REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Aurora State Airport
Septic Drain Field Improvements for HDSE Sewer System
Aurora, Oregon
Project: AronFA-2-01

For
Aron Faegre and Associates
November 8, 2021

Project: AronFA-2-01

NV5

NV5

November 8, 2021

Aron Faegre and Associates
520 SW Yamhill Street, PH1
Portland, OR 97204

Attention: Aron Faegre

Report of Geotechnical Engineering Services
Aurora State Airport
Septic Drain Field Improvements for HDSE Sewer System
Aurora, Oregon
Project: AronFA-2-01

NV5 is pleased to present this report of geotechnical engineering services for subgrade improvements atop a proposed septic drain field for the HDSE sewer system in the runway safety area at the southern end of the Aurora State Airport located in Aurora, Oregon. Our services were conducted in accordance with our proposal dated August 26, 2021.

We appreciate the opportunity to be of continued service to you. Please call if you have questions regarding this report.

Sincerely,

NV5

Brett A. Shipton, P.E., G.E.
Principal Engineer

BAS:sn
Attachments
One copy submitted (via email only)
Document ID: AronFA-2-01-110821-geor.docx

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
CBR	California bearing ratio
DCP	dynamic cone penetrometer
ESWL	equivalent single wheel load
FAA	Federal Aviation Administration
pcf	pounds per cubic foot
psi	pounds per square inch

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for improving the subgrade atop a future drain field located at the southern end of the runway at the Aurora State Airport located in Aurora, Oregon. The same solution could be used for the existing drain fields if needed. Figure 1 shows the site relative to existing physical features.

The proposed drain fields are located in the runway safety area (RSA). The FAA Advisory Circular AC No. 150/5300-13A states that RSA be should be capable, "under dry conditions, of supporting snow removal equipment, aircraft rescue and fire fighting . . . equipment, and the occasional passage of aircraft without causing damage to the aircraft." It also states, "Compaction of RSAs must comply with Specification P-152, Excavation, Subgrade and Embankment, found in AC 150/5370-10."

According to the FAA Airport Construction Standards (AC150/5370-10) Item P-152, the subgrade outside of paved areas must be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698. No compaction is required in the top 4 inches of the subgrade, and any soil that has become compacted from construction or other traffic in the upper 4 inches must be scarified to a loose state.

From Item P152-2.1:
Areas outside the limits of the pavement areas where the top layer of soil has become compacted by hauling or other Contractor activities shall be scarified and disked to a depth of 4 inches (100 mm), to loosen and pulverize the soil. Stones or rock fragments larger than 4 inches (100 mm) in their greatest dimension will not be permitted in the top 6 inches (150 mm) of the subgrade.

From Item P152-2.6:
"On all areas outside of the pavement areas, no compaction will be required on the top 4 inches (100 mm), which shall be prepared for a seedbed in accordance with Item T-901, T-906."

From Item P152-2.10:
The subgrade in areas outside the limits of the pavement areas shall be compacted to a depth of 12 inches (300 mm) and to a density of not less than 95 percent of the maximum density as determined by ASTM D698.

Such stringent compaction is not permitted in the soil cover of drain fields, and this study provides recommendations for preparing a subgrade in the RSA over the drain fields that is capable, under dry conditions, of supporting snow removal equipment, aircraft rescue and fire fighting equipment, and the occasional passage of aircraft without causing damage to the aircraft.

2.0 PURPOSE AND SCOPE

The purpose of our scope was to provide recommendations for improving the soil cover over the drain fields such that it is capable, under dry conditions and without rigorous compaction, of supporting snow removal equipment, aircraft rescue and fire fighting equipment, and the occasional passage of aircraft without causing damage to the aircraft. Specifically, we have conducted the following tasks:

- Reviewed information provided to us by Aron Faegre and Associates and other available information in our files.
- Visited the site to observe the subgrade and conduct the following:
  - Collected bulk soil samples in order to establish moisture density relationships in accordance with ASTM D698
  - Measured the in situ density at the location of the proposed drain fields in general accordance with ASTM D6938, Procedure A, using a Troxler 3430 nuclear density gauge
  - Conducted DCP testing in general accordance with ASTM D6951 at the locations shown on Figure 2
- Conducted a laboratory testing program including proctor analyses in accordance with ASTM D698.
- Provided recommendations for subgrade stabilization that do not require significant compaction of the subgrade soil.
- Provided calculations showing that the subgrade atop the proposed drain fields can support emergency vehicles and occasional aircraft.
- Documented our findings, conclusions, and recommendations in this report.

3.0 SITE RECONNAISSANCE

Our site reconnaissance included collecting bulk samples to determine the moisture density relationship of the subgrade soil, conducting DCPs in order to estimate the resilient modulus of the subgrade, and measuring the in situ density of the subgrade soil. Figure 2 shows the locations of sampling and tests.

3.1 SOIL SAMPLING

Bulk soil samples were collected from the near-surface soil in the areas of the future drain fields. A moisture density relationship was determined on a combined bulk sample collected from the surface soil in the area of the proposed drain field. Groundcover at the sampling locations consisted of short grass. The vegetation was removed before sampling, and soil below a depth of 4 inches was placed in a sample bucket and transported to NV5's geotechnical laboratory in Wilsonville, Oregon, for testing. The soil was visually classified as silt in accordance with the soil classification system presented in Figure 3. A moisture density test was performed on the bulk sample in general accordance with ASTM D698. The test results are presented in Appendix A.

3.2 DCP TESTING

We performed DCP testing in general accordance with ASTM D6951 to estimate subgrade resilient modulus (M_r) at the locations shown on Figure 2. The DCP test results are presented on Appendix B. Since it is required that the upper 4 inches of the subgrade be loose, the upper 4 inches of soil was removed before testing was performed. We plotted the depth of penetration versus blow count and used the slope of the data to estimate the resilient modulus of the subgrade. We correlated the DCP test results to resilient modulus using the methods presented in *The Structural Design of Bituminous Roads*. The computed resilient modulus was converted to CBR using the following relationship:

CBR $= M_r/1500$

Table 1 summarizes the estimated resilient moduli and corresponding CBR for the subgrade.

Table 1. DCP Test Results and Corresponding CBR

Location	Resilient Modulus (psi)	CBR (percent)
DCP-1	24,300	16.2
DCP-2	18,700	12.5
DCP-3	21,200	14.1
DCP-4	14,000	9.3
DCP-5	12,400	8.3
DCP-6	18,000	12.0
DCP-7	10,400	6.9
DCP-8	8,800	5.9

Some of the DCP tests were performed at a depth of 12 inches in order to avoid damaging the drain pipe in the existing drain field.

3.3 IN SITU DENSITY

The in situ density was measured at the locations shown on Figure 2. The density measurements were conducted in accordance with ASTM D6938, Procedure A. Since it is required that the upper 4 inches of the subgrade be loose, the tests were performed deeper than than 4 inches below ground surface. The tests were compared to the maximum dry density determined in the laboratory. Table 2 presents a summary of the in situ density measurements.

Table 2. Measured In Situ Density

Location	Measured Dry Density (pcf)	Measured Moisture Content (percent)	Relative Density ASTM D698 (percent)
D-1	97.0	8.0	92[1]
D-2	89.1	8.3	85[1]
D-3	80.0	6.9	80[2]
D-4	83.4	8.5	84[2]
D-5	109.4	19.7	103[1]
D-6	101.1	21.3	95[1]
D-7	91.1	19.5	92[2]
D-8	87.1	22.4	88[2]

1. Based on a maximum dry density of 105.4 pcf and an optimum moisture content of 18.4 percent
2. Based on maximum dry density of 99.5 pcf and an optimum moisture content of 20.5 percent

We tested the compaction at the existing drain field at locations D-4 and D-8. The other locations were taken randomly throughout the site. The varying degrees of compaction found to exist in the RSA are summarized in Table 1.

Because the FAA's intent is that fire trucks and other vehicles may operate in the RSA, it brings up the question of whether relative compaction definitively relates to the depth of a vehicle rut in the RSA. Although the compaction does not meet the FAA requirement at some locations, the estimated resilient modulus indicates that the subgrade in these areas is capable of supporting similar wheel loads as the areas in which the compaction requirement is met.

4.0 PROPOSED DRAIN FIELD

The proposed drain field consists of a series of subsurface drainage trenches that are approximately 24 inches wide and approximately 3.5 to 4 feet on center. The base of each trench is to have a minimum depth of 18 inches below the capping fill. Twelve inches of ¾- to 2½-inch washed gravel will be placed in the trench. A perforated pipe will be placed in the washed gravel through which the effluent will be drained. A maximum of 10 inches of capping fill will be placed over the trench.

5.0 SUBGRADE IMPROVEMENT

The drain fields are located in the RSA of Aurora State Airport. The FAA Advisory Circular AC No. 150/5300-13A states that the RSA should be capable, ". . . under dry conditions, of supporting snow removal equipment, aircraft rescue and fire fighting . . . equipment, and the occasional passage of aircraft without causing damage to the aircraft." It also states, "Compaction of RSAs must comply with Specification P-152, Excavation, Subgrade and Embankment, found in AC 150/5370-10, which requires that upper 4 inches of the subgrade be uncompacted and scarified to be in a loose state." The underlying 12 inches of subgrade soil should be compacted to at least 95 percent of the maximum dry density, as determined by

ASTM D698. Because a drain field will be beneath the subgrade in the RSA, it cannot be compacted to the standard required by AC 150/5370-10. It must also be capable of growing vegetation.

We have considered the following design vehicles to model emergency equipment and aircraft that may traffic the RSA:

- Emergency Vehicle: AASHTO H20 or a 16,000-pound wheel load
- Aircraft: GulfStream G550 with a gross weight of 91,000 pounds or a 30,300-pound ESWL

To accommodate design traffic, the subgrade located over the drainage trenches should be stabilized using a product such as the Presto GeoSystems Geoweb. We have determined that the GW30V Geocells will create a subgrade that can support both the AASHTO H20 and Gulfstream 550 ESWL with an adequate margin of safety. Our supporting calculations are presented in Appendix C. Table 3 summarizes the input parameters and results of our analysis.

Table 3. Subgrade Stabilization

Design Vehicle	ESWL (pounds)	Tire Pressure (psi)	CBR Beneath Geoweb (percent)	Product Specification	Bearing Capacity Safety Factor
AASHTO H20	16,000	110	5	GW30V 6-inch depth	1.5
Gulfstream 550	30,300	200	5	GW30V 8-inch depth	1.3

A 6-inch-deep cell may be sufficient if the RSA is only subject to ESWLs of 16,000 pounds, such as those of the AASHTO H20 axle load. The geoweb cells should be filled with a blend of two-thirds crushed aggregate and one-third topsoil mix. The crushed aggregate should be 3/8 to 1 inch in nominal diameter and have a D50 of 0.5 inch and a void space of 30 percent. The geoweb should extend beyond each drainage trench by a distance of at least 18 inches. The geoweb should be overfilled by at least 1 inch with the selected fill. In addition, the geoweb should be installed in accordance with the manufacturer's recommendations. A 4-inch layer of loose, uncompacted material can be placed on the improved subgrade to meet the requirement of Item P152-2.6

6.0 LIMITATIONS

We have prepared this report for use by Aron Faegre and Associates and members of the design team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time our report was prepared. No warranty, express or implied, should be understood.

♠ ♠ ♠

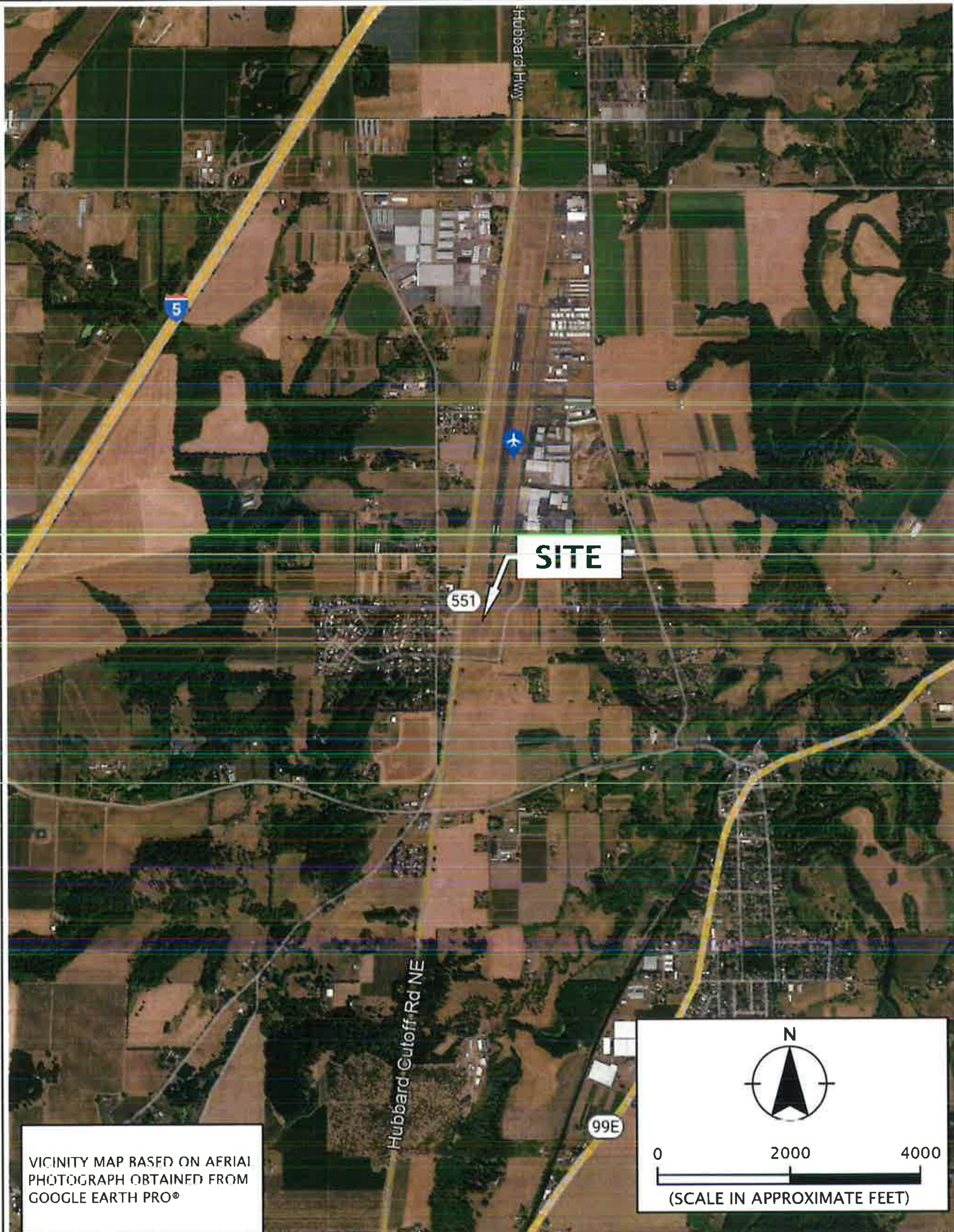
We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

NV5

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/22



NV5	ARONFA-2-01	VICINITY MAP	
	NOVEMBER 2021	SEPTIC DRAIN FIELD IMPROVEMENTS AURORA, OR	FIGURE 1

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LEGEND:

DCP-1 / D-1 — DCP TEST AND DENSITY MEASUREMENT

N

0 120 240

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®
SEPTEMBER 14, 2021

NV5

ARONFA-2-01

NOVEMBER 2021

SITE PLAN

SEPTIC DRAIN FIELD IMPROVEMENTS
AURORA, OR

FIGURE 2

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RELATIVE DENSITY - COARSE-GRAINED SOIL			
Relative Density	**Standard Penetration Test (SPT) Resistance**	**Dames & Moore Sampler (140-pound hammer)**	**Dames & Moore Sampler (300-pound hammer)**
Very loose	0 - 4	0 - 11	0 - 4
Loose	4 - 10	11 - 26	4 - 10
Medium dense	10 - 30	26 - 74	10 - 30
Dense	30 - 50	74 - 120	30 - 47
Very dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL				
Consistency	**Standard Penetration Test (SPT) Resistance**	**Dames & Moore Sampler (140-pound hammer)**	**Dames & Moore Sampler (300-pound hammer)**	**Unconfined Compressive Strength (tsf)**
Very soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 - 6	2 - 5	0.25 - 0.50
Medium stiff	4 - 8	6 - 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 - 25	9 - 19	1.0 - 2.0
Very stiff	15 - 30	25 - 65	19 - 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION	
Term	**Field Test**
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS					
Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	**Silt and Clay In: Fine-Grained Soil**	**Silt and Clay In: Coarse-Grained Soil**	**Percent**	**Sand and Gravel In: Fine-Grained Soil**	**Sand and Gravel In: Coarse-Grained Soil**
< 5	trace	trace	< 5	trace	trace
5 - 12	minor	with	5 - 15	minor	minor
> 12	some	silty/clayey	15 - 30	with	with
			> 30	sandy/gravelly	Indicate %

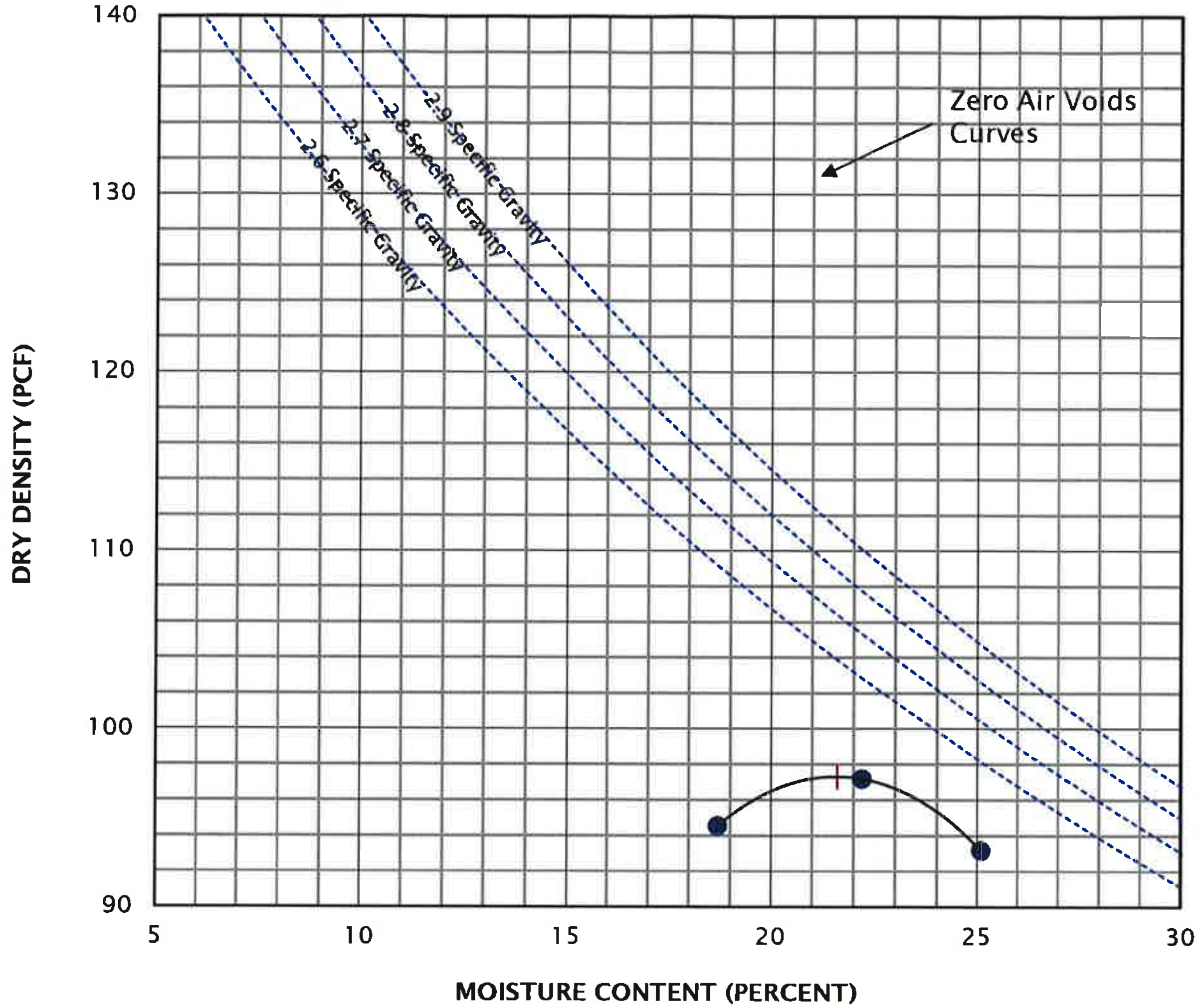
NV5	SOIL CLASSIFICATION SYSTEM	FIGURE 3

APPENDIX A

APPENDIX A

MOISTURE DENSITY RELATIONSHIP

We determined the moisture density relationship of samples collected from the near-surface soil at the location of the proposed drain field in general accordance with ASTM D698. The compaction curves for each sample are presented in this appendix.



EXPLORATION/ LOCATION	DEPTH (FEET)	SOURCE	MATERIAL DESCRIPTION
NA	NA	Onsite	Silt (ML) Existing Drain Fields

TEST RESULTS

TEST METHOD	AS RECEIVED MOISTURE CONTENT (PERCENT)	OVERSIZE (PERCENT)
Standard Proctor-ASTM D698 Method A	22.3	5.6

UNCORRECTED

MAXIMUM DRY DENSITY, lb/ft^3	**97.3**
OPTIMUM WATER CONTENT, %	**21.6**

OVERSIZE CORRECTION

MAXIMUM DRY DENSITY, lb/ft^3	**99.5**
OPTIMUM WATER CONTENT, %	**20.5**

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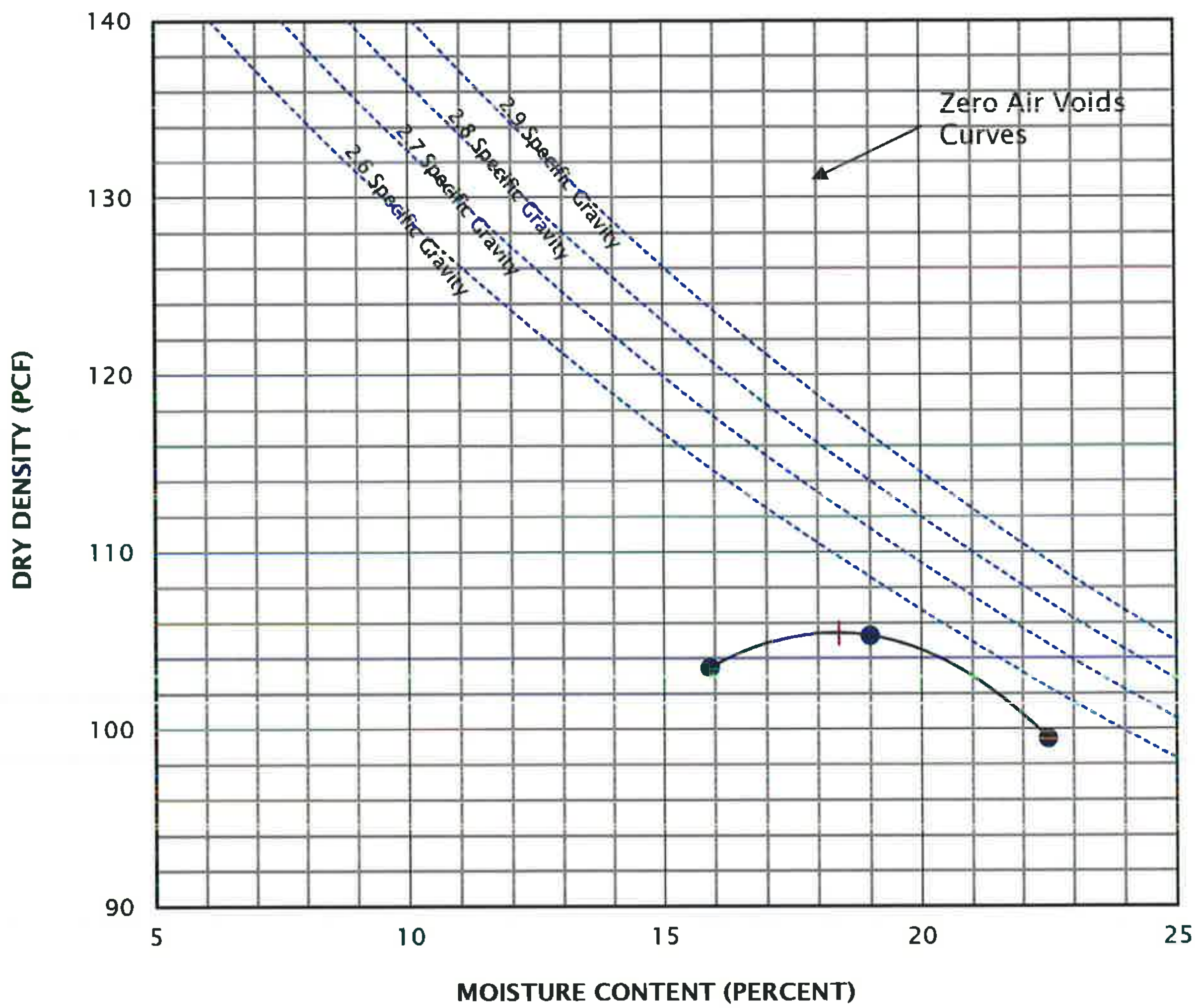
NV5	ARONFA-2-01	**LABORATORY TEST (MODIFIED PROCTOR)**	
	NOVEMBER 2021	SEPTIC DRAIN FIELD IMPROVEMENTS AURORA, OR	**FIGURE A-1**



EXPLORATION/ LOCATION	DEPTH (FEET)	SOURCE	MATERIAL DESCRIPTION
NA	NA	Onsite	Silt (ML) Proposed Drain Fields

TEST RESULTS

TEST METHOD	AS RECEIVED MOISTURE CONTENT (PERCENT)	OVERSIZE (PERCENT)
Standard Proctor-ASTM D698 Method A	19.0	0.1

UNCORRECTED

MAXIMUM DRY DENSITY, lb/ft^3	**105.4**
OPTIMUM WATER CONTENT, %	**18.4**

OVERSIZE CORRECTION

MAXIMUM DRY DENSITY, lb/ft^3	**NA**
OPTIMUM WATER CONTENT, %	**NA**

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NV5	ARONFA-2-01	**LABORATORY TEST (STANDARD PROCTOR)**	
	NOVEMBER 2021	SEPTIC DRAIN FIELD IMPROVEMENTS AURORA, OR	**FIGURE A-2**

APPENDIX B

APPENDIX B

DCP TESTING

We performed DCP testing at the locations shown in Figure 2. The tests were performed in general accordance with ASTM D6951. We correlated the DCP test results to resilient modulus using the methods presented in *The Structural Design of Bituminous Roads*. The results of each test are presented in this appendix.

DYNAMIC CONE PENETROMETER RESULTS - DCP 1

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	6.9	24,300
2	---	---	---
3	---	---	---

$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

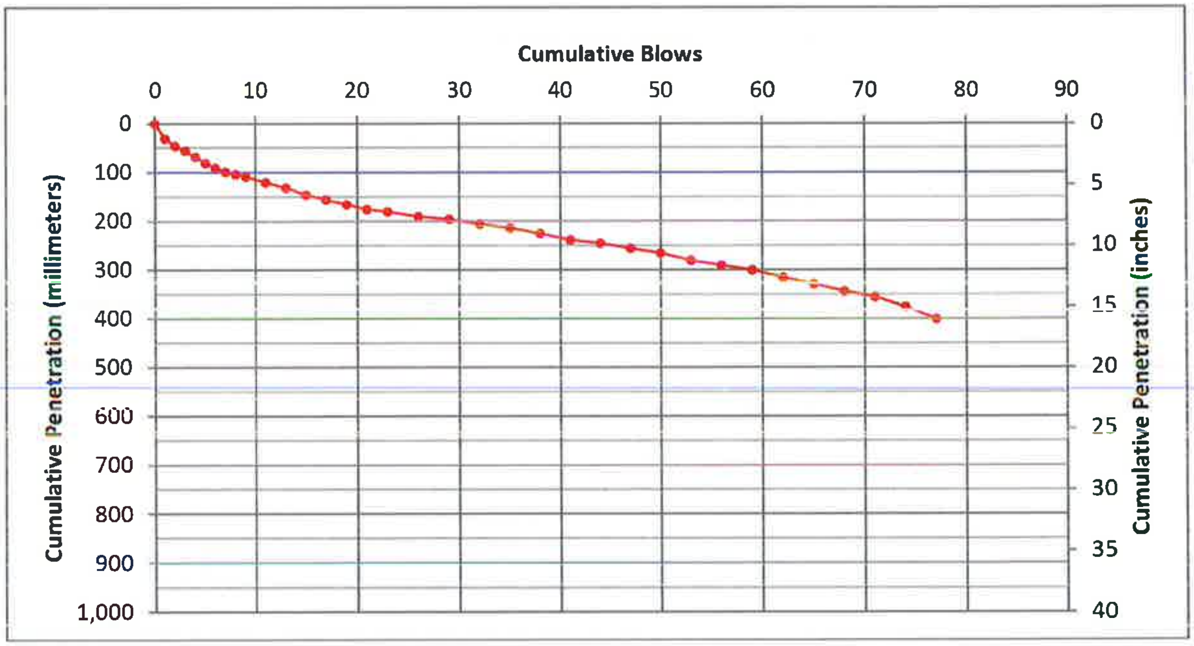
References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 2

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	9.9	18,700
2	---	---	---
3	---	---	---



$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

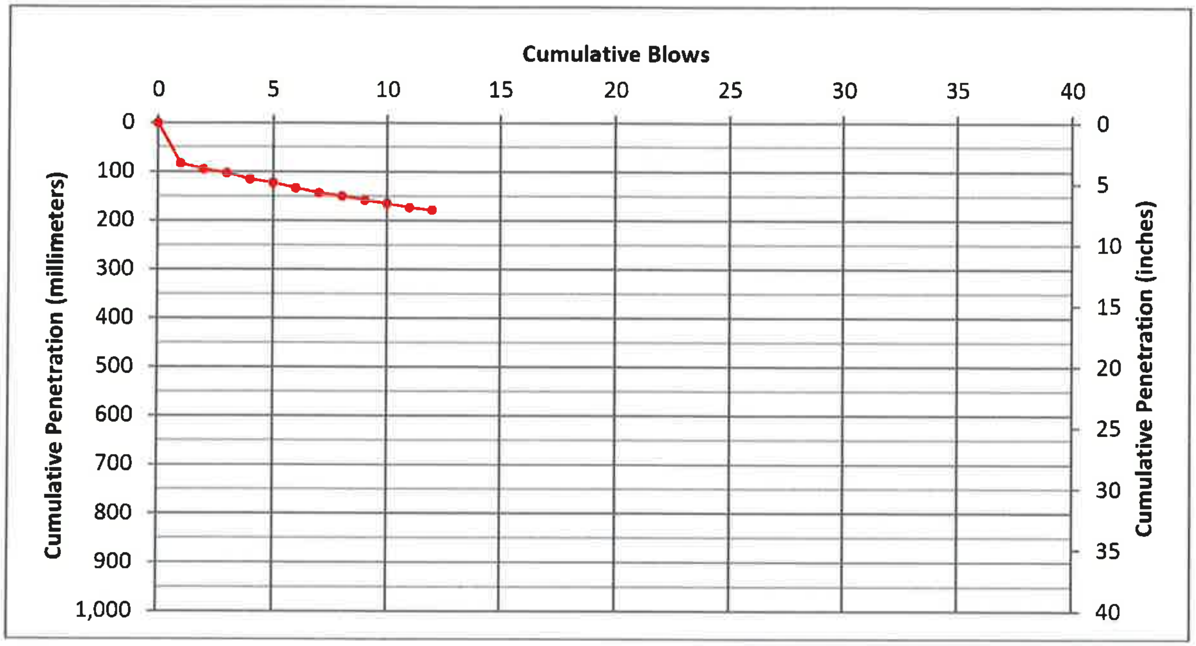
References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 3

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	8.3	21,200
2	---	---	---
3	---	---	---



$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

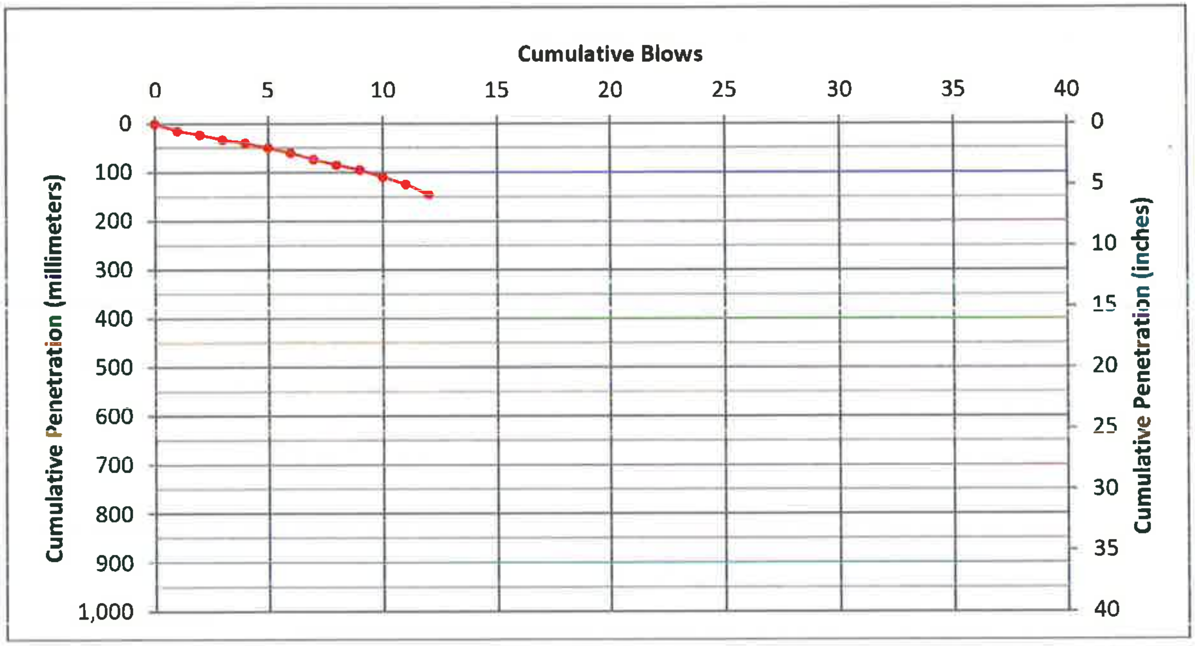
References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 4

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	14.8	14,000
2	---	---	---
3	---	---	---



$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 5

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	17.6	12,400
2	---	---	---
3	---	---	---

$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

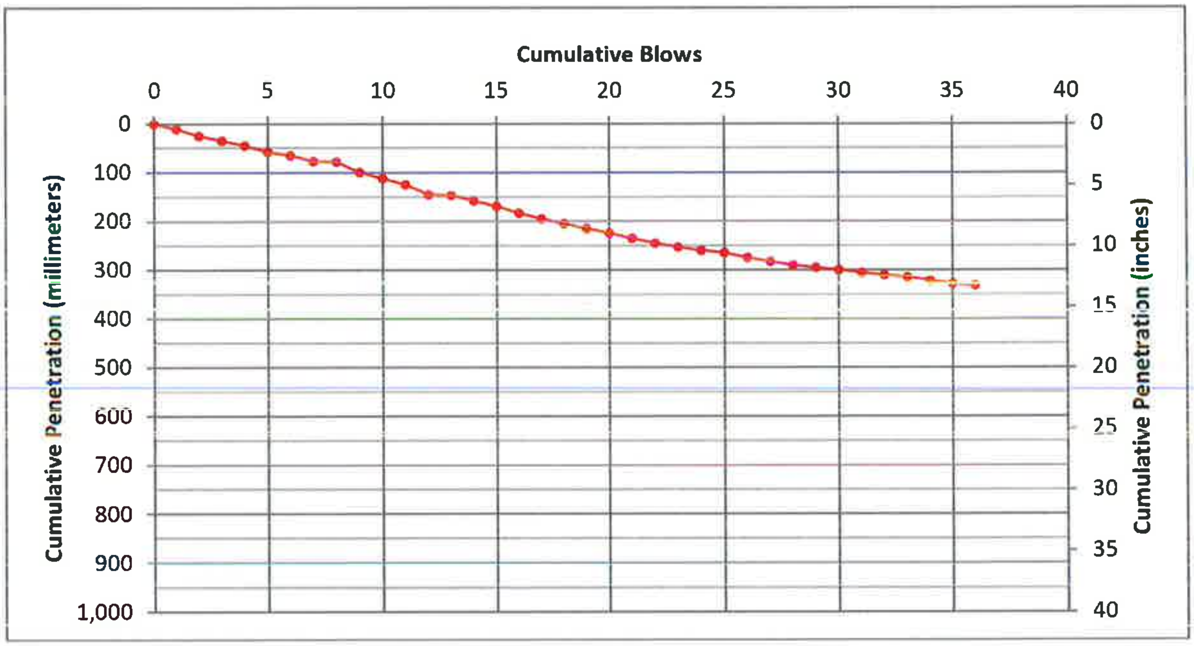
References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 6

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	10.4	18,000
2	---	---	---
3	---	---	---



$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

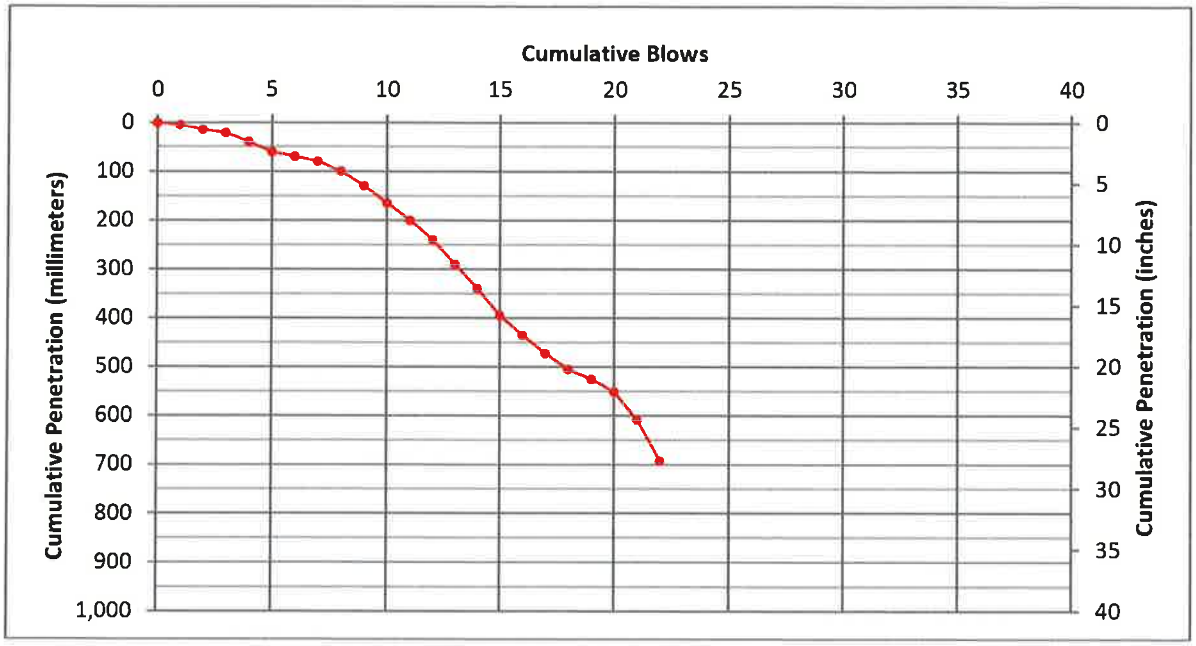
References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 7

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	22.5	10,400
2	---	---	---
3	---	---	---



$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R = resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

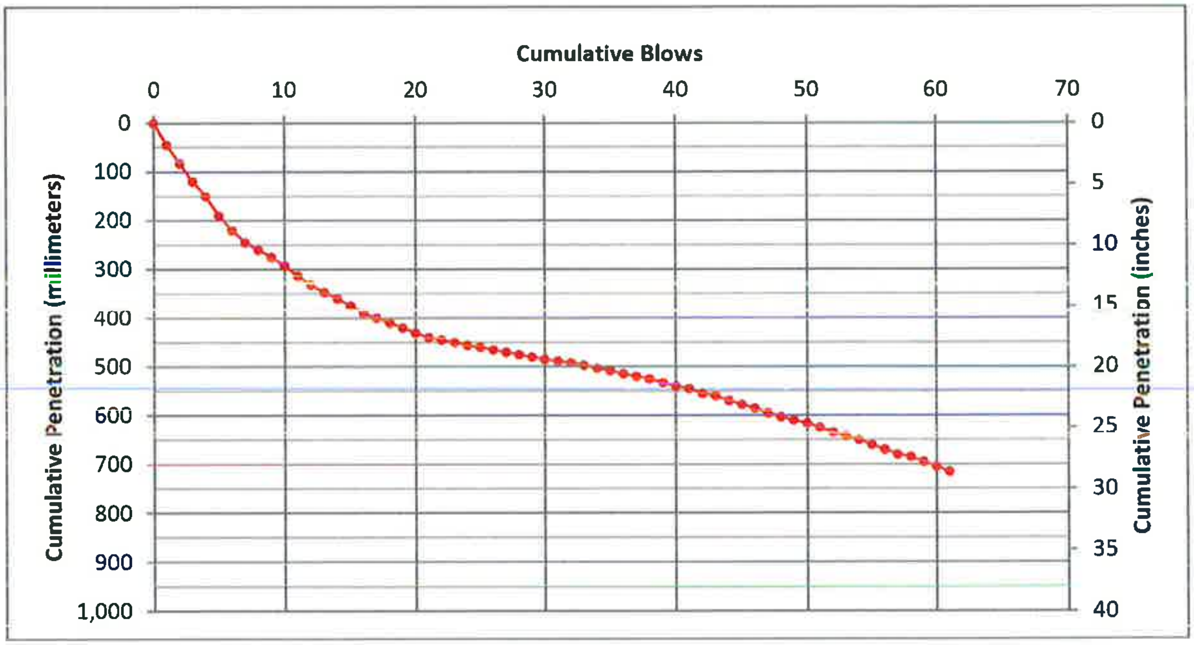
References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

DYNAMIC CONE PENETROMETER RESULTS - DCP 8

Layer	Soil Type	Hammer weight =	17.6 pounds
		Slope (mm/blow)	M_R (psi)
1	Soil not CL, CBR < 10 or not CH	28.1	8,800
2	---	---	---
3	---	---	---



$M_R = 96658 \times S^{-0.7168}$; soil not CL, CBR < 10 or not CH

$M_R = 469673 \times S^{-1.28}$; CL soil, CBR < 10

$M_R = 108206 \times S^{-0.64}$; CH soil

M_R – resilient modulus (pounds per square inch)

S = slope (millimeters per blow); multiplied by two (2) if 10-pound hammer is used

References:

ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer In Shallow Pavement Applications.

W.D. Powell, J.F. Foster, H.C. Mayhew, and M.E. Nunn, "The Structural Design of Bituminous Roads," TRRL Laboratory Report 1132, Transport and Road Research Laboratory, Department of Transport, United Kindgom, 1984.

APPENDIX C

APPENDIX C

DESIGN CALCULATIONS

This appendix presents our deign calculations for the use of Presto GeoSystems Geoweb for subgrade improvement.

AASHTO H20

CBR (%)	5
Cu (psi) from table 4.	21.7
Nc (low traffic, high rutting)	3.3
P (lb)	16000
p (psi)	100
r - see GW30V spec sheet	0.95
δ (deg)	26.6
Φ	28
Zt	1
Zb	7
H (in.)geoweb depth	6
D (in.)effective cell diam.	9.5

Variable Names

c_u	Subgrade shear strength
N_c	Bearing capacity coefficient - based on design traffic - see below
P	Design wheel load
p	Contact pressure
r	Geoweb cell wall/infill peak friction angle ratio
δ	Angle of shear resistance between the granular infill and Geoweb cell wall
ϕ	Angle of internal friction of the Geoweb infill material
z_t	Depth from surface to top of Geoweb cell walls
z_b	Depth from surface to bottom of Geoweb cell walls

Table 4 Correlation of Subgrade Soil Strength Parameters for Cohesive (Fine-Grained) Soils

California Bearing Ratio CBR (%)	Undrained Shear Strength c_u kPa (psi)	Standard Penetration Resistance SPT (blows/ft)	Field Identification
< 0.4	< 11.7 (1.7)	< 2	Very soft (extruded between fingers when squeezed)
0.4 - 0.8	11.7 - 24.1 (1.7) - (3.5)	2 - 4	Soft (molded by light finger pressure)
0.8 - 1.6	24.1 - 47.6 (3.5) - (6.9)	4 - 8	Medium (molded by strong finger pressure)
1.6 - 3.2	47.6 - 95.8 (6.9) - (13.9)	8 - 15	Stiff (readily indented by thumb but penetrated with great effort)
3.2 - 6.4	95.8 - 191 (13.9) - (27.7)	15 - 30	Very stiff (readily indented by thumbnail)
> 6.4	> 191 (27.7)	> 30	Hard (indented with difficulty by thumbnail)

N_C = 2.8 (High Traffic, Low Rutting - from U.S. Forest Service guidelines)
N_C = 3.3 Low Traffic, High Rutting - from U.S. Forest Service guidelines)

max allowable stress	qa (psi)	71.61	$q_a = N_C c_u$
radius of loaded area	R	7.1	where R = Radius of loaded area (i.e. effective radius of single or dual tires) $R = \sqrt{\frac{P}{p\pi}}$
vertical stress top of geoweb	σvt	99.7	
vertical stress bottom of geoweb	σvb	65.7	
Active earth pressure coefficient	Ka	0.4	
horizontal stress top of geoweb	σht	36.0	
horizontal stress bottom of geoweb	σhb	23.7	
average horizontal stress	σave	29.9	
stress reduction beneath loaded area	σr	18.9	

$$\sigma_{vt} = p\left[1-\left(\frac{1}{1+\left(\frac{R}{z_t}\right)^2}\right)^{3/2}\right] \qquad \sigma_{vb} = p\left[1-\left(\frac{1}{1+\left(\frac{R}{z_b}\right)^2}\right)^{3/2}\right]$$

$$\sigma_{ht} = K_a \sigma_{vt} \qquad \sigma_{hb} = K_a \sigma_{vb} \qquad \sigma_{avge} = \frac{(\sigma_{ht} + \sigma_{hb})}{2}$$

$$\sigma_r = 2\left(\frac{H}{D}\right)\sigma_{avge} \tan\delta$$

Allowable Stress on Subgrade	71.61
Stress on Subgrade	46.8
Factor of Safety	1.5 acceptable

Gulfstream 550

CBR (%)	5
Cu (psi) from table 4.	21.7
Nc (low traffic, high rutting)	3.3
P (lb)	30333
p (psi)	200
r - see GW30V spec sheet	0.95
δ (deg)	26.6
ϕ	28
Zt	1
7h	9
H (In.)geoweb depth	8
D (in.)effective cell dlam.	9.5

Variable Names

c_u	Subgrade shear strength
N_c	Bearing capacity coefficient - based on design traffic - see below
P	Design wheel load
p	Contact pressure
r	Geoweb cell wall/infill peak friction angle ratio
δ	Angle of shear resistance between the granular infill and Geoweb cell wall
ϕ	Angle of internal friction of the Geoweb infill material
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1.6 - 3.2	47.6 - 95.8 (6.9) - (13.9)	8 - 15	Stiff (readily indented by thumb but penetrated with great effort)
3.2 - 6.4	95.8 - 191 (13.9) - (27.7)	15 - 30	Very stiff (readily indented by thumbnail)
> 6.4	> 191 (27.7)	> 30	Hard (indented with difficulty by thumbnail)

N_c = 2.8 (High Traffic, Low Rutting - from U.S. Forest Service guidelines)
N_c = 3.3 Low Traffic, High Rutting - from U.S. Forest Service guidelines)

max allowable stress	qa (psi)	71.61	$q_a = N_c c_u$
radius of loaded area	R	6.9	where R = Radius of loaded area (i.e. effective radius of single or dual tires) $R = \sqrt{\frac{P}{p\pi}}$
vertical stress top of geoweb	σvt	199.4	$\sigma_{vt} = p\left[1 - \left(\frac{1}{1+\left(\frac{R}{z_t}\right)^2}\right)^{3/2}\right]$
vertical stress bottom of geoweb	σvb	100.8	$\sigma_{vb} = p\left[1 - \left(\frac{1}{1+\left(\frac{R}{z_b}\right)^2}\right)^{3/2}\right]$
Active earth pressure coefficient	Ka	0.4	
horizontal stress top of geoweb	σht	72.0	$\sigma_{ht} = K_a \sigma_{vt}$
horizontal stress bottom of geoweb	σhb	36.4	$\sigma_{hb} = K_a \sigma_{vb}$
average horizontal stress	σave	54.2	$\sigma_{avge} = \frac{(\sigma_{ht} + \sigma_{hb})}{2}$
stress reduction beneath loaded area	σr	45.7	$\sigma_r = 2\left(\frac{H}{D}\right)\sigma_{avge} \tan\delta$
Allowable Stress on Subgrade		71.61	
Stress on Subgrade		55.1	
Factor of Safety		1.30 acceptable	

Performance Handbook **Gulfstream G550**

Equivalent Single Wheel Loading (ESWL) GV-GER-1212

1. **Introduction:**

One consideration in operating Gulfstream aircraft is the strength of runway and taxiway pavements in relation to aircraft operating weight. This can limit operational weights in some airports. One common method of evaluating an aircraft for a given runway is the Equivalent Single Wheel Loading (ESWL). ESWL accounts for the extra tire flotation for multi-wheel landing gear struts such as the dual wheel struts used on the Gulfstream aircraft. This section provides information on how to compute ESWL for the G550 and G500 airplanes.

2. **G550 and G500 Main Landing Gear Parameters:**

Max Ramp Weight (pounds)	MLG Tire Size (inches)	Tire Spacing (inches)	Max Tire Pressure (psi)	Reduction Factor -	Maximum ESWL (pounds)
91,400	35 X 11.0	18.5	198	1.25	32,904

The reduction factor in the table above assumes a rigid pavement with a radius of equivalent stiffness of 40 inches, roughly equivalent to a 13.5 inch thick concrete slab. Thinner pavements would give higher reduction factors, so the factors presented are conservative.

3. **ESWL Computation for Lower Operating Weights:**

ESWL can be computed for lower operating weights as follows:
ESWL = (Gross Weight) x (0.9) x (0.5) / (Reduction Factor)

Aircraft	Gulfstream G550	
Gross Weight (lb)	91000	
Reduction Factor	1.35	assume 1.35, since rutting is allowed
ESWL (lb)	30333.33	
tire presure (psi)	200	

PRESTO | GEOSYSTEMS
STRENGTH FROM THE GROUND UP
Since 1979

Product Specification - GEOWEB® GW30V Geocells

GENERAL

GEOWEB® product is manufactured from textured, perforated strips of high density polyethylene that are bonded together to create a network of interconnected cells. The GEOWEB® cells can be filled with soil, aggregate, concrete, pulverized debris, recycled asphalt pavement, or other infill material for geotechnical applications such as: 1) load support for unpaved and paved roads, railways, ports, heavy-duty pavements, container yard, and basal embankments stabilization; 2) retaining structures, free-standing structures, and fascia walls; and, 3) slope, channel, and geomembrane protection.

DIMENSIONS

Parameter	Units	Value
Cell Depth (Available in 5 Depths)[1]	inches (mm)	3 (75), 4 (100), 6 (150), 8 (200), 12 (300)
Cell Size (Length x Width +/- 10%)	inches (mm)	11.3 x 12.6 (287 x 320)
Expanded Section Width	No. Cells	8
	Feet (m)	Varies: 7.7 to 9.2 (2.3 to 2.8)
Expanded Section Length	No. Cells	18, 21, 25, 29, or 34
	Feet (m)	Varies: 15.4 to 35.1 (4.7 to 10.7)

STRUCTURAL INTEGRITY AND SYSTEM PERFORMANCE

Parameter	Units	Value
Minimum Short Term Seam Peel Strength	lbf/in (N/cm)	≥80 (142)
Long-Term Seam Peel Strength (standard 4-inch sample width)[2]	lb (N)	160 (710)
Internal Junction Efficiency[3]	%	≥100
Mechanical Junction Efficiency (Connection Type: ATRA Key)[3]	%	≥100
Peak Friction Angle Ratio (δ/Ø)[4]	Unitless	0.95

MATERIAL PROPERTIES

Parameter	Test Method	Units	Value
Polymer Density	ASTM D1505 or D792	g/cm^3	0.935 - 0.965
Carbon Black Content[5]	ASTM D1603	%	1.5 - 2.0
Sheet Thickness Prior to Texture	ASTM D5199	mm (mil)	1.27 (50), -5% +10%
Sheet Thickness After Texture	ASTM D5199	mm (mil)	1.52 (60), -5% +10%
Texture Type/Shape	—	—	Rhomboidal
Texture Density	—	$indentations/cm^2$	22 - 31

DURABILITY

Parameter	Test Method	Units	Value
Environmental Stress Crack Resistance	ASTM D1693	hrs	>5,000
Resistance to Oxidation[6]	EN ISO 13438	yrs	≥50
Resistance to Weathering[7]	EN 12224	%	100

Notes:

1) 12-inch cell depth available in 21-cell panel length only.
2) A 100 mm (4.0 in.) wide seam sample shall support a 72.5 kg (160 lb) load for a period of 7 days minimum in a temperature-controlled environment undergoing a temperature change on a 10 hour cycle from ambient room to 54° C (130° F). Ambient room temperature is per ASTM E 41.
3) Junction efficiency determined as a percentage of junction performance (EN ISO 13426-1) to perforated strip performance (EN ISO 10319).
4) Typical design value for clean granular infill material (i.e. - coarse sand or crushed aggregate). Consult with manufacturer to confirm value for other types of infill materials.
5) Standard black HDPE strips. For tan/green GEOWEB, hindered amine light stabilizer (HALS) content will be 2.0% by weight of carrier.
6) Predicted to be durable for a minimum of 50 years in natural soil with a pH between 4 and 9 and at a soil temperature ≤ 25°C.
7) 100% of original tensile strength retained following exposure to intense UV radiation and accelerated weathering in accordance with EN 12224.

UK CA CE
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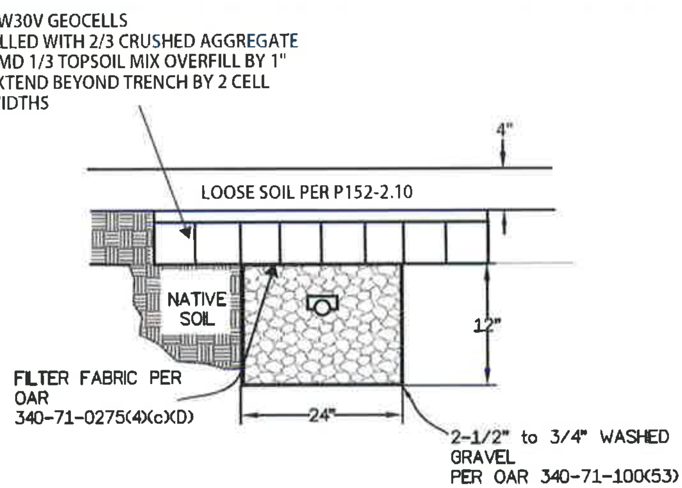

TYPICAL DETAIL (NOT TO SCALE)

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