Geotechnical Engineering Report
Victor Road Reconstruction
Anchorage, Alaska

December 2009

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ANCHORAGE, ALASKA

1.0 INTRODUCTION

This report presents the results of our engineering study for the proposed Victor Road reconstruction in Anchorage, Alaska. The purpose of this geotechnical study was to evaluate existing subsurface information and to formulate engineering recommendations for road improvements to Victor Road between West Dimond Boulevard and West 100th Avenue. The data used in formulating the following recommendations can be found in the December 2009 companion report entitled *Geotechnical Data Report, Victor Road Reconstruction, Anchorage, Alaska.*

We received authorization to proceed with reformatting our original reports to ADOT&PF standards in the form of a signed contract from James Sawhill of Lounsbury and Associates on December 27, 2006.

2.0 SITE AND PROJECT DESCRIPTION

As shown on the vicinity map included as Figure 1, the site is located in Anchorage, Alaska extending south from Dimond Boulevard to West 100th Avenue. The length of the project is approximately 2,600 feet between the centerlines of the above mentioned end streets. Currently, this section of Victor Road is straight but undulates with nearly 37 vertical feet of relief between high and low points, which occur approximately 1,100 and 2,050 feet south of the Dimond Boulevard centerline, respectively. At the low point of Victor Road (near its intersection with Olympic Drive), the banks of Campbell Lake are approximately 250 feet west of the Victor Road Centerline. The existing pavement conditions on this road are marginal with numerous longitudinal and transverse cracks and occasional frost heaves that suggest frost susceptible soils as a root cause of the pavement degradation. At present, Victor Road has relatively well defined ditches that serve to carry surface water away from the road prism. We understand that the proposed improvements to Victor Road will include improving the vertical alignment (cutting and filling the respective high and low points mentioned above), widening, potentially constructing a retaining wall along a portion of the alignment, and adding curb and gutter. A site plan showing the project alignment is included as Figure 2.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Given the subsurface conditions discussed in our Geotechnical Data Report, we conclude that pavement distress on Victor Road is likely due to a combination of frost susceptible subgrade and presence of a water table within 10 feet of the pavement surface. Although the pavement section appears strong enough to support the traffic during thawed ground conditions (little alligator cracking is evident to suggest that the pavement section has failed), the section is inadequate to resist stresses caused by heaving of the subgrade during the winter, which leads to splitting of the pavement along longitudinal, and to a lesser extent, transverse cracks.

In April 2002, Shannon & Wilson conducted a Phase I site assessment for the project area and identified three Leaking Underground Storage Tanks (LUST) sites cited in the ADEC databases. Two of these sites are located upgradient with respect to groundwater flow from Victor Road. At the Chevron fueling station located on the northeast corner of Victor Road and West Dimond Boulevard, groundwater has been documented as flowing northwest. Because of the location of the contamination at this property, it is our opinion that petroleum hydrocarbons released from this site could potentially affect a portion of the Victor Road ROW. The potential for handling impacted groundwater should be evaluated during the design of the project and methods to avoid dewatering or to properly handle the generated water should be included in the design.

Given the potential presence of contaminated groundwater, we believe the most cost-effective approach to improving Victor Road will be to incorporate subdrains on the southern end of the project to no more than 100 feet north of the low point on the project alignment. The presence of potential contamination on the north end of the project limit the possibilities for improvements because subdrains could yield contaminated water that would need to be treated prior to discharge. Additionally, hydrocarbon contaminated soil and water are not amenable to using insulated sections (due to deterioration of the insulation) or overexcavation into contaminated soil and groundwater for a thicker structural section.

In addition to subdrains on the south end of the alignment, the designer should also seek to avoid abrupt contrasts in subsurface conditions (soil type, moisture, thermal characteristics, etc.) that could cause differential movements over short distances. If subdrains, insulated sections, or thicker structural sections are desired on the northern end of the project, we recommend that a

Phase II environmental assessment be conducted to further define the nature of the potential contamination.

3.1 Site Preparation and Subgrade Development

Based on the approach described above, we recommend designing the road improvements to incorporate an improved pavement section overlying geotextile fabric to separate it from the frost susceptible native subgrade as follows. The existing asphalt pavement should be removed within the project limits and the shoulder areas to receive fill should be cleared and grubbed of organic matter to the width of the new embankments. After the asphalt pavement has been removed, the quality of subgrade soils and the salvage of existing road base fill must be addressed in preparing the new subgrade for construction of the pavement structural section.

Soil descriptions on Borings HLAB-3 and HLAB-4 indicate that organic soils were once present beneath the Victor Road embankment although later borings drilled through the embankment (Borings SWB-11, SWB-12A, and SWB-13) did not encounter organics. However, Boring SWB-16A encountered peat beneath about 5 feet of road base fill. These data suggest that organic soils are likely to be encountered in embankment expansion areas near Olympic Drive and also that organic soils may be present in isolated areas beneath existing embankments. We recommend that local areas of soft or compressible soils (including organics) should be over-excavated and replaced with structural fill to improve the subgrade support for the new embankment. For planning purposes, we estimate that subgrade mitigation could involve a 5 to 6-foot deep sub-cut over 50 percent of the embankment expansion area, but depending on the actual conditions encountered during construction, the required sub-cut may be greater.

Based on data presented in the geotechnical data report, we anticipate that the existing road base fill will vary from clean (less than 5 percent fines) to silty (greater than 12 percent fines). We believe that the average gradation of the existing fill will not meet the gradation requirements provided herein for base and sub-base fill, but also believe that it may fall within the gradation requirements for Selected Material Type B, which we believe is appropriate for embankment construction up to subgrade level. Therefore, we recommend that the fill to raise the grade of Victor Road in the sag curve close to Olympic Drive consist of existing road base fill (that meets Selected Material Type B requirements) salvaged from other portions of the project. Salvaged fill should be carefully segregated from native silts to avoid contaminating the granular fill. The salvaged material should be transported to the fill area, blended by windrowing, and then placed in lifts with moisture/density control per the requirements of Section 3.7.

When new embankments are to be constructed against existing embankments or hillsides, the new earthwork should be knit together with the existing earthwork by benching. Benching is accomplished by cutting horizontal notches in the sides of existing embankments and incorporating the cut material, if the soil type is suitable, into the new embankment. Each horizontal cut should start at the intersection of the original ground surface with the vertical side of the previous bench. Bench widths should be sufficient to permit placing and compacting the new fill with moisture/density control per Section 3.7.

3.2 Site Drainage

Because of the frost susceptibility of the native soils beneath the subgrade in this area, site drainage around the road should be provided to reduce the effects of seasonal frost in the new road surface. We assume that positive drainage will be maintained on the pavement surface with surface water being directed to the storm drain via catch basins in the curb and gutter. In addition, subdrains should be installed to intercept subsurface seepage and carry it away from the subgrade beneath the road prism on the southern end of the alignment extending north no further than 100 feet north of the low point of the alignment (see potential contamination discussion in Section 3.0). In our opinion, there is probably sufficient cross drainage toward Campbell Lake that a competent subdrain behind the curb line on the east side of Victor Road would protect the project. However, we recommend that the project include a contingency for a second subdrain behind the curb on the west side of Victor Road and that the decision to construct the drain should be made by the project engineer when the east drain is being constructed and the subsurface conditions can be visually evaluated.

The subdrain should consist of perforated pipe bedded in a clean granular drainage material and hydraulically connected to the road prism fill. If coarse rock (“drain rock”) is used to backfill around the drain, a geotextile filter material should be used to separate it from backfill above the drain pipe. Alternatively, the subdrain trench can be filled with inorganic, clean, well-graded sand and gravel consistent with Selected Material Type A material as shown in Figure 3.

3.3 Pavement Structural Section

Our analysis for design of a pavement structural section is based on the ADOT&PF Excess Fines Method. Based on our subsurface exploration, the native silt material (soil that will comprise the road subgrade) in this area contains an average of approximately 80 percent fines. Design values for the Equivalent Single Axle Loads (ESALS) and Average Annual Daily Traffic (AADT) were not available to us at the time of this report, so assumed values were used in our analysis. We developed a design pavement section tailored to the anticipated subgrade soil properties following site preparation as described in Section 3.1. The pavement structural section should conform to one of the options in the following table.

Structural Section #1	Structural Section #2
2 inches AC Pavement	2 inches AC Pavement
	4.5 inches ATB
6 inches D-1	6 inches D-1
18 inches Selected Material Type A	12 inches Selected Material Type A
36 inches Selected Material Type B	24 inches Selected Material Type B

Asphalt Treated Base (ATB), included in structural Section #2 in the above table, is a sub-base material that can be used to transition from the D-1 Base Course to the AC Pavement. ATB allows for a thinner structural section to be used for the road (approximately 4 feet versus 5 feet). ATB also allows for the AC Pavement to be milled and replaced with reduced effort to repair the subgrade material. Figure 3 provides the gradation requirements for the Selected Material Types A and B as well as D-1 Base Course fill materials.

The ESALS and AADT values used in analysis for this pavement structural section are assumed to be 1,400,000 for ESALS and 50,000 for AADT. If the actual values for these design parameters are found to differ from the numbers assumed, then the structural section of the pavement may also need to be adjusted. In the event that the actual ESALS and AADT values are greater than our estimates, then Shannon & Wilson should be advised so that the recommendations can be reviewed and our pavement structural section can be adjusted if necessary.

3.4 Utility or Subdrain Trenches

Buried utilities may be involved in the Victor Road upgrade project and trenches will be needed for installation of subdrains. Trenches excavated for installation of these utilities or drains should be constructed as presented in Figure 4. The bedding and structural fill material around the buried utility should be densely compacted to support and hold the pipe firmly in place.

Observations from boring logs, attached in the companion report, suggests that the typical utility trenches extending 10 feet below the existing grade will likely encounter groundwater. Dewatering with sumps and pumping equipment may be required if the trenches must be carried more than about 4 feet below the water table. The native soils are predominantly fine grained, which should limit the amount of seepage that will be experienced. However, the contractor should be prepared to work quickly and should avoid leaving trenches open for lengthy periods as water will collect and the soils will soften making them more frost susceptible.

Since the native soils in this area are generally cohesive and moist, trench slopes will tend to stand steeply initially, but in the presence of seepage they will soften and slump in time to relatively flat-lying slopes, which for planning purposes is estimated at about 3 horizontal (H) to 1 vertical (V), or flatter. Below the water table, running ground can occur as seepage occurs through cohesionless soils toward the excavation. The trench side slope and bottom conditions should be made the responsibility of the contractor as he is present on a day to day basis and can adjust his efforts to obtain the needed stability, and meet the applicable Alaska and Federal (OSHA) safety regulations.

Below areas to receive pavement sections, trench backfill should be placed in maximum 12-inch loose lifts and compacted to at least 95 percent of the Modified Proctor maximum dry density. In areas where no paving is planned, less compaction is required and material may be placed in thicker lifts (14 to 18 inches) and moderately compacted to achieve at least 90 percent compaction. Utility trenches should be backfilled with existing inorganic native soils as much as practical between the top of the pipe bedding and the bottom of the road subgrade, or to original ground surface in areas where no pavement is needed. Bulking of backfill into the trench should be discouraged as this can cause voids and lead to large future surface settlements.

3.5 Embankment Construction and Slope Stability

The construction of embankments will be necessary to develop the final grade of Victor Road, particularly through the low spot near Olympic Drive. Embankments should be constructed with compacted Selected Material Type B structural fill up to the bottom of the pavement structural section. Embankment material must be placed and compacted as stated in Section 3.7. Exposed embankment slopes (as well as exposed drainage ditch slopes if used), should be stable at a slope of 2H to 1V. Slopes should be covered with vegetation to prevent raveling or rilling of soil grains down slope.

3.6 Retaining Wall

A retaining wall approximately 360 feet long may be constructed on the west side of the alignment starting approximately 350 feet north of the intersection of 100th Avenue and Victor Road. If it is constructed, the wall will extend approximately 100 feet north of the intersection of Olympic Drive and Victor Road. We understand that the wall may be needed to address right-of-way issues and allow for the inclusion of a bicycle/pedestrian walkway on the west side of the road. Accordingly, the wall could be approximately 12 to 16 feet in height and will likely be a gravity wall constructed with gabion baskets.

Soil conditions along the proposed alignment of the retaining wall are described in Borings B-12A, B-13, B-14, B-15, and B-16A; the logs of which can be found in our Geotechnical Data Report for this project. It should be noted that the borings conducted in that study focused on exploring near-surface soils for road and embankment construction and only penetrated approximately 15 to 16 feet below the ground surface (bgs). In general, we encountered silt with isolated areas of interbedded sand and gravel. Boring B-16A also encountered buried peat and organic material from 5.5 to 10 feet bgs. According to standard penetration testing in our borings, the relative density of the soils varies with penetration resistances of between 6 and 14 blows per foot in the dominant cohesive soils. The conditions encountered in our borings are consistent with our experience around the Campbell Lake area. Based on our area experience, we expect that the soft to medium stiff fine grained soils likely extend to 10 to 20 feet below the water level of the lake, where conditions transition to stiffer, normally consolidated, fine grained soils. However, if a wall is included in the project and its design and performance is sensitive to long term settlements, we recommend that we be retained to perform additional explorations and engineering analysis to confirm the deeper soil conditions at the site.

We recommend that the gabion basket base of the wall be positioned such that it is bearing on a minimum of 2 feet of compacted, Select Material Type A over the existing firm, unyielding native soil. The Select Material Type A should be placed with moisture density control as described in Section 3.7. Note that buried organic material was encountered near the proposed south end of the wall in Boring B-16A. If buried organic material is encountered in the wall excavation, it should be removed and replaced with compacted, classified structural fill. Additionally, we recommend excavating test pits every 50 feet along the base of the wall (perpendicular to the wall alignment) to at least 10 feet below the design base of the wall to detect zones of buried organic material. Excavations to allow room for the 2 feet of Select Material Type A and to remove buried organic debris should extend under the entire thickness of the wall structure and beyond such that a plane extending down and out from the toe and heel of the wall at 1H to 1V only intersects compacted structural fill.

3.6.1 Retaining Wall and Slope Conditions

Based on the information provided, it appears that if a wall is constructed the critical section will be approximately 150 feet south of Olympic Drive. Note that this "critical" section represents the highest portion of the wall at approximately 16 feet of free-standing height. In our analyses, we assume that the soils below the retaining wall are fine-grained, cohesive soils and the wall will support new embankment fills consisting of Selected Material Type B or A, placed with moisture/density control. A surcharge load from vehicles on top of the embankment was not included in our analysis, but a surcharge load could exist if vehicles will be traveling within a 1:1 slope extending up from the bottom corner on the back side of the wall. Because the likely wall type is a gravity, gabion basket wall, we assumed that the wall structure will be a minimum of 12 feet thick (based on approximately 70 percent of the critical section free standing height) and that the base of the wall is embedded at least 5 feet below the existing ground surface (excluding surface organics and vegetative mat). Based on these assumptions and an average backslope above the wall of less than 5H to 1V, it is our opinion that the critical gabion wall section has factors of safety against rotational failure above 1.5 and 1.1 for static and dynamic (seismic) loading conditions, respectively. This stability assessment is only for external stability of the wall and embankment system, additional internal stability (sliding, overturning, etc.) was not addressed in our analysis. Note that this assessment is based on assumptions regarding soil conditions beyond the depths of our borings. Additional stability analyses can be conducted, but would require advancing several deeper borings to evaluate deeper soil conditions under the proposed wall.

Assuming that the wall is constructed as described above and the embankment surface and ground surface at the toe of the wall are graded to maintain drainage of surface water away from the wall, the forces applied to the wall from frost jacking should be negligible when compared to other earth pressures. We recommend that the retaining wall footing be designed with an allowable bearing capacity of 3,000 pounds per square foot (psf). These bearing values may be increased by 1/3 for short-term wind and seismic loading.

We assume that the retaining wall structure will be developed based on a proprietary welded-wire and/or mesh gabion system. We recommend that the wall be configured to meet the minimum embedment and structure thickness described above. The design of the wall should also address internal stability such as sliding and overturning and as such, the embedment and wall thickness should be increased beyond our recommendations if needed. The basket fills should be clean, well graded gravels and cobbles, appropriate for the selected gabion basket opening size. We recommend selecting a material with the durability requirements established by the ADOT for durability and wear resistance for aggregates provided on Figure 3. The ground surface around the wall should be designed to direct surface drainage away from the wall. To allow drainage through the gabion wall system, impermeable membranes or soils should not be installed on or behind the face of the wall.

3.6.2 Estimated Settlements

The magnitude of the settlements that could develop at the project site are dependent upon the applied loads, the density or stiffness of the support material, and the care with which structural fills are placed and compacted beneath foundations. The compaction recommendations and procedures described in Section 3.7 should be strictly adhered to for satisfactory results. Assuming that the local loose or compressible soils (if present) are excavated from beneath the footings and replaced, and adequate compaction of fill materials is achieved, we estimate that total maximum settlements will be about 4 to 6 inches or less with differential settlements being about 1/2 of the total settlements over a horizontal distance of 100 feet. The greatest amount of settlement should occur during the first three to five years, such that long term settlements of the new wall should be approximately half of the total settlements.

3.6.3 Lateral Earth Pressures and Lateral Resistance

The retaining wall will support earth fills and, therefore, should be designed to resist associated lateral earth pressures. The magnitude of the pressures is dependent on the method of placement of backfill, the type of backfill material, the backslope of the material above the top of the wall, drainage provisions, and whether the wall is allowed to deflect during or after placement of backfill.

If the wall is permitted to deflect laterally or rotate an amount equal to about 0.001 times the height of the wall, an active earth pressure condition under static loading would prevail, but rigid walls that are restrained from deflecting at the top, an at rest earth pressure condition would prevail. We believe that the site layout is configured in a way so that the top of the wall will not be anchored into place, so an active condition will likely be applicable.

In our analyses, we estimated earth pressures using the Coulomb equation allowing for the backslope above the wall and friction between the soil/wall interface of 25 degrees. We assumed that hydrostatic, frost heave, and parking surcharge forces behind the wall are negligible. To simulate seismic conditions, active driving forces were increased using the Mononobe-Okabe

Method for Calculating of Dynamic Earth Pressures assuming a horizontal acceleration value of 0.3 times gravity (g). The table below outlines how the calculated earth pressures are affected by adjusting the backslope of the soil above the top of the wall.

Slope of Backslope - degrees	Active Earth Pressure (pcf*)	At Rest Earth Pressure (pcf*)	Seismic Active Earth Pressure Increase (psf/ft**)
0 (No Backslope)	31	54	15.5
5 (11.43H : 1V)	32	90	18.1
9.5 (6H : 1V)	34	97	21.5
11.3 (5H : 1V)	35	100	23.4
14 (4H : 1V)	36	107	27.4
18.4 (3H : 1V)	39	127	42.9

* Active and At Rest Earth pressures are given in pounds per cubic foot (pcf)

** Seismic Active Earth Pressure Increase values are given in pounds per square foot per foot of wall height (psf/ft)

Lateral earth pressures during a seismic event should be estimated using a uniformly distributed active pressure increase of the indicated pressure per foot of wall height. Except for the sloping backfill, surcharge loading above the wall was not taken into account, but a surcharge load could exist if vehicles are parked or building foundations are constructed within a 1:1 slope extending up from the bottom footing corner on the back side of the wall. The surcharge load would likely be small and have a minimal impact on the earth pressures experienced by the wall as long as the vehicle loads are temporary loads from light cars or trucks.

Lateral forces from wind or seismic loading may be resisted by passive earth pressures against the sides of footings and the portion of the wall below grade. In our opinion, these resisting pressures can be estimated using an equivalent fluid weight of 250 pcf. This value includes a factor of safety of at least 1.5 on the full passive earth pressure to limit deflections. This value assumes that backfill around the footing is densely compacted.

Lateral resistance may also be developed in friction against sliding along the base of the wall structure on grade. These forces may be computed using a coefficient of 0.25 between the gabion wall structure and soil.

3.7 Structural Fills and Compaction

Backfill will be required to develop the prescribed pavement structural section, in the construction of embankments, and potentially in a retaining wall. Fill that is placed should be clean, granular soil to provide drainage and frost protection. These soils should meet the gradation requirements of Selected Material Type A or B, depending on location as addressed in this report. Selected Material Types A and B fill material can typically be placed in both wet and dry conditions. Gradation requirements for this material are presented on Figure 3.

There is insufficient information to determine whether existing road-prism fills will meet the gradation requirements for Selected Material Type A or B classified fill, however, it is our opinion that soils classified as sandy gravel or gravelly sand (See logs of Borings SWB-1, 2, 3, 8, 9, 10, 13, 14, 15, 16A, and 17 in the Geotechnical Data Report) may be suitable for re-use as structural fill on site. Where it is desirable to reuse excavated material, these granular soils must be segregated from fills containing greater than 5 to 6 percent fines (See logs of Borings SWB-5, 6, 7, 11, and 12A in the Geotechnical Data Report).

Structural fills beneath pavements and in embankments should be placed in lifts not to exceed 10- to 12-inches loose thickness, and compacted to 95 percent of the maximum density as determined by the Modified Proctor compaction procedure (ASTM D-1557). During fill placement, we recommend that large cobbles or boulders with dimensions in excess of 8 inches be removed from structural fills. In the initial lift over fine grained native soils, the compaction criteria may need to be relaxed somewhat to establish a firm surface to resist compaction efforts.

4.0 CLOSURE AND LIMITATIONS

This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions and recommendations contained in this report are based on information provided from the observed site conditions and other conditions described herein. The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of explorations. It is assumed that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If, during construction, subsurface conditions different from those encountered in these and prior explorations are observed or appear to be present, Shannon & Wilson, Inc. should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

We recommend that we be retained to review those portions of the plans and specifications pertaining to earthwork and foundations to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly the installation of shoring and site excavations, preparation of subgrade for footing foundations and compaction of structural fill, and also to make field measurements of ground displacements and such other field observations as may be necessary.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachment in Appendix A *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact the undersigned.

We appreciate this opportunity to be of service. Please contact the undersigned at (907) 561-2120 with questions or comments concerning the contents of this report.

Sincerely,

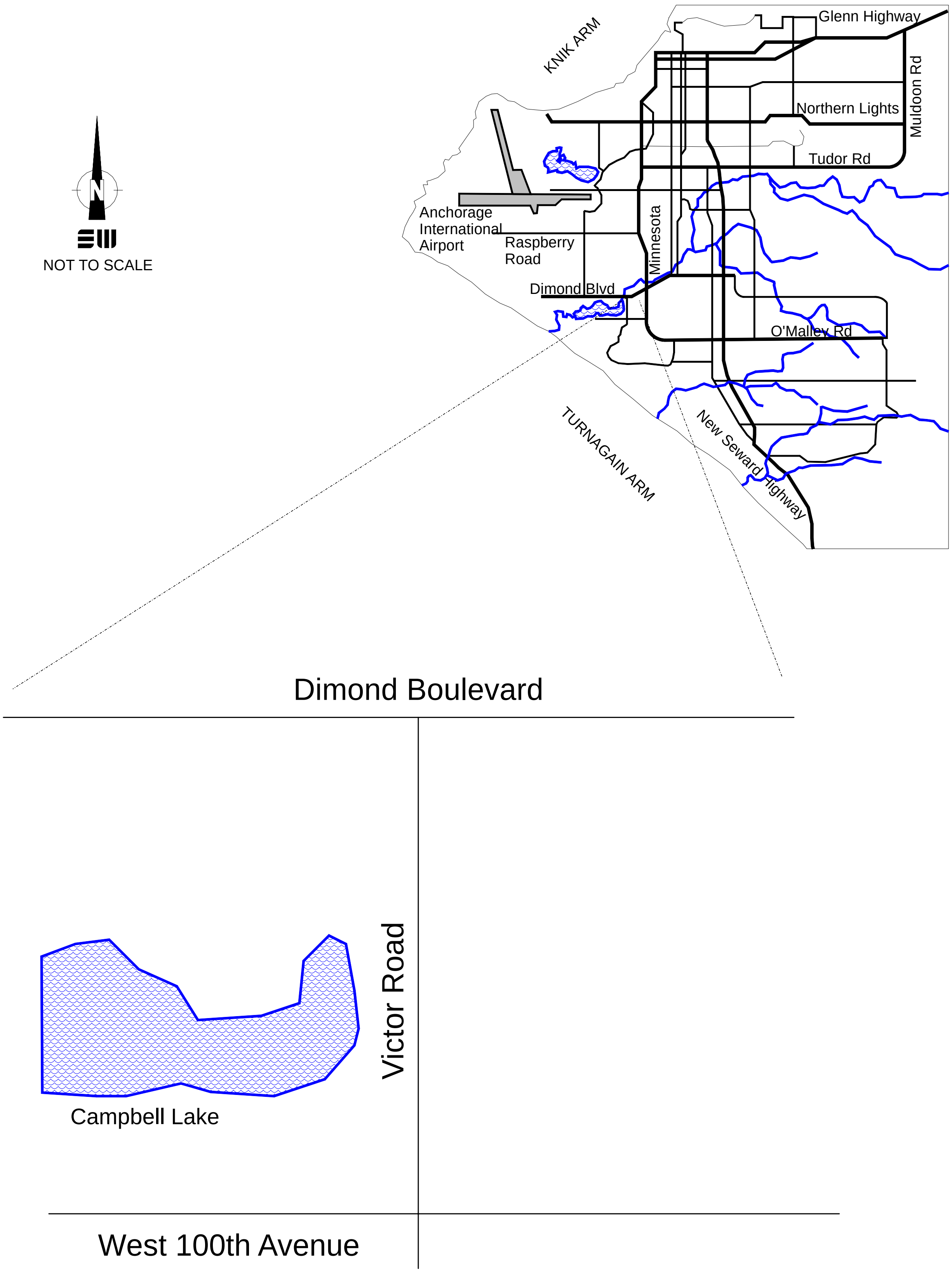
SHANNON & WILSON, INC.

Prepared by:

Kyle Brennan, P.E.
Senior Geotechnical Engineer

Reviewed by:

Stafford Glashan, P.E.
Vice President



Victor Road Reconstruction
Anchorage, Alaska

VICINITY MAP

December 2007

32-1-01870

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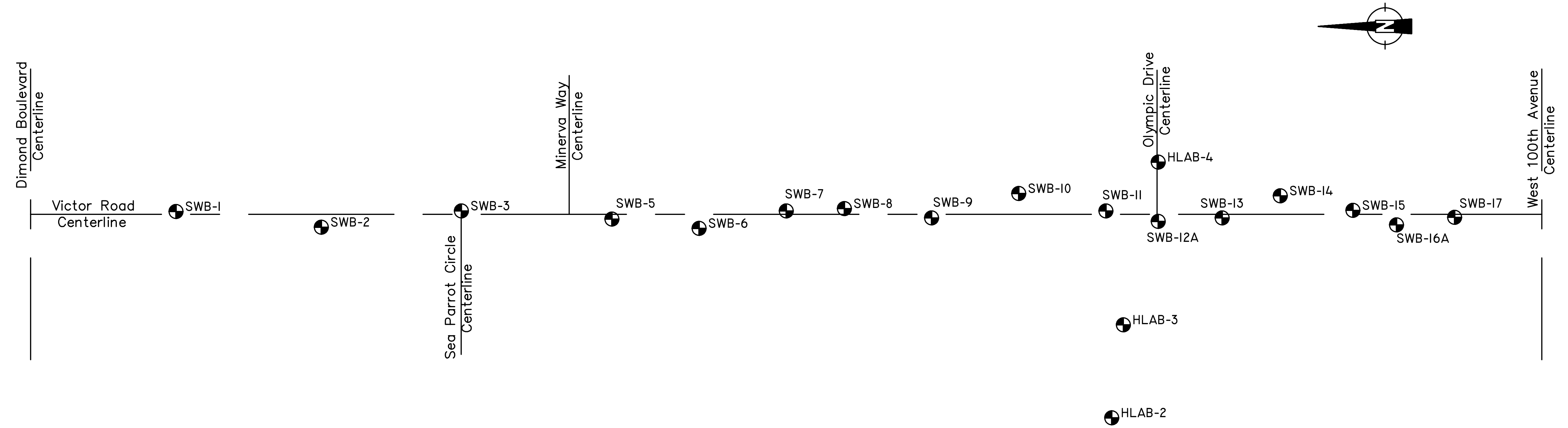
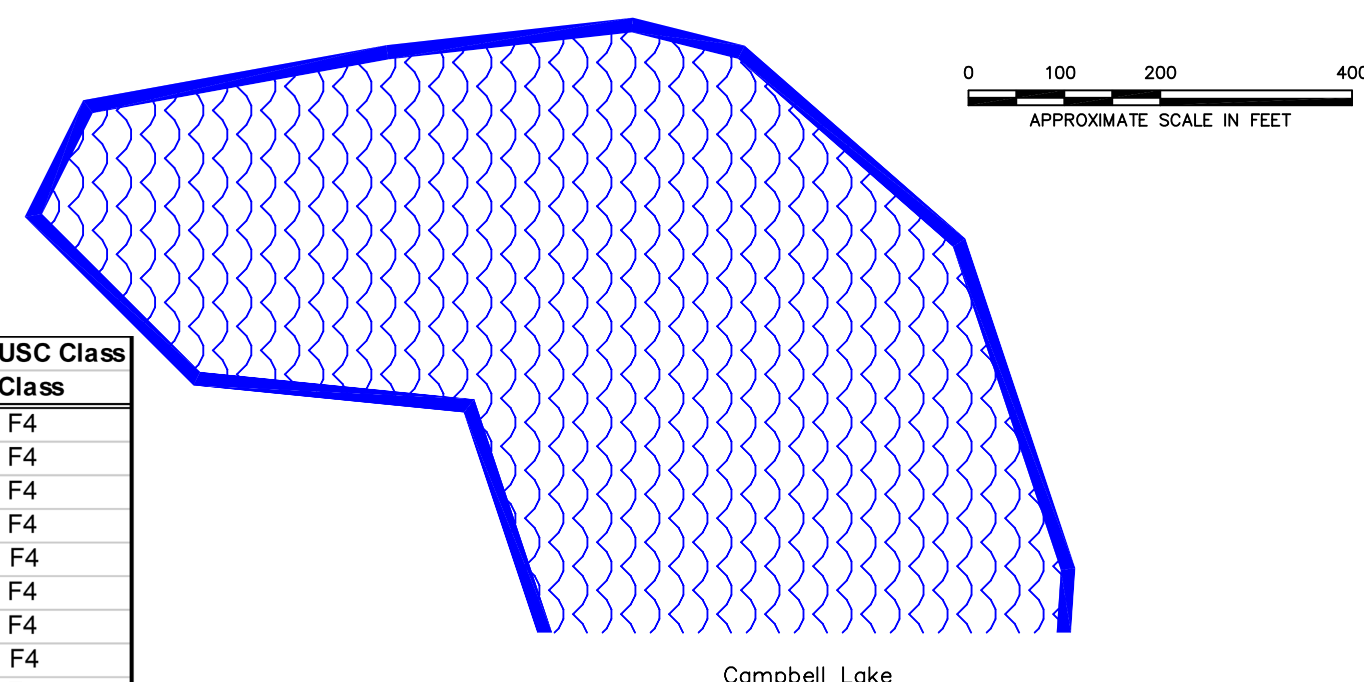
Fig. 1





LEGEND

SWB-1 Number and approximate location of boring B–1 advanced by Shannon & Wilson, Inc. from June 1994 "Geotechnical Report Victor Road, Dimond Blvd to West 100th Ave."

HLAB-2 Number and approximate location of boring B–2 advanced by Harding Lawson Associates from August 1985 "Soil Investigation, 97th Avenue Storm Drain"

Approximate Boring Locations +

Boring Number	Distance South of Dimond Blvd. CL (ft)	Distance From Victor Rd. CL (ft) *	Asphalt Thickness (in)	Fill Thickness (ft) USC Class	Subgrade USC Class Frost Class
SWB-1	250	5	3	5, GP	ML, F4
SWB-2	500	-22	3	5, GP	ML, F4
SWB-3	741	6	4	2, GP	ML, F4
SWB-5	1000	-8	3	3.5, GP-GM	ML, F4
SWB-6	1150	-6	NA	5.5, SP-SM	MH, F4
SWB-7	1300	6	3	4, ML/SM	ML, F4
SWB-8	1400	10	NA	2.5, SP	ML, F4
SWB-9	1550	-6	6	2.5, SP	MH, F4
SWB-10	1700	36	NA	7.5, SP	ML, F4
SWB-11	1850	6	3	5, ML/SM	MH, F4
SWB-12A	1940	-12	NA	2.5, ML/SM	GP, F2
SWB-13	2050	-6	5	5.5, ML	ML, F4
SWB-14	2150	32	NA	5.5, GP	ML, F4
SWB-15	2275	7	4	2.5, SP	ML, F4
SWB-16A	2350	-18	NA	5.5, SP	MH/PEAT, F4
SWB-17	2450	-5	4	5, GP	MH, F4
HLAB-2	1860	-350	NA	4, SM	SP, F1
HLAB-3	1880	-190	NA	1, SM	ML/PEAT, F4
HLAB-4	1940	90	NA	NA	ML/PEAT, F4
	* Negative distance denotes Westerly offset				

+ Locations measured relative to existing roadway centerlines in the field with a surveyor's wheel.

Victor Road Reconstruction
Anchorage, Alaska

SITE PLAN

December 2007 32–1–01870

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FIG. 2

GRADATION AND DURABILITY REQUIREMENTS

After: Alaska Department of Transportation
Standard Specifications for Highway Construction

D1

U.S. STANDARD SIEVE SIZE English	Metric	PERCENT PASSING BY WEIGHT
1 in.	25 mm	100
3/4 in.	19 mm	70 - 100
3/8 in.	9.5 mm	50 - 80
No. 4	4.75 mm	35 - 65
No. 8	2.36 mm	20 - 50
No. 50	0.300 mm	8 - 30
No. 200	0.075 mm	0 - 6

Selected Material Type A

U.S. STANDARD SIEVE SIZE English	Metric	PERCENT PASSING BY WEIGHT
No. 4	4.75 mm	20 - 55
No. 200	0.075 mm	6 Max. on minus 3-in. portion

Aggregate containing no muck, frozen material, roots, sod or other deleterious matter and with a plasticity index not greater than 6 as tested by WAQTC FOP for AASHTO T 89/T 90. Meet the gradation as tested by WAQTC FOP for AASHTO T 27/T 11.

Selected Material Type B

U.S. STANDARD SIEVE SIZE English	Metric	PERCENT PASSING BY WEIGHT
No. 200	0.075 mm	10 Max. on minus 3-in. portion

Aggregate containing no muck, frozen material, roots, sod or other deleterious matter and with a plasticity index not greater than 6 as tested by WAQTC FOP for AASHTO T 89/T 90. Meet the gradation as tested by WAQTC FOP for AASHTO T 27/T 11.

Coarse Aggregate Durability

Retained on #4 Sieve

Test Type	Percent Loss
L.A. Abrasion	45 - 50 max. *
Sulfate Soundness	9 max.

* Asphalt and Surface Course = 45% max
Base Course = 50% max

Victor Road Reconstruction
Anchorage, Alaska

GRADATION REQUIREMENTS

December 2009 32-1-01870

SHANNON & WILSON, INC.
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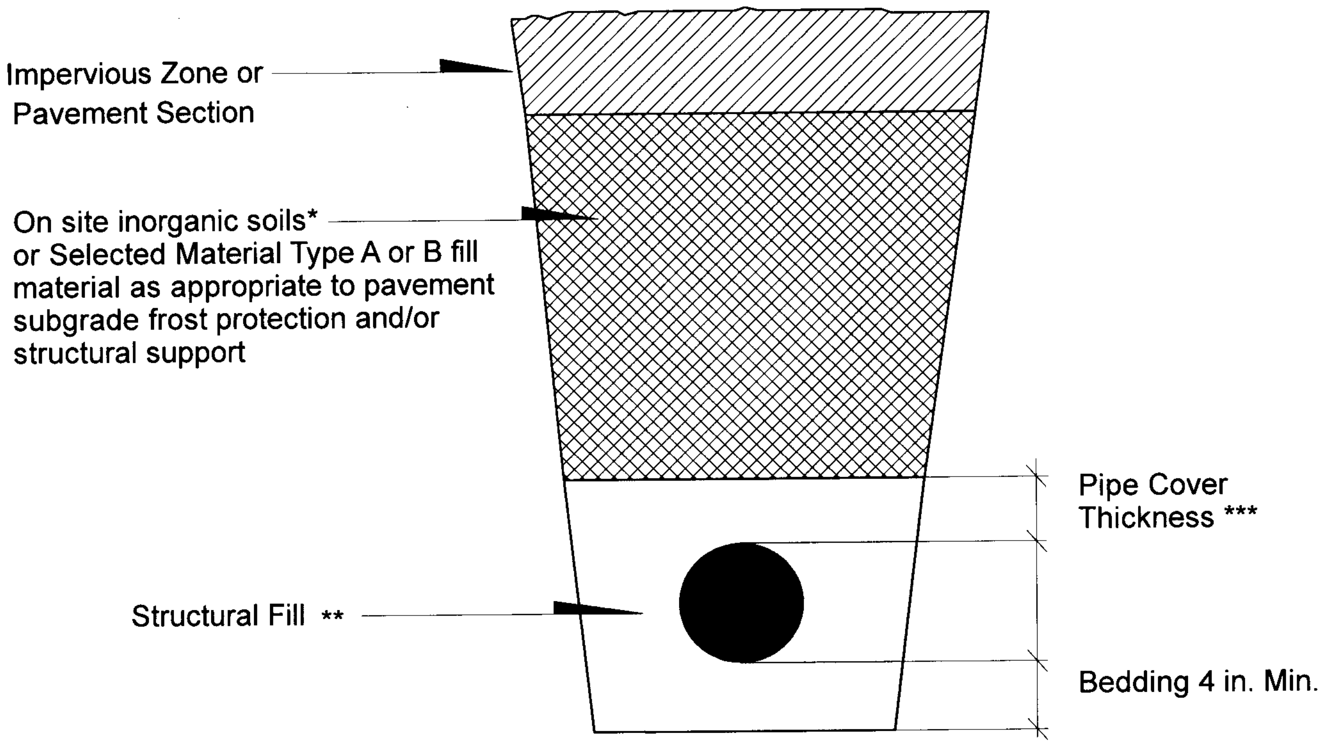
Fig. 3



* Inorganic soils, 95% compaction below structural fill supporting footings, streets, etc., 90% compaction in non structural support areas.

** Inorganic clean sand or well-graded sand and gravel (max. particle size 2-inch diameter) with less than 6 percent fines. Fill to be compacted to 95% Modified Proctor maximum dry density (ASTM D 1557) or as recommended by pipe manufacturer for specific application.

*** Pipe cover thickness as specified by pipe manufacturer for specific application. Absent manufacturer specifications, pipe cover thickness depends on corrosion and structural support properties. In non-structural support and non-corrosive environment, minimum bedding fill thickness should be at or above springline of pipe. In non-structural support area with corrosive environment, pipe cover should extend at least 6-inches above top of pipe. In structural support area, minimum pipe cover should be 6-inches or one pipe diameter above top of pipe, whichever is greater.

NOTE:

OSHA requires slope protection and support for all trenches greater than 4 feet deep. Side slope requirements are variable depending upon soil type and the duration of time in which the trench remains open. The contractor should be made responsible for compliance to these regulations as he/she is at the project on a day to day basis and is aware of changing conditions.

DRAWING NOT TO SCALE

Victor Road Reconstruction Anchorage, Alaska	
UTILITY TRENCH DETAIL	
December 2009 32-1-01870	
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	**Fig. 4**

APPENDIX A

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to 32-1-01870

Date: December 2009
To: Lounsbury & Associates
Re: Victor Road Reconstruction
Anchorage, Alaska

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland