Project No. 40310A.00
January 22, 2024

Jibboom Street LLC
PO Box 5638
Tahoe City, California 96145

Attention: Sean Whelan

Reference: Residences at Jibboom
Preliminary Geotechnical Engineering Report, April 7, 2003
Truckee, California

Subject: Geotechnical Engineering Report Update Letter

This letter presents our geotechnical engineering report update for the proposed commercial development to be constructed at 10002-10012 Jibboom Street (currently designated as Residences at Jibboom) in Truckee, California. Our current scope of services included a site visit, review of our previously prepared geotechnical engineering report for the subject property, and preparation of this letter. NV5 (formerly Holdrege & Kull) prepared a geotechnical engineering report for the subject property dated April 7, 2003 (formerly designated as High Street Frontage Improvements, Project No. 40310-01). Our previous report for the site was prepared for a planned two-story retail building, nine single-family residences along High Street, relocation of an existing hotel in the east portion of the site and widening of High Street that included steep cut slopes with rock slope protection. NV5 (formerly Holdrege & Kull) also provided earthwork observation and material testing services during construction of an existing retaining wall and rock slope protection located near the north edge of the site in 2004. This letter is considered a part of the geotechnical engineering report for the subject property and should be bound to it.

Our professional opinion is that the recommendations provided in our aforementioned geotechnical engineering report for the subject property are not applicable to the current planned project. This letter contains updated geotechnical recommendations that will supersede all of the recommendations sections presented in the Geotechnical Engineering Report for the subject property dated April 7, 2003. This report update letter should be used in conjunction with our previously prepared geotechnical engineering report for project design, submittal, and construction.

The current project planned for the site is significantly different from that previously proposed. Therefore, NV5 must be onsite during grading to observe subsurface conditions exposed in foundation excavations at the time of construction and provide additional recommendations, as needed.

Site Description

Based on our review of parcel information contained on the Nevada County My Neighborhood map website (https://maps.nevadacountyca.gov/myneighborhood/), the project site consists of seven contiguous parcels that total approximately 0.98-acre of previously developed land in downtown Truckee, California. The Nevada County Assessor's Parcel Numbers (APNs) for the site are 019-102-011, -012, -013, -014, -015, -016, and -017-000. The physical addresses for the site are 10002, 10004, 10006, 10008, 10060, 10090, 10010, and 10012 Jibboom Street. The project site is bounded by a steep cut slope containing rock slope protection and High Street to the northwest and northeast, Jibboom Street to the southeast, and developed commercial property to the southwest.

APNs 019-102-011, -012, -013, -014, and -015-000 consist of a gravel surface parking lot. An approximately 10-foot high wooden and concrete retaining wall is present along the northwest edge of most of these parcels. Rock slope protection is present on the steep cut slope above the retaining wall. An existing single-family residence is present on APN 019-102-016-000. The single-family residence appears to be located on top of approximately 3 to 4 feet of existing fill.

Due to the previously developed nature of the site, there is essentially no vegetation except for very scattered brush. At the time of our site visit completed on January 5, 2024, the site was covered with approximately one to two inches of snow. Therefore, we could not observe the ground surface over most of the site.

The site is located at 39.3293°N latitude and 120.1850°W longitude (WGS84 datum). According to the preliminary site plan (Sheet 1.0) prepared by MWA, Inc. dated August 7, 2023, site elevations range from approximately 5,866 feet above mean sea level (MSL) near the west property corner to approximately 5,818 feet above MSL near the southeast property corner. The site slopes gently to very steeply down in a general northwest to southeast direction. Surface water drainage consists of overland flow that is controlled through existing back-of-wall drains.

Proposed Improvements

Information about the proposed project was obtained from our site visit, correspondence with you and Lindy Winter of MWA, Inc.; review of partial preliminary project plans (Sheet 1.0, Site Plan; Sheet 3.1A, Elevations Building A; Sheets 3.2 and 3.3, Elevations Building B-C; Sheets 3.4 and 3.5, Elevations Building D; and Sheets 3.6 and 3.7, Exterior Finish Materials Building D) prepared by MWA, Inc. dated August 7, 2023, and January 17, 2024; and our previous experience at the site and project area. As currently proposed, the project will involve removal of the existing single-family residence and constructing three residential buildings (Buildings A, B-C, and D). Buildings A and B-C will be three-story wood- and steel-frame structures with lower-level parking and/or retail space. Building D will be a four-story wood- and steel-frame structure with lower-level lobby area. Each planned building will be supported by slab-on-grade floors with conventional shallow spread footings.

We understand that the existing concrete retaining wall and rock slope protection behind Buildings A and B-C will remain and that decks associated with Building A will be supported by piers anchored to the slope beneath the rock slope protection. The existing retaining wall and rock slope protection behind Building D will be removed and replaced with a new concrete retaining wall. Excavations on the order of approximately 22 feet are planned for the reconstructed slope behind Building D. New retaining wall heights were not available at the time we prepared this letter report, however we understand that a tiered retaining wall approach will be used behind Building D. Rock slope protection may also be included for slopes behind Building D. Appurtenant construction will include nine off-street asphalt concrete paved parking spaces, two asphalt concrete access driveways, underground utilities, and landscaping.

Structural loads were not available at the time we prepared this letter. Maximum anticipated wall and column loads will be about 6 kips per lineal foot and 100 kips, respectively. We anticipate average cut and fill depths will be about 2 to 4 feet and will not exceed about 22 feet. No detailed grading plans were available for our review.

Updated Site Geology and Regional Faulting

The project is located in a potentially active seismic area. To evaluate the site geology and location of mapped faults relative to the project site, we reviewed the following maps:

- *Fault Activity Map of California <http://maps.conservation.ca.gov/ cgs/fam/>;* by Charles W. Jennings and William A. Bryant, California Geological Survey, Geologic Data Map No. 6, 2010;
- Google Earth/KMZ files provided by USGS Earthquakes Hazards Program. *Quaternary Faults & Folds in the U.S.* Retrieved January 5, 2024. <https://earthquake.usgs.gov/ learn/kml.php>;
- *EQ Zapp: California Earthquake Hazards Zone Application*. *http://www.conservation.ca.gov/cgs/geohazards/eq-zapp;* California Geological Survey.
- *Geologic Map of the North Lake Tahoe-Donner Pass Region, Northern Sierra Nevada, California*, by Arthur Gibbs Sylvester et al., California Geological Survey, 2012; and
- *Geologic Map of the Lake Tahoe Basin, California and Nevada*, by George J. Saucedo, California Geological Survey, 2005.

The geologic maps indicate that the site is generally underlain by Pliocene aged glacial outwash deposits that are comprised of silt, sand, gravel, cobbles, and boulders. Based on our previous subsurface investigation, described below and our experience in the area, near-surface soil conditions appear to be consistent with the published geologic maps.

The potential risk of fault rupture is based on the concept of recency and recurrence. The more recently a particular fault has ruptured, the more likely it will rupture again. The California State Mining and Geology Board define an “active fault” as one that has had surface displacement within the past 11,000 years (Holocene). Potentially active faults are defined as those that have ruptured between 11,000 and 1.6 million years before the present (Quaternary). Faults are generally considered inactive if there is no evidence of displacement during the Quaternary period.

The referenced geologic maps and interactive fault databases show several active and potentially active faults located near the project site, including the Dog Valley Fault (active, approximately 4.0 miles northwest), a group of unnamed faults southeast of Truckee (active and potentially active, through the northeast edge of the site [Saucedo, 2005] approximately 160 feet southwest and 1.6 miles southeast of the west edge of the site [KMZ files and Fault Activity Map of California]), the Polaris Fault (active, approximately 2.5 miles northeast), the West Tahoe – Dollar Point Fault Zone (potentially active, approximately 3.9 miles southeast), the Tahoe Sierra Frontal Fault Zone (potentially active, approximately 6.0 miles south-southwest), the West Tahoe Fault (active, approximately 16.8 miles south-southeast), and the North Tahoe Fault (active, approximately 13.3 miles southeast). Earthquakes associated with these faults may cause strong ground shaking at the project site.

Potential Seismic Hazards

Primary hazards associated with earthquake faults include strong ground motion and surface rupture. Based on the geologic map prepared by Saucedo (2005), a northwest-southeast trending fault associated with the unnamed faults southeast of Truckee is mapped near the northeast edge of the site. Due to the scale of the map, it is difficult to discern the exact location of the fault, but it appears to cross the northeast edge of the site. This fault trace is shown as discontinuous (approximately 2 miles in length) and approximately located (concealed or dotted beneath the glacial outwash deposits). Due to the discontinuous nature of the fault trace, it is not considered active and the potential for surface rupture is considered low. This fault trace is not shown on the geologic map prepared Sylvester et al (2012) but is shown approximately 160 feet southwest of the west edge of the site on the KMZ files and Fault activity Map of California. It should be noted that NV5 did not perform a site-specific surface reconnaissance investigation nor a subsurface investigation of the project site and vicinity for the purpose of identifying the exact locations of any faults that may negatively impact the project site.

Earthquakes centered on regional faults in the area, such as the West Tahoe Fault, would likely result in higher ground motion at the site than earthquakes centered on smaller faults that are mapped closer to the site.

Based on our review of the *California Earthquake Hazards Zone Application (EQ Zapp – https://www.conservation.ca.gov/cgs/geohazards/eq-zapp),* the site is not located within an active Earthquake Hazards Zone (formerly referred to as the Alquist-Priolo fault zone).

Secondary seismic hazards include liquefaction, lateral spreading, and seismically induced slope instability. These potential hazards are discussed below.

Soil Liquefaction

Liquefaction is a phenomenon where loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup. Cyclic loading, such as that caused by an earthquake, typically causes an increase in pore water pressure and subsequent liquefaction. Based on the results of our previous subsurface investigation completed at the site, near-surface soil at the site generally consists of medium dense to dense granular soil types with varying amounts of gravel, cobbles, and boulders. This soil profile will have a low potential for liquefaction.

Lateral Spreading

Lateral spreading is the lateral movement of soil resulting from liquefaction of subadjacent materials. Since we anticipate that there is a low potential for liquefaction of soil at the site, the potential for lateral spreading to occur is also considered low.

Slope Instability

Slope instability includes landslides, debris flows, and rockfall. No landslides or debris flows hazards were observed in the project area. The north portion of the site consists of steep slopes armored with rock slope protection and supported at the base by an existing concrete retaining wall. Care should be taken during construction to avoid dislodging existing rock slope protection. Due to the planned excavations within the steep slopes near the northeast edge of the site, we recommend that Support of Excavation (SOE) engineering be completed to provide design criteria for soil nail walls and/or other methods of cut stabilization. We can provide this service at an additional cost once project plans have become more finalized.

Subsurface Conditions

Our previous geotechnical engineering report prepared for the site was completed for the existing concrete retaining wall in the north portion of the site as well as for a planned two-story retail building, nine single-family residences along High Street, relocation of an existing hotel in the east portion of the site and widening of High Street that included steep cut slopes with rock slope protection. At the time of our previous investigation, several granitic and volcanic boulders up to approximately 14 feet in diameter were observed at the ground surface across the site. Five exploratory trenches were excavated within the former cut slope below High Street to depths ranging from approximately 4 to 5.5 feet below the ground surface (bgs). Subsurface conditions encountered in the trenches consisted of approximately 10 to 12 inches of loose silty Sand (SM) containing organic material and cobbles (topsoil). Underlying the topsoil, approximately 3 feet of existing fill was encountered in one trench that consisted of very loose silty Sand (SM) with varying amounts of gravel, cobbles, organic material, and debris. Native soil conditions consisted of loose (mostly slough from the cut slope face) to dense silty Sand (SM) and well-graded Gravel with silt and sand (GW-GM) and varying amounts of cobbles and boulders to the maximum depth explored of approximately 5.5 feet bgs.

Groundwater was not encountered during our previous investigation completed at the site. However, fluctuations in soil moisture content and groundwater levels should be anticipated depending on precipitation, irrigation, runoff conditions and other factors. Based on our experience in the project area, seasonal saturation of near-surface soil should be anticipated, especially during and immediately after seasonal snowmelt or heavy rain events.

Conclusions

The following conclusions are based on our field observations, our experience in the project area, and the results of our previously prepared geotechnical engineering report.

1. We anticipate that the proposed foundations will bear on undisturbed granular native soil. A <u>significant</u> amount of over-sized material (cobbles and boulders) should be anticipated in site excavations. We do not anticipate that highly plastic, potentially expansive, or compressible soil conditions exist at the site. The potential for liquefaction and lateral spreading at the site is considered low. Due to the limited nature of our previous subsurface investigation completed at the site and current planned project, a representative of NV5 must be onsite during construction to observe subsurface conditions encountered in footing excavations and provide additional recommendations, as needed.
2. Due to planned excavations within the steep slopes near the northeast corner of the site, we recommend SOE engineering to provide design criteria for soil nail walls and/or other methods of cut stabilization. Furthermore, we recommend that deck piers for Building A extend through the rock slope protection and bear directly on the underlying native soil. We have provided updated recommendations for foundation design criteria in the section below.
3. Although groundwater was not encountered in our trenches to the maximum depth explored, near-surface soil layers will likely become seasonally saturated. Positive surface water drainage will be important across the site to help reduce the potential for groundwater to cause moisture migration through concrete slabs-on-grade, cause degradation of asphalt concrete pavements, and contribute to frost heave and other adverse conditions. We have provided updated recommendations to reduce the potential for these adverse effects in the recommendations section below.
4. Based on our field observations, it appears that approximately 3 to 4 feet of existing fill is present in the eastern portion of the site and area of the existing residence. We also anticipate that existing fill is present across the current parking lot at the site, likely at a depth less than 2 feet bgs. Based on our current understanding of the project as currently proposed, it is likely that existing fill will be removed during grading. Due to the potential for excessive settlement, existing fill will not be suitable for direct support of structural improvements at the site. Existing fill should either be replaced with compacted structural fill or improvements may be founded directly on properly prepared underlying native soil. We have provided updated recommendations for removal and replacement of existing fill in the section below.
5. We anticipate that site soil will generally be suitable for reuse as structural fill; however, processing to remove oversize material will likely be necessary.
6. Similar to most of California, the site is located within a seismically active area. We have provided seismic design parameters consistent with the 2022 California Building Code in the recommendations section below.

Revised Recommendations

The following recommendations are based on our understanding of the project as currently proposed, our site visit, and our experience in the project area. The recommendations provided below supersede all of those provided in our April 7, 2003 Geotechnical Engineering Report for the subject property.

5.1 Earthwork

The following sections present our recommendations for site clearing and grubbing, preparation for and placement of fill material, cut/fill slope grading, temporary excavations, utility trench construction, and construction dewatering.

5.1.1 Clearing and Grubbing

Areas proposed for fill placement, road and driveway construction, and building areas should be cleared and grubbed of vegetation and other deleterious materials. Existing vegetation, organic topsoil, and any debris should be stripped and hauled offsite or stockpiled outside the construction limits. Due to the developed nature of most of the site, we anticipate that the average depth of stripping will be minimal. Based on our field observations, the average depth of stripping may be on the order of 2 to 4 inches in the eastern portion of the site and area of the existing residence. An additional approximately 6 inches of pine needle duff should be anticipated along the slope above the existing residence in the previously undisturbed area at the site. Organic surface soil and pine needle duff may be stockpiled for future use in landscape areas but is not suitable for use as structural fill. We anticipate that the actual depth of stripping will vary across the site and may be greater in wooded areas.

Man-made debris and backfill soil in our exploratory test pits or any other onsite excavations should be over-excavated to underlying, competent material and replaced with compacted structural fill. Grubbing may be required where concentrations of organic soil or tree roots are encountered during site grading.

Existing fill should be removed in areas that will support foundation elements, earth retention structures, concrete slabs-on-grade, and pavement sections. Based on our field observations, it appears that approximately 3 to 4 feet of existing fill is present in the eastern portion of the site and area of the existing residence. We also anticipate that existing fill is present across the current parking lot at the site, likely at a depth less than 2 feet bgs. Based on our current understanding of the project as currently proposed, it is likely that existing fill will be removed during grading. Existing fill should either be replaced with compacted structural fill or improvements may be founded directly on properly prepared underlying native soil. Existing fill material will likely be suitable for re-use as structural fill material provided any debris exceeding eight inches in maximum dimension and all organic or deleterious material are removed prior to placement. Preparation of the subgrade exposed by over-excavation and requirements for structural fill should be in accordance with recommendations provided below.

All rocks greater than 8 inches in greatest dimension (oversized rock) should be removed from the top 12 inches of soil, if encountered. Oversized rock may be used in landscape areas, *rock faced slopes, or removed from the site. Oversized rock should not be placed in fill without prior approval by the project geotechnical engineer.*

5.1.2 Preparation for Fill Placement

Prior to fill placement, all areas of existing fill material, man-made debris, or backfill soil should be removed to expose non-expansive native soil as discussed in the previous section.

Where fill placement is planned, the near-surface soil should be scarified to a depth of about 12 inches or to competent material and then uniformly moisture conditioned to within 2 percent of the optimum moisture content. Scarified and moisture conditioned soil should be recompacted with appropriate compaction equipment and proof rolled with a loaded, tandem-axle truck under the observation of an NV5 representative. Any areas that exhibit pumping or rutting should be over-excavated and replaced with compacted structural fill placed according to the recommendations below.

5.1.3 Fill Placement

All fill placed beneath structural improvements (e.g., foundation elements, pavements, and utility lines) and as part of a fill slope or retaining structure should be considered structural fill. Material used for structural fill should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. Structural fill should consist of granular material, nearly free of organic debris, with a liquid limit of less than 40, a plasticity index less than 15, 100 percent passing the 8-inch sieve, and less than 30 percent passing the No. 200 sieve. In general, the near-surface on-site soil has less than 30 percent passing the No. 200 sieve and meets the above recommendations. This site soil may be used for structural fill; however, processing to remove over-sized material will likely be needed. Based on our previous experience in the area, site soil may be above optimum moisture content even in late summer and may require air drying or additional compaction effort to reach the specified compaction. Moisture content, dry density, and relative compaction of fill should be evaluated by our firm at regular intervals during fill placement. Rock used in fill should be broken into fragments no larger than eight inches in diameter. Rocks larger than eight inches are considered oversized material and should be stockpiled for offhaul, later use in rock-faced slopes, or placement in landscape areas.

Imported fill material should be predominantly granular, non-expansive, and free of deleterious or organic material. Import material that is proposed for use on site should be submitted to NV5 for approval and laboratory analysis at least 72 hours prior to import.

If site grading is performed during periods of wet weather, near-surface site soil may be significantly above its optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact fill materials to the recommended compaction criteria. Fill material may require drying to facilitate placement and compaction, particularly during or following the wet season or spring snowmelt. Suitable compaction results may be difficult to obtain without processing the soil (e.g., discing during favorable weather, covering stockpiles during periods of precipitation, etc.).

Compaction requirements (maximum dry density and moisture content) specified in this report reference ASTM D1557 – Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort. Structural fill should be uniformly moisture conditioned to within 2 percent of the optimum moisture content and placed in maximum 8-inch thick, loose lifts (layers) prior to compacting. Structural fill should be compacted to at least 90 percent of the maximum dry density. The upper 8 inches of structural fill in paved areas should be compacted to at least 95 percent of the maximum dry density. Moisture content, dry density, and relative compaction of fill should be evaluated by our firm at regular intervals during fill placement. The earthwork contractor should assist our representative by preparing test pads with the onsite earth moving equipment.

Structural fill material with more than 30 percent rock larger than ¾-inch cannot be reliably tested using conventional compaction testing equipment. We recommend that a procedural approach, or method specification, be used for quality assurance during rock fill placement rather than a specified relative compaction. The procedural requirements will depend on the equipment used, as well as the nature of the fill material, and will need to be determined by the geotechnical engineer on site. Based on our experience in the area, we anticipate that the procedural specification will require a minimum of six passes with a Cat 563 or similar, self-propelled vibratory compactor to compact a maximum 8-inch thick loose lift. Processing or screening of the fill may be required to remove rocks larger than 8-inches in maximum dimension. Continuous observation by an NV5 representative will be required during fill placement to confirm that procedural specifications have been met.

Differential fill depths beneath the structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet across a building pad, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad were used in this example, the cut portion would need to be over-excavated 3 feet and rebuilt with compacted structural fill.

5.1.4 Cut/Fill Slope Grading

Permanent cut and fill slopes at the subject site should be stable at inclinations up to 2H:1V (horizontal to vertical); however, we recommend re-vegetating or armoring all cut/fill slopes to reduce the potential for erosion. Steeper slopes may be possible at the site provided slopes are protected from excessive erosion using rock slope protection or similar slope reinforcement. Slopes steeper than 2H:1V (horizontal to vertical) should be evaluated on a case-by-case basis.

Fill should be placed in horizontal lifts to the lines and grades shown on the project plans. Slopes should be constructed by overbuilding the slope face and then cutting it back to design slope grades. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

Equipment width keyways and benches should be provided where fill is placed on side-slopes with gradients steeper than 5H:1V. The keyway should be excavated at the toe of the slope and extend into competent material. Benching must extend through loose surface soil into *suitable material and be performed at intervals such that no loose soil is left beneath the fill. NV5 should observe keyways and benches prior to fill placement.*

The upper two to five feet of cut slopes should be rounded into the existing terrain above the slope to remove loose material and produce a contoured transition from cut face to natural ground. Scaling to remove unstable cobbles and boulders may be necessary. Fill slopes should be compacted as recommended for the placement of structural fill. The upper four to eight inches may be scarified to help promote revegetation.

5.1.5 Permanent Slope Angles and Rock Slope Protection

Stability of cut and fill slopes involves two separate aspects. The first concerns true slope stability related to mass wasting and landslides. True slope stability is dependent upon the shear strength, unit weight, and moisture content of soil as well as the slope angle. Site soil generally consists of medium dense to dense, granular material. Cut and fill slopes at the subject site should be stable from a slope stability standpoint at inclinations up to 1.5H:1V (horizontal to vertical) for heights up to about 10 feet.

The second aspect of slope stability involves erosion potential and is dependent on numerous factors including grain size distribution, cohesion, moisture content, slope inclination, slope length, and velocity of water or wind on the slope face. Steep constructed slopes reduce the amount of disturbed area and result in shorter slope lengths. Slope performance will be primarily affected by surface runoff and erosion control. Permanent cut and fill slopes may be constructed at inclinations up to 1.H:1V provided slopes are protected from excessive erosion. Steeper slopes should be evaluated on an individual basis.

All slopes steeper than 2H:1V should be stabilized with rock slope protection (RSP). We recommend 12- to 18-inch RSP for slopes up to 12 feet in vertical extent and 18- to 30-inch RSP for slopes between 12 and 20 feet in vertical extent. RSP should be placed in accordance with Caltrans Standard Specifications Method A Placement. Placing of rocks by dumping should not be permitted. Rocks should decrease in size from the bottom to the top of the slope. Stones should have a specific gravity of at least 2.5 and a percentage of wear (per ASTM C535) of not more than 45. Rocks should be of such shape as to form a stable protection structure of the required section. Rounded, flat, or needle-shaped boulders will not be accepted unless the thickness of the individual rocks is greater than 1/3 the length. A minimum 12-inch-deep footing trench should be excavated along the toe of the slope. The larger rocks should be placed in the footing trench with full contact between the rock and soil. Rocks should be placed with their longitudinal axis horizontal and arranged so that each rock above the foundation course has 3 points of bearing on the underlying rocks. Rocks should be placed in such a manner so as to avoid continuous joint planes in the vertical and lateral directions. Bearing on smaller rocks that may be used for chinking voids will not be acceptable. Local surface irregularities of the slope protection should not vary from the planned slope by more than one foot measured at right angles to the slope.

In the event seepage is encountered emanating from a cut slope, we recommend a well-graded filter layer consisting of ¾ to 2½-inch crushed gravel be placed behind the RSP to *help reduce the loss of soil through the face of the RSP. The filter layer should be at least 12 inches thick.*

The upper two feet to five feet of cut slopes should be rounded into the existing terrain above the slope to remove loose material and produce a contoured transition from cut face to natural ground. Scaling to remove unstable cobbles and boulders may be necessary. Fill slopes should be compacted as recommended for the placement of structural fill. The top 4 to 8 inches may be scarified to help promote revegetation.

5.1.6 Temporary Unconfined Excavations

Based on our understanding of the proposed project, temporary unconfined excavations up to about 22 feet may be necessary for construction of the proposed project. For planning purposes, temporary excavations that extend through near-surface soil should be designed at inclinations of 1H:1V (Horizontal to Vertical) or flatter. We understand that the planned excavations will require vertical slopes. We recommend Support of Excavation (SOE) design (soil nail walls and/or other methods of cut stabilization) for the planned steep and large excavations at the site. We can provide this service for an additional fee and present the results under separate cover.

These temporary slope inclinations may require modification in the field during construction or where loose soil, groundwater seepage, or existing fill is encountered. The slope should be scaled of loose cobbles and boulders. Higher slopes should be covered with strong wire or fabric, firmly secured to prevent roll down of cobbles or other deleterious materials. The contractor is responsible for the safety of workers and should strictly observe federal and local Occupational Safety and Health Administration (OSHA) requirements for excavation shoring and safety. Some raveling of temporary cut slopes should be anticipated. During wet weather, surface water runoff should be prevented from entering excavations. To reduce the likelihood of sloughing or failure, temporary cut slopes must not remain over the winter.

5.1.6 Underground Utility Trenches

We anticipate that the contractor will be able to excavate underground utility trenches using conventional earthmoving equipment across the majority of the site. However, confined excavations for underground utilities will likely encounter over-sized material (cobbles and boulders). Excavations that extend into boulders will likely be difficult and may require a large track-mounted excavator equipped with a ripper tooth or hydraulic hammer. An excavator with a “thumb” attachment may increase ease of boulder removal at the site.

We expect that some caving and sloughing of utility trench sidewalls will occur. OSHA requires all utility trenches deeper than five feet bgs be shored with bracing equipment or sloped back prior to entry.

Shallow subsurface seepage may be encountered in trench excavations, particularly if utility trenches are excavated during the spring or early summer. The earthwork contractor may need to employ dewatering methods as discussed in the Construction Dewatering section below to excavate, place, and compact trench backfill materials.

We recommend utility trench cutoff walls and relief drains be constructed where utility lines enter structures and for all utility lines that slope toward planned structures. We can provide details for cutoff-wall and relief-drain construction upon request.

Soil used as trench backfill should be non-expansive and should not contain rocks greater than 3 inches in maximum dimension. Trench backfill should consist of uniformly moisture conditioned soil and be placed in maximum 8-inch thick loose lifts prior to compacting. Unless otherwise specified by the applicable local utility district, pipe bedding and trench backfill should be compacted to at least 90 percent of the maximum dry density. Trench backfill placed within 8 inches of building subgrade and driveway areas should be compacted to at least 95 percent of the maximum dry density. The moisture content, density, and relative compaction of fill should be tested by NV5 at regular intervals during fill placement.

5.1.7 Construction Dewatering

During our subsurface exploration, we did not encounter groundwater seepage in our exploratory test pits. If grading is performed during or immediately following the wet season or spring snowmelt, seepage may be encountered during grading. We should observe those conditions, if they are encountered, and provide site specific subsurface drainage recommendations. The following recommendations are preliminary and are not based on a groundwater flow analysis.

We anticipate that dewatering of excavations can be performed by gravity or by constructing sumps to depths below the excavation and removing water with pumps. To maintain stability of the excavation when placing and compacting trench backfill, groundwater levels should be drawn down at least two feet below the lowest point of the excavation.

If seepage is encountered during trench excavation, it may be necessary to remove underlying saturated soil and replace it with free draining, open-graded, crushed rock (drain rock). Soil backfill may be placed after backfilling with drain rock to an elevation higher than encountered groundwater.

5.2 Surface Water and Foundation Drainage

This section of the report presents our recommendations to reduce the possibility of surface water and near-surface groundwater entering below grade areas. Care should be taken to reduce water and moisture introduced into the building interior during construction.

Based on our observations and past experience with geotechnical investigations in the project vicinity, there is a relatively high potential for seasonal saturation of near-surface soil and groundwater seepage into foundation areas. Near-surface groundwater may migrate through concrete floor slabs, degrade asphalt concrete pavements, increase frost heave, and contribute to other adverse conditions.

Final site grading should be planned so that surface water is directed away from all foundations and pavements. Ponding of surface water should not be allowed near pavements or structures. Paved areas should be sloped away from structures a minimum of 2 percent and drainage gradients should be maintained to carry all surface water to a properly designed *infiltration facility. The surface drainage system should generally be kept separate from the foundation (subsurface) drainage system. Surface water should not be infiltrated at elevations above the lowest foundation elements.*

Drains should be constructed on the upslope side of exterior foundations and should be placed along continuous interior wall foundations. Drains should extend to a properly designed infiltration facility. Recommended subsurface drain locations can be provided at the time of construction and when foundation elevations and configuration are known. Due to the gentle topography in the south portions of the site, elevations of foundations should be carefully planned so that it is possible to install gravity-fed drains that daylight a minimum of 10 feet from structures. Subsurface and foundation drain locations should be included on the project plans.

All foundation and slab-on-grade concrete should have a water to cement ratio of 0.45 or less. Underslab or blanket drains should be considered in slab-on-grade floor areas to reduce moisture transmission through the floor and help maintain subgrade support, particularly if the floor surface is lower than the adjacent exterior grade.

Where utility trenches slope toward structures, potential flow paths through utility trench backfill should be plugged with a less permeable material at the exterior of the foundation. All utility pipes should have sealed joints.

Roof drip-lines should be protected from erosion with a gravel layer and riprap. Roof downspouts should be directed to a closed collector pipe that discharges flow to positive drainage. Backfill soil placed adjacent to building foundations should be placed and compacted such that water is not allowed to pond or infiltrate. Backfill should be free of deleterious material and placed and compacted in accordance with the above earthwork recommendations.

5.3 Structural Improvement Design Criteria

The following sections provide design criteria for foundations, seismic design, slabs-on-grade, retaining walls, and pavement sections.

5.3.1 Foundations

Our opinion is that shallow spread foundations are suitable for support of the proposed structures. The following paragraphs discuss foundation design parameters and construction recommendations.

Exterior foundations should be embedded a minimum of 18 inches below the lowest adjacent exterior finish grade for frost protection and confinement. The bottom of interior footings should be at least 12 inches below lowest adjacent finish grade for confinement. Reinforcing steel requirements for foundations should be determined by the project structural engineer.

Foundations constructed on steep slopes for deck piers behind Building A should extend through existing rock slope protection and bear directly on medium dense to dense native granular soil. Foundation footings for deck piers should be embedded into native soil such *that there is a minimum of four lateral feet between the bottom of the pier and the ground surface.*

Foundations founded in competent, undisturbed native soil or compacted fill may be designed using an allowable bearing capacity of 3,000 psf for dead plus live loads. Allowable bearing pressures may be increased by 33 percent for transient loading such as wind or seismic loads.

Resistance to lateral loads (including transient loads) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the sides of foundations. Lateral resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at the ground surface to a maximum of 350d psf, where d equals the depth of the foundation in feet. A coefficient of friction of 0.40 may be used between poured-in-place concrete foundations and the underlying native soil. Lateral load resistance provided by passive soil pressure and friction may be used in combination without reduction.

Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loading. Based on anticipated foundation dimensions and loads, we estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be on the order of ½ inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than ¼ inch, provided footings are founded on similar materials (e.g., all on structural fill, native soil, or rock). Differential settlement between adjacent footings founded on dissimilar materials (e.g., one footing on soil and an adjacent footing on rock) may approach the maximum anticipated total settlement. Settlement of foundations is expected to occur rapidly and should be essentially complete shortly after initial application of loads.

Loose material remaining in footing excavations should be removed to expose firm, unyielding material or compacted to at least 90 percent relative compaction. Footing excavations should be moistened prior to placing concrete to reduce risk of problems caused by wicking of moisture from curing concrete. NV5 should observe footing excavations prior to reinforcing steel and concrete placement.

5.3.2 Seismic Design Criteria

In accordance with the 2022 California Building Code (CBC), the seismic design criteria shown in the table below should be used for the project site. The values were obtained for the site using the online Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps tool found at https://seismicmaps.org. Input values included the site's approximate latitude and longitude obtained from Google Earth and the Site Class. Site Class selection was based on our literature review, our subsurface investigation, our experience in the area, and the Site Class definitions provided in Chapter 20 of ASCE 7-16.

Table 5.3.2.1 – 2022 CBC Seismic Design Parameters

Description	**_Value_**	**_Reference_**
Approximate Latitude/Longitude	*39.3292ºN/120.1850 ºW*	*Google Earth*
Site Class	*C*	*Table 20.3-1, ASCE 7-16*
Mapped Short-Period Spectral Response Acceleration Parameter	$S_S = 1.334\ g$	*Figure 1613.2.1(1), 2022 CBC*
Mapped 1-Second Period Spectral Response Acceleration Parameter	$S_1 = 0.44\ g$	*Figure 1613.2.1(2), 2022 CBC*
Short Period Site Coefficient	$F_A = 1.2$	*Table 1613.2.3(1), 2022 CBC*
1-Second Period Site Coefficient	$F_V = 1.5$	*Table 1613.2.3(2), 2022 CBC*
Site Adjusted Short-Period Spectral Response Acceleration Parameter	$S_{MS} = 1.601\ g$	*Equation 16-36, 2022 CBC*
Site Adjusted 1-Second Period Spectral Response Acceleration Parameter	$S_{M1} = 0.659\ g$	*Equation 16-37, 2022 CBC*
Design Short-Period Spectral Response Acceleration Parameter	$S_{DS} = 1.067\ g$	*Equation 16-38, 2022 CBC*
Design 1-Second Period Spectral Response Acceleration Parameter	$S_{D1} = 0.44\ g$	*Equation 16-39, 2022 CBC*
Peak Ground Acceleration	*PGA = 0.576 g*	*Figure 22-7, ASCE 7-16*
Risk Category	*II*	*Table 1604.5, 2022 CBC*
Seismic Design Category	*D*	*Tables 1613.2.5 (1) & (2) 2022 CBC*

5.3.3 Slab-on-Grade Construction

Concrete slabs-on-grade may be used in conjunction with perimeter concrete footings. Slabs-on-grade should be a minimum of four inches thick. If floor loads higher than 250 psf, intermittent live loads, or vehicle loads are anticipated, the project structural engineer should provide slab thickness and steel reinforcing requirements.

Prior to constructing concrete slabs, the upper eight inches of slab subgrade should be scarified, uniformly moisture conditioned to within two percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density. Scarification and compaction may not be required if floor slabs are placed directly on undisturbed compacted structural fill.

Slabs should be underlain by at least four inches of Class 2 aggregate base placed over the prepared subgrade. The aggregate base should be compacted to a minimum of 95 percent of the maximum dry density. If a subdrain is installed as described below, slabs may be *constructed over the crushed gravel layer provided a moisture barrier will be placed over the gravel.*

To reduce the potential for groundwater intrusion, the project architect and/or owner should consider constructing a drain beneath concrete slabs-on-grade in areas where groundwater and/or saturated soil may be present during wet periods. Subdrains should consist of a minimum of four inches of clean crushed gravel placed over native subgrade leveled or sloped at two percent towards a 4-inch diameter perforated drain pipe. The drain pipe should be placed with perforations faced down in a minimum 12-inch wide gravel-filled trench. The depth of the trench may vary depending on cover requirements for the drain pipe and the slope required to drain water from beneath the slab to a properly constructed infiltration facility. A minimum of one pipe should be installed in each area of the slab surrounded by continuous perimeter foundation elements.

In slab-on-grade areas where moisture sensitive floor coverings are proposed, a vapor barrier (e.g., 15 mil Stego® Wrap) should be placed over the base course or gravel subdrain to reduce the migration of moisture vapor through the concrete slab. The vapor barrier should be installed in accordance with the manufacturer's instructions. Concrete should be placed directly on the vapor barrier. All slab concrete should have a water-cement ratio of 0.45 or less. Alternatively, two inches of spray insulation may be placed between the gravel layer and slab-on-grade.

Regardless of the type of vapor barrier used, moisture can wick up through a concrete slab. Excessive moisture transmission through a slab can cause adhesion loss, warping, and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor, and fungi growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. Commercial sealants, moisture retarding admixtures, fly ash, and a reduced water-to-cement ratio can be incorporated into the concrete to reduce slab permeability. To further reduce the chance of moisture transmission, a waterproofing consultant should be contacted.

Exterior slabs-on-grade such as sidewalks should be placed on a minimum 6-inch thick compacted aggregate base section to help reduce the potential for frost heave. Deleterious material should be removed from floor slab subgrades prior to concrete placement. For exterior slabs, the upper eight inches of native soil should be scarified, moisture conditioned, and compacted to at least 90 percent of the maximum dry density. We recommend a minimum concrete thickness of four inches. Where traffic loads are possible, we recommend a minimum concrete thickness of six inches. Concrete used for sidewalk construction should meet the durability requirements of Section 1904 of the 2022 CBC. The Exposure Class should be F2 unless the surface will be exposed to deicing chemicals, in which case the Exposure Class should be F3.

Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion, freeze-thaw cycles, or differential loading.

5.3.4 Retaining Wall Design Criteria

Retaining walls should be designed to resist lateral earth pressures exerted by retained soil plus additional lateral forces (i.e., surcharge loads) that will be applied to walls. Pressures exerted against retaining walls may be calculated by modeling soil as an equivalent fluid with unit weights presented in the following table. The equivalent fluid weights are for well-drained walls.

Table 5.3.4.1 – Equivalent Fluid Unit Weights*

Loading Condition	***Retained Cut or Compacted Fill (Level Backfill)***	***Retained Cut or Compacted Fill (Backfill Slopes up to 2H:1V)***	***Retained Cut or Compacted Fill (Backfill Slopes up to 1H:1V)***
At-Rest Pressure (pcf)	*45*	*65*	*75*
Active Pressure (pcf)	*30*	*50*	*65*
Passive Pressure (pcf)	*350*	*350*	*350*
Coefficient of Friction	*0.40*	*0.40*	*0.40*

**Equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. Passive pressures provided assume footings are founded in competent native soil or compacted and tested fill.*

The values presented in Table 5.3.4.1 assume that the retained soil will not exceed approximately ten feet in height and that no surcharge loads (e.g., footings, vehicles) are anticipated within a horizontal distance of approximately six feet from the face of the wall. Fifty percent of any uniform areal surcharge placed at the top of a restrained wall (at-rest condition) may be assumed to act as a uniform horizontal pressure over the entire height of the wall. This may be reduced to 30 percent for unrestrained walls (active condition). In addition, we can provide retaining wall and rockery wall design criteria for specific loading and backfill configurations, if requested.

The use of the tabulated active pressure unit weight requires that the wall design accommodate sufficient deflection for mobilization of the retained soil to occur. Typically, a wall yield of at least 0.1 percent of the wall height is sufficient to mobilize active conditions in granular soil (Caltrans Bridge Design Specifications, August 2004). If the walls are rigid or restrained to prevent rotation, at-rest conditions should be used for design.

We recommend including additional lateral loading (ΔP_{ae}) on retaining structures due to seismic accelerations when designing walls greater than six feet in height. The USGS Seismic Design Maps tool was used to establish seismic design parameters and provides an estimated peak ground acceleration (PGA) corresponding to the maximum considered earthquake (MCE_R) ground motion.

For an earthquake producing a design PGA of 0.576g and a horizontal seismic coefficient (k_h) equal to one-third the PGA, and following the Mononobe-Okabe procedure to evaluate seismic *loading on retaining walls, we recommend that the resulting additional lateral force applied to retaining structures with drained level backfill be estimated as $\Delta P_{ae}=4H^2$ (pounds per foot), where H is the height of the wall in feet. The additional seismic force may be assumed to be applied at a height of H/3 above the base of the wall. This seismic loading is for standard retaining walls with drained, level backfill conditions only. NV5 should be consulted to provide seismic loading values for more critical walls or walls with non-level or non-drained backfill conditions. The use of reduced factors of safety is often appropriate when reviewing overturning and sliding resistance during seismic events.*

Heavy compaction equipment or other loads should not be used in close proximity to retaining walls unless the wall is designed or braced to resist the additional lateral forces. If planned surface loads are closer to the top of the retaining wall than one-half of its height, NV5 should review the loads and loading configuration.

Retaining wall backfill should consist of granular material, nearly free of organic debris, with a liquid limit less than 40, a plasticity index less than 15, 100 percent passing the 8-inch sieve, and less than 30 percent passing the No. 200 sieve. Backfill should be uniformly moisture conditioned to within two percent of the optimum moisture content and compacted with appropriate compaction equipment to at least 90 percent of the maximum dry density. If the retaining wall backfill will support foundations or rigid pavements, the backfill should be compacted to at least 95 percent of the maximum dry density. An NV5 representative should review and provide specific backfill criteria for all retaining walls over 10 feet in height. Utilities that run through retaining wall backfill should allow for vertical movement where they pass through the wall.

Retaining wall design criteria presented in Table 5.3.4.1 assume that retaining walls are well-drained to reduce hydrostatic pressures. Back-of-wall drainage consisting of graded gravel drains and geosynthetic blankets should be installed to reduce hydrostatic pressures. Gravel drains should consist of at least 18 inches of open-graded, crushed rock placed directly behind the wall, wrapped in non-woven geotextile filter fabric such as Mirafi 140N or approved equivalent. Drains should have a minimum 4-inch diameter, perforated drain pipe placed at the base of the wall, inside the drain rock, with perforations placed down. The pipe should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent should also be placed against the back of the wall. Backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

We recommend that subsurface walls and slabs be treated to resist moisture migration. Moisture retarding material should consist of sheet membrane rubberized asphalt, polymer-modified asphalt, butyl rubber, or other approved material capable of bridging nonstructural cracks, applied in accordance with the manufacturers' recommendations. A manufactured water-stop and/or key should be placed at all cold joints. The project architect or contractor may wish to consult with a waterproofing expert regarding additional options for reducing moisture migration into living areas.

5.3.5 Pavement Sections

Based on our experience in the Tahoe-Truckee area, environmental factors, such as freeze-thaw cycles and thermal cracking will usually govern the life of asphalt concrete (AC) pavements. Thermal cracking of asphalt pavement allows more water to enter the pavement section, which promotes deterioration and increases maintenance costs. In addition, snow removal activities on site may result in heavy traffic loads. For these reasons, we recommend a minimum driveway/parking area pavement section of three inches of AC on six inches of aggregate base (AB). A minimum pavement section of 3 inches AC on 8 inches Class 2 AB should be used in roadways. Based on an assumed Resistance value for the onsite sand soil of at least 50, this pavement section should provide support of a traffic index of at least 6.0. We can provide additional pavement section recommendations, as needed.

We recommend that paving stones in non-traffic areas be supported by a minimum of four inches of Caltrans Class 2 AB. For light traffic areas, the AB section should be increased to at least six inches. An underlying concrete slab is not necessary for light traffic and non-traffic areas. Prior to placing aggregate base, the subgrade should be prepared in accordance with the recommendations provided below.

Due to seasonal saturation of the underlying AB and freeze-thaw cycles, some vertical movement of paving stones over time should be anticipated. This movement can likely be reduced by constructing a drainage layer beneath paving stone pavements. The drainage layer should consist of at least 4 inches of compacted clean angular gravel under the AB layer. The drainage layer should contain a minimum 4-inch diameter perforated pipe, sloped to drain water from beneath the pavement towards an infiltration facility. All open-graded gravel should be consolidated using vibratory compaction equipment. A minimum 4-ounce non-woven filter fabric such as Mirafi 140N or approved equivalent should be placed between the compacted gravel subdrain and aggregate base course.

The upper six inches of native soil should be compacted to at least of 95 percent of the maximum dry density prior to placing AB. AB should also be compacted to a minimum of 95 percent of the maximum dry density. Subgrade and AB dry densities should be evaluated by NV5. In addition to field density tests, the subgrade should be proof rolled under NV5's observation prior to AB placement. If temporary pavement is used during construction, we recommend preparation of the subgrade and AB as outlined above prior to construction of the temporary pavement.

To improve pavement performance and lifespan, we recommend promoting drainage of the pavement subgrade. Drainage can be accomplished through roadway layout and design, subdrains, and/or roadside ditches. An NV5 representative should evaluate pavement subgrade at the time of construction and provide location-specific recommendations for subdrains. Typical subdrains consist of a shallow trench with a minimum 4-inch diameter perforated pipe encased in open-graded gravel wrapped in filter fabric. Pavement subgrade should be graded and prepared such that water drains from beneath the pavement section to a properly designed infiltration facility. Subdrains may be used in conjunction with roadside ditches located on one or both sides of the roadway. Roadside ditches should be constructed *to a depth greater than the proposed pavement and subdrain section. Ditches should be rock-lined or vegetated to help reduce erosion and convey water to a properly designed infiltration facility.*

We recommend installing cut-off curbs where paved areas abut landscaped areas to reduce migration of irrigation water into subgrade soil or baserock, promoting asphalt failure. Cut-off curbs should be a minimum of 4-inches wide and extend through the aggregate base a minimum of four inches into subgrade soil.

Revised Limitations

The following paragraphs should replace all of the text in our Geotechnical Engineering Report for the project site.

Our services were performed consistent with our agreement with our client. We are not responsible for the impacts of changes in environmental standards, practices, or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others or the use of segregated portions of this letter. This letter is solely for the use of our client. Reliance on this letter by a third party is at the risk of that party.

If changes are made to the nature or design of the project as described in this letter, then the conclusions and recommendations presented in the letter should be reviewed by NV5 to assess the relevancy of our conclusions and recommendations. Additional field work and laboratory tests may be required to revise our recommendations. Costs to review project changes and perform additional field work and laboratory testing necessary to modify our recommendations are beyond the scope of services provided for this letter. Additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.

Analyses, conclusions, and recommendations presented in this letter are based on site conditions as they existed at the time we performed our subsurface exploration. We assumed that subsurface soil conditions encountered at the locations of our subsurface explorations are generally representative of subsurface conditions across the project site. Actual subsurface conditions at locations between and beyond our explorations may differ. If subsurface conditions encountered during construction are different than those described in this letter, we should be notified so that we can review and modify our recommendations as needed. Our scope of services did not include evaluating the project site for the presence of hazardous materials or petroleum products.

The elevation or depth to groundwater and soil moisture conditions underlying the project site may differ with time and location. The project site map shows approximate exploration locations as determined by pacing distances from identifiable site features. Therefore, exploration locations should not be relied upon as being exact.

The findings of this letter are valid as of the present date. Changes in the conditions of the property can occur with the passage of time. These changes may be due to natural processes or human activity, at the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or a broadening of *knowledge. Therefore, the recommendations presented in this letter should not be relied upon after a period of two years from the issue date without our review.*

Closing

We have prepared this letter for your exclusive use in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our services. No warranty, express or implied, is intended.

We appreciate the opportunity to provide assistance on this project. If you have any questions regarding this letter, please contact the undersigned.

Sincerely,
NV5

Prepared By:

Pamela J. Raynak, P.G.
Senior Geologist

Reviewed By:

Nicole C. Berry, P.E.
Project Engineer

Attachment: *Geotechnical Engineering Report for High Street Frontage Improvements, Truckee, California*, prepared by Holdrege & Kull dated April 7, 2023.

GEOTECHNICAL ENGINEERING REPORT
for
HIGH STREET FRONTAGE IMPROVEMENTS
Truckee, California

Prepared for:
McManus / Horst
c/o Truckee River Associates
11430 Deerfield Drive, Suite B
Truckee, California 96161

Prepared by:
Holdrege & Kull
15826 Donner Pass Road Suite 101
Truckee, California 96161

Project No. 40310-01
April 7, 2003

Project No. 40310-01
April 7, 2003

McManus / Horst
Truckee River Associates
11430 Deerfield Drive, Suite B
Truckee, CA 96161

Attn: Tom Watson

Reference: ***High Street Frontage Improvements***
Truckee, California

Subject: ***Geotechnical Engineering Report***

This report presents the results of our geotechnical engineering investigation for the proposed High Street Frontage Improvements to be located in Truckee, California. The proposed development will involve three projects within a one block area between Jibboom Street and High Street. The projects involve construction of a two-story retail building along Jibboom Street, construction of nine single family homes along High Street and relocation of an existing motel at the east end of the block and adding a kitchen at the back of the reconstructed building. Appurtenant construction for all three projects will include concrete retaining walls, asphalt concrete pavements, concrete curb, gutter and sidewalk, underground utilities and landscaping. In addition, a widening of High Street is planned that will include steep cut slopes with rock slope protection.

Based on our findings, it is our professional opinion that the site is suitable for the proposed development using conventional earthwork grading and foundation construction techniques. No highly compressible or potentially expansive soil conditions were encountered during our subsurface exploration. Specific recommendations regarding the geotechnical aspects of project design and construction are presented in the following report.

During construction, a representative of Holdrege & Kull should observe site preparation, foundation preparation, fill placement, rockery wall construction and subdrain installation to check for compliance with the recommendations provided in this report.

Please contact us if you have any questions regarding this report or if we can be of additional service.

Sincerely,
HOLDREGE & KULL

Bryan J. Hanna
Staff Engineer

John K. Hudson, P.E., C.E.G.
Senior Engineer

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APPENDICES

1 INTRODUCTION

Holdrege & Kull (H&K) is pleased to present the results of our geotechnical engineering investigation for the proposed High Street Frontage Improvements to be located between High Street and Jibboom Street in Truckee, California. We performed our investigation in general accordance with our January 31, 2003 proposal for the project. A copy of the proposal is included as Appendix A of this report. For your review, Appendix B contains a document prepared by ASFE entitled *Important Information About Your Geotechnical Engineering Report*. This document summarizes the general limitations, responsibilities and use of geotechnical engineering reports.

1.1 Site Description

The project site consists of four parcels totaling approximately 1.71 acres of land between Jibboom Street and High Street in Truckee, California. The approximate site location is shown on the Vicinity Map, Plate 1. Both Jibboom Street and High Street provide paved access to the project site, which is bounded by High Street to the north and northwest. Jibboom Street bounds the east portion of the site to the south and commercially developed parcels along Jibboom Street bound the west portion of the site to the south. Spring Street and Bridge Street bound the site to the west and east respectively. An unimproved parking lot currently exists on the south portion of the project site adjacent to Jibboom Street. An existing motel with two ancillary structures is located at the east end of the block. At the time of our site reconnaissance, vegetation consisted of predominantly Bitter Brush and Mule's Ear with some conifer trees. Numerous large boulders were observed on the project site.

In general, site topography sloped from the northwest portion of the site (High Street) down to the southeast. The northeast portion of the site, was moderately sloped. A large cut slope was located in the north and northwest portion of the site, resulting in a very steep slope between High Street and the existing parking lot on Jibboom Street. A total relief of approximately 47 feet was present across the project site, with a total relief ranging between approximately 10 to 20 feet at any one proposed building location. No surface water was observed during our site visit; however, it appeared that drainage across most of the site consisted of overland flow. After surface water would flow off site, it would be directed into various storm drain drop inlets along Jibboom Street.

1.2 Project Description

The proposed development will involve three projects on four parcels of land within a one block area between Jibboom Street and High Street. The McManus/Horst "A" Project will involve construction of a long, narrow two-story retail building. The project will include a cast-in-place concrete retaining wall along the back (north) side of the parcel and a mini-storage building. We anticipate that the retail building will be wood-frame with a slab-on-grade floor. Earthwork cuts for the retaining wall are expected to be up to 10 feet in vertical extent. Rock slope protection may be necessary above the concrete wall. The second project will include construction of nine single family homes along High Street. We understand the houses will be two-story, wood frame structures with raised wood and/or slab-on-grade floors. The third project will involve relocation of the existing motel at the east end of the block and adding a kitchen at the back of the reconstructed building. The kitchen will likely require excavations up to 12 feet with a concrete retaining wall. Appurtenant construction for all three projects will include asphalt concrete pavements, concrete curb, gutter and sidewalk, underground utilities and landscaping. In addition, High Street is planned to be widened with cut slopes of 1:1 and fill slopes of 1.5:1 (H:V). Rock slope protection (RSP) is planned for the constructed slopes.

Structural loads were not available at the time this report was prepared. Therefore, we estimate that loads for all planned structures will not exceed 80 kips at isolated columns and 4 kips per linear foot along continuous wall foundations for long-term loading conditions. Earthwork cuts and fills for building pad grading of the project are expected to be up to approximately 5 feet in vertical extent with the exception of the kitchen.

1.3 Purpose

The purpose of our investigation was to explore and evaluate the subsurface conditions at the project site, and to provide our geotechnical engineering recommendations for project design and construction. Our findings are based on our site reconnaissance and our experience in the project area.

1.4 Scope of Services

To prepare this report we performed the following scope of services:

- A site reconnaissance, literature review and subsurface exploration by logging and sampling soil exposed in an existing cut slope;
- A review of available subsurface information contained in our files pertinent to the proposed construction and project site;
- Engineering analyses to develop geotechnical engineering recommendations for project design and construction; and
- Preparation of this report which includes:
    - A discussion of the general soil and groundwater conditions at the project site, with emphasis on how the conditions are expected to affect the proposed construction;
    - Recommendations for earthwork construction, including site preparation, the reuse of existing near surface soils as structural or non-structural fill, and structural fill placement;
    - Recommendations for trench excavation and backfill;
    - Recommendations for permanent cut and fill slopes;

- Recommendations for conventional shallow spread foundation design including soil bearing values, minimum footing depth, resistance to lateral loads and estimated settlements, and Uniform Building Code Soil Profile Type for use in structural design.
- Lateral earth pressures for retaining structures;
- Recommendations for subsurface and surface drainage;
- Recommendations for subgrade preparation for slab-on-grade concrete, paving stone and asphalt pavement;
- Asphalt concrete pavement design recommendations; and
- Rockery wall recommendations.

1.5 Authorization

Authorization to proceed with our work on this project was provided by Tom Watson on February 10, 2003 in the form of a signed Terms and Conditions agreement.

1.6 Reference

The following information was provided to H&K in the course of this investigation.

- A site plan sheet entitled "Preliminary Plan, High Street Frontage Improvements, Truckee, California", prepared by Acumen Engineering, dated January 17, 2003.

In addition, the following published reference was reviewed during preparation of this report.

- "Regional Geologic Map of the Chico Quadrangle", prepared by Saucedo and Wagner, 1992.

2 SUBSURFACE EXPLORATION

We performed our subsurface exploration to characterize typical subsurface conditions at the site. Our exploration included literature review and field exploration as described below.

2.1 Literature Review

We reviewed the Geologic Map of the Chico Quadrangle, California. The geological map indicates that the project site is underlain by Pleistocene age glacial deposits.

2.2 Field Exploration

The subsurface conditions at the site were explored on February 26, 2003 by hand excavating the soil exposed in the existing cut slopes at two locations between High Street and the parking lot on Jibboom Street. Our staff engineer hand excavated loose slough material present along the cut slope in 5 locations to reveal the underlying native soil. Excavation locations were selected based on locations of proposed improvements and location of the steepest existing cuts, which more readily revealed native subsurface soil conditions. The approximate locations of the hand excavations are shown in the Test Pit Location Plan, Plate 2.

Our staff engineer logged the soil conditions exposed in the hand excavations, visually classified the soil, and collected bulk soil samples for laboratory testing. Soil samples were packaged and sealed in the field to reduce moisture loss.

2.3 Subsurface Soil Conditions

Near surface soil along High Street, at the top of the cut slope and in the north portion (undisturbed areas) of the site consisted of 10 to 12 inches of loose silty Sand (SM) containing organic material and cobbles up to 12 inches in diameter. Several granitic and volcanic boulders up to 14 feet in diameter were observed on the soil surface across the entire portion of the currently unimproved project site. In the more vegetated areas of the site, up to 6 inches of pine needles and duff material exist on the surface soil. Along High Street, fill was observed as evidenced by the presence of approximately ¾-inch aggregate, chunks of asphalt concrete up to 6 inches in diameter and other debris. This fill material appears to be up to 3 feet or more in some locations.

Underlying the loose silty sand we encountered medium dense to dense silty Sand (SM) with gravel and cobbles up to 10 inches to a depth of approximately 8 feet below ground surface. At approximately 2½ feet below ground surface in Test Pit 5, friable granitic boulders were encountered. This material can be expected to be excavated as a well-graded Gravel (GW-GM) with silt and sand. We anticipate numerous granitic and volcanic boulders up to several feet in diameter will be encountered. Depths described above are below the native ground surface. In the area of the existing cut face, the dense sand and boulders can be expected to be encountered nearer the existing cut surface.

More detailed descriptions of the subsurface conditions observed are presented in our trench logs in Appendix C.

2.4 Groundwater Conditions

We did not observe free groundwater during our subsurface exploration. However, fluctuations in soil moisture content should be anticipated depending on precipitation, irrigation, runoff conditions and other factors. Based on our experience in the project area, seasonal saturation of near-surface soil should be anticipated. Near-surface water may become perched on denser soil layers or rock encountered at varying depths.

3 LABORATORY TESTING

We performed laboratory tests on bulk soil samples collected from our subsurface exploration to help evaluate their engineering properties. Laboratory test results are included in Appendix C. The following laboratory tests were performed:

- Atterberg Limits / Plasticity (ASTM D4318)
- Sieve Analysis (ASTM D422)
- Direct Shear (ASTM D3080)

Laboratory test results are included as Appendix D.

4 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results and our experience in the area. These conclusions may change if additional information becomes available.

- From a geotechnical engineering standpoint, no severe soil or groundwater constraints were observed which would preclude construction as planned. Site soil consists of silty Sand (SM) with gravel, cobbles and boulders. Site soil should provide adequate support for the planned improvements using conventional shallow spread footings. No highly plastic, compressible or potentially expansive soil was encountered during our subsurface exploration.\

- No existing landslides or other slope failures were observed at the project site. However, due to steep slopes at the site, minor modifications in the schedule and/or approach to site grading, and possibly to planned structures may be required during site development. We anticipate that retaining walls at the bottom of the slope will be constructed prior to foundation construction at the upper portion of the site.

- Based on our observations and experience in the site area, we anticipate that site grading can be performed with conventional earthwork equipment. Excavations for footing and utility trenches will likely require equipment such as a track-mounted excavator. Deeper or confined trenches may be difficult due to numerous boulders and dense soil conditions. Site soil is generally suitable for reuse as structural fill, but processing to remove oversize material will likely be necessary.

- Although groundwater was not encountered during our subsurface investigation, near-surface soil will likely become seasonally saturated. Surface flows and near surface groundwater may adversely affect the long term performance of structures and pavements if inadequate drainage exists under pavements or behind retaining walls and footings. We have provided recommendations in this report for site surface and subsurface drainage, back of retaining wall drains, crawl space drainage and subsurface pavement drainage.

Specific recommendations for project design and construction are presented in the following sections of this report.

5 RECOMMENDATIONS

The following geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of our laboratory tests, engineering analysis and our experience in the area.

5.1 Earthwork

5.1.1 Clearing and Grubbing

Areas proposed for fill placement, pavement or slab-on-grade construction, and building areas should be cleared and grubbed of vegetation and other deleterious materials as described below.

- Prior to site grading, surface vegetation, organic soil and any debris should be stripped and disposed of outside the construction limits. Based on our subsurface exploration, we anticipate stripping depths of 4 to 6 inches and an additional 6 inches of pine needles in localized areas along High Street. We anticipate stripping depths of 0 to 4 inches in the area of the existing cut slope. Deeper stripping and more extensive grubbing of organic soil, tree roots, etc. may be required in localized areas. Tree root balls should be removed and the resulting voids backfilled with adequately compacted backfill soil.

- Organic surface soil may be stockpiled for reuse in landscape areas to promote revegetation, but is not suitable for use as structural fill.

5.1.2 Preparation for Fill Placement

Where fill placement is planned, we recommend near-surface soil exposed by site clearing and grubbing be prepared as described below.

- The near-surface soil should be scarified to a depth of 12 inches below existing ground surface or to competent material and then uniformly moisture conditioned to within 2% of optimum moisture content as determined by ASTM D1557.

- Areas to receive fill should be compacted with appropriate compaction equipment and proof rolled with a loaded, tandem-axle truck under the observation of a representative of H&K. Any areas that exhibit pumping or rutting should be over-excavated and replaced with compacted fill placed according to the recommendations discussed below.

5.1.3 Fill Placement

Fill should be placed according to the following recommendations.

- Material used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. In general, near-surface, on-site soil similar to those encountered in our subsurface exploration may be used in structural fills provided all oversized material is removed prior to placement and compaction. Rock used in fill should be broken into pieces no larger than 8 inches in diameter. Rocks larger than 8 inches are considered oversized material and should be hauled offsite, used as rock slope protection or placed in landscape areas.

- Imported fill material should be predominantly granular, non-expansive and free of deleterious or organic material. Import material that is proposed for use on site should be submitted to H&K for approval and laboratory analysis at least 72 hours prior to import.

- If site grading is performed during or after periods of wet weather, near-surface site soil may be significantly above optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact fill materials to the recommended compaction criteria. Fill material may require drying to facilitate placement and compaction, particularly during or following the wet season or spring snow melt. No fill material shall be placed, spread or rolled while it is frozen or thawing, or during unfavorable weather conditions.

- Fill should be uniformly moisture conditioned to within ±2% of optimum moisture content and placed in maximum 8-inch thick, loose lifts (layers) prior to compacting.

- Soil used for structural fill (soil supporting structures, pavements, slabs-on-grade or other improvements, including foundation backfill) should be compacted to at least 90% relative compaction*. The upper 8 inches of fill in paved areas should be compacted to at least 95% relative compaction. Fill placed in non-structural areas should be compacted to at least 85% relative compaction.

- The moisture content and relative compaction of fill should be evaluated by our firm at regular intervals during fill placement.

5.1.4 Permanent Slope Grading

Stability of cut and fill slopes involves two separate aspects. The first concerns true slope stability related to mass wasting and landslides. True slope stability is dependent upon shear strength, unit weight, moisture content of soil and slope angle.

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* Wherever referenced in this report, relative compaction should be determined by comparing field dry density and moisture content to the maximum dry density and optimum moisture content determination conducted in accordance with ASTM D1557 Test Method.

Cut and fill slopes at the subject site should be stable from a slope stability standpoint at inclinations up to 1:1 for heights up to about 10 feet.

The second aspect of slope stability involves erosion potential and is dependent on numerous factors including grain size distribution, cohesion, moisture content, slope inclination, slope length and velocity of water or wind on the slope face. Steep constructed slopes reduce the amount of disturbed area and result in shorter slope lengths. Slope performance at this site will be primarily affected by surface runoff and erosion. Slopes may be constructed at inclinations up to 1:1 for heights up to about 10 feet provided they are armored with rock slope protection (RSP).

Permanent slope grading (cut/fill slopes) should be constructed according to the following recommendations.

- Fill should be placed in horizontal lifts to the lines and grades shown on the project grading plans. Slopes should be constructed by overbuilding the slope face and then cutting it back to the design slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

- Equipment width keyways and benches should be provided where fill is placed on side-slopes with gradients steeper than 5 horizontal to 1 vertical. Benching must extend through loose surface soil into suitable material, and be performed at intervals such that no loose soil is left beneath the fill. Keyways and benches should be observed by H&K prior to fill placement. All fill placed on slopes, particularly foundation backfill, should be compacted in accordance with recommendations provided in *Section 5.1.3 - Fill Placement* of this report.

- All disturbed slopes should be stabilized with a combination of vegetation and rock slope protection (RSP). We recommend RSP for all slopes steeper than 2 horizontal to 1 vertical. We recommend 8-inch to 12-inch diameter cobbles as RSP for slopes up to 8 feet in vertical extent, and 12-inch to 24-inch rocks for slopes up to 12 feet in vertical extent. If RSP is used to stabilize slopes greater than 12 feet in vertical extent or if steeper slopes are necessary, H&K should be contacted to provide additional recommendations.

- RSP should be placed in accordance with Caltrans Standard Specifications Method A Placement. Placing of rocks by dumping should not be permitted. Rocks should decrease in size from the bottom to the top of the slope. Rocks should have a specific gravity of at least 2.5 and a percentage of wear (per ASTM C535) of not more than 45. Rounded, flat or needle shaped rocks will not be accepted unless the thickness of the individual pieces is greater than ⅓ the length. A footing trench should be excavated to at least 12 inches below the lowest adjacent site grade along the toe of the slope. Bearing on smaller rocks that may be used for chinking voids will not be acceptable.

- In the event seepage is encountered emanating from a cut slope, we recommend a well-graded filter layer consisting of ¾ to 2½-inch crushed gravel be placed behind the RSP to help reduce the loss of soil through the face of the RSP. The filter layer should be at least 12 inches thick.

- The upper 2 to 5 feet of cut slopes should be rounded into the existing terrain above the slope to remove loose material and produce a contoured transition from cut face to natural ground. Scaling to remove unstable cobbles and boulders may be necessary. Fill slopes should be compacted as recommended for the placement of structural fill. The top 4 to 8 inches may be scarified in planter areas to help promote vegetation.

5.1.5 Temporary Unconfined Excavations

We understand that deep cuts of up to about 12 or 15 feet will be necessary to construct proposed retaining walls. The use of steepened, temporary cut slopes will be needed to construct these structures. The following criteria have been developed and may be used for construction of temporary cut slopes adjacent to the proposed structures.

Table 1. Temporary Slopes

Temporary Slope Inclination	
(Horizontal to Vertical)	Maximum Height (Feet)
0.5 : 1	16
0.75 : 1	20

These layback requirements may require modifications where loose or cohesionless soils are encountered. We anticipate a layered excavation, with upper slopes at a flatter inclination than at the bottom. The contractor is ultimately responsible for the safety of workers and should observe federal and local OSHA requirements for excavation shoring and safety. Due to the granular nature of the surface soils, some raveling of temporary cut slopes should be anticipated. Scaling to remove unstable cobbles and boulders may be necessary. During wet weather, runoff water should be prevented from entering excavations. Heavy construction equipment, building materials, excavated soil and vehicle traffic should not be allowed within a distance of one-third the slope height of any excavation.

5.1.6 Trench Excavations and Trench Backfill

Based on our experience in the site area, excavations for footing and utility trenches will likely require a track-mounted excavator. Boulders may be encountered and make deep or confined excavations difficult. An excavator with a "thumb" attachment may increase ease of boulder removal at the site.

Underground utility trenches should be excavated and backfilled in accordance with the following recommendations.

- Soil used as trench backfill should be non-expansive and should not contain rocks greater than 4 inches in maximum dimension.

- Trench backfill should be uniformly moisture conditioned to within ±2% of optimum moisture content and be placed in 8-inch thick loose lifts prior to compaction.

- Unless otherwise specified by the applicable local utility district, pipe bedding and trench backfill should be compacted to at least 90% relative compaction. Jetting and flooding should not be permitted.

- Trench backfill placed within 8 inches of subgrade in structural improvements (e.g., buildings, pavements, slabs-on-grade, etc.) should be compacted to at least 95% relative compaction. Poor compaction in utility trench backfill may cause excessive settlements resulting in damage to the pavement section or other overlying improvements.

- The moisture content and relative compaction of trench backfill should be evaluated by our firm at regular intervals during underground utility construction.

- Due to the granular nature of the on site soil, we expect that some caving and sloughing of utility trenches will occur. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 5 feet below ground surface to be shored with bracing equipment or sloped back prior to entry.

- Although we did not observe groundwater during our subsurface exploration, shallow subsurface seepage may be encountered in trench excavations, particularly if utility trenches are excavated during the spring or early summer. The earthwork contractor may need to employ dewatering methods as discussed in the *Construction Dewatering* section below to excavate, place and compact trench backfill materials.

5.1.7 Construction Dewatering

We did not encounter groundwater during our subsurface exploration. However, if grading in performed during or immediately following the wet season or spring snowmelt, seepage may be encountered in excavations. If groundwater or saturated soil conditions are encountered during grading, we should observe those conditions and provide site specific subsurface drainage recommendations. The following recommendations are preliminary and are not based on a groundwater flow analysis.

- We anticipate that dewatering of excavations can be performed by gravity or by constructing sumps to depths below the excavation and removing water with pumps. To maintain stability of the excavation when placing and compacting the trench backfill, groundwater levels should be drawn down a minimum of 2 feet below the lowest point of excavation.

- If seepage is encountered during trench excavation, it may be necessary to remove underlying saturated soil and replace it with free draining, open graded crushed rock. Soil backfill may be placed after backfilling with drain rock to an elevation higher than encountered groundwater.

5.1.8 Surface Water and Near-Surface Groundwater Drainage

Proper surface water and near-surface groundwater drainage is important for the performance of project structures and pavements. We recommend the following measures to help reduce potential drainage problems:

- Final grades at the site should be planned so that surface water drains away from all foundations at a minimum 2% slope for a minimum distance of 5 feet. Positive drainage gradients should carry surface water off site or to a properly designed infiltration gallery.

- Foundation drains are recommended on the uphill side of all continuous cross-slope wall foundations.

- Compact backfill soil placed adjacent to building foundations such that water is not allowed to pond or infiltrate.

- Direct roof downspouts to a closed collector pipe, which discharges flow to positive drainage.

- Roof drip-lines should be protected from erosion with a gravel layer and riprap.

- Paved areas should be sloped and drainage gradients maintained to carry all surface water to properly designed infiltration areas.

- Interior grades in the crawl space beneath proposed structures should be higher than the exterior ground surface. The crawl space should be sloped to collect and divert water to drains that exit under or through the foundation (positive crawl space drainage). We recommend placing 3 to 4 inches of clean gravel over the crawl space grade.

- Provide adequate ventilation in all crawl space areas to promote drying.

- If open graded gravel or other highly permeable material is used for underground utilities, the trench should slope away from the structure or the potential flow path along the trench backfill should be plugged at the exterior of the foundation.

5.1.9 Erosion Control

Based on our onsite observations and experience in the area, the predominantly granular onsite soil will be moderately susceptible to erosion. Best management practices (BMPs) should be incorporated into the design and construction of this project in accordance with The California Regional Water Quality Control Board, Lahontan Region, Best Management Practices Plan.

The earthwork operations at the project should be complete in one construction season. All permanent erosion control should be in place prior to October 15. All areas disturbed by construction should be protected against erosion resulting from direct rain impact and melting snow until permanent vegetation can be established.

Erosion and sediment control measures can be categorized as temporary or permanent. Temporary measures are to be installed to provide short-term protection until the permanent measures are installed and effective. Temporary erosion and sediment control includes ground surface treatments and installation of sediment barriers and detention facilities down gradient from all disturbed areas during construction and after construction until permanent measures become effective. Typical temporary measures include properly installed silt fences, straw bales, sediment logs, water bars, detention basins, covering of exposed soil, channel linings and inlet protection.

These structures are designed to slow down water flow and intercept suspended sediment to prevent sediment discharge from the construction area while allowing runoff to continue down gradient. The requirement to install a sediment barrier is dependent on the following factors: 1) slope angle, 2) slope length, and 3) soil type (texture and coarse fragment content). Sediment barriers should be installed down gradient and at the edges of all disturbed areas and around topsoil and spoil piles where necessary. Sediment barriers should be placed as needed on slope contours, within small drainages and gently sloping swales. The slope length above each barrier should not exceed 100 feet. Following completion of construction and planting/seeding, temporary erosion control measures may be left in place, possibly for a complete growing season. Temporary erosion control measures require regular inspection and maintenance.

Permanent erosion and sediment control measures may include rock slope protection (RSP), rock lined ditches and inlet/outlet protection, rock energy dissipaters, infiltration/detention basins and vegetation. Existing vegetation should be protected and undisturbed where possible. Revegetation should consist of native brush and grass species or hybrid plants that will grow well at the subject site. All areas disturbed by construction should be revegetated or protected with permanent erosion control measures.

Surface water drainage should not be directed to flow over unprotected slopes. Interceptor (brow) ditches should be considered at the top of slopes in order to collect and divert runoff which otherwise would flow over the slope face. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.

5.2 *Structural Improvement Design Criteria*

5.2.1 Foundations

Our opinion is that shallow spread foundations are suitable for support of the proposed structures. The following paragraphs discuss foundation design parameters and construction recommendations.

- Exterior foundations should be embedded a minimum of 24 inches below the lowest adjacent exterior finish grade for frost protection and confinement. The bottom of interior footings should be at least 12 inches below lowest adjacent finish grade for confinement. Foundations constructed on steeper slopes may require deeper embedment to provide adequate lateral resistance on the downhill side of the foundation. We anticipate the footings for the single family residences along High Street will require deeper embedment. We recommend the exterior and interior footings for these structures be embedded beyond the 24 inches to provide a minimum horizontal distance of 4 feet between the toe of the footing and the ground surface as shown in Figure 1, below. The Geotechnical Engineer should review all foundation excavations during or immediately after excavation. Reinforcing steel requirements for foundations should be provided by the design engineer.

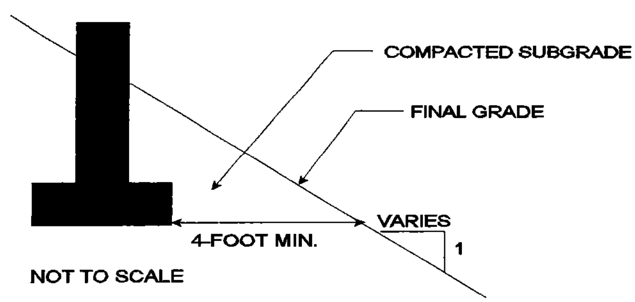


Figure 1. Minimum lateral distance on steep slopes

- Foundations founded in competent, previously undisturbed native soil or compacted structural fill may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf) for dead loads plus long-term live loads. An increase of 600 psf per foot of additional embedment (beyond the minimum 24 inches) may also be used, up to a limiting value of 4,200 psf. The allowable bearing pressure may be increased by one-third for total loading conditions, including wind and seismic forces. The allowable bearing pressure is a net value; therefore, the weight of the foundation that extends below grade and backfill may be neglected when computing dead loads.

- Resistance to lateral loads may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil and by passive soil pressure against the sides of the foundations. Lateral resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at the ground surface to a maximum of 350d psf, where d equals the depth of the foundation in feet. Due to potential variability of soil consistency at finish grade, potential surface soil desiccation and disturbance, we recommend the upper 6 inches of soil be neglected when estimating lateral resistance. A coefficient of friction of 0.4 may be used between poured-in-place concrete foundations and underlying native soil.

- Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on anticipated foundation dimensions and loads, we estimate that total post-construction settlement of footings designed and constructed in accordance with the preceding recommendations will be on the order of ¾-inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than ¼-inch, provided footings are founded on similar materials (e.g., all on native soil). Differential settlement between adjacent footings founded on dissimilar materials (e.g., one footing on structural fill and one footing on rock) may approach the maximum, anticipated total settlement. Settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads.

- Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. Any loose soil in the bottom of footing excavations should be recompacted to at least 90% relative compaction or removed to expose firm, unyielding material. Footing excavations should be moistened prior to placing concrete to reduce risk of problems caused by wicking of moisture from curing concrete. All footing excavations should be observed by a representative of H&K prior to placing steel or concrete.

5.2.2 Seismic Design Criteria

The site is located in UBC Seismic Zone 3 of the 1997 Uniform Building Code (UBC) Seismic Zone Map. Therefore, structural improvements should be designed using a seismic zone factor of Z=0.30 (UBC Table 16-I). We recommend using the S_c Soil Profile Type (dense soil), (UBC Table No. 16-J) to evaluate seismic loads.

5.2.3 Concrete Slab-on-Grade Construction

Concrete slab-on-grade floors may be used in conjuction with perimeter concrete footings. We make the following recommendations for the design and construction of slabs-on-grade.

- Prior to constructing concrete slabs, the upper 8 inches of slab subgrade should be scarified, uniformly moisture conditioned to within ±2% of optimum moisture content, and uniformly compacted to at least 90% relative compaction. Scarification and recompaction may not be required if floor slabs are placed directly on undisturbed compacted structural fill.

- All concrete slabs should have a minimum thickness of 4 inches. Slab thickness and structural reinforcing requirements within the slab should be determined by the design engineer.

- At least 4 inches of Class 2 aggregate base should be placed beneath slab-on-grade floors to provide uniform support. The aggregate base should be compacted to at least 95% relative compaction. The slab subgrade should be protected against drying until concrete placement. All slabs should be confined by perimeter foundations or at least a 2-foot wide compacted fill shoulder. If groundwater is encountered in slab areas, subsurface drains should be constructed.

- In slab-on-grade areas where moisture sensitive floor coverings are planned, a vapor retarder (e.g. 10-mil thick polyethylene) should be placed over the base course to reduce the migration of moisture vapor through the concrete slabs. The vapor retarder should be protected by two to three inches of fine, moist sand placed above the vapor retarder. The sand cover will provide protection for the retarder and will promote uniform curing of the concrete slab. The sand cover should be moistened (not wet) and tamped prior to slab placement.

- Exterior slabs-on-grade such as sidewalks may be placed directly on compacted structural fill or scarified and recompacted native soil without the use of an aggregate baserock section. Deleterious material should be removed from slab subgrades prior to concrete placement. For exterior slabs, the native soil should be ripped, moisture conditioned and recompacted to an 8-inch depth.

- Exposed concrete slabs should be moisture cured for at least seven days after placement.

- Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion, freeze-thaw cycles, or differential loading.

5.2.4 Retaining Walls

Retaining walls should be designed to resist the lateral earth pressure exerted by the retained, compacted backfill plus any additional lateral forces that will be applied to the walls. The pressure exerted on retaining walls depends on the slope inclination above the wall. The following active and passive pressures are for well drained walls retaining native soil. If import soil is used for fill or backfill, we should review our recommendations. Pressures exerted against retaining walls may be calculated by modeling soil as an equivalent fluid with the unit weights presented in Table 2.

Table 2. Equivalent unit weights for retaining wall design

Equivalent Unit Weights			
Loading Condition	Retained Cut or Compacted Fill		
	Horizontal	Slopes up to 2(H):1(V)	Slopes up to 1:1
Active Pressure (pcf)	30	50	65
Passive Pressure (pcf)	350	350	350
At-Rest Pressure (pcf)	45	65	75
Coefficient of Friction	0.45	0.45	0.45

Note: The equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. Passive pressure values provided assume footings are founded in competent native soil or compacted and tested fill.

The at-rest earth pressure is applicable for braced walls that are restrained at the top. Fifty percent of any uniform area surcharge placed at the top of a restrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall. Where rotational movement is possible, the active earth pressure applies. Thirty percent of any uniform surcharge placed at the top of a non-restrained wall may be assumed to act as a uniform horizontal pressure over the entire height of the wall.

The tabulated values in Table 2 assume that the retained soil will not exceed approximately 10 feet in height and that no surcharge loads (e.g., footings, vehicles) are anticipated within a horizontal distance of approximately 8 feet from the face of the wall. If additional surcharge loads are anticipated, we should review the proposed loading configuration to provide loading-specific design criteria. In addition, we can provide retaining wall and rockery wall design criteria for specific loading and backfill configurations, if requested. Additional recommendations for design and construction of retaining walls are listed below.

- Retaining wall design criteria presented in Table 2 assume that retaining walls are well drained to reduce hydrostatic pressures. A drainage blanket should be installed to reduce additional lateral forces and reduce saturation of backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.

- Rock drains should consist of a minimum 12 inches of open-graded crushed rock, and placed directly behind the wall, wrapped in non-woven geotextile filter fabric such as Amoco 4545™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40 PVC pipe placed at the base of the wall, inside the drain rock, with perforations placed down. The PVC pipe should be sloped so that water is directed away from the wall by gravity. If a rock drain is used and is wrapped with a non-woven geotextile, backfill must be adequately compacted for the filter fabric to function appropriately. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the rock drain, provided that water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

- Additional lateral loading on retaining structures due to seismic accelerations may be considered at the designer's option. For an earthquake producing a design horizontal acceleration of 0.3g, we recommend that the resulting additional lateral force applied to unrestrained (cantilevered) retaining structures on site be estimated as $P_{ae} = 14H^2$ pounds, where H is the height of the wall in feet. The additional seismic force may be assumed to be applied at a height of 0.6H above the base of thewall.

- Where retaining walls will enclose useable interior space or floors below grade, the wall should be waterproofed. Waterproofing material should consist of sheet membrane rubberized asphalt, polymer-modified asphalt, butyl rubber, or other approved material capable of bridging nonstructural cracks. Joints in the membrane should be lapped and sealed in accordance with the manufacturer's recommendations. Extra attention should be paid to concrete cold joints between the wall and footing. A manufactured water-stop or key should be placed at all cold joints.

- Heavy compaction equipment or other loads should not be allowed within a distance of ½ the wall height to the wall. This could result in lateral pressures higher than those recommended above, unless planned for in the structural design.

5.2.5 Asphalt Concrete and Paving Stone Pavement

Site soils should provide good support for on-site and roadway asphalt concrete (AC) pavement. Based on our experience in the Tahoe Truckee area, environmental aspects, such as freeze-thaw cycles and thermal cracking will probably govern the life of AC pavements. Thermal cracking of the asphalt pavement allows more water to enter the pavement section, which promotes deterioration and increases maintenance costs. In addition, it is likely that snow removal equipment will operate on the pavement section and result in heavy traffic loads. Based on the anticipated traffic, soil and environmental conditions at the site, we recommend a minimum pavement section of 3 inches AC on 6 inches Class 2 aggregate base (AB) for driveway and parking areas. A minimum pavement section of 3 inches AC on 8 inches Class 2 AB should be used in roadways. Based on an assumed R-value for the on-site sand soil of at least 50, this pavement section should support a traffic index of at least 6.0.

The aggregate base should be uniformly moisture conditioned to near optimum and compacted to at least 95% relative compaction. The material type, placement and compaction for base materials and asphalt concrete should conform to the Standard Specification for the State of California (Caltrans).

Paving stone pavements may be used for exterior patios and walkways. Paving stones should be placed in accordance with the manufacturer's recommendations on a minimum of 6 inches aggregate base with an 18-inch deep perimeter foundation. Subdrains should be constructed under all paving stone pavements.

It should be noted that the subgrade soil is likely to be prone to frost action during the winter and saturation during the wet spring months. The primary impact of frost action and subgrade saturation is the loss of subgrade and aggregate base strength. Pavement life and performance will be increased if efforts are made to reduce accumulation of excessive moisture in the subgrade soils. Consideration should be given to the installation of sub-drains below the subgrade, which allow drainage of the pavement section.

5.3 Plan Review and Construction Monitoring

Construction monitoring includes review of plans and specifications and observation of onsite activities during construction. H&K should review final grading and foundation plans prior to construction to evaluate whether our recommendations have been implemented and to provide additional and/or modified recommendations, if necessary. We also recommend that our firm be retained to perform construction monitoring and testing services during site grading, foundation, retaining wall, underground utility and road construction to observe subsurface conditions with respect to our engineering recommendations.

6 LIMITATIONS

Our professional services were performed consistent with the generally accepted geotechnical engineering principals employed in the site area at the time the report was prepared. This warranty is in lieu of all other warranties, either expressed or implied.

Our services were performed consistent with our agreement with our client. We are not responsible for the impacts of changes in environmental standards, practices or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client. Reliance on this report by a third party is at the risk of that party.

- If changes are made to the nature of design of the project as described in this report, then our conclusions and recommendations presented in this report should be reviewed by H&K to review our conclusions and recommendations. Additional field work and laboratory tests may be required to revise our recommendations. Costs to review project changes, perform additional field work and laboratory testing necessary to modify our recommendations are beyond the scope of services provided for this report. Additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.

- Analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our subsurface exploration. We have assumed that subsurface soil conditions encountered at the location of our subsurface exploration are generally representative of subsurface conditions across the project site. Actual subsurface conditions at locations between and beyond our subsurface exploration locations may differ. If subsurface conditions encountered during construction are different than those described in this report, we should be notified so that we can review and modify our recommendations as needed.

- The elevation or depth to groundwater and soil moisture conditions underlying the project site may differ with time and location.

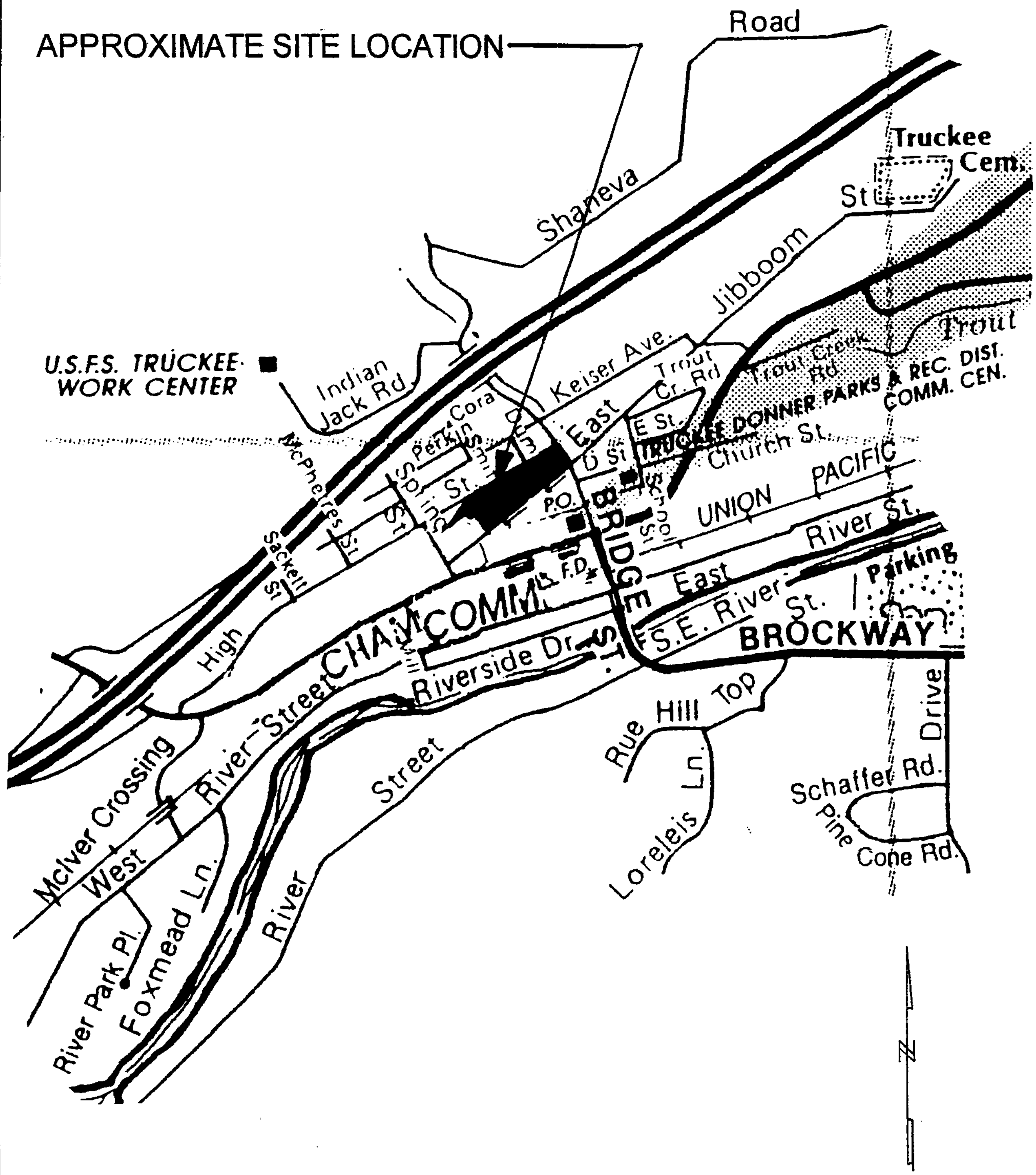
- The project site map shows approximate subsurface exploration locations as determined by pacing distances from identifiable site features. Therefore, exploration locations should not be relied upon as being exact.

- Our scope of services did not include evaluating the project site for the presence of hazardous materials or petroleum products. Although we did not observe evidence of hazardous materials or petroleum products at the time of our field investigation, project personnel should be careful and take necessary precautions should hazardous materials be encountered during construction.

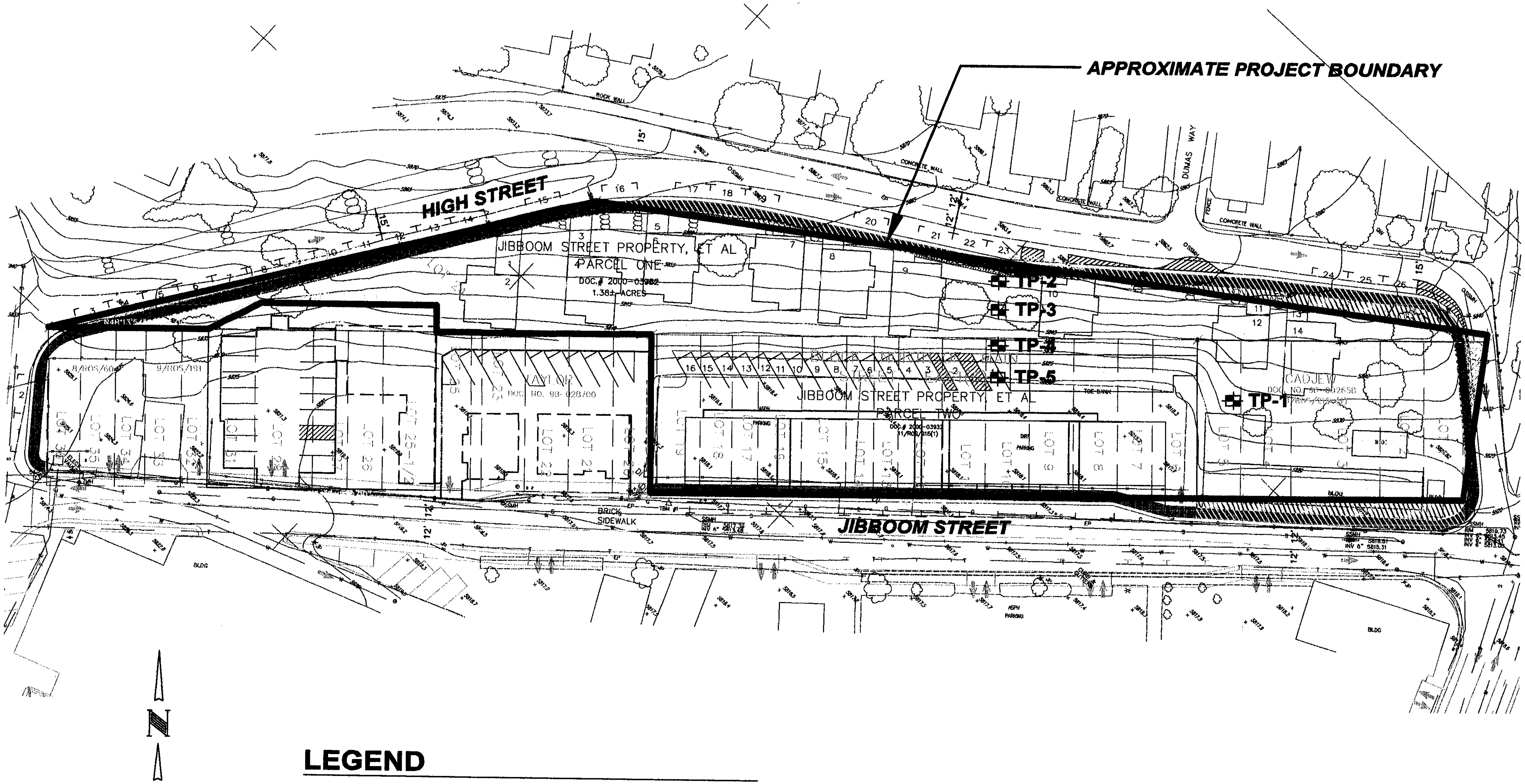
- The findings of this report are valid as of the present date. Changes in the conditions of the property can occur with the passage of time. These changes may be due to natural processes or work of man, at the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

PLATES

Plate 1 **Site Vicinity Map**
Plate 2 **Test Pit Location Map**



HOLDREGE & KULL CONSULTING ENGINEERS • GEOLOGISTS HK 15826 DONNER PASS ROAD TRUCKEE, CA 96160 (530) 587-5156	SITE VICINITY MAP HIGH STREET FRONTAGE IMPROVEMENTS TRUCKEE, CALIFORNIA	PROJECT NO.: 40310-01 DATE: APRIL 4, 2002 PLATE: 1



SCALE: 1" = 60'

LEGEND

TP-1 APPROXIMATE TEST PIT LOCATION

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
15826 DONNER PASS ROAD SUITE 101
TRUCKEE, CA 96160
(530) 587-5156 FAX 587-5196

TEST PIT LOCATION MAP
HIGH STREET FRONTAGE
IMPROVEMENTS
TRUCKEE, CALIFORNIA

DRAWN BY: BJH	CHECKED BY: JKH
PROJECT NO.: 40310-01	
DATE: APRIL 4, 2003	
PLATE NO.: 2	

APPENDIX A PROPOSAL

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

January 31, 2003

John McManus
c/o Truckee River Associates
11430 Deerfield Drive, Suite B5
Truckee, CA 96161

Attention: Tom Watson

Reference: ***Jibboom Street-High Street Development***
Truckee, California

Subject: ***Proposal for Geotechnical Engineering Investigation***

We appreciate the opportunity to present this proposal to perform a geotechnical investigation for the proposed development between Jibboom Street and High Street in downtown Truckee. The purpose of our investigation will be to explore and evaluate the subsurface conditions at the project site and provide geotechnical engineering recommendations for project design and construction.

Project Description

This proposal is based on conversations with MWA Architecture Engineering, review of a preliminary site plan and a site visit. The development will involve three projects on four parcels of land within a one block area between Jibboom Street and High Street. The McManus/Horst "A" Project will involve construction of a long, narrow two-story retail building. The project will include a cast-in-place concrete retaining wall along the back (north) side of the parcel and a mini-storage building. We anticipate the retail building will be wood-frame with a slab-on-grade floor. Earthwork cuts for the retaining wall are expected to be up to 8 or 10 feet in vertical extent. Some rock slope protection may be necessary above the concrete wall. The second project will include nine single family homes constructed along High Street. We understand the houses will be two-story, wood frame structures with raised wood and/or slab-on-grade floors. The third project will involve relocation of the existing Motel at the east end of the block and adding a kitchen at the back of the reconstructed building. The kitchen will likely require excavation up to 10 or 12 feet with a concrete retaining wall.

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Appurtenant construction for all three of the projects will include asphalt concrete parking areas, concrete curb, gutter and sidewalk, underground utilities and landscaping. In addition, High Street is planned to be widened with cut slopes of 1:1 and fill slopes of 1.5:1(H:V). Rock slope protection (RSP) is planned for the constructed slopes.

Structural loads were not available at the time this proposal was prepared. Therefore, we estimate for all the buildings are not expected to exceed 80 kips at isolated columns and 4 kips per linear foot along continuous wall foundations for long-term loading conditions. Earthwork cuts and fills for building pad grading of the project are expected to be up to approximately 5 feet in vertical extent with the exception of the kitchen.

Anticipated Conditions

In preparation of this proposal, we have made a site visit and reviewed previous work in the site area. Based on this information, we anticipate the site is underlain by glacial outwash deposits consisting of silty sand soil with numerous cobbles and boulders. Several large boulders are present in the existing cut slope and on the ground surface. Groundwater is not expected to be present within the depth of planned construction. However, groundwater seepage from slope excavations is expected at isolated portions of the site.

Scope of Services

Based on our current understanding of the project, we propose to perform a design level geotechnical engineering investigation of the project site, which will include the following scope of services.

Subsurface Exploration

The soil conditions at the project site are well exposed at the existing cut slope at the proposed retail building area. We propose exploring the subsurface conditions at the site by logging the existing cut slope and obtaining bulk and drive soil samples for laboratory testing. The slope will be logged by a engineer from our firm.

Laboratory Testing

Depending on the soil/rock conditions encountered, we anticipate that laboratory testing will include moisture-density determinations, direct shear tests, sieve analyses and plasticity tests. Direct shear test results will be used to derive foundation design criteria and strength parameters for slope stability analysis. Moisture-density determinations will establish the dry density and moisture content of relatively undisturbed soil samples.

Geotechnical Engineering Report Preparation

Following field exploration and laboratory testing, we will prepare an engineering report, which will present the following:

- Logs of subsurface conditions including depths to groundwater, if encountered;
- Site plan showing the approximate soil profile locations;
- Description of the subsurface conditions encountered at the site and how the conditions are expected to affect the planned construction;
- Earthwork grading recommendations including site preparation recommendations, a discussion of reuse of existing near surface soil as structural or non-structural fill, and a discussion of anticipated excavation conditions;
- Design criteria for foundations including an allowable bearing pressure, minimum footing depth, resistance to lateral loads and estimated settlements, and Uniform Building Code Soil Profile Type for use in structural design;
- Lateral earth pressures and drainage recommendations for retaining structures;
- Slab-on-grade recommendations;
- Recommendations for rock slope protection (RSP); and
- Asphalt concrete pavement design recommendations.

Schedule

We can begin our investigation immediately upon your authorization to proceed. We anticipate submitting our final written report within three weeks of completion of our field exploration. We could provide verbal information to you as it is developed in order to reduce project delays.

Fees

We propose to provide our geotechnical services for a lump sum fee of $3,800. This fee is based on the enclosed 2003 Fee Schedule. Progress billing will be on a monthly basis.

Enclosed with this proposal is our terms and conditions agreement. Please sign and return one copy of the attached terms and conditions to our attention if this proposal meets with your approval. All terms and conditions will be in effect upon receipt of your verbal authorization to proceed. If you have any questions, please contact the undersigned in our Truckee office.

Sincerely,
HOLDREGE & KULL

Gregory N. Porter
Staff Engineer

John K. Hudson, P.E., C.E.G.
Senior Engineer

encl: 2003 Fee Schedule
Terms and Conditions

cc: Michael Mason, MWA Architecture Engineering

TERMS AND CONDITIONS

This schedule sets forth the Terms and Conditions under which Holdrege and Kull (hereafter referred to as Engineer) will provide consulting engineering services to John McManus c/o Truckee River Associates (representing the Client and hereafter referred to as Client), pursuant to the attached proposal for geotechnical/geological services January 31, 2003 for the project titled Gibson Street-High Street Development. Acceptance of these terms and conditions by the Client is a condition precedent to the Engineer performing services on the project. The terms and conditions of this agreement are an attachment and part of the proposal.

Right of Entry: The Client will provide for right of entry and all necessary permits in order for the Engineer to perform services. It is understood by Client that in the normal course of field work some site damage may occur, including damage to vegetation, the correction of which is not the responsibility of the Engineer. Client agrees to indemnify and hold the Engineer harmless for any damage to subterranean structures or utilities which are not called to the attention of the Engineer or correctly shown in the field or on the plans furnished to the Engineer.

Ownership of Samples and Documents: All samples, boring logs, field data, field notes, laboratory test data, calculations, estimates, and other documents prepared by the Engineer shall remain property of the Engineer, except for reports submitted to the Client. All documents furnished to the Client which are not paid for, will be returned to the Engineer upon demand and will not be used by the Client or others. Soil samples will be disposed of 60 days following the submission of our report unless Client requests otherwise.

Standard of Care: Services performed by the Engineer under this agreement will be conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the local engineering profession currently practicing in California under similar conditions. No other warranty, either expressed or implied, is made. It is agreed that the Engineer's scope of services does not include the evaluation of the site for hazardous or toxic substances or chemical analysis of the soil unless specifically addressed in the attached proposal. The recommendations and conclusions are considered preliminary in nature, therefore, the engineer should be retained during construction to confirm and verify or modify the findings. If this is not possible, the Engineer will not be liable for the accuracy of the preliminary opinions.

Limitation of Liability: In consideration for the Engineer's provision of consulting services to the Client, Client expressly limits their right to sue or otherwise make any claim against the Engineer to an amount not to exceed the Engineer's fee, at law or otherwise, for any alleged error, omission, or any alleged negligent act or failure to act arising out of the performance of the Engineer's service, in contract, indemnity, tort arbitration, or any other action. In addition, Client expressly waives their right to sue, or otherwise make any claim against, any of Engineer's officers or employees as individuals for any cause.

In the event that any person or party, including a lending institution or property owner or occupier, makes a claim against the Engineer or any of Engineer's employees, at law or otherwise, for any alleged error, omission, or any other act arising out of the performance of professional services, the Client shall defend, indemnify, and hold the Engineer or employee harmless against the claim.

Work performed by others on nearby properties, heavy rainfall or man induced conditions beyond the control of the Engineer could adversely affect the property. Client agrees that the consequences of earth movement risks are assumed by Client. This investigation does not include a geologic hazards study or stability analysis of the general area within which the site is located unless specified in the proposal.

The Engineer shall not be responsible for safety during construction or the work of other contractors and third parties. It is expressly agreed and understood that the Engineer shall not be held responsible for delays or events caused by factors beyond his control, or by factors which could not reasonably have been foreseen at the time of the execution of the Agreement.

Assignment: Neither the Client nor the Engineer may delegate, add, sublet or transfer their duties or interest in the Agreement without written consent of the other party.

Entire Agreement: It is expressly agreed that these Terms and Conditions are a part of the Agreement between Client and Engineer. These combined documents shall be the entire Agreement and shall supersede any other agreement between Client and the Engineer relating to the project. In case of conflict or inconsistency between these Terms and Conditions and any other contract documents, these Terms and Conditions shall control.

The parties have read the foregoing, understand completely the terms and willingly enter into this agreement which will become effective on the date signed by the client below.

CLIENT

Signed By

Agent

Title

2/10/2003

Date

HOLDREGE & KULL

Signed By

Senior Engineer

Title

January 31, 2003

Date

APPENDIX B *IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT*

(included with permission of ASFE, Copyright 1998)

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one—not even you*—should apply the report for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject To Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations", many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

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Telephone: 301-565-2733 Facsimile: 301-589-2017
email: info@asfe.org www.asfe.org

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APPENDIX C EXPLORATORY TRENCH LOGS

TRENCH NO. 1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40310-01	HIGH STREET FRONTAGE IMPROVEMENTS	5830' MSL	2/26/03	1 OF 1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND EXCAVATED INTO EXISTING CUT SLOPE	BULK (BAG)	NONE	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
1-1	--	--	1		BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL AND COBBLES TO 8" DIAMETER, MOIST, LOOSE TO MEDIUM DENSE
			2		
			3	SM	BECOMES MEDIUM DENSE
			4		
			5		
			6		EXCAVATION TERMINATED
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

HOLDREGE & KULL

TRENCH NO. 2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40310-01	HIGH STREET FRONTAGE IMPROVEMENTS	5858' MSL	2/26/03	1 OF 1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND EXCAVATED INTO EXISTING CUT SLOPE	BULK (BAG)	NONE	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM (FILL)	DARK BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL AND COBBLES UP TO 12" DIAMETER, FILL, MOIST, VERY LOOSE, HIGHLY ORGANIC, NUMEROUS ROOTS, CONTAINS ¾" GRAVEL AND CHUNKS ASPHALT CONCRETE UP TO 6" DIMATER AND OTHER DEBRIS
2-1	--	--	2		
			3	SM	BROWN TO RED-BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, MOIST, MEDIUM DENSE TO DENSE
			4		
			5		EXCAVATION TERMINATED
			6		
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

TRENCH NO. 3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40310-01	HIGH STREET FRONTAGE IMPROVEMENTS	5848' MSL	2/26/03	1 OF 1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND EXCAVATED INTO EXISTING CUT SLOPE	BULK (BAG)	NONE	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN TO RED-BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, MOIST, LOOSE, (SLOUGH FROM CUT-FACE)
3-1	--	--	2	SM	BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL AND COBBLES UP TO 10", MOIST, MEDIUM DENSE TO DENSE
			3		
			4		TERMINATED EXCAVATION
			5		
			6		
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

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TRENCH NO. 4

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40310-01	HIGH STREET FRONTAGE IMPROVEMENTS	5833' MSL	2/26/03	1 OF 1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND EXCAVATED INTO EXISTING CUT SLOPE	BULK (BAG)	NONE	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN TO RED-BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, MOIST, LOOSE, (SLOUGH FROM CUT-FACE)
			2		
			3	SM	BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL AND COBBLES AND BOULDERS UP TO 4' DIAMETER, OBSERVED A 6" DIAMETER COBBLE OF FRIABLE GRANITE, MOIST, DENSE
			4		
			5		TERMINATED EXCAVATION
			6		
			7		
			8		
			9		
			10		
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			12		
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			20		

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TRENCH NO. 5

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40310-01	HIGH STREET FRONTAGE IMPROVEMENTS	5820' MSL	2/26/03	1 OF 1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
HAND EXCAVATED INTO EXISTING CUT SLOPE	BULK (BAG)	NONE	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN TO RED-BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, MOIST, LOOSE, (SLOUGH FROM CUT-FACE)
			2	SM	BROWN SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL AND VOLCANIC COBBLES TO 12" DIAMETER AND NUMEROUS GRANITIC COBBLES AND BOULDERS TO 3' DIAMETER, MOIST, DENSE
5-1	--	--	3		
			4	GW-GM	BROWN FINE TO COARSE GRAVEL WITH SILT AND COARSE SAND, WET, DENSE, (DECOMPOSED GRANITE), NUMEROUS BOULDER UP TO SEVERAL FEET IN DIAMETER
			5		TERMINATED EXCAVATION
			6		
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

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APPENDIX D LABORATORY TEST RESULTS

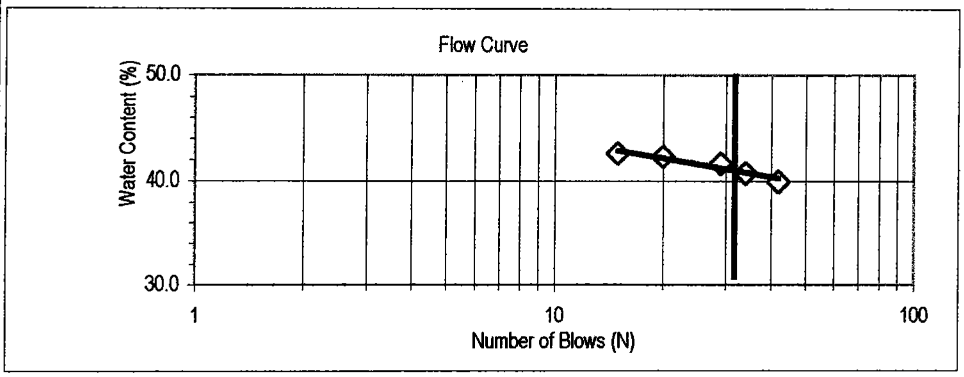
Atterberg Indices

ASTM D4318

Project No.:	**40310-01** Project Name: **High Street Frontage Improvements**	Date:	**3/8/2003**
Sample No.:	**5-1** Boring/Trench: **5** Depth, (ft.): **'2.5 to 3.5**	Tested By:	**JCS**
Description:	**Well Graded Gravel with Silt and Sand**	Checked By:	**MLH**
Sample Location:	**See Plate 2 - Test Pit Location Map**	Lab. No.:	**3-50**

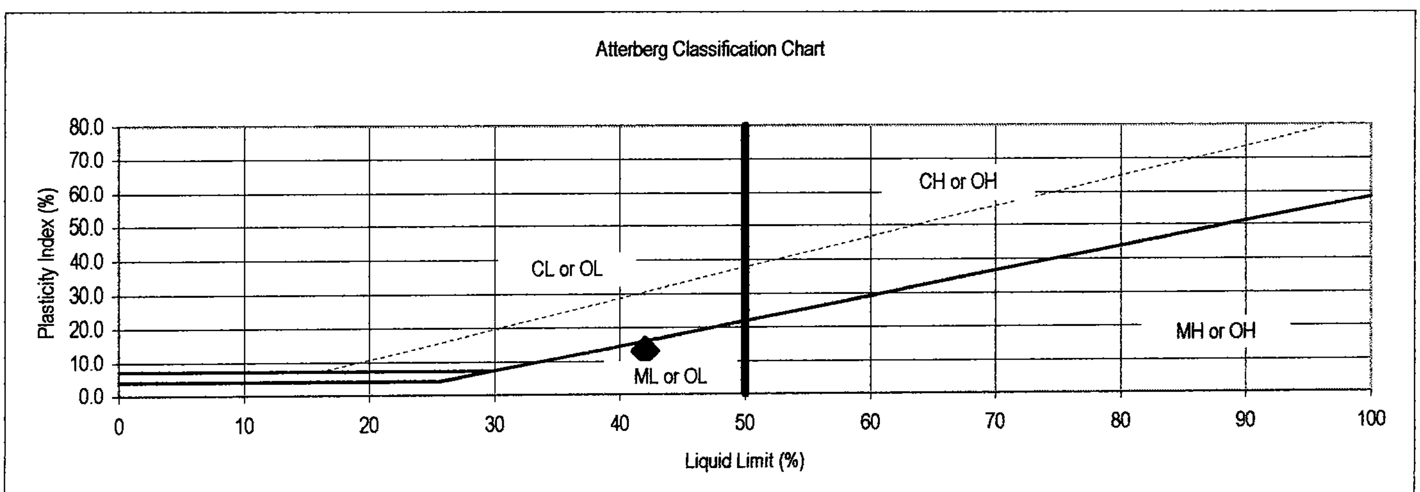
Estimated % of Sample Retained on No. 40 Sieve: 90 Sample Air Dried: Yes

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	30	AC	AB	AA	R2	30	35	28
Wt. Pan (gr)	21.34	21.45	22.59	21.38	21.54	8.11	8.13	8.06
Wt. Wet Soil + Pan (gr)	58.28	60.21	54.23	52.01	53.39	12.78	14.04	14.13
Wt. Dry Soil + Pan (gr)	47.24	48.68	45.20	43.00	44.17	11.75	12.71	12.75
Wt. Water (gr)	11.04	11.53	9.03	9.01	9.22	1.03	1.33	1.38
Wt. Dry Soil (gr)	25.90	27.23	22.61	21.62	22.63	3.64	4.58	4.69
Water Content (%)	42.6	42.3	39.9	41.7	40.7	28.3	29.0	29.4
Number of Blows, N	15	20	42	29	34			
				LIQUID LIMIT =	42		PLASTIC LIMIT =	28.9



Plasticity Index = 13.1

Group Symbol = ML



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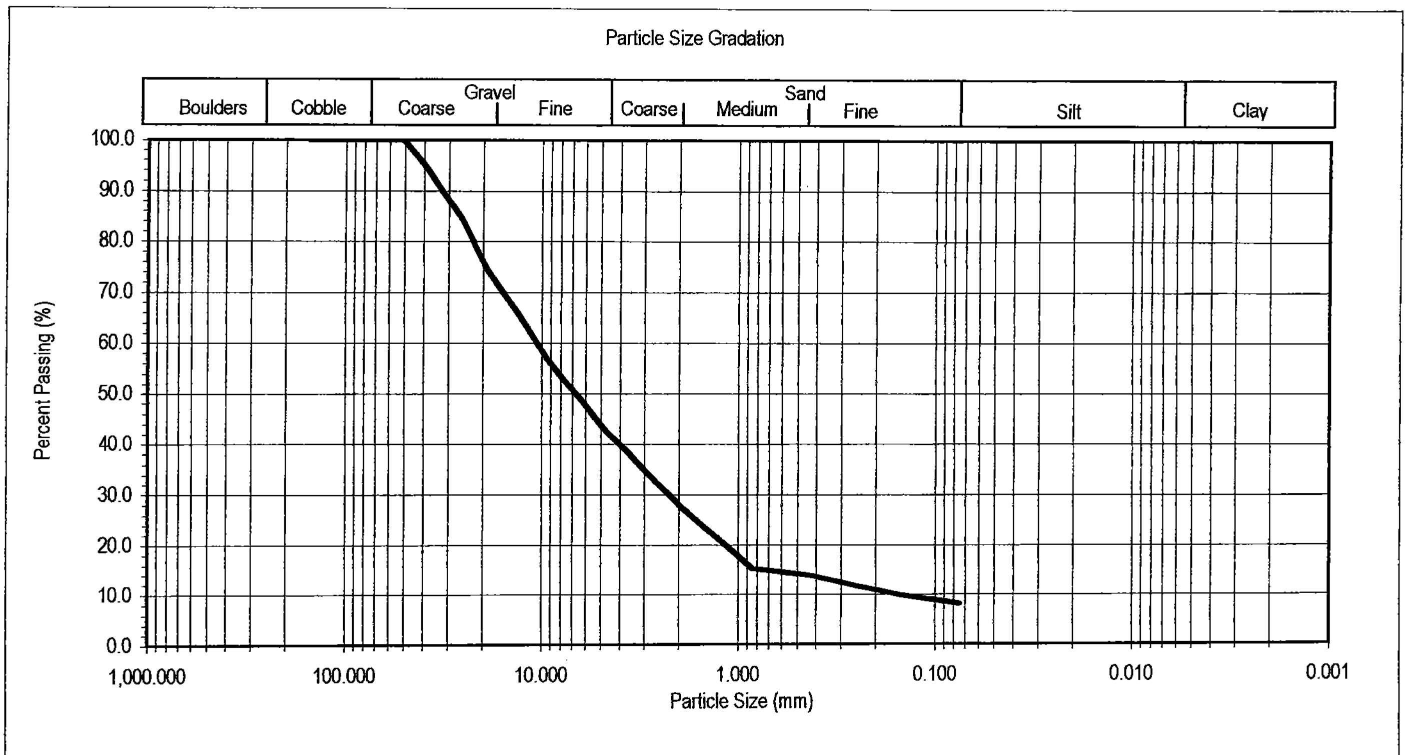
Particle Size Distribution

ASTM D422

Project No.: **40310-01** Project Name: **High Street Frontage Improvements** Date: **3/5/2003**
Sample No.: **5-1** Boring/Trench: **5** Depth, (ft.): **2.5 to 3.5** Tested By: **JCS**
Description: **Well Graded Gravel with Silt and Sand** Checked By: **MLH**
Sample Location: **See Plate 2 - Test Pit Location Map** Lab. No.: **3-50**

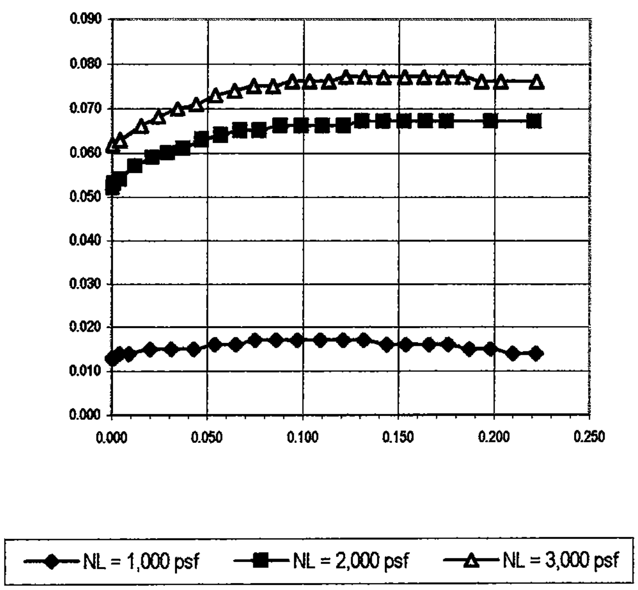
Sieve Size	Particle Diameter		Dry Weight on Sieve			Percent Passing
(U.S. Standard)	Inches (in.)	Millimeter (mm)	Retained On Sieve (gm)	Accumulated On Sieve (gm)	Passing Sieve (gm)	(%)
6 Inch	6.0000	152.4	0.00	0.0	3,749.3	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,749.3	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,749.3	**100.0**
1.5 Inch	1.5000	38.1	232.00	232.0	3,517.3	**93.8**
1.0 Inch	1.0000	25.4	366.00	598.0	3,151.3	**84.1**
3/4 Inch	0.7500	19.1	362.00	960.0	2,789.3	**74.4**
1/2 Inch	0.5000	12.7	358.00	1,318.0	2,431.3	**64.8**
3/8 Inch	0.3750	9.5	294.00	1,612.0	2,137.3	**57.0**
#4	0.1870	4.7500	532.00	2,144.0	1,605.3	**42.8**
#10	0.0787	2.0000	555.37	2,699.4	1,049.9	**28.0**
#20	0.0335	0.8500	472.02	3,171.4	577.9	**15.4**
#40	0.0167	0.4250	56.49	3,227.9	521.4	**13.9**
#60	0.0098	0.2500	81.21	3,309.1	440.2	**11.7**
#100	0.0059	0.1500	69.97	3,379.1	370.2	**9.9**
#200	0.0030	0.0750	64.70	3,443.8	305.5	**8.1**



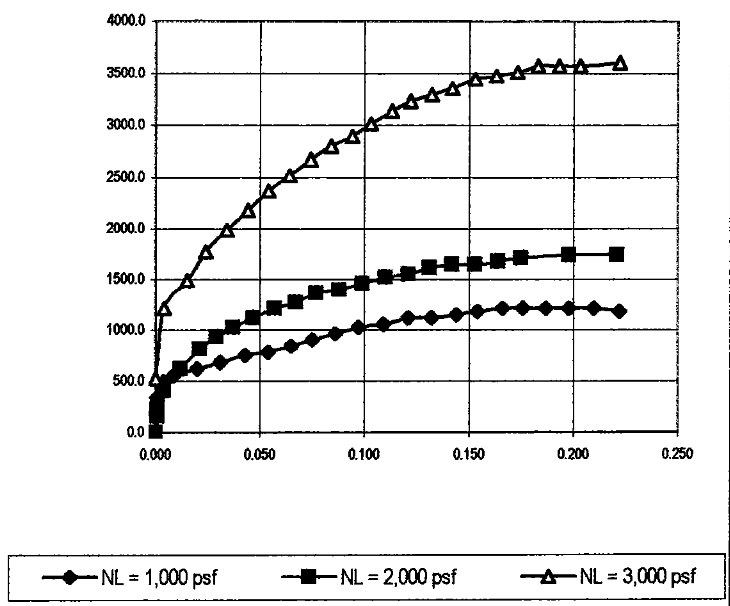
HOLDREGE & KULL

DIRECT SHEAR
TEST RESULTS

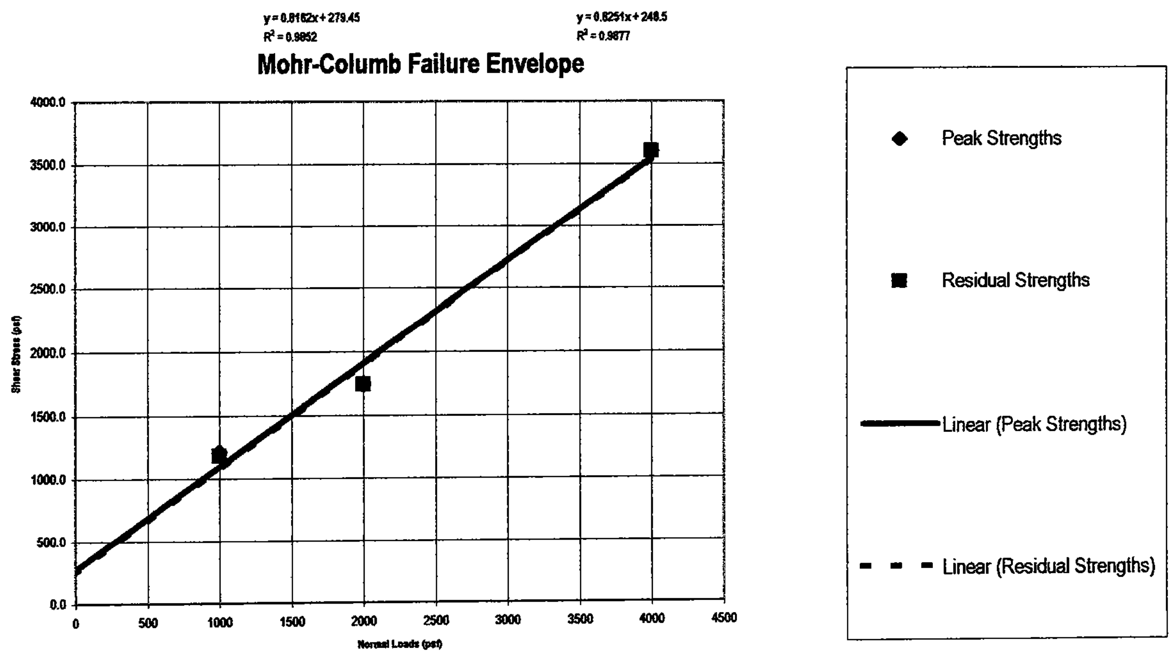
Shear Strain vs. Normal Strain



Shear Strain vs. Shear Stress



Mohr-Columb Failure Envelope



SHEAR STRENGTH TEST RESULTS		
PARAMETERS	PEAK STRENGTH:	RESIDUAL STRENGTH:
FRICTION ANGLE, (Degree)	39.2	39.5
COHESION, (psf)	279.5	248.5

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PROJECT NAME: High Street Frontage Improvements
PROJECT NO.: 40310-01 **DATE:** 3/6/2003
BORING / TRENCH NO.: 1 **LAB NO.** 3-50
SAMPLE NO.: 1-1 **SAMPLE DEPTH (ft.):** 1 to 2
DESCRIPTION: Brown Silty Sand with Gravel