STORM DRAINAGE REPORT

Johnson Hangars
Arlington Airport
Arlington, WA

August 22, 2003

By

Two Rivers Engineering
307 N. Olympic Ave.
Arlington, WA 98223
(360) 435-8167

MN-03-061

1.0 AUTHORIZATION – The report was authorized by Mr. Johnson in June 2003.

2.0 PURPOSE – this report has been prepared to comply with the City of Arlington storm drainage requirements for new development. This report describes the drainage system improvements which have been designed to control runoff quantity and quality.

2.0 PROJECT SITE – The project is located at the Arlington Airport west of 59th Drive and north of 188th. It is in Section 15, Township 31 N, Range 5 E. The project is composed of two separate parcels. The southern parcel is approximately 1.4 acres south of the taxiway. The northern parcel is approximately acres north of the taxiway.

3.0 EXISTING LAND USE – The south parcel is an undeveloped grassed tie down area. The north parcel contains two structures. The westerly structure is approximately 11,900 sf manufacturing plant. The easterly structure is a small building being used for vehicle detailing. The taxiway that separates the two parcels is a paved area. Site photographs are in Appendix A.

4.0 PROPOSED DEVELOPMENT – Development will proceed in two phases. The southern parcel is planned to be constructed first. The building will be four aircraft hangars. The northern parcel will be modified by removing the small easterly structure and constructing a larger aircraft hangar along with an entry road and parking. The westerly building will remain. The taxiway between the parcels will be rehabilitated as part of the overall project.

5.0 TOPOGRAPHY – Sheet C- 5 shows the layout of the proposed drainage system. The area is extremely flat with less than 2 feet of elevation change across the entire site. The site drainage is controlled by existing facilities on all sides. The taxiway between the north and south property also controls elevation changes.

6.0 SOIL TYPE - Site soil conditions are uniform across the area. The soil profile is characterized by a shallow layer of topsoil between 6 inches to one foot. The lower soils typically are an overburden of fine to medium sand overlaying a fine to course sand with gravel. Numerous soil test pits were excavated to determine the soil type and depth of groundwater. Test pits were excavated to an average of 15 feet below grade at the proposed infiltration basin sites. No groundwater was encountered. The recommended soil percolation rate was 20” per hour. A soils report has been prepared by Cornerstone Geotechnical, Inc. that presents a detailed description of conditions encountered along with recommended design criteria. The report is in Appendix B.

7.0 EXISTING STORM DRAINAGE SYSTEMS – The site has minimal drainage features at this time. This condition reflects the very permeable soil conditions that exist at the site. There is a minimal shallow drainage ditch on the western side of 59th. There are no road culverts at any street or driveway crossings. Drainage from the existing structures is directed to permeable areas adjacent to buildings. Most structures drain directly onto grass areas adjoining the buildings. No drainage system is included in the present parking or road areas.

8.0 UPSTREAM DRAINAGE AREAS – There is no drainage from upstream areas that crosses through the site. A small amount of drainage sheet flows from 59th Dr west to the ditch then immediately infiltrates into the ground. A small amount of runoff from the access road on the south boundary of the south parcel may flow northward but the pavement is old with differential settlement and extremely flat. Any runoff would be negligible. Taxiway drainage does drain to the north and south edges before percolating into the soil. The taxiway generally drains from the east to the west. Aircraft requirements dictate very level surfaces. Lack of any drainage system support the favorable site conditions and present proof that drainage is not a concern at the airport.

10.0 DOWNSTREAM DRAINAGE - No downstream drainage systems are present. No runoff is anticipated to leave the site.

11.0 DESIGN STORM – the project site is located at latitude N 48° 9.6', longitude W 122° 8.45'. The design rainfall criteria for a 24-hour storm have been obtained from the NOAA Atlas 2, Volume IX. Strom volumes for the 24-hour events are shown in Table 1.

Table 1, Arlington WA 24 Hour Storm Volumes

Return Frequency		
2 Year	10 Year	100 Year
1.8"	2.8"	3.75"

12.0 DESIGN CRITERIA – The drainage system is designed to control runoff via infiltration up to the 100-year event. Biofiltration and oil water separation shall provide runoff water quality treatment before entering the infiltration basins. The basins are sized utilizing a safety factor of 2 for the soil percolation rate and a safety factor of 1.5 to accommodate long-term storms. These factors are consistent with the 1992 Department of Ecology Storm Drainage Manual.

12.1 PERCOLATION RATE – The percolation rate has been calculated as shown below:

20 "/hr / (12in/ft. x 60 min./hr. x 60 sec. / min.) X 0.5 = 0.0001157 cfs/sf.

Department of Ecology recommends 3 feet separation between the bottom of an infiltration basin and the ground water table. Various test pits were excavated between 14 to 16 feet below grade on the site. The average site elevation is 300. Infiltration basins can be constructed at an elevation of 287 or higher and meet the recommended separation.

12.3 DESIGN CALCULATIONS - The design calculations used to develop the infiltration basin sizing was Watershed G 2 by Engenious Systems, Inc. Calculation printouts are in Appendix B.

13.0 PROPOSED DRAINAGE SYSTEM – Drainage systems for each phase will be constructed to accommodate runoff for each improved area. Phase 1 will be the southern parcel and the taxiway. Phase 2 will be the northern hangars and new road and parking. The existing manufacturing building will remain.

13.1 13.1 PHASE 1 SOUTH PARCEL – Runoff from the south area and taxiway will flow to two infiltration basins. The basins are labeled 1 E (east) and 1 W (west). The runoff will collect ½ of the taxiway, the buildings, part of 59th drive and the south access road. Runway runoff will be routed to a splitter box that can meter flow into two directions. The runoff will be delivered to the bioswales that will be constructed over each infiltration basin, flow to a catch basin with an oil water separator then infiltrate via a pipe system into the ground. Each basin will be 45' x 5' base infiltration basin with 1:1 sidewall slope filled with gravel for a 33% void ratio.

13.2 PHASE 2 NORTH PARCEL – The north parcel drainage shall collect ½ of the taxiway, the new hangar building and the existing structure. Runoff shall be collected on the east and west portions of the area and routed to bioswales for treatment passed through an oil water separator then into infiltration basins. The 2E basin shall be 95' x 10' based infiltration basin with 0.5 to 1 sidewalls, gravel filled for a 33% void ration. Its base elevation shall be 293. The 2 W basin will be a 100' x 18' vertical walled infiltration basin, gravel filled for a void ratio of 33%. The base elevation shall be 294.

Table 2,Johnson Hangars Infiltration Basin Performance for the 100-Year Storm before Enlargement by 50%

Basin	Base Dimension	Side Slope	Base Elevation	100 Yr. Volume, cf	Max Stage	Time To Drain, Hr.
1 E	30' x 5'	1: 1	291.5	1140	298.5	24.77
1 W	30' X 5'	1: 1	291.5	1140	298.5	24.77
2 E	63' x 10'	0.5: 1	293	1273	297.25	24.27
2 W	67' x 18'	Vert.	293	1570	296.95	25.12

14.0 CONSTRUCTION SEQUENCE – The project may be constructed in two or more phases. Initial plans envision construction of Phase 1, the southern area, followed by Phase 2 at a later time. The taxiway rehabilitation shall be constructed as part of Phase 1. The site is small with very permeable soils and no drainage entering or leaving the site. The well-drained native soil and the lack of any off site erosion or sedimentation concerns allow construction at any time of the year.

15.0 OPERATION AND MAINTENANCE – A separate operation and maintenance plan is included in Appendix C. Those operations should be carried out during and after construction to obtain the longest life from any infiltration basin. While the design of each basin adequately addresses the site conditions, the actual construction and maintenance of the systems shall determine the system longevity.

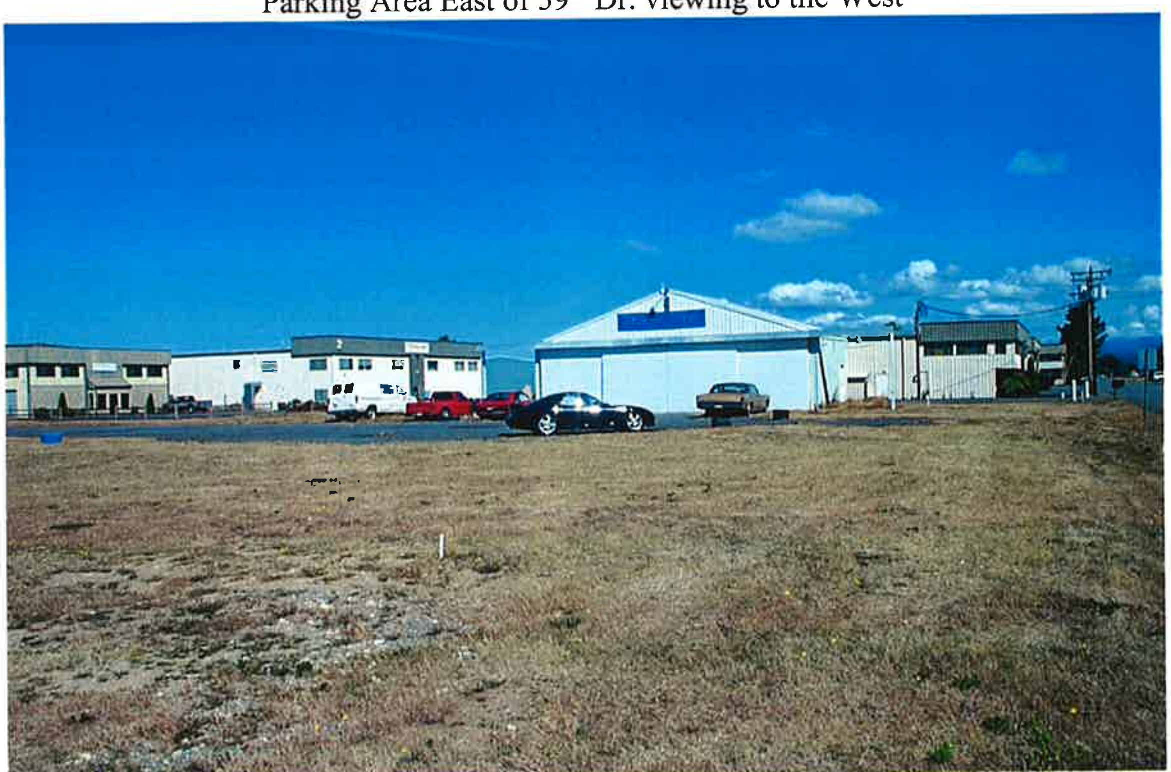
Appendix A

Site Photographs

Phase 1 Area viewing west from 59^{th} Dr.

East Project Area & 59^{th} Dr., Viewing North from Access Road

Parking Area East of 59th Dr. viewing to the West



East Portion of Phase 2 Area, viewing from South

Phase 2 E Area viewing to the East

Infiltration Basin Area for Phase 2 W viewing to the West

Appendix B

Geotechnical Report

Geotechnical Investigation
Johnson Hangers
Arlington, Washington
For
Mr. Steve Johnson

Cornerstone
Geotechnical, Inc.

Cornerstone
Geotechnical, Inc.

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March 14, 2003

Mr. Steve Johnson
24005 – 13th Avenue NE
Arlington, WA 98223-9040

Geotechnical Investigation
Johnson Hangers
Arlington, Washington
CG File No. 1432

Dear Mr. Johnson:

INTRODUCTION

This report presents the results of our geotechnical engineering investigation at your proposed new hanger facilities at the Arlington Airport. The site is located at 19033 - 59th Drive NE, off the northeast corner of the Arlington Airport, as shown in the Vicinity Map on Figure 1. You plan to expand your airplane hanger facilities with two new hangers. Site storm water runoff will be directed into infiltration trenches or covered basins. Mr. Tom Kearns with Two Rivers Engineering provided us with site drawings, which showed the locations of planned new hanger buildings and infiltration areas. The site consists of two parcels separated by a paved taxiway.

Mr. Kearns indicated that the infiltration design for the site should be completed under the 1992 Department of Ecology manual using the grain size distribution of a sample from the site to compare to the textural triangle in the manual. You requested that we complete this report to provide a recommended infiltration rate for design of storm water infiltration at the site, and geotechnical design parameters and general recommendations for site development.

PROJECT DESCIPTION

You plan to construct metal-frame buildings for use as airplane hangers and add paved parking areas. Two separate hangers with associated parking areas are planned to be constructed in separate phases of the work. Phase 1 will be development of the southern parcel and Phase 2 will be development of the northern parcel.

Infiltration is expected to consist of trenches or basins excavated down to suitable soil, backfilled with gravel, and covered with filter fabric and topsoil. Storm water runoff will be routed through an oil/water separator and into a bioswale area before entering the infiltration system. Approximately 100 feet of travel over a vegetated bioswale is intended to provide filtration and removal of fine soil particles (fines) before the water enters the infiltration system. The size of the infiltration areas will be completed based on the results of this report. Infiltration areas are planned at the east and west side of each parcel, as shown on the Site Plan in Figure 2.

An existing hanger is located on the west end of the northern parcel. An existing building on the east end of the north parcel will be removed. An east-west trending gas line located through the center of the Phase 1 hanger on the southern parcel will be relocated. There are no significant cuts or fills planned relative to the existing grade.

SCOPE OF SERVICES

The purpose of this study is to explore the subsurface conditions at the site and provide recommendations for design. Specifically, our scope of services will be:

1. Evaluate subsurface soil and ground water conditions by excavating test pits with a subcontracted excavator. Include at least one pit in each of the four planned infiltration areas to a depth of at least 15 feet if possible.
2. Complete grain-size analysis on samples at the planned infiltration locations.
3. Compare grain size results to 1992 DOE manual and provide a recommended infiltration rate for design.
4. Prepare a letter to document our findings and recommendations.
5. Provide site preparation, grading, and drainage recommendations.
6. Provide recommendations for foundation support and subgrade preparation for slab and paved areas.
7. Prepare a geotechnical report to document our findings and recommendations.

SITE CONDITIONS

Surface Conditions

The site consists of two parcels on the east side of 59th Drive NE at the northeast corner of the Arlington airport. The parcels are separated by a paved taxiway, as shown on Figure 2. The Phase 1 hanger is planned in the southern parcel and the Phase 2 hanger is planned in the northern parcel. A developed property is located to the north and a paved access drive is located along the south boundary. To the east is 59th Drive NE.

The site and the surrounding area are nearly level. The vegetation on the site is minimal with only grass-covered areas where the two hangers are planned and a row of low shrubs along portions of the site. There were no areas of standing water observed. 59th Drive NE did not appear to have a storm water collection system. It appears that runoff is infiltrated on the sides of the roadway in shallow trenches.

Geology

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 10,000 to 11,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, the Puget Sound region was overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. Part of a typical glacial sequence includes recessional outwash sand underlain by glacial till. We reviewed the Geologic Map of the Arlington West Quadrangle, Snohomish County, Washington (USGS 1985). The map indicates the site and the surrounding area are underlain by Qvrm, which is the recessional sand deposit of the Marysville Sand Member.

Our site explorations encountered recessional outwash consistent with the geologic map designation as Marysville Sand. Recessional outwash was not overridden by the glacial ice sheet and is generally loose to medium dense, not cemented, and moderate to well draining.

Explorations

Subsurface conditions were explored at the site on February 19, 2003, by excavating six test pits with a track-mounted excavator. The test pits were excavated to depths of 4.0 to 16.0 feet below the ground surface. The explorations were located in the field by a representative from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the test pits. The approximate locations of the test pits are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the test pits are presented in Figures 4 and 5.

Subsurface Conditions

A brief summary of conditions encountered in our explorations is included below. For a more detailed description review the test pit logs in Figures 4 and 5.

Our explorations encountered a very thin topsoil layer of only a few inches. A highly-weathered zone of orange-brown sand with silt extended to a depth of about 1.5 feet. Gray to tan weathered soils with silt extended to a depth of about 3.5 feet in 4 of the explorations and was not encountered in TP-5 and TP-3. Below the weathered soils, most of the explorations encountered fine to coarse sand with gravel and scattered cobbles with very coarse zones of gravel with sand and cobbles. This was the predominant soil encountered at each of the explorations at the planned infiltration areas (TP-1, TP-2, TP-3, and TP-4). In TP-3, there was a fine to medium sand with trace silt and gravel that extended to a depth of 8 feet. The more coarse sand and gravel was encountered between 8.0 and 14.5 feet. A fine to medium sand was encountered below a depth of 14.5 feet. In TP-1, TP-2, and TP-6, there were seams of a layer of fine to medium sand with trace silt at about 4 to 6 feet in depth.

Hydrologic Conditions

Ground water seepage was not encountered in our test pit explorations. Temporary ground water monitoring piezometers were installed at TP-1, TP-2, and TP-3 to the bottom of the exploration depths, which were 15.0, 14.0, and 16.0, respectively. We visited the site on March 11, 2003, and did not measure any water in the piezometers. Based on available information, the seasonal high ground water level is expected to be at least 14 feet below the ground surface.

Laboratory Results

A grain-size analysis was completed for a representative sample of the coarse sand and gravel deposit that is the predominant soil type below the planned infiltration systems. The results are presented on Figure 6. The sample was classified as gravel with sand since the gravel portion was slightly greater than the sand portion. The moisture content of the sample was 5.2 percent. The dry weight of the sample used in the grain size analysis was 2,097 grams.

CONCLUSIONS AND RECOMMENDATIONS

General

In our opinion, the site is suitable for the proposed improvements. The site soils should provide adequate support for the planned buildings and pavements provided the subgrade is prepared as recommended in this report.

Using the 1992 Department of Ecology manual, we interpret that the site soils are suitable for an ultimate infiltration rate of 8.2 inches per hour for a minimum depth of 3.5 feet and 20 inches per hour for a minimum depth of 6 feet. There may be areas where additional excavation is needed, as described in the **Infiltration** section of this report. Appropriate safety factors should be used in design and protection of the infiltration system should be maintained. It appears that the seasonal high water elevation at the site is at least 14 feet below grade. Therefore, there should be adequate separation between the bottom of the infiltration system and the seasonal high water table.

Infiltration

We have used the United States Department of Agriculture (U.S.D.A.) soil group classification (Figure III-3.1) as presented in the "Storm Water Management Manual for the Puget Sound Basin", (Ecology 1992) to classify the soil samples analyzed. A grain-size analysis was completed on a representative sample of the coarse deposit encountered at the site. The sieve results, and our observation, indicate this soil should be classified as coarse sand or gravel with sand. The fines content was just under 1 percent. Based on this manual, an infiltration rate of 20 inches per hour should be used for the design of the infiltration system. Therefore, an ultimate rate of 20 inches per hour for the infiltration system design is appropriate based on the use of the 1992 Department of Ecology manual. This rate is considered applicable for the coarse sand and gravel unit. To use this rate the bottom of the infiltration trenches must extend through beds of fine sand into the coarse sand and gravel deposit. In most cases, a minimum depth of 6 feet should be sufficient. In some cases, excavation may need to extend deeper than 6 feet as in TP-3 where the coarse soil was encountered at a depth of 8 feet.

If more shallow infiltration systems will be used, the design will have to consider the layers of fine to medium sand encountered in the upper soils. These soils meet the classification as sand on the 1992 Ecology manual and an ultimate infiltration rate of 8.2 inches per hour is obtained from the manual for these soils. The minimum depth for infiltration designed using this rate should be 4.0 feet. Again there is potential that limited overexcavation may be needed. Overexcavated material could be replaced with additional gravel.

Mr. Kearns indicated that the design will be completed based on one-half of the recommended infiltration rates presented above in order to provide a safety factor in the design. He indicated that the design would be completed based on infiltration out the bottom of the system and that infiltration on the sides is ignored.

Site Preparations and Grading

The first step of site preparation should be to strip the vegetation, topsoil, or loose soils to expose medium dense native soils in pavement and building areas. The stripped material should be removed from the site, or stockpiled for later use as landscaping fill. The resulting subgrade should be compacted to a firm, non-yielding condition. Areas observed to pump or weave should be repaired prior to placing hard surfaces.

Structural Fill

General: All fill placed beneath buildings, pavements or other settlement sensitive features, should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field-monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve.

The upper 1.5 to 3.5 feet of soils contain a moderate to high amount of fines. We expect these soils would be difficult to compact to structural fill specifications in wet weather. The soils below that depth generally contain only trace fines and should be suitable for use as structural fill in most weather conditions. Some addition of water may be needed in the summer season to obtain compaction with these deeper soils.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas, and within a depth of 2 feet below pavement and sidewalk subgrade, should be compacted to at least 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D 1557 compaction test procedure. Fill more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or ground water. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations, since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and ground water conditions encountered.

We anticipate temporary cuts for installation of utilities and the infiltration systems. For planning purposes, we recommend that temporary cuts over 4 feet deep be no greater than 1 Horizontal to 1 Vertical (1H:1V) where worker access is needed. We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. Cut slope heights and inclinations should also conform to local and WISHA/OSHA standards.

Foundations

Conventional shallow spread foundations should be founded on undisturbed, medium dense or better native soils or be supported on structural fill extending to those soils. If the soil at the planned bottom of footing elevation is not medium dense, it should be compacted to a medium dense condition or overexcavated to expose suitable bearing soil. The area of overexcavation should be filled with structural fill, or taller foundation stem walls may be used.

Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection and bearing capacity considerations. Minimum foundation widths of 16 and 24 inches should be used for continuous and isolated spread footings, respectively. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable design bearing pressure of 2,000 pounds per square foot (psf) be used for the footing design. Uniform Building Code (UBC) guidelines should be followed when considering short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between footings or across a distance of about 30 feet.

Lateral loads can be resisted by friction between the foundation and subgrade soil, and by passive soil resistance acting on the below-grade portion of the foundation. For the latter, the foundation must be poured "neat" against undisturbed soil or backfilled with clean, free-draining, compacted structural fill. Passive resistance may be calculated as a triangular equivalent fluid pressure distribution. We recommend that an equivalent fluid density of 220 pounds per cubic foot (pcf) be used to calculate the allowable lateral passive resistance for the case of a level ground surface adjacent to the footing. An allowable coefficient of friction between footings and soil of 0.45 may be used, and should be applied to the vertical dead load only. A factor of safety of 2.0 has been applied to the passive pressure to reduce the amount of deflection needed to generate passive resistance.

Slabs- On-Grade

Slab-on-grade areas should be prepared as recommended in the **Site Preparation and Grading** subsection. Slabs should be supported on medium dense native soils, or on structural fill extending to these soils. Where moisture control is a concern, we recommend that slabs be underlain by 6 inches of free-draining sand or gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting, should be placed over the capillary break. If desired, a sand blanket could be placed over the vapor barrier to aid in curing of the concrete.

Drainage

Final site grades should allow for drainage away from any buildings. We suggest that the finished ground surface be sloped at a gradient of 3 percent minimum for a distance of at least 10 feet away from the buildings.

Since the surrounding soils are fairly well drained, we do not consider that footing drains are necessary. However, if moisture control inside the structure is important, the slab should be a minimum of 1 foot above surrounding grades.

Where used, footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. Roof drains should not be connected to wall or footing drains.

Additional Comments on Protection of Infiltration Areas

Long term performance of infiltrations systems depends for the most part on limiting the amount of fine soil particles (fines) that enter the infiltration system. The bioswale should provide suitable removal of fines once it is well vegetated and provided that the water directed into the system is not heavily laden with fines. Storm water runoff from bare soil areas during site grading should not be allowed into any prepared infiltration areas.

USE OF THIS REPORT

We have prepared this report for Mr. Steve Johnson and his agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our work does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

We should be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, and to provide recommendations for design changes, should the conditions revealed during the work differ from those anticipated. As part of our services, we would also evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

Within the limitations of scope, schedule and budget for our work, we have strived to take care that our work has been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

o O o

We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

Sincerely,

Cornerstone Geotechnical, Inc.

Doug Bath, PE
Project Manager

3/14/03

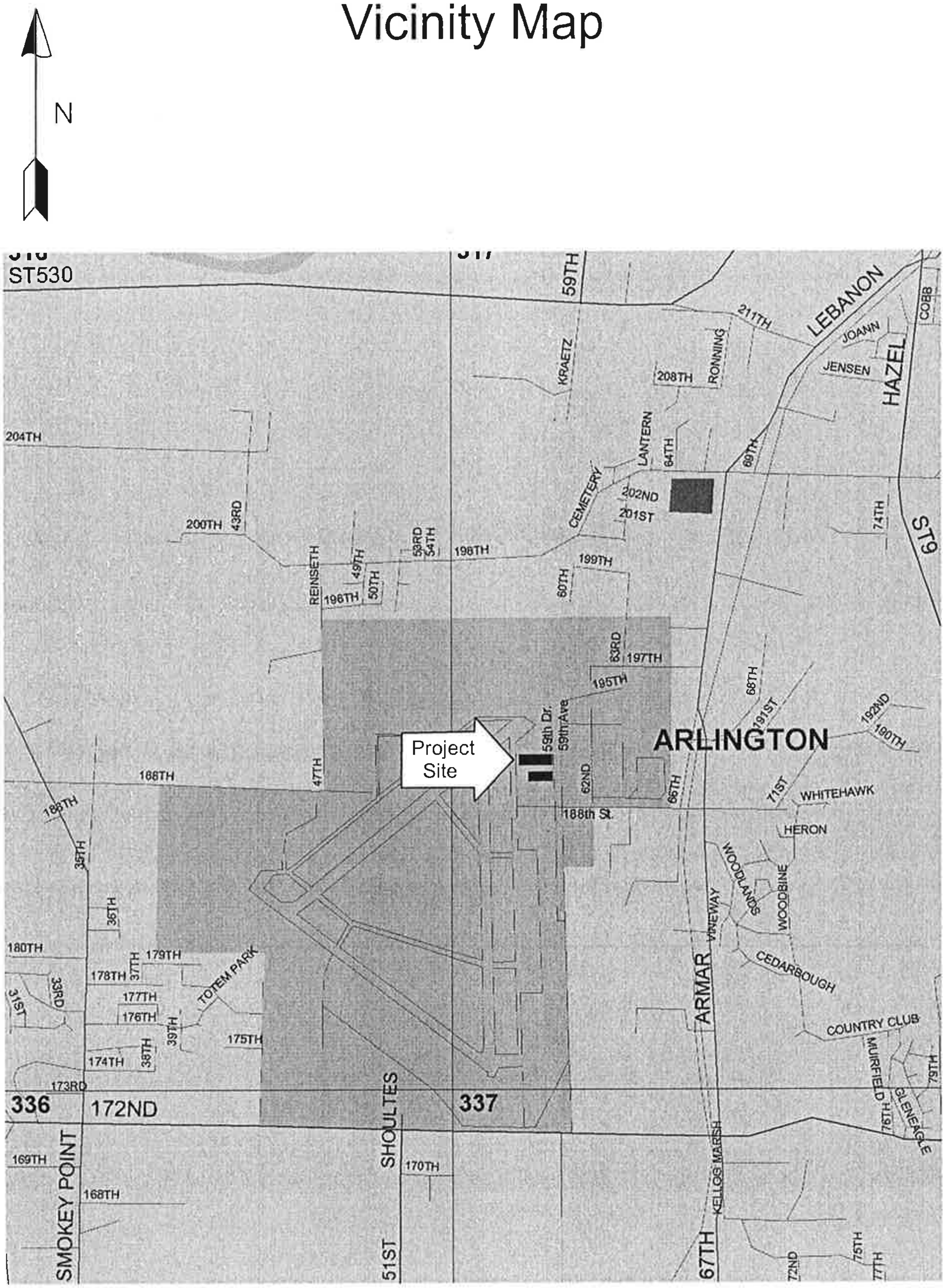
Charles P. Couvrette, PE
Principal Engineer

DJB:CPC:nt

One Copy Submitted
Six Figures

cc: Tom Kearns, Two Rivers Engineering (3 copies)

Vicinity Map



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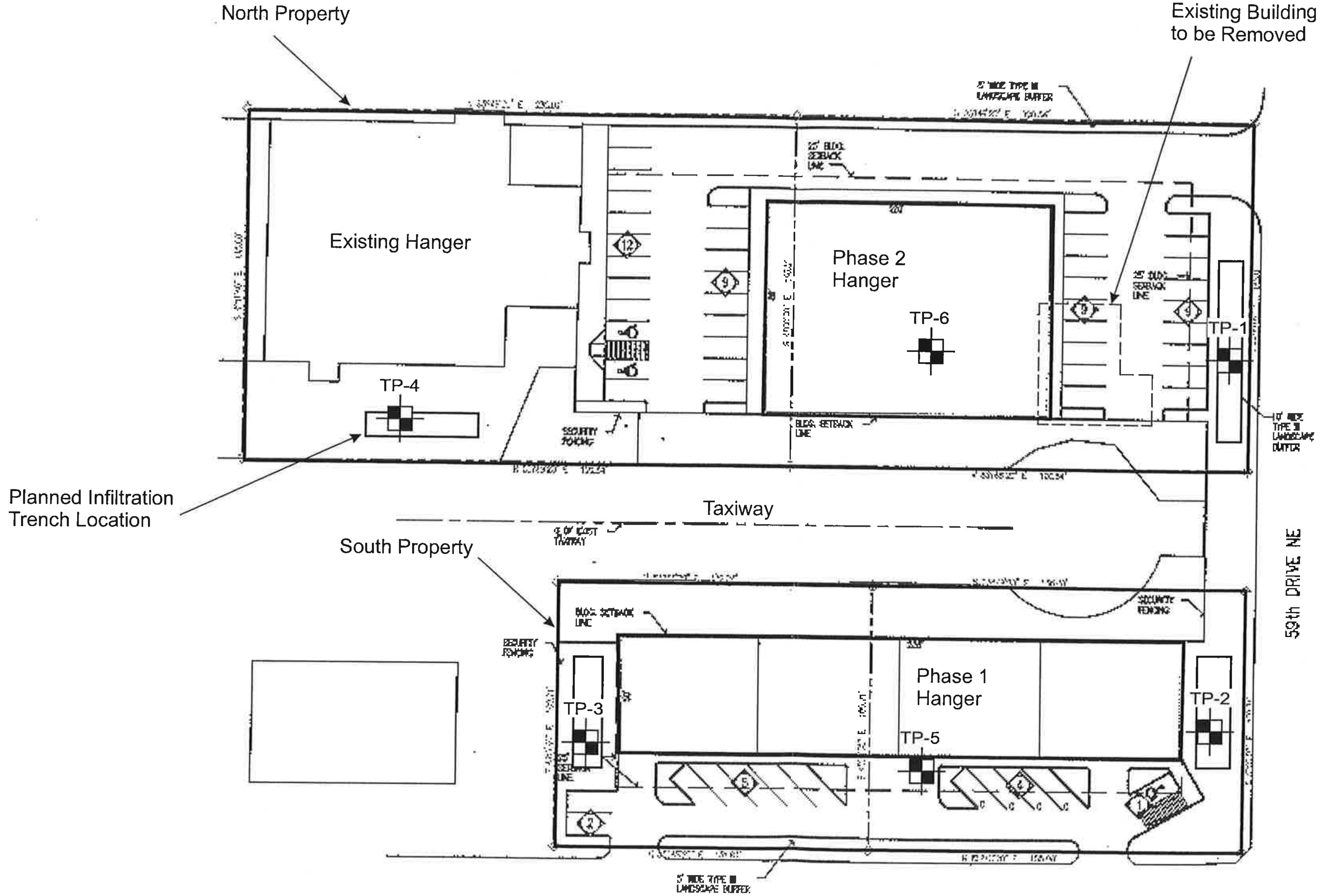
Johnson Hangers

File Number	Figure
1432	1

Site Plan



LEGEND

TP-6 — Test Pit Number and Approximate Location

0 50 100

Scale 1" = 50'

Reference: Site Plan based on drawing provided by Two Rivers Engineering.

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Johnson Hangers

File Number	Figure
1432	2

Unified Soil Classification System

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON number 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-83.
2) Soil classification using laboratory tests is based on ASTM D 2487-83.
3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS

Dry- Absence of moisture, dusty, dry to the touch

Moist- Damp, but no visible water

Wet- Visible free water or saturated, usually soil is obtained from below water table

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Unified Soil Classification System

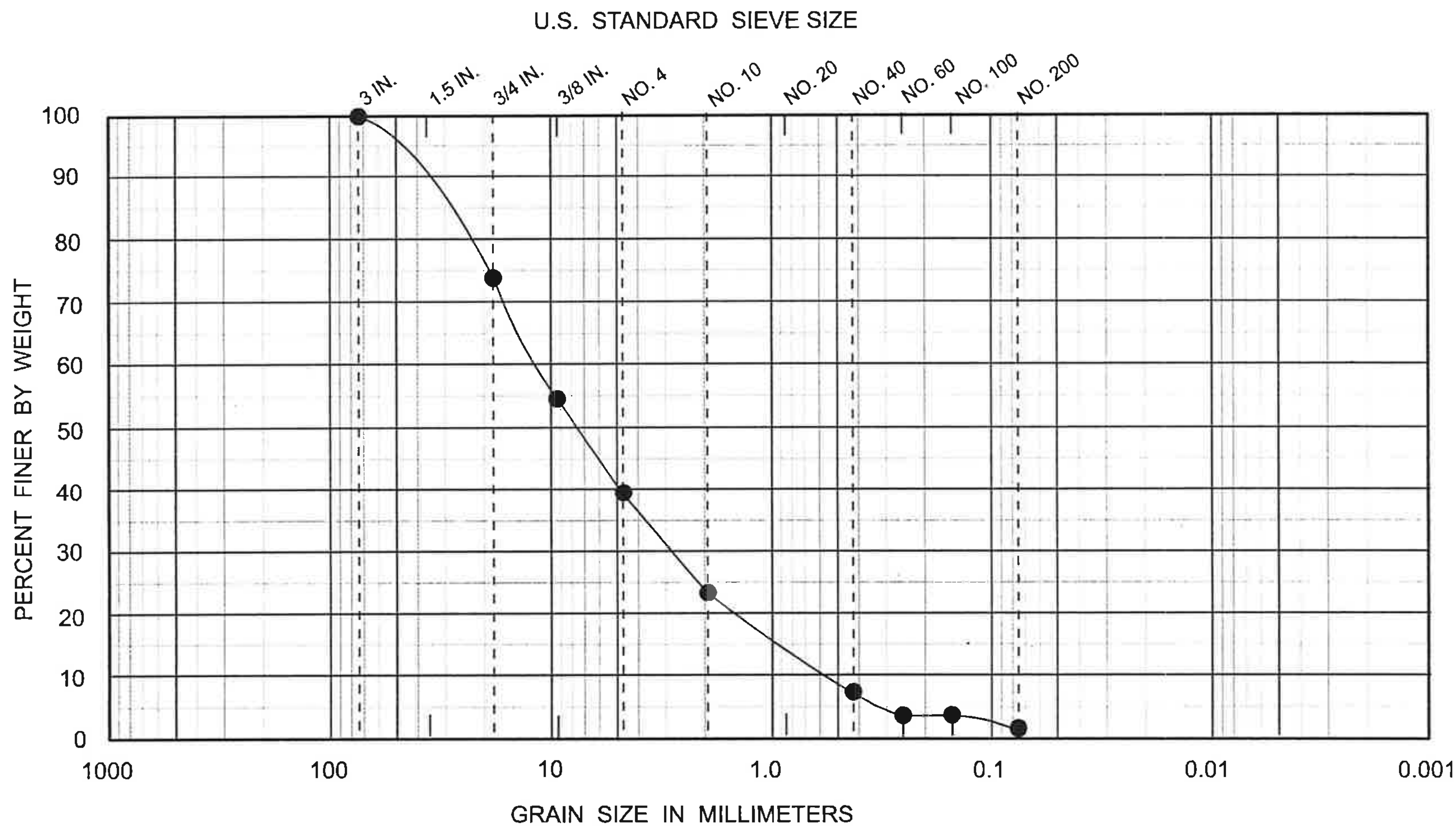
Figure 3

LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 0.4	SM	DARK BROWN SILTY FINE SAND WITH ORGANICS (LOOSE, WET) (TOPSOIL)
0.4 – 1.6	SP-SM	ORANGE-BROWN FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST TO WET)
1.6 – 2.8	SP-SM	GRAY FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE, MOIST)
2.8 – 3.2	SM	TAN SILTY FINE TO MEDIUM SAND (MEDIUM DENSE, MOIST)
3.2 – 4.8	SW/GW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SCATTERED COBBLES WITH ZONES OF GRAVEL WITH COBBLES AND SAND (MEDIUM DENSE, MOIST)
4.8 – 5.8	SP	GRAY FINE TO MEDIUM SAND WITH TRACE SILT (MEDIUM DENSE, MOIST)
5.8 – 15.0	SW/GW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SCATTERED COBBLES WITH ZONES OF GRAVEL WITH COBBLES AND SAND (MEDIUM DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 3.0, 5.0, AND 12.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS OBSERVED FROM 1 TO 7 FEET TEST PIT WAS COMPLETED AT 15.0 FEET ON 2/19/03
TEST PIT TWO		
0.0 – 0.2	SM	DARK BROWN SILTY FINE SAND WITH ORGANICS (LOOSE, WET) (TOPSOIL)
0.2 – 1.0	SP-SM	ORANGE-BROWN FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST TO WET)
1.0 – 2.6	SW-SM	GRAY FINE TO COARSE SAND WITH SILT AND GRAVEL (MEDIUM DENSE, MOIST)
2.6 – 3.2	SM	TAN, RUST MOTTLED, SILTY FINE TO MEDIUM SAND (MEDIUM DENSE, MOIST)
3.2 – 4.8	SW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SCATTERED COBBLES AND SLIGHT SILT COATING ON GRAVELS IN SOME POCKETS (MEDIUM DENSE, MOIST)
4.8 – 5.6	SP/SW	INTERBEDDED FINE TO MEDIUM SAND WITH TRACE SILT AND FINE TO COARSE SAND WITH GRAVEL. (MEDIUM DENSE, MOIST)
5.6 – 14.0	SW/GW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SCATTERED COBBLES WITH ZONES OF GRAVEL WITH COBBLES AND SAND (MEDIUM DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 3.0, 3.6, 5.0, 10.0, AND 14.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS OBSERVED FROM 1 TO 7 FEET TEST PIT WAS COMPLETED AT 14.0 FEET ON 2/19/03
TEST PIT THREE		
0.0 – 0.3	SM	DARK BROWN SILTY FINE SAND WITH ORGANICS (LOOSE, WET) (TOPSOIL)
0.3 – 1.5	SP-SM	ORANGE-BROWN FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST TO WET)
1.5 – 8.0	SP-SM	GRAY FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE, MOIST)
8.0 – 14.5	SW/GW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SCATTERED COBBLES WITH ZONES OF GRAVEL WITH COBBLES AND SAND (MEDIUM DENSE, MOIST)
14.5 – 16.0	SP	GRAY FINE TO MEDIUM SAND WITH TRACE SILT (MEDIUM DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 3.5, 8.5, 10.0, 14.6, AND 16.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED SLIGHT TEST PIT CAVING WAS OBSERVED FROM 1 TO 7 FEET TEST PIT WAS COMPLETED AT 16.0 FEET ON 2/19/03

LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT FOUR		
0.0 – 0.3	SM	DARK BROWN SILTY FINE SAND WITH ORGANICS (LOOSE, WET) (TOPSOIL)
0.3 – 1.6	SP-SM	ORANGE-BROWN AND BROWN FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST TO WET)
1.6 – 2.8	SP-SM	GRAY FINE TO MEDIUM SAND WITH SILT AND GRAVEL (MEDIUM DENSE, MOIST)
2.8 – 3.5	SM	TAN SILTY FINE TO MEDIUM SAND WITH ROOTS (MEDIUM DENSE, MOIST)
3.5 – 14.0	SW/GW	GRAY FINE TO COARSE SAND WITH GRAVEL AND SCATTERED COBBLES WITH ZONES OF GRAVEL WITH COBBLES AND SAND, MINOR SILT COATING ON SOME OF THE GRAVEL IN A FEW POCKETS (MEDIUM DENSE, MOIST)
		SAMPLE WAS COLLECTED AT 8.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS OBSERVED FROM 1 TO 10 FEET TEST PIT WAS COMPLETED AT 14.0 FEET ON 2/19/03
TEST PIT FIVE		
0.0 – 0.4	SM	DARK BROWN SILTY FINE SAND WITH ORGANICS (LOOSE, WET) (TOPSOIL)
0.4 – 1.7	SP-SM	ORANGE-BROWN FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST TO WET)
1.7 – 4.0	SP-SM	GRAY FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE, MOIST)
		SAMPLE WAS COLLECTED AT 1.5 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS OBSERVED FROM 1 TO 7 FEET TEST PIT WAS COMPLETED AT 4.0 FEET ON 2/19/03
TEST PIT SIX		
0.0 – 0.3	SM	DARK BROWN SILTY FINE SAND WITH ORGANICS (LOOSE, WET) (TOPSOIL)
0.3 – 1.8	SP-SM	ORANGE-BROWN FINE TO MEDIUM SAND WITH SILT AND TRACE GRAVEL (LOOSE TO MEDIUM DENSE, MOIST TO WET)
1.8 – 3.4	SP-SM	TAN FINE TO MEDIUM SAND WITH SILT (MEDIUM DENSE, MOIST)
3.4 – 5.5	SW/SP	GRAY FINE TO COARSE SAND WITH TRACE SILT AND GRAVEL WITH A 4-INCH THICK SEAM OF FINE TO MEDIUM SAND WITH TRACE SILT (MEDIUM DENSE, MOIST)
		SAMPLES WERE COLLECTED AT 3.0, 3.6, 5.0, 10.0, AND 14.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS OBSERVED FROM 1 TO 7 FEET TEST PIT WAS COMPLETED AT 5.5 FEET ON 2/19/03



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION
●	TP-3	8.5 feet	Gray gravel with fine to coarse sand and trace silt (D_{10} ~ 0.6 mm)

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Johnson Hangers

File Number	Figure
1432	6

Appendix C

Drainage Calculation Input/Output

Johnson Hangars Basin 1 E

Summary Report of all Detention Pond Data

Precipitation Data

Event	Precip (in)
2 year	1.8000
10	3.0000
100 year	3.7500

Record Id: 1 E Developed

Design Method	SBUH	**Rainfall type**	TYPE1A
Hyd Intv	10.00 min	**Peaking Factor**	484.00
		Abstraction Coeff	0.20
Total Area	0.55 ac	**DCIA**	0.00 ac
Pervious CN	92.55	**DC CN**	0.00
Pervious TC	5.83 min	**DC TC**	0.00 min

Pervious CN Calc

Description	SubArea	Sub cn
Impervious surfaces (pavements, roofs, etc)	0.45 ac	98.00
Open spaces, lawns,parks (>75% grass)	0.10 ac	68.00
Pervious Composited CN (AMC 2)		92.55

Pervious TC Calc

Type	Description	Length	Slope	Coeff	Misc	TT
Sheet	roof	60.00 ft	25.00%	0.0110	2.50 in	0.33 min
Channel (cont)	drain pipe	15.00 ft	2.00%	0.0110		0.02 min
Shallow	Paved and gravel areas (n=0.012)	125.00 ft	0.50%	0.0120		1.83 min
Channel (cont)	High grass (n=0.035)	160.00 ft	0.10%	0.0350		3.65 min
Pervious TC						5.83

Record Id: 30 x 5 Infil Stage Storage

Descrip:	30 x 5 1 SS 33 % Voids	Increment	0.10 ft
Start El.	291.5000 ft	Max El.	300.0000 ft

Stage Volume	
Stage (ft)	**Volume (cf)**
291.50	0.0000
293.50	256.0000
295.50	492.0000
300.00	1566.0000

Record Id: 30 x 5 10 in perc

Descrip:	30 x 10 x 1 SS basin 10 in per hr rate	Increment	0.10 ft
Start El.	293.0000 ft	Max El.	300.0000 ft

Stage (ft)	Discharge (cfs)
293.00	0.0350
300.00	0.1910

LPOOLCOMPUTE [30 x 5 Infil Pond] SUMMARY using Puls
[1 E Developed] Using [TYPE1A] As [10]
[1 E Developed] Using [TYPE1A] As [100 year]

Event	Match Q (cfs)	Peak Q (cfs)	Peak Stg (ft)	Vol (cf)	Vol (acft)	Time to Empty
10	0.0000	0.1500	5.1587	839.39	0.0193	24.60
100 year	0.0000	0.1910	7.0067	1140.09	0.0262	24.77

Johnson Hangars Basin 2 E

Summary Report of all Detention Pond Data

Precipitation Data

Event	Precip (in)
2 year	1.8000
10 year	3.0000
100 year	3.7500

Record Id: 2 E

Design Method	SBUH	**Rainfall type**	TYPE1A
Hyd Intv	2.00 min	**Peaking Factor**	484.00
		Abstraction Coeff	0.20
Total Area	0.77 ac	**DCIA**	0.00 ac
Pervious CN	95.27	**DC CN**	0.00
Pervious TC	2.35 min	**DC TC**	0.00 min

Pervious CN Calc

Description	SubArea	Sub cn
Impervious surfaces (pavements, roofs, etc)	0.70 ac	98.00
Open spaces, lawns,parks (>75% grass)	0.07 ac	68.00
Pervious Composited CN (AMC 2)		95.27

Pervious TC Calc

Type	Description	Length	Slope	Coeff	Misc	TT
Sheet	Pavement	110.00 ft	1.00%	0.0110	2.50 in	1.95 min
Channel (cont)	drain pipe	125.00 ft	0.50%	0.0110		0.40 min
Pervious TC						2.35 min

Record Id: 63 x 10 1/2 SS Stage Storage

Descrip:	63' x 10 ' 0.5 SS 33% void ratio	Increment	0.10 ft
Start El.	293.0000 ft	Max El.	300.0000 ft

Stage Volume	
Stage (ft)	**Volume (cf)**
293.00	0.0000
300.00	2102.0000

Record Id: 63 x 10 10 in perc Stage Discharge

Descrip:	63 x 10 infil 10 in per hr.	Increment	0.10 ft
Start El.	293.0000 ft	Max El.	300.0000 ft

Stage (ft)	Discharge (cfs)
293.00	0.1460
300.00	0.2750

LPOOLCOMPUTE [63 x 10 Infil basin] SUMMARY using Puls

[2 E] Using [TYPE1A] As [10 year]
[2 E] Using [TYPE1A] As [100 year]

Event	Match Q (cfs)	Peak Q (cfs)	Peak Stg (ft)	Vol (cf)	Vol (acft)	Time to Empty
10 year	0.0000	0.1957	2.6972	809.92	0.0186	24.27
100 year	0.0000	0.2242	4.2425	1273.96	0.0292	24.27

Johnson Hangars Basin 2W

Summary Report of all Detention Pond Data

Precipitation Data

Event	Precip (in)
2 year	1.8000
10 year	3.0000
100 year	3.7500

Record Id: 2 W

Design Method	SBUH	**Rainfall type**	TYPE1A
Hyd Intv	1.00 min	**Peaking Factor**	484.00
		Abstraction Coeff	0.20
Total Area	1.07 ac	**DCIA**	0.00 ac
Pervious CN	96.04	**DC CN**	0.00
Pervious TC	9.74 min	**DC TC**	0.00 min

Pervious CN Calc

Description	SubArea	Sub cn
Impervious surfaces (pavements, roofs, etc)	1.00 ac	98.00
Open spaces, lawns,parks (>75% grass)	0.07 ac	68.00
Pervious Composited CN (AMC 2)		96.04

Pervious TC Calc

Type	Description	Length	Slope	Coeff	Misc	TT
Sheet	Smooth Paved Surfaces.: 0.011	210.00 ft	1.00%	0.0110	2.50 in	3.27 min
Channel (cont)	pipe	80.00 ft	0.50%	0.0110		0.26 min
Shallow	High bioswale grass (n=0.035)	190.00 ft	0.10%	0.0350		6.21 min
Pervious TC						9.74 min

Record Id: 67 x 18 infil Stage Storage

Descrip:	67' x 18' vert 33% voids	Increment	0.10 ft
Start El.	293.0000 ft	Max El.	300.0000 ft

Stage Volume	
Stage (ft)	**Volume (cf)**
293.00	0.0000
300.00	2786.0000

Record Id: 67 x 18 10 in perc Stage Discharge

Descrip:	67' x 18' 10"/hr perc rate vert side walls	Increment	0.10 ft
Start El.	293.0000 ft	Max El.	300.0000 ft

Stage (ft)	Discharge (cfs)
293.00	0.2790
300.00	0.2790

LPOOLCOMPUTE [67 x 18] SUMMARY using Puls

[2 W] Using [TYPE1A] As [10 year]
[2 W] Using [TYPE1A] As [100 year]

Event	Match Q (cfs)	Peak Q (cfs)	Peak Stg (ft)	Vol (cf)	Vol (acft)	Time to Empty
10 year	0.0000	0.2790	2.2525	896.49	0.0206	25.08
100 year	0.0000	0.2790	3.9463	1570.61	0.0361	25.12

Appendix D

Operation & Maintenance

1.0 DRAINAGE SYSTEM – The drainage system for the Johnson Hangars is composed of three basic elements; collection, treatment and infiltration. The collection system is composed of a series of buried pipes, overland sheet flow and ditches. The treatment system is composed of bioswales and gravity oil water separators. The disposal system is composed of four infiltration basins.

2.0 OPERATION AND CONTROLS – In order for the system to function properly, several simple, routine procedures should be followed on a regular basis. Following these procedures will lengthen the life of the system and provide proper service. Lack of proper operation and maintenance will result in shorten system life, property damage and extra cost.

2.01 CONVEYANCE SYSTEM OPERATION – The conveyance system is composed of sheet flow over impermeable surfaces and pipe systems. The system shall be inspected on a semi annual basis to determine its condition. Areas of debris accumulation, broken features and areas in need of cleaning or repair shall be identified and corrected. There is one catch basin located at the east end of the taxiway that contains a spitter box. The purpose of the box is to direct ½ of the runoff to each infiltration basin to the north and south. The condition of the box and the level weir logs should be inspected to insure they are level and in good repair. Non-level stop logs or shorten weir lengths will modify flow distribution with disproportioning flow between the basins. Correct any condition to its original design state.

2.02 RUNOFF TREATMENT - The treatment system is composed of four bioswales located before each infiltration basin and oil/water separators. Bioswales grass shall be maintained in a healthy state with adequate fertilizer and water to maintain a healthy grown cover. The ground cover shall be maintained to a height of 3 to 6". If mowing is required, the clippings shall be removed from the area as to not cause a debris problem for downstream elements. The vegetation height shall be controlled to be above 3" entering into the dormant period. Impermeable pavement areas shall be swept at the same interval as City street maintenance. Pavement sweeping is recommended twice a year. Catch basins shall be inspected at the same time to remove accumulated sediment and debris. Remove sediment when depth is 33% or more of the catch depth. Inspect the oil/water tees installed in the infiltration catch basin. Remove any debris or accumulated oil or sediment. Repair the tee as necessary.

2.03 Infiltration basin - Inspect the inlet catch basin for accumulated debris sediment or other conditions. Remove or repair as necessary. Inspect the ground water level during a reasonable dry state when the basin should be empty. Open the observation well by removing the cap and noting any standing water. If standing water is observed, inspect surrounding observation wells to determine if a state of general high water exists. If after inspection, it is suspected a basin may have failed, inspect the piping system via closed circuit TV or other methods to determine if a break may have occurred. Repair the condition to return the basin to its proper function.

3.0 System Operation – The system should provide a reasonable period of satisfactory operation provided it is constructed with care and operated using sound procedures. Infiltration basins will become less permeable over time. The actual longevity of the system is dependent upon control of fine material entering the basin. Initial construction, pavement sweeping, catch basin cleaning and bioswale maintenance all impact the systems performance and longevity.