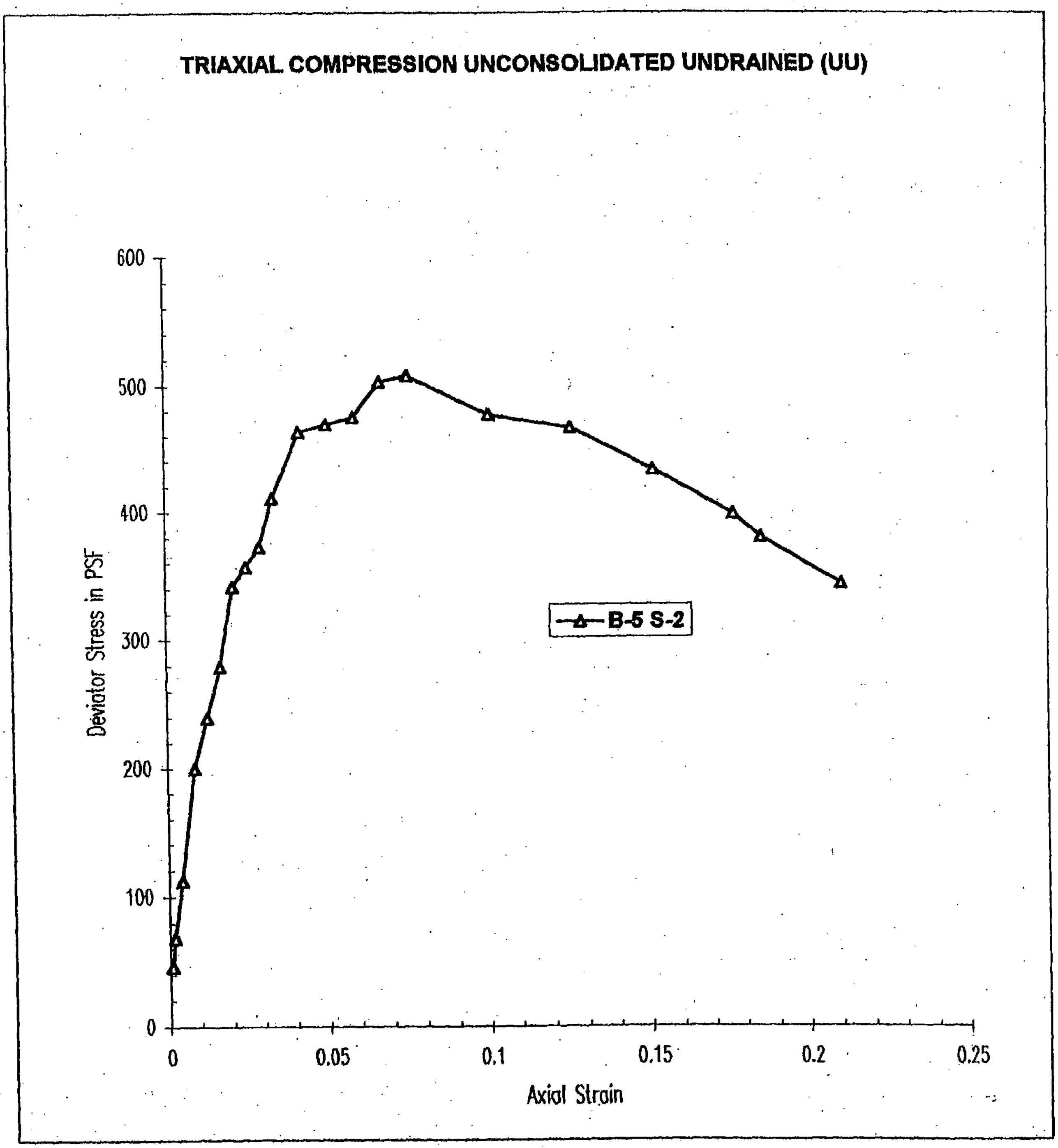



| Symbol | Boring Number | Sample Number | Depth ft | Water Content in Percent | | | | Unit Weight | | Cell Pressure psi |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Natural | Liquid Limit | Plastic Limit | Plasticity Index | Wet pcf | Dry pcf | |
| Δ | B-4 | S-12 | 58.5-61.0 | 44 | 60 | 27 | 33 | 112 | 78 | 50 |

Figure B-9

Soil Technology, Inc.
J-997

Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300



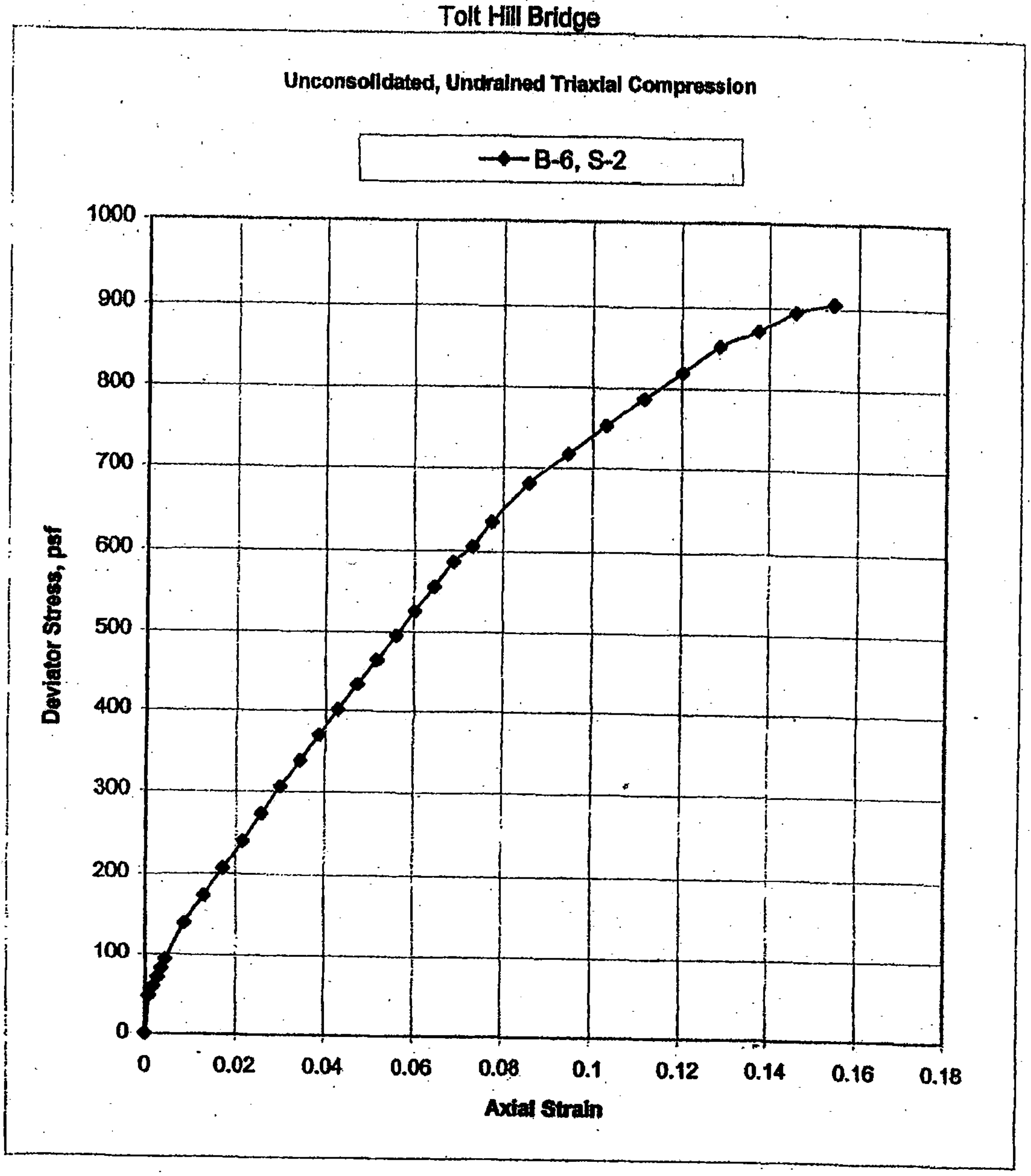

| Symbol | Boring Number | Sample Number | Depth ft | Water Content in Percent | | | | Unit Weight | | Cell Pressure psi |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Natural | Liquid Limit | Plastic Limit | Plasticity Index | Wet pcf | Dry pcf | |
| Δ | B-5 | S-2 | 7.5-9.5 | 42 | 49 | 25 | 24 | 113 | 81 | 7 |

**Figure B-10**

Soil Technology, Inc.

Rosa Environmental and Geotechnical Laboratory, LLC

Lorilla Engineering
Tolt Hill Bridge



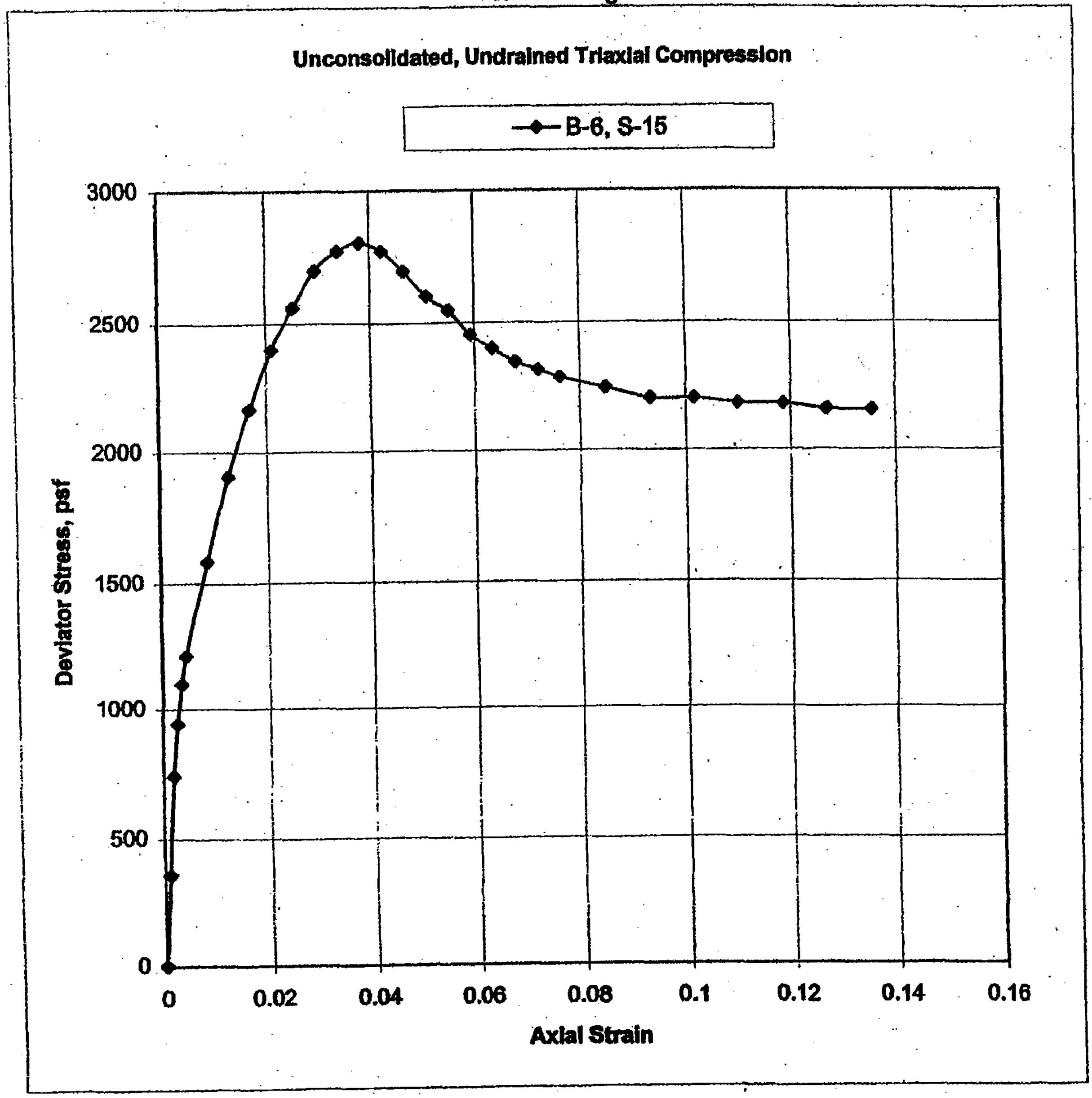

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-6, S-2 | 8.5 | 7.0 | 117.4 | 30.2 | 90.1 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a bulging failure.

Figure B-11

Rosa Environmental and Geotechnical Laboratory, LLC

Lorilla Engineering
Tolt Hill Bridge



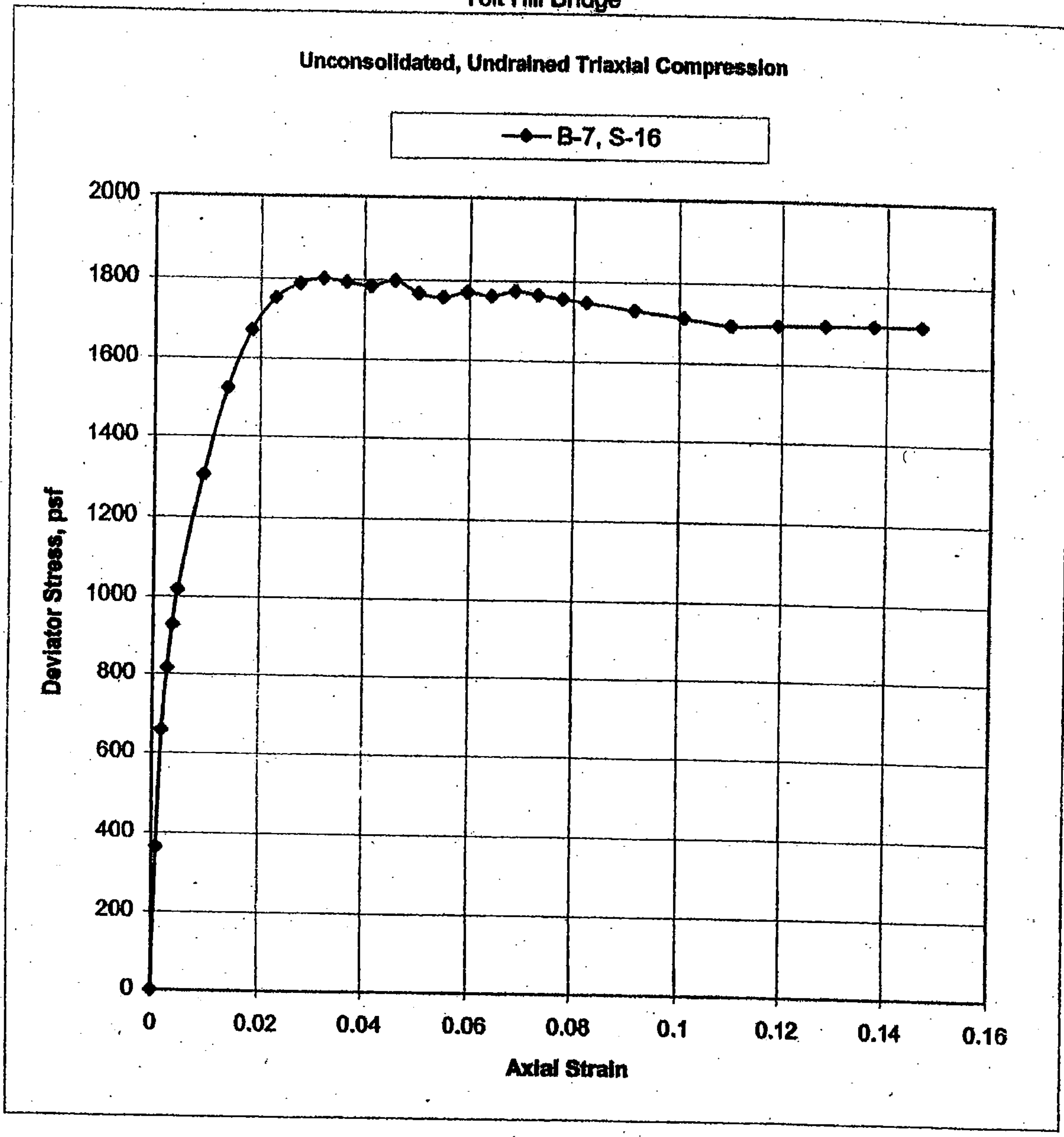

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-6, S-15 | 73.5 | 35.0 | 112.4 | 45.1 | 77.5 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a shear failure, along a very thin sand lense, approximately 45 degrees from horizontal.

Figure B-12

Rosa Environmental and Geotechnical Laboratory, LLC

Lorilla Engineering
Tolt Hill Bridge



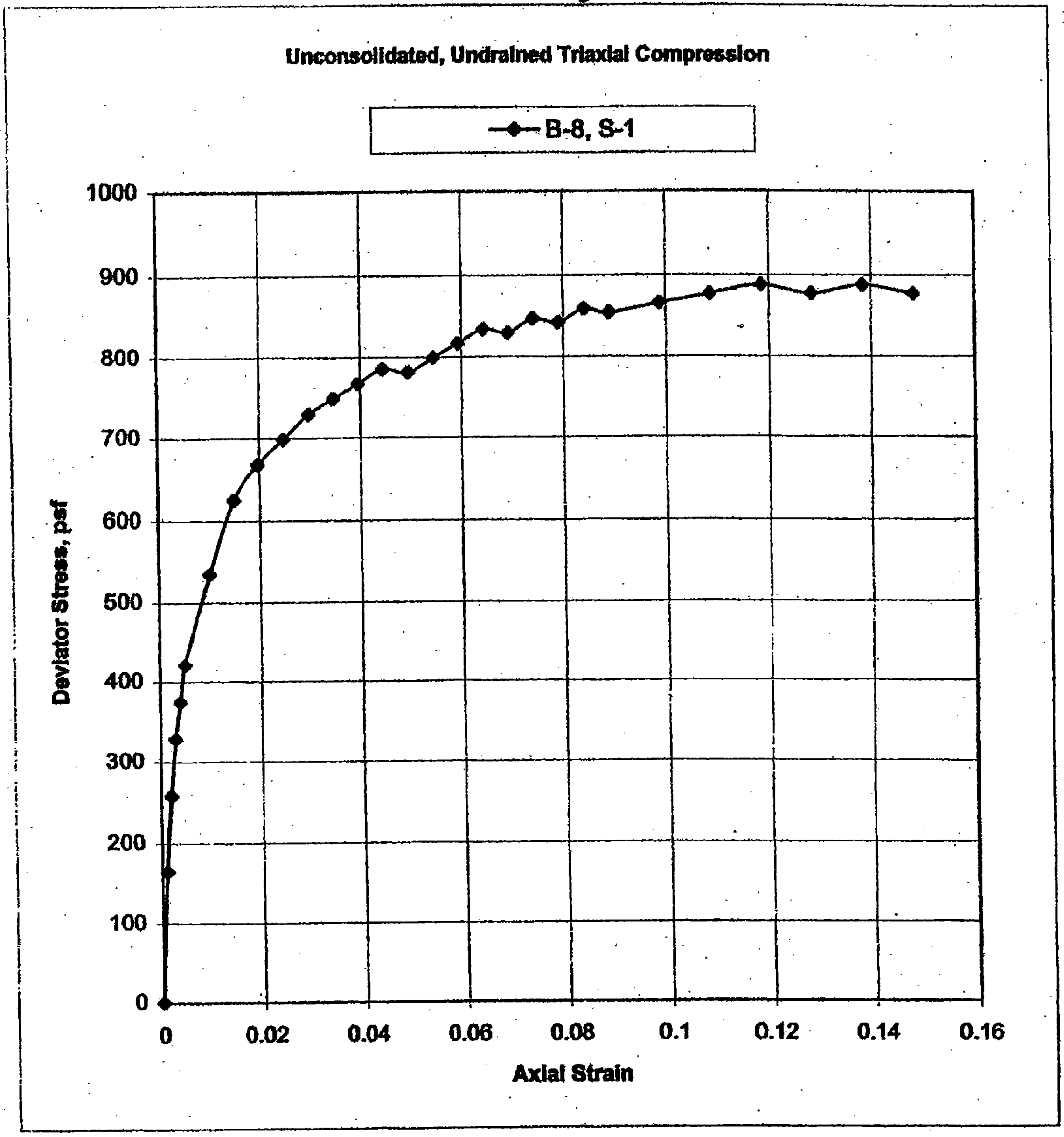

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-7, S-16 | 73.5 | 36.0 | 110.3 | 51.6 | 72.8 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a shear failure with shear planes going in opposite directions forming an "X" in the middle of the sample.

Figure B-13

Rosa Environmental and Geotechnical Laboratory, LLC

Lorilla Engineering
Tolt Hill Bridge



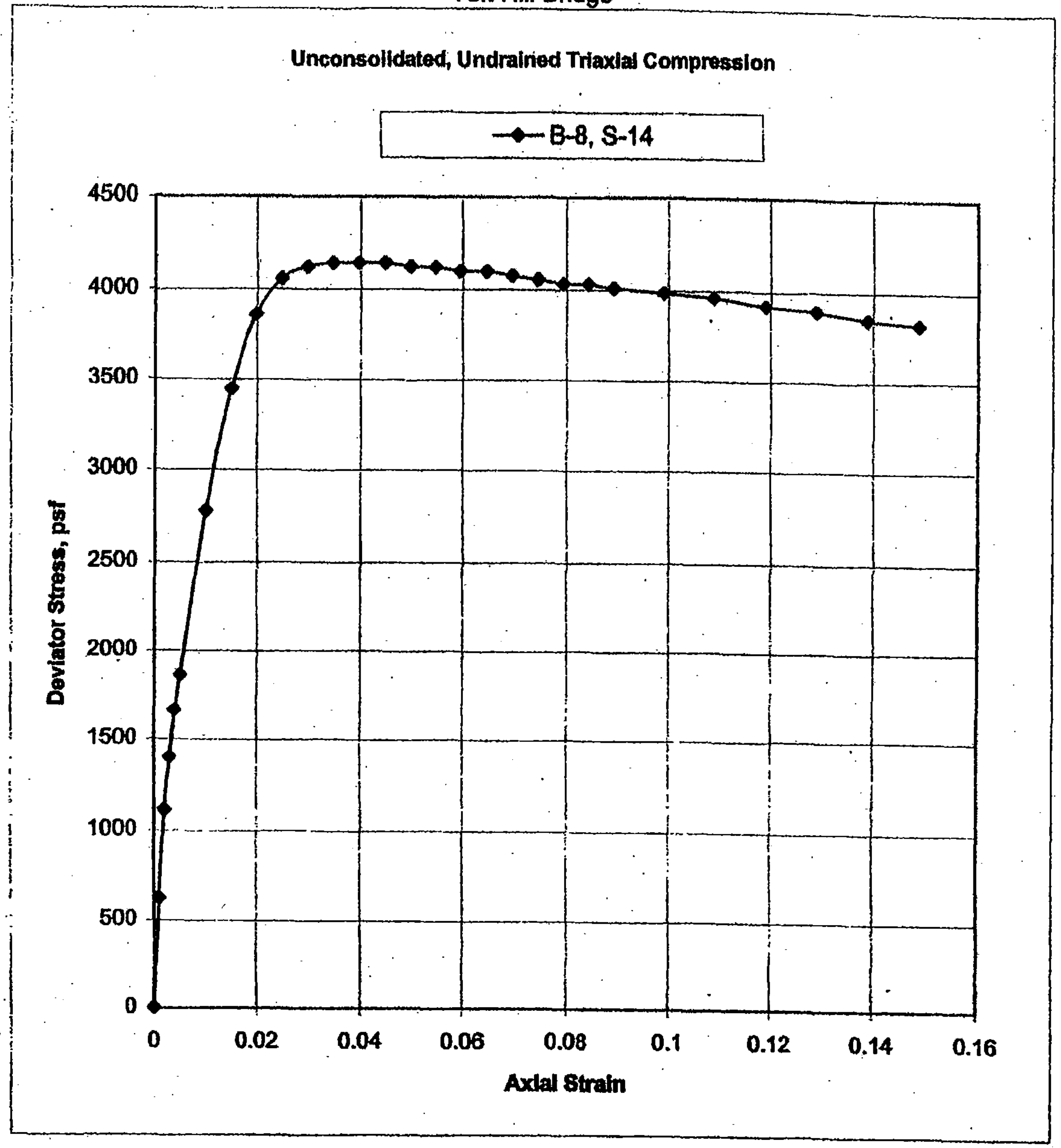

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-8, S-1 | 8.5 | 7.0 | 113.6 | 41.0 | 80.5 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a bulging failure.

Figure B-14

Rosa Environmental and Geotechnical Laboratory, LLC

Lorilla Engineering
Tolt Hill Bridge



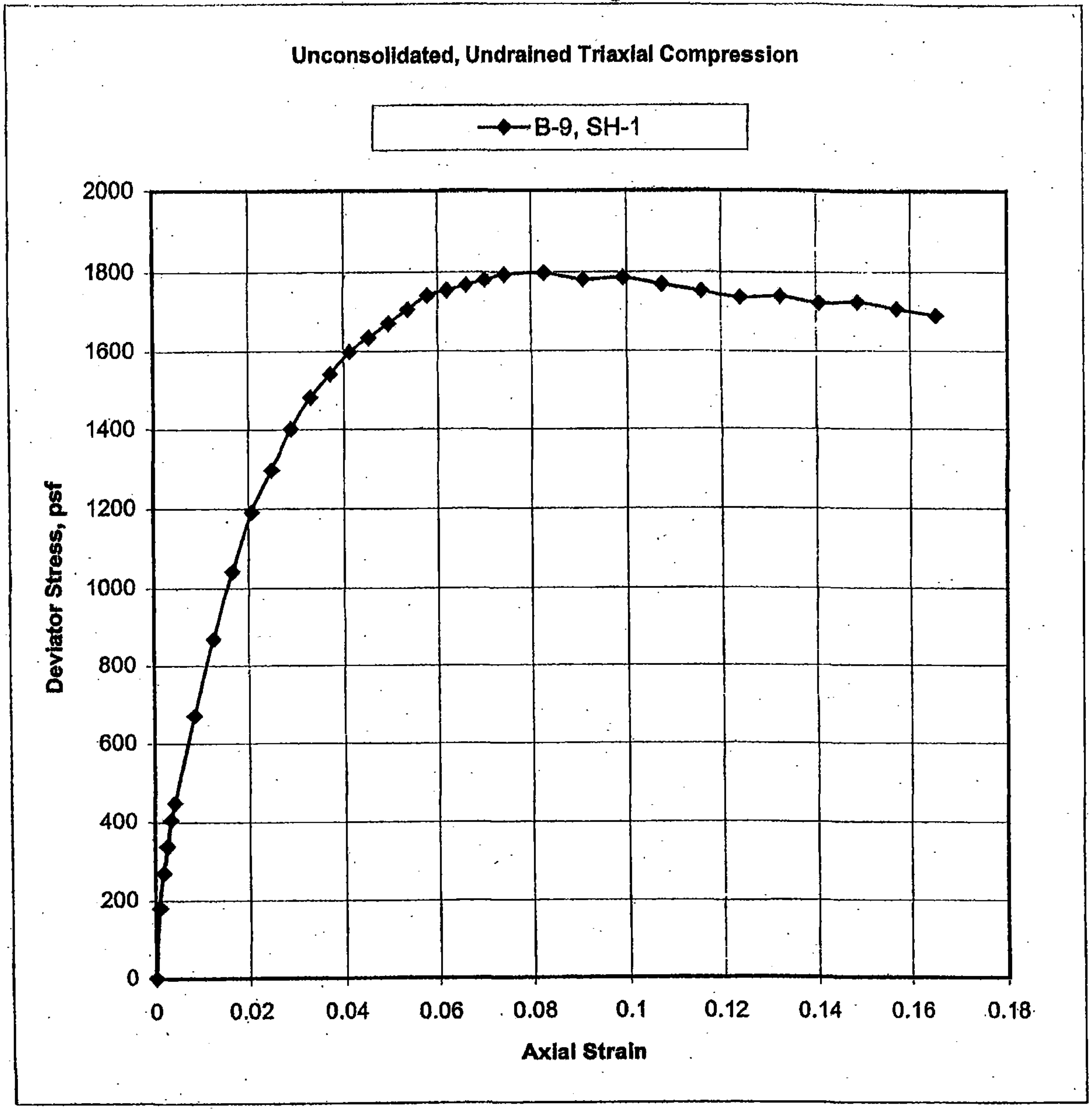

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-8, S-14 | 68.5 | 34.0 | 113.4 | 34.2 | 84.6 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a shear failure at the top of the sample and several silt/fine sand lenses just below the shear plane.

Figure B-15

ANALYTICAL RESOURCES INCORPORATED

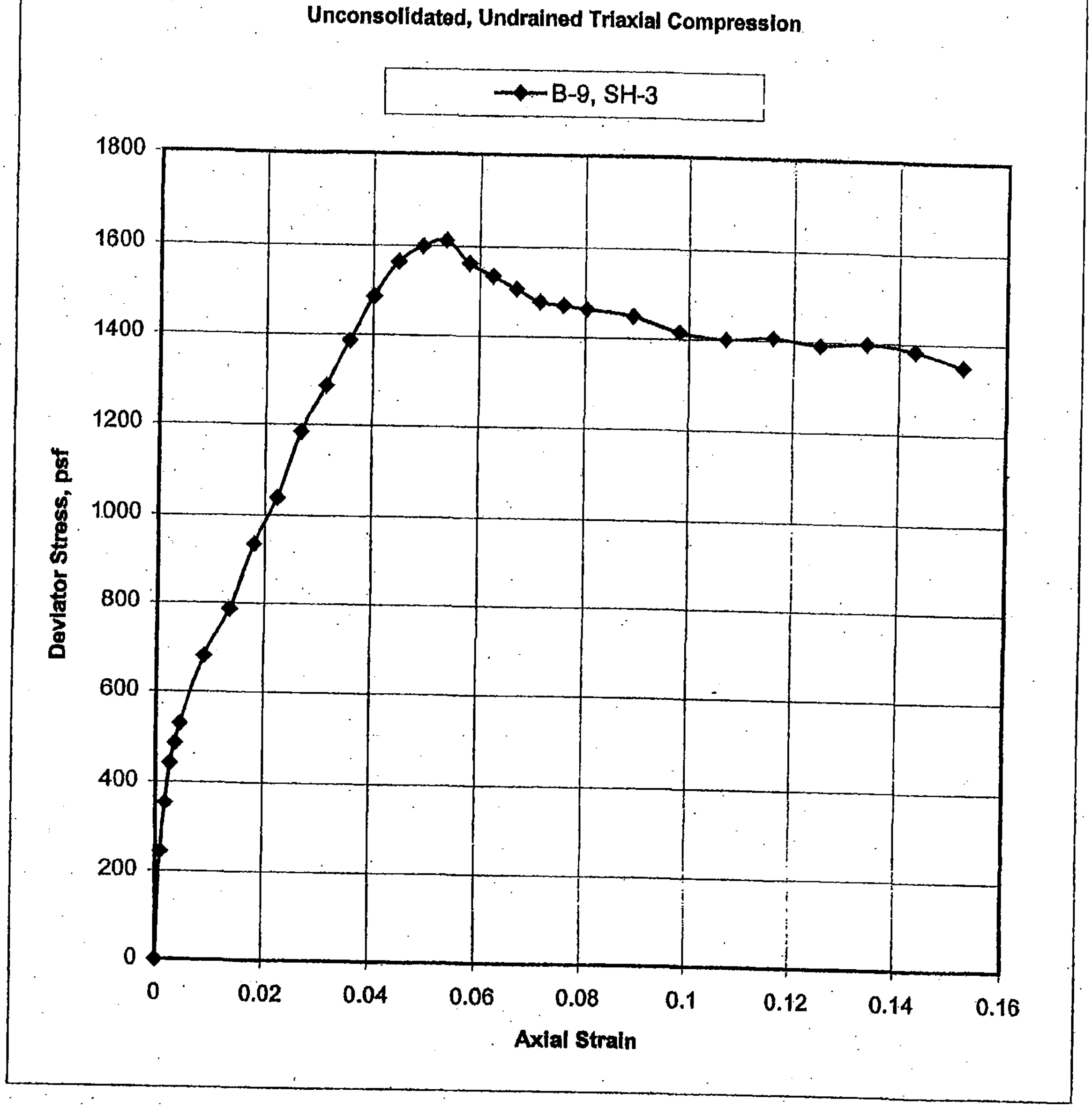
Lorilla Engineering, Inc.
Tolt Hill Bridge




| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-9, SH-1 | 17.5-19 | 10.0 | 104.7 | 50.5 | 69.6 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a shear failure.

Figure B-16



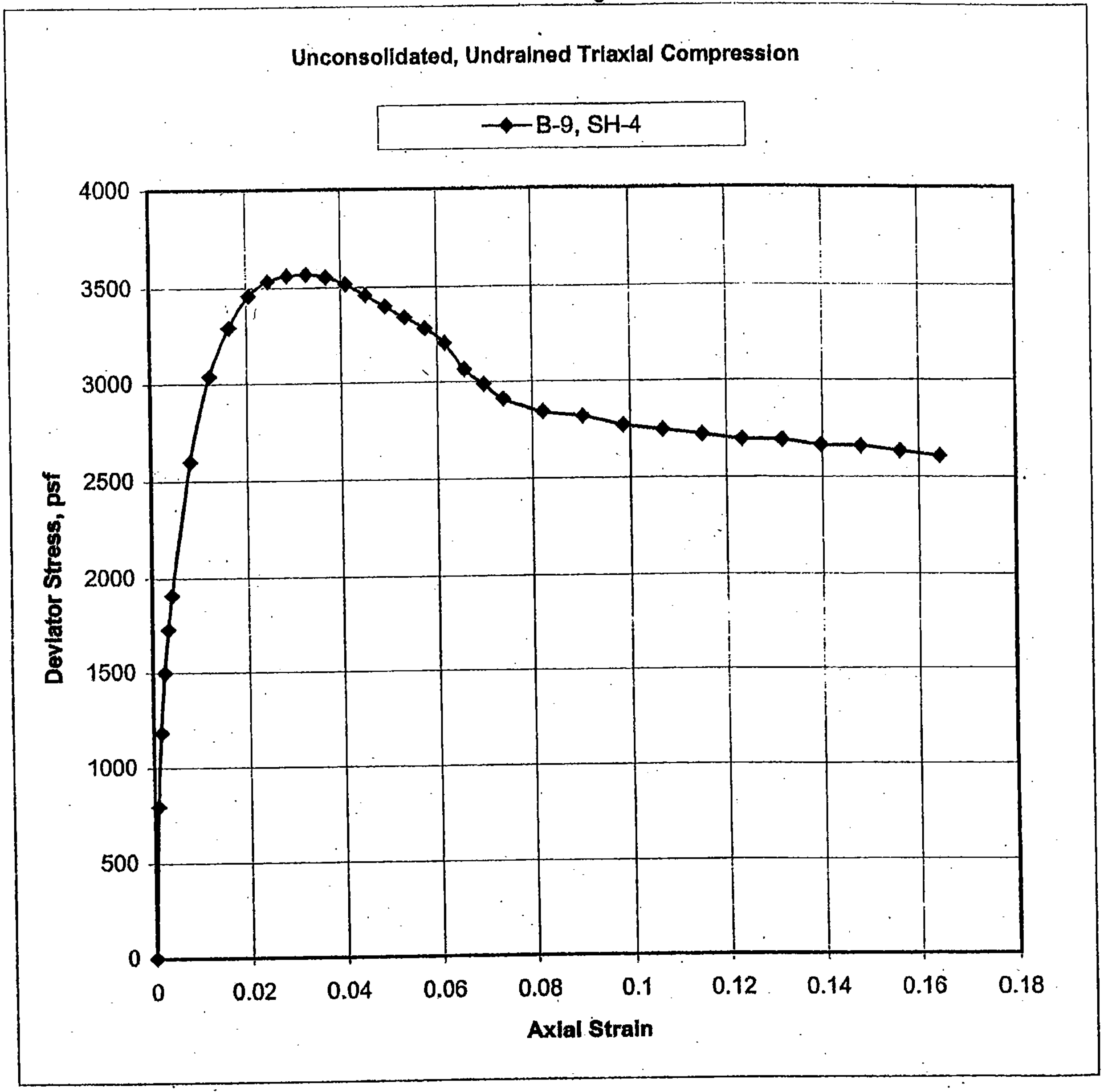

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-9, SH-3 | 82.5-85 | 38.0 | 109.4 | 43.3 | 76.3 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a shear failure, along two shear planes perpendicular to each other.

Figure B-17

Lorilla Engineering, Inc.
Tolt Hill Bridge

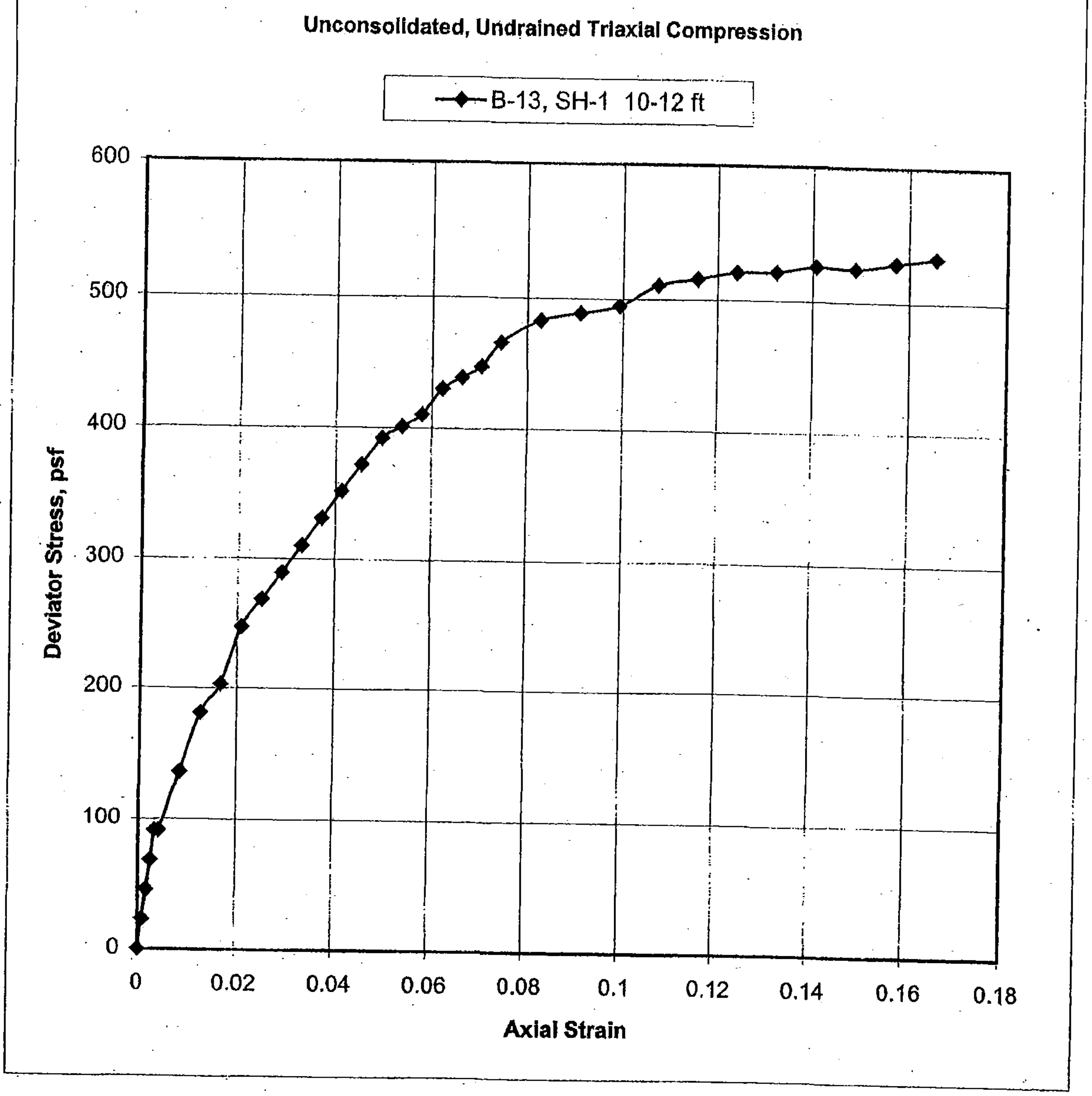
ANALYTICAL RESOURCES INCORPORATED




| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-9, SH-4 | 87.5-90 | 40.0 | 108.6 | 51.3 | 71.8 |

Notes to the testing:
1. The testing was performed according to ASTM D-2850.
2. The sample had a shear failure.

Figure B-18



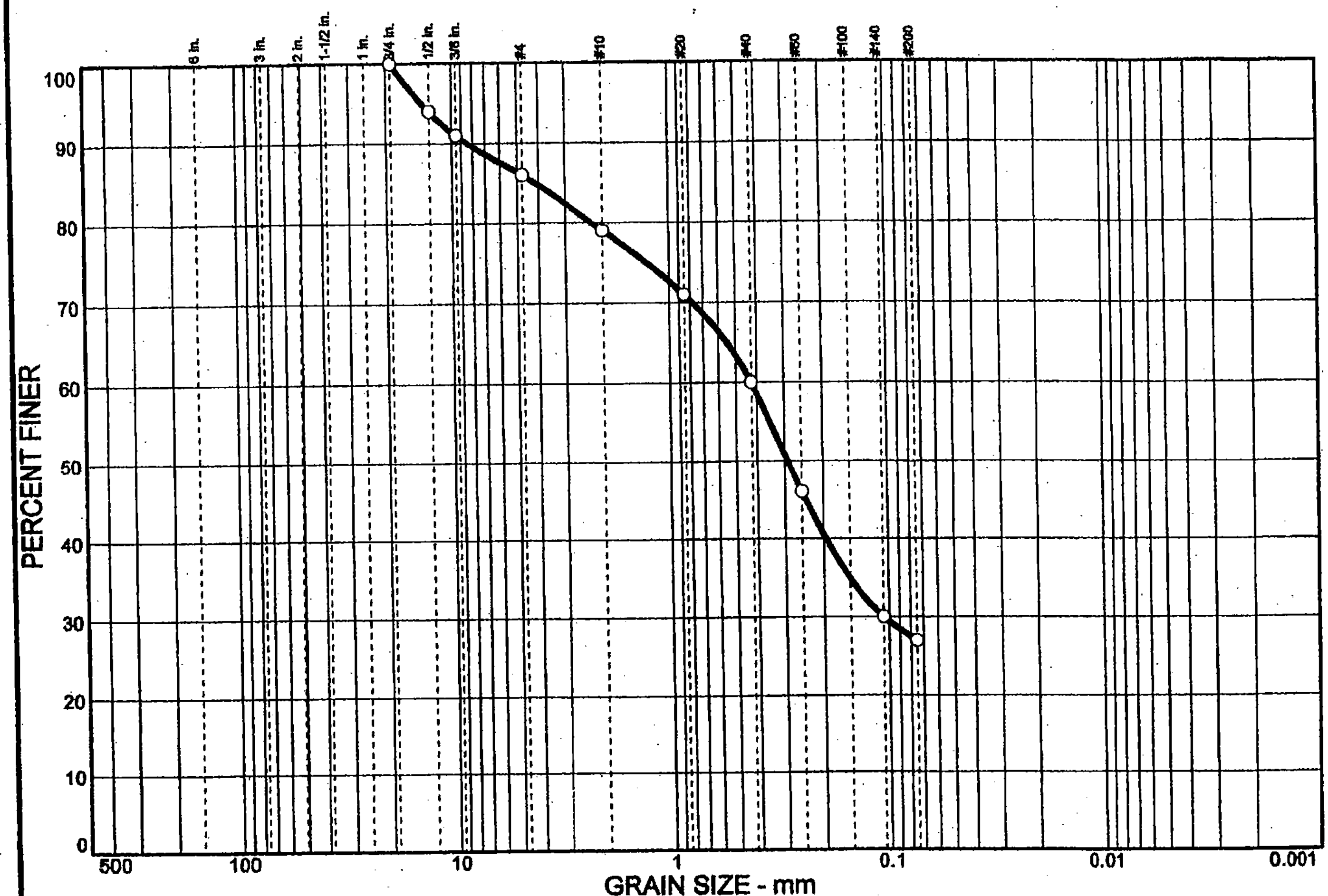

| Sample ID | Depth (ft) | Confining Pressure (psi) | Wet Density (pcf) | Moisture Content (%) | Dry Density (pcf) |
|---|---|---|---|---|---|
| B-13, SH-1 | 10-12 ft | 8.0 | 116.0 | 34.9 | 86.0 |

Notes to the testing:

1. The testing was performed according to ASTM D-2850.
2. The sample had a bulging failure.

Figure B-19

# PARTICLE SIZE DISTRIBUTION TEST REPORT




| % COBBLES | % GRAVEL | % SAND | % SILT | % CLAY |
|---|---|---|---|---|
| 0 | 14 | 59 | 27 | |

| SIEVE SIZE | PERCENT FINER | SPEC.* PERCENT | PASS? (X=NO) |
|---|---|---|---|
| .75 | 100 | | |
| .5 | 94 | | |
| .375 | 91 | | |
| #4 | 86 | | |
| #10 | 79 | | |
| #20 | 71 | | |
| #40 | 60 | | |
| #60 | 46 | | |
| #140 | 30 | | |
| #200 | 27 | | |

* (no specification provided)

**Soil Description**

Silty sand

**Atterberg Limits**

PL= LL= PI=

**Coefficients**

$D_{85}$= 4.14 $D_{60}$= 0.425 $D_{50}$= 0.290
$D_{30}$= 0.106 $D_{15}$= $D_{10}$=
$C_u$= $C_c$=

**Classification**

USCS= SM AASHTO=

**Remarks**

Classification based on grain size only. No Atterberg Limits performed.

**Sample No.:** B-1 S-7 **Source of Sample:** **Date:**
**Location:** Carnation, WA **Elev./Depth:**

**SOIL TECHNOLOGY INC.**

**Client:** Lorilla Engineering, Inc.
**Project:** Tolt Hill Bridge
KC 300
**Project No:** J-997

**Figure B-20**

ANALYTICAL RESOURCES INCORPORATED

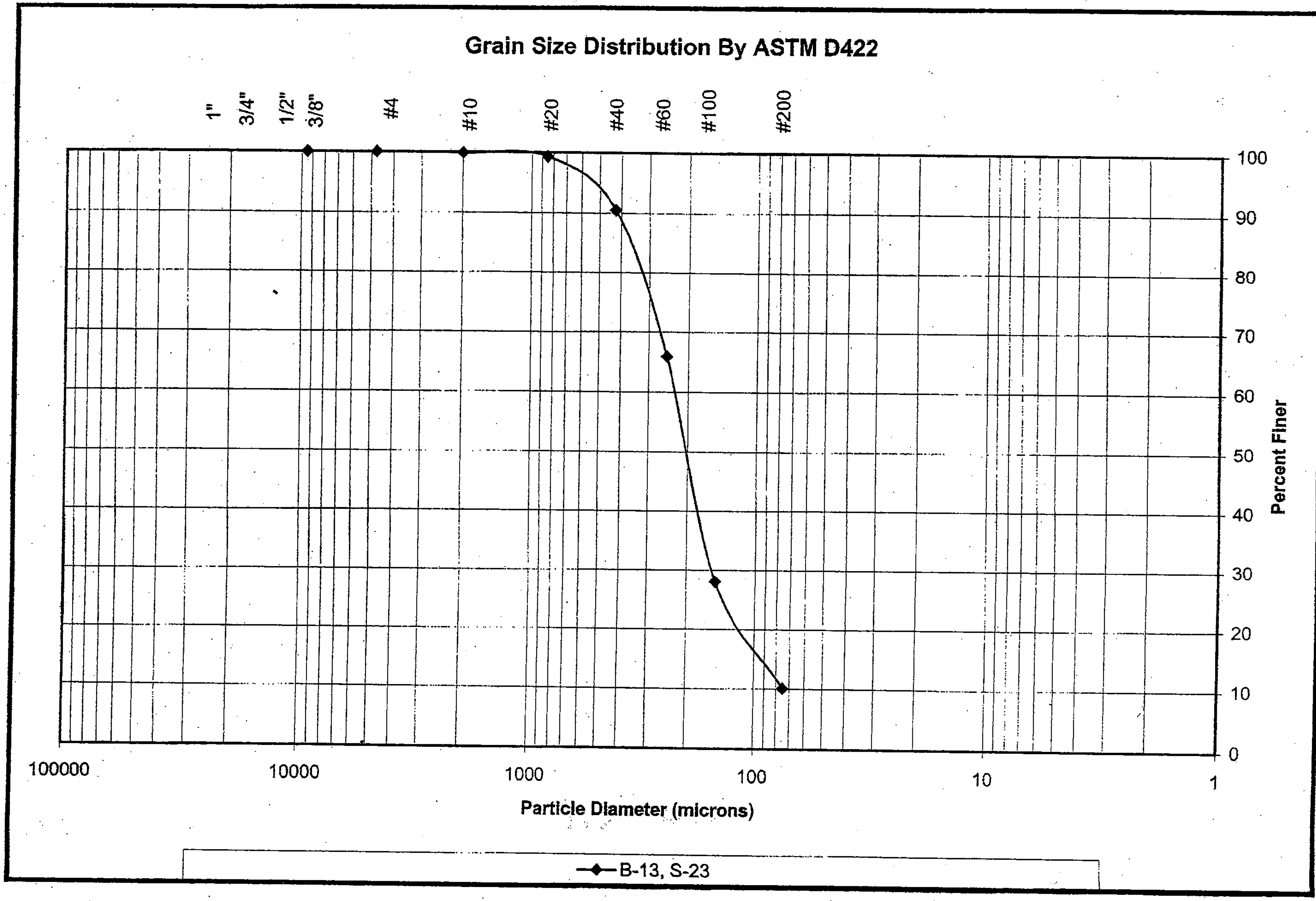
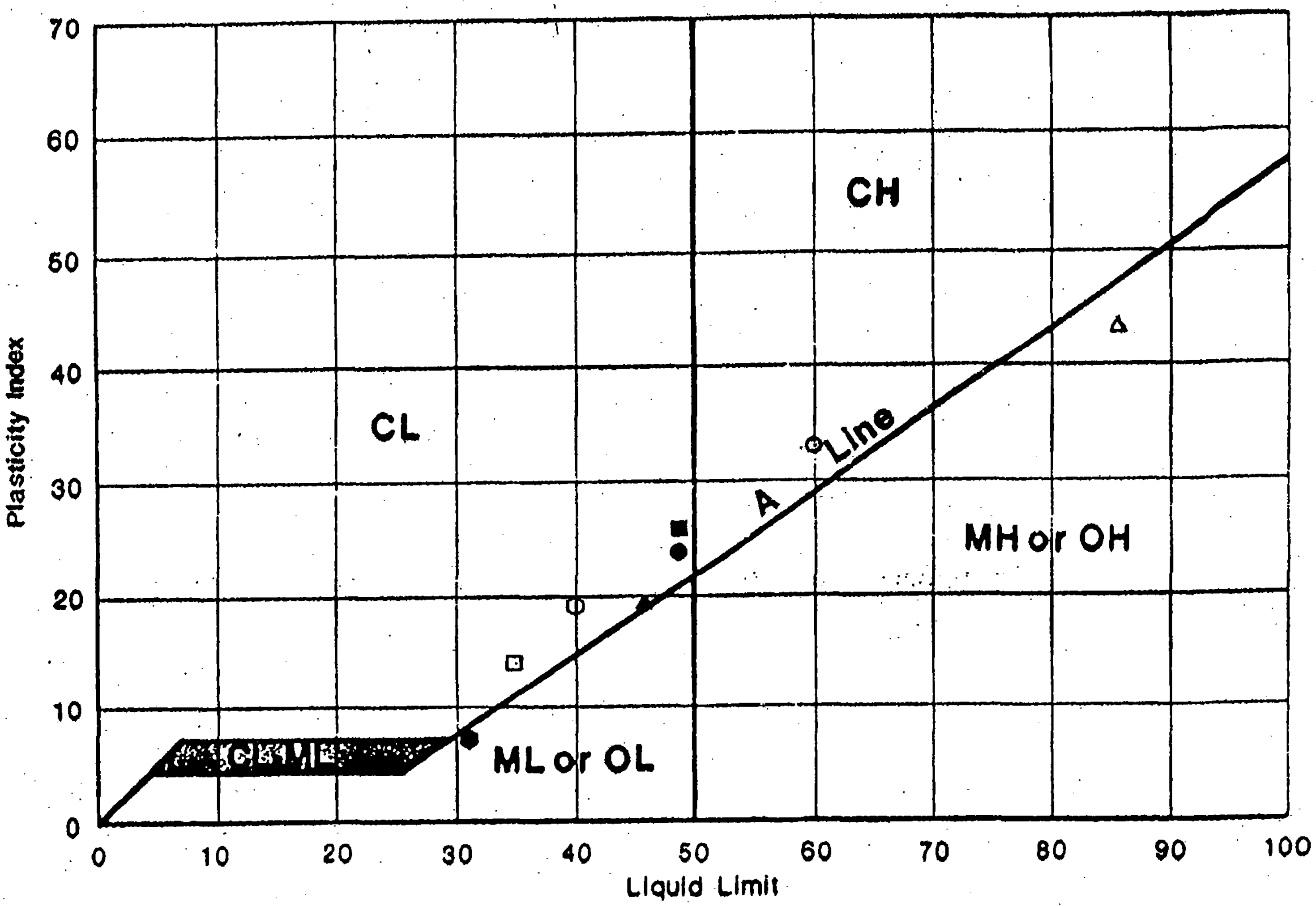



Figure B-21

# Plasticity Chart

Lorilla Engineering, Inc.
Tolt Hill Bridge
Project No. KC 300




| Symbol | Boring Number | Sample Number | Depth in feet | Water Content in Percent Nat. | L.L. | P.L. | P.I. | Classification |
|---|---|---|---|---|---|---|---|---|
| △ | B-1 | S-1 | 5.0-6.5 | 86 | 86 | 43 | 43 | OH |
| ▲ | B-1 | S-13 | 65.0-66.5 | 44 | 46 | 27 | 19 | CL |
| □ | B-2 | S-12 | 50.0-52.0 | 32 | 35 | 21 | 14 | CL |
| ■ | B-2 | S-14 | 60.0-61.3 | 42 | 49 | 23 | 26 | CL |
| ○ | B-4 | S-12 | 58.5-61.0 | 44 | 60 | 27 | 33 | CH |
| ● | B-5 | S-2 | 7.5-9.5 | 42 | 49 | 25 | 24 | CL |
| ○ | B-5 | S-17 | 77.5-79.0 | 44 | 40 | 21 | 19 | CL |
| ⬢ | B-5 | S-24 | 122.5-124.0 | 35 | 31 | 24 | 7 | ML |

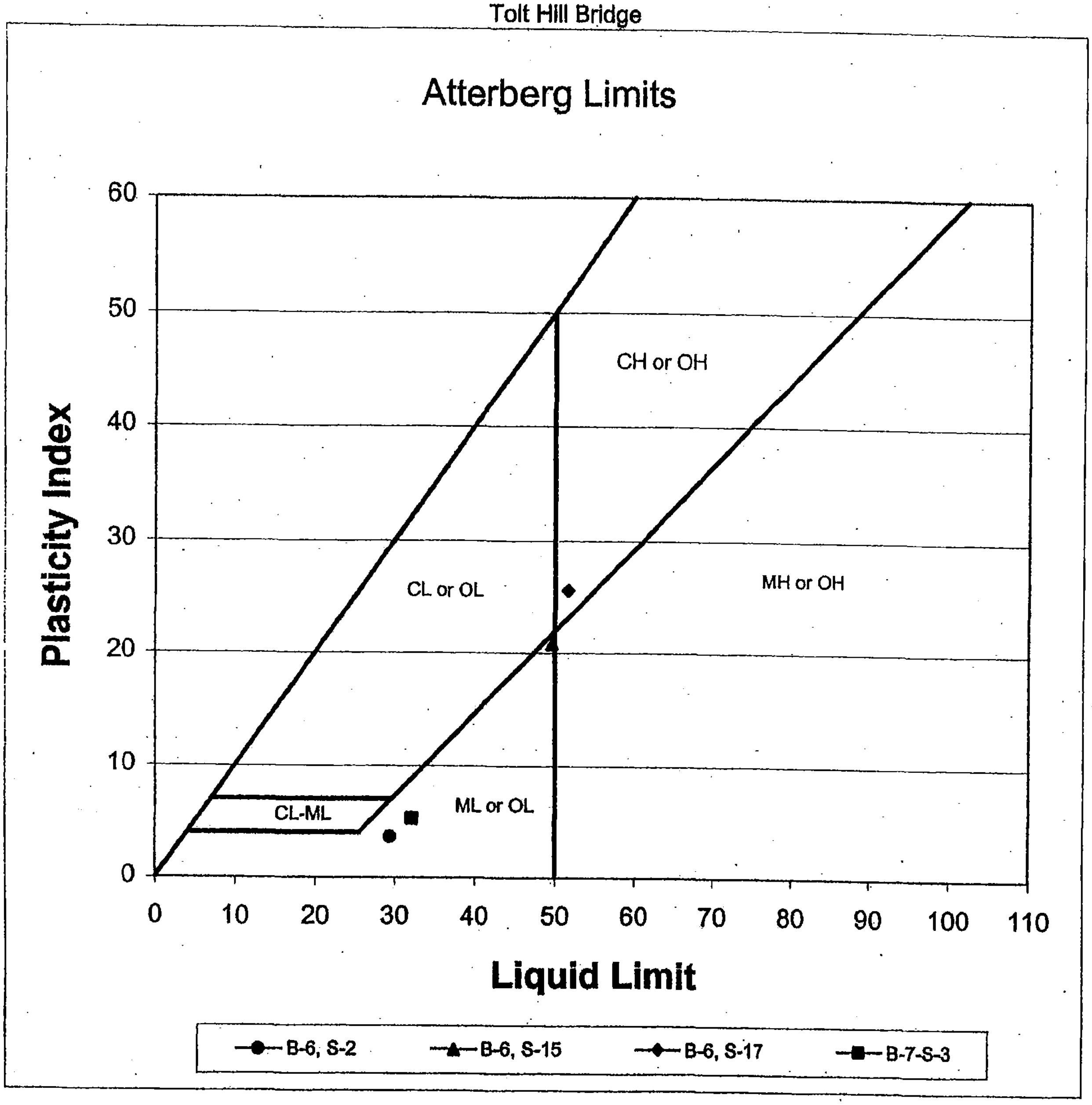
Nat. Natural
L.L. Liquid Limit
P.L. Plastic Limit
P.I. Plasticity Index

Figure B-22

Soil Technology, Inc.
J-997

ROSA ENVIRONMENTAL AND GEOTECHNICAL LABORATORY, LLC

Lorilla Engineers
Tolt Hill Bridge

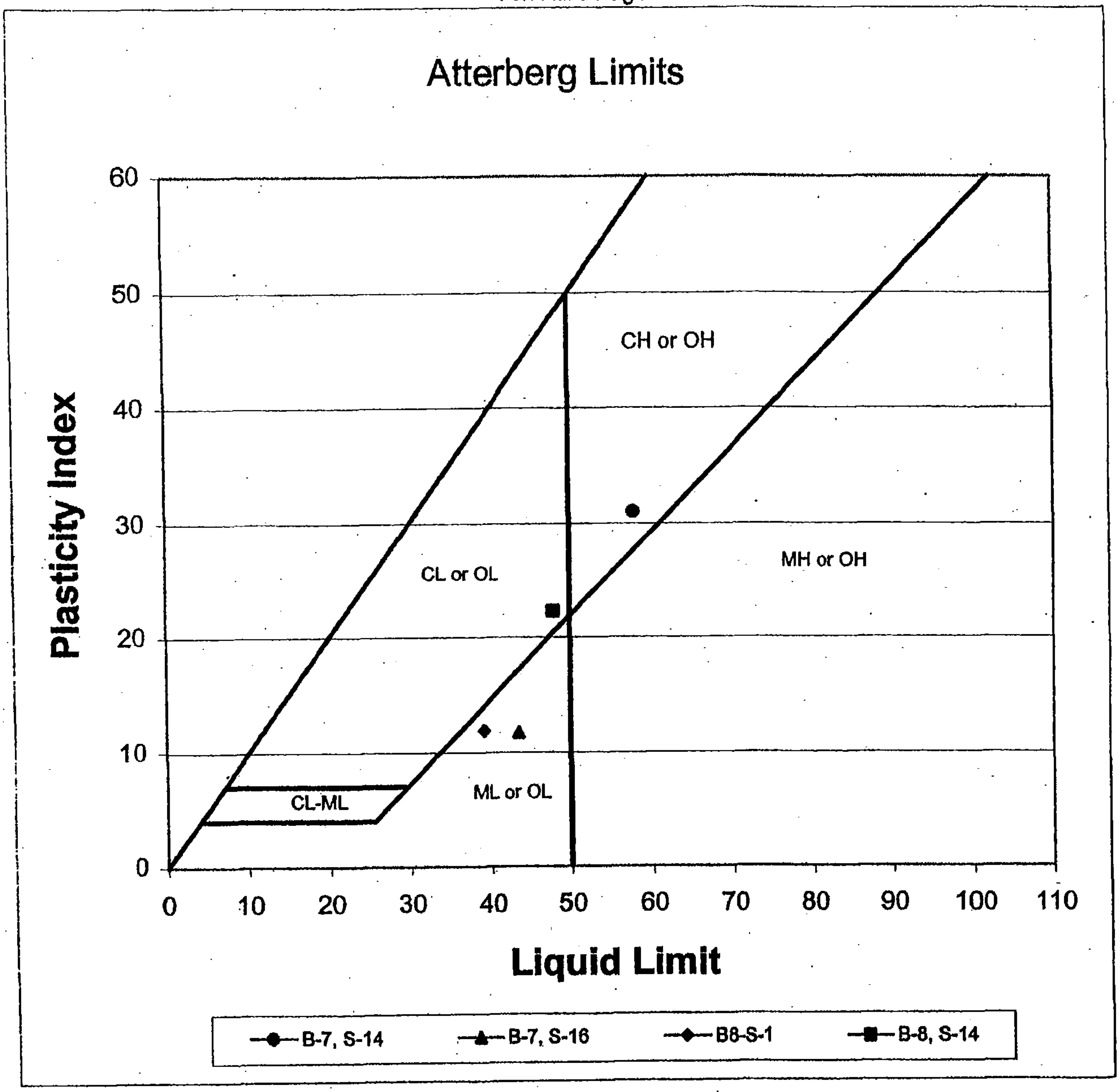



| Sample Number | Depth | Plasticity Index | Liquid Limit | Plastic Limit | Classification |
|---|---|---|---|---|---|
| B-6, S-2 | 8.5 | 3.6 | 29.5 | 25.8 | ML |
| B-6, S-15 | 73.5 | 20.8 | 49.7 | 28.9 | ML |
| B-6, S-17 | 83 | 25.5 | 51.7 | 26.2 | CH |
| B-7-S-3 | 11 | 5.3 | 32.1 | 26.9 | ML |

**Figure B-23**

ROSA ENVIRONMENTAL AND GEOTECHNICAL LABORATORY, LLC

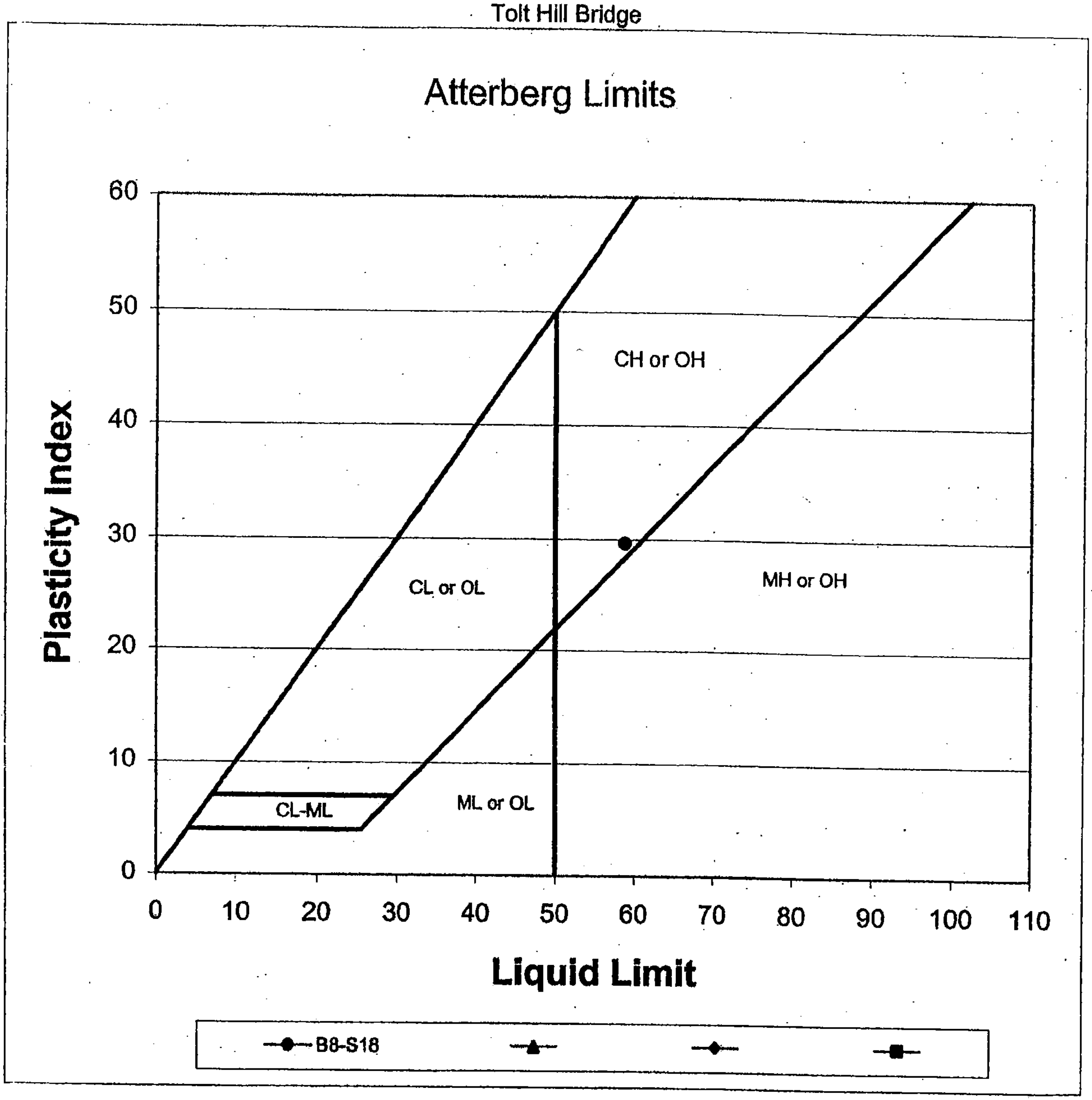
Lorilla Engineers
Tolt Hill Bridge




| Sample Number | Depth | Plasticity Index | Liquid Limit | Plastic Limit | Classification |
|---|---|---|---|---|---|
| B-7, S-14 | 63 | 31.0 | 57.9 | 26.9 | CH |
| B-7, S-16 | 74.5 | 11.7 | 43.5 | 31.8 | ML |
| B8-S-1 | 8.5 | 11.8 | 39.2 | 27.4 | ML |
| B-8, S-14 | 68.5 | 22.2 | 47.9 | 25.7 | CL |

Figure B-24

Lorilla Engineers
Tolt Hill Bridge

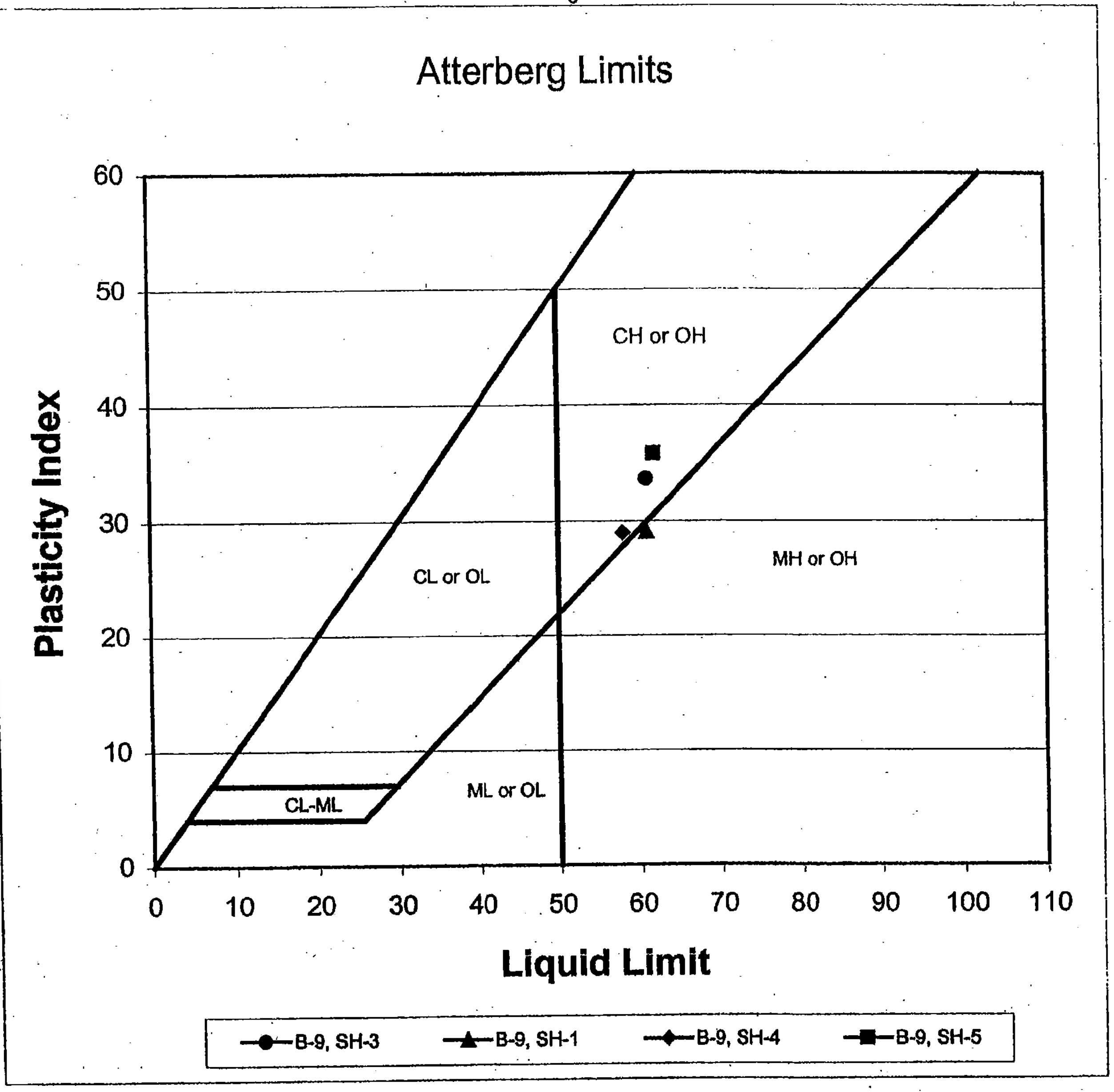



| Sample Number | Depth | Plasticity Index | Liquid Limit | Plastic Limit | Classification |
|---|---|---|---|---|---|
| B8-S18 | 87 | 29.7 | 58.8 | 29.2 | CH |

Figure B-25

ANALYTICAL RESOURCES INCORPORATED

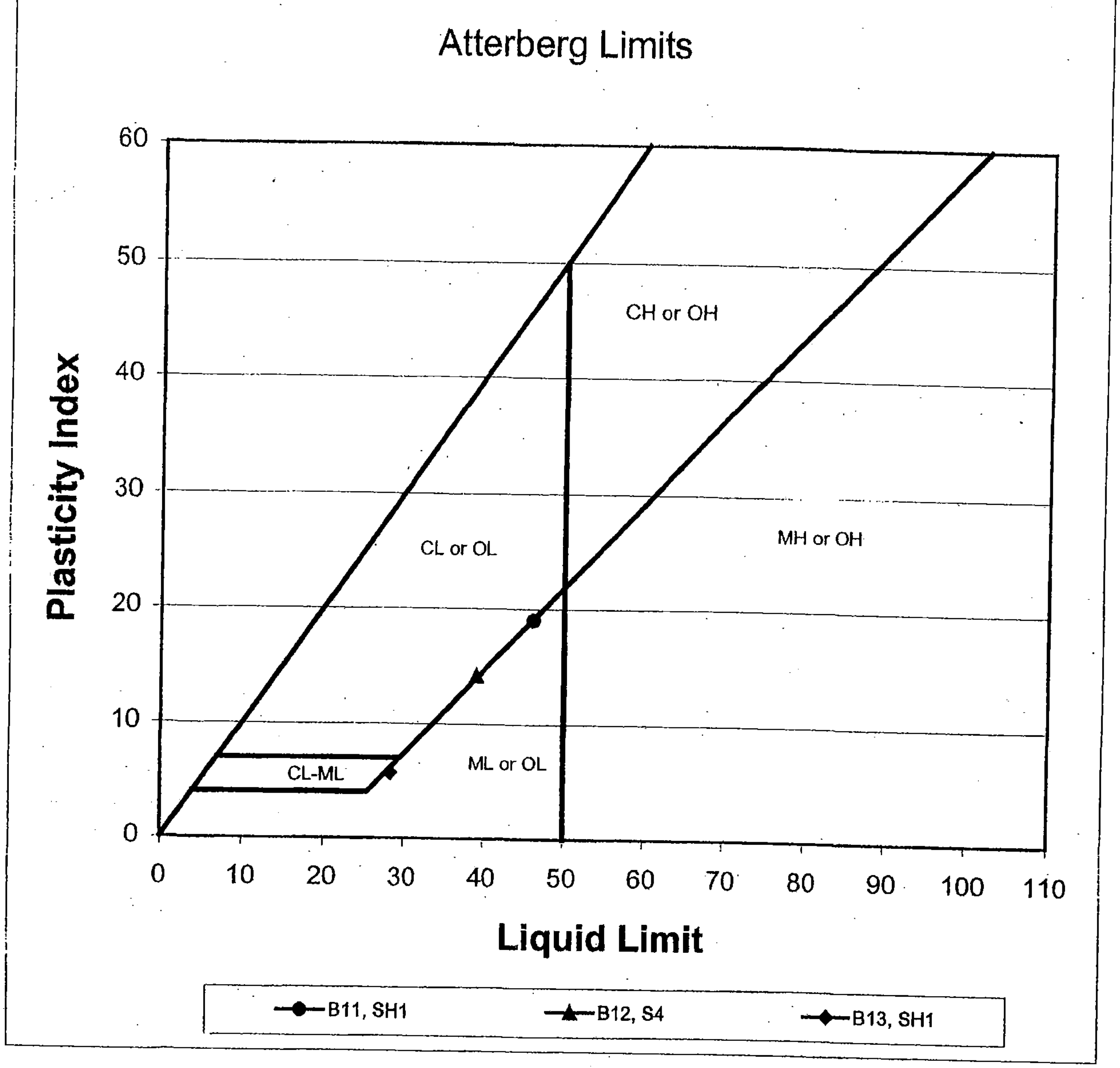
Lorilla Engineering
Tolt Hill Bridge KC-300




| Sample Number | Depth | Plasticity Index | Liquid Limit | Plastic Limit | Classification |
|---|---|---|---|---|---|
| B-9, SH-3 | 82.5-85 | 33.7 | 60.9 | 27.3 | CH |
| B-9, SH-1 | 17.5-19.5 | 29.1 | 60.9 | 31.9 | MH |
| B-9, SH-4 | 87.5-90 | 29.0 | 58.0 | 29.0 | CH |
| B-9, SH-5 | 92.5-95 | 35.8 | 61.9 | 26.0 | CH |

Figure B-26

Lorilla Engineering
Tolt Hill Road Bridge




| Sample Number | Depth | Plasticity Index | Liquid Limit | Plastic Limit | Classification |
|---|---|---|---|---|---|
| B11, SH1 | 10-12 | 19.0 | 46.2 | 27.2 | ML |
| B12, S4 | 10-11.5 | 14.1 | 39.0 | 24.9 | ML |
| B13, SH1 | 10-12 | 5.6 | 28.4 | 22.7 | ML |

Figure B-27

# APPENDIX C

## REFERENCES

## APPENDIX C

## REFERENCES

American Association of Highway Transportation Officials, 1996 and 2002, Standard Specifications for Highway Bridges.

Anderson, J.B., and F.C. Townsend. (2001) "SPT and CPT Testing for Evaluating Lateral Loading of Deep Foundations." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(11), 920-925.

Bartlett, S.F. and T.L. Youd. (1992). "Empirical analysis of horizontal ground displacement generated by liquefaction –induced lateral spread." Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, Buffalo, New York.

Brown, D. (2002) "Effect of construction on Axial Capacity of Drilled Foundations in Piedmont Soils." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(12), 967-973.

Duncan, J.M. and A.L. Buchignani. (1976). An Engineering Manual for Settlement Studies. University of California, Berkeley.

Elhakim, A.F. and P.W. Mayne. (2002). Discussion of "Side Resistance in Piles and Drilled Shafts". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(5), 448-449

Fellenius, B.H. (2002). Discussion of "Side Resistance in Piles and Drilled Shafts". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(5), 446-448.

Fitzpatrick, B.T. and K.J. Wissmann (2002) GeoPier Shear Reinforcement for Global Stability and Slope Stability, Technical Bulletin No. 5, GeoPier Foundation Company, Inc., Scottsdale, AZ

Fox, N.S. and M.J. Cowell (1998) GeoPier Foundation and Soil Reinforcement Manual, GeoPier Foundation Company, Inc., Scottsdale, AZ.

Idriss, I.M. and J.I. Sun. (1992). User's manual for SHAKE91, a computer program for conducting linear seismic response analyses of horizontally layered soil deposits: Davis, California, University of California, Department of Civil and Environmental Engineering.

Ishihara, K. and M. Yoshimine. (1992) "Evaluation of settlements in sand deposits following liquefaction during earthquakes." Soils and Foundations, 32(1), 173-188.

Kramer, S.L. (1996) Geotechnical Earthquake Engineering, Prentice-Hall, Upper Saddle River, New Jersey.

Kulhawy, F.H. (1991). Chapter 14: Drilled Shaft Foundations. Foundation Engineering Handbook, Van Nostrand Reinhold, New York.

Ng, C.W.W., T.L.Y. Yau, J.H.M. Li, W.H. Tang. (2001) "Response of Laterally Loaded Large-Diameter Bored Pile Groups." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(8), 658-669.

O'Neill, M.W. (2001). "Side Resistance in Piles and Drilled Shafts." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(1), 3-16.

O'Neill, M.W. (2002). Closure to "Side Resistance in Piles and Drilled Shafts". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 128(5), 450.

Report of a Task Force Sponsored by the G-I Deep Foundation Committee, (2000). "Nondestructive Evaluation of Drilled Shafts." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 126(1), 92-95.

Seed, H.B. and I.M. Idriss. (1971). "Simplified procedure for evaluating soil liquefaction potential." Journal of Soil Mechanics and Foundations Division, ASCE, 97(9), 1249-1273.

Seed, H.B., K. Tokimatsu, L.F. Harder, and R. Chung. (1985). "The influence of SPT procedures in soil liquefaction resistance evaluations." Journal of Geotechnical Engineering, ASCE, 111(12), 1425-1445.

Turner, J.P. (1992) "Constructability for Drilled Shafts." Journal of Construction Engineering and Management, 118(1), 77-93.

United States Geological Survey website providing 1996 and 2002 probabilistic ground motion values.

Vesic, A.S. (1977). Design of Pile Foundations, Synthesis of Highway Practice 42, Transportation Research Board, Washington, D.C.

West Consultants (April 2005) Bridge Hydraulics and Scour Assessment for Tolt Bridge No. 1834A. (To be published)

Wissmann, K.J., B.T. FitzPatrick, D.J. White and B.H. Lien (2002) Improving global stability and controlling settlement with GeoPier soil reinforcing elements. Fourth International Conference on Ground Improvement Techniques, Kuala Lumpur, Malaysia, March 26-28.

Youd, T.L etal (2001) "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 127(10), 817-833.

**LORILLA ENGINEERING, INC., P.S.**
P.O. Box 46018, Seattle, WA 98146

**Geotechnical Consulting**
206-241-7287/Fax:433-0512

March 10, 2006

Mr. Raymond Moy, P.E.
Lin & Associates, Inc.
719 Second Avenue, Suite 1218
Seattle, WA 98104

Re: Addendum No. 1
Geotechnical Engineering Study, Final Design Report
Tolt Bridge No.1834A Replacement dated September 2005
King County, WA

Dear Mr. Moy,

This letter should be considered as an addendum to our final design report referenced above. The purpose of this addendum is to address the changes made to the east approach fill after submittal of our final report referenced above.

**Project Changes**

Mechanically stabilized earth (MSE) walls were proposed to support the east approach fill. Because of the depth of the utilities to be installed within the roadway fill, gabion walls will now be used instead of the MSE walls. We understand that the contractor will provide shop drawings for the gabions to be reviewed by the project team. This will allow flexibility in the use of surcharge filling techniques to accelerate fill settlements if the contractor or project team chooses this option.

The north side of the east approach fill was to be built over the roadway fill currently supporting NE Tolt Hill Road. We understand that the existing roadway fill will be removed to allow construction of gabion walls of similar heights on the north and south sides of the realigned NE Tolt Hill Road.

Lin & Associates provided Sheets S55 through S57 dated 1/06 for our use.

**Stability of North Side of East Approach Fill**

Our scope of work was to review the stability issues on the north side of the east approach fill given that the existing roadway fill will be removed and correspondingly higher than anticipated walls will be constructed. Instead of a maximum of 5 feet of fill, the new maximum wall height will be 10.4 feet on the north side.

Based on a review of wall sections provided by the project team, the ground surface elevation difference between the north wall (Wall 3) and the south wall (Wall 4) ranges from 6 to 8 feet along the alignment of the north wall. This higher grade extends to the north of Wall 3 a minimum of 20 feet.

This existing fill has served to preload the soils along the north wall alignment. With the removal of the existing fill and replacement with the new gabion wall configuration, a net maximum increase in load of 5 feet of fill is anticipated. This maximum 5 feet includes an 18-inch high slope above the gabion and an 18-inch preload.

The underlying native soil along the Wall 3 alignment has been preloaded with the existing roadway fill. This preloading has caused settlement and increase in shear strength in these soils. This was taken into consideration in our stability analyses.

Our stability analyses included a fill height of 18 inches with a 2H:1V slope above a maximum 10.4 foot high gabion wall. We included an 18-inch preload fill at the top of the fill. The results of our stability analyses indicated a minimum factor of safety against global failure of 1.8. As such, stabilizing berms are not required for the construction of Wall 3.

It should be noted however, that if the total preload height approaches 4 feet, the need for stabilizing berms north of Wall 3 should be reassessed.

Stabilizing berms are still required for the south wall (Wall 4). In reviewing the temporary stabilizing berm schematic on Sheet S57, we provide recommended changes on the attached drawing.

Design parameters for the gabion wall are presented in Table 1. The design values assume that the backfill soils are placed and compacted in accordance with Washington State Department of Transportation Standard Specifications for Road, Bridge and Municipal Construction, Section 2-09.3(1)E.

**TABLE 1: Gabion Wall Design Parameters**

| **Soil Properties** | **Backfill Soils** | **Foundation Subgrade Soils** | | |
|---|---|---|---|---|
| | Retained Soil | South Wall (Wall 4) Alluvial Silt and Clay Prior to Fill Placement | South Wall (Wall 4) Alluvial Silt and Clay after Fill Settlement | North Wall (Wall 3) Alluvial Silt and Clay below Excavated Roadway Fill |
| Unit Weight (pcf) | 125 | 110 | 115 | 115 |
| Friction Angle (degrees) | 36 | 0 | 0 | 0 |
| Cohesion (psf) | 0 | 250 | 800 | 800 |
| Allowable Soil Bearing Pressure (psf) | N/A[Note1] | 450[Note2] | 1500 | 1500 |

Note 1: Not applicable
Note 2: Stabilizing berms required to support south wall (Wall 4)

Lorilla Engineering, Inc., P.S.
P.O. Box 46018
Seattle, WA 98146
206-241-7287

Should you have any questions regarding this addendum, please contact Lorilla Engineering at your convenience.

Sincerely,

**LORILLA ENGINEERING, INC., P.S.**

Michele Lorilla, P.E.
Geotechnical Consultant

Enclosure: Recommended changes on Sheet S57, Structural Earth Walls 3 & 4-2

MICHELE M. LORILLA
WASHINGTON
3.10.2006
27837
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 06-05-2006

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| Allowable Soil Bearing Pressure (psf) | N/A[Note1] | 450[Note2] | 1500 | 1500 |

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P.O. Box 46018
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Should you have any questions regarding this addendum, please contact Lorilla Engineering at your convenience.

Sincerely,

**LORILLA ENGINEERING, INC., P.S.**

Michele Lorilla, P.E.
Geotechnical Consultant

Enclosure: Recommended changes on Sheet S57, Structural Earth Walls 3 & 4-2


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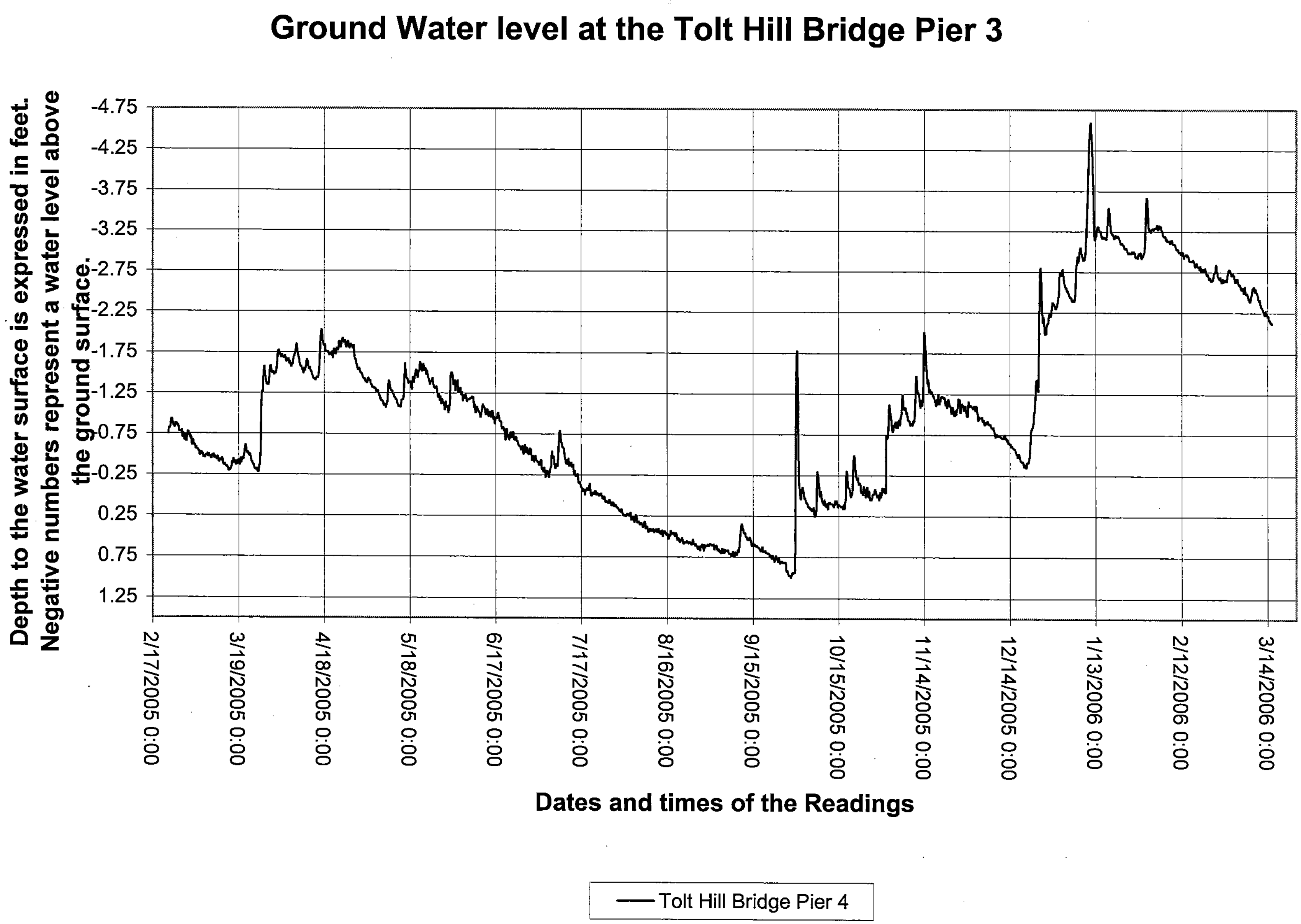
# Tolt Hill Bridge

## MEASURED GROUNDWATER SURFACE LEVELS AT PIER 3

| Date of Measurement | Groundwater Depth Below Ground Surface (feet) | |
|---|---|---|
| | Pier 3 Observation Well[1] | |
| February 22,2005 | -0.83 | |
| February 23,2005 | -0.84 | |
| March 3, 2005 | -0.61 | |
| May 9, 2005 | -1.1 | |
| May 17, 2005 | -1.41 | |
| July 18, 2005 | -0.07 | |
| July 20, 2005 | -0.06 | |
| October 6,2005 | 0.15 | |
| November 29, 2005 | -0.98 | |
| January 12, 2006 | -2.78 | |
| March 15, 2006 | -2.12 | |
| | | |
| | | |

Notes

1. Negative numbers indicate that groundwater level is above the ground surface.
2. On January 12, 2006 groundwater was flowing from the top of the protective well cover, indicating a groundwater level somewhat higher than that measured.
3. The top of the protective cover is 2.78 feet above the ground surface.


Ground Water level at the Tolt Hill Bridge Pier 3
Depth to the water surface is expressed in feet. Negative numbers represent a water level above the ground surface.
-4.75
-4.25
-3.75
-3.25
-2.75
-2.25
-1.75
-1.25
-0.75
-0.25
0.25
0.75
1.25
2/17/2005 0:00
3/19/2005 0:00
4/18/2005 0:00
5/18/2005 0:00
6/17/2005 0:00
7/17/2005 0:00
8/16/2005 0:00
9/15/2005 0:00
10/15/2005 0:00
11/14/2005 0:00
12/14/2005 0:00
1/13/2006 0:00
2/12/2006 0:00
3/14/2006 0:00
Dates and times of the Readings
Tolt Hill Bridge Pier 4