Aron Faegre, AIA, PE   13200 Fielding Road   Lake Oswego   Oregon   97034   503-880-1469   faegre@earthlink.net

February 25, 2025

Alex Thomas, Planning and Programs Manager
Tony Beach, State Airports Manager
Oregon Department of Aviation
Brandy Steffen, JLA
Oregon Department of Aviation
3040 25th Ste SE
Salem, OR 97602
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Re: Aurora State Airport Master Plan Proposed Preferred Alternative
HDSE Septic Drainfield Correction of the Record and Next Steps Forward

Mr. Thomas, Mr. Beach, and Ms. Steffen:

This letter is to provide comment on ODAV's draft master plan for the Aurora State Airport, with particular attention to the issue of the HDSE drainfield at the south end of the airport. Please share this letter with the ODAV and FAA design team, and enter it into the record for the Oregon Department of Aviation's (ODAV) proposed "Preferred Alternative" for the Aurora State Airport Master Plan.

1. Correcting the Record: History of the HDSE Proposal to Strengthen the Drainfield– ODAV was not awaiting any answer from HDSE to ODAV questions. ODAV Advised HDSE that ODAV Preferred to study HDSE effluent being taken to the Columbia Helicopter drainfield.

There has been much discussion during the past two PAC meetings about HDSE's proposal to modify the existing drainfield in the Runway Safety Area at the south end of the runway, by using a top layer of modern geofabrics through which grass can grow up through. A very detailed geotechnical report by NV5 dated November 8, 2021 (Exhibit 6 to this letter) was provided to ODAV showing through engineering analysis of existing soils at the site that the proposed modification would result in soil strength consistent with FAA's guidance for soil in Runway Safety Areas.

During the PAC meetings, ODAV asserted that it had questions about the proposed modification to the existing drainfield that were not answered, and that it was HDSE's failure to respond that caused ODAV to decide to remove HDSE's drainfield.

For example, in the ODAV issued transcript of PAC Meeting #9 on February 11, 2025 Tony Beach states:

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[Tony Beach] 17:36:59
We have… gone thoroughly over the report that you've submitted and And we had
questions that as far as we're aware have not been answered.
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In fact, ODAV had asked numerous questions about the NV5 report, to which the NV5 and the HDSE team had provided written answers on December 20, 2021. See attached Exhibit 1. ODAV next asked follow-up questions in an email dated February 7, 2022. See attached Exhibit 2. The follow-up questions were extremely detailed geotechnical ones, the answers for which would surely trigger further ODAV questions. At this point drainfield strengthening was not an issue, just the specific design. Therefore, the HDSE team requested that all questions be answered in a meeting with ODAV's geotechnical engineer (GRI) and HDSE's engineer (NV5), to enable a drainfield strengthening plan to move forward.

Thus, ODAV's Tony Beach on February 7, 2022 issued an invitation for a Teams meeting on February 16, 2022 at 10am (see attached Exhibit 3: Aron Faegre meeting confirmation). The meeting invitation went to Tony Beach (ODAV), Betty Stansbury (then the ODAV director), James Kirby (an engineer with Century West), Tony Helbling (then the President of HDSE), Ted Millar (HDSE Board member), and Aron Faegre. Also attending the meeting was Brett Shipton (an engineer with NV5) using Aron Faegre's link. ODAV's last round of detailed questions were discussed and resolved with ODAV.

The next communication with ODAV was an email from Tony Beach on February 16, 2022, sent after the meeting (attached as Exhibit 4). It summarized the general conditions ODAV wished that the Runway Safety Area meet, and suggested the next step would be "stamped engineering plans that we can review before we agree." There were no more questions asked, that were unanswered.

However, the Exhibit 4 email added that ODAV was forbidding NV5 from speaking with ODAV's geotechnical engineer GRI. That email also suggested that HDSE continue to search for other drainfield locations, stating: "Have you considered locating the drainfields on the new Aurora Airport Business Center (AABC) property, or have you tried reaching out to HTS?" HDSE reported back verbally to ODAV that they had already searched for other locations, including AABC and HTS, and none were available.

The next step in trying to resolve this issue, was the ODAV Director Betty Stansbury and Aviation Board Chair Martha Meeker suggesting that perhaps the HDSE effluent could be piped to the north end of the airport, and use Columbia Helicopter's existing septic system and drainfield, located at the north end of the runway. ODAV indicated it believed that the Columbia Helicopter septic system had capacity for HDSE's effluent. HDSE agreed they would cooperate with this goal of looking at some wider options before settling on the geofabric option for the existing drainfield. This resulted in ODAV hiring Century West Engineering to do a study of the possibility of sending HDSE effluent (and perhaps other effluent from other airport businesses) to Columbia Helicopter's system. Tony Helbling (then HDSE's president) even provided volunteer assistance to this study by calling other airport companies to gather their effluent flow information to be used in ODAV's study.

A copy of an email from ODAV director Betty Stansbury is attached as Exhibit 5 which shows ODAV's continuing to examine piping HDSE effluent to the Columbia Helicopter drainfield. Director Stansbury's email also flagged FAA concerns but notes that "If a drainage field Engineer" were able to provide documented evidence that the drainage field will not compromise the safety area's load bearing capacity over the length of time the drainage fields remain under the safety area" that ODAV and FAA could "consider it acceptable." FAA noted that the drainfield was not funded by FAA grant money and so, in its view, if it remained in place as strengthened, would simply be considered a "nonstandard" condition – it did not require an MOS. We pause to point out here that FAA's claim that the strengthened the drainfield would be nonstandard, is technically inaccurate because strengthening the drainfield consistent with FAA's AC guidance would make the drainfield a wholly standard, not nonstandard, condition.

Regardless, in response to her Exhibit 5 email, HDSE pointed out to Director Stansbury that HDSE's 2021 geotechnical study provided the specific information by an engineering company – NV5 – that provides engineering expertise for airfields all over the United States, demonstrating that HDSE's proposed geofabric strengthening was consistent with FAA's AC guidance and the FAA representative's email that Stansbury cited in Exhibit 5.

Unfortunately, HDSE is still awaiting the outcome of that septic study by ODAV. But it is important to be clear that it is ODAV that had asked for a hold on the HDSE geofabric project. It is time for ODAV to share what their study found.

2. HDSE has submitted detailed geotechnical engineering analysis showing that the proposed reconstruction of the drainfield with geofabric will comply with FAA standards, and that the septic drainfield will continue to function per DEQ standards.

The REPORT OF GEOTECHNICAL ENGINEERING SERVICES, Aurora State Airport, Septic Drain Field Improvements for HDSE Sewer System, Aurora, Oregon November 8, 2021 was prepared by NV5 (https://www.nv5.com/) and is attached as Exhibit 6, for ease of reference. NV5 is an internationally recognized geotechnical firm with an office in Wilsonville, Oregon, and has provided extensive engineering work on airports all over the United States. The report was prepared by Brett Shipton, Principal Engineer, and contains his stamp as an Oregon Registered Professional Engineer. The report discusses FAA guidance in detail and shows that the use of the geofabric ensures the drainfields are fully consistent with the Runway Safety Area soil compaction guidance. When HDSE completes its work, its southend drainfield will not be a "nonstandard" condition.

The designer of the HDSE septic system, including the drainfield, is Environmental Management Systems (EMS) https://envmgtsys.com/ located in Portland, Oregon. A letter is attached as Exhibit 7 from EMS principal Bob Sweeney, confirming that the addition of the strengthening geofabric demonstrates that the drainfield will continue to operate fully in compliance with all DEQ standards for drainfields. Bob Sweeney was integral to suggesting that the geofabric material to strengthen the drainfield consistent with FAA's guidance.

Finally, my firm Aron Faegre, AIA, PE, Airport Planning is ready, willing, and able to oversee the project as a whole for HDSE. Aron Faegre is an architect, civil engineer, physicist, and pilot who has been the lead planner and designer on over two hundred airport planning and development projects in Oregon, Washington, California, New York, and British Columbia over the past 35 years. He has a Master of Architecture from MIT and a Bachelor of Physics from Reed College. It is noted that FAA's Airport Design AC150-5300-13B acknowledges that utility systems can be located in the Runway Safety Area as noted in Section 3.10.1.5 since it specifically discusses the requirements for "foundations, inlets, and manholes" that are located in the Runway Safety Area. Aron Faegre will coordinate additional civil engineering and survey work to ensure: a) overall longitudinal and transverse grading is fully consistent with FAA standards for Runway Safety Areas; b) all utility control boxes for valves and controls have traffic rated lids to match the soil load capacity requirements; and c) the drainfield area remains object free above ground per FAA standards.

Respectfully submitted,

Aron Faegre

Aron Faegre, AIA, PE
Aron Faegre Airport Planning and Design

Attached Exhibits:

- Exhibit 1: NV5 and Faegre response to ODAV Tony Beach by email December 20, 2021, 145 pages, which includes the Attachments 1 through 6.

- Exhibit 2: ODAV Tony Beach email February 7, 2022 at 8:06am with additional questions to HDSE, 10 pages.

- Exhibit 3: ODAV Tony Beach email February 7, 2022 invitation for an HDSE Teams Meeting February 16, 2022 at 10am, 1 page.

- Exhibit 4: ODAV Tony Beach email February 16, 2022 at 12:56pm following the HDSE meeting, 1 page.

- Exhibit 5: ODAV Betty Stansbury email May 26, 2022 discussing the Columbia Helicopters septic system option relative to HDSE, 2 pages.

- Exhibit 6: NV5 geotechnical report, REPORT OF GEOTECHNICAL ENGINEERING SERVICES, Aurora State Airport, Septic Drain Field Improvements for HDSE Sewer System, Aurora, Oregon November 8, 2021, 35 pages.

- Exhibit 7: EMS septic system letter: Suitability of Proposed Modifications to the Onsite Wastewater Treatment System Drainfield at Aurora State Airport, February 25, 2025, 6 pages.

Exhibit 1

NV5 and Aron Faegre response to ODA email from Tony Beach dated December 9, 2021 4:20 PM concerning HDSE drainfield expansion area at UAO December 20, 2021

Aron,

Here is our response to the questions from the airport's geotechnical consultant along with all of the attachments.

Brett

==

- Field Data Collection
  - Date of soil sampling
    We conducted 2 site visits: September 9, 2021 and October 11, 2021. The samples for proctor testing were collected on October 11, 2021.
  - Were any logs prepared to describe the bulk sampling results?
    Logs were not prepared for bulk samples. A bulk sample was collected from each area. Each bulk sample was not collected from a discrete test location. Soil collected from the testing locations were combined to form the bulk sample that was tested in the laboratory. Separate bulk samples from the existing and proposed drain field were prepared and tested in the laboratory.
  - Was a sieve analysis and/or Atterberg Limits test performed to validate the Silt visual classification?
    Sieve tests and/or Atterberg Limits tests were not conducted. The samples were visually classified in the field and in the laboratory. Other geotechnical studies at Aurora State Airport confirm our classification. Laboratory tests from these studies were used in conjunction with our visual classification to classify the soil. We have attached a copy of pertinent information from these studies (Attachment 1 - Lab Data).
  - Was infiltration testing performed? If not, why?
    Drain field design will be conducted by others and therefore we did not conduct infiltration testing as part of scope of services. A drain field feasibility study was conducted by Environmental Management Systems, Inc. A November 5, 2020 report that documents their study is attached (Attachment 2 - EMS drainfield feasibility report.pdf).
- As-builts or other construction documents pertaining to the existing drain field

To be provided by others. [Note: Attachment 6 added by Aron Faegre to this memo for providing this information to Tony Beach.]

- Report references
  - Geoweb design procedure
    The Geoweb design procedure is attached: "GeoWeb Load Support System, Technical Overview" (Attachment 3 - Geoweb Technical Overview.pdf)
  - Provide addition discussion on how the 6-inch geoweb, with 2/3 aggregate and 1/3 topsoil, replaces 12 inches of compacted soil.
    According to the FAA Airport Construction Standards (AC150/5370-10) Item P-152, the specified method of stabilizing the subgrade outside of paved areas is to compact the upper 12-inches to at least 95 percent of the maximum dry density, as determined by ASTM D698. It is further specified that the upper 4 inches must be scarified and be in a loose state. The intent of this is to provide a subgrade that can support snow removal equipment, aircraft rescue and firefighting equipment, and an occasional aircraft without causing damage to the aircraft. The intent of the geoweb is to provide a subgrade that will provide a subgrade that will support such traffic. It does so by confining the infill soil with the cells which gives the infill soil added shear strength when it is loaded from the top. It reduces the stress directly below the loaded area by transferring stress to the cell walls. Our calculation shows that the Geoweb provides a subgrade with an adequate factor of safety.
  - Equivalent Single Wheel Load source
    AASHTO H20: AASHTO HB-17 Standard Specifications for Highway Bridges, 17th Edition standard

    Gulfstream 550: Gulfstream Flight Ops, Operations Briefing, Pavement Weight Bearing Capacity (CAN/PCN) a copy is attached (Attachment 4 - Gulfstream Flight Ops.pdf)
  - Source identifying the critical aircraft type
    A Gulfstream G-V aircraft was selected based on a report prepared by Geotechnical Resources, Inc., dated September 16, 2019, that documents a pavement evaluation of Runway 17-35 at Aurora State Airport. We have attached a copy of that report (Attachment 5 – GRI Report)
- Report figures
  - Figure A-1: graphic does not show up in the provided pdf
  - Figure A-2: graphic does not show up in the provided pdf

We have attached another copy of our report that a shows Figures A-1 and A-2 when opened with Bluebeam Revu X64 Version 2016.5.1 and with Google Chrome Version 96.0.4664.110

- "Such stringent compaction is not permitted in the soil cover of drain fields"
  - Where does this statement come from?
    This statement was written by NV5 based on the requirement from drain filed designer that the drain field cover material must allow evapotranspiration and oxygen exchange to function efficiently. Compacted soil will inhibit both of these processes.

In addition to the list above, we will also need specifics on the proposed Geoweb reinforced drain field construction.

- Materials/Construction Proposed
  - What materials specification is to be used (ODOT, proprietary, etc.) for the aggregate?
    Per the GeoWeb Manufacturer the infill material should consist of one third pulverized topsoil and two thirds crushed aggregate. The aggregate portion should be crushed rock that has a particle size range from 0.375 to 1.0 inches with a D50 of 0.5 inches and a 30 percent void space. The engineered fill should lightly be compacted to allow vegetation growth.

  - What compaction specifications and test methods are proposed to achieve the proposed Geoweb strengths?
    After the cells have been filled the prepared ground surface should be proofrolled with a fully loaded dump truck. Some rutting and deflection is acceptable considering that the FAA specifies the upper 4-inches of subgrade consist loose uncompacted soil over 12-inches of compacted subgrade.

  - What compaction specifications and test methods are proposed for soil layers to be placed along with the Geoweb?
    The only other soil that will be placed is the washed gravel or drain rock in the drainage trenches. We recommend only light compaction of this material until it is well keyed. Even at this level of compaction we believe its load bearing characteristics will be superior to the soil that exists in the RSA. Over compacting this material will inhibit its drainage characteristics

- What subgrade compaction specifications and test methods are proposed for the expanded drain field areas?
  See our response to the two prior questions.

- What materials are proposed for use in the rest of the elements of the drain field system (pipes, manifolds, perf spec., etc.)?
  To be addressed by others. [[Note: Attachment 6 added by Aron Faegre to this memo for providing this information to Tony Beach.]

Attachments:

Attachment 1 – Lab Data
Attachment 2 – EMS drainfield feasibility report
Attachment 3 – Geoweb Technical Overview
Attachment 4 – Gulfstream Flight Ops
Attachment 5 – GRI Report
Attachment 6 – Construction Documents for HDSE Drainfield

Exhibit 1, Attachment 1

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	CENTEREXCON-4-01	**VICINITY MAP**	
	MARCH 2019	AURORA AIRPORT FUEL FARM AURORA, OR	**FIGURE 1**

Printed By: mmiller | Print Date: 3/6/2019 4:52:23 PM
File Name: J:\A-D\CentrexCon\CentrexCon-4\CentrexCon-4-01\Figures\CAD\CentrexCon-4-01-VM01.dwg | Layout: FIGURE 1

LEGEND:

— - - —	SITE BOUNDARY
B-1	BORING
CPT-1	CONE PENETRATION TEST

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, MARCH 6, 2019

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CENTEREXCON-4-01

SITE PLAN

MARCH 2019

AURORA AIRPORT FUEL FARM
AURORA, OR

FIGURE 2

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling one boring (B-1) to a depth of 26.5 feet BGS and completing one CPT probe (CPT-1) to a depth of approximately 58.7 feet BGS. The boring was drilled on February 22, 2019 using a trailer-mounted drill rig and solid-stem drilling techniques by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon. The exploration log is presented in this appendix. The CPT data are presented in Appendix B.

The approximate locations of the explorations are shown on Figure 2. Exploration locations were chosen based on preliminary site plan provided to our office by N.D. Eryou, PhD, P.E. The exploration locations were determined by pacing from existing site features and should be accurate implied by the methods used.

SOIL SAMPLING

Samples were collected from the boring using 1½-inch-inner diameter SPT split-barrel sampler in general accordance with ASTM D1586. The sampler was driven into the soil with a 140-pound hammer free-falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration log, unless otherwise noted. Samples were generally collected at 2.5- to 5-foot intervals throughout the depth of the boring. In addition, relatively undisturbed samples were collected by pushing thin-walled standard Shelby tubes into the base of the exploration in general accordance with ASTM D1587. Sampling methods and intervals are shown on the exploration log.

We understand that calibration of the SPT hammer used by Dan J. Fischer Excavating, Inc. has not been completed. The SPT blows completed by Dan J. Fischer Excavating, Inc. were conducted using two wraps around a cathead.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Explorations Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration log indicates the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration log.

LABORATORY TESTING

We visually examined soil samples collected from the exploration to confirm field classifications. We also performed the following laboratory testing.

MOISTURE CONTENT

We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

ATTERBERG LIMITS TESTING

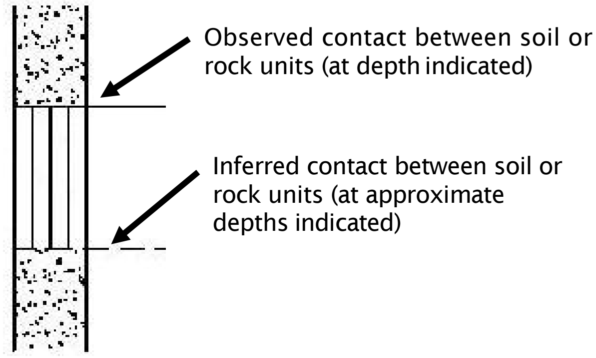
Atterberg limits (plastic and liquid limits) testing was performed on a select soil sample in general accordance with ASTM D4318. The plastic limit is defined as the moisture content where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSES

Particle-size analysis was completed on select soil samples in general accordance with ASTM D1140. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
◧	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
◧	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▤	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample obtained using Dames & Moore sampler and 140-pound hammer or pushed with recovery
⊠	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
⊠	Location of grab sample
▨	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Nonplastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration 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 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:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	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 %

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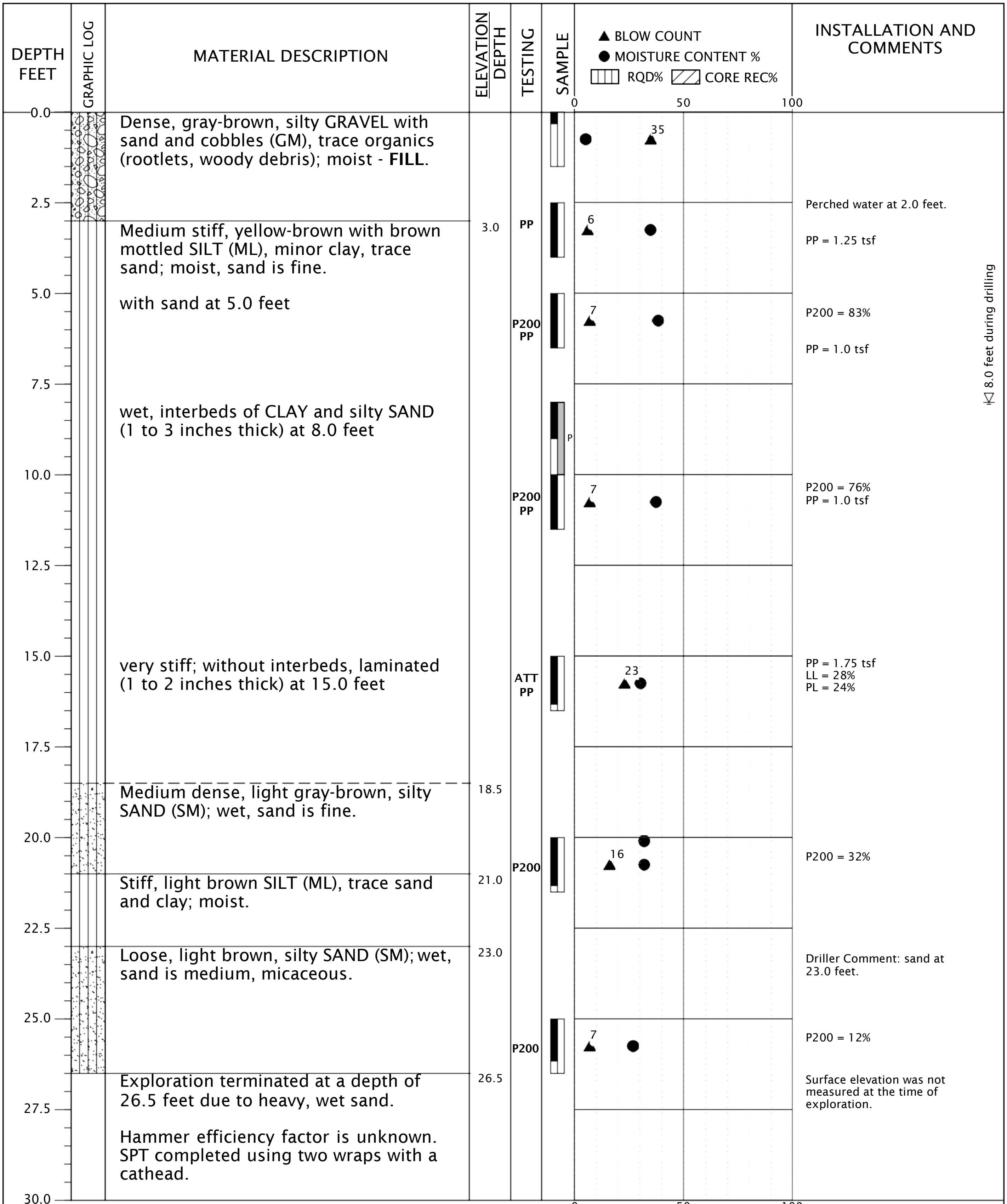
SOIL CLASSIFICATION SYSTEM

TABLE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Dense, gray-brown, silty GRAVEL with sand and cobbles (GM), trace organics (rootlets, woody debris); moist - **FILL**.			35	
2.5					Perched water at 2.0 feet.
	Medium stiff, yellow-brown with brown mottled SILT (ML), minor clay, trace sand; moist, sand is fine.	3.0	PP	6	PP = 1.25 tsf
5.0	with sand at 5.0 feet		P200 PP	7	P200 = 83% PP = 1.0 tsf
7.5	wet, interbeds of CLAY and silty SAND (1 to 3 inches thick) at 8.0 feet				
10.0			P200 PP	7	P200 = 76% PP = 1.0 tsf
12.5					
15.0	very stiff; without interbeds, laminated (1 to 2 inches thick) at 15.0 feet		ATT PP	23	PP = 1.75 tsf LL = 28% PL = 24%
17.5	Medium dense, light gray-brown, silty SAND (SM); wet, sand is fine.	18.5			
20.0			P200	16	P200 = 32%
	Stiff, light brown SILT (ML), trace sand and clay; moist.	21.0			
22.5	Loose, light brown, silty SAND (SM); wet, sand is medium, micaceous.	23.0			Driller Comment: sand at 23.0 feet.
25.0			P200	7	P200 = 12%
	Exploration terminated at a depth of 26.5 feet due to heavy, wet sand. Hammer efficiency factor is unknown. SPT completed using two wraps with a cathead.	26.5			Surface elevation was not measured at the time of exploration.
27.5					
30.0					

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: J. Hook

COMPLETED: 02/22/19

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 4 inches

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CENTREXCON-4-01

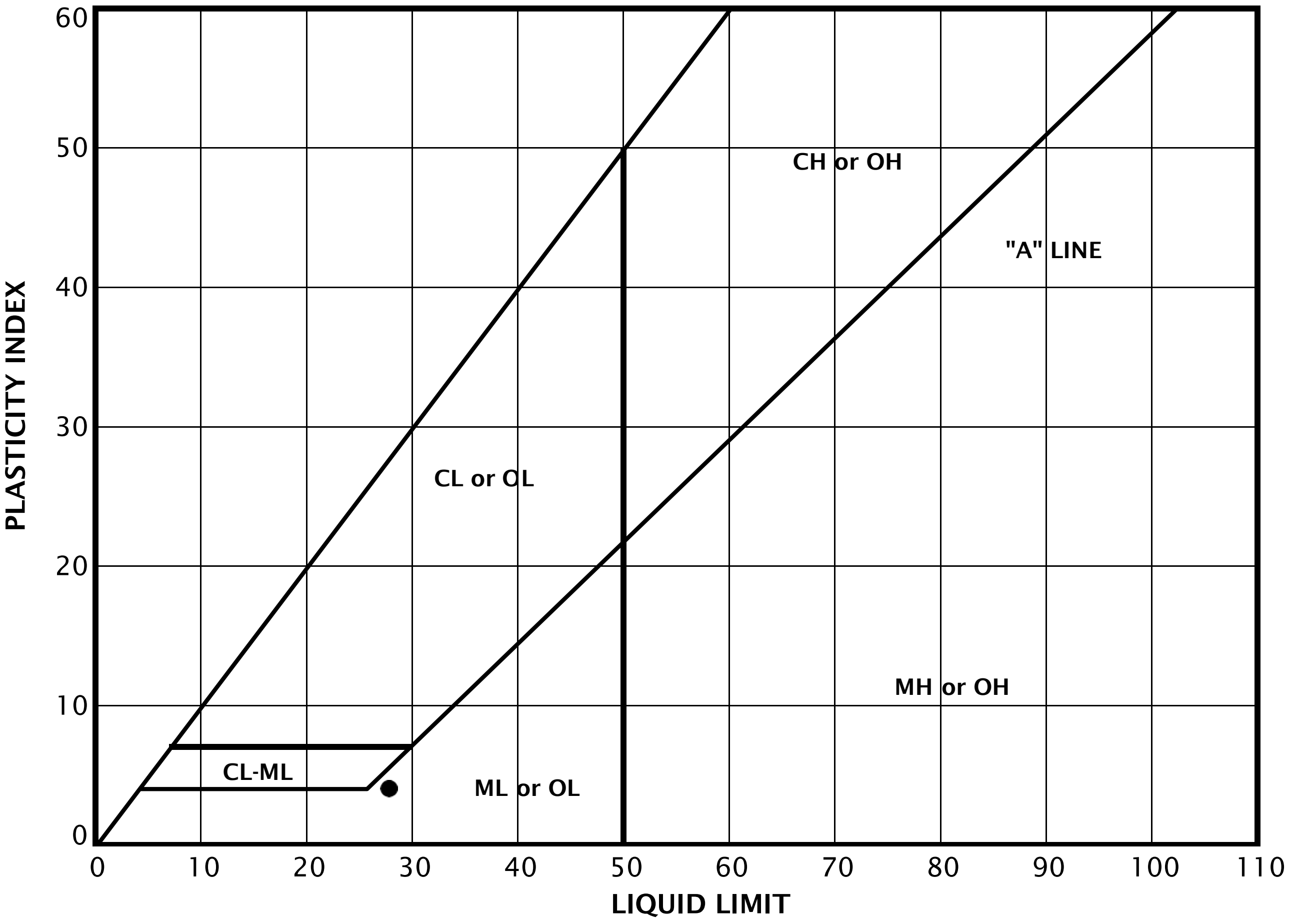
MARCH 2019

BORING B-1

AURORA AIRPORT FUEL FARM
AURORA, OR

FIGURE A-1

BORING LOG CENTREXCON-4-01-B1.GPJ GEODESIGN.GDT PRINT DATE: 3/22/19:KM:KT



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	15.0	30	28	24	4

ATTERBERG_LIMITS 7 CENTREXCON-4-01-B1.GPJ GEODESIGN.GDT PRINT DATE: 3/13/19:KM

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CENTREXCON-4-01

MARCH 2019

ATTERBERG LIMITS TEST RESULTS

AURORA AIRPORT FUEL FARM
AURORA, OR

FIGURE A-2

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	0.0		5							
B-1	2.5		35							
B-1	5.0		38				83			
B-1	10.0		37				76			
B-1	15.0		30					28	24	4
B-1	20.0		32				32			
B-1	20.1		32							
B-1	25.0		27				12			

GEODESIGN INC

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CENTREXCON-4-01

MARCH 2019

SUMMARY OF LABORATORY DATA

AURORA AIRPORT FUEL FARM
AURORA, OR

FIGURE A-3

Geotechnical Investigation

Aurora State Airport
Parallel Taxiway Relocation

Aurora, Oregon

Prepared for:

W&H Pacific
Portland. Oregon

February 9, 2007

Professional Geotechnical Services

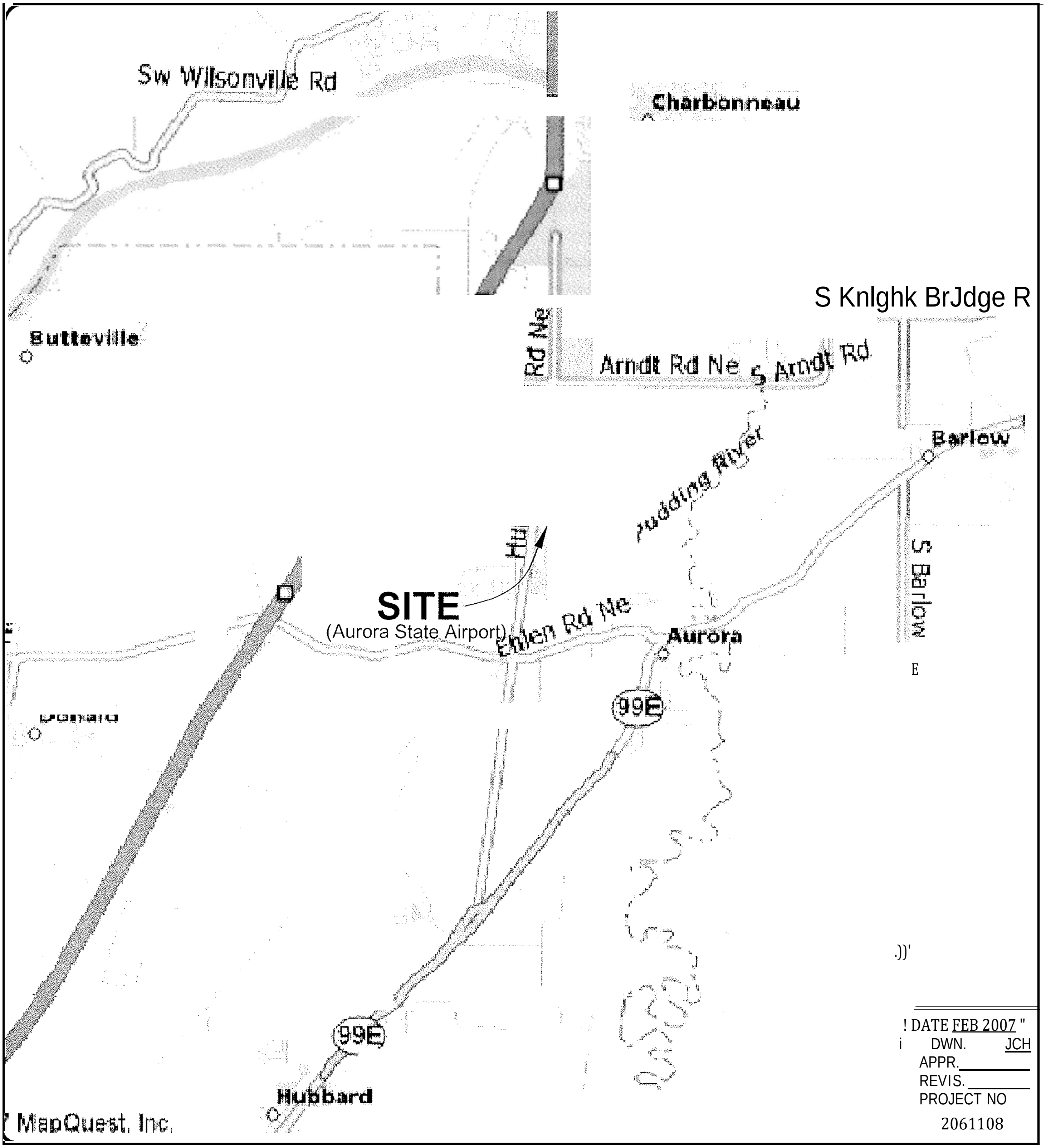
Foundation Engineering, Inc.

Appendix **A**

Figures

Professional
Geotechnical
Services

Foundation Engineering, Inc.



! DATE FEB 2007 "
i DWN. JCH
APPR.________
REVIS.________
PROJECT NO
2061108

FOUNDATION ENGINEERING INC.
M PROFESSIOHAL GEOTECHNICAL SERVICES
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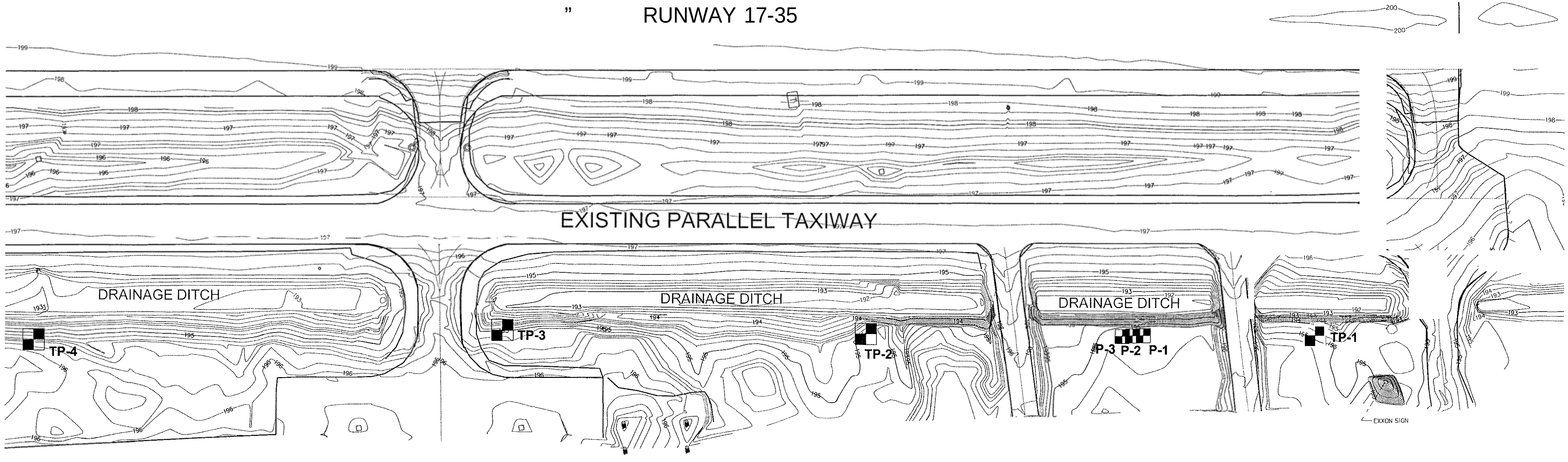
CORVALLJS, OR 7330-4517
BUS. (54t) 757-?845 r"1X (54t) 757—7es0

VICINITY MAP
AURORA STATE AIRPORT
PARALLEL TAXI WAY RELOCATION
AURORA, OREGON

FIGURE NO.

1A



SCALE: I" = i 00'

0 50 100 200

NOTES:

I. TEST PIT, CORE HOLE AND PERMEABILITY TEST LOCATIONS WERE ESTABLISHED USING A MEASURING WHEEL AND ARE APPROXIMATE ONLY.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY W&H PACIFIC.

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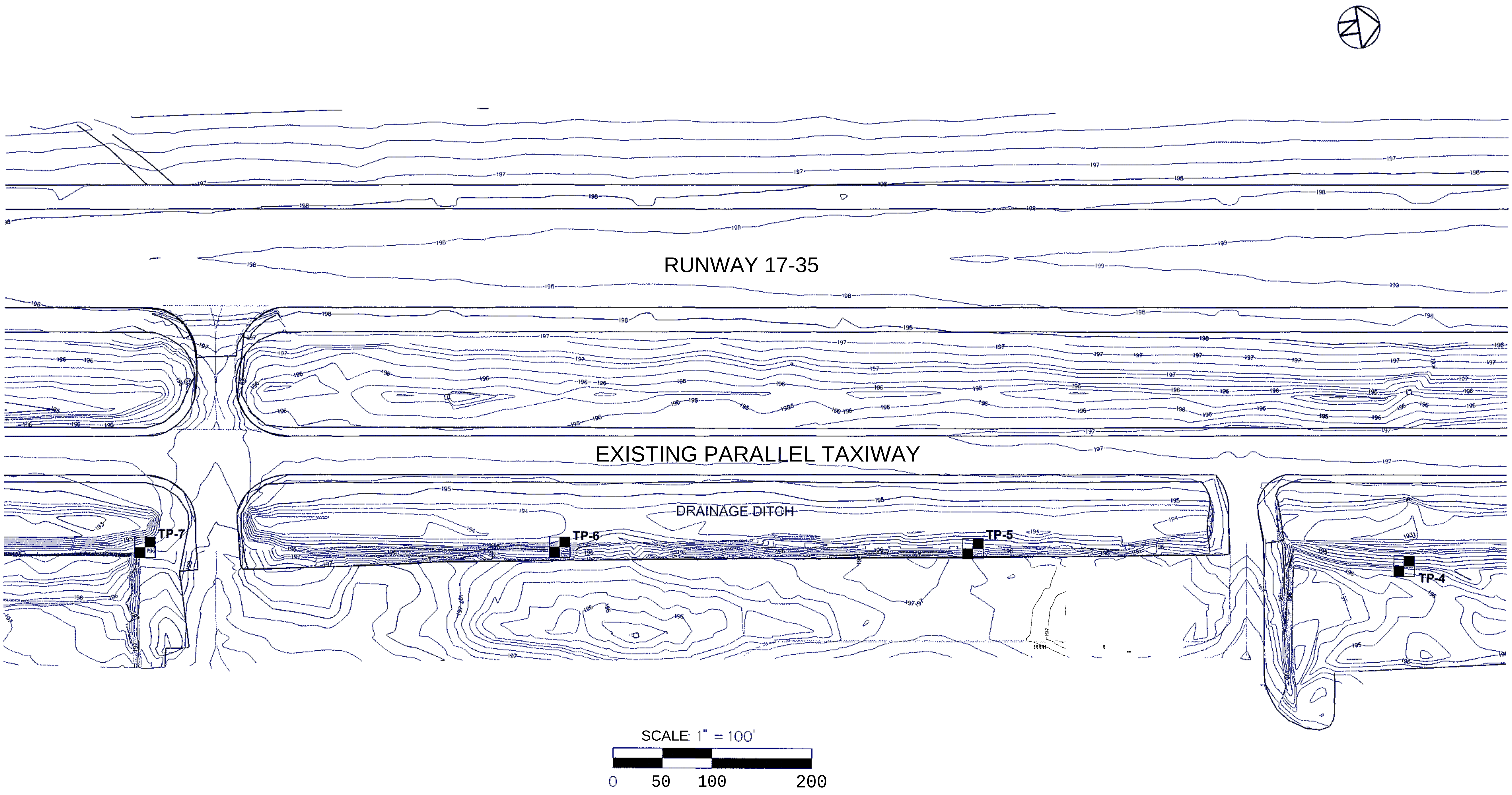
DATE FEB 2007
DWN. JCH
APPR. ______
REVIS. ______
PROJECT NO.
2061108

SITE LAYOUT AND EXPLORATION LOCATIONS
AURORA STATE AIRPORT
PARALLEL TAXIWAY RELOCATION PROJECT
AURORA, OREGON

FIGURE NO.
2A



NOTES:

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d h
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820 flW CORfI£LL 4¥CNuC
CORV4LMS, OR 97330-4*!*
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PROJECT NO.
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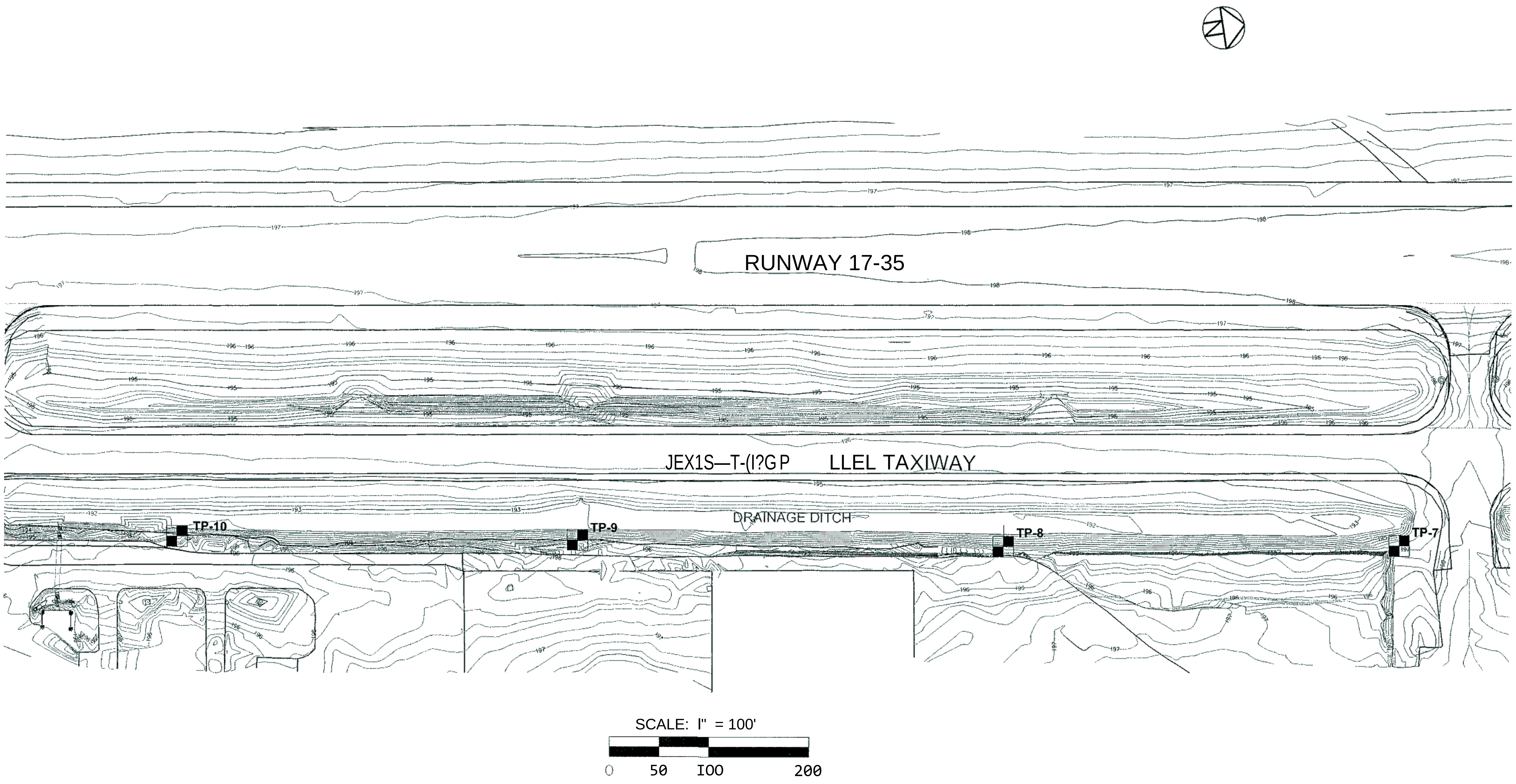
SITE LAYOUT AND EXPLORATION LOCATIONS
AURORA STATE AIRPORT
PARALLEL TAXIWAY RELOCATION PROJECT
AURORA, OREGON

FIGURE NO.
3A



NOTES:

I. TEST PIT, CORE HOLE AND PERMEABILITY TEST LOCATIONS WERE ESTABLISHED USING A MEASURING WHEEL AND ARE APPROXIMATE ONLY.

2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.

S. BASE MAP WAS PROVIDED BY W&H PACIFIC.

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PROFKSIONAk GEOTECHNICAL SERVICES
820 NY CORNCLL JtCxos
CORYALMS. OR 97330-•* t*
8u3. (5«) t5t-z6‹5 FxX {54i} '75't-z6s0

DATE /E02IXI/
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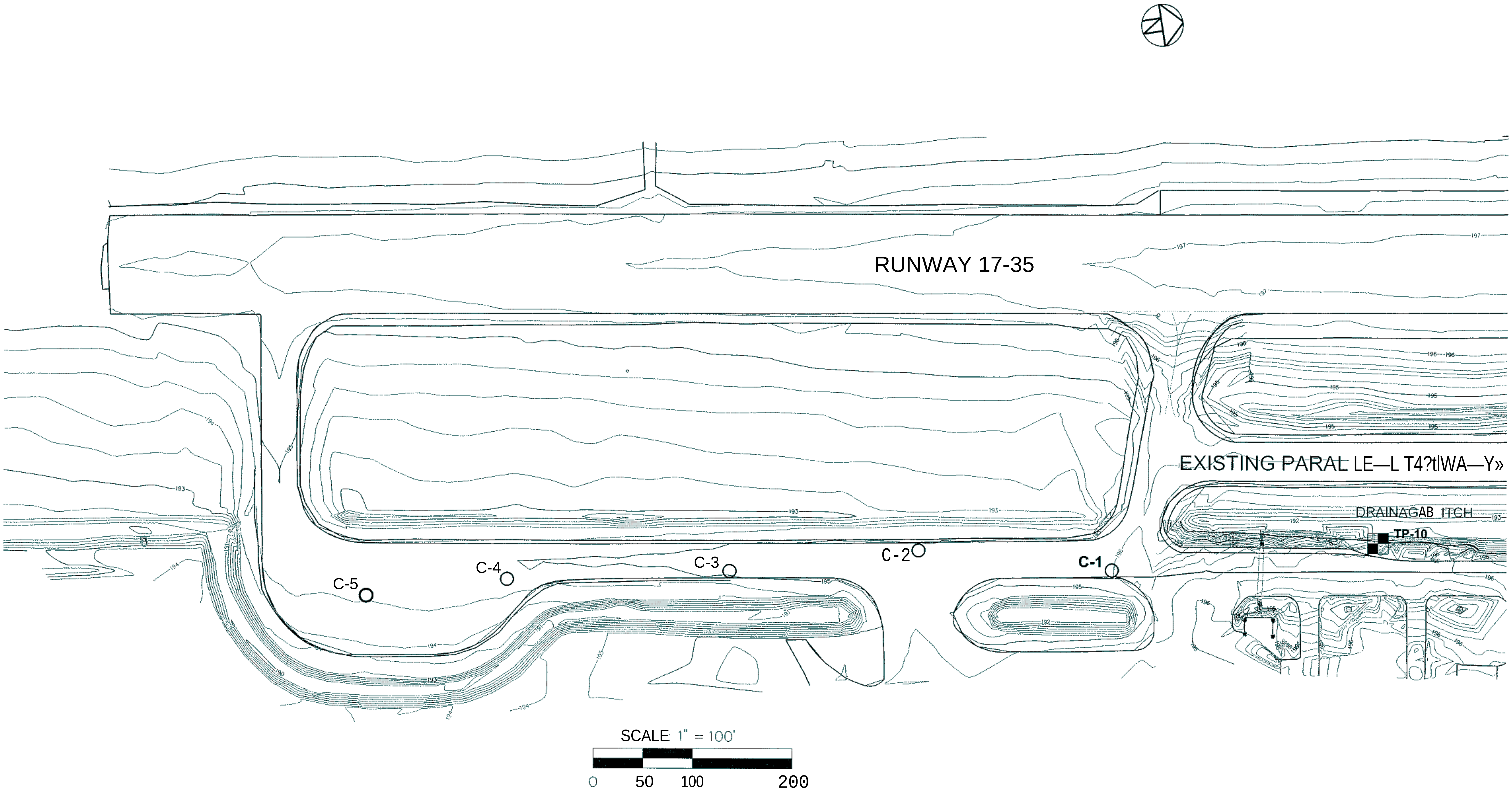
REV IS.
PROJECT NO
2061108

SITE LAYOUT AND EXPLORATION LOCATIONS
AURORA STATE AIRPORT
PARALLELTAXIWAYRELOCATIONPROJECT
AURORA, OREGON

FIGURE NO.
4A



NOTES:

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2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY W&H PACIFIC.

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APPR. ______
REVIS ______
PROJECT NO.
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SITE LAYOUT AND EXPLORATION LOCATIONS
AURORA STATE AIRPORT
PARALLEL TAXIWAY RELOCATION PROJECT
AURORA, OREGON

FIGURE NO.
5 A

Appendix B

Test Pit and Core Hole Logs

Professional
Geotechnical
Services

Foundation Engineering, Inc.

DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various moterials such as grovel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs *and* the results of the laboratory examinations and tests. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

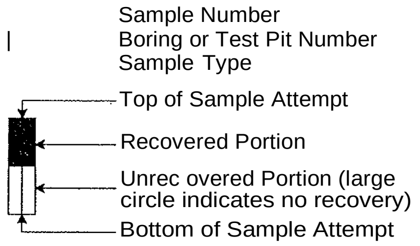
VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

The dnol log ond reloted infomnoGon depict subsudoce condiGons only ot the speciWc locoGon ond on the dote indicoted. Those using the informotion contained herein should be aware that soil conditions at other locations *or* on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

TRANSITION B EEN SOIL OR ROCK TYPES

The lines designating the interface between soil, fill or rock on the final logs and on subsurface profiles presented in the report are determined by interpolation ond are therefore approximote. The transition between the materials may be obrupt or gradual. Only at boring or test pit locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes thereon.

SAMPLE OR TEST SYMBOLS



S — Grab Samples
SS — Standard Penetration Test Sample (split—spoon)
SH — Thin—wolled Shelby Tube Sample
C — Core Sample
CS — Continuous Sample

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 3O in. is required to drive a standard split— spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (@).

UNIMED SOIL OLASSIMCAION SYMBOLS

G — Gravel
S — Sand
M — Silt
C — Clay
Pt — Peat

W — Well Graded
P — Poorly Graded
L — Low Plasticity
H — High Plosticity
O — Organic

MELO SHEAR STRENGTH TEST

Shear strength measurements on test pit side wolfs, blocks of soil or Shelby tube samples are typically made with Torvane or pocket penetrometer devices.

TYPICAL SOIL/ROCK SYMBOLS

Sond
▥ Silt
▨ Cloy
Gravel
▩ Basalt
Siltstone

WATER TABLE

Water Table Location

(1/31/00) Date of Measurement

Piezometer Tip Location (it used)

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SYMBOL KEY

BORING AND TEST PIT LOGS

Explanation of Common Terms Used in Soil Descriptions

field Identification	Cohesive Soils			Gronulor Soils ” ""	
	SPT	s„° (tel)	Term	SPT	Term
Eosily penetrated several inches by fist.	0 — 1	0.125	Very Soft	0 — 4	Very Loose
Eosily penetrated several inches by thumb.	2 — 4	0.123 0.25	Soft	5 — 10	Loose
Can be penetrated several inches by thumb with moderate effort.	5 — 8	0.25 — 0.50	Medium Stiff (Firm)	11 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	9 — 1g	0.50 — 1.0	Stiff	31 — 50	Dense
Readily indented by thumbnail.	16 — 30	1.0 — 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	31 – 60	> 2.0	Hard		

+ Undrained shear strength

	Soil Moisture Field Description
Dg	Absence of moisture. Dusty. Dry to the touch.
Domp	Soil has moisture. Cohesive soils ore below plastic limit and usually moldable.
Moist	Groins appear darkened, but no visible woter. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand ond cohesionless silt exhibit dilotancy. Cohesive silt/clay can be readily remolded. Soil leaves wetness on the frond when squeezed. "Wet" indicotes that the soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plosticity Field Test
Nonplastic	0 — Z	Connot be rolled into a thread.
Low Plasticity	3 — 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 — 30	Easily rolled into thread.
High Plasticity	TO	Easily rolled and rerolled into thread.

Tenm	Soil Structure Criterio “
Strotitieb	Alternating foyers at least 1 inch thick — describe variation.
Laminated	Alternating layers at less th on 1 inch thick — describe variation.
Fissured	Contains shears and partings along planss of weakness.
Slickensides	Partings appear glossy *or* striated.
Blocky	Breaks into lumps — crumbly.
Lensed	Contains pockets of different soils — describe variation.

Term	Soil Cementotion Criterio "
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not breok with finger pressure.

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COMMON TERMS
SOIL DESCRIPTIONS

Comments

pth, Fe | mple # | O W * O | θc′ Cl | %W | C, | Sy5

Soil and Rock Description

Surface: short grass.

Fine roots extend to +12 inches.

Moderate seepage noted at +3 feet.

1
2
3
4
8
9
10—
11

S-1-1

020

Medium stif, clayey SILT, (ML); brown, moist, low plasticity, blocky structure, (topsoil).

Soft to medium stiff, clayey SILT, (ML); brown-grey, trace iron-staining, moist to wet, low plasticity, micaceous, (alluvium).

Medium stiff SILT, some sand, (ML) brown-grey, wet, non-plastic to low plasticity, fine sand, (alluvium).

BOTTOM OF TESTPIT

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP-1

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments

σpth, | ≡mpl | σcati | Class | Water | T: | m

Soil and Rock Description

Fine roots extend to +18 inches.

Slow seepage noted at +4 feet.

1
2
3
4–
5–
6–
8
9
10–

S-2-1

5-2-2

52

i st)P ayeySd.QT tpaa L
a!k bro d , uo ticity, becky n turM,"
(topsoi).

Medium stiff to stiff, SILT, some clay, trace sand, (CL-ML); brown-prey, trace iron-staining, moist, medium plasticity, semi-blocky structure, micaceous, (alluvium).

Stiff SILT, some clay, trace sand, (CL-ML); brown-grey, moist to wet, low to medium plasticity, fine sand, micaceous, (alluvium).

BOTTOM OF TEST PT

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 2”

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	pth, Fe	mple #	ocation	lass Sy	/ater Ta	, TSF	m	Soil and Rock Description
Su aces short grass.								Soft to stiff, SILT, some clay, trace sand, (ML); brown-grey, trace iron-staining, moist, low to medium plasticity, fine sand, micaceous, (alluvium).
	'	S-3-1						
	2							
	3-							
Slow seepage noted at a3.5 feet.	4–	S-3-2						Stiff, clayey SILT, trace sand, (CL-ML); brown-grey; moist to wet, low to mediumplasticity, fine sand, micaceous, (alluvium).
	5–							
Rapid seepage noted at +6.5 feet.	y							
	8							BOTTOM OF TEST PIT
	9							
	10-—							
	11							

Project No.: 2061108

Surface Elevation: N/A

DateofTestPit: January 9, 2007

Test Pit Log: TP- 3

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: short grass.							▨	Stiff, SILT, some clay, trace sand, (ML); brown-grey, trace iron-staining, moist to wet, low plasticity, micaceous, (alluvium).
	1-						▨	
Fine roots extend to ±2 feet.	2-	S-4-1	■				▨	Blocky structure noted in upper±5 feet.
Moderate seepage noted at ±3 feet.	3-						▨	
	4-						▨	
	5-						▨	
	6-						▨	
	7-						▨	
	8-							BOTTOM OF TEST PIT
	9-							
	10—							
	11-							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 4

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: short grass and trace gravel fill.								Medium stiff, gravelly ”SLT, some clay, (CL-ML); dark brown, moist to wet, medium plasticity, fine to coarse, subrounded to rounded gravel, blocky structure, (fill).
	1							
Fine roots extend to +2 feet.	2–	S-5-1	■			0.80		Medium stiff to stiff, clayey SILT, (CL-ML); brown-grey, trace iron-staining, moist to wet, medium plasticity, micaceous, (alluvium).
Slow to moderate seepage noted at +3 feet.	3							
	4							Stiff SILT, trace clay and sand (ML)) brown-greys moist to wet, low plasticity, fine sand, micaceous, (alluvium).
	5							
	6–							
	7							
	8–							BOTTOM OF TEST PIT
	10–							
	11–							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 5

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments		m	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: short grass and trace gravel.								Medium stiff, clayey SILT, trace gravel, (CL-ML); dark brown, moist, medium plasticity, blocky structure, (topsoil/fill).
		5-6-1	■					Medium stiff, clayey SILT, (CL-ML); brown-grey, trace iron-staining, moist to wet, low to medium plasticity, blocky structure, micaceous, (alluvium).
Fine roots extend to +2 feet.	2	S—6—2	■					Stiff SILT, some clay, trace sand, (ML); brown-grey, moist to wet, low plasticity, fine sand, micaceous, (alluvium).
Slow to moderate seepage noted at +3 feet.								
	4							
	5							
	7							
	9							BOTTOM OF TEST PIT
	10—							
	11							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 6

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	m	Location	Class Symbol	Water Table	C, TSF	m	Soil and Rock Description
	:							Medium stiff, gravelly SILT, some clay, (ML)/ brown, moist, medium plasticity, fine to coarse, subrounded gravel, (fill).
Fine roots extend to +2 feet.	2-	S-7-1	■					Stiff clayey SILT, (ML), brown-grey, trace ironstaining, moist to wet, low plasticity, micaceous, (alluvium).
	3-							
Slow seepage noted at +4 feet.	4							
	5-							
Rapid seepage noted at +5.5 feet.	g							
	7-							
								BOTTOM OF TEST PIT
	40-							
	11-							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 7

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	**Depth, Feet**	**Sample #**	**Location**	**Class Symbol**	**Water Table**	**C, TSF**	**Symbol**	**Soil and Rock Description**
Surface: tall grass.								Medium stiff to stiff SILT, trace clay and sand, (ML); brown-grey, moist to wet, low plasticity, fine sand, micaceous, (alluvium).
Fine roots extend to ±12 inches.	1-							
	2-	S-8-1	■					
Slow seepage noted at ±3 feet.	3-							
	4-							
	5-							
	6-							
	7-							
	8-							
Rapid seepage noted at ±8.5 feet.	9-							
								BOTTOM OF TEST PIT
	10-							
	11-							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 8

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symb	Water Table	C, TSF	m	Soil and Rock Description
Surface: tall grass.								Soft to medium stiff, clayey SILT, (ML) dark brown, moist, low to medium plasticity (topsoils
	1-	S-9-1						Medium stiff, clayey SILT, (CL-ML); grey-brown, trace iron-staining, moist to wet, medium plasticity, blocky structure, micaceous, (alluvium).
	2	g 2						
Slow seepage noted at +2.5 feet.	3-							Stiff SILT, some clay, (ML); brown-grey, moist to wet, low to medium plasticity, micaceous, (alluvium).
	4—							
	5-							
	6-							
	7—							
	9							
	10-							BOTTOM OF TEST PIT
	11							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP- 9

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	pth,	mpl	catio	Class S	Water T	C, TSF	Symbol	Soil and Rock Description
Surface: short grass.								Medium stiff to stiff, clayey SILT, (ML); dark brown, moist, low plasticity, (possible topsoil).
		5-10-1						
	2							
	3	S-10-2						Stiff, SILT, some clay, trace sand, (CL-ML); brown-grey, moist, medium plasticity, fine sand, micaceous, (alluvium).
	4					045		
	5-							
	6							
No ground water encountered to the limit of excavation.	9							BOTTOM OF TEST PIT
	40—							
	11—							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: TP-10

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments

Depth, Feet	Sample #	Location	Class Sym	Water Tabl	C, T	Sym
	C-1-1					
1						
2	SHC-1-2					
3						
4						
5						

No ground water encountered to the limit of excavation.

Soil and Rock Description

ASPHALTIC CONCRET E (+4 inches).

Dense CRUSHED ROCK (+14inches), (GW); grey, moist, +2-inch minus, (base rock).

+3?•-inch, rounded cobble encountered at +18 inches.

Stiff, clayey SILT, (CL-ML), grey, moist, medium plasticity, (alluvium).

BOTTOM OF CORE HOLE

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 10, 2007

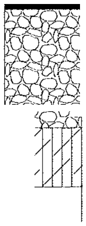
Core Hole Log: C-1

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments

Depth,	Sampl	Locati	Class	Water T	C, TSF	Symbol
	C-2-1					
	SHC-2-2					
3						
4						
5						



No ground water encountered to the limit of excavation.

Soil and Rock Description

ASPHALTIC CONCRETE (14"é inches).

Dense CRUSHED ROCK (/+14a inches); (GW); grey to brown, damp, 1-inch minus, (base rock).

Stiff, clayey SILT, (CL-ML); brown-grey, moist, medium plasticity, (alluvium).

BOTTOM OF CORE HOLE

Project No.: 2061408

Surface Elevation: N/A

Date of Test Pit: January 10, 2007

Core Hole Log: C-2

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
								ASPHALTIC CONCRETE (±4 inches).
		C-3-1	■					Dense CRUSHED ROCK (±13 inches), (GW); grey, moist, 2-inch minus, (base rock).
	1							
No ground water encountered to the limit of excavation.		C-3-2	■					Stiff, clayey SILT, (CL-ML); grey-brown, trace iron-staining, moist, medium plasticity, micaceous, (alluvium).
	2							BOTTOM OF CORE HOLE
	3							
	4							
	5							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 10, 2007

Core Hole Log: C-3

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
								ASPHALTIC CONCRETE (±5½ inches).
								Dense CRUSHED ROCK (±13½ inches), (GW); grey, moist ±2-inch minus, (base rock).
	1	C-4-1	■					
Slow seepage noted at +1.5 feet								Stiff, clayey SILT, (CL-ML); grey, moist, medium plasticity, (alluvium).
	2	SHC-4-2	■					
								BOTTOM OF CORE HOLE
	3							
	4							
	5							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 10, 2007

Core Hole Log: C-4

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	pth, Feet	mpl #	ocati n	Class E	Water o	T	m	Soil and Rock Description
								ASPHALTIC CONCRETE (+5 inches).
								Dense CRUSHED ROCK (+27 inches), (GW); grey, moist, +2-inch minus, (base rock).
	1	C-5-1						
	2-							
No groundwater encountered to the limit of excavation.	3							BOTTOM OF CORE HOLE
	4’							
	5							

Project No.: 2061408

Surface Elevation: N/A

Date of Test Pit: January 10, 2007

Core Hole Log: C-5

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample#	0	Class Symbol	Water Table	C, TSF	E	Soil and Rock Description
								Medium stff, clayey SILT, (ML); dark brown, moist, low to medium plasticity, blocky structure, (topsoil).
	1							Soft to medium stiff, clayey SILT, (CL-ML); light brown-grey, trace iron-staining, wet, medium plasticity, blocky structure, (alluvium).
Moderate seepage noted at ±1.5 feet.	2							
	3							BOTTOM OF PERMEABILITY TEST
	4							
	6							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: P-1

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symb	Water Table	C, TSF	m	Soil and Rock Description
"								Medium stiff, clayey”SILT, (ML); dark brown, moist, low to medium plasticity, blocky structure, (topsoil).
	1							”Soft to medium stiff, clayey SILT, (CL-ML); brown-grey, trace iron-staining, wet, medium plasticity, blocky structure, (alluvium).
Moderate seepage noted at +1.5 feet.	2						▨	
	3	P-2-1						Stiff, clayey SILT, trace sand, (CL-ML); brown-grey, wet, medium plasticity, (alluvium).
	4	z,						
	5							BOTTOM OF PERMEABILITY TEST
	7							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: P-2

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Comments	Depth, Feet	[illegible]	io	Class Symbol	Water Table	C, TSF	Ú Symbol	Soil and Rock Description
Moderate seepage noedat +1.5 feet.								Medium stiff, clayey SILT, (ML): dark brown, moist, low to medium plasticity, blocky structure, (topsoil).
	1						"	Soft to medium stiff, clayey S"ILT, (CL-ML); brown-grey, trace iron-staining, wet, medium plasticity, blocky structure, (alluvium).
	2							
	3–					"		”Stiff, clayey SILT, trace sand, (CL-ML); wet, brown-grey, medium plasticity, (alluvium).
	4-							
	5							
	6							
	7							BOTTOM OF PERMEABILITY TEST
	8-							

Project No.: 2061108

Surface Elevation: N/A

Date of Test Pit: January 9, 2007

Test Pit Log: P-3

Aurora State Airport Parallel Taxiway Relocation

Aurora, Oregon

Appendix **C**

Field and Laboratory Test Results

Professional
Geotechnical
Services

Foundation Engineering, Inc.

Table 1C. Summary of Field Permeability Testing

Test Location	Test Depth (feet)	Soil Description at Test Depth	Average k Value (cm/sec)
P—1	2.9	Medium stiff, brown-grey, medium plasticity, Clayey SILT (CL—ML)	i3x1O-’
P-2	5	Stiff, brown—grey, medium plasticity, Clayey SILT; trace sand (CL—ML)	+ 3x10^{7}
P-3	7	Stiff, brown-grey, medium plasticity, Clayey SILT; trace sand (CL ML)	+ 5x10

Note: Tests were conducted on January 10 and 12, 2007.

Table 2C. Natural Water Content and Atterberg Limits

Sample Number	Sample Depth (feet)	Natural Water Content (percent)	LL	PL	PI	FAA/USCS Classification
S-1 -1	2.0 — 3.0	33.0				
S—2—1	1.0 — 1.5	33.7				
S-2-2	2.0 — 3.0	30.3	44	26	17	CL—ML
S—2-3	3.5 — 4.0	47.8				
S-3-1	1.0 — 1.5	386				
5-3-2	3. 5 — 4. 0	38.8				
S-4-1	2.0 3.0	37.6				
S-5-1	2.0 2.5	42.7				
S-6-1	1.0 -1.5	42.4				
5-6-2	20 4.0	33.8	42	29	13	ML
S—7—1	2.0 2.5	30.5				
S-8-1	2.0 — 3.0	38.1				
5-9-1	1.0—1.5	34.1				
S—9—2	2.5 — 3.5	36.4				
S-10—1	1. 0 1. 5	31.0				
S—10—2	3.0 3. 5	39.7				
SEC—1-2	1. 8 2.1	25.4				
SHC-2—2	1.7 2.2	27.7				
SHC-4—2	1.9 — 2.7	25.2	42	24	18	CL
C-3—2	1.5 — 1.8	29.6				

Table 3C. Summary of Previous and Recent Moisture-Density and CBR Test Results

Test Date	Location	Soil Description	FAA/USCS Classification	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	CBR at 95% Relative Compaction
1999	Apron	Brown, silty CLAY	CL	100.0	21.0	5.8
2005	Runway	Grey, Clayey SILT; trace sand	ML—OL	100.5	20.0	6.1
2005	Runway	Brown—Grey SILT; some clay, trace sand	ML	103.5	19.0	5.5
2005	Runway	Brown-Grey SILT; some clay, trace sand	ML	980	23.0	5.5
2007	Taxiway	Brown—Grey SILT; some clay, trace sand	CL—ML	97.4	19.9	5.7
2007	Taxiway	Brown-Grey SILT; some clay, trace sand	ML	95.9	20.5	7.2
			Average =	99.2	20.6	6.0

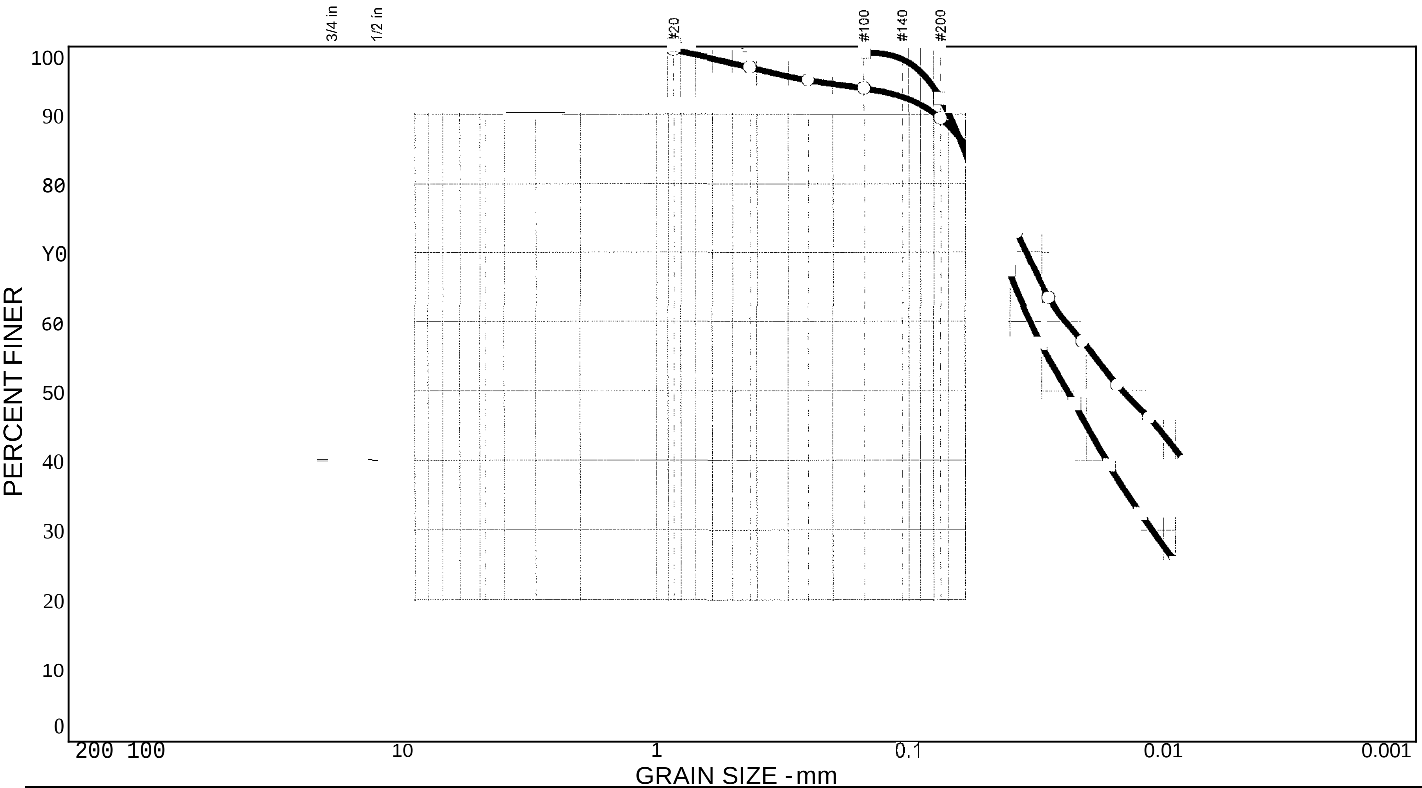
Note: Maximum dry densities and Optimum moisture contents are based on ASTM D698 moisture-density test results.

Table 4C. Bulk Densities

Sample Number	Sample Depth (feet)	Soil Description	Water Content (%)	Moist Bulk Density (pcf)	Dry Density (pcf)	Relative Compaction
SEC-1-2	1.8 - 2.1	Grey, clayey SILT	25.4	124.8	99.4	100
SHC-2-2	4 7-2.2	Light brown, clayey SILT	27.7	117.0	91.6	94
SHC-4-2	1.9 - 2.7	Grey, clayey SILT	25.2	12 1.4	97.0	98

Note: Relative compaction is based on a maximum dry density of 99.2 pcf, which is based on the average results of six moisture-density tests (ASTM D698) on subgrade from Aurora Airport.

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	3.1	7.5	65.2	24.2
00	00	00	00	û0	77	80.3	12.0

LL	PL	D_{es}	D_{60}	D_{so}	D_{uo}	D_{15}	D_{10}	C_c	C_u
44	26	0.0583	0.0242	0.0144	0.004 1				
		0.06 10	0.0334	0.0238	0.0111	0.0044			

MATERIAL DESCRIPTION	USCS	AASHTO
Brown—grey Sl LT; soiaae clay, t1‘ace sand	ML—CL	A-7-6(18)
Brown-grey *SI* LT; some clay, trace sand	ML	A-4(0)

Project No. 20G I ! 08 Client: FoutJdatiotJ Eng\‹ eei \‹Jg, I\Jc.

Project: Art otaState Aitpoi-t Tax\way; Arltota, O\cgo‹

‹. Source: 3423 Sample No.: S-2-2 Elev./Depth: 2.0-3.0

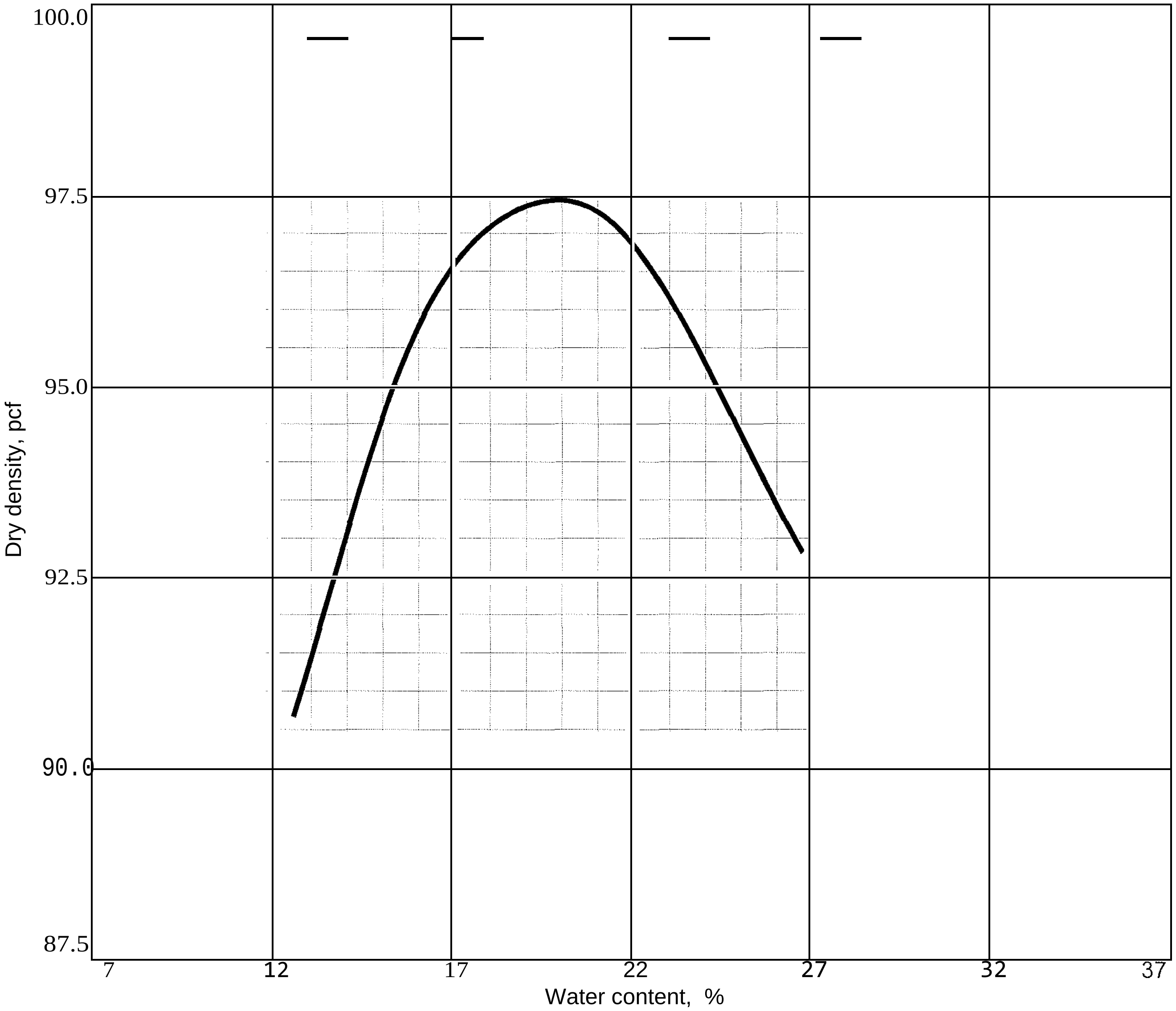
Source: 3423 Sample No.: S-8-1 Elev./Depth: 2.0-3.0

Remarks:

Particle Size Distribution Report
FEI Testing & Inspection, Inc.
Corvallis, OR

Fig. No: IC

MOISTURE - DENSITY RELATIONSHIP TEST



Test specification: ASTM D G98-00a Met) oJ A StatJda\ d

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
2.0-3.0	CL—ML		303		44	18	0.0	24.2

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 97.4 pcf Optimum moisture — 19.9 %	Drown-grey SILT; some clay, ti'ace sand

Project N o. 2061 108 Client: Foundation Engineering, Inc.

Project: Aurora State Airport Taxiway; Au1‘ora, Oregon

u Source: 3425 Sample No.: S-1-1/S-2-2 Elev./Depth: 2.0-3.0

Remarks:

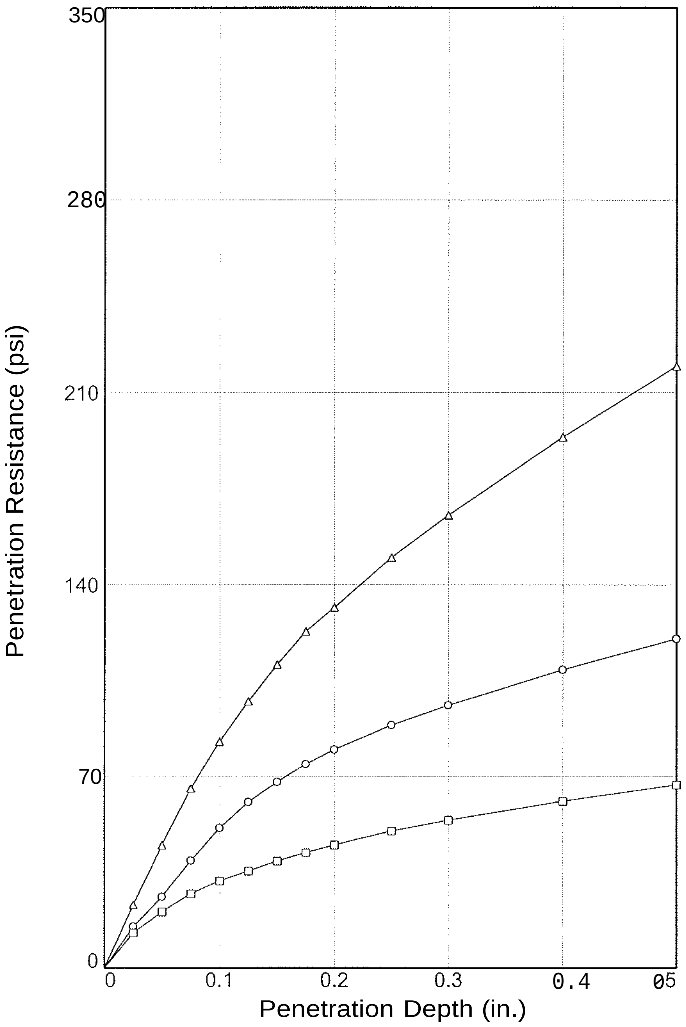
Datc: ! -! 8-07

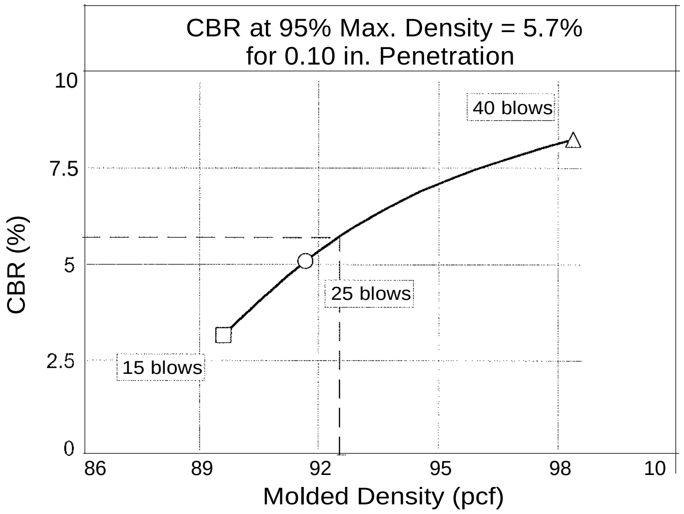
MOISTURE - DENSITY RELATIONSHIP TEST

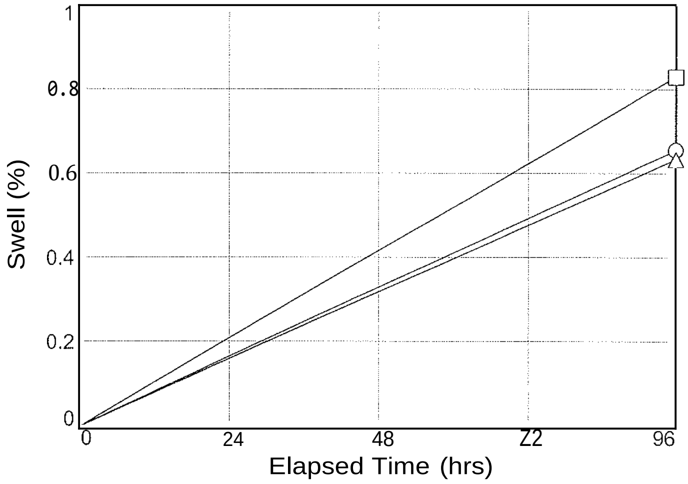
FEI Testing & Inspection, Inc.
Corvallis, OR

Fig. No: 2C

BEARING RATIO TEST REPORT
ASTM D 1883-99







	Molded			Soaked			CBR (%)		Linearity Correction	Surcharge	Max. Swell
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.	(in.)	(lbs.)	(%)
1 ○	91.7	94.1	19.6	91.1	93.5	30.2	5.1	5.3	0.000	32	0.7
2 △	98.4	101	18.7	97.8	100.4	28.2	8.3	8.8	0.000	32	0.6
3 □	89.6	92	19.0	88.9	91.2	30.8	3.2	3.0	0.000	32	0.8

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
Brown—grey SILT; some clay, trace sand	CL—ML	97.4	19.9	44	18

Project No: 2061108

Project: Auiola State Aiipoil Taxiway; Auiola, Oiegon

Source of Sample: 3425 Depth: 2.0-3.0

Sample Number: S-1-1/S-2-2

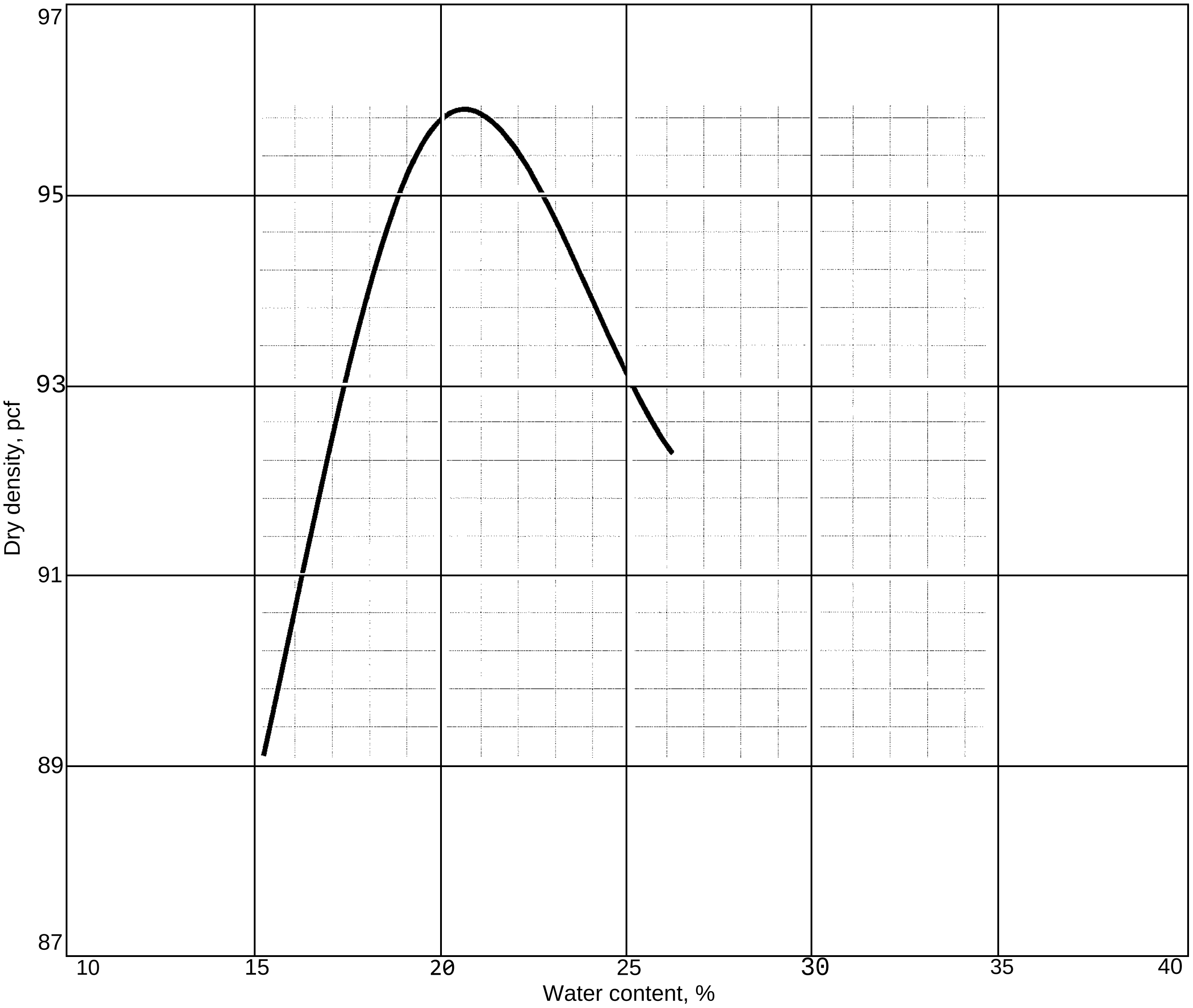
Date: 1-29-07

BEARING RATIO TEST REPORT
FEI Testing & Inspection, Inc.
Corvallis, OR

Test Description/Remarks:

Fig. No: 3C

MOISTURE - DENSITY RELATIONSHIP TEST



Test specification: ASTM D 698-00a Method A Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% ≥ No.4	% ≤ No.200
	USCS	AASHTO						
2.0-4.0	ML		33.8		42	13		

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 95.9 pcf Optimum moisture = 20.5 %	Brown-grey SILT; some clay, ti‘ace sand

Project No, 2061108 Client: Foundation Engineering, Inc.

Project: A tu‘ora State Airport Taxiway; Abu or a, Oregon

o Source: 3425 Sample No.: S-5-2 Elev./Depth: 2.0-4.0

MOISTURE - DENSITY RELATIONSHIP TEST

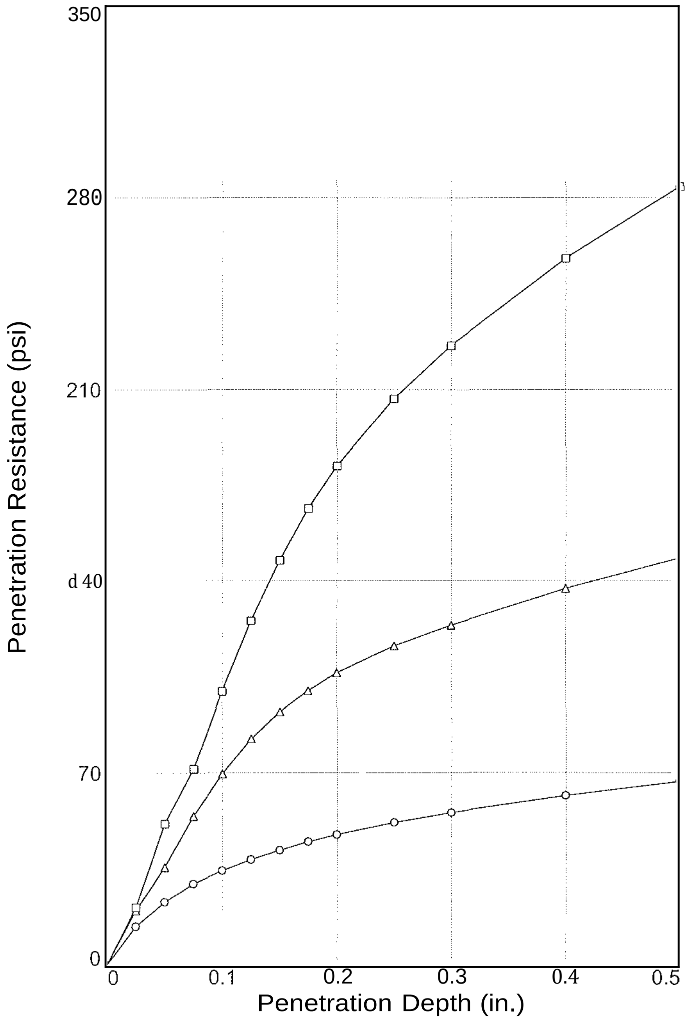
FEI Testing & Inspection, Inc.
Corvallis, OR

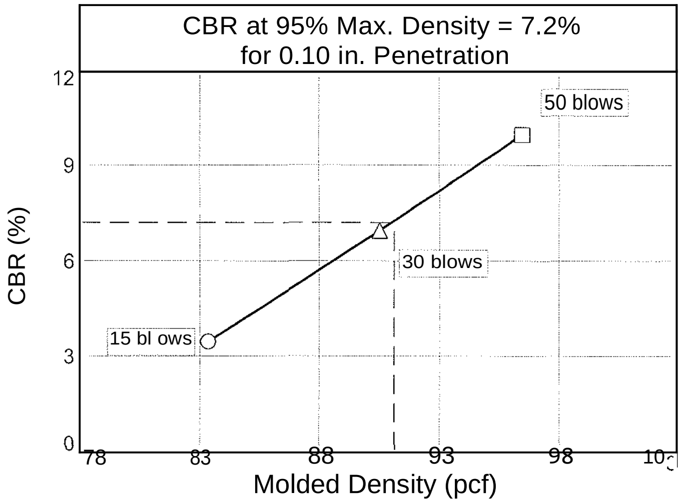
Remarks:

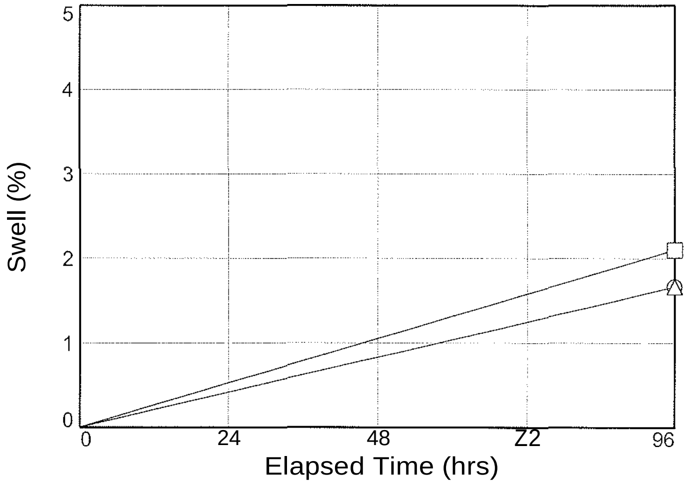
Fig. No: 4C

BEARING RATIO TEST REPORT

ASTM D 1883-99







	Molded			Soaked			CBR (%)		Linearity Correction (in.)	Surcharge (lbs.)	Max. Swell (%)
	Density (pcf)	Percent of Max. Dens.	Moisture (%)	Density (pcf)	Percent of Max. Dens.	Moisture (%)	0.10 in.	0.20 in.			
1 O	83.4	87	20.2	82.0	85.5	35.0	3.4	3.2	0.000	32	1.7
2 A	90.5	94.4	19.1	89.0	92.8	32.2	6.9	7.1	0.000	32	1.7
3 D	96.5	100.6	19.4	94.5	98.5	29.7	10.0	12.1	0.000	32	2.1

Material Description	USCS	Max. Dens. (pcf)	Optimum Moisture (%)	LL	PI
Brown-grey SILT; some clay, trace sand	ML	95.9	20.5	42	13

Project No: 2061108

Project: Autoia State Aiipoi1 Taxiway; Auioia, Oiegon

Source of Sample: 3425 Depth: 2.0-4.0

Sample Number: S—6-2

Date: 1-29-07

Test Description/Remarks:

BEARING RATIO TEST REPORT
FEl Testing & Inspection, Inc.
Corvaiiis, OR

Fig. No: DC

Exhibit 1, Attachment 2

EMS
ENVIRONMENTAL
MANAGEMENT
SYSTEMS, INC.

OR: 503-353-9691
FAX: 503-353-9695
WA: 360-735-1109
www.envmgtsys.com
4080 SE International Way
Suite B-112
Milwaukie, OR 97222

5 November 2020
Report # 19-0054-02

Mr. Ted Millar
c/o: Aron Faegre & Associates
520 SW Yamhill St., Roofgarden 1
Portland, OR 97204

REGARDING: Winter Evaluation for feasibility of onsite wastewater treatment, HDSE Sewer System Association, Aurora State Airport, adjacent to Keil Rd. NE and Hubbard Cuttoff Rd. NE, Aurora, OR 97002. T: 4S, R: 1W, Sec: 11, T.L: 800, 17.79 Acres

Dear Mr. Millar & Mr. Faegre,

As requested, Environmental Management Systems, Inc. (EMS) has performed the following services and provides this report for your use.

PROJECT DESCRIPTION:

The goal of this project is to expand the approved drainfield area for onsite wastewater treatment to serve future expansion of your existing HDSE Sewer System Association facilities located at the Aurora State Airport. The subject property is leased from the Oregon Department of Aviation by the HDSE Sewer System Association. The lease was recently renewed to accommodate expansion to include enough drainfield area to double the existing system's capacity in support of future development. The existing drainfields in this area were approved by DEQ in 2005 and have been functioning with no problems in the intervening 15 years since installation. There have been no documented drainfield problems in these soils. On September 25th, 2019, twelve test pits adjacent to the existing drainfields were evaluated by Marion County for feasibility for onsite wastewater treatment. EMS's analysis was that the soils are similar to the adjacent existing soils and will function acceptably. However, Marion County staff initially denied the application on October 8th, 2019 because they felt there was potential for seasonally high groundwater which could be a problem, and because they believed there was a presence of fill in this area. They recommended that for re-evaluation a tile dewatering system be installed to drain the area, and that a winter evaluation be conducted to determine the actual depth to seasonal water table. EMS designed a tile dewatering system which was installed in January of 2020. A winter evaluation was conducted through the winter of 2020. This report details our methods, findings, and recommendations for next steps and continues to recommend approval of the soils for the expansion use.

SUMMARY:

The average water depth across all twelve wells was 28 inches from the surface, after the tile dewatering system (TDS) was installed on January 23rd, 2020. The longest consecutive number of days that the water table rose above 12" below ground surface anywhere in the drainfield was about 3.8 days. On average, the water table rose above 12" for less than 1 day, with five out of the twelve wells having no shallow water table readings after the TDS was completed. Each well was dry when they were re-inspected in June following excessive rainfall during the previous six weeks. Based on success of the existing system and this study, we recommend approval of the drainfield areas for installation of a shallow pressure distribution drainfield, following Treatment Standard 1 or 2 similar to that currently in use. Permits require review and approval by DEQ.

METHODS: The following methods were used:

Observation x   Measurement x   Staking x   Soil Evaluation x
Sampling x   Inspection x   Laser Elevations x Total Station x
Gov Records x   Interview x   Aerial Photo _x_   Soil Survey x
Geologic Maps _x_   Wetland Inventories x   other (specify) Weather tracking x_

LIMITATIONS: This investigation is limited by the precipitation frequency and duration.

LANDSCAPE SETTING:
The study area consists of Tax Lot 800 in Township 4S, Range 1W, Section 11, in Marion County Oregon, totaling 17.79 acres. The site is outside of the urban growth boundary for Aurora and is zoned P (public) by Marion County. The site is part of a complex of many lots all making up the Aurora State Airport. The onsite wastewater treatment system is owned and operated under a common entity known as the HDSE Sewer System Association. Lot 800 is owned by Oregon Department of Aviation, with part of the site leased by the Association as a private septic system easement. The proposed drainfield area is within the easement, south of the airport runway and on either side (east and west) of the runway flight path and instrument landing system (FAA localizer). An existing drainfield is located at the southeast corner of the easement, south and southeast of the new proposed drainfields. An approved reserve area is in the southwest corner of the easement. No signs of failure, such as surfacing or odors, have been observed in the existing system since its installation in 2005. Also, this state-owned property is fenced and monitored to protect it from unauthorized public access and or contact with sewage.

The site is situated in the lowlands of the Willamette Valley, northwest of the town of Aurora. The average elevation of the site is approximately 193 feet above sea level. The site is fairly flat, sloping 1-2% east and west, with a crown along the runway flight path. The soils in this area were established in 1993 when the runway was extended over existing farmland. There has been no disturbance of those soils in the intervening 27 years. Two drainage swales are located along the east and west property lines, draining surface runoff to the south. Concrete culverts at the southwest and southeast corners of the site convey drainage off site. The property is open and vegetated with grasses and other low-lying forbs. No wetlands are mapped on the property by the National Wetlands Inventory (US Fish & Wildlife), and none were observed during the site visit. According to Oregon Department of Geology and Mineral Industries (DOGAMI) geology of the site is mapped as Quaternary surficial deposits (fine grained sediments) of the Missoula Flood Deposits formation.

The soil on site is mapped by the Natural Resource Conservation Service (NRCS) as Amity silt loam. Amity is described as somewhat poorly drained with a depth to water table of 6 to 16 inches, and depth to restrictive layer over 80 inches. Conditions associated with saturation (redoximorphic features) were observed at 6-16 inches from the soil surface, indicating potential for a seasonally high-water table. Runway construction resulted in the deposition of fill soil along the sides. This soil has remained essentially undisturbed for 23 years.

The new drainfield lease area was surveyed prior to conducting the study. Enough area was included for two new drainfields and reserve areas to support a design flow of approximately 10,000 gpd, thereby doubling the existing system’s capacity. Twelve test pits were dug across the site in the summer of 2019, with six on the eastern proposed drainfield area, and six in the western proposed drainfield area. Various depths of the (at least) 27 year old fill were observed over the native silt loam in the 6 eastern test pits dugs on the site (TP’s 5-10).

TILE DEWATERING SYSTEMS

Tile dewatering systems (TDS) were installed on the site in mid-January 2020, with completion on January 23rd, 2020. In both the east and west drainfield areas, two adjacent 70' by 350' rectangular dewatering trenches were installed. The field collection tile was installed with a slope of 0.2-0.4 percent at the bottom of the trenches; trench depths vary between 15 and 52 inches from ground surface. The trenches are 1 foot wide and are filled with EZFlow synthetic drain media. Each drainage system is connected to a 4" tight line installed on a 1% slope, which discharges to either the east or west drainage swale. Sediment basins were installed at the inlet end of each outfall pipe.

WATER TABLE MONITORING

While DEQ does not provide guidance on how to evaluate data, research has demonstrated that 21 days per season of actual saturation is needed to create the Redoximorphic Features which form the basis for Oregon DEQ to judge depth to water table. Published guidance from several sources, primarily the Recommended Procedures and Standards for Conducting a Water Table Study from Virginia Tech University[1] (2008) was used for conducting the water table study. On December 4th, 2019, thirteen (13) monitoring wells (piezometers) were installed on the site by registered geologist and licensed well constructor, Roger N. Smith (RG, License #10225).

Within each 70' x 350' tile dewatering area, 3 piezometers wells were installed (12 total). One additional well was installed approximately 20 feet north of the eastern tile system to collect barometric pressure. Each monitoring well consists of a 5-foot long, 1-inch diameter plastic PVC pipe capped with a plastic lid. The wells were installed approximately 3 feet below the surface, with 22-29 inches of pipe above ground surface. Special Standards were requested from and approved by the Oregon Water Resources Department. Silica filter sand was placed in the hole around the piezometer at the lowest 26 inches, followed by a 12-inch bentonite seal to the soil surface. A slit was sawed in the top of each pipe to allow the lid to be easily removed, and to release air pressure inside the well from the rising and lowering water table. Each well was assigned a number (Pz1 – Pz13) which was noted on metal start card tags and written in permanent marker on the pipe itself. Start cards for the wells were registered with the Oregon Water Resources Department.

Table 1. Measured and calculated Barodiver cord lengths relative to grade

Piezometer	Cord length (in.)	Cord length above grade (in.)	Cord length below grade (in.)
Pz1	57	27	30
Pz2	56	25	31
Pz3	57	29	28
Pz4	57	27	30
Pz5	57	28	29
Pz6	58	29	29
Pz7	57.5	27	30.5
Pz8	58	27	31
Pz9	56	27	29
Pz10	57.5	25.5	32
Pz11	57.5	25	32.5
Pz12	57	22	35

---

[1] Cobb, PR, Conta, JF, Steverson, ED, and Stull RL. Recommended Procedures and Standards for Conducting a Water Table Study, Version 1.0. Crop and Soil Environmental Sciences Department, Virginia Tech, Blacksburg, VA

DATA COLLECTION.

Barodiver data loggers were placed inside Pz's 1-12, between 28 and 35 inches below grade to collect water column pressure. One additional Barodiver was placed in Pz13 above the soil surface to collect atmospheric pressure for the study area. Technical specifications for the Barodiver data loggers are enclosed at the end of this report. The total cord length (CL) and cord length above grade (COG) for each Barodiver was measured manually and recorded (see Table 1). Data was collected automatically every four hours (6 times per day) from January 9th, 2020 until approximately 9:00 am on May 1st, 2020. Data for the date of the installation (January 8th) was omitted to avoid false readings caused by system testing, and an artificially high-water table immediately after the wells were dug. Each piezometer was surrounded by wooden stakes and caution tape for protection (see Figure 1 below).

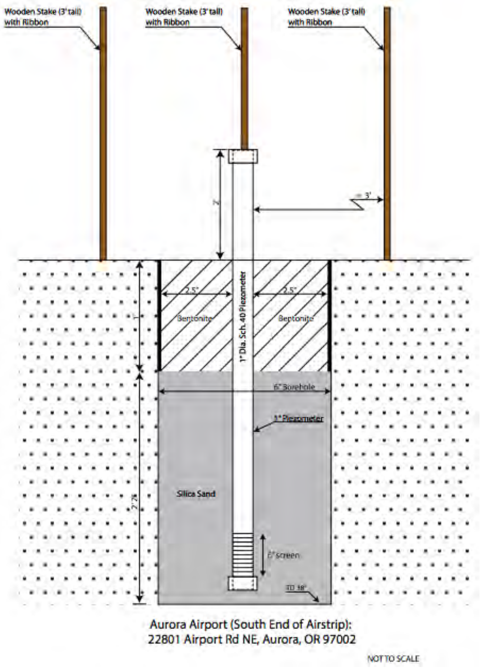


Figure 1. Piezometers were installed approximately 3 feet below grade and pressure sensors were hung from the top of the pipe. The well was sealed with bentonite clay.

The site was visited once each month during the study; a total of 5 times after setup. Each site visit consisted of the following:

1. Inspect each well to ensure they are still fully functioning and had not been tampered with
2. Download data from Barodiver data loggers onto laptop using USB data port
3. Visually inspect the tile dewatering system and assess flow

After all data was collected, the water level (WL) for each well was then determined using the following equation, where ρ is the density of water (1000 kg/m3) and g is the acceleration due to gravity (9.80665 m/s2s). :

$$WL = COG - CL + 9806.65 \frac{P_{diver} - P_{baro}}{\rho * g}$$

RAINFALL MONITORING

Precipitation data for January 2020 through April 2020 was collected from the Aurora State Airport weather station in Aurora, Oregon (45.2485, -122.7686). Normal precipitation levels were determined using the US Normal Data (1981-2010) from the National Oceanic and Atmospheric Administration (NOAA), obtained from the NRCS National Water and Climate

Center (https://www.wcc.nrcs.usda.gov/climate/navigate_wets.html). However, because the NOAA uses data from 1981-2010 to determine Climate Normals, this year's precipitation was also compared to the previous two years (2018 and 2019). Precipitation was found to be only 5% drier than last year (2019). Daily precipitation levels were monitored and compared to water table levels.

FINDINGS:

Precipitation

The precipitation for the past three years in the Aurora area has been less than what previously has been considered "normal" based on long term records. Table 2 below shows the monthly precipitation for 2020, 2019, and 2018 from data from the airport weather station. It is unknown whether there is going to be a new normal, however we can say this study was performed under precipitation conditions that were only 5% different than the previous year.

Table 2 – Monthly precipitation totals in inches for 2018, 2019, 2020.

Month	2020	2019	2018
January	7.06	3.49	5.57
February	1.64	3.97	2.06
March	2.53	1.54	2.97
April	1.32	4.24	5.04
Total	**12.55**	**13.24**	**15.64**

Table 3 shows total precipitation for the months of January through April 2020. Although the month of January was above normal, February, March, and April were drier than normal. The expected normal and the measured precipitation for the months of the study were totaled, and overall, the precipitation was found to be 70% of historic normal. Daily precipitation levels are graphed in Figure 2, below.

Table 3 – Percent of NOAA Normal precipitation for January 2020 – April 2020

Month	Normal (inches)	Measured (inches)	Percent of Normal
January	5.87	7.06	120
February	4.75	1.64	35
March	4.23	2.53	60
April	3.13	1.32	42
Total	**17.98**	**12.55**	**70**

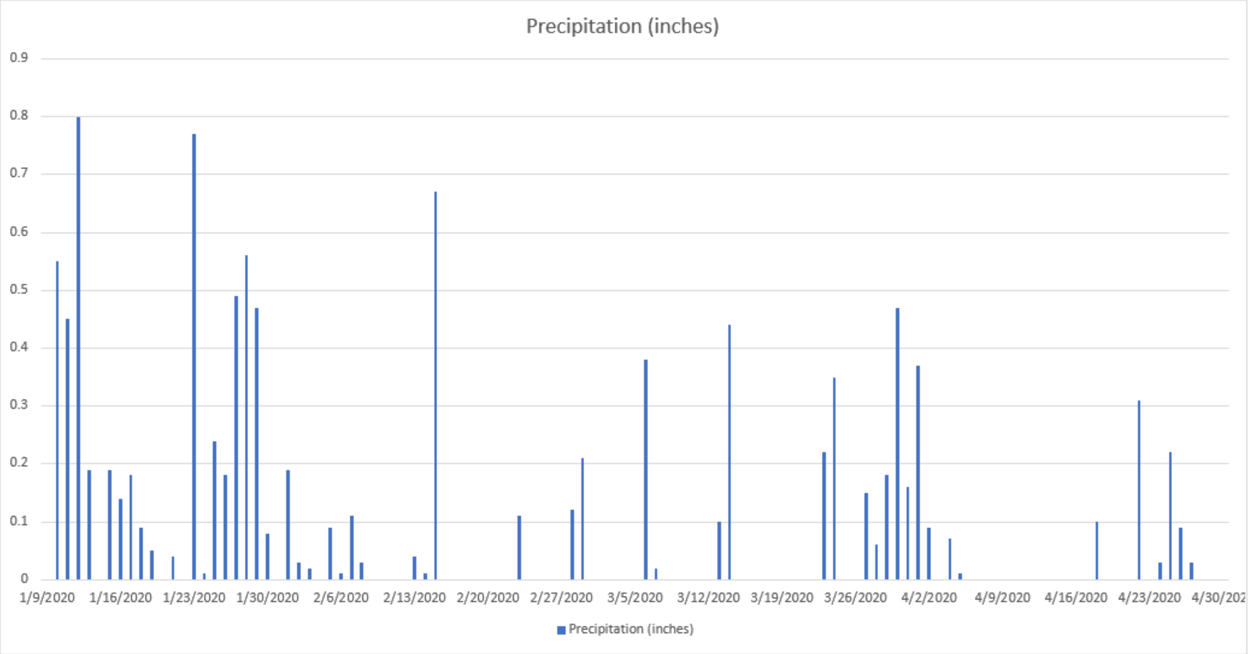


Figure 2 –Daily precipitation (inches) from the Aurora State Airport weather station (June 9th – May 1st).

Precipitation for May and June of 2020 was greater than normal, with 2.82 inches of rain in May (119% of normal), and 2.96 inches of rain in the first half of June (147% of monthly normal as of June 16th). EMS returned to the site on June 16th to manually measure the water table in each well. Each of the 12 piezometers was dry (no standing water in the well). 0.24 inches of rain fell on the day the measurements were taken. The ten days prior to the measurements each had precipitation, with the biggest rain event being on June 15th when 0.84 inches of rain fell. Daily climate data for each month is enclosed at the end of this report.

Well data and water table levels

A total of 681 readings were automatically collected every 4 hours from each piezometer during the study. The results were variable across all wells. Some of the wells exhibited periods of time where the water table was less than 12” from the ground surface (up to 37 readings a row in Pz9) whereas others had none at all. The average water table depth across all wells was 21” and 28” from ground surface, before and after the installation of the TDS respectively. Pz4 and Pz11 were always deeper than 12” throughout the study. The shallowest water table depth was in Pz12, at 3” on the dates of 01/16/2020 and 1/29/2020. Most shallow water table readings occurred in January, which had 120% of normal rainfall, and prior to the tile dewatering system being installed. Average and minimum water table depths before the tile dewater system was installed are summarized in Table 4, below. Piezometers are located on either the east or west side of the runway approach and departure areas.

Table 4. Average and highest water table levels, in inches, before the TDS installation (01/09/2020 – 01/22/2020.

Piezometer	Average water level	Highest water level	Location
Pz1	20	8	West
Pz2	19	6	West
Pz3	22	7	West
Pz4	26	13	West
Pz5	28	22	East
Pz6	20	9	East
Pz7	21	9	East
Pz8	17	6	East
Pz9	14	4	East
Pz10	23	12	East
Pz11	29	23	West
Pz12	17	3	West
Average	**21**	**12**	

After the tile dewatering system was completed, only seven of the twelve wells had occurrences of the water table being less than 12" from the surface (Table 5). These shallow water table events were brief periods that to correlate with significant rain events of 0.5 inches of rain or more over a 24-hour period. The average water table depth across all wells was 28" inches from the surface between 01/23/2020 and 05/01/2020.

Table 5. Average and highest water table levels, in inches, after TDS installation (01/23/2020 – 05/2020)

Piezometer	Average water level	Highest water level	Location
Pz1	24	5	West
Pz2	26	7	West
Pz3	27	5	West
Pz4	29	17	West
Pz5	28	10	East
Pz6	28	10	East
Pz7	31	30	East
Pz8	31	28	East
Pz9	28	17	East
Pz10	30	6	East
Pz11	30	17	West
Pz12	25	3	West
Average	**28**	**13**	

Daily precipitation is graphed along with water table levels in the enclosed hydrographs. All shallow water table readings occurred in January, which had 120% of normal rainfall, except for Pz1, which had one reading on 2/16/2020, and Pz12, which had three readings on 2/16/2020. 0.67 inches of rainfall occurred on the previous day (2/15/2020). The longest duration that any well had a shallow water table of 12" or less was 23 consecutive readings (about 3.8 days). See Table 6 below. In Pz1, Pz2, Pz5, Pz6, Pz10, and Pz12, the longest duration of shallow water table conditions occurred around the dates of 01/27/2020 - 01/29/2020, when approximately 1.5 inches of rain fell. On average, the water table was only above 12 inches for about 0.9 days after significant rain events. According to the standards recommended by Virginia Tech, less than 21 consecutive days of high-water table conditions is considered acceptable.

Table 6. Consecutive time of shallow water table conditions for each piezometer, after installation of TDS (01/23/2020 – 05/01/2020).

Piezometer	# of readings	Consecutive hrs.	Consecutive days	Dates
Pz1	13	5	2.2	1/28 - 1/30
Pz2	12	48	2.0	1/27 - 1/29
Pz3	0	0	0	--
Pz4	0	0	0	--
Pz5	4	16	0.7	1/28
Pz6	4	16	0.7	1/23, 1/28
Pz7	0	0	0	--
Pz8	0	0	0	--
Pz9	0	0	0	--
Pz10	10	40	1.7	1/27 - 1/29
Pz11	0	0	0	--
Pz12	23	92	3.8	01/27 - 01/31
Average	**6**	**22**	**0.9**	

Since May and June were wetter than normal, EMS returned to the site on June 16th to manually measure the water table in each well. Each well was dry, with no standing water at the bottom of the well.

Tile Dewatering System

The tile dewatering system was completed on January 23rd, 2020. During each site visit, water was observed flowing from the field collection tile into the outfall pipes. Water was also observed draining from the outlet of the pipe and discharging to the swales near the east and west property lines. Prior to the installation of the TDS, ten out of twelve wells had a high-water table of 12” or less from the surface. After the installation of the TDS, only seven out of twelve wells had a high-water table, and only for relatively short periods during significant rain events. The TDS is functioning as designed and has contributed to lowering the water table.

Figure 3 –Tile dewatering trenches were installed 15-52 inches below grade and filled with 12” EzFlow bundles. 4” pipes at the bottom of the trench sloped are at 0.2-0.4%.

Figure 4 –24" silt traps were installed at the inlet end of each tight line outfall, which discharged toward existing drainage swales on the site. Photo taken facing west toward the west property line (fence) with Hubbard Cuttoff Rd. NE in the background.

CONCLUSIONS:

1. Precipitation for the months of January through April 2020 was only 5% drier than 2019. When compared to the NOAA 1981-2010 Climate Normals, precipitation during the study was 70% of “normal”.

2. May and June were wetter than normal. May had 119% of normal precipitation. In June, 147% of the monthly normal precipitation had accumulated in the first half of the month. EMS returned to the site in mid-June to manually measure the water table levels.

3. Between January 9th and January 22nd, the average water table depth for each piezometer ranged between 14” (Pz9) and 29” (Pz10) from the ground surface and averaged 21” across all wells.

4. After installation of the tile dewatering system on January 23rd, the average water table depth across all wells increased to 28”. In half of the wells, the water table never rose above 12” from the surface after the TDS was installed.

5. Most shallow water table readings (less than 12” from the ground surface) occurred in January, which had 120% of normal precipitation. Spikes in the water table levels appear to correlate with significant rain events of 0.5 inches or more over 24 hours.

6. The most consecutive number of days that the water table was rose above 12” from the soil surface was about 3.8 days in Pz12. On average, the water table lingered above 12” for about 0.9 days, although five out of twelve wells had no shallow water table readings after the TDS was installed. Less than 21 consecutive days of shallow water table is considered acceptable for onsite wastewater treatment.

7. No water was observed in the bottom of the wells when manual measurements were taken on June 16th, 2020. This was following an unusually wet June, which had already accumulated 2.96 inches of the total normal 2.02 inches of monthly precipitation in the first half of the month. 0.84 inches of rain fell the previous day (June 15th). The first half of June's 2.96 inches amounts to 146% of the whole months normal or 293% of the first half's expected 1.01 inches.

8. Onsite wastewater treatment appears feasible. Effluent will be highly treated to Treatment Standard 2 and disinfected, using the existing Advantex AX100 textile filters, or similar technology with Ultra Violet Disinfection when these future repair drainfields are needed. High water table levels only occur after significant rain events and for relatively short durations (less than 21 consecutive days).

9. This site is protected from public access by fencing and constant observation, thereby further limiting the risk of human contact with sewage.

10. Further, the existing drainfield has been in use for fifteen years in similar soils and treatment with no signs of failure.

RECOMMENDATIONS: The following additional steps or services appear to be needed:

1. Feasibility review. The result of this study will need to be presented to and assessed by Marion County and/or Oregon Department of Environmental Quality (DEQ) to reevaluate feasibility of the site for on-site wastewater treatment.

2. On-site Wastewater Treatment System Design. A final design will need to be prepared that meets DEQ specifications for a Water Pollution Control Facility Permit.

DISCLOSURE: The information and statements in this report are true and accurate to the best of our knowledge. Neither Environmental Management Systems, Inc., nor the undersigned have any economic interests in the project.

Thank you for your business. We look forward to assisting you to achieve your development goals. If you have any questions, please contact Emma Eichhorn, REHS, or me at 503-353-9691.

Sincerely

Robert F. Sweeney, MS, REHS
President
ENVIRONMENTAL MANAGEMENT SYSTEMS, INC.

Enclosures:
1. Site Plan
2. Tile Dewatering System Details
3. Tax Lot Map
4. Hydrographs for piezometers Pz1 – Pz12
5. Barodiver data logger spec sheet
6. Precipitation data for the Aurora State Airport weather station

Aurora Airport
Located at Airport & Keil Roads NE
Aurora, OR 97002

DRAINFIELD AREA
SEE PAGE 2

HUBBARD CUTOFF RD NE

THIS DOCUMENT IS NOT A SURVEY. LOCATIONS OF SITE FEATURES ARE APPROXIMATE.

SCALE 1"= 250'

19-0054

12/10/2019

1 OF 3

AABC-HD AVIATION

T: 4S, R: 1W, Sec 2 & 11, TL: 200, 203, 400, 401, 800, 900 & 1600

AIRPORT SITE PLAN

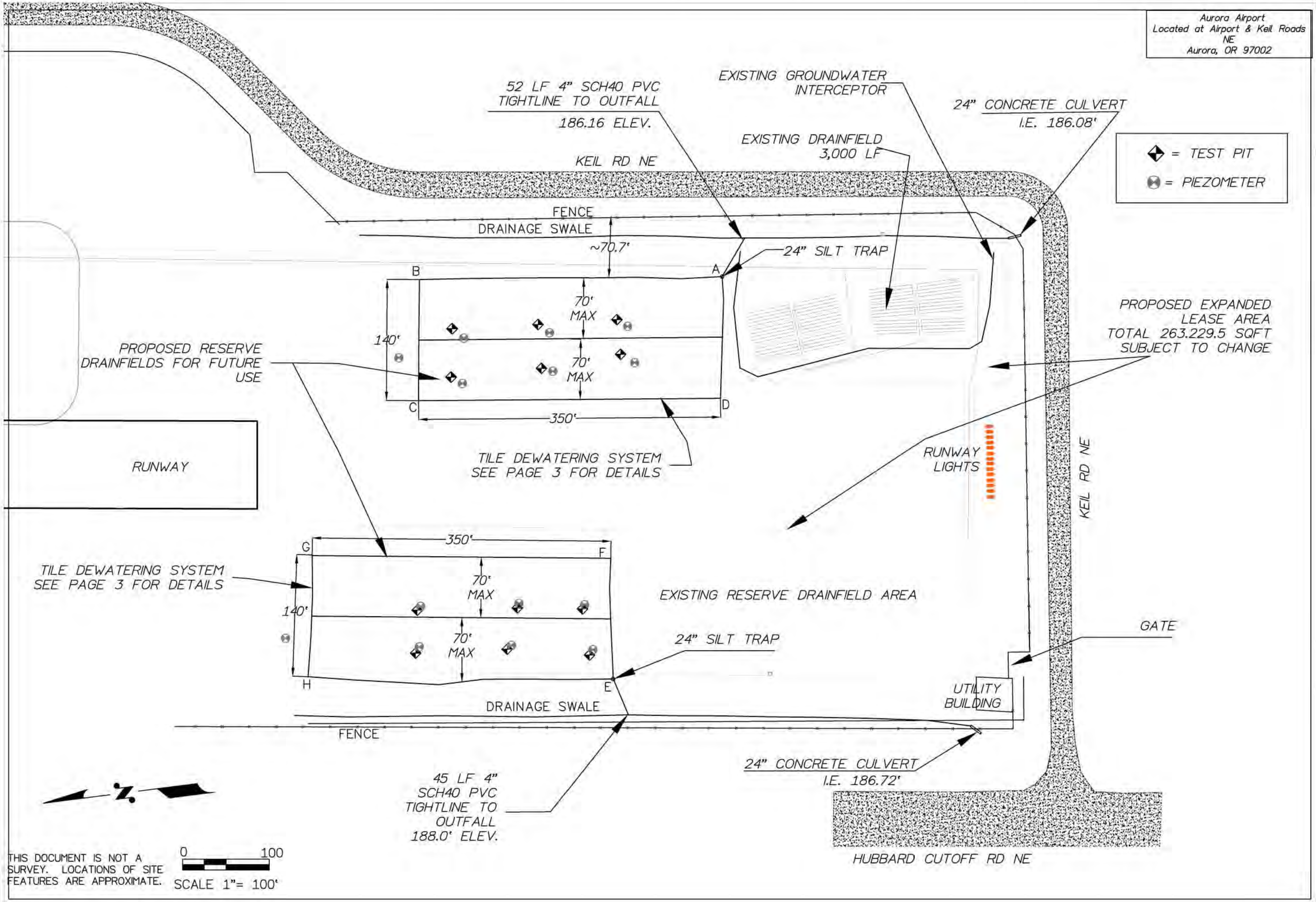
CHECKED:

DRAWN:

4080 SE International Way
Suite B112
Milwaukie, OR 97222

503-353-9691
360-735-1109
www.envmgtsys.com

ENVIRONMENTAL MANAGEMENT SYSTEMS, INC.



THIS DOCUMENT IS NOT A SURVEY. LOCATIONS OF SITE FEATURES ARE APPROXIMATE.

19-0054

12/10/2019

2 OF 3

AABC-HD AVIATION

T: 4S, R: 1W, Sec 2 & 11, TL: 200, 203, 400, 401, 800, 900 & 1600

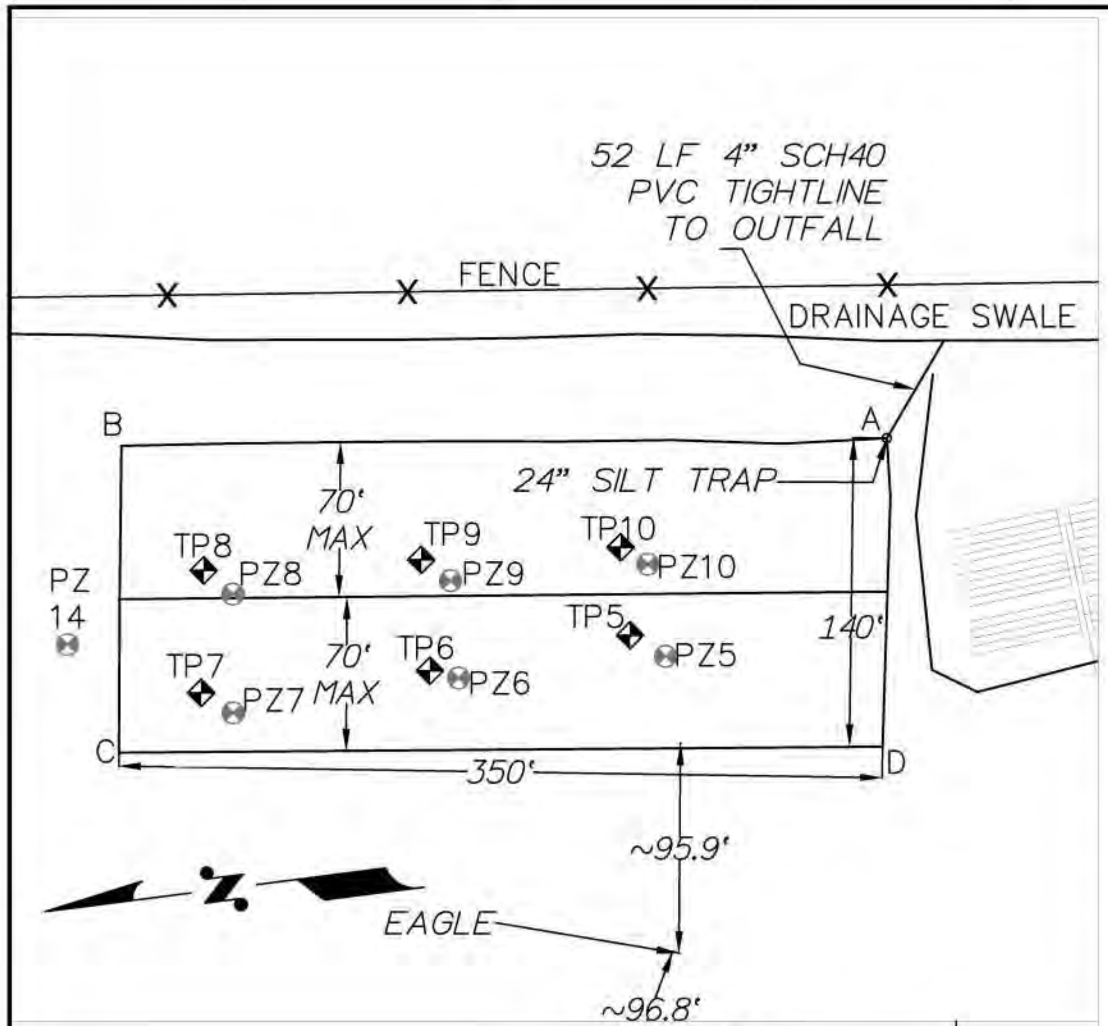
SITE PLAN

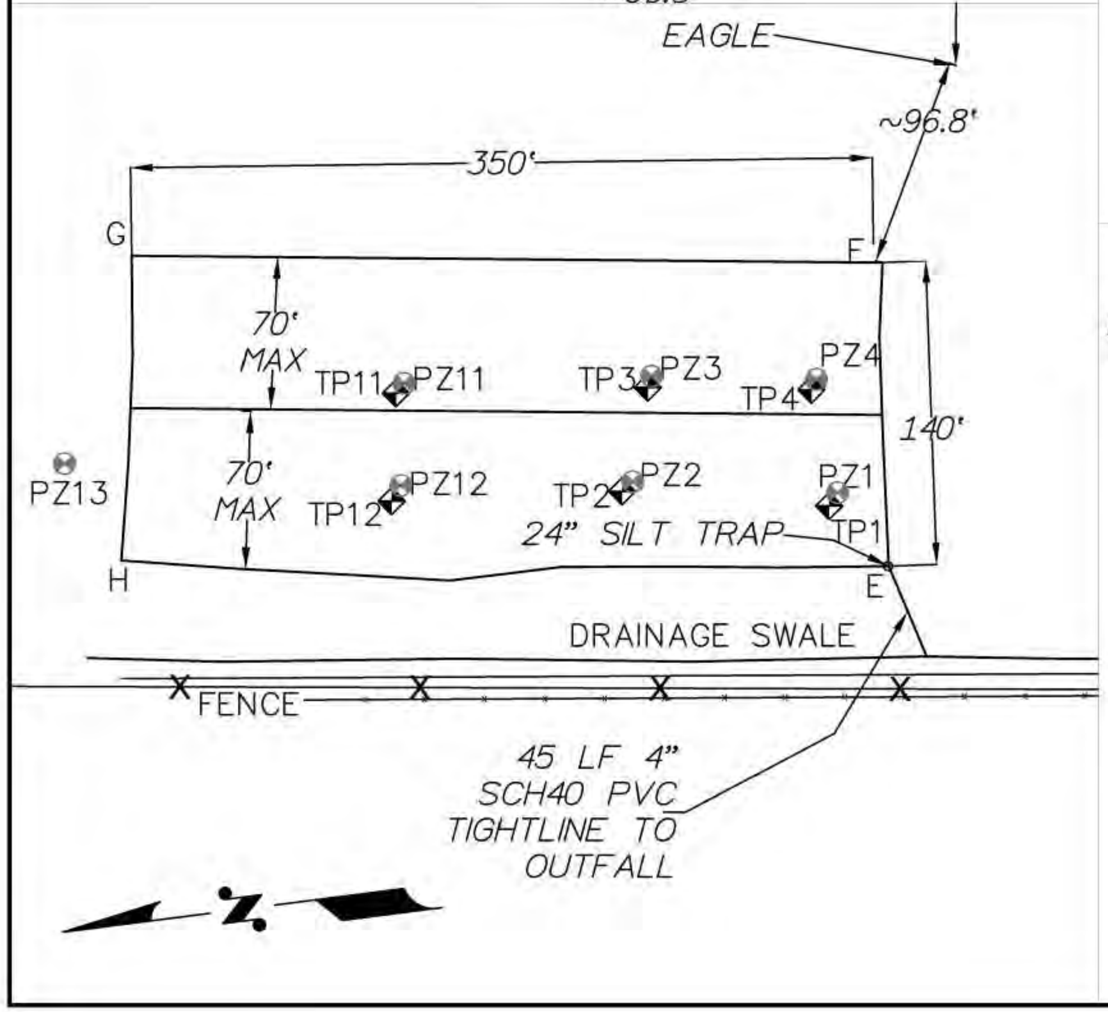
CHECKED:

DRAWN:

EMS
4080 SE International Way
Suite B112
Milwaukie, OR 97222
503-353-9691
360-735-1109
www.envmgtsys.com
ENVIRONMENTAL MANAGEMENT SYSTEMS, INC.

Aurora Airport
Located at Airport & Keil Roads NE
Aurora, OR 97002



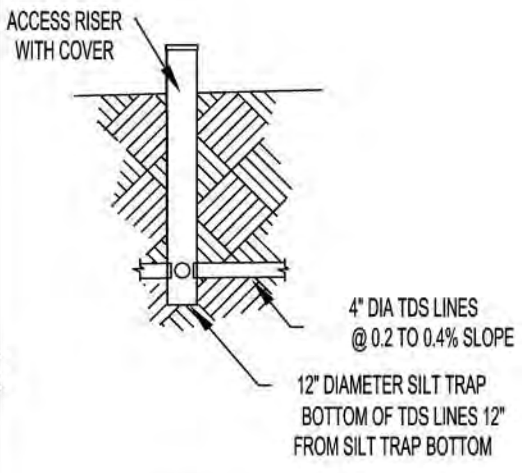


East TDS

POINT	Trench Elevation (feet)	Surface Elevation (feet)	Trench Depth (feet)	Trench Depth (inches)
A	186.72	189.28	2.57	30.79
B	187.59	189.71	2.11	25.36
C	187.94	191.44	3.50	41.94
D	187.07	191.42	4.35	52.24

West TDS

POINT	Trench Elevation (feet)	Surface Elevation (feet)	Trench Depth (feet)	Trench Depth (inches)
E	188.45	189.73	1.29	15.42
F	188.73	192.07	3.35	40.17
G	189.43	192.42	3.00	35.94
H	189.15	190.28	1.13	13.59



SECTION - SILT TRAP

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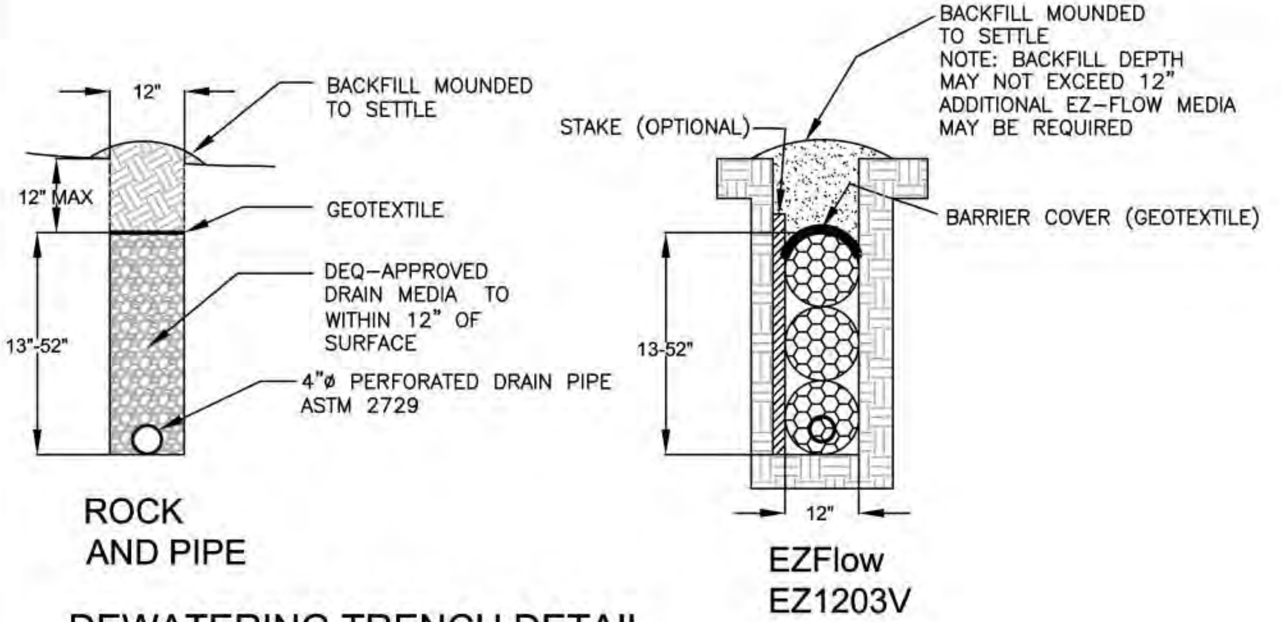


DEWATERING TRENCH DETAIL

SCALE : NTS

REFER TO TABLE FOR TRENCH DEPTHS. SLOPE MUST BE 0.2–0.4% IN FIELD COLLECTION DRAINAGE TILE.

19-0054

12/10/2019

3 OF 3

AABC-HD AVIATION

T: 4S, R: 1W, Sec 2 & 11, TL: 200, 203, 400, 401, 800, 900 & 1600

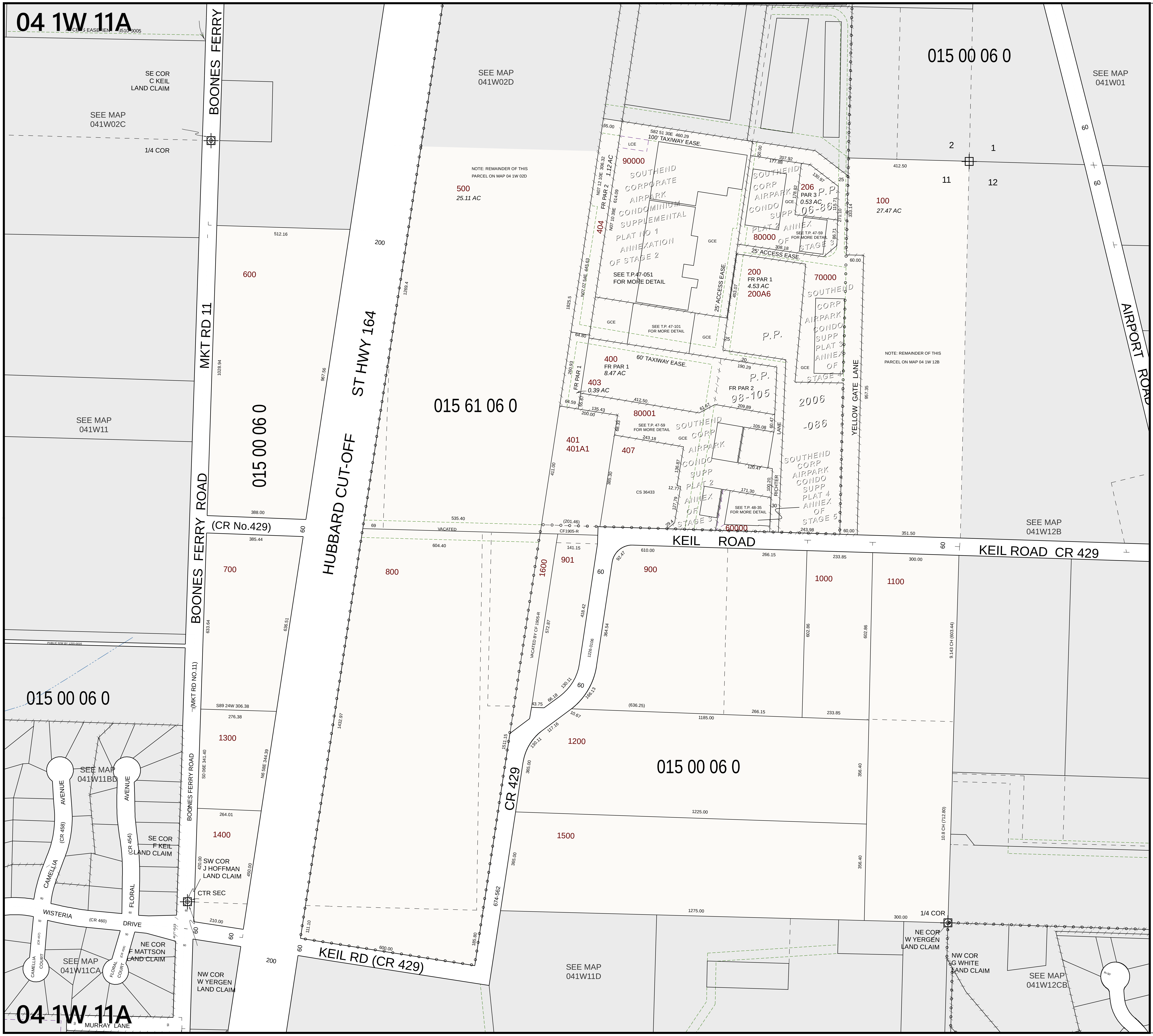
TILE DEWATERING SYSTEM DETAIL

CHECKED:

DRAWN:

EMS
503-353-9691
360-735-1109
www.envmgtsys.com
4080 SE International Way
Suite B112
Milwaukie, OR 97222
ENVIRONMENTAL MANAGEMENT SYSTEMS, INC.

04 1W 11A



MARION COUNTY, OREGON

NE1/4 SEC11 T4S R1W W.M.

SCALE 1" = 200'

LEGEND

LINE TYPES

Taxlot Boundary	Historical Boundary
Road Right-of-Way	Easement
Railroad Right-of-Way	Railroad Centerline
Private Road ROW	Taxcode Line
Subdivision/Plat Bndry	Map Boundary
Waterline - Taxlot Bndry	Waterline - Non Bndry

CORNER TYPES

+ 1/16TH Section Cor.	⌖ 1/4 Section Cor.
◎ DLC Corner	16 15 / 21 22 Section Corner

NUMBERS

Tax Code Number

000 00 00 0

Acreage 0.25 AC — All acres listed are Net Acres, excluding any portions of the taxlot within public ROWs

NOTES

Tick Marks: A tick mark in the road indicates that the labeled dimension extends into the public ROW

200.00

175.00

CANCELLED NUMBERS

101
200A1
200A2
200A5
201
202
203
204
207
300
401
402
405
406

DISCLAIMER: THIS MAP WAS PREPARED FOR ASSESSMENT PURPOSES ONLY

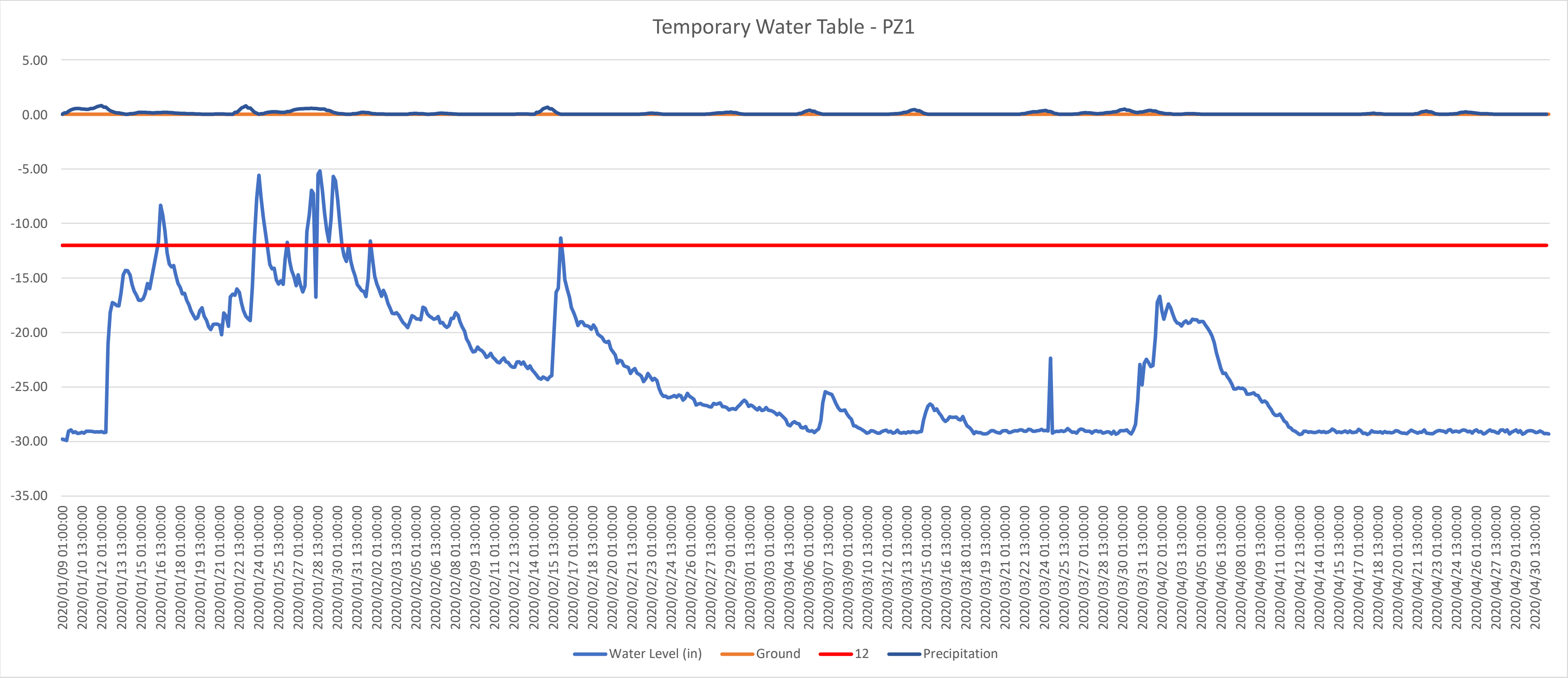
Marion County OREGON

Assessors Office Cartography Dept

FOR ADDITIONAL MAPS VISIT OUR WEBSITE AT www.co.marion.or.us

PLOT DATE: 2/13/2020

04 1W 11A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Ground
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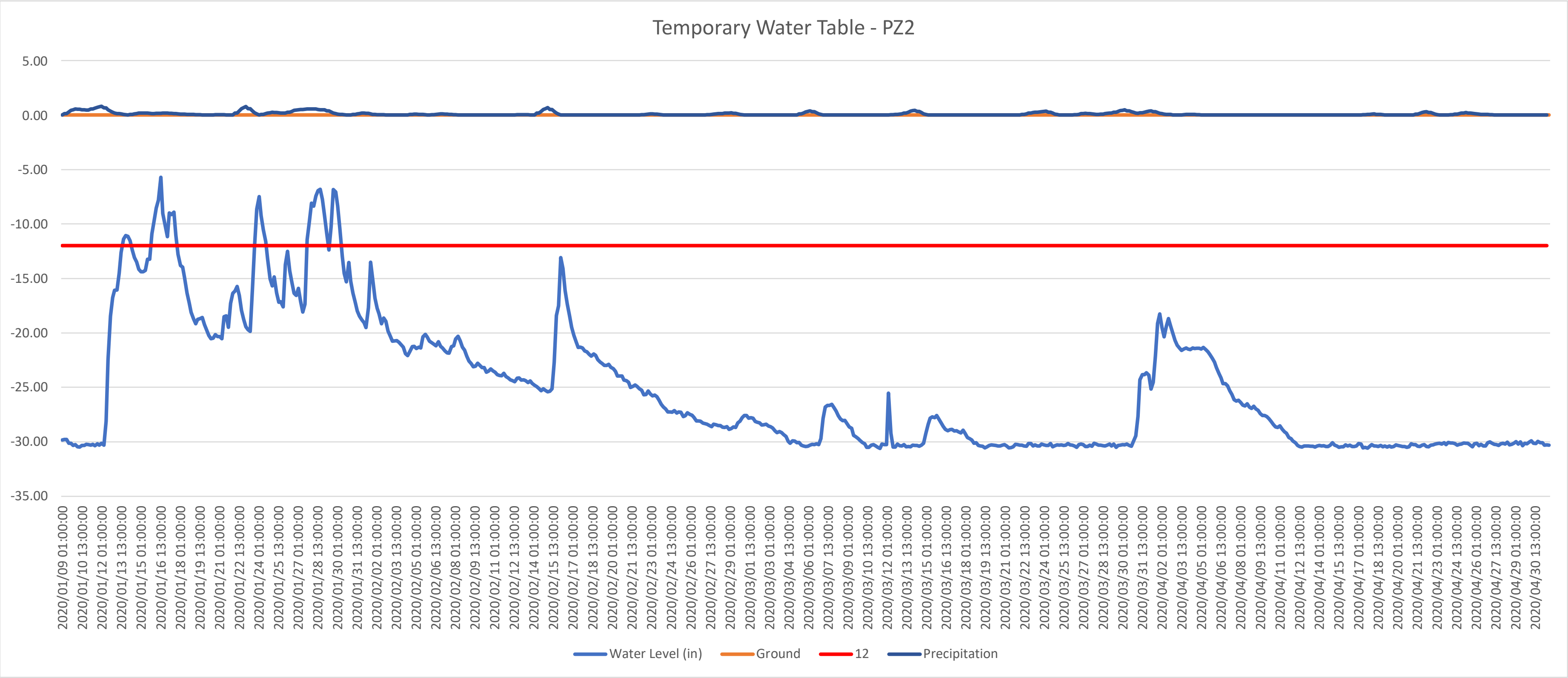
Precipitation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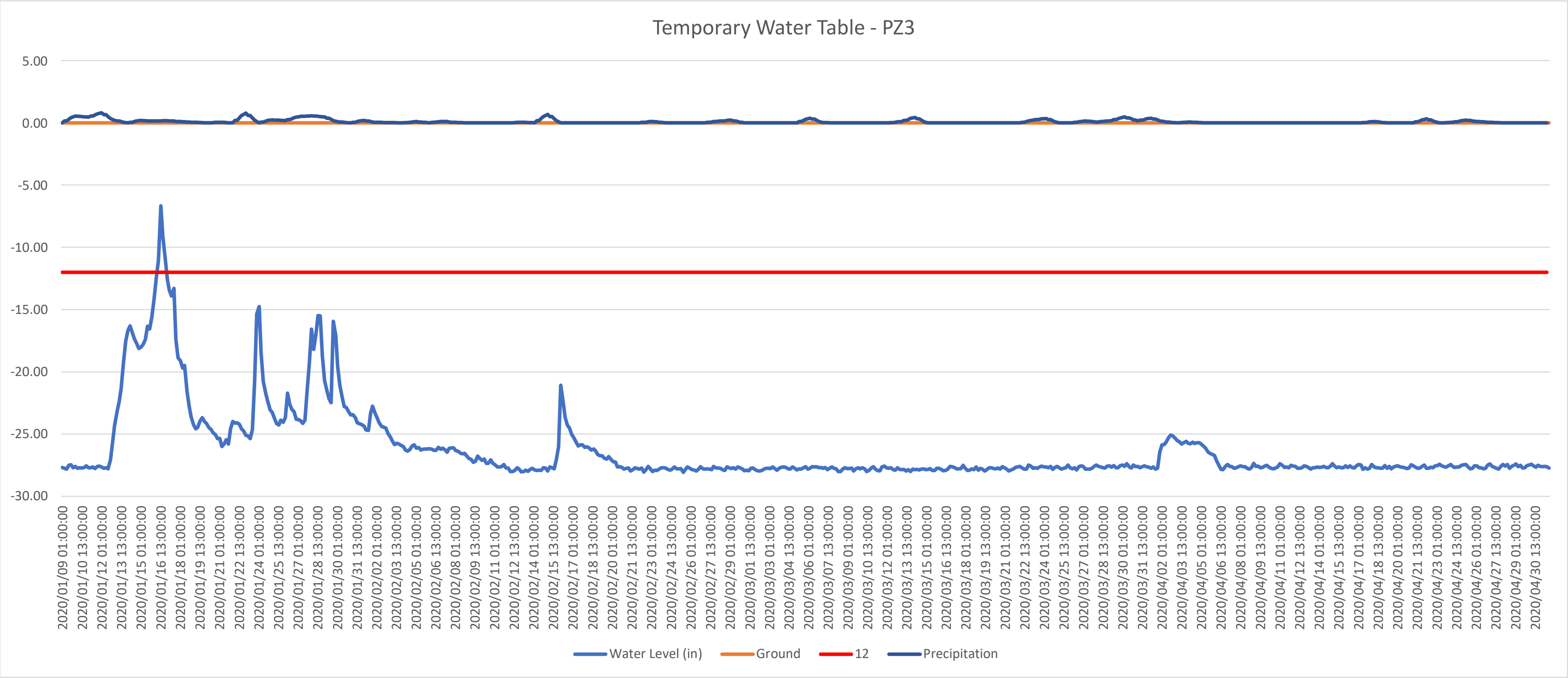
Ground
12
Precipitation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Ground
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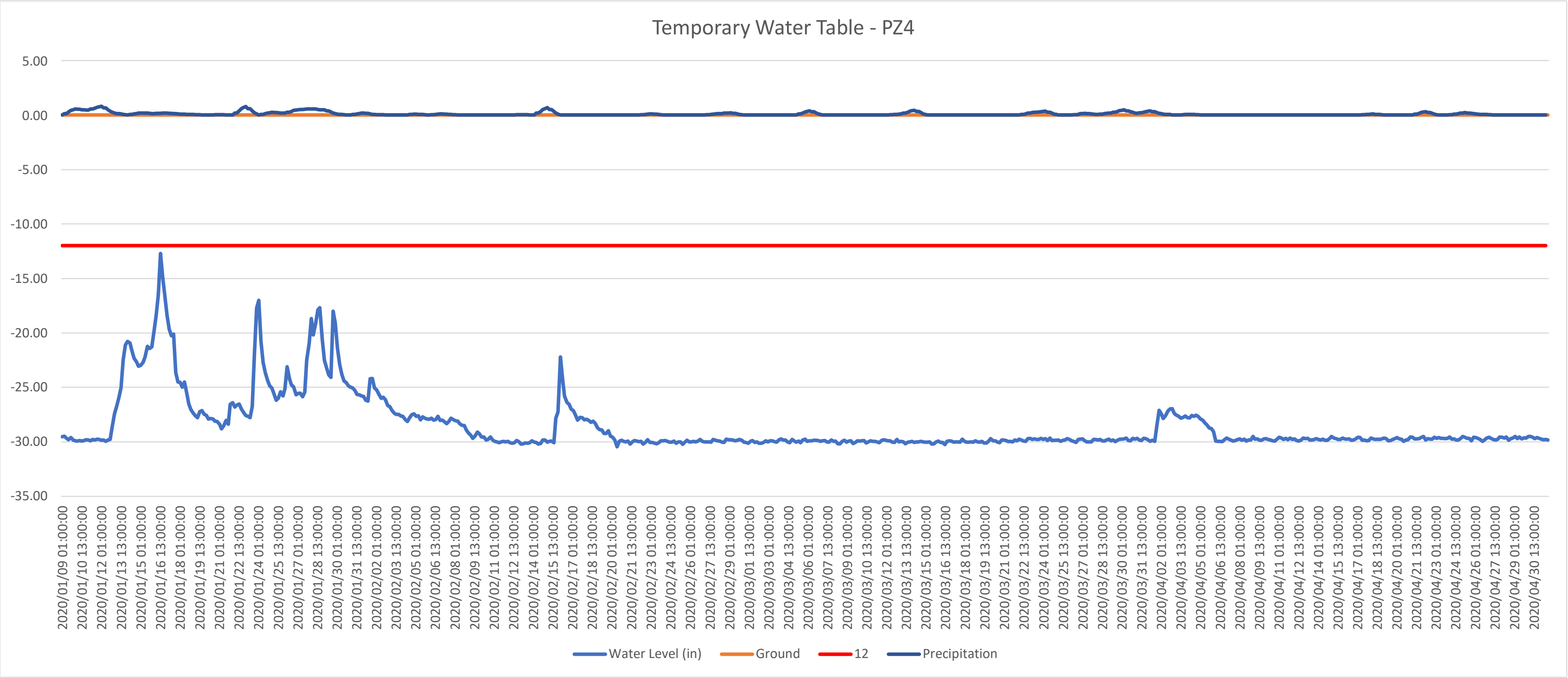
Precipitation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Ground
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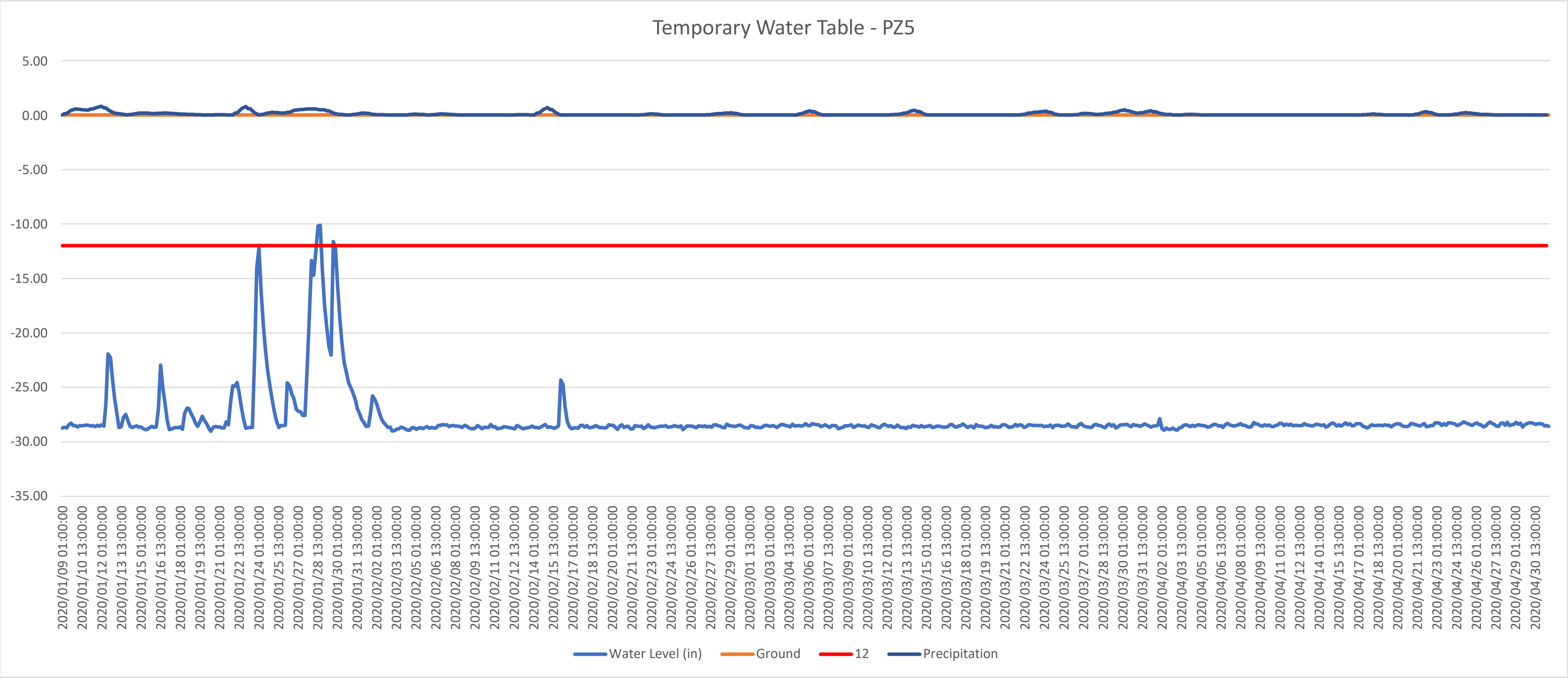
Precipitation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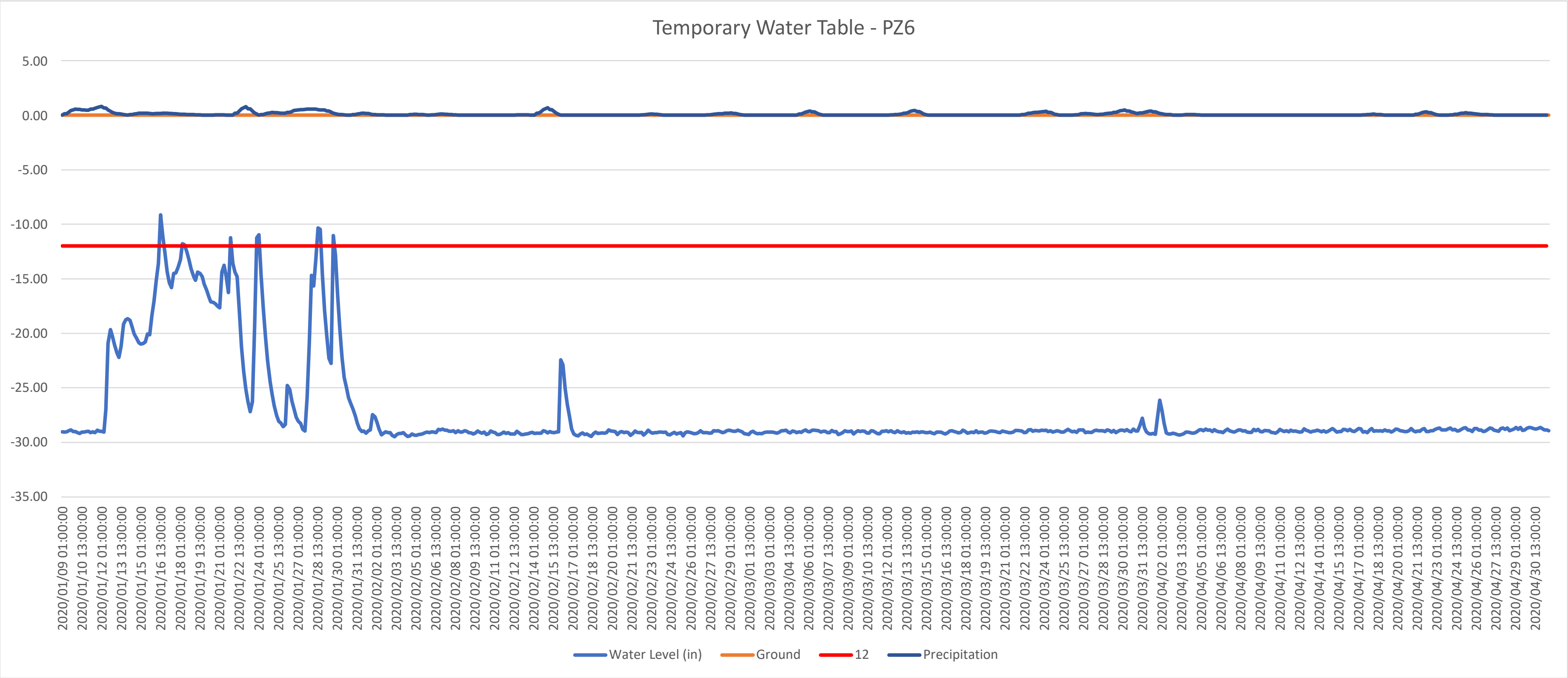
Water Level (in)
Ground
12
Precipitation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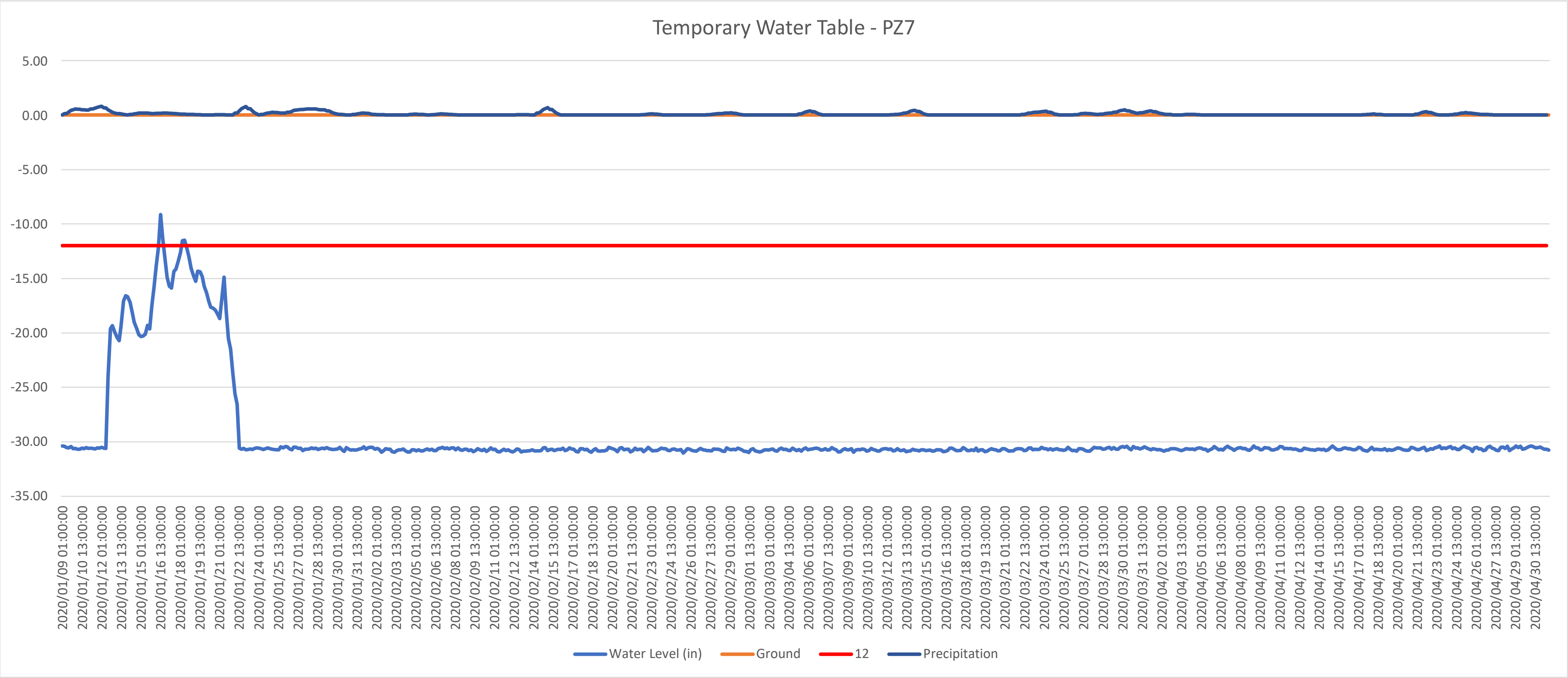
Water Level (in)
Ground
12
Precipitation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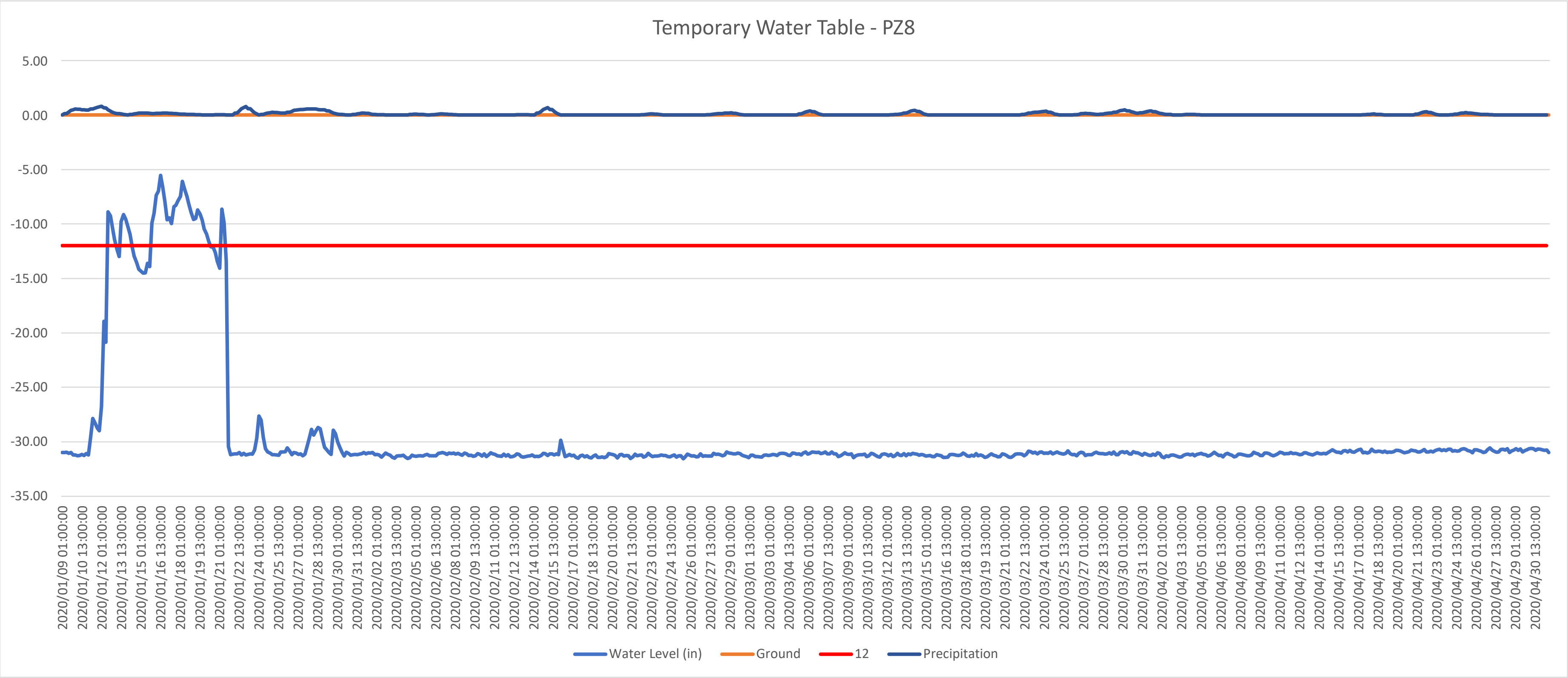
Water Level (in)
Ground
12
Precipitation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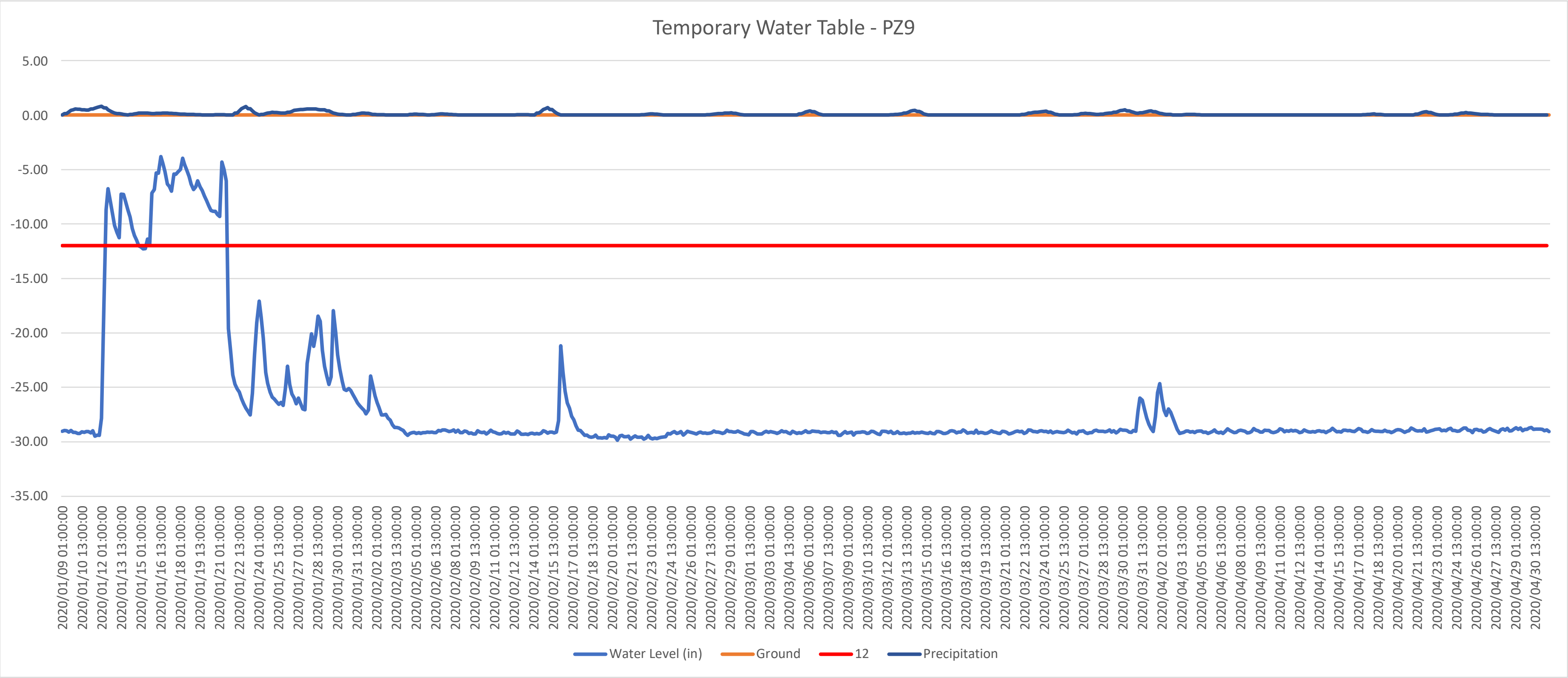
Water Level (in)
Ground
12
Precipitation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Water Level (in)
Ground
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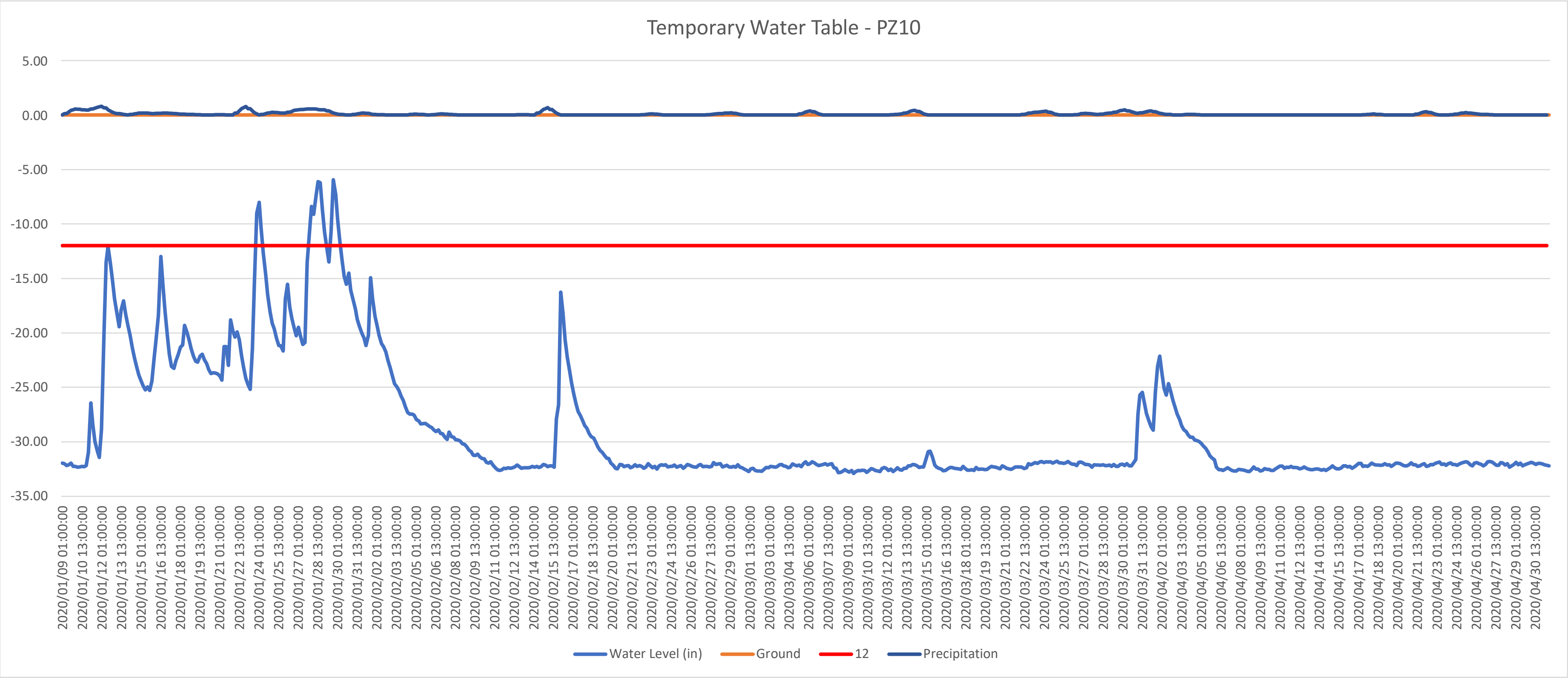
Precipitation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Water Level (in)
Ground
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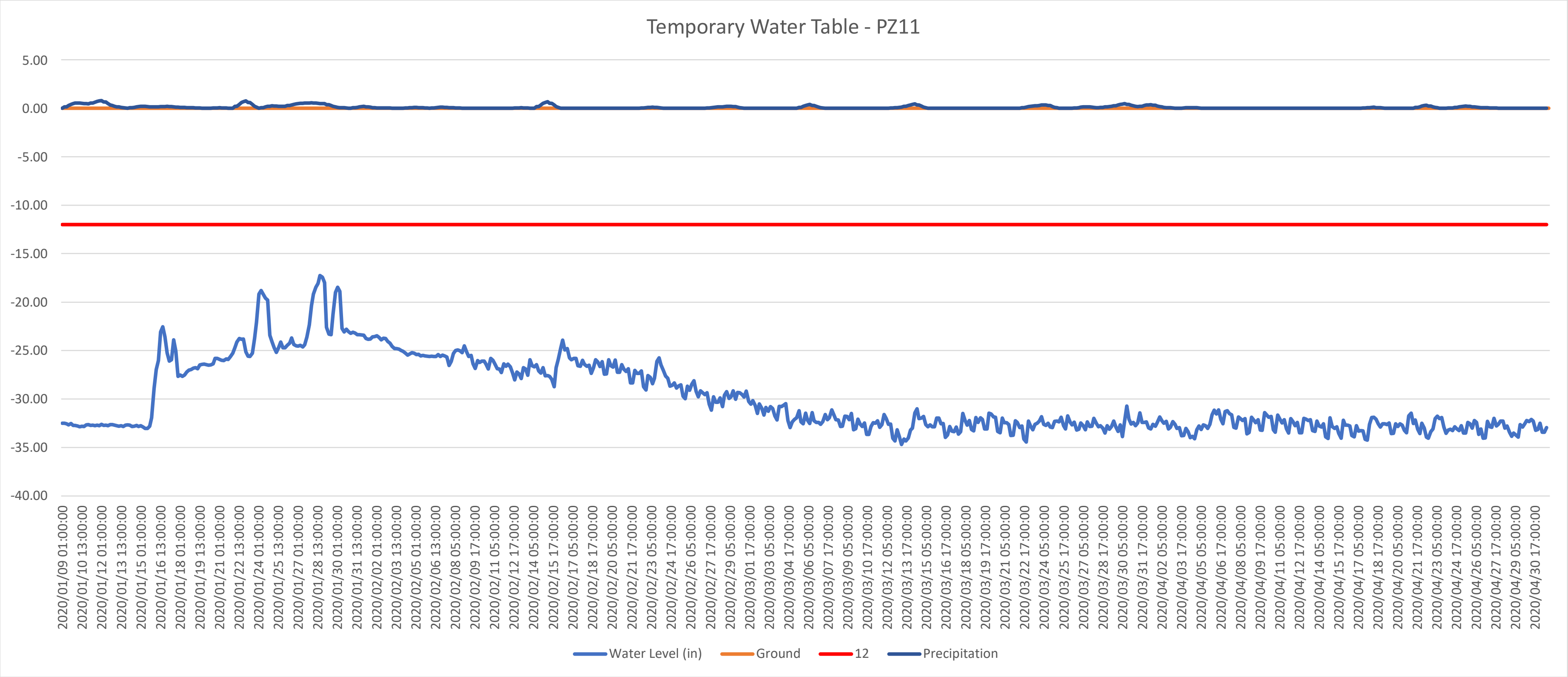
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Ground
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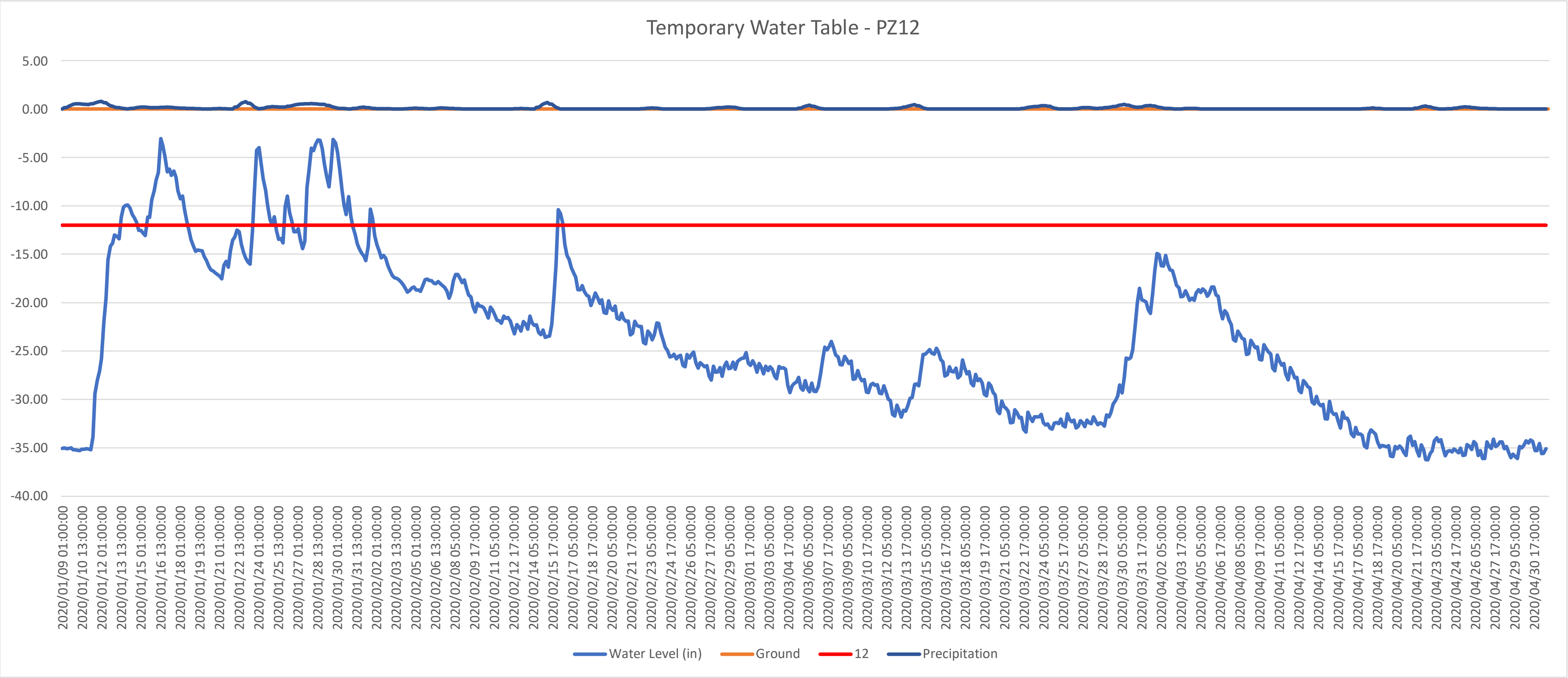
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2020/03/16 17:00:00
2020/03/18 05:00:00
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2020/04/05 05:00:00
2020/04/06 17:00:00
2020/04/08 05:00:00
2020/04/09 17:00:00
2020/04/11 05:00:00
2020/04/12 17:00:00
2020/04/14 05:00:00
2020/04/15 17:00:00
2020/04/17 05:00:00
2020/04/18 17:00:00
2020/04/20 05:00:00
2020/04/21 17:00:00
2020/04/23 05:00:00
2020/04/24 17:00:00
2020/04/26 05:00:00
2020/04/27 17:00:00
2020/04/29 05:00:00
2020/04/30 17:00:00
Water Level (in)
Ground
12
Precipitation

Technology Sheet

Baro-Diver – DI800

Technical Specifications

Length	4.33 in
Diameter	0.87 in
Weight	3.67 oz
Memory	72,000 measurements with backup; continuous and fixed length memory
Wetted parts	
housing	stainless steel (316L)
o-rings	Viton ®
pressure sensor	piezo resistive ceramic (Al_2O_3) with thermal compensation
cap	Nylon PA6 30% glass fiber
nose cone	ABS
Battery life	up to 10 years (dependent on usage)
Sample interval	½ second to 99 hours
Sample method	fixed interval
Communication	RS232

Pressure

Part number	**DI 800**	
Range	4.9	ftH_2O
Accuracy[+]	± 0.2	inH_2O
Resolution	0.01	inH_2O

Temperature

Range	-4 to 176 °F
Calibrated	14 to 122 °F
Accuracy[+]	± 0.18 °F
Resolution	0.018 °F

[+] typical

Actual size

M = membrane
Dimensions in mm

Climatological Data for AURORA STATE AP, OR - January 2020

Date	Max Temperature	Min Temperature	Avg Temperature	GDD Base 40	GDD Base 50	Precipitation	Snowfall	Snow Depth
2020-01-01	55	47	51.0	11	1	0.09	M	M
2020-01-02	51	43	47.0	7	0	0.00	M	M
2020-01-03	62	45	53.5	14	4	0.16	M	M
2020-01-04	50	40	45.0	5	0	0.29	M	M
2020-01-05	50	42	46.0	6	0	0.21	M	M
2020-01-06	52	45	48.5	9	0	0.44	M	M
2020-01-07	56	46	51.0	11	1	0.24	M	M
2020-01-08	47	38	42.5	3	0	0.15	M	M
2020-01-09	42	33	37.5	0	0	T	M	M
2020-01-10	47	37	42.0	2	0	0.55	M	M
2020-01-11	46	42	44.0	4	0	0.45	M	M
2020-01-12	46	38	42.0	2	0	0.80	M	M
2020-01-13	40	37	38.5	0	0	0.19	M	M
2020-01-14	42	32	37.0	0	0	T	M	M
2020-01-15	49	27	38.0	0	0	0.19	M	M
2020-01-16	43	29	36.0	0	0	0.14	M	M
2020-01-17	43	30	36.5	0	0	0.18	M	M
2020-01-18	51	40	45.5	6	0	0.09	M	M
2020-01-19	55	42	48.5	9	0	0.05	M	M
2020-01-20	48	39	43.5	4	0	0.00	M	M
2020-01-21	51	41	46.0	6	0	0.04	M	M
2020-01-22	M	M	M	M	M	M	M	M
2020-01-23	56	51	53.5	14	4	0.77	M	M
2020-01-24	57	48	52.5	13	3	0.01	M	M
2020-01-25	58	47	52.5	13	3	0.24	M	M
2020-01-26	55	46	50.5	11	1	0.18	M	M
2020-01-27	53	41	47.0	7	0	0.49	M	M
2020-01-28	53	46	49.5	10	0	0.56	M	M
2020-01-29	49	46	47.5	8	0	0.47	M	M
2020-01-30	55	41	48.0	8	0	0.08	M	M
2020-01-31	62	54	58.0	18	8	0.00	M	M
Average|Sum	50.8	41.1	46.0	201	25	7.06	M	M

Climatological Data for AURORA STATE AP, OR - February 2020

Date	Max Temperature	Min Temperature	Avg Temperature	GDD Base 40	GDD Base 50	Precipitation	Snowfall	Snow Depth
2020-02-01	58	41	49.5	10	0	0.19	M	M
2020-02-02	46	31	38.5	0	0	0.03	M	M
2020-02-03	47	30	38.5	0	0	0.02	M	M
2020-02-04	42	27	34.5	0	0	0.00	M	M
2020-02-05	52	42	47.0	7	0	0.09	M	M
2020-02-06	57	49	53.0	13	3	0.01	M	M
2020-02-07	54	44	49.0	9	0	0.11	M	M
2020-02-08	51	38	44.5	5	0	0.03	M	M
2020-02-09	47	34	40.5	1	0	0.00	M	M
2020-02-10	51	34	42.5	3	0	0.00	M	M
2020-02-11	44	31	37.5	0	0	0.00	M	M
2020-02-12	53	33	43.0	3	0	0.00	M	M
2020-02-13	44	33	38.5	0	0	0.04	M	M
2020-02-14	50	39	44.5	5	0	0.01	M	M
2020-02-15	47	42	44.5	5	0	0.67	M	M
2020-02-16	51	37	44.0	4	0	T	M	M
2020-02-17	51	33	42.0	2	0	0.00	M	M
2020-02-18	54	33	43.5	4	0	0.00	M	M
2020-02-19	61	32	46.5	7	0	0.00	M	M
2020-02-20	56	28	42.0	2	0	0.00	M	M
2020-02-21	57	29	43.0	3	0	0.00	M	M
2020-02-22	58	31	44.5	5	0	0.00	M	M
2020-02-23	51	42	46.5	7	0	0.11	M	M
2020-02-24	49	34	41.5	2	0	0.00	M	M
2020-02-25	55	32	43.5	4	0	0.00	M	M
2020-02-26	58	40	49.0	9	0	0.00	M	M
2020-02-27	64	33	48.5	9	0	0.00	M	M
2020-02-28	56	32	44.0	4	0	0.12	M	M
2020-02-29	47	31	39.0	0	0	0.21	M	M
Average|Sum	52.1	35.0	43.6	123	3	1.64	M	M

Climatological Data for AURORA STATE AP, OR - March 2020

Date	Max Temperature	Min Temperature	Avg Temperature	GDD Base 40	GDD Base 50	Precipitation	Snowfall	Snow Depth
2020-03-01	49	32	40.5	1	0	T	M	M
2020-03-02	50	39	44.5	5	0	T	M	M
2020-03-03	60	47	53.5	14	4	T	M	M
2020-03-04	57	40	48.5	9	0	0.00	M	M
2020-03-05	59	34	46.5	7	0	0.00	M	M
2020-03-06	48	41	44.5	5	0	0.38	M	M
2020-03-07	49	36	42.5	3	0	0.02	M	M
2020-03-08	52	32	42.0	2	0	0.00	M	M
2020-03-09	57	29	43.0	3	0	0.00	M	M
2020-03-10	61	29	45.0	5	0	0.00	M	M
2020-03-11	57	36	46.5	7	0	0.00	M	M
2020-03-12	56	31	43.5	4	0	0.00	M	M
2020-03-13	41	34	37.5	0	0	0.10	M	M
2020-03-14	44	33	38.5	0	0	0.44	M	M
2020-03-15	48	33	40.5	1	0	0.00	M	M
2020-03-16	61	34	47.5	8	0	0.00	M	M
2020-03-17	59	36	47.5	8	0	0.00	M	M
2020-03-18	59	36	47.5	8	0	0.00	M	M
2020-03-19	63	34	48.5	9	0	0.00	M	M
2020-03-20	68	37	52.5	13	3	0.00	M	M
2020-03-21	60	37	48.5	9	0	0.00	M	M
2020-03-22	63	32	47.5	8	0	0.00	M	M
2020-03-23	51	43	47.0	7	0	0.22	M	M
2020-03-24	50	38	44.0	4	0	0.35	M	M
2020-03-25	53	37	45.0	5	0	T	M	M
2020-03-26	51	35	43.0	3	0	0.00	M	M
2020-03-27	51	38	44.5	5	0	0.15	M	M
2020-03-28	53	46	49.5	10	0	0.06	M	M
2020-03-29	59	48	53.5	14	4	0.18	M	M
2020-03-30	51	43	47.0	7	0	0.47	M	M
2020-03-31	52	40	46.0	6	0	0.16	M	M
Average|Sum	54.6	36.8	45.7	190	11	2.53	M	M

Climatological Data for AURORA STATE AP, OR - April 2020

Date	Max Temperature	Min Temperature	Avg Temperature	GDD Base 40	GDD Base 50	Precipitation	Snowfall	Snow Depth
2020-04-01	48	39	43.5	4	0	0.37	M	M
2020-04-02	52	36	44.0	4	0	0.09	M	M
2020-04-03	50	34	42.0	2	0	T	M	M
2020-04-04	52	37	44.5	5	0	0.07	M	M
2020-04-05	64	42	53.0	13	3	0.01	M	M
2020-04-06	62	41	51.5	12	2	0.00	M	M
2020-04-07	62	40	51.0	11	1	0.00	M	M
2020-04-08	74	36	55.0	15	5	0.00	M	M
2020-04-09	79	48	63.5	24	14	0.00	M	M
2020-04-10	71	41	56.0	16	6	0.00	M	M
2020-04-11	64	43	53.5	14	4	0.00	M	M
2020-04-12	66	36	51.0	11	1	0.00	M	M
2020-04-13	68	36	52.0	12	2	0.00	M	M
2020-04-14	69	37	53.0	13	3	0.00	M	M
2020-04-15	64	43	53.5	14	4	0.00	M	M
2020-04-16	71	43	57.0	17	7	0.00	M	M
2020-04-17	76	43	59.5	20	10	0.00	M	M
2020-04-18	53	45	49.0	9	0	0.10	M	M
2020-04-19	63	42	52.5	13	3	0.00	M	M
2020-04-20	72	40	56.0	16	6	0.00	M	M
2020-04-21	62	45	53.5	14	4	0.00	M	M
2020-04-22	60	47	53.5	14	4	0.31	M	M
2020-04-23	62	49	55.5	16	6	0.00	M	M
2020-04-24	63	50	56.5	17	7	0.03	M	M
2020-04-25	67	44	55.5	16	6	0.22	M	M
2020-04-26	68	39	53.5	14	4	0.09	M	M
2020-04-27	68	49	58.5	19	9	0.03	M	M
2020-04-28	72	50	61.0	21	11	0.00	M	M
2020-04-29	71	52	61.5	22	12	T	M	M
2020-04-30	65	46	55.5	16	6	0.00	M	M
Average|Sum	64.6	42.4	53.5	414	140	1.32	M	M

Climatological Data for AURORA STATE AP, OR - May 2020

Date	Max Temperature	Min Temperature	Avg Temperature	GDD Base 40	GDD Base 50	Precipitation	Snowfall	Snow Depth
2020-05-01	62	41	51.5	12	2	0.07	M	M
2020-05-02	57	43	50.0	10	0	0.49	M	M
2020-05-03	60	43	51.5	12	2	0.06	M	M
2020-05-04	70	39	54.5	15	5	0.01	M	M
2020-05-05	73	49	61.0	21	11	0.01	M	M
2020-05-06	64	46	55.0	15	5	0.13	M	M
2020-05-07	75	41	58.0	18	8	0.00	M	M
2020-05-08	85	57	71.0	31	21	0.00	M	M
2020-05-09	87	62	74.5	35	25	0.00	M	M
2020-05-10	88	54	71.0	31	21	0.00	M	M
2020-05-11	70	50	60.0	20	10	0.13	M	M
2020-05-12	63	49	56.0	16	6	0.25	M	M
2020-05-13	63	49	56.0	16	6	0.04	M	M
2020-05-14	57	50	53.5	14	4	0.68	M	M
2020-05-15	69	51	60.0	20	10	0.01	M	M
2020-05-16	69	54	61.5	22	12	0.16	M	M
2020-05-17	69	50	59.5	20	10	T	M	M
2020-05-18	60	52	56.0	16	6	0.39	M	M
2020-05-19	63	49	56.0	16	6	T	M	M
2020-05-20	61	51	56.0	16	6	0.02	M	M
2020-05-21	60	48	54.0	14	4	0.02	M	M
2020-05-22	62	45	53.5	14	4	0.03	M	M
2020-05-23	64	47	55.5	16	6	0.00	M	M
2020-05-24	74	48	61.0	21	11	0.00	M	M
2020-05-25	69	53	61.0	21	11	0.02	M	M
2020-05-26	75	56	65.5	26	16	0.00	M	M
2020-05-27	85	49	67.0	27	17	0.00	M	M
2020-05-28	92	55	73.5	34	24	0.00	M	M
2020-05-29	86	55	70.5	31	21	0.00	M	M
2020-05-30	62	53	57.5	18	8	0.30	M	M
2020-05-31	66	51	58.5	19	9	0.00	M	M
Average|Sum	69.7	49.7	59.7	617	307	2.82	M	M

Climatological Data for AURORA STATE AP, OR - June 2020

Date	Max Temperature	Min Temperature	Avg Temperature	GDD Base 40	GDD Base 50	Precipitation	Snowfall	Snow Depth
2020-06-01	73	44	58.5	19	9	0.00	M	M
2020-06-02	79	49	64.0	24	14	0.00	M	M
2020-06-03	76	51	63.5	24	14	0.00	M	M
2020-06-04	74	50	62.0	22	12	0.00	M	M
2020-06-05	70	52	61.0	21	11	0.00	M	M
2020-06-06	61	49	55.0	15	5	0.26	M	M
2020-06-07	62	48	55.0	15	5	0.21	M	M
2020-06-08	65	52	58.5	19	9	0.20	M	M
2020-06-09	66	51	58.5	19	9	0.42	M	M
2020-06-10	78	59	68.5	29	19	T	M	M
2020-06-11	76	58	67.0	27	17	0.04	M	M
2020-06-12	59	54	56.5	17	7	0.13	M	M
2020-06-13	60	51	55.5	16	6	0.58	M	M
2020-06-14	68	49	58.5	19	9	0.04	M	M
2020-06-15	64	52	58.0	18	8	0.84	M	M
2020-06-16	64	52	58.0	18	8	0.24	M	M
2020-06-17	M	M	M	M	M	M	M	M
2020-06-18	M	M	M	M	M	M	M	M
2020-06-19	M	M	M	M	M	M	M	M
2020-06-20	M	M	M	M	M	M	M	M
2020-06-21	M	M	M	M	M	M	M	M
2020-06-22	M	M	M	M	M	M	M	M
2020-06-23	M	M	M	M	M	M	M	M
2020-06-24	M	M	M	M	M	M	M	M
2020-06-25	M	M	M	M	M	M	M	M
2020-06-26	M	M	M	M	M	M	M	M
2020-06-27	M	M	M	M	M	M	M	M
2020-06-28	M	M	M	M	M	M	M	M
2020-06-29	M	M	M	M	M	M	M	M
2020-06-30	M	M	M	M	M	M	M	M
Average|Sum	68.4	51.3	59.9	322	162	2.96	M	M

AgACIS

Month	Total Precipitation Normal (inches)
January	5.87
February	4.75
March	4.23
April	3.13
May	2.36
June	2.02
July	0.68
August	0.66
September	1.73
October	3.23
November	6.63
December	6.58
Annual	41.87

Station Information

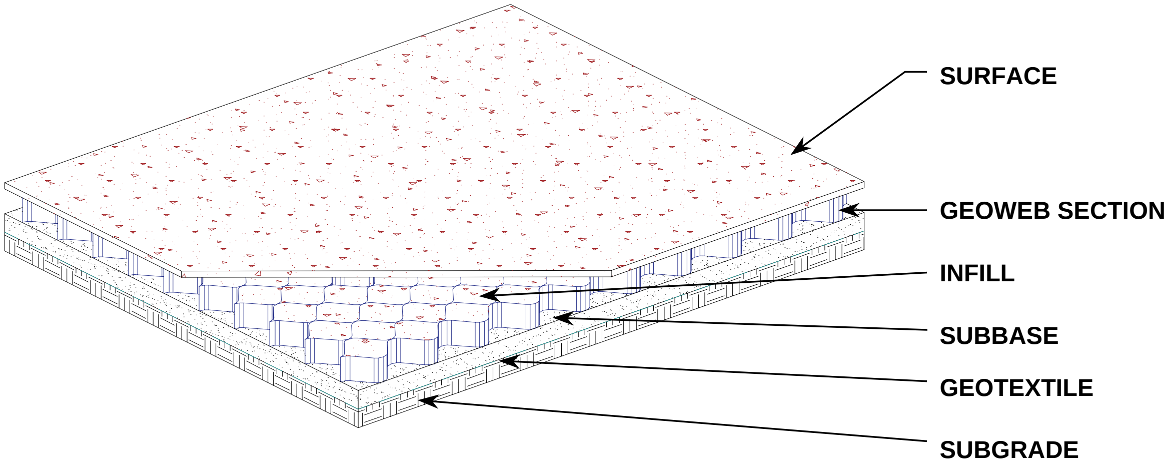
Station name:	AURORA STATE AP
State:	OR
County:	(FIPS 41047)
Station ids:	94281 (WBAN)UAO (FAA)3S2 (FAA)KUAO (ICAO)USW00094281 (GHCN)
Latitude:	45.2486 degrees
Longitude:	-122.7686 degrees
Elevation:	196 feet
Available date ranges:	Max Temperature 1997-06-01 - 2020-05-12 Min Temperature 1997-06-01 - 2020-05-12 Precipitation 1998-04-01 - 2020-05-12 Snowfall 2009-08-01 - 2018-12-12 Snow Depth 1998-07-16 - 2018-10-10

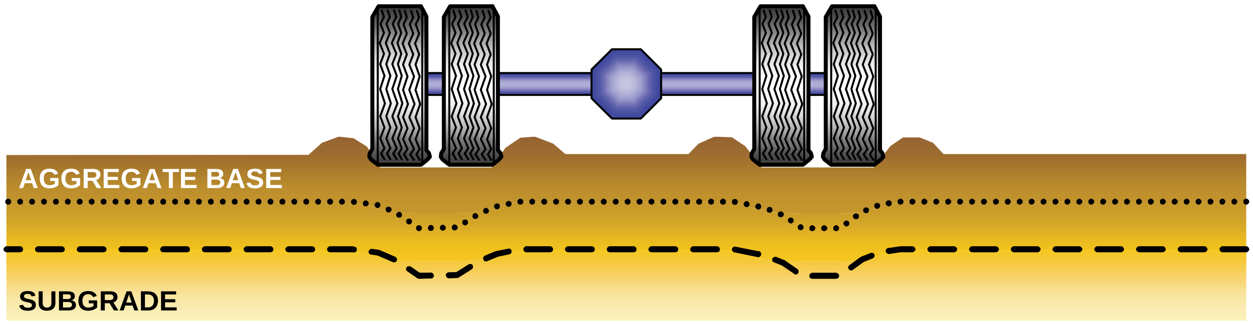
Exhibit 1, Attachment 3

PRESTO

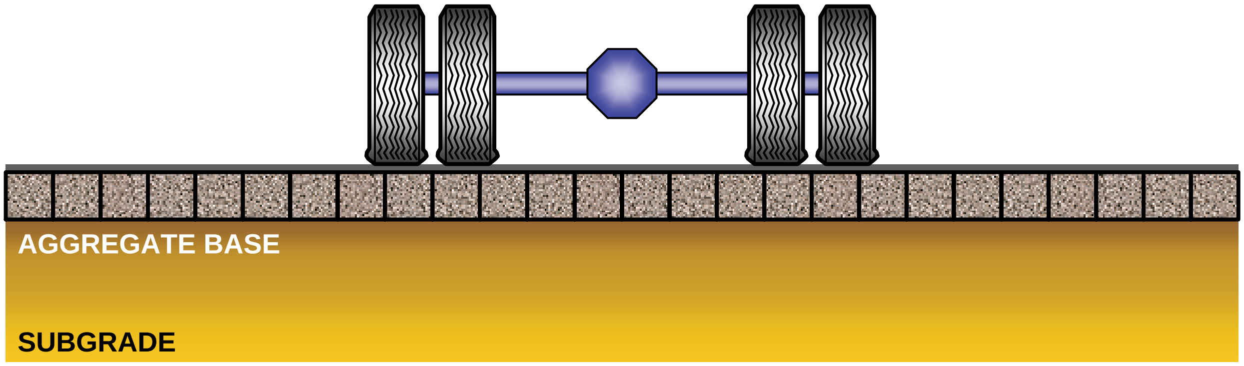
GEOSYSTEMS

GEOWEB® LOAD SUPPORT SYSTEM
TECHNICAL OVERVIEW





UNCONFINED GRANULAR PAVEMENT SYSTEM



THE GEOWEB GRANULAR PAVEMENT SYSTEM

Presto Geosystems
670 N Perkins Street, Appleton, Wisconsin, USA 54914
Ph: 920-738-1707 or 800-548-3424 ■ Fax: 920-738-1222
e-mail: info@prestogeo.com www.prestogeo.com/
GWLSTO 15 Feb-08

PRESTO

GEOSYSTEMS

GEOWEB® LOAD SUPPORT SYSTEM TECHNICAL OVERVIEW

Table of Contents

PRESTO
GEOSYSTEMS

GEOWEB® LOAD SUPPORT SYSTEM TECHNICAL OVERVIEW

Introduction

Natural aggregate / soil construction materials for road base and other load support applications are inherently unstable compared to other construction materials such as steel and reinforced concrete. This is because they are comprised of discrete particles of varying sizes that can roll, or slide, over one another. They have relatively low shear resistance and will eventually fail as a result of single or multiple load applications. However, this *weak link* property also makes these natural construction materials easily workable relative to stockpiling, transporting and placing over large areas or long roadways.

Asphalt cement and Portland cement are commonly used to improve the stability of aggregate materials to make them suitable for the wearing course of load support structures. In addition, most load support structures also require a good base and/or subbase layer to distribute surface loads over the subgrade. Unbound aggregate materials are ideal for this purpose because they are easy to place, are flexible and improve the ride quality of the structure. However, because of their inherent weakness, road builders have long sought new ways to increase the long-term stability of unbound aggregate materials. Many products have been developed and tested to bind together or reinforce aggregate materials but often with limited success.

Fine and uniformly graded sands best exemplify the inherent weakness of granular materials. Desert sands and dry beach sands cannot support channelized traffic loading without significant rutting occurring due to localized shear failure of the near surface material. For this reason, the U.S. Army Corps of Engineers, Waterways Experiment Station, began a research project in the mid 1970's to investigate methods for rapid construction of sand roads for beach landings and desert operations. In order to achieve surface stability without the requirement for chemical additives, mixing and curing time, three-dimensional cellular confinement of loose sands was determined to be the most practical alternative. Through field trials and experimentation, the optimum cell depth to diameter ratio was determined to be approximately 1.0 for heavy military and civilian wheel loads. In the late 1970's Presto Products Company developed the Geoweb cellular confinement system, based on the Corps of Engineers research, as a commercial product to stabilize unbound aggregate materials. The Geoweb system consists of an assembly of polyethylene sheet strips connected in a series of off-set, full-depth ultrasonic welded seams, aligned perpendicular to the longitudinal axis of the strips. When expanded, the interconnected strips form the walls of the cellular confinement structure into which granular fill materials can be placed. Various cell depths have been developed to satisfy load and subgrade strength design criteria based on optimum cell to diameter ratios. Recent improvements to the Geoweb system include surface texturization and cell wall perforations for improved frictional resistance and lateral drainage.

Examples of Geoweb Load-Support System Applications

Granular Access Roads
Grass Access Roads
Porous Pavements
Pavement Subbases
Parking Lots
Storage Yards
Intermodal and Port Facilities
Boat Ramps / Low Level Crossings
Retaining Wall Spread Footings
Foundation Mattresses
Trench Invert Stabilization
Stabilized Drainage Layer

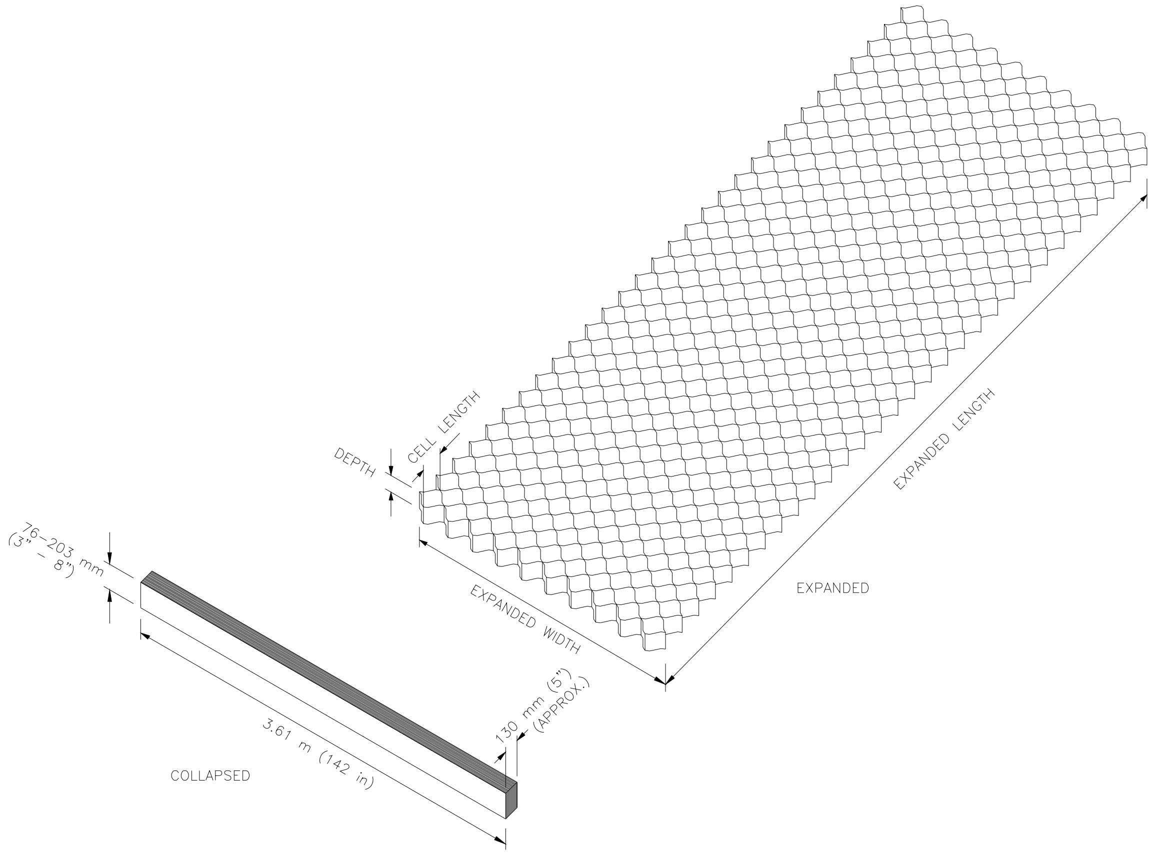


Figure 1 Typical Geoweb Sections

Features and Benefits of the Geoweb Cellular Confinement System

The Geoweb cellular confinement system improves the load-deformation performance of granular infill materials due to the hoop strength of individual cells, the passive resistance of infill material in adjacent cells and vertical stress transfer to adjoining cells. When compared to 2-dimensional sheet reinforcement materials, the stiffness of the 3-dimensional Geoweb system is significantly greater and does not require initial deformation to support the design load.

The Geoweb cellular confinement system dramatically increases the shear resistance of granular infill materials allowing the use of lower quality aggregates (e.g. sand, gravel) to carry concentrated loads that would otherwise require crushed stone or bituminous mixes to prevent localized, near-surface, shear failure. The cellular structure also distributes concentrated loads to surrounding cells thus reducing the stress on the subgrade directly beneath the load and the required total thickness of the structure.

The Geoweb load support system can offer several advantages over conventional solutions and alternative systems. When very soft soils and/or heavy loads are a factor, the system can reduce costs by reducing the required section thickness. Where aggregate materials are expensive or unavailable, the system can reduce costs by incorporating locally available materials. Since Geoweb sections are very compact for shipping and reduce total thickness requirements, a small quantity can be used to replace truckloads of imported aggregate that may have to be hauled over long distances. Finally, when extended pavement life and/or low maintenance requirements are desired, the Geoweb system can ensure that the integrity of granular infill materials will be maintained indefinitely.

Identifying Load Support Problems and Geoweb Solutions

Load support design problems most commonly arise when:

- soft subgrade soils are encountered,
- surface soils are unstable, (i.e. good quality aggregates are locally unavailable or uneconomical) or,
- there are aesthetic and/or environmental consideration.

To identify load support problems where Geoweb cellular confinement should be considered, the following questions should be asked.

Soft Subgrades Problems

Are there any constraints on undercutting or designing a thick structure? If yes, consider the Geoweb cellular confinement system to reduce the section thickness.

Is it impossible to build a stable foundation mattress below the load structure because of a very soft, unstable subgrade condition? If yes, consider the Geoweb cellular confinement system, with a geotextile underlayer, to bridge over the soft soil and support construction equipment while using a minimum thickness of cover material.

Conventional, non-Geoweb solutions to soft subgrades problems, may include:

- excavation of the soft soil and replacement with imported fill (usually granular),
- chemical stabilization of the subgrade soil, or
- design of a thick, multi-layered structure which may include high quality aggregate materials, asphaltic concrete and/or Portland cement concrete.

Thick pavement structures and/or deep excavation may not always be possible due to existing curbs and buried utilities in existing roads.

Surface Stability Problems

Do the locally available soils (e.g. sands and gravels) have adequate shear strength to be used as a wearing surface for a temporary or low-volume access road? If not, confinement of the local materials in the Geoweb system should be weighed against the cost of importing higher quality aggregate materials.

Will aggregate degradation and lateral spreading of the pavement base course result in rutting and premature failure of the pavement structure? If the subgrade is relatively competent, deformation and rutting of the base course is likely to be the cause of maintenance problems and reduce the potential life of the pavement structure. Using the Geoweb system to confine the base course will totally restrict lateral movement that causes rutting and will minimize abrasion and wear on the aggregate infill material.

Few, if any, conventional solutions exist for this problem.

Aesthetic / Environmental Problems

Would a grass surfaced, low volume access road for maintenance vehicles be more aesthetically pleasing than a gravel or asphalt concrete surfaced pavement? If yes, the Geoweb cellular confinement system infilled with an aggregate/topsoil mix and vegetated is an attractive solution.

Is a porous pavement required for groundwater regeneration? If yes, the Geoweb cellular confinement system infilled with porous stone should definitely be considered. However, without confinement, porous aggregates are inherently unstable as surface materials.

Geoweb Load Support Systems - The Key Components

Textured Geoweb system

Engineered surface-textured polyethylene strips used in manufacturing Geoweb sections improve the frictional interaction between the Geoweb cell walls and granular infill materials. The increase in cell-wall / infill-interface friction provides structural benefits in certain Geoweb applications.

In load support applications, the higher cell wall/infill interface friction increases the resistance to vertical deformation of the infill soil relative to the cellular structure. Therefore, a more efficient transfer of vertical stress is provided to the surrounding cells. The result is a further reduction in vertical stress on the subgrade compared to a smooth walled geocell. For certain combinations of wheel loads and infill material properties, the surface texture makes it possible to further reduce the total required thickness of granular pavement over smooth-walled geocells.

Results of small and large scale shear box tests on sand and stone materials with textured Geoweb materials have demonstrated that Peak Coefficient Ratios (i.e. peak interface friction coefficient of textured Geoweb sections divided by the peak interface friction coefficient of granular infill soil in-isolation) varied from 0.63 (crushed stone materials) to 0.81 (coarse sand materials) compared to 0.64 (crushed stone materials) and 0.61 (coarse sand materials) with smooth Geoweb materials. Note that texturization does not increase the interface friction with some crushed stone infills. The Peak Coefficient Ratio should not be confused with the Peak Friction Angle Ratio defined in the section titled Geoweb Cell Wall/Infill Friction Angle Ratio

Perforated Geoweb system

Similar tests using sand and stone materials with the perforated Geoweb material demonstrated that the interface frictional characteristics are similar, or in some cases better, than those with surface textured Geoweb cells. Specifically, the Peak Coefficient Ratios of perforated Geoweb materials with crushed stone and coarse sand infills were found to be 0.75 and 0.89 respectively.

The latter test results indicate that perforated cell walls can be as effective as textured cell walls in increasing the interface friction. Therefore, the structural capacity of the perforated Geoweb load support system with certain sand/gravel infills is more effective than the textured Geoweb system. Since perforations also offer the advantage of lateral drainage, which is particularly useful over impermeable subgrades, the perforated Geoweb system is the recommended choice for many pavement applications. Refer to Table 1 for an illustration of the significance of the performance advantage using textured and perforated cell wall type.

Infill materials

Infill materials for Geoweb load support applications should always be predominately granular with a maximum particle size of 50 mm (2 in). For best performance, the fines fraction (i.e. material passing the #200 sieve - 75 μm) should not be greater than 10%. Soils with greater than 10% fines have low permeability and lose strength dramatically when they become wet. Pure granular materials are not affected by moisture fluctuations but are not as stable as granular materials with 5% - 10% fines. A small fraction of fines will increase stability by reducing the voids ratio and binding the soil.

The Geoweb cellular confinement system is effective in increasing the stability of lower quality granular infill materials such as poorly graded sands and gravels. With cellular confinement, poor quality granular infills can be used as the surface or near-surface material of access roads where driving speeds are relatively slow and ride quality is not a major concern. Higher quality aggregates are recommended for granular surfaced pavements where traffic speeds are higher and a smoother riding surface is required. Good quality aggregates typically include well graded crushed stones or gravels with a maximum particle size of 40 mm (1.5 in) and less than 8%, by weight, passing the #200 sieve. For long-term durability, the coarse fraction of the aggregate should have a Los Angeles Abrasion test wear less than 50%. The fines fraction (i.e. passing the #200 sieve) should not be greater than two-thirds of the fraction passing the #40 sieve and the fraction passing the #40 sieve should have a liquid limit no greater than 25%. The plasticity index should be less than 6%.

Table 1 Total Thickness of Coarse Sand / Gravel Base Including Geoweb Section

Subgrade CBR	**Wheel Load**		**Smooth Cell**	**Textured Cell**	**Perforated Cell**	**Unconfined Gravel**
%	**kN**	**(lbf)**	**Relative Total Thickness of Road Base**			
0.2	**27**	**(6,000)**	32%	28%	27%	100%
	53	**(12,000)**	59%	25%	25%	100%
	111	**(25,000)**	72%	23%	23%	100%
	222	**(50,000)**	80%	22%	22%	100%
0.5	**27**	**(6,000)**	46%	40%	40%	100%
	53	**(12,000)**	43%	38%	37%	100%
	111	**(25,000)**	40%	35%	34%	100%
	222	**(50,000)**	38%	33%	32%	100%
1.0	**27**	**(6,000)**	58%	54%	54%	100%
	53	**(12,000)**	55%	49%	48%	100%
	111	**(25,000)**	52%	45%	44%	100%
	222	**(50,000)**	49%	43%	42%	100%
2.0	**27**	**(6,000)**	81%	81%	81%	100%
	53	**(12,000)**	65%	58%	58%	100%
	111	**(25,000)**	59%	52%	51%	100%
	222	**(50,000)**	60%	52%	51%	100%

NOTE: **This table is based on theoretical methodologies outlined herein. Values are for comparative purposes only and are not a substitute for project specific design.**

Geotextile underlayer

When the Geoweb section is to be placed directly above a fine-grained or cohesive soil subgrade, a nonwoven geotextile is typically recommended for separation of the native soil and the granular infill. Separation is important to prevent contamination and loss of shear strength of the granular infill and to prevent punching or migration of the infill material into the subgrade. With a geotextile underlayer, the infill material is totally confined on all sides and at the bottom of individual cells.

When specific designs require a granular subbase below the Geoweb section, a woven or nonwoven geotextile may be recommended for separation as well as temporary load support during placement of the subbase layer. If the subbase is a well-compacted granular material, a geotextile separator is not typically required between the subbase and Geoweb infill.

Surface materials

In order to prevent trafficking directly on top of the Geoweb cell walls, it is generally recommended to place a minimum 50 mm (2 in) of granular cover (i.e. overtopping) above the Geoweb cell walls. The surface material should be dense-graded crushed stone that is resistant to surface rutting. If traffic volumes are high, a bituminous surface treatment can increase the stability of the riding surface.

If an asphalt concrete base or surface layer is to be placed over the infilled Geoweb base, the depth of granular cover above the cell walls should be at least 25 mm (1 in) to allow for minor consolidation of the infill material and to insulate the polyethylene from direct contact with the hot mix asphalt concrete.

Design Considerations and Methods

There is no single design method that encompasses the full range of Geoweb load support applications. A theoretical design method, based on empirically derived design methods for unpaved roads over soft soils, has been developed for the Geoweb granular pavement system. Design methods for flexible pavements, spread footings, and granular pavements with unstable infill soils have yet to be developed. However, it was this latter function for which Geoweb was originally invented and developed and has proven effective, particularly with sand infill materials.

Recent results of large scale triaxial compression testing of the Geoweb cell infilled with granular materials demonstrate that the Geoweb system imparts an apparent cohesion of approximately 150 kPa (3000 psf) to the confined material. This effective cohesion is in addition to the natural frictional shear strength of the granular material. Presto Geosystems is currently using this information to develop bearing-capacity design procedures for Geoweb load support structures that takes into account the additional shear strength provided by the apparent cohesion. These design procedures will apply to large spread footing and granular pavement applications with poor-quality infill materials.

A discussion of currently available design procedures follows for Geoweb granular pavement systems and the design approaches used for other Geoweb load support applications.

Flexible Pavements

Conventional flexible pavement design methods (e.g. AASHTO, Asphalt Institute, Caltrans, etc.) are all based on empirical data collected from either full-scale road tests or ongoing testing and monitoring of pavement performance within various geographical areas. Structural values of conventional road construction materials (e.g. crushed stone, gravel, asphalt concrete, etc.) have been determined by federal and local agencies based on years of in-service performance history. While many new materials (e.g. stabilizers, geosynthetics, etc.) have been introduced in recent years to enhance the structural value of conventional construction materials, it is difficult and can take several years to obtain structural values for these components to use with existing design methods. For this reason, there are no agency-accepted structural values or equivalencies that can be used with current pavement design methods for the Geoweb system.

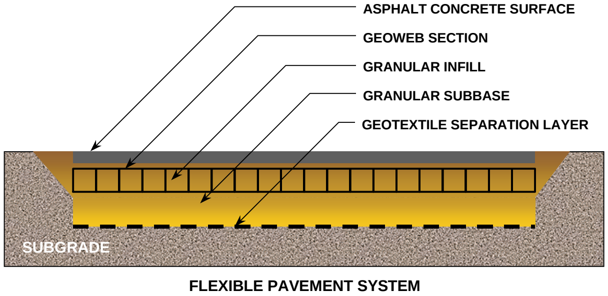


Figure 2 Flexible Pavement Detail

By combining conventional pavement design methods with a theoretical method for determining the structural equivalency of a confined pavement layer, it is possible to design pavement structures that incorporate the Geoweb system.

Granular Pavements

Design of Geoweb confined granular pavements (e.g. access roads) over soft soils is relatively straight forward and has been well documented for general design purposes. Refer to the Design Parameters – Granular Pavements and Design Calculations Granular Pavements sections of this document for specific details about the required design input data and the design calculations.

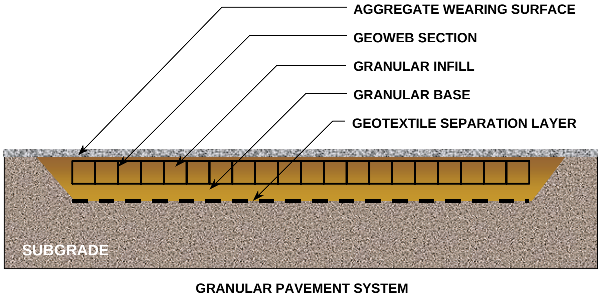


Figure 3 Granular Pavement Detail

Spread Footings

Geoweb spread footings may be considered for a wide range of load support applications such as building footings, buried pipes and segmental retaining walls. They may also be considered for a variety of soil problems such as low bearing capacity, settlement and inadequate shear resistance of near surface foundation soils. Footing loads may be relatively large or small with respect to individual cell or section size of Geoweb spread footings. Due to the versatility of the Geoweb cellular confinement system, the function and design method may change with varying combinations of application, problem and footing loads. In some cases the governing design factor may be:

- the overall shear resistance of the Geoweb spread footing,
- the redistribution of stresses within individual Geoweb cells or
- the increase in bearing area provided by a Geoweb spread footing.

The design approach used for granular pavement structures can also be used for design of Geoweb spread footings with relatively small rigid footing loads by modifying the design criteria for bearing capacity from local shear failure mode to general shear failure mode. For conventional bearing capacity and settlement calculations of larger footing loads, the recommended effective bearing area of a Geoweb mattress should extend no more than 500 mm (18 in) beyond the edges of the rigid footing. In most cases, this will provide a significant decrease in the calculated bearing pressure without compromising the basic assumption that the Geoweb mattress will be effectively rigid.

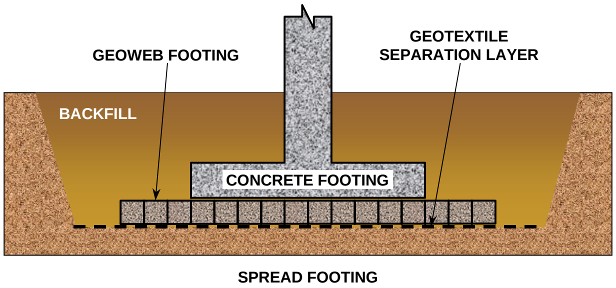


Figure 4 Spread Footing Detail

As stated above, development of a design method for Geoweb spread footings, which will take into account the effective cohesion of the cellular structure, is currently underway.

Design Parameters - Granular Pavements

The following information and input parameters are required for design of the Geoweb load support system for granular pavements.

Wheel Load

The design wheel load is the heaviest single or dual wheel load that the granular pavement will be required to support over the proposed life of the structure.

Tire Pressure

The tire pressure is the tire inflation pressure of the design wheel load and is approximately equal to the ground contact pressure. An input value is required for determination of the effective contact radius of the design wheel load.

Bearing Capacity Coefficient

Bearing capacity coefficients are mathematically or empirically derived coefficients used within standard equations for determination of the bearing capacity of a soil. For unpaved roads over soft cohesive soils, the US Forest Service and others have developed bearing capacity coefficients for determination of the bearing capacity of soils subjected to dynamic loading wherein punching (local) shear failure is more prevalent than general shear failure. The US Forest Service developed the following bearing coefficients for unpaved haul roads for two broad ranges of traffic loading.

$N_c = 2.8$	High traffic with little rutting (i.e. > 1000, < 10000)
$N_c = 3.3$	Low traffic with significant rutting (i.e. < 1000)

Depth to Top of Geoweb section

The depth of placement of the Geoweb layer influences the distribution of stresses through the system and has a significant effect on the design. Since vertical stresses are higher near the surface, optimum performance and maximum thickness reduction are obtained by placing the Geoweb as close to the surface as possible. However, in order to protect the top of the Geoweb cell walls, a 25 mm - 50 mm (1 in - 2 in) aggregate wearing surface is typically recommended.

Subgrade Strength

There are several laboratory and field test methods available to determine the strength of subgrade soils for design purposes. The calculations require soil strength to be expressed in terms of shear strength or cohesion. Shear strength can be determined in the field by the vane shear test or in the laboratory by the shear box or triaxial compression tests. Soil strength is also commonly determined by the Standard Penetration Test and the California Bearing Ratio (CBR) test. For cohesive soils, shear strength of a soil can be estimated from the standard penetration resistance (N) or the California Bearing Ratio (CBR). In the absence of field or laboratory test data, the strength of the subgrade soil can be estimated by it's consistency (see the Field Identification section of Table 4). When estimating a soil's strength by it's consistency, the soil sample should be taken from a test pit which is deep enough to ensure it's properties have not been affected by changing surface conditions (e.g. rain water, hot dry weather, etc.).

Brief descriptions of the most common test methods for determining the strength of subgrade soils are given below.

California Bearing Ratio (CBR) Test

The California Bearing Ratio test is an index test used to determine the relative strength of a soil compared to a standard high-quality crushed stone material. The test specimen is prepared by compacting a sample of the soil, in multiple lifts, into a 6 inch diameter cylinder, applying a surcharge in the form of circular plates to approximate the confining stress of the final pavement on the soil and soaking the entire sample for a period of 4 days. The test consists of loading the soil sample with a 3 square inch (1935 square mm) circular piston, through holes in the surcharge plates, at a rate of 0.10 inch (2.54 mm)/minute up to a maximum of 0.5 inches (13 mm). The CBR value is the ratio of the unit load at 0.10 inch (2.54 mm) or 0.20 inch (5.04 mm) to that of the standard crushed stone material at the same depth of penetration (whichever is higher). The unit loads are given in Table 2.

Table 2 Unit Loads for Standard Crushed Stone Material

0.1 inch penetration	1000 psi
0.2 inch penetration	1500 psi
0.3 inch penetration	1900 psi
0.4 inch penetration	2300 psi
0.5 inch penetration	2600 psi

Standard Penetration Test

The standard penetration test provides an indication of the density, and the angle of internal friction of cohesionless soils and the shear strength of cohesive soils. The tests consists of driving a split spoon sampler, equipped with a cutting shoe and attached to the end of a drill rod, into a soil by dropping a 140 lb (63.6 kg) hammer a distance of 30 inches (0.76 m). A split spoon sampler is a thick-walled steel tube, split lengthwise, used to obtain undisturbed samples of soil from drill holes. The number of blows required for each 6 inches (150 mm) of penetration of the split spoon sampler is recorded. The standard penetration resistance is the sum of the blows for the second and third increments of 6 inches (150 mm) and is termed N in blows/ft (blows/300 mm).

Shear Strength Tests

The shear strength of a soil is the stress at which the soil fails in shear. It can be calculated by dividing the shear force at which a soil fails by the cross-sectional area of shear or, if the cohesion and angle of internal friction are known, by the general Coulomb equation.

$$s = c + \sigma \tan \phi$$

where c is the soil's cohesion (i.e. interparticle attraction) expressed in terms of force per unit area

σ is the overburden or surcharge pressure in terms of force per unit area

ϕ is the soil's angle of internal friction (i.e. resistance to interparticle slip) in degrees

Granular soils do not have cohesion and therefore shear strength is governed by overburden pressure that explains why granular pavement surface materials are inherently unstable. Undrained cohesive soils (e.g. soft and saturated clays) do not have internal friction and therefore shear strength is governed by cohesion that can vary with moisture content. Drained cohesive soils can have both cohesion and internal friction.

The shear strength of granular soils can be measured in a laboratory by the shear box test. Cohesion and the angle of internal friction of cohesive soils can be measured in a laboratory for drained and undrained conditions by triaxial compression tests. In the field, shear strength can be measured by the field vane shear test. Refer to a textbook on soil mechanics or geotechnical engineering for more information about the shear strength of soils and test methods.

Angle of Internal Friction - Geoweb Infill Material

The angle of internal friction of a cohesionless granular soil can be determined by measuring the maximum shear stress at failure over a range of normal stresses (i.e. confining pressures) and plotting the results on a graph. The angle formed by the best-fit straight line through the origin and the horizontal axis is a close approximation of the angle of internal friction. See Figure 5. For compacted granular materials, the angle of internal friction is typically within a range of 30° to 40°. The higher the quality of the granular material (e.g. angularity, gradation, hardness, etc.) the higher the angle of internal friction.

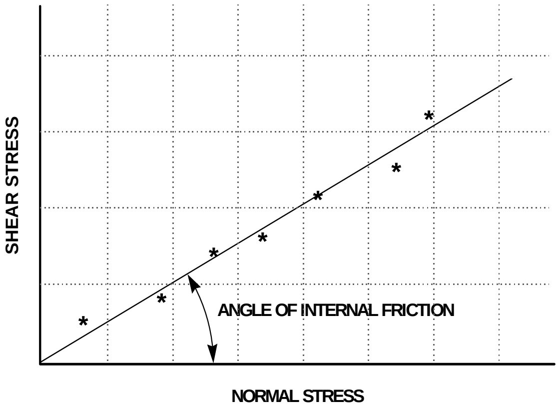


Figure 5 Angle of Internal Friction

Geoweb Cell Wall/Infill Friction Angle Ratio

The Geoweb cell wall/infill material friction angle ratio is the ratio of angle of shearing resistance between the infill material and the Geoweb cell wall over the peak friction angle of the infill soil in-isolation. It will vary depending upon the gradation and particle angularity of the infill material and the roughness of the cell wall or the size and spacing of perforations in the cell wall.

Shear box tests have been carried out to determine angles of shearing resistance between standard Geoweb cell wall treatments and typical granular materials. The results were expressed in terms of peak friction angle ratios (or Geoweb Cell Wall/Infill Friction Angle Ratio), where Peak Friction Angle Ratio is defined as the angle of shearing resistance between the granular infill and the Geoweb cell wall divided by the peak friction angle of the infill material in-isolation. Geoweb Cell Wall/Infill Friction Ratios for standard cell wall treatments and typical compacted granular materials are given in Table 3. The values presented in Table 3 are used to develop the relationships in Table 1 and base thickness in Table 5.

Table 3 Recommended Peak Friction Angle Ratio

Granular Infill Material	**Cell Wall Type**	**$r = \delta/\phi$**
Coarse Sand / Gravel	Smooth	0.71
	Textured	0.88
	Textured - Perforated	0.90
#40 Silica Sand	Smooth	0.78
	Textured	0.90
	Textured - Perforated	0.90
Crushed Stone	Smooth	0.72
	Textured	0.72
	Textured - Perforated	0.83

Design Calculations Granular Pavements

Illustrated here are the design procedures and calculations for determining aggregate thickness requirements for granular-surfaced pavements (e.g. access, utility and haul roads) both with and without the Geoweb cellular confinement system. Empirically derived bearing capacity coefficients are first used to determine the maximum allowable stress on a subgrade with either known or estimated shear strength. The maximum allowable stress is that stress which would cause local punching / shear failure of the subgrade under sustained loading conditions. Since granular pavement loads are transient, the effective strength of the soil is typically higher than it would be under static loading. Therefore, the maximum allowable stress is the limiting stress for design purposes. Boussinesq theory is then used to determine the required depth of granular cover beneath the design wheel load to ensure that the maximum allowable stress is not exceeded. The calculations outlined herein are intended for low volume roads where minor deformations are tolerable or for design of pavement subbase layers over soft soils. They are not intended for design of flexible pavement structures with paved surfaces. The calculations are only valid for granular pavement design over cohesive subgrade soils with CBR values less than 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

Calculations

Determine the subgrade shear strength. Refer to Table 4 if the subgrade strength is reported in terms of Standard Penetration Resistance, CBR or by Field Identification.

Determine the maximum allowable stress on the subgrade, q_a

$$q_a = N_c c_u$$

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

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

Determine the required thickness of granular pavement, z_U, without the Geoweb cellular confinement system using the following equation (Boussinesq equation for estimating vertical stress at a given depth below a circular load re-written to calculate the depth of cover above a given vertical stress, q_a).

$$z_U = \frac{R}{\sqrt{\dfrac{1}{\left(1 - \dfrac{q_a}{p}\right)^{2/3}} - 1}}$$

where R = Radius of loaded area (i.e. effective radius of single or dual tires)

$$R = \sqrt{\frac{P}{p\pi}}$$

Determine the required thickness of granular pavement, z_G, with the Geoweb cellular confinement system.

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)

The total required thickness of granular pavement with the Geoweb cellular confinement system is a function of the Geoweb cell depth, the depth of placement below the applied load, the wheel load and tire pressure and the infill material properties. Surface stress (i.e. wheel load contact pressure) is distributed both vertically and horizontally through the Geoweb cellular structure. Horizontal stresses, in turn, are converted into vertical resisting stresses along the cell walls thus reducing the total vertical stress directly beneath the center of the loaded area. The total resisting stress provided by the Geoweb cell structure is calculated and added to the maximum allowable stress on the subgrade for determination of the total required thickness of granular pavement with the Geoweb cellular confinement system.

The first step is to select the Geoweb section placement depth, z_t within the granular pavement structure. Since vertical stresses are higher near the surface, optimum performance and maximum thickness reduction are obtained by placing the Geoweb as close to the surface as possible. However, to protect the top of the Geoweb cell walls, a 25 mm to 50 mm (1 in to 2 in) aggregate wearing surface is typically recommended.

After selecting a trial depth of placement, calculate the vertical stress, σ_{vt}, at the top of the Geoweb section using the following equation.

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

Next, calculate the vertical stress, σ_{vb}, at the bottom of the Geoweb section. The bottom depth, z_b, is the top depth, z_t, plus the thickness (or depth) of the Geoweb section.

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

Calculate the horizontal stress at the top, σ_{ht}, and bottom, σ_{hb}, of the Geoweb section using the following equations.

$$\sigma_h = K_a \sigma_v$$

where K_a is the coefficient of active earth pressure.

$$K_a = \tan^2\left(45 - \phi/2\right)$$

Horizontal stress at the top of the Geoweb section, σ_{ht}

$$\sigma_{ht} = K_a \, \sigma_{vt}$$

Horizontal stress at the bottom of the Geoweb section, σ_{hb}.

$$\sigma_{hb} = K_a \, \sigma_{vb}$$

The average horizontal stress on the Geoweb cell walls is then determined as follows.

$$\sigma_{avge} = \frac{\left(\sigma_{ht} + \sigma_{hb}\right)}{2}$$

Next, calculate the reduction in stress, σ_r, directly beneath the center of the loaded area due to stress transfer to the Geoweb cell walls using the following equation.

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

where H = Geoweb cell depth in mm (in)

D = Effective Geoweb cell diameter = 190 mm (7.5 in)

δ = Angle of shearing resistance between granular infill material and Geoweb cell walls.

$\delta = r\phi$ (obtain test data or estimate r from Table 3)

Determine the design allowable stress, q_G, on the subgrade with the Geoweb cellular confinement system using the following equation.

$$q_G = q_a + \sigma_r$$

Determine the total required thickness of granular pavement, z_G, with the Geoweb cellular confinement system.

$$z_G = \frac{R}{\sqrt{\frac{1}{\left(1 - \frac{q_G}{p}\right)^{2/3}} - 1}}$$

If the total required thickness is greater than the surface thickness (i.e. depth to the top of the Geoweb section); in addition, the depth of the Geoweb section, a subbase layer is required. The thickness of the subbase layer must be equal to the total required thickness minus the surface thickness and the Geoweb section depth.

Using the equations presented herein, Table 5 gives base/subbase thickness requirements vs. cell wall type for the Geoweb load support system, under the following load condition:

- 203 mm (8 in) depth of Geoweb section,
- crushed stone infill,
- 38 degree friction angle,
- 690 kPa (100 psi) tire pressure,
- 25 mm (1 in) depth of cover over the Geoweb section,
- 2.8 bearing capacity coefficient.

Table 5 Total Thickness of Coarse Sand / Gravel Base Including Geoweb Section

Subgrade CBR	Wheel Load		Smooth r = 0.71		Textured r = 0.88		Textured - Perforated r = 0.90		Unconfined Stone	
%	**kN**	**(lbf)**	**mm**	**(in)**	**mm**	**(in)**	**mm**	**(in)**	**mm**	**(in)**
0.2	**27**	**(6,000)**	277	(10.9)	241	(9.5)	236	(9.3)	876	(34.5)
	53	**(12,000)**	366	(14.4)	315	(12.4)	310	(12.2)	1240	(48.8)
	111	**(25,000)**	490	(19.3)	419	(16.5)	411	(16.2)	1788	(70.4)
	222	**(50,000)**	655	(25.8)	556	(21.9)	546	(21.5)	2527	(99.5)
0.5	**27**	**(6,000)**	251	(9.9)	221	(8.7)	218	(8.6)	546	(21.5)
	53	**(12,000)**	335	(13.2)	292	(11.5)	287	(11.3)	772	(30.4)
	111	**(25,000)**	450	(17.7)	389	(15.3)	384	(15.1)	1113	(43.8)
	222	**(50,000)**	605	(23.8)	518	(20.4)	511	(20.1)	1575	(62.0)
1.0	**27**	**(6,000)**	218	(8.6)	203	(8.0)	203	(8.0)	376	(14.8)
	53	**(12,000)**	292	(11.5)	257	(10.1)	254	(10.0)	531	(20.9)
	111	**(25,000)**	396	(15.6)	345	(13.6)	340	(13.4)	767	(30.2)
	222	**(50,000)**	536	(21.1)	465	(18.3)	457	(18.0)	1085	(42.7)
2.0	**27**	**(6,000)**	203	(8.0)	203	(8.0)	203	(8.0)	251	(9.9)
	53	**(12,000)**	231	(9.1)	206	(8.1)	203	(8.0)	353	(13.9)
	111	**(25,000)**	315	(12.4)	279	(11.0)	274	(10.8)	536	(21.1)
	222	**(50,000)**	429	(16.9)	376	(14.8)	368	(14.5)	721	(28.4)

NOTE: The above wheel load values are from either single or dual wheels. For axle loads multiply by 2. **This table is based on theoretical methodologies outlined herein. Values are for comparative purposes only and are not a substitute for project specific design.**

Available Tools & Services

Presto and Presto's authorized distributors and representatives offer assistance to anyone interested in evaluating, designing, building or purchasing a **Geoweb Load Support System**. You may access these services by calling 800-548-3424 or 920-738-1707. In addition to working directly with you, the following design and construction resources are available for your use with the **Geoweb Load Support System**.

Design	Material and CSI-format Specifications, System Components Guideline, Request for Project Evaluation, AutoCAD® Drawings, SPECMaker® Specification Development Tool, Technical Resources Library CD, videos
Construction	Installation Guidelines, SPECMaker® Specification Development Tool, Technical Resources Library CD, videos

Disclaimer

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Exhibit 1, Attachment 4

Gulfstream FLIGHT OPS

Operations Briefing

Pavement Weight Bearing Capacity (ACN/PCN)

05202020
Scope: All Aircraft
Briefing Owner: Flight Operations

All Aircraft | Pavement Weight Bearing Capacity

Executive Summary

If you have a question regarding Airport Classification Number (ACN) and Pavement Classification Number (PCN), reference the following sources:

- **Mid Cabin Aircraft**: QRH: Supplemental Data
- **GIV & GV**: QRH : Performance -> Landing Planning
- **G450/G550/G650**: Performance Handbook -> Landing Planning
- **GVII-G500/G600** : Operating Manual >Supplemental Data

Once you have established your aircraft classification number, Gulfstream recommends you contact your flight plan provider as well as the appropriate airport authority/manager for an updated accurate advertised Pavement Classification Number as well as the latest assessment of permissible movement areas.

The PCN is calculated using the verbiage "unrestricted operations.” While it has obviously a calculation of pavement strength, it also is derived to extend the life of the runway environment. While PCNs are published for repeated use, a one-time event (one takeoff/one landing) should be acceptable with the appropriate authorizations. *Caution must be given as PCN does not usually apply to taxiways or ramps and only within 50 feet of runway centerline*. When ACN/PCN is close, make sure to inquire from the airport manager about all movement areas, paying particular attention to the taxiways and ramp areas due to the runway PCN not always guaranteeing the taxiways.

All Aircraft | Pavement Weight Bearing Capacity

Executive Summary (continued)

Keep in mind that even when obtaining the airport manager's approval for an exemption for operation, stay alert to the fact that the real concern is the weight bearing capability of the ramp and taxiways, as it is undoubtedly lower than the runway surface itself. Even with an exemption, tight turns and prolonged duration on the ramp would not help the situation.

If the airport you are operating into has a small number PCN, it may be prudent to acquire a copy of the engineering runway analysis, as well as an explanation as to why the PCN is valued so low. While the average PCN may be acceptable in many cases, some airport movement areas may contain weaker pavement, and as such a smaller PCN is published.

Your flight plan provider and the airport authority will also be able to help you establish confirmed prior aircraft type operated into and out of that particular airfield and whether operators are using surrounding airports for tech stop purposes to add additional fuel for the departure enabling lighter weights at the lower PCN airfield. Heavier weight aircraft historical value and confirming known design value for the runway from the airport manager will assist in making the decision. If there is any doubt, conservatism should always trump and operation should be avoided.

If you still require assistance, please forward your question via the submit your question in the appropriate aircraft section and our team of pilot advocates will be happy to provide further guidance to your situation.

All Aircraft | Pavement Weight Bearing Capacity

Background Briefing

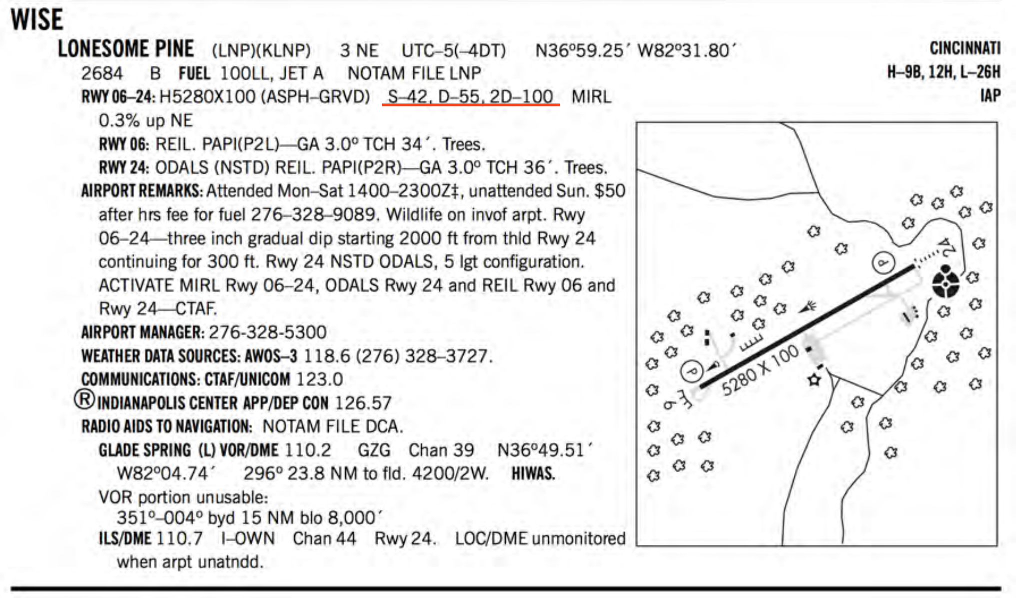
This briefing addresses the two most common forms of pavement weight bearing capacity metrics. A brief, top-level overview of weight bearing capacity is discussed. Where to find such data and how to conduct a pavement analysis follows. Additional factors are discussed at the conclusion.

What are the two most common ways to determine pavement weight bearing capacity?

- **Wheel Weight Bearing Limits (commonly used in the United States).**

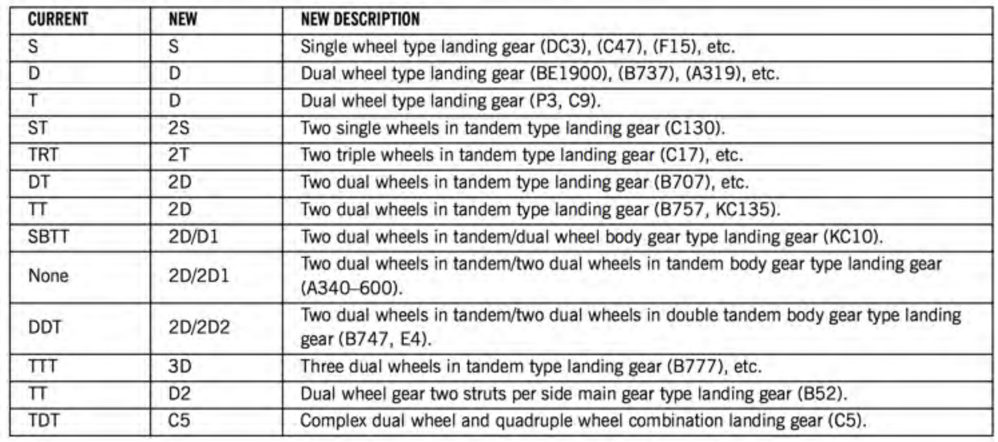
- **ACN/PCN (ICAO Standard)**

All Aircraft | Pavement Weight Bearing Capacity



WISE

LONESOME PINE (LNP)(KLNP) 3 NE UTC–5(–4DT) N36°59.25´ W82°31.80´ **CINCINNATI**
2684 B **FUEL** 100LL, JET A NOTAM FILE LNP **H–9B, 12H, L–26H**
RWY 06–24: H5280X100 (ASPH–GRVD) S–42, D–55, 2D–100 MIRL **IAP**
0.3% up NE
RWY 06: REIL. PAPI(P2L)—GA 3.0° TCH 34´. Trees.
RWY 24: ODALS (NSTD) REIL. PAPI(P2R)—GA 3.0° TCH 36´. Trees.
AIRPORT REMARKS: Attended Mon–Sat 1400–2300Z‡, unattended Sun. $50 after hrs fee for fuel 276–328–9089. Wildlife on invof arpt. Rwy 06–24—three inch gradual dip starting 2000 ft from thld Rwy 24 continuing for 300 ft. Rwy 24 NSTD ODALS, 5 lgt configuration. ACTIVATE MIRL Rwy 06–24, ODALS Rwy 24 and REIL Rwy 06 and Rwy 24—CTAF.
AIRPORT MANAGER: 276-328-5300
WEATHER DATA SOURCES: AWOS–3 118.6 (276) 328–3727.
COMMUNICATIONS: CTAF/UNICOM 123.0
®**INDIANAPOLIS CENTER APP/DEP CON** 126.57
RADIO AIDS TO NAVIGATION: NOTAM FILE DCA.
GLADE SPRING (L) VOR/DME 110.2 GZG Chan 39 N36°49.51´ W82°04.74´ 296° 23.8 NM to fld. 4200/2W. **HIWAS.**
VOR portion unusable:
351°–004° byd 15 NM blo 8,000´
ILS/DME 110.7 I–OWN Chan 44 Rwy 24. LOC/DME unmonitored when arpt unatndd.



CURRENT	NEW	NEW DESCRIPTION
S	S	Single wheel type landing gear (DC3), (C47), (F15), etc.
D	D	Dual wheel type landing gear (BE1900), (B737), (A319), etc.
T	D	Dual wheel type landing gear (P3, C9).
ST	2S	Two single wheels in tandem type landing gear (C130).
TRT	2T	Two triple wheels in tandem type landing gear (C17), etc.
DT	2D	Two dual wheels in tandem type landing gear (B707), etc.
TT	2D	Two dual wheels in tandem type landing gear (B757, KC135).
SBTT	2D/D1	Two dual wheels in tandem/dual wheel body gear type landing gear (KC10).
None	2D/2D1	Two dual wheels in tandem/two dual wheels in tandem body gear type landing gear (A340–600).
DDT	2D/2D2	Two dual wheels in tandem/two dual wheels in double tandem body gear type landing gear (B747, E4).
TTT	3D	Three dual wheels in tandem type landing gear (B777), etc.
TT	D2	Dual wheel gear two struts per side main gear type landing gear (B52).
TDT	C5	Complex dual wheel and quadruple wheel combination landing gear (C5).

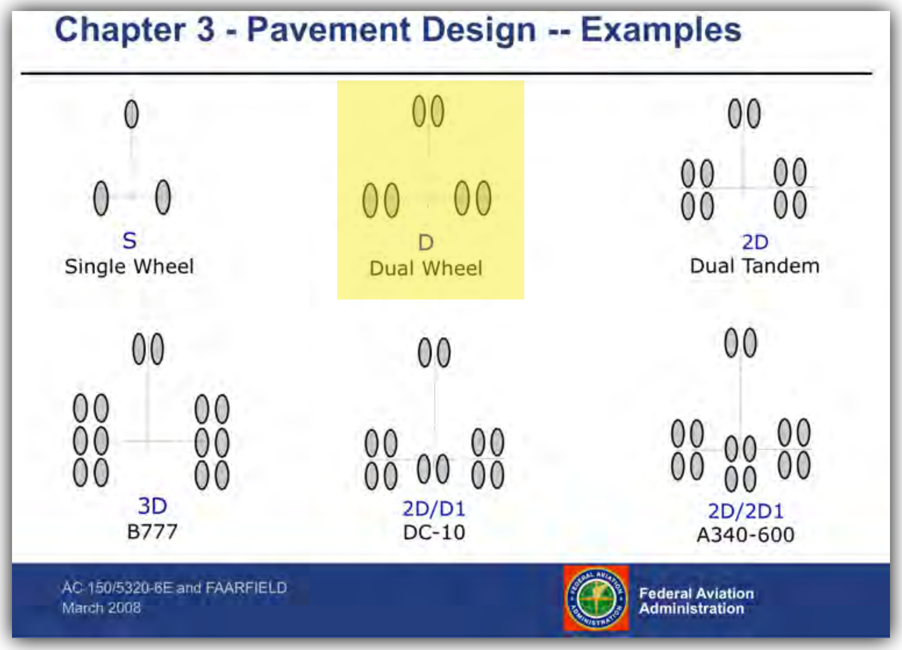
FAA Airport/Facility Directory Front Matter

Wheel Weight Bearing Limits

- **FAA Wheel Weight Bearing Limits specify a maximum aircraft weight based on the number of wheels that the aircraft rests upon.**
- **This data is available in the Airport/Facility Directory.**
- **Add “000” to the numerical figure.**
- **It is imperative to emphasize that, per the FAA, this is based on total aircraft weight, *not weight per wheel.***

All Aircraft | Pavement Weight Bearing Capacity

Graphical Wheel Description (Examples)



Chapter 3 - Pavement Design -- Examples

S
Single Wheel

D
Dual Wheel

2D
Dual Tandem

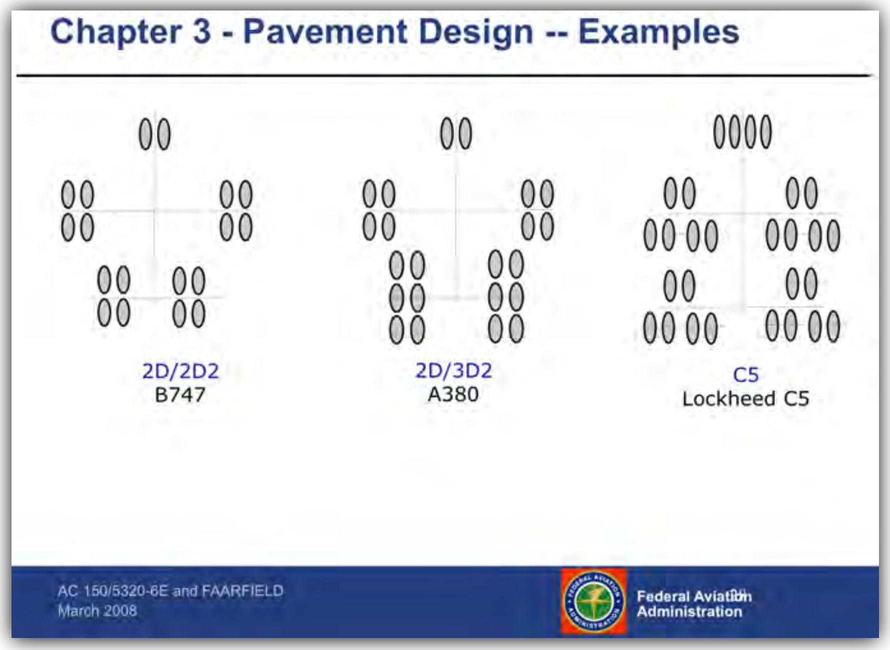
3D
B777

2D/D1
DC-10

2D/2D1
A340-600

AC 150/5320-6E and FAARFIELD
March 2008

Federal Aviation Administration



Chapter 3 - Pavement Design -- Examples

2D/2D2
B747

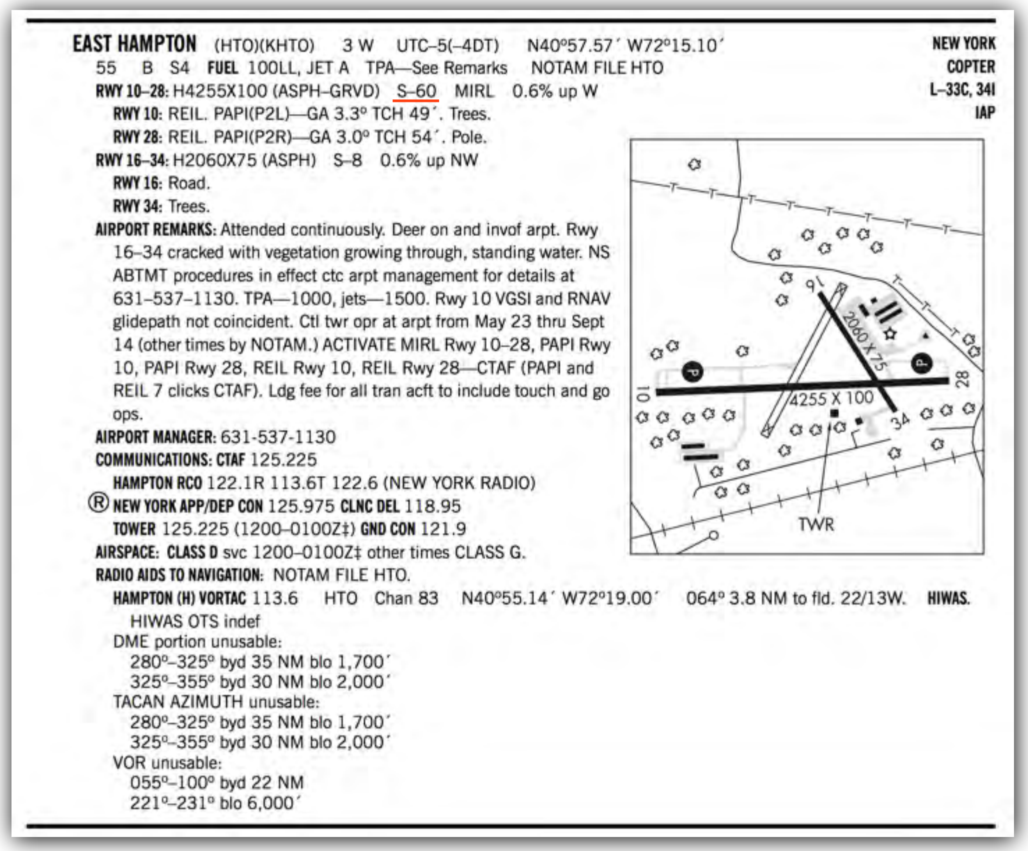
2D/3D2
A380

C5
Lockheed C5

AC 150/5320-6E and FAARFIELD
March 2008

Federal Aviation Administration

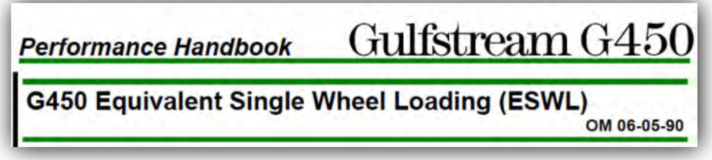
All Aircraft | Pavement Weight Bearing Capacity



EAST HAMPTON (HTO)(KHTO) 3 W UTC–5(–4DT) N40°57.57´ W72°15.10´ **NEW YORK**
55 B S4 **FUEL** 100LL, JET A TPA—See Remarks NOTAM FILE HTO **COPTER**
RWY 10–28: H4255X100 (ASPH–GRVD) S–60 MIRL 0.6% up W **L–33C, 34I**
RWY 10: REIL. PAPI(P2L)—GA 3.3° TCH 49´. Trees. **IAP**
RWY 28: REIL. PAPI(P2R)—GA 3.0° TCH 54´. Pole.
RWY 16–34: H2060X75 (ASPH) S–8 0.6% up NW
RWY 16: Road.
RWY 34: Trees.
AIRPORT REMARKS: Attended continuously. Deer on and invof arpt. Rwy 16–34 cracked with vegetation growing through, standing water. NS ABTMT procedures in effect ctc arpt management for details at 631–537–1130. TPA—1000, jets—1500. Rwy 10 VGSI and RNAV glidepath not coincident. Ctl twr opr at arpt from May 23 thru Sept 14 (other times by NOTAM.) ACTIVATE MIRL Rwy 10–28, PAPI Rwy 10, PAPI Rwy 28, REIL Rwy 10, REIL Rwy 28—CTAF (PAPI and REIL 7 clicks CTAF). Ldg fee for all tran acft to include touch and go ops.
AIRPORT MANAGER: 631-537-1130
COMMUNICATIONS: CTAF 125.225
HAMPTON RCO 122.1R 113.6T 122.6 (NEW YORK RADIO)
® **NEW YORK APP/DEP CON** 125.975 **CLNC DEL** 118.95
TOWER 125.225 (1200–0100Z‡) **GND CON** 121.9
AIRSPACE: CLASS D svc 1200–0100Z‡ other times CLASS G.
RADIO AIDS TO NAVIGATION: NOTAM FILE HTO.
HAMPTON (H) VORTAC 113.6 HTO Chan 83 N40°55.14´ W72°19.00´ 064° 3.8 NM to fld. 22/13W. **HIWAS.**
HIWAS OTS indef
DME portion unusable:
280°–325° byd 35 NM blo 1,700´
325°–355° byd 30 NM blo 2,000´
TACAN AZIMUTH unusable:
280°–325° byd 35 NM blo 1,700´
325°–355° byd 30 NM blo 2,000´
VOR unusable:
055°–100° byd 22 NM
221°–231° blo 6,000´

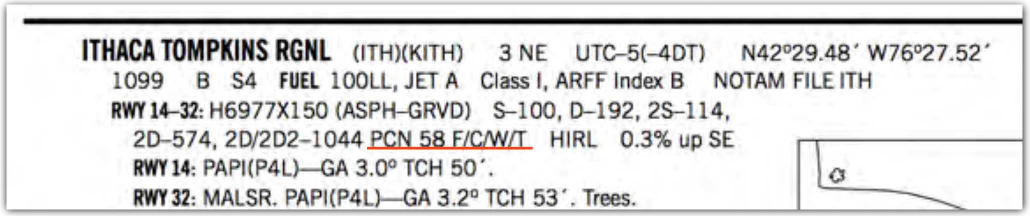
What if the A/FD only includes information pertaining to a single-wheel gear configuration?

- **Call the airport auhtority. They may have additional information.**
- **Most Gulfstream aircraft have a “Equivalent Single Wheel Loading (ESWL)” table. The G280 may have this information in the near future.**



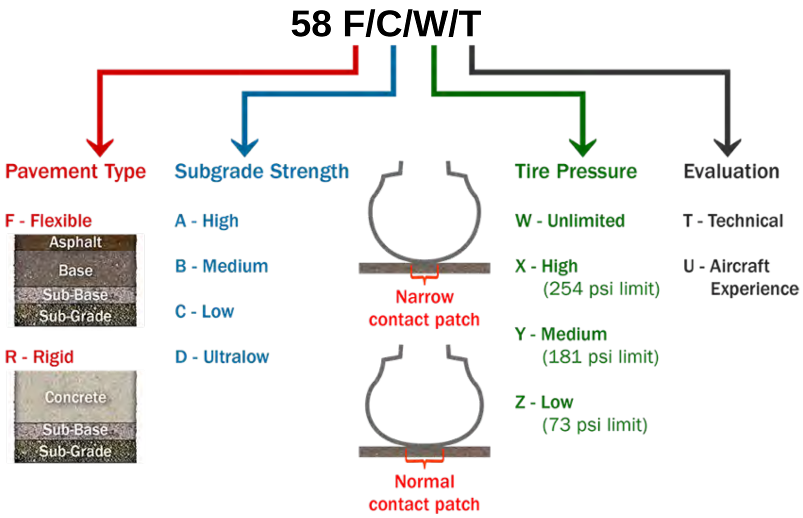
Performance Handbook Gulfstream G450
G450 Equivalent Single Wheel Loading (ESWL)
OM 06-05-90

All Aircraft | Pavement Weight Bearing Capacity



ITHACA TOMPKINS RGNL (ITH)(KITH) 3 NE UTC–5(–4DT) N42°29.48´ W76°27.52´
1099 B S4 **FUEL** 100LL, JET A Class I, ARFF Index B NOTAM FILE ITH
RWY 14–32: H6977X150 (ASPH–GRVD) S–100, D–192, 2S–114,
2D–574, 2D/2D2–1044 PCN 58 F/C/W/T HIRL 0.3% up SE
RWY 14: PAPI(P4L)—GA 3.0° TCH 50´.
RWY 32: MALSR. PAPI(P4L)—GA 3.2° TCH 53´. Trees.

FAA Airport/Facility Directory



58 F/C/W/T

Pavement Type

F - Flexible

R - Rigid

Subgrade Strength

A - High

B - Medium

C - Low

D - Ultralow

Narrow contact patch

Normal contact patch

Tire Pressure

W - Unlimited

X - High (254 psi limit)

Y - Medium (181 psi limit)

Z - Low (73 psi limit)

Evaluation

T - Technical

U - Aircraft Experience

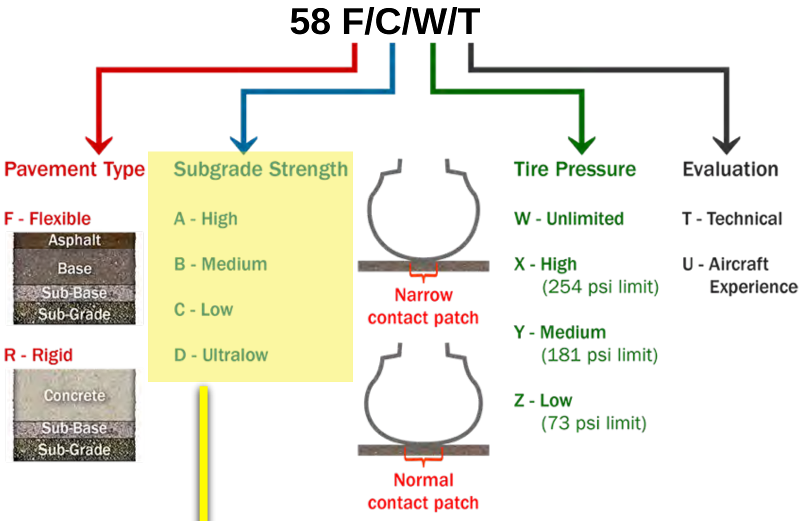
Credit: Aviation Week

What is PCN?

- **Pavement Classification Number (PCN)**: Single unique number to express the load-carrying capacity of a pavement, without specifying a particular airplane or pavement structure.

- As shown in the graphic, tire pressure also affects the amount of force applied to a given portion of the pavement. This will be addressed later.

All Aircraft | Pavement Weight Bearing Capacity



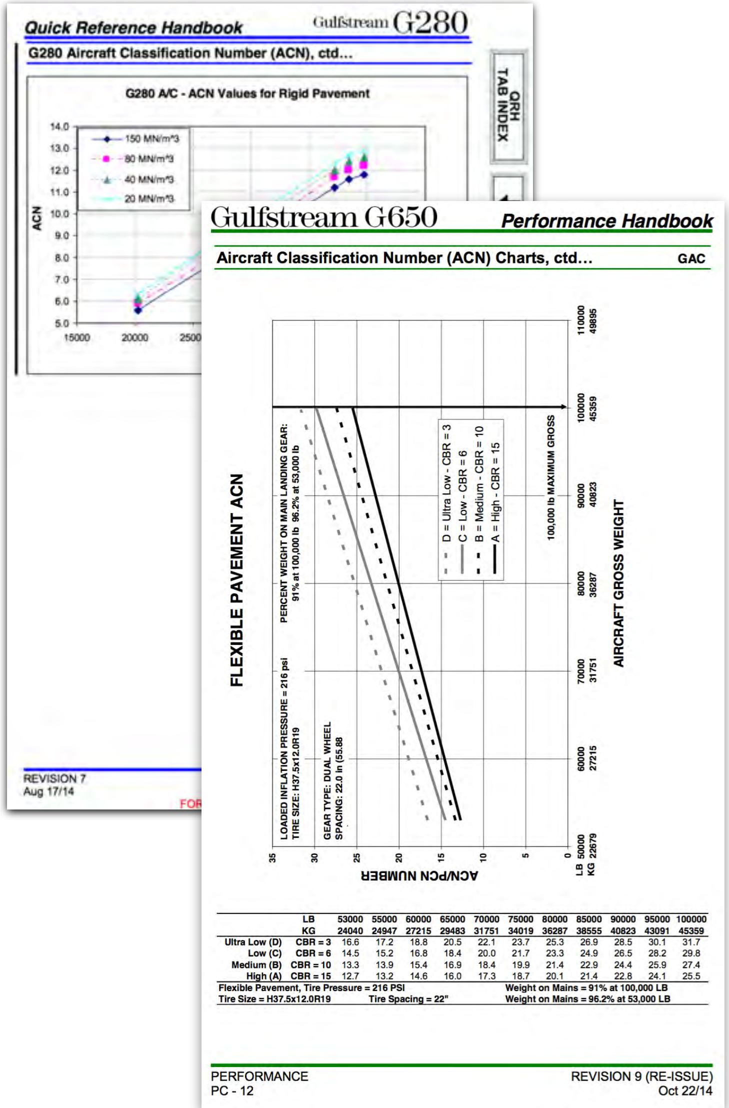
Credit: Aviation Week

What is PCN (continued)?

- **Subgrade strength can be translated into California Bearing Ratio (CBR), which is the ICAO-preferred unit.**
- **It can also be translated into a K-value.**
- **Many of these terms are present in Gulfstream performance guidance.**

Subgrade Strength	CBR Value	K-Value
A	15	150
B	10	80
C	6	40
D	3	20

All Aircraft | Pavement Weight Bearing Capacity



Quick Reference Handbook Gulfstream G280

G280 Aircraft Classification Number (ACN), ctd...

G280 A/C - ACN Values for Rigid Pavement

REVISION 7
Aug 17/14

Gulfstream G650 **Performance Handbook**

Aircraft Classification Number (ACN) Charts, ctd... GAC

	LB	53000	55000	60000	65000	70000	75000	80000	85000	90000	95000	100000
	KG	24040	24947	27215	29483	31751	34019	36287	38555	40823	43091	45359
Ultra Low (D)	CBR = 3	16.6	17.2	18.8	20.5	22.1	23.7	25.3	26.9	28.5	30.1	31.7
Low (C)	CBR = 6	14.5	15.2	16.8	18.4	20.0	21.7	23.3	24.9	26.5	28.2	29.8
Medium (B)	CBR = 10	13.3	13.9	15.4	16.9	18.4	19.9	21.4	22.9	24.4	25.9	27.4
High (A)	CBR = 15	12.7	13.2	14.6	16.0	17.3	18.7	20.1	21.4	22.8	24.1	25.5

Flexible Pavement, Tire Pressure = 216 PSI Weight on Mains = 91% at 100,000 LB
Tire Size = H37.5x12.0R19 Tire Spacing = 22" Weight on Mains = 96.2% at 53,000 LB

PERFORMANCE
PC - 12

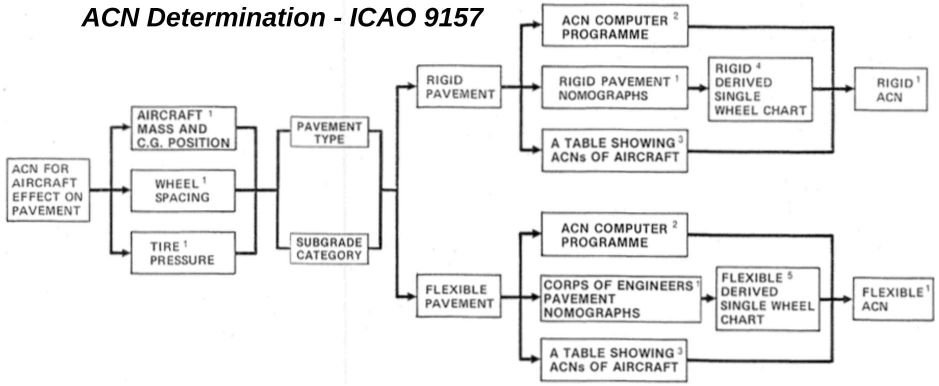
REVISION 9 (RE-ISSUE)
Oct 22/14

What is ACN?

- **Aircraft Classification Number (ACN)**: Single unique number to express effect of an individual airplane on *different pavements*.

Generally, ACN must be less than or equal to PCN. Exceptions are discussed in the executive summary.

ACN Determination - ICAO 9157



ACN FOR AIRCRAFT EFFECT ON PAVEMENT

AIRCRAFT MASS AND C.G. POSITION [1]

WHEEL SPACING [1]

TIRE PRESSURE [1]

PAVEMENT TYPE

SUBGRADE CATEGORY

RIGID PAVEMENT

FLEXIBLE PAVEMENT

ACN COMPUTER PROGRAMME [2]

RIGID PAVEMENT NOMOGRAPHS [1]

A TABLE SHOWING ACNs OF AIRCRAFT [3]

RIGID DERIVED SINGLE WHEEL CHART [4]

RIGID ACN [1]

ACN COMPUTER PROGRAMME [2]

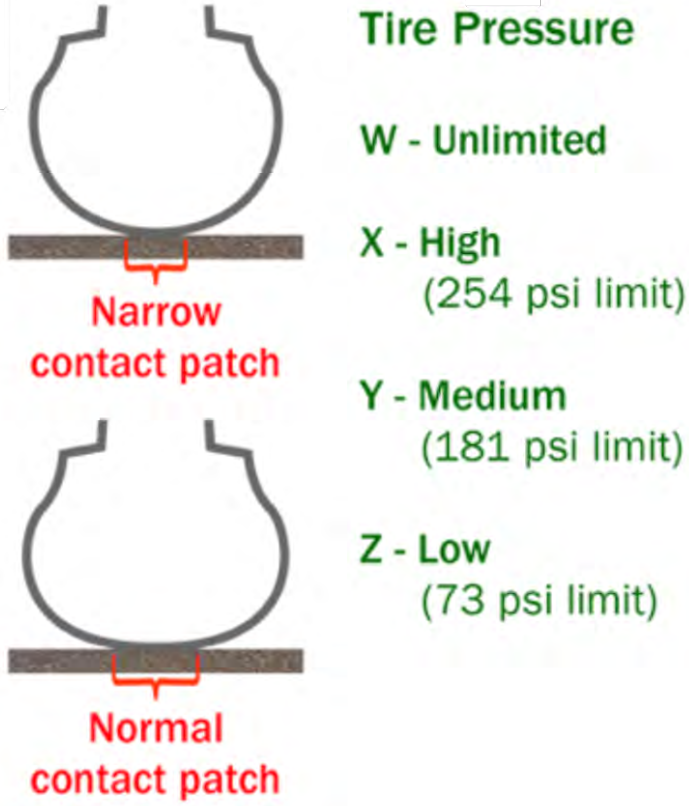
CORPS OF ENGINEERS PAVEMENT NOMOGRAPHS [1]

A TABLE SHOWING ACNs OF AIRCRAFT [3]

FLEXIBLE DERIVED SINGLE WHEEL CHART [5]

FLEXIBLE ACN [1]

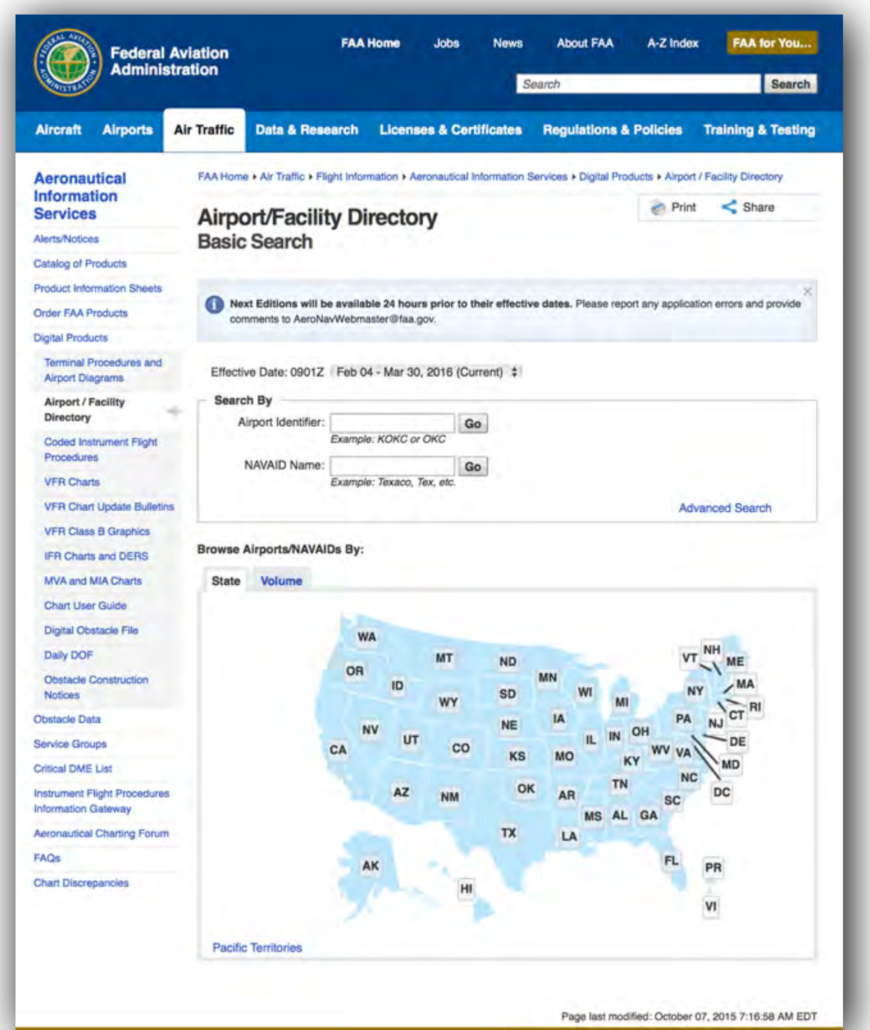
All Aircraft | Pavement Weight Bearing Capacity



Tire Pressure

- **Tire pressure effects effects the amount of contact a wheel has with a surface, thereby affecting how much weight a given amount of pavement is exposed to. Maximum pressure limits are assigned to pavement to ensure that a minimum amount of contact is provided.**
- **The codes and numbers in the graphic to the left are updated to reflect new ICAO standards, whereas the codes/numbers in Gulfstream publications reflect older standards (including a “very low” rating).**
- ***Due to further aircraft weight restrictions when lowering tire pressures, lowering tire pressure is not a recommended method for normal operations to meet a desired PCN and will not be addressed in this briefing.***

All Aircraft | Pavement Weight Bearing Capacity



Airport/Facility Directory

Basic Search

Next Editions will be available 24 hours prior to their effective dates. Please report any application errors and provide comments to AeroNavWebmaster@faa.gov.

Effective Date: 0901Z Feb 04 - Mar 30, 2016 (Current)

Search By

Airport Identifier:

Example: KOKC or OKC

NAVAID Name:

Example: Texaco, Tex, etc.

Browse Airports/NAVAIDs By:

Page last modified: October 07, 2015 7:16:58 AM EDT

Click image to access website

Where can I access PCN/Runway Weight Bearing information for US Airports?

- **The Airport/Facility Directory (A/FD) is a good source for this information.**

All Aircraft | Pavement Weight Bearing Capacity

Where can I find PCN/Runway Weight Bearing information for International Airports?

Examples include:

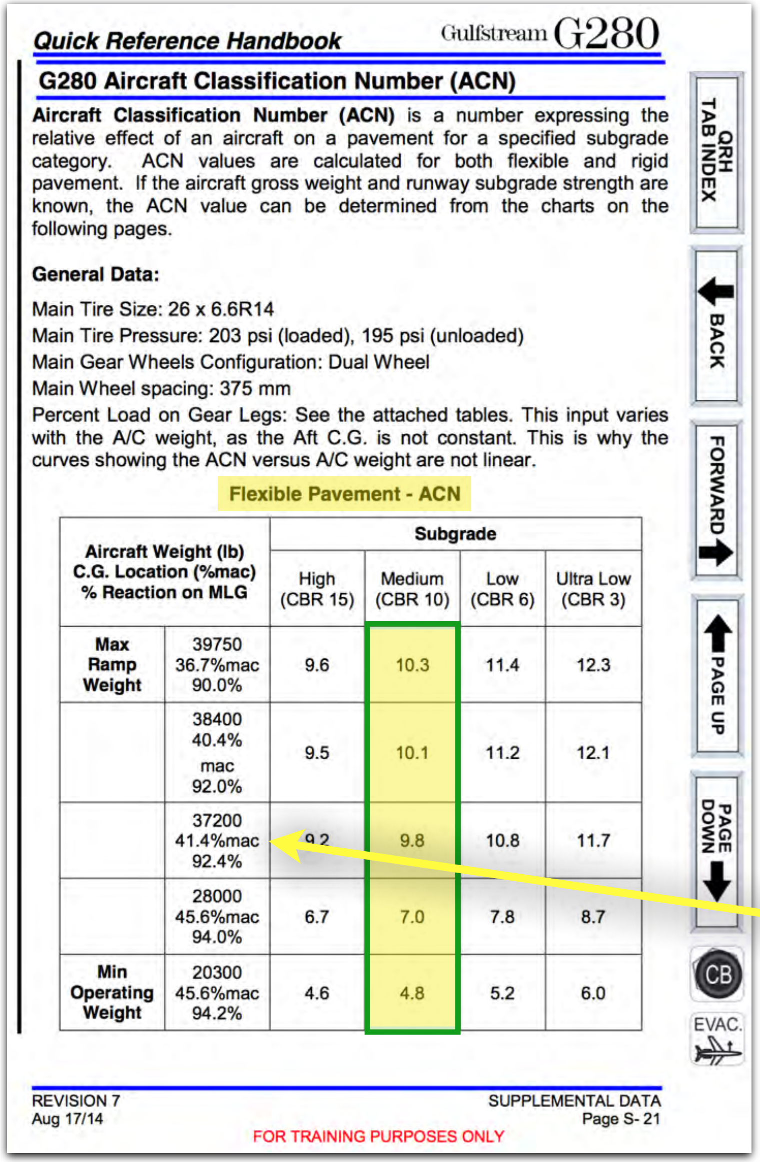
- **The Jeppesen Airport Directory, much like the FAA A/FD, contains PCN information.**

- **AC-U-KWIK also contains this data.**

- **NOTE: if wheel weight bearing capacity is listed in lieu of PCN for international airports, weights may be *per wheel*, not total aircraft weight (opposite of FAA numbers).**

All Aircraft | Pavement Weight Bearing Capacity

MID CABIN (G280) EXAMPLE



Quick Reference Handbook Gulfstream G280

G280 Aircraft Classification Number (ACN)

Aircraft Classification Number (ACN) is a number expressing the relative effect of an aircraft on a pavement for a specified subgrade category. ACN values are calculated for both flexible and rigid pavement. If the aircraft gross weight and runway subgrade strength are known, the ACN value can be determined from the charts on the following pages.

General Data:

Main Tire Size: 26 x 6.6R14
Main Tire Pressure: 203 psi (loaded), 195 psi (unloaded)
Main Gear Wheels Configuration: Dual Wheel
Main Wheel spacing: 375 mm

Percent Load on Gear Legs: See the attached tables. This input varies with the A/C weight, as the Aft C.G. is not constant. This is why the curves showing the ACN versus A/C weight are not linear.

Flexible Pavement - ACN

Aircraft Weight (lb) C.G. Location (%mac) % Reaction on MLG		**Subgrade**			
		High (CBR 15)	Medium (CBR 10)	Low (CBR 6)	Ultra Low (CBR 3)
Max Ramp Weight	39750 36.7%mac 90.0%	9.6	10.3	11.4	12.3
	38400 40.4% mac 92.0%	9.5	10.1	11.2	12.1
	37200 41.4%mac 92.4%	9.2	9.8	10.8	11.7
	28000 45.6%mac 94.0%	6.7	7.0	7.8	8.7
Min Operating Weight	20300 45.6%mac 94.2%	4.6	4.8	5.2	6.0

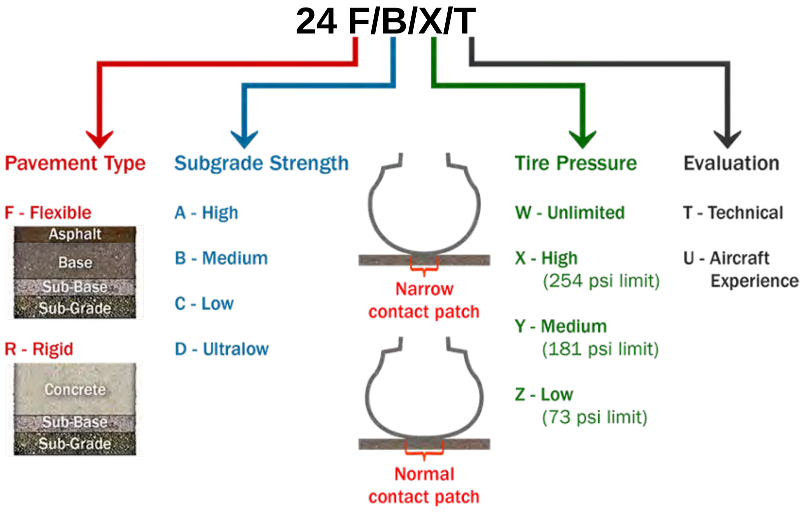
REVISION 7
Aug 17/14

SUPPLEMENTAL DATA
Page S- 21

FOR TRAINING PURPOSES ONLY

Example: Lake Placid, NY

- PCN: 24 F/B/X/T



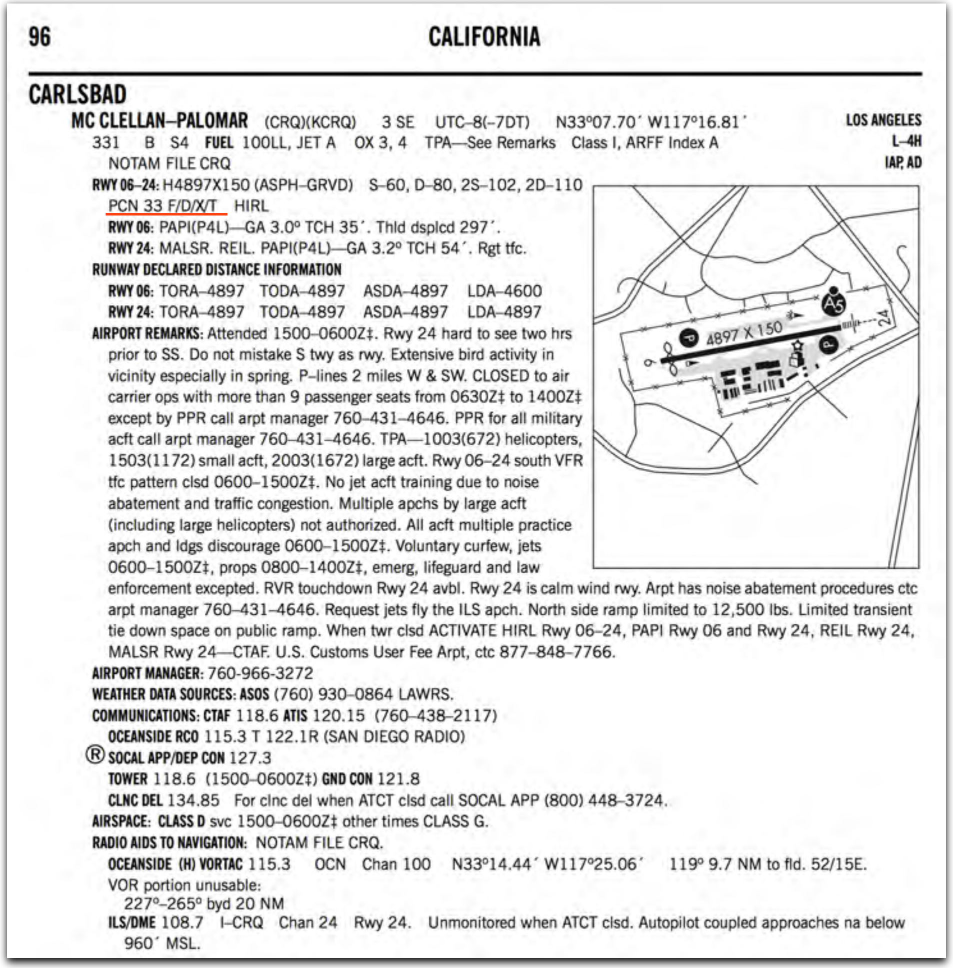
Credit: Aviation Week

Note that the %MAC is at its rearward extreme, thereby placing the most weight possible on the main gear (92.4%). This is the most limiting condition. All Gulfstream ACNs are determined using this conservative methodology.

Entire weight band acceptable for PCN

All Aircraft | Pavement Weight Bearing Capacity

LARGE CABIN (G650) EXAMPLE - PCN



96 CALIFORNIA

CARLSBAD

MC CLELLAN–PALOMAR (CRQ)(KCRQ) 3 SE UTC–8(–7DT) N33°07.70´ W117°16.81´ **LOS ANGELES**

331 B S4 **FUEL** 100LL, JET A OX 3, 4 TPA—See Remarks Class I, ARFF Index A **L–4H**

NOTAM FILE CRQ **IAP, AD**

RWY 06–24: H4897X150 (ASPH–GRVD) S–60, D–80, 2S–102, 2D–110

PCN 33 F/D/X/T HIRL

RWY 06: PAPI(P4L)—GA 3.0° TCH 35´. Thld dsplcd 297´.

RWY 24: MALSR. REIL. PAPI(P4L)—GA 3.2° TCH 54´. Rgt tfc.

RUNWAY DECLARED DISTANCE INFORMATION

RWY 06: TORA–4897 TODA–4897 ASDA–4897 LDA–4600

RWY 24: TORA–4897 TODA–4897 ASDA–4897 LDA–4897

AIRPORT REMARKS: Attended 1500–0600Z‡. Rwy 24 hard to see two hrs prior to SS. Do not mistake S twy as rwy. Extensive bird activity in vicinity especially in spring. P–lines 2 miles W & SW. CLOSED to air carrier ops with more than 9 passenger seats from 0630Z‡ to 1400Z‡ except by PPR call arpt manager 760–431–4646. PPR for all military acft call arpt manager 760–431–4646. TPA—1003(672) helicopters, 1503(1172) small acft, 2003(1672) large acft. Rwy 06–24 south VFR tfc pattern clsd 0600–1500Z‡. No jet acft training due to noise abatement and traffic congestion. Multiple apchs by large acft (including large helicopters) not authorized. All acft multiple practice apch and ldgs discourage 0600–1500Z‡. Voluntary curfew, jets 0600–1500Z‡, props 0800–1400Z‡, emerg, lifeguard and law enforcement excepted. RVR touchdown Rwy 24 avbl. Rwy 24 is calm wind rwy. Arpt has noise abatement procedures ctc arpt manager 760–431–4646. Request jets fly the ILS apch. North side ramp limited to 12,500 lbs. Limited transient tie down space on public ramp. When twr clsd ACTIVATE HIRL Rwy 06–24, PAPI Rwy 06 and Rwy 24, REIL Rwy 24, MALSR Rwy 24—CTAF. U.S. Customs User Fee Arpt, ctc 877–848–7766.

AIRPORT MANAGER: 760-966-3272

WEATHER DATA SOURCES: ASOS (760) 930–0864 LAWRS.

COMMUNICATIONS: CTAF 118.6 **ATIS** 120.15 (760–438–2117)

OCEANSIDE RCO 115.3 T 122.1R (SAN DIEGO RADIO)

Ⓡ **SOCAL APP/DEP CON** 127.3

TOWER 118.6 (1500–0600Z‡) **GND CON** 121.8

CLNC DEL 134.85 For clnc del when ATCT clsd call SOCAL APP (800) 448–3724.

AIRSPACE: CLASS D svc 1500–0600Z‡ other times CLASS G.

RADIO AIDS TO NAVIGATION: NOTAM FILE CRQ.

OCEANSIDE (H) VORTAC 115.3 OCN Chan 100 N33°14.44´ W117°25.06´ 119° 9.7 NM to fld. 52/15E.

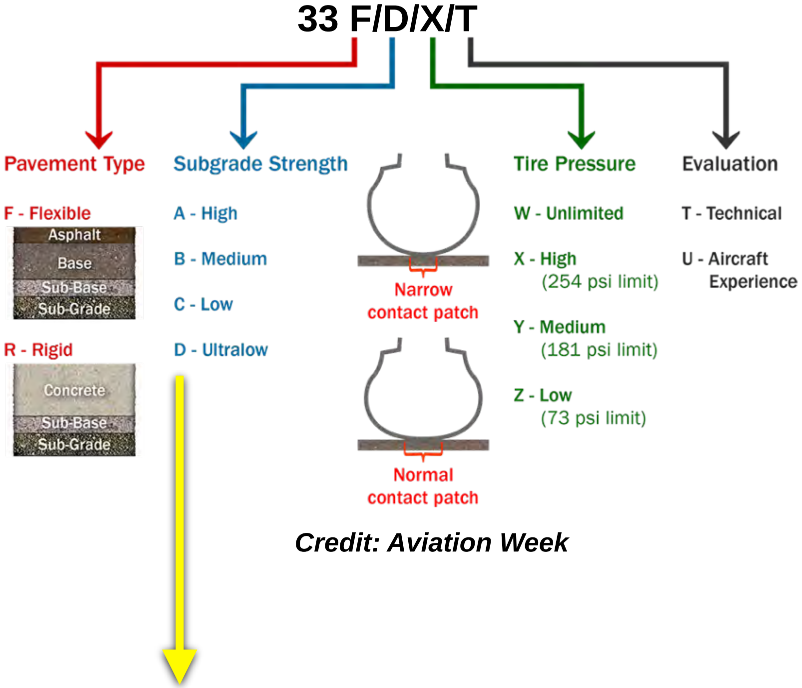
VOR portion unusable:

227°–265° byd 20 NM

ILS/DME 108.7 I–CRQ Chan 24 Rwy 24. Unmonitored when ATCT clsd. Autopilot coupled approaches na below 960´ MSL.

Example: Carlsbad, CA

- PCN: 33 F/D/X/T

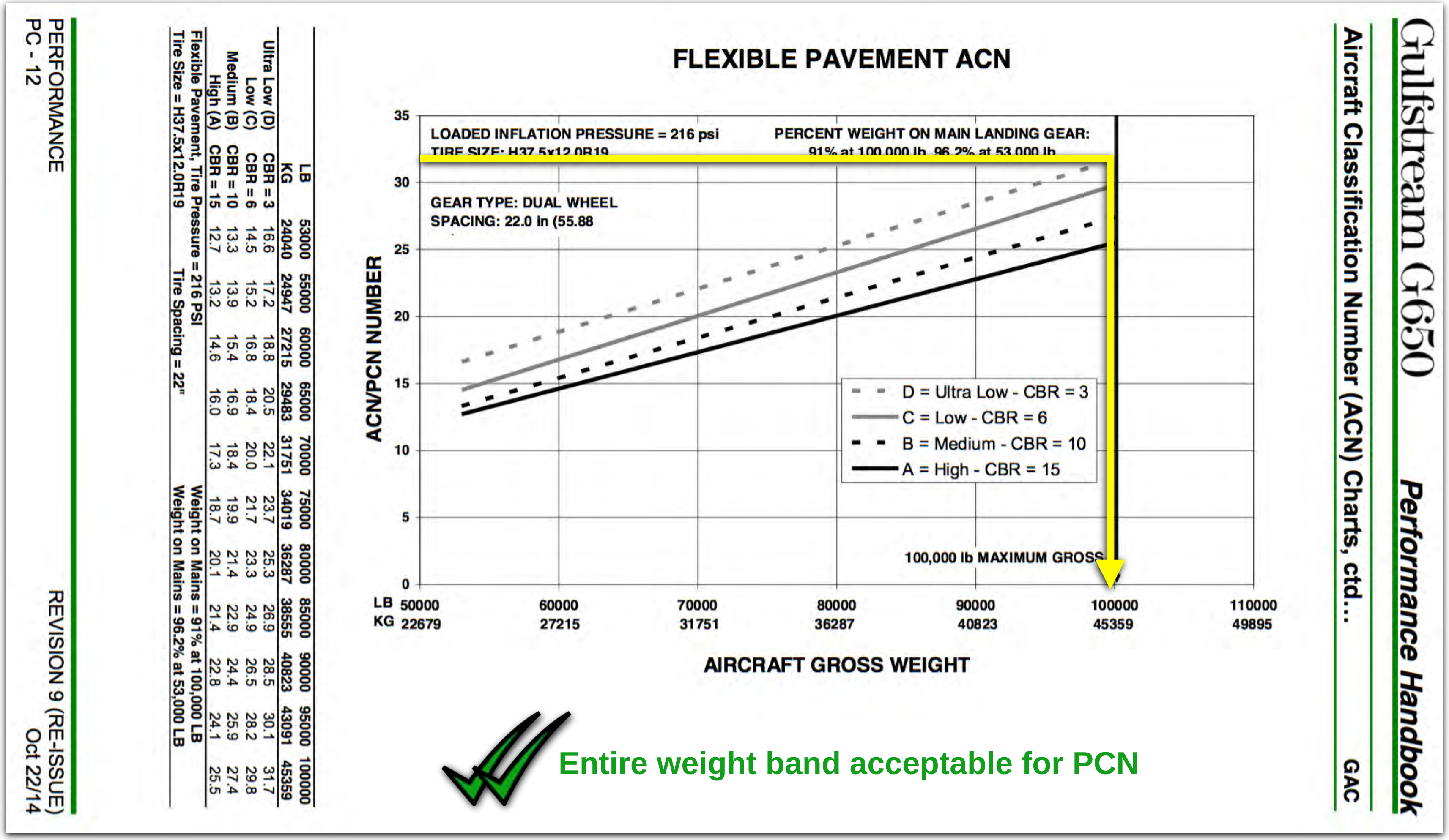


Credit: Aviation Week

Subgrade Strength	CBR Value	K-Value
D	3	20

All Aircraft | Pavement Weight Bearing Capacity

LARGE CABIN (G650) EXAMPLE - PCN



Gulfstream G650 ***Performance Handbook***

Aircraft Classification Number (ACN) Charts, ctd... **GAC**

FLEXIBLE PAVEMENT ACN

	LB	53000	55000	60000	65000	70000	75000	80000	85000	90000	95000	100000
	KG	24040	24947	27215	29483	31751	34019	36287	38555	40823	43091	45359
Ultra Low (D)	CBR = 3	16.6	17.2	18.8	20.5	22.1	23.7	25.3	26.9	28.5	30.1	31.7
Low (C)	CBR = 6	14.5	15.2	16.8	18.4	20.0	21.7	23.3	24.9	26.5	28.2	29.8
Medium (B)	CBR = 10	13.3	13.9	15.4	16.9	18.4	19.9	21.4	22.9	24.4	25.9	27.4
High (A)	CBR = 15	12.7	13.2	14.6	16.0	17.3	18.7	20.1	21.4	22.8	24.1	25.5

Flexible Pavement, Tire Pressure = 216 PSI Weight on Mains = 91% at 100,000 LB

Tire Size = H37.5x12.0R19 Tire Spacing = 22" Weight on Mains = 96.2% at 53,000 LB

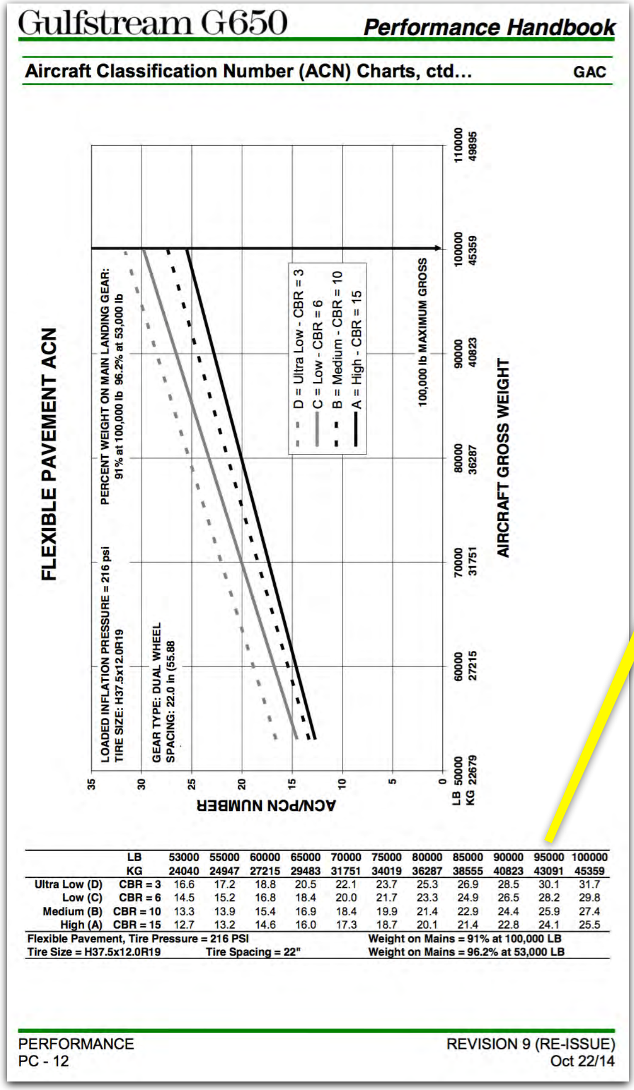
PERFORMANCE
PC - 12

REVISION 9 (RE-ISSUE)
Oct 22/14

✓✓ **Entire weight band acceptable for PCN**

All Aircraft | Pavement Weight Bearing Capacity

LARGE CABIN (G650) EXAMPLE - PCN



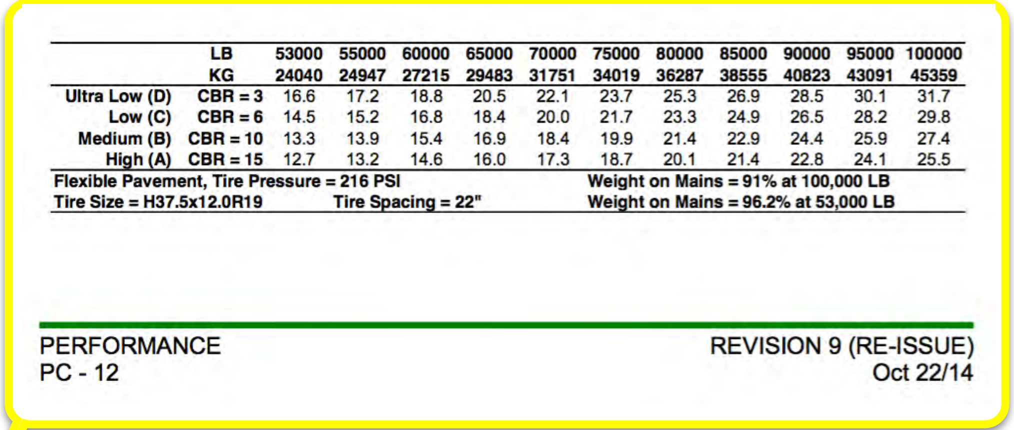
Gulfstream G650 ***Performance Handbook***

Aircraft Classification Number (ACN) Charts, ctd... **GAC**

	LB	53000	55000	60000	65000	70000	75000	80000	85000	90000	95000	100000
	KG	24040	24947	27215	29483	31751	34019	36287	38555	40823	43091	45359
Ultra Low (D)	CBR = 3	16.6	17.2	18.8	20.5	22.1	23.7	25.3	26.9	28.5	30.1	31.7
Low (C)	CBR = 6	14.5	15.2	16.8	18.4	20.0	21.7	23.3	24.9	26.5	28.2	29.8
Medium (B)	CBR = 10	13.3	13.9	15.4	16.9	18.4	19.9	21.4	22.9	24.4	25.9	27.4
High (A)	CBR = 15	12.7	13.2	14.6	16.0	17.3	18.7	20.1	21.4	22.8	24.1	25.5

Flexible Pavement, Tire Pressure = 216 PSI
Weight on Mains = 91% at 100,000 LB
Tire Size = H37.5x12.0R19
Tire Spacing = 22"
Weight on Mains = 96.2% at 53,000 LB

PERFORMANCE
PC - 12
REVISION 9 (RE-ISSUE)
Oct 22/14



	LB	53000	55000	60000	65000	70000	75000	80000	85000	90000	95000	100000
	KG	24040	24947	27215	29483	31751	34019	36287	38555	40823	43091	45359
Ultra Low (D)	CBR = 3	16.6	17.2	18.8	20.5	22.1	23.7	25.3	26.9	28.5	30.1	31.7
Low (C)	CBR = 6	14.5	15.2	16.8	18.4	20.0	21.7	23.3	24.9	26.5	28.2	29.8
Medium (B)	CBR = 10	13.3	13.9	15.4	16.9	18.4	19.9	21.4	22.9	24.4	25.9	27.4
High (A)	CBR = 15	12.7	13.2	14.6	16.0	17.3	18.7	20.1	21.4	22.8	24.1	25.5

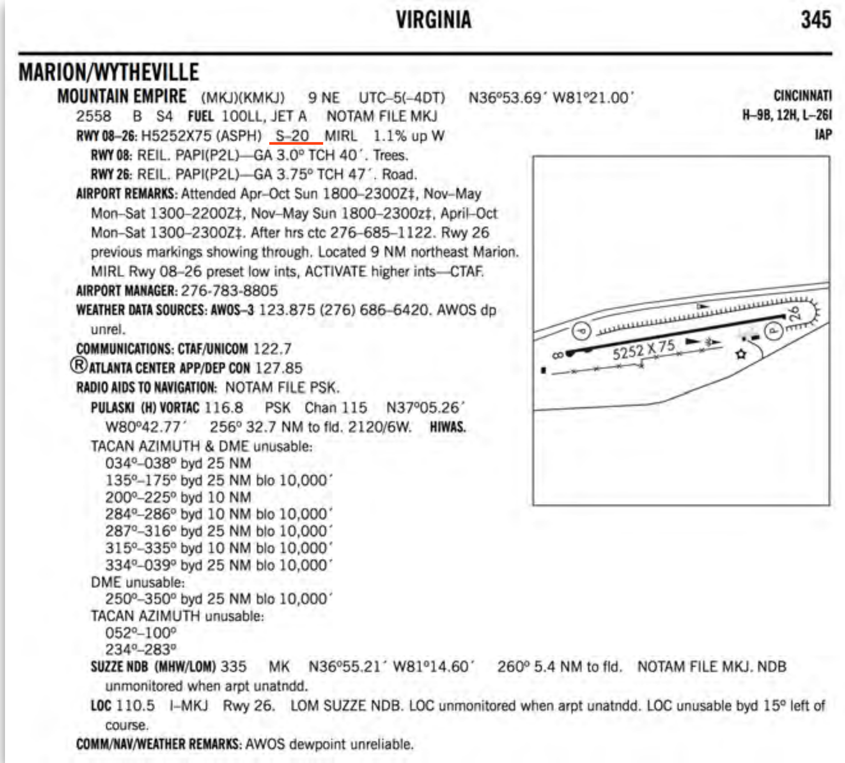
Flexible Pavement, Tire Pressure = 216 PSI
Weight on Mains = 91% at 100,000 LB
Tire Size = H37.5x12.0R19
Tire Spacing = 22"
Weight on Mains = 96.2% at 53,000 LB

PERFORMANCE
PC - 12
REVISION 9 (RE-ISSUE)
Oct 22/14

- As an alternative to consulting the line graph, the tables provided at the bottom of the page can be used to interpolate and find more accurate values.

All Aircraft | Pavement Weight Bearing Capacity

LARGE CABIN (G550) EXAMPLE - ESWL



VIRGINIA **345**

MARION/WYTHEVILLE

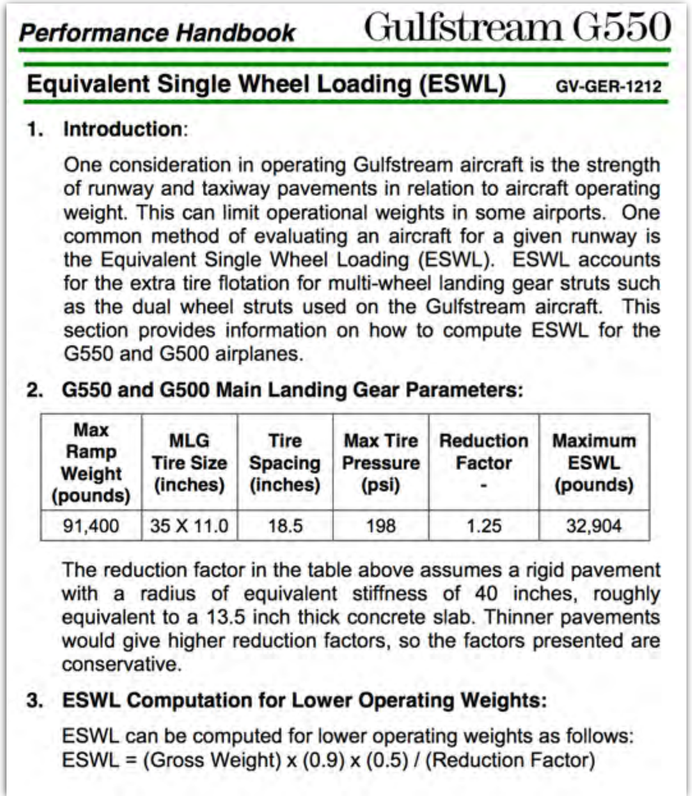
MOUNTAIN EMPIRE (MKJ)(KMKJ) 9 NE UTC–5(–4DT) N36°53.69´ W81°21.00´ **CINCINNATI**
2558 B S4 **FUEL** 100LL, JET A NOTAM FILE MKJ **H–9B, 12H, L–26I**
RWY 08–26: H5252X75 (ASPH) S–20 MIRL 1.1% up W **IAP**
RWY 08: REIL. PAPI(P2L)—GA 3.0° TCH 40´. Trees.
RWY 26: REIL. PAPI(P2L)—GA 3.75° TCH 47´. Road.
AIRPORT REMARKS: Attended Apr–Oct Sun 1800–2300Z‡, Nov–May Mon–Sat 1300–2200Z‡, Nov–May Sun 1800–2300z‡, April–Oct Mon–Sat 1300–2300Z‡. After hrs ctc 276–685–1122. Rwy 26 previous markings showing through. Located 9 NM northeast Marion. MIRL Rwy 08–26 preset low ints, ACTIVATE higher ints—CTAF.
AIRPORT MANAGER: 276-783-8805
WEATHER DATA SOURCES: **AWOS–3** 123.875 (276) 686–6420. AWOS dp unrel.
COMMUNICATIONS: CTAF/UNICOM 122.7
®**ATLANTA CENTER APP/DEP CON** 127.85
RADIO AIDS TO NAVIGATION: NOTAM FILE PSK.
PULASKI (H) VORTAC 116.8 PSK Chan 115 N37°05.26´ W80°42.77´ 256° 32.7 NM to fld. 2120/6W. **HIWAS.**
TACAN AZIMUTH & DME unusable:
034°–038° byd 25 NM
135°–175° byd 25 NM blo 10,000´
200°–225° byd 10 NM
284°–286° byd 10 NM blo 10,000´
287°–316° byd 25 NM blo 10,000´
315°–335° byd 10 NM blo 10,000´
334°–039° byd 25 NM blo 10,000´
DME unusable:
250°–350° byd 25 NM blo 10,000´
TACAN AZIMUTH unusable:
052°–100°
234°–283°
SUZZE NDB (MHW/LOM) 335 MK N36°55.21´ W81°14.60´ 260° 5.4 NM to fld. NOTAM FILE MKJ. NDB unmonitored when arpt unatndd.
LOC 110.5 I–MKJ Rwy 26. LOM SUZZE NDB. LOC unmonitored when arpt unatndd. LOC unusable byd 15° left of course.
COMM/NAV/WEATHER REMARKS: AWOS dewpoint unreliable.

(55,000lbs) x (0.9)x(0.5)/(1.25) =

19,800lbs Equivalent Single Wheel Loading

Example: Mountain Empire, VA

- **FAA Wheel Weight Bearing Limit: S-20**
- **Landing weight: 55,000lbs.**



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)

Gulfstream FLIGHT OPS

Operations Briefing

Exhibit 1, Attachment 5

GRI
9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
p | 503-641-3478 f | 503-644-8034

September 16, 2019

6289 AURORA STATE AIRPORT RUNWAY 17-35 PCN EVALUATION
(ISSUED 11/12/2019)

Century West Engineering Corporation
5331 SW Macadam Avenue, Suite 287
Portland, OR 97239

Attention: James Kirby, PE
Senior Project Manager

SUBJECT: Pavement Classification Number (PCN) Evaluation of Runway 17-35
Aurora State Airport (UAO)
Aurora, Oregon

As requested, GRI conducted a pavement evaluation at Aurora State Airport (UAO) in support of the Oregon Department of Aviation (ODA) to develop a pavement classification number (PCN) for Runway 17-35.

PROJECT DESCRIPTION

Our work included review of relevant ODA records for Runway 17-35, falling weight deflectometer (FWD) testing, core explorations, and engineering analyses in accordance with Federal Aviation Administration (FAA) Advisory Circular 150/5335-5C, *Standardized Method of Reporting Airport Pavement Strength – PCN*. According to the FAA, the PCN is a number that expresses the load-carrying capacity of a pavement for unrestricted operations. We determined the PCN using the Technical Evaluation Method specified in Advisory Circular 150/5335-5C.

BACKGROUND

Based on information provided in the ODA pavement evaluation/maintenance management program report prepared by Pavement Consultant Inc. in 2018, a 4,100-ft-long segment on the north end of the runway was first constructed in 1943 and in 1993, a 900-ft-long extension was built to the south. The last major rehabilitation on the runway was conducted in 2005 and generally consisted of a 2- to 3-in. overlay.

The current Airport Master Record, FAA Form 5010, lists the gross weight limit for a single-wheel, main-gear aircraft and a dual-wheel, main-gear aircraft at 30,000 and 45,000 lbs, respectively. UAO currently does not have an established PCN.

FIELD WORK

Site Reconnaissance

A visual pavement reconnaissance was performed by GRI engineers on August 12, 2019, to assess the general surface condition of the pavements within the project and to identify core exploration locations.

Falling Weight Deflectometer Tests

GRI conducted FWD testing on August 20, 2019, along the full length of the runway. The testing was conducted in accordance with FAA Advisory Circular 150/5370-11b, *Use of Nondestructive Testing in the Evaluation of Airport Pavements,* using our KUAB 2m Model 150 FWD device.

FWD testing was completed along test lines located at 7 ft west and 12 ft east of the runway centerline. The tests were spaced at approximately 200-ft intervals within the runway keel section. The approximate locations of the test lines are shown on Figure 1.

The FWD test procedures are described in Appendix A. The data were normalized to a 30,000-lb load basis and the FWD deflection data are shown in Table 1A.

We also reviewed the load-response data measured by the FWD to provide a preliminary understanding of the overall stiffness of the pavement structure. Although this information does not provide information about the stiffness of individual soil and pavement layers, it does provide a quick assessment of the overall stiffness of the pavement system to gauge the variability of pavement stiffness within a particular pavement facility. Impact stiffness modulus (ISM) is inversely proportional to deflection and is therefore a direct measurement of the combined stiffness, or resistance to deflection induced by FWD loading, of the pavement and subgrade soils. As such, it is usually a relative measure of the pavement's ability to support loads, i.e., high ISM modulus values usually correspond to high pavement strength and vice versa. The profile of relative pavement strength along the two FWD test lines, as measured by resistance to deflection under FWD loading, is plotted for each FWD test location on Figure 4A. Additional discussion regarding ISM is provided in Appendix A.

Coring Explorations

General. On August 20, 2019, GRI conducted three core explorations, all of which were located over cracks. The approximate locations of the explorations are shown on the Site Plan, Figure 1. Details of our field investigations are further discussed in Appendix A of this report and the core explorations are summarized in Table 1.

Table 1: SUMMARY OF CORING EXPLORATION RESULTS

Core No.	FWD Test No.	Test Line	Station	Asphalt Concrete Thickness, in.	Aggregate Base Thickness, in.	Drilled Over a Crack?	Depth of Crack, in.
B-1	26	7 ft west	56+81	8.75	15.00	Yes	2.50
B-2	16	7 ft west	39+51	9.00	15.00	Yes	3.25
B-3	32	12 ft east	19+41	9.00	15.00	Yes	2.50

Existing Pavement Conditions

Overall, the pavement surface of Runway 17-35 appears to be in good condition. The primary distresses observed on the runway are low- to medium-severity longitudinal cracking, primarily at paving-panel joints or along the centerline; low-severity weathering; and isolated low-severity alligator cracking within the gear paths.

Since the alligator cracking within the gear paths (noted above) is a load-associated distress, in our opinion, it warranted further investigation and we therefore conducted the three core explorations in areas of alligator cracking on the runway. As shown in Table 1 and the photo logs on Figures 1A through 3A in Appendix A, the cracking is top down and extends to a depth of 2.5 in. in cores B-1 and B-3 and to a depth of 3.25 in. in B-2. These types of cracks may be induced by excessive shear stresses imposed by aircraft wheel loads at the runway surface and can typically be repaired by milling to the depth of cracking and overlaying. In our opinion, pavement exhibiting this type of distress should be rehabilitated when the cracking progresses to the point that spalling begins to occur and therefore represents a significant Foreign Object Damage (FOD) potential. The core samples also exhibit delamination (separation of asphalt concrete [AC] layers) at a depth of 2.5 and 3.25 in. in cores B-2 and B-3, respectively. The depth of delamination generally agrees with the thickness of the 2005 overlay.

DESIGN PROCEDURES AND ANALYSIS

Traffic Loading

Century West Engineering Corporation (CWE) provided an estimate of the aircraft traffic-volume data consisting of the number of operations (i.e., either an arrival or departure) for Runway 17-35 in 2018 from the FAA Traffic Flow Management System Counts (TFMSC). Our traffic-loading estimate is based on an annual growth rate of 1.58% per year, which is based on the aviation forecasts provided in the current master plan for UAO (WHPacific, 2012).

The COMFAA 3.0 software used to compute the PCN has inputs for each aircraft type (in the mix), which include the type of aircraft, gross weight, and number of annual departures over a 20-year period. The program does not take into account the annual growth rate, so we calculated the total departures from 2020 to 2040 to determine the equivalent annual number of departures for the analysis. The aircraft mix and annual number of departures we input into COMFAA are provided in Table 2.

Table 2: RUNWAY 17-35: AIRCRAFT TYPES AND DEPARTURE VOLUMES

Aircraft Type	Maximum Takeoff Weight, lbs	Design Aircraft for COMFAA	2018 Annual Operations	2040 Annual Operations	Values Entered into COMFAA	
					Equivalent Airplane	Annual # of Departures
Bombardier Global Express	92,500	Gulfstream G-V	50	61	Gulfstream G-V	64
Gulfstream G600	91,600	Gulfstream G-V	2	3		
Gulfstream V	76,850	Gulfstream G-IV	2	3	Gulfstream G-IV	7
Gulfstream IV	73,200	Gulfstream G-IV	2	3		
Dassault Falcon 900	45,503	Falcon-900	68	83	Falcon-900	83
Bombardier Challenger 600	45,100	Challenger CL-604	58	70	Challenger CL-604	176
Bombardier Challenger 300	38,850	Challenger CL-604	88	106		
Dassault Falcon 2000	41,000	Falcon-2000	34	42	Falcon-2000	42
Dassault Falcon 50	37,480	Falcon-50	276	332	Falcon-50	424
Dassault Falcon 20	28,650	Falcon-50	76	92		
Cessna Citation 750	36,600	Citation X	104	126	Citation X	292

Aircraft Type	Maximum Takeoff Weight, lbs	Design Aircraft for COMFAA	2018 Annual Operations	2040 Annual Operations	Values Entered into COMFAA	
					Equivalent Airplane	Annual # of Departures
Cessna Citation 680	30,775	Citation X	138	167		
Hawker 800	28,000	Hawker-800	34	42	Hawker-800	42
Gulfstream G150	26,100	D-35	80	97	D-35	97
Astra 1125	24,650	D-30	96	117	D-30	117
Cessna Citation 650	22,000	Citation VI/VII	98	119	Citation VI/VII	119
Learjet 60	23,500	Learjet-55	30	36	Learjet-55	57
Learjet 55	21,500	Learjet-55	4	6		
Learjet 75	21,500	Learjet-55	12	15		
Learjet 45	20,500	Learjet-35A/65A	110	133	Learjet-35A/65A	254
Learjet 35	18,000	Learjet-35A/65A	8	10		
Learjet 31	15,500	Learjet-35A/65A	92	111		
Cessna Citation 560	20,000	Citation 550B	704	847	Citation 550B	1,102
Cessna Citation 550	13,300	Citation 550B	212	255		
Phenom 300/ Embraer 300	17,968	D-25	56	68	D-25	68
		Total Operations:	**2,434**			**2,944**

Backcalculation Analysis of FWD Test Data

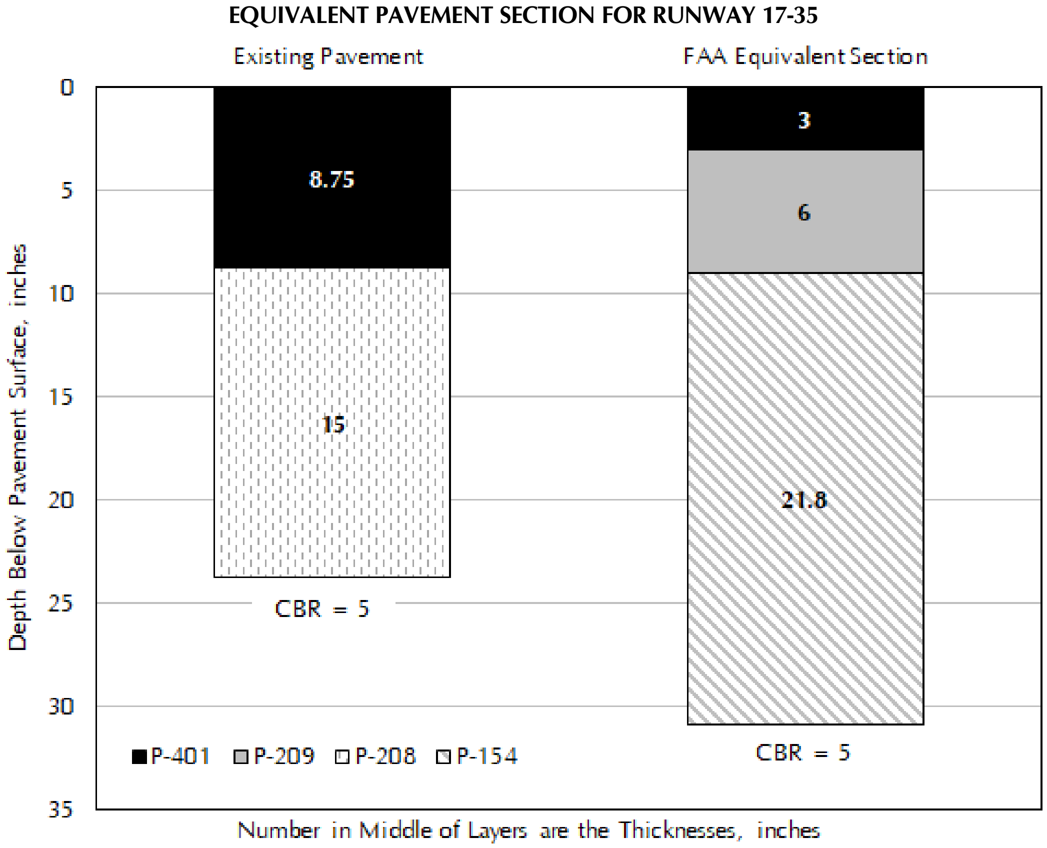
The elastic moduli of the subgrade soil at the boring locations were backcalculated from the FWD test data. The average minus-one standard deviation subgrade moduli for each analysis unit (design modulus) are shown at the bottom of the backcalculation analysis results in Table 2A in Appendix A.

PAVEMENT CLASSIFICATION NUMBER (PCN) CALCULATIONS

As requested by the ODA, we calculated the PCN for Runway 17-35 for each aircraft in the fleet mix based on the critical pavement-layer thickness and subgrade-support characteristics developed herein. The California bearing ratio (CBR) used in the PCN analysis is based on the backcalculated design modulus from Analysis Unit 2 in Table 2A in Appendix A and was calculated using the typical correlation between CBR and Resilient Modulus (M_r) and the correlation adopted by the FAA in Advisory Circular 150/5320-6F, *Airport Pavement Design and Evaluation*, which is represented by the following:

$$CBR = M_r / 1{,}500$$

The analysis was conducted using the FAA's Support Spreadsheet, COMFAA 3.0. The pavement-layer thicknesses were converted into an equivalent pavement section using the appropriate subgrade-support code and the default values for the conversion factors given in Advisory Circular 150/5335-5C. Based on our analysis, the equivalent pavement section is also shown on the following figure.



Results of the PCN computations summarized in Table 3 are based on the departure traffic provided by CWE. For Runway 17-35, we recommend publishing the PCN value shown in Table 3. The corresponding PCN elements of the runway are summarized in Form 5010 (Table 1B) in Appendix B.

Table 3: RECOMMENDED UPDATES TO FAA FORM 5010 FOR UAO RUNWAY 17-35

		Aircraft Gross Weight, thousands lbs	
Runway	**PCN**	**Single Wheel Main Gear**	**Dual Wheel Main Gear**
17-35	40/F/C/X/T	102	145

Our recommended single-wheel, main-gear and dual-wheel, main-gear aircraft gross weights are 102,000 and 143,000 lbs, respectively. The increase in wheel-load capacity (as compared to the current Airport Master Record, FAA Form 5010) is likely due to the increased structural capacity related to the 2005 overlay. Additional discussion regarding the PCN methodology and reporting is provided in Appendix B.

LIMITATIONS

This pavement report has been prepared for use by the Oregon Department of Aviation and Century West Engineering Corporation and should not be relied upon by any other entity without the written permission of an authorized representative. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the analysis of the pavements at the time of publication.

PCN system is only intended as a method that airport operators can use to evaluate acceptable operations of aircraft. It is not intended as a pavement design or pavement evaluation procedure, nor does it restrict or replace the methodology used to design or evaluate a pavement structure.

Our work has been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the locale. The results and conclusions submitted in this report are based on the data obtained from our sources of information discussed in this report. No other warranty, expressed or implied, is made.

Please contact the undersigned if you have any questions regarding this report or any other pavement considerations associated with this project.

Submitted for GRI,

REGISTERED PROFESSIONAL
ENGINEER
15,335
OREGON
MARCH 26, 1991
MICHAEL J. MALONEY

Renews 12/2020

Lindsi Hammond

Michael J. Maloney, PE
Principal

Lindsi A. Hammond, PE
Associate

This document has been submitted electronically.

References

WHPacific, Inc., 2012, Aurora State Airport, Airport Master Plan Update.

Pavement Consultants Inc., 2018, 2018 Pavement Evaluation / Maintenance Management Program: Aurora State Airport.

● PAVEMENT CORE COMPLETED BY GRI
(AUGUST 20, 2019)

— FWD TESTING COMPLETED BY GRI
(AUGUST 20, 2019)

SITE PLAN FROM GOOGLE EARTH (IMAGE DATE JULY 2018)

North

0 600 1,200 FT

GRI

CENTURY WEST ENGINEERING CORPORATION
AURORA STATE AIRPORT – RUNWAY 17-35 PCN EVALUATION

SITE PLAN

SEP. 2019 JOB NO. 6289 FIG. 1

APPENDIX A

Field Explorations and FWD Data

APPENDIX A

FIELD EXPLORATIONS AND FWD DATA

FIELD EXPLORATIONS

Existing pavement and subsurface conditions on Runway 17-35 were investigated by GRI on August 20, 2019, with three core explorations, designated B-1 through B-3. The approximate locations of the explorations are shown on the Site Plan, Figure 1. The field exploration and laboratory programs completed for this project are described below.

Pavement Core Explorations

The pavement was cored at each exploration location to assist in evaluation of the type of cracking and/or the thickness and condition of the asphalt concrete (AC). The pavement was cored using an electric drill owned and operated by GRI. Photographs of the core locations and core samples are shown on Figures 1A through 3A. Below the AC, we excavated to a maximum total depth of 24 in. below ground surface to observe the condition of the aggregate base (AB) and subgrade, if encountered. The subgrade was not encountered during our explorations and the AB was classified as silty sandy gravel ranging from angular to rounded and up to 1 to 1.5 in. in diameter.

FWD DATA

Falling weight deflectometer (FWD) tests were conducted by GRI on August 20, 2019, using our KUAB Model 150 FWD. The annual reference calibration for the FWD was accomplished in October 2019 at the KUAB manufacturing facility in Savoy, Illinois.

The FWD testing on Runway 17-35 was accomplished along test lines located at 7 ft west and 12 ft east of the runway centerline. The tests were completed at approximately 200-ft intervals within the keel section of the runway.

General

Geodetic coordinates of all test locations were measured from GPS signal using a submeter-capable Trimble™ GPS receiver with the antenna mounted on the FWD above the load plate.

The FWD load is generated by a two-mass/two-buffer, falling-weight system that produces a nearly haversine-shaped load-pulse waveform. The buffer and weight combination used for these tests produces a load rise time of approximately 14 milliseconds with an equivalent haversine frequency of approximately 32 Hz. The load pulse was applied to the pavement surface through a 450-mm-diameter (8.86-in.-radius), four-part, segmented plate designed to apply uniform surface pressure distribution despite irregularities in the pavement surface. Air temperature and pavement surface temperature (the latter measured by infrared thermometer) were recorded for each test.

Test Data

The average deflections from the two nominal 32,000-lb impact loads were linearly normalized to a 30-kip (30,000-lb) load basis and are tabulated in Table 1A of this appendix. The measurement units for the test data are distance in feet, deflections in mil units (1 mil = 0.001 in.), load in pounds, sensor distance in inches, load plate radius in inches, and temperature in degrees Fahrenheit.

Impact Stiffness Modulus (ISM)

The Impact Stiffness Modulus (ISM) shown in units of kips per square inch (ksi) is the composite stiffness, or dynamic plate bearing modulus, of all the materials beneath the pavement/roadway surface. It is computed using the Boussinesq formula for surface deflection beneath the center of a uniformly loaded circular area on a linear-elastic half space, with a Poisson's ratio of 0.50. The surface deflection measured at the center of the FWD load plate (D0) was used to compute the surface modulus. The magnitude of the ISM is inversely proportional to deflection and comparable to the elastic modulus. The difference between the pavement ISM and elastic modulus is that the elastic modulus represents the elastic load-deformation response of an individual pavement layer or the subgrade soil, whereas the pavement ISM represents the composite elastic load-deformation response of all materials (pavement layers and subgrade soil) below the pavement surface. Therefore, the ISM (as computed from the deflection measured beneath the FWD load plate) cannot be taken as representative of the elastic modulus of any single pavement layer or the subgrade soil. However, since it is a measurement of the combined stiffness of the pavement structure and subgrade soil, it is often useful for evaluation of variation in pavement stiffness and for assessment of relative pavement strength. Plots of the ISMs are shown on Figure 4A.

Table 1A - FWD NORMALIZED DEFLECTION TEST DATA
RUNWAY 17-35: AURORA STATE AIRPORT (UAO)

Test Section: RW 17-35
Start Point: North edge of runway, 10+00
Test Date: 8/20/2019
Test File: 6289-Aurora Airport.fwd
Load Plate Radius, in: 8.86
Sensor Distance, in: 0 12 18 24 36 48 60 72

Deflections Normalized to 30000 lbf Basis

Test No.	Test Station	Test Line	Core	D 1, mils	D 2, mils	D 3, mils	D 4, mils	D 5, mils	D 6, mils	D 7, mils	D 8, mils	Surface Temp., °F	Time	Surface Modulus, Ksi	ISM, kips/in	Comments
1	10+50	7' w		28.54	24.85	21.17	18.56	13.73	10.05	7.37	5.54	68	1:24:59	57	1,051	7' west
2	12+50	7' w		25.28	20.28	16.82	14.62	10.56	7.81	5.80	4.50	71	1:26:36	64	1,187	
3	14+49	7' w		30.42	25.52	21.55	18.73	13.50	9.84	7.24	5.55	71	1:27:52	53	986	
4	16+51	7' w		29.35	24.82	20.94	18.25	13.29	9.74	7.15	5.47	71	1:29:09	55	1,022	
5	18+50	7' w		24.65	20.46	17.12	14.81	10.62	7.71	5.71	4.47	71	1:30:14	66	1,217	
6	20+56	7' w		27.93	22.60	18.54	15.81	11.05	7.98	5.87	4.66	71	1:31:20	58	1,074	
7	22+50	7' w		25.72	21.22	17.71	15.34	11.10	8.13	6.06	4.70	71	1:32:26	63	1,166	
8	24+51	7' w		26.54	21.58	17.98	15.18	10.67	7.71	5.71	4.47	71	1:33:33	61	1,130	
9	26+53	7' w		26.28	20.74	17.15	14.64	10.47	7.67	5.83	4.64	70	1:34:39	62	1,142	
10	28+55	7' w		26.82	22.10	18.49	15.98	11.58	8.49	6.34	4.95	71	1:35:42	60	1,119	
11	30+54	7' w		26.27	21.60	18.22	15.84	11.70	8.66	6.45	4.96	71	1:37:01	62	1,142	
12	32+54	7' w		30.95	25.88	21.81	19.07	13.97	10.26	7.67	5.78	71	1:38:07	52	969	
13	34+52	7' w		36.96	27.64	22.18	18.81	13.26	9.67	7.12	5.56	71	1:39:22	44	812	
14	36+57	7' w		32.41	26.67	22.42	19.26	13.87	10.02	7.26	5.44	70	1:40:28	50	926	
15	38+52	7' w		28.76	23.55	19.60	16.84	12.06	8.67	6.34	4.88	70	1:41:38	56	1,043	
16	39+51	7' w	B-2	34.09	27.13	22.55	19.48	14.13	10.46	7.65	5.72	70	1:43:21	47	880	B-2
17	40+51	7' w		27.27	22.43	18.67	16.13	11.60	8.44	6.11	4.75	70	1:44:29	59	1,100	
18	42+51	7' w		31.58	25.74	21.56	18.44	13.11	9.35	6.80	5.10	70	1:45:38	51	950	
19	44+51	7' w		29.21	23.02	18.77	15.98	11.24	7.90	5.76	4.52	70	1:46:46	55	1,027	
20	46+50	7' w		29.41	23.54	19.35	16.44	11.40	7.92	5.78	4.50	70	1:47:53	55	1,020	
21	48+52	7' w		28.25	23.01	19.08	16.26	11.38	8.17	6.06	4.66	70	1:49:02	57	1,062	
22	50+52	7' w		39.77	29.04	22.94	19.04	12.53	8.69	6.21	4.86	70	1:50:10	41	754	
23	52+50	7' w		34.37	27.28	22.48	18.86	12.83	8.94	6.47	5.08	70	1:51:20	47	873	
24	54+51	7' w		44.23	34.59	27.53	22.75	14.74	9.70	6.77	5.20	69	1:52:33	37	678	
25	56+40	7' w		37.32	28.83	22.75	18.62	11.88	7.81	5.61	4.42	67	1:53:49	43	804	
26	56+81	7' w	B-1	35.88	28.79	23.20	19.31	12.57	8.38	5.79	4.55	70	1:55:03	45	836	B-1
27	58+50	7' w		35.45	27.78	22.05	18.05	11.74	7.82	5.60	4.34	65	1:56:22	46	846	5875=s end end 7' west
28	11+50	12' e		25.22	21.35	18.22	15.93	11.88	8.90	6.66	5.09	68	2:05:27	64	1,190	12' east
29	13+50	12' e		30.01	25.29	21.29	18.67	13.66	10.11	7.43	5.70	70	2:07:03	54	1,000	
30	15+51	12' e		30.03	25.22	21.26	18.42	13.46	9.89	7.28	5.64	70	2:08:15	54	999	
31	17+53	12' e		28.42	22.94	19.00	16.27	11.53	8.38	6.20	4.83	70	2:09:28	57	1,056	
32	19+41	12' e	B-3	34.02	25.85	20.87	17.26	11.79	8.33	6.13	4.74	70	2:13:56	48	882	B-3
33	21+50	12' e		21.06	17.31	14.42	12.49	9.07	6.79	5.19	4.17	70	2:16:05	77	1,425	
34	23+52	12' e		25.55	21.01	17.53	15.14	11.13	8.27	6.23	4.95	70	2:17:18	63	1,174	
35	25+52	12' e		21.98	17.91	15.02	13.04	9.69	7.31	5.60	4.43	69	2:18:26	74	1,365	
36	27+51	12' e		26.27	20.79	16.87	14.33	10.21	7.48	5.62	4.44	69	2:19:33	62	1,142	
37	29+50	12' e		34.66	28.16	23.24	19.76	13.95	10.10	7.48	5.79	69	2:20:42	47	866	

Table 1A - FWD NORMALIZED DEFLECTION TEST DATA
RUNWAY 17-35: AURORA STATE AIRPORT (UAO)

Deflections Normalized to 30000 lbf Basis

Test No.	Test Station	Test Line	Core	D 1, mils	D 2, mils	D 3, mils	D 4, mils	D 5, mils	D 6, mils	D 7, mils	D 8, mils	Surface Temp., °F	Time	Surface Modulus, Ksi	ISM, kips/in	Comments
38	31+52	12' e		27.24	22.35	18.84	16.39	12.19	9.20	6.99	5.47	69	2:21:52	59	1,101	
39	33+49	12' e		26.34	21.87	18.38	15.90	11.64	8.78	6.71	5.25	69	2:23:00	61	1,139	
40	35+53	12' e		24.64	20.22	16.91	14.67	10.73	8.01	6.08	4.83	69	2:24:09	66	1,218	
41	37+51	12' e		29.65	24.86	20.96	18.32	13.45	9.99	7.38	5.60	69	2:25:16	55	1,012	
42	39+50	12' e		25.27	21.38	17.99	15.86	11.68	8.77	6.56	5.13	69	2:26:26	64	1,187	
43	41+51	12' e		25.80	21.67	18.35	15.90	11.67	8.62	6.43	4.94	69	2:27:34	63	1,163	
44	43+50	12' e		27.58	23.19	19.57	17.18	12.51	9.22	6.76	5.14	69	2:28:38	59	1,088	
45	45+51	12' e		26.22	21.41	17.71	15.13	10.72	7.77	5.72	4.51	69	2:29:48	62	1,144	
46	47+54	12' e		28.02	22.49	18.48	15.60	10.83	7.75	5.68	4.46	69	2:30:56	58	1,071	
47	49+51	12' e		27.34	22.44	18.36	15.67	11.04	7.94	5.90	4.62	69	2:32:04	59	1,097	
48	51+53	12' e		30.35	24.69	20.12	17.00	11.60	8.11	5.96	4.66	69	2:33:11	53	988	
49	53+55	12' e		31.95	26.02	21.17	17.69	11.99	8.46	6.17	4.85	69	2:34:18	51	939	
50	55+50	12' e		36.26	28.03	22.28	18.48	12.16	8.34	6.04	4.75	69	2:35:31	45	827	
51	57+51	12' e		32.67	26.40	21.38	17.62	11.50	7.75	5.50	4.31	67	2:36:47	49	918	5878=s end end 12' east

Table 2A - BACKCALCULATION ANALYSIS SUMMARY
RUNWAY 17-35: AURORA STATE AIRPORT (UAO)

Runway 17-35: Aurora State Airport (UAO)
Based on FWD Testing Conducted: 8/20/2019
Start Station: North edge of runway, 10+00

FWD Test #	Test Station	Test Line	Core Exploration	Analysis Unit	D0, mils	AC Thickness, inches	AB Thickness, inches	Subgrade Modulus, psi
1	10+50	7' w		1	28.54	9.00	15.00	10,402
2	12+50	7' w		1	25.28	9.00	15.00	15,441
3	14+49	7' w		1	30.42	9.00	15.00	11,553
4	16+51	7' w		1	29.35	9.00	15.00	11,570
5	18+50	7' w		1	24.65	9.00	15.00	12,902
6	20+56	7' w		1	27.93	9.00	15.00	11,768
7	22+50	7' w		1	25.72	9.00	15.00	14,630
8	24+51	7' w		1	26.54	9.00	15.00	12,567
9	26+53	7' w		1	26.28	9.00	15.00	15,004
10	28+55	7' w		1	26.82	9.00	15.00	14,486
11	30+54	7' w		1	26.27	9.00	15.00	13,228
12	32+54	7' w		1	30.95	9.00	15.00	10,155
13	34+52	7' w		1	36.96	9.00	15.00	9,847
14	36+57	7' w		1	32.41	9.00	15.00	10,365
15	38+52	7' w		1	28.76	9.00	15.00	10,556
16	39+51	7' w	B-2	1	34.09	9.00	15.00	9,726
17	40+51	7' w		1	27.27	9.00	15.00	10,489
18	42+51	7' w		1	31.58	9.00	15.00	11,108
19	44+51	7' w		1	29.21	9.00	15.00	11,314
20	46+50	7' w		1	29.41	9.00	15.00	11,087
21	48+52	7' w		1	28.25	9.00	15.00	14,129
22	50+52	7' w		2	39.77	8.75	15.00	8,814
23	52+50	7' w		2	34.37	8.75	15.00	9,367
24	54+51	7' w		2	44.23	8.75	15.00	6,713
25	56+40	7' w		2	37.32	8.75	15.00	9,796
26	56+81	7' w	B-1	2	35.88	8.75	15.00	7,615
27	58+50	7' w		2	35.45	8.75	15.00	9,512
28	11+50	12' e		1	25.22	9.00	15.00	12,541
29	13+50	12' e		1	30.01	9.00	15.00	11,399
30	15+51	12' e		1	30.03	9.00	15.00	9,781
31	17+53	12' e		1	28.42	9.00	15.00	11,645
32	19+41	12' e	B-3	1	34.02	9.00	15.00	10,977
33	21+50	12' e		1	21.06	9.00	15.00	17,720
34	23+52	12' e		1	25.55	9.00	15.00	13,364
35	25+52	12' e		1	21.98	9.00	15.00	14,811
36	27+51	12' e		1	26.27	9.00	15.00	14,236
37	29+50	12' e		1	34.66	9.00	15.00	11,837
38	31+52	12' e		1	27.24	9.00	15.00	10,942
39	33+49	12' e		1	26.34	9.00	15.00	11,421
40	35+53	12' e		1	24.64	9.00	15.00	14,477
41	37+51	12' e		1	29.65	9.00	15.00	10,835
42	39+50	12' e		1	25.27	9.00	15.00	11,501
43	41+51	12' e		1	25.80	9.00	15.00	13,236
44	43+50	12' e		1	27.58	9.00	15.00	11,913

Table 2A - BACKCALCULATION ANALYSIS SUMMARY
RUNWAY 17-35: AURORA STATE AIRPORT (UAO)

FWD Test #	Test Station	Test Line	Core Exploration	Analysis Unit	D0, mils	AC Thickness, inches	AB Thickness, inches	Subgrade Modulus, psi
45	45+51	12' e		1	26.22	9.00	15.00	12,250
46	47+54	12' e		1	28.02	9.00	15.00	11,825
47	49+51	12' e		1	27.34	9.00	15.00	12,606
48	51+53	12' e		2	30.35	8.75	15.00	11,238
49	53+55	12' e		2	31.95	8.75	15.00	10,326
50	55+50	12' e		2	36.26	8.75	15.00	9,761
51	57+51	12' e		2	32.67	8.75	15.00	9,341

Statistical Summary

Structural Unit#	From Sta	To Sta	PAVER PMP Unit	Average D0, mils	Average AC Thickness, in.	Average AB Thickness, in.	Average Subgrade Modulus, psi
1	0+00	49+51	R17AU-01	28.10	9.00	15.00	12,235
2	0+00	58+50	R17AU-02	35.83	8.75	15.00	9,248

Design Subgrade Resilient Modulus

Structural Unit #	From	To	PAVER PMP Unit	Average Subgrade Modulus, psi	Standard Deviation, psi	Average Subgrade – Standard Deviation, psi	CBR, Mr (psi)/1500
1	10+50	49+51	R17AU-01	12,235	1,800	10,435	7
2	50+52	58+50	R17AU-02	9,248	1,294	7,955	5

Core B-1 (RW 17-35 8′ West of Centerline, Station 56+81, FWD 26)

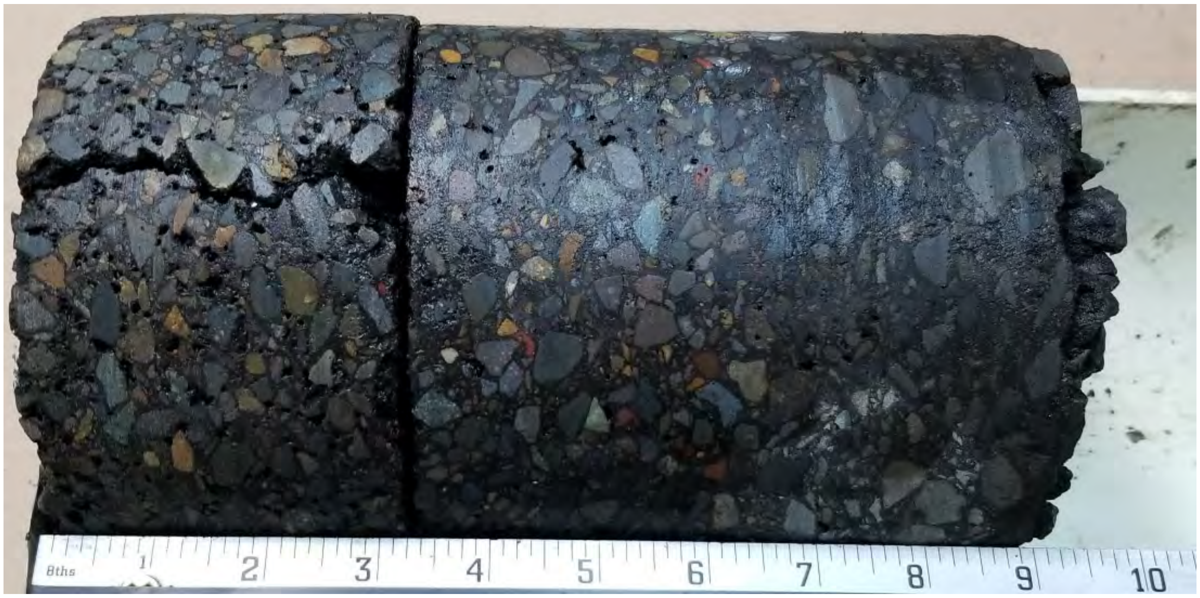
B-1 (Pavement Core Sample, 8.75 in.)

GRI

PAVEMENT CORE PHOTOGRAPHS

SEP. 2019 JOB NO. 6289 FIG. 1A

Core B-2 (RW 17-35 8′ West of Centerline, Station 39+51, FWD 16)



B-2 (Pavement Core Sample, 9.0 in.)

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PAVEMENT CORE PHOTOGRAPHS

SEP. 2019 JOB NO. 6289 FIG. 2A

Core B-3 (RW 17-35 12′ East of Centerline, Station 19+41, FWD 32)

B-3 (Pavement Core Sample, 9.0 in.)

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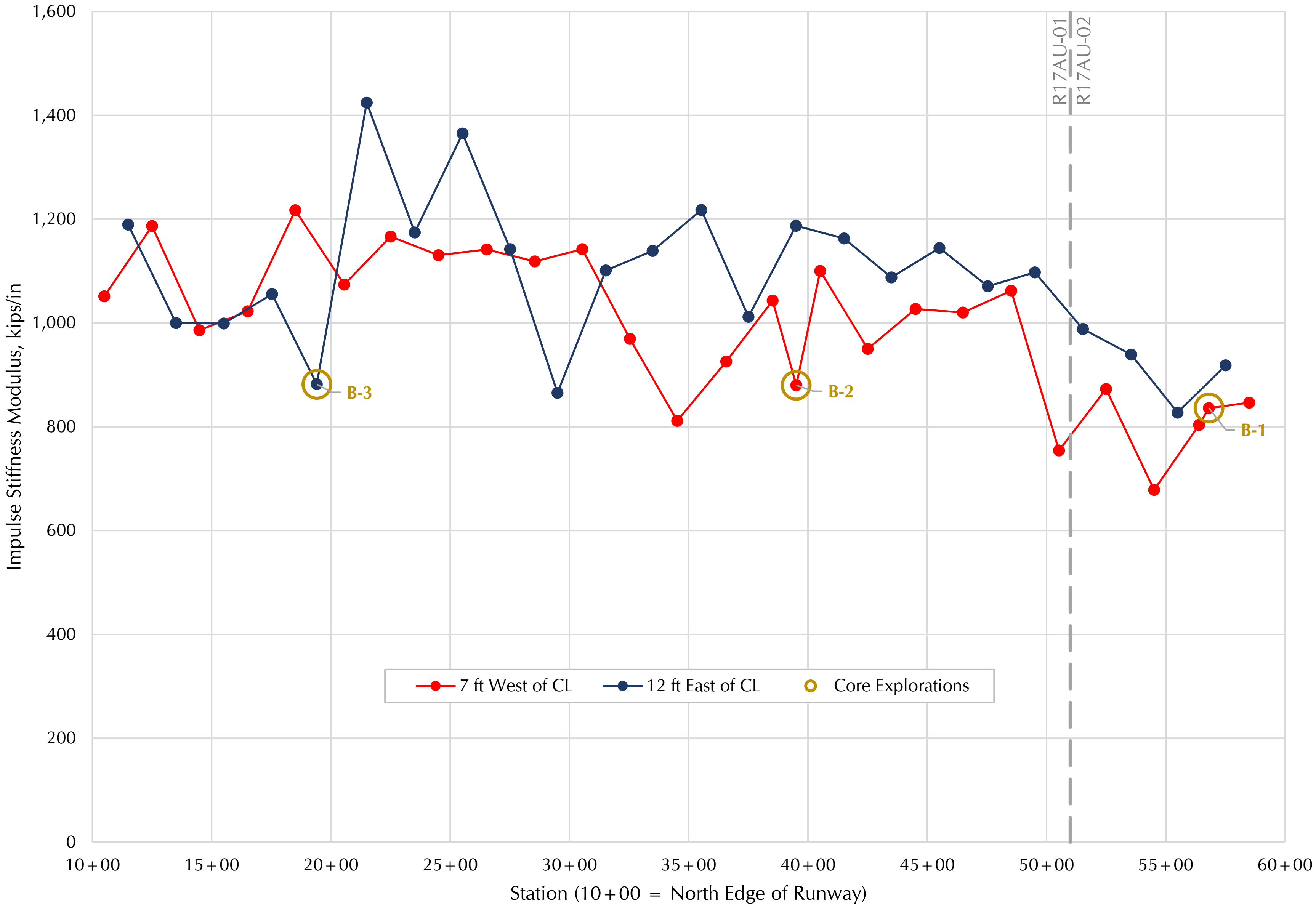
PAVEMENT CORE PHOTOGRAPHS

SEP. 2019 JOB NO. 6289 FIG. 3A



GRI

IMPULSE STIFFNESS MODULUS

SEP. 2019 JOB NO. 6289 FIG. 4A

APPENDIX B

Pavement Classification Number Analysis

APPENDIX B

PAVEMENT CLASSIFICATION NUMBER ANALYSIS

BACKGROUND

In 2014, the FAA instituted a requirement that Part 139-certified airports be assigned pavement classification number (PCN) data. The PCN is required because the United States is a member state of the International Civil Aviation Organization (ICAO), the international regulatory body for air traffic. ICAO adopted the Aircraft Classification Number (ACN)-Pavement Classification Number (ACN-PCN) method to allow any airport a standardized method for reporting the effect of aircraft that use the facility, as well as the load-carrying capacity of the pavement (ICAO, 1999).

The ACN is a number that expresses the relative effect of an aircraft at a given configuration on a pavement structure for a specified standard subgrade strength. Conversely, the PCN is defined as a number that expresses the load-carrying capacity of a pavement for unrestricted operations. Therefore, the ACN-PCN system is structured so that a pavement with a particular PCN value can support unlimited repetitions of an aircraft that has an ACN equal to or less than the pavement's PCN value.

In the ACN/PCN method, the PCN, pavement type, subgrade strength category, tire pressure category, and evaluation method are all reported together. A code system has been implemented to allow an abbreviated presentation of the necessary information. The pavement type is abbreviated "R" for rigid (portland cement concrete [PCC]) and "F" for flexible (AC) pavements. Four subgrade categories, A, B, C, and D, indicate high, medium, low, and ultra-low subgrade strengths, respectively. The four tire-pressure categories, W, X, Y, and Z, indicate high, medium, low, and very low tire pressures, respectively. The evaluation methods are T for a technical evaluation and U for an evaluation based on the type and weight of the aircraft that commonly use the airfield. For example, the PCN code 90/F/C/W/T indicates that the PCN number is 90, that the pavement is flexible, that there is a low-strength subgrade, that high-pressure tires are allowed, and that a technical evaluation was performed to determine the PCN rating.

METHODOLOGY

As noted above, the pavement strength evaluation was accomplished in accordance with the Technical Method described in Advisory Circular 150/5335-5C. To complete the analysis, the following information was used for Runway 17-35:

> **Aircraft Traffic Volume:** The traffic volume estimate was provided by Century West Engineering Corporation in terms of operations for Runway 17-35. The COMFAA 3.0 program includes a library of standard aircraft types, and we used the default gear weight for each aircraft in the aircraft fleet mix.
>
> **Pavement Structure:** As noted earlier herein, the pavement thickness and subgrade support characteristics were estimated based on the FWD backcalculation results and core explorations.

The results of our PCN analysis are summarized in Form 5010 – Airport Master Record (Table 1B) and presented on Figure 1B of this appendix.

Reference

ICAO, 1999, Aerodrome standards – aerodrome design and operations, Annex 14, Third Edition.

Table 1B - FORM 5010 AIRPORT MASTER RECORD

○ A Flexible Category (CBR 15)
○ B Flexible Category (CBR 10)
● C Flexible Category (CBR 6)
○ D Flexible Category (CBR 3)

TIRE PRESSURE

○ W Unlimited
● X 254 psi
○ Y 145 psi
○ Z 73 psi

METHOD USED

○ Using Aircraft
● Technical

Project info

Aurora State Airport

○ A Rigid Category (k 552 pci)
○ B Rigid Category (k 295 pci)
○ C Rigid Category (k 147 pci)
○ D Rigid Category (k 74 pci)

AIRCRAFT GEAR TYPE IN TRAFFIC MIX

☑ S (single wheel gear)
☑ D (dual wheel gear)
☐ 2D (dual tandem wheel gear)
☐ 3D (triple tandem wheel gear) e.g B-777
☐ DDT or W/B (tandem gear under wing AND tandem gear under body) e.g. B-747, A-340-600, A-380

Airport LOC-ID UAO

Pavement ID RW 17-35

Enter PCN 40

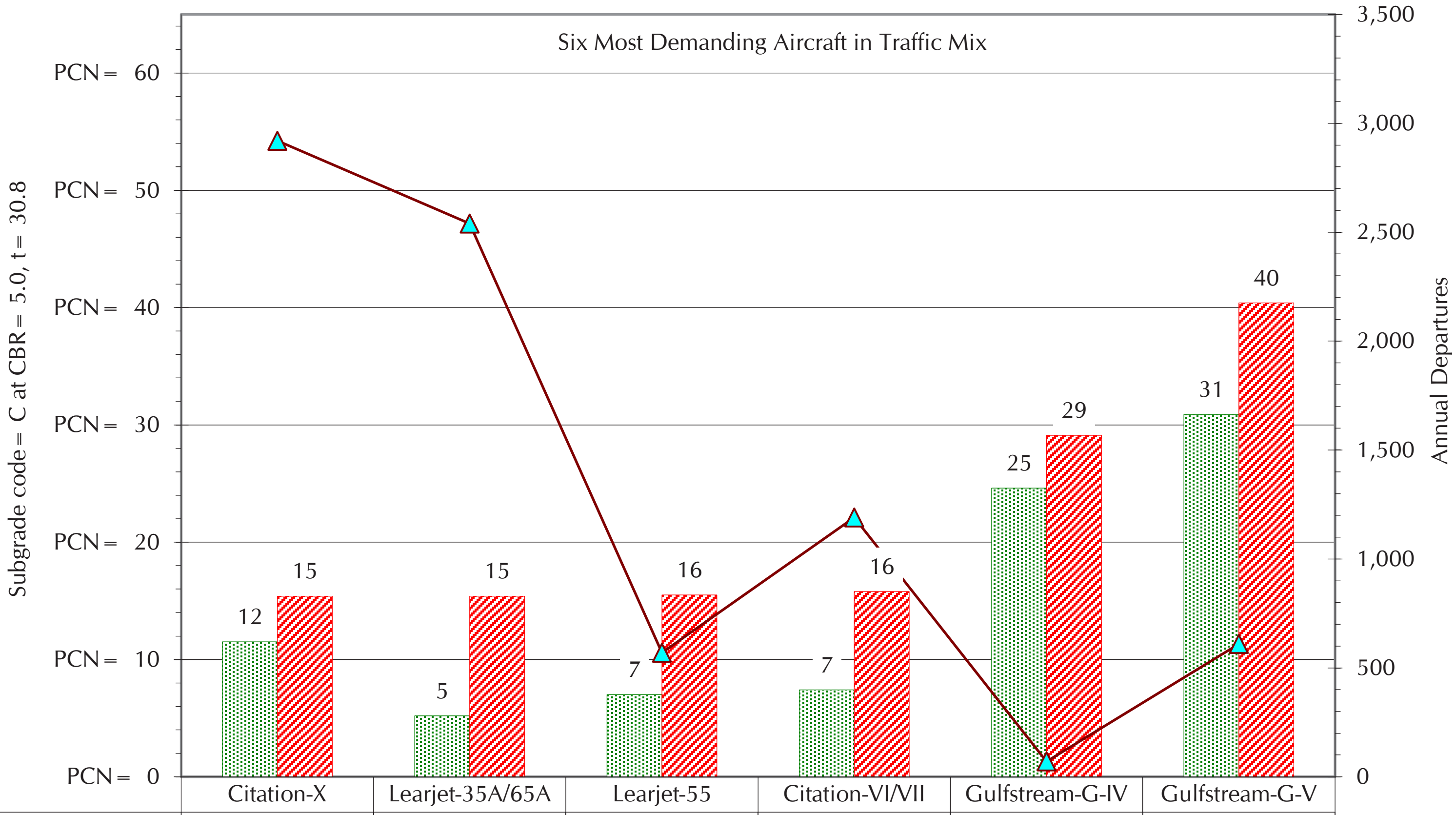
IF 3D or W/B Gear Checked, #38 = PCN
Please Add Data Element #38 Remark

Form 5010 Data Element	Gross Weight and PCN
#35 S gear	102
#36 D gear	143
#37 DT gear	
#38 DDT gear	
#39 PCN	40/F/C/X/T

3D	
2D/2D2	
2D/3D2W	
2D/3D2B	

} **Report Minimum Gross Weight**

Airport LOC-ID	Pavement ID	#35 S GW	#36 D GW	#37 DT GW	#38 DDT GW	#39 PCN
UAO	17-35	102	143			40/F/C/X/T



	Citation-X	Learjet-35A/65A	Learjet-55	Citation-VI/VII	Gulfstream-G-IV	Gulfstream-G-V
🟩 1. Aircraft ACN at traffic mix GW	11.5	5.2	7.0	7.4	24.6	30.9
🟥 2. Calculated PCN at CDF max. GW	15.4	15.4	15.5	15.8	29.1	40.4
▲ 3. Annual Departures from traffic mix	2,920	2,540	570	1,190	70	610

GRI

PAVEMENT CLASSIFICATION CHART

SEP. 2019 JOB NO. 6289 FIG. 1B

Exhibit 1, Attachment 6

HD AVIATION & SOUTHEND AIRPARK

T.4S, R.1W, SEC, 2D & 11A, T.L. 200, 203, 400, 401, 1600
14401 KEIL ROAD N.E.
AURORA, OREGON 97002

PLAN APPROVED

Date 6/22/05 Signed
* with changes noted herein

REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
STATE OF OREGON
Robert F. Sweeney
#0206202513
Expires 30 Sep 05

INSTALLER: TBA

LICENSE #: TBA

Table of Contents

Project Description

Proposed expansion of existing wastewater treatment facility for a regional airport.
System design = 10,000 GPD.
Residential strength waste flows to eight new Septic Tanks and three new 2-compartment Septic/Dosing Tanks. Effluent flows by gravity from the Septic Tanks to five new Dosing Tanks. Three existing Dosing Septic Tanks will be converted to Sewage Lift Stations, pumping to a new 3000-gallon Co-mingle Tank with effluent filter and then to a 3000-gallon Dosing Tank. Accumulated sludges to be removed by a licensed Sewage Disposal Service.

Effluent is pumped from the Dosing Tanks and Septic/Dosing Tanks to a new 3000-gallon Co-mingle Tank and new 3000-gallon Recirculation Tank that is to be intertied to existing 2x3000-gallon Recirculation Tanks. The Recirculation Tanks will dose two AX-100 Recirculating Textile Filters.

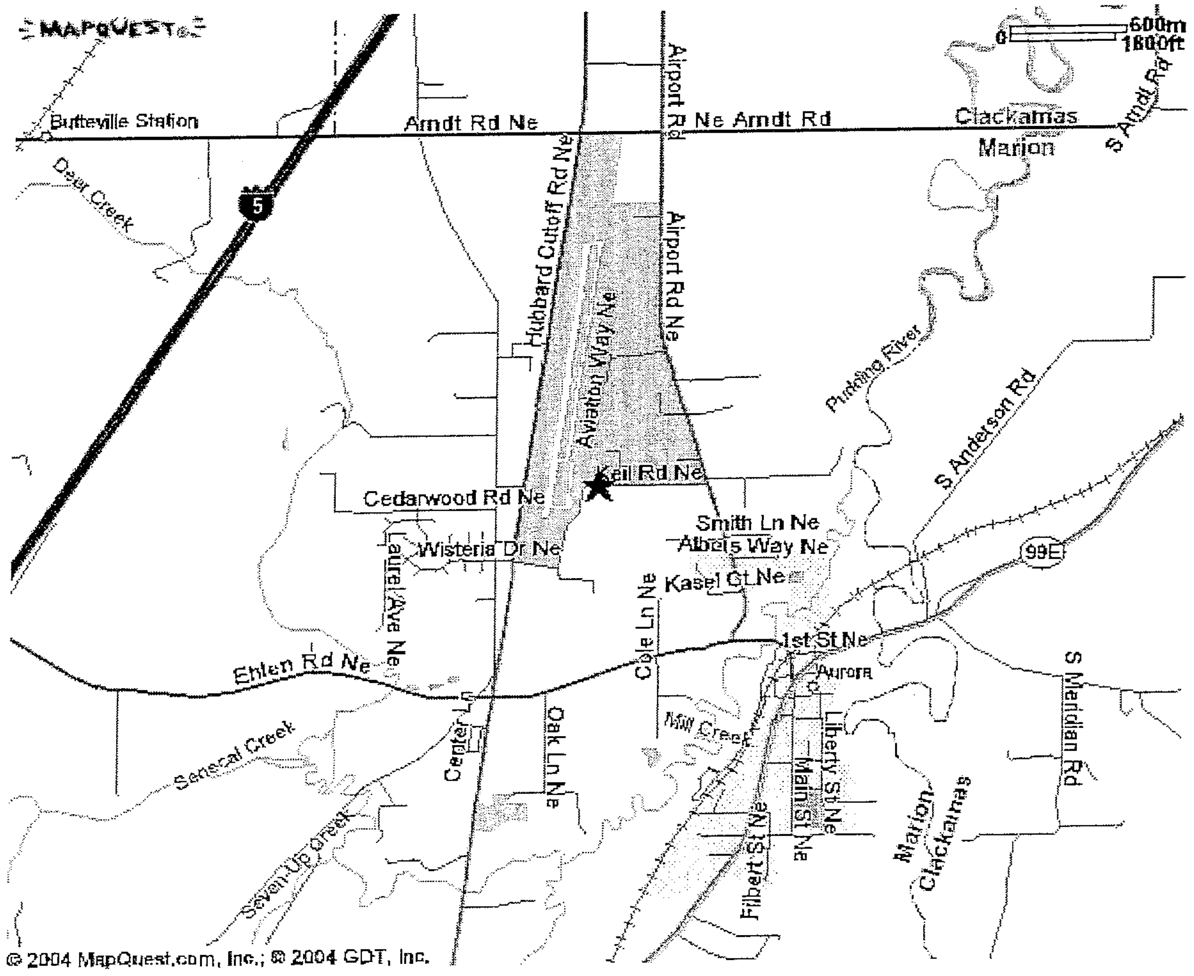
Final disposal via an existing Dosing Tank will be retrofitted with new pumping systems. The existing drainfield size to be doubled to 3000 lft by using the previously identified reserve area. Existing drainfield laterals to be removed and replaced. New Distribution System is detailed in this design. New reserve area will be located west of present drainfield.

Existing Recirculating Gravel Filter to be disassembled and removed. Used gravel media will be deposited in empty NW corner of Tax Lot 400.
Site is served by a private water well.

Site and Soils (Profile Details, Pg. 4)

Amity Silt Loam
Slope 0 – 2%
Reference: Existing File / Permit No.: DEQ110707

VICINITY MAP



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PAGE 1
11 JAN 2005
SCALE: NONE

H.D. AVIATION & SOUTHEND AIRPARK
T.4S, R.1W, SEC. 2D & 11A, T.L. 200, 203, 400, 401, 1600
COVER SHEET

CHECKED BY:
DRAWN BY:

Environmental Management Systems Inc.
4080 SE International Way, Ste.B112
Milwaukie, OREGON 97222
OR (503)353-9691 WA (360)735-1109
FAX (503)353-9695

CHECK OFF	
	GENERAL STANDARDS
☐	All work and material shall conform with OAR 340 Div. 71 & 73 approved design permit, and appropriate laws. Permits relating (but not limited) to plumbing, electrical, and grading must be coordinated with the on-site system installer and designer.
☐	Minor modifications to accommodate stumps, boulders or other unforeseen obstacles may be needed. Major modification cannot be performed without re-design and regulatory approval.
☐	If the installation contractor (installer) notes any conflicts with applicable State and/or local laws, rules or requirements, he shall request a clarification before ordering or installing affected materials or work. This may include and is not limited to such factors as: land-use regulations, grading ordinances, erosion control districts, hauling limits, typographical errors, etc.
☐	Installer is to obtain copies of all necessary permits, authorizations, licenses etc. prior to initiating construction, including that specialty work designated to a subcontractor which is directly or indirectly related to the system construction.
☐	Installer shall request and obtain utility locates by a qualified service for all potential underground utilities before excavation work commences.
☐	Installer shall maintain any and all survey monuments, which are affected by work and activities, related to the projects. Monuments, if damaged by the installer, shall be reset by a licensed surveyor at the installer's expense.
☐	All materials and equipment shall be of the type, model and brand listed for the manufacturers specified, unless otherwise authorized by the system designer. Substitution of materials and equipment shall receive pre-authorization and the contractor/installer will be responsible for providing performance and operating data.
☐	Installer shall prepare, maintain and provide, at completion of the project, drawings detailing the construction "as-built" and shall provide the owner & designer with the manufacturer's current specification and operating data on all equipment installed prior to final payment to the installer.
	TANK (S)
☐	Grout: Grout watertight using hydraulic-adhesive type cement or grout material. Grout interior and exterior.
☐	Seal all joints and seams with manufacturer-approved sealants.
☐	Odor proof: Seal riser lid to contact with closed cell plastic foam sheet, or single-side adhesive neoprene foam tape.
☐	All tanks must be Traffic Rated.
☐	Tanks must be fitted with manhole covers in steel rings set in pavement minimum 2" above the tank risers.
☐	Riser: Tank must be equipped with a watertight riser, with minimum 18" inside diameter, with tank access brought to or above finish grade. Riser seams must be grouted interior and exterior.
☐	Knockouts: Perforations and unused knockouts must be grouted.
☐	Watertight: Tank must be subject to overnight test for watertightness prior to calling for inspection. Fill to a maximum 2" into riser. Mark water level, initials, time and date.
	PUMP (S)
☐	Air-lock hole: Install a 5/32" diameter hole in discharge pipe below off level and below check level.
☐	Disconnect: Provide a quick disconnect of non-corrosive material within 12" of riser top. Position to allow for removal of pump and pump screen for annual maintenance.
☐	Isolate valve: Provide a gate or ball valve within 12" of riser top, on discharge side of disconnect. Position to allow for removal of pump and pump screen for annual maintenance.
☐	Flush transport pipe and check for equal distribution from splitters, valves, and/or distribution box.
☐	Float control assembly: Float controls must be connected to a separate stand pipe, not discharge line, which is serviceable and accessible.
☐	Pump screen: Provide a corrosion-resistant screen with minimum twelve sq. ft. surface area, with maximum 1/8" openings, surrounding pump extending above maximum effluent level.
☐	Trace wire: Provide an electrically continuous 18 gauge, green-jacketed copper wire in trench for the full length of all transport lines, accessible at the source end.
	ELECTRICAL COMPONENTS
☐	Wiring of pumps and controls shall be performed by a licensed electrician under the auspices of a permit secured from the local jurisdiction. For details of electrical system, pump controls, floats, and the level of the float settings see the manufacturer's instructions and approved design.
☐	Splicing of wires at the splice box inside the tank risers shall be done using the heat shrink connectors provided by the manufacturer or with an approved watertight electrical connector nut.
☐	Wiring from the splice box to the source or the control panel shall be protected in UL approved PVC conduit, constructed watertight. Pump line voltage shall have water proof insulation such as THW, THWN, or HHW. Wire for all connections shall be 14 gauge wire or larger.
☐	"Seal offs" shall be installed between the splice box and the power source or control panel, either in the horizontal just outside the riser or in the vertical just below the control panel or per connection. "Seal offs" shall be installed to manufacturer's specifications and shall be equal to or better than the following: Appleton EYF seal off box, PVC terminal adapters (threaded), Killark sealing compound, Killark packing fiber.
☐	Wiring shall be color coded or numbered and the schedule written inside the control panel or on the wiring diagram.
☐	Upon completion, the apparatus shall be tested for operation and correctness. Available voltage, pump run voltage and pump run amperage shall be measured and recorded inside the control panel.
☐	The wiring diagram shall be retained on site (preferably in control panel enclosure) and any as-built notes or comments entered, initialed, and dated.
	CONTROL PANEL (S)
☐	The electrician shall label the control panel or electrical panel with his business name and the permit number and date of installation.
☐	Control panel shall be installed per manufacturer's instructions; alarm shall be audible from the living/working space. Pump and alarm must be on separate circuits. Location of panels to be based on electrical access.
☐	The control panel for all pumps must have the capability to record the number of alarms, pump events and override events, if applicable.
☐	Use a padlock or other locking device to prevent unauthorized access to the control panel. Panel to be installed on 4" X 4" post, <u>NOT</u> on wall.
☐	Panel shall be in accordance with NEMA 4X rating. Panel enclosure shall meet NEMA 4X requirements.
	OTHER
☐	Setbacks: Maintain required setbacks.
	COLLECTION SYSTEM
☐	Plumbing permit required
	DISTRIBUTION AND TRANSPORT LINES
☐	Pressure piping: Must meet or exceed Class 200 PVC, (ASTM 2241), or class 160 for pipes greater than an inch in diameter.
☐	Road crossing: Sleeve transport pipe in Sch. 40 PVC, installed a minimum of 18" below grade, and bedded in ¾ minus to the surface.
☐	All work and materials shall conform with Chapter 246-272 WAC, approved design permit, and appropriate laws. Permits relating (but not limited) to plumbing, electrical, and grading must be coordinated with the on-site system installer and designer.

Robert F. Sweeney
#0206202513
Expires 30 Sep 05

Environmental
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4080 SE International Way, Ste.B112
Milwaukie, OREGON 97222
OR (503)353-9691 WA (360)735-1109
FAX (503)353-9695

CHECKED BY:
DRAWN BY:

H.D. AVIATION & SOUTHEND AIRPARK
T.4S, R.1W, SEC. 2D & 11A, T.L. 200, 203, 400, 401, 1600
CONSTRUCTION SPECIFICATIONS

PAGE 2
11 JAN 2005
SCALE: NONE

Table 1: Maximum Daily Design Flow

	Source					Gallons Per Day
HD AVIATION	Existing Building 1	55	occupants @	15	gpd	825
	Existing Building 2	63	occupants @	15	gpd	945
	Existing Building 3	25	occupants @	15	gpd	375
	Future Building 4	6	occupants @	15	gpd	90
	Future Building 5	5	occupants @	15	gpd	75
	Future Building 6	7	occupants @	15	gpd	105
	Future Building 7	11	occupants @	15	gpd	165
	Future Building 8	10	occupants @	15	gpd	150
	Future Building 9	6	occupants @	15	gpd	90
SOUTHEND AIRPARK	Future Building A	32	occupants @	15	gpd	480
		7	occupants @	75	gpd	525
	Existing Building B	3	occupants @	15	gpd	45
	Existing Building C	15	occupants @	15	gpd	225
	Existing Building D	9	occupants @	15	gpd	135
		6	occupants @	75	gpd	450
	Future Building E	3	occupants @	15	gpd	45
	Existing Building F	3	occupants @	15	gpd	45
	Future Building G	12	occupants @	15	gpd	180
	Future Building H	12	occupants @	15	gpd	180
	Existing Building I	21	occupants @	15	gpd	315
	Existing Building J	13	occupants @	15	gpd	195
	Future Building K	33	occupants @	15	gpd	495
	Future Building L	34	occupants @	15	gpd	510
		7	occupants @	75	gpd	525
	Future Building M	7	occupants @	15	gpd	105
	Future Building N	15	occupants @	15	gpd	225
	Projected Peak Flow					**7500**
	Design Flow Max					**10000**

Table 2: Capacity for Existing Drainfield

Effluent Quality	Gallons Per Day	Loading Rate
Advantex Effluent	5000	45 lft / 150 gpd

Table 3: Capacity for New Drainfield

Effluent Quality	Gallons Per Day	Loading Rate
Advantex Effluent	5000	45 lft / 150 gpd

Table 4: Effluent Quality Expectations

Parameter	Not to Exceed
BOD	20 mg/L
TSS	20 mg/L
TN	30 mg/L

System Components-HD Aviation

	Tank #	*Volume (gal)*	*Dose Schedule*	*Pump Type*	*# of Pumps*	*Pump Model*	*Pump Size*	*Gal/min*
Pump Stations	S1	3000	Timed	Solids Handling	2	PSE4011	4/10 hp	10
	S2	3000	Timed	Solids Handling	2	PSE4011	4/10 hp	10
	S3	1500	Demand	Solids Handling	2	PSE4011	4/10 hp	10
Septic Tanks	S5	1000	na	na	na	na	na	na
	S6	3000	na	na	na	na	na	na
	S10	3000	na	na	na	na	na	na
	S12	1500	na	na	na	na	na	na
	S13	1500	na	na	na	na	na	na
	S14	1000	na	na	na	na	na	na
	S15	1000	na	na	na	na	na	na
	S16	1000	na	na	na	na	na	na
Septic/Dosing Tanks	S4	1500	Demand	Turbine	2	P100511	1/2 hp	10
	S7	1500	Demand	Turbine	2	P100511	1/2 hp	10
	S8	1500	Demand	Submersible	1	Myers	1/2 hp	10
	S9	1500	Demand	Turbine	2	P100511	1/2 hp	10
	S11	1500	Demand	Turbine	2	P100511	1/2 hp	10
Dosing Tanks	D1	2000	Timed	Turbine	2	P100511	1/2 hp	10
	D2	1000	Timed	Turbine	2	P100511	1/2 hp	10
	D3	1500	Timed	Turbine	2	P100511	1/2 hp	10
	D4	1000	Timed	Turbine	2	P100511	1/2 hp	10
	D5	1000	Timed	Turbine	2	P100511	1/2 hp	10
	D6	1000	Timed	Turbine	2	P100511	1/2 hp	10
	D7	3000	Timed	Turbine	2	P501512	1-1/2 hp	50
Recirculation Tanks	R3	3000	Timed	Turbine	2	P500712	3/4 hp **	50

Primary Septic Tankage Volume = 25,500 gallons
Total Tankage Volume = 47,500 gallons

*** Pumps to be upgraded to size and model determined and verified by RTF Manufacturer (Orenco).*

Recirculating Textile Filter

Max. Design Capacity = 10,000gpd
Projected Peak Flow = 7500 gpd
Projected Average Daily Flow = 3750 gpd
Pod Surface Area = 100 sqft.
Number of pods = 2
Recirculation Rate = 4 to 1
Acutal Flow (w/ Recirc.) = 15000 gpd
Design Hydraulic Loading = 50 gal/sqft/day
Design Recirculation Loading = 250 gal/sqft/day

Distribution Components

Quantity	*Component*
1	OSI-Hydrotek V6602 Automatic Distribution Valve (RTF Pod Distribution)
1	OSI-Hydrotek V6606 Automatic Distribution Valve (Drainfield Distribution)

Pressure Distribution Drainfield

Max. Design Capacity = 10,000gpd
Projected Peak Flow = 7500 gpd
Projected Average flow = 3750 gpd
Design Hydraulic Loading Rate = 45 Lft./ 150gpd

REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
STATE OF OREGON
R Sweeney
Robert F. Sweeney
#0206202513
Expires 30Sep05

Environmental Management Systems Inc.
4080 SE International Way, Ste. B112
Milwaukie, OREGON 97222
OR (503)353-9691 WA (360)735-1109
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CHECKED BY:
DRAWN BY:

H.D. AVIATION & SOUTHEND AIRPARK

T.4S, R.1W, SEC. 2D & 11A, T.L. 200, 203, 400, 401, 1600

DESIGN DATA TABLE

PAGE 3

11 JAN 2005

SCALE: NONE

ENVIRONMENTAL MANAGEMENT SYSTEMS Inc. - ON-SITE SEWAGE TREATMENT SITE EVALUATION DATA

4080 SE International Way, Suite #B-112, Milwaukie, OR 97222-8867 (503) 353-9691 fax: (503) 353-9695 (Vancouver, WA 360-735-1109)

APPLICANT: HD AVIATION
PREVIOUS EVALUATIONS: Y / N | ID#:
ADDRESS OF SITE: 1400 KEIL RD, AURORA, OR 97002
Sub Division:
LEGAL DESCRIPTION: TOWNSHIP: 2S RANGE: 1W SECTION: 2, 11 PARCEL/TAX LOT: 800
other:

SOIL PROFILES: DRAINFIELD AREA - EAST SET OF HOLES

			Redoximorphic Features						
TEST HOLE	DEPTH (IN)	TEXTURE	MATRIX COLOR	Oxidized COLOR	Reduced Color	Coarse Fragments	ROOTS	STRUCTURE & Other Comments:	APPL. RATE w/ Pretreatment
#1 NE	0" - 8"	gSiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	8" - 16"	SiCL	7.5YR 3/2	--	--	--	cf	Faint Mottles / sbk	45 LFt/150g
	16" - 36"	SiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles / abk	50 LFt/150g
Slope= 2% E	36" - 64"	SiCL	7.5YR 2.5/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
#2 NW	0" - 13"	gSiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	13" - 18"	SiCL	7.5YR 3/2	--	--	--	cf	Faint Mottles / sbk	45 LFt/150g
	18" - 40"	gSiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles / abk	50 LFt/150g
Slope= 2% E	40" - 76"	SiCL	7.5YR 2.5/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
#3 SE	0" - 10"	gSiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	10" - 30"	gSiCL	7.5YR 3/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles / abk	50 LFt/150g
	30" - 62"	SiCL	7.5YR 2.5/1	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
Slope= 2% E	62" - 66"	SiCL	7.5YR 3/2	--	--	--	--	Massive	75 LFt/150g
#4 SW	0" - 14"	gSiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	14" - 30"	gSiCL	7.5YR 3/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles / abk	50 LFt/150g
	30" - 48"	SiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
Slope= 2% E	48" - 66"	SiCL	7.5YR 3/2	--	--	--	--	Massive	75 LFt/150g

WEATHER CONDITIONS: (X) CLEAR ( ) CLOUDY ( ) PRECIPITATION ( ) EXTREME DRYNESS ( ) OTHER:
LANDFORMS: ( ) FLOODPLAIN (X) TERRACE (X) UPLAND ( ) SIDESLOPE ( ) OPEN ( ) OTHER:
VEGETATION: (X) PASTURED ( ) FORESTED (X) CLEARED ( ) WETLAND ( ) OTHER:
EFFECTIVE SOIL DEPTH ~ 30-48" OTHER: SURFACE LAYERS INCLUDE OLD FILL FROM AIRSTRIP CONSTRUCTION.
MAJOR LIMITING FACTOR(S): (X) WATER TABLE / (X) SOIL SUITABILITY / ( ) SET BACKS / ( ) LOT SIZE
Soil Survey Mapping: Marion County Sheet #6= Amity Silt Loam
SOIL TYPE: **SiL / SiCL** PROPOSED SYSTEM: **RGF to Pressure Drainfield**
ADDITIONAL COMMENTS: **Soil Pits Observed with Randy Trox, RS of DEQ-Western Region.**
EVALUATOR'S NAME: ROBERT F. SWEENEY, RS
EVALUATORS SIGNATURE: R Sweeney
DATE: 14 July 1999

ENVIRONMENTAL MANAGEMENT SYSTEMS Inc. - ON-SITE SEWAGE TREATMENT SITE EVALUATION DATA

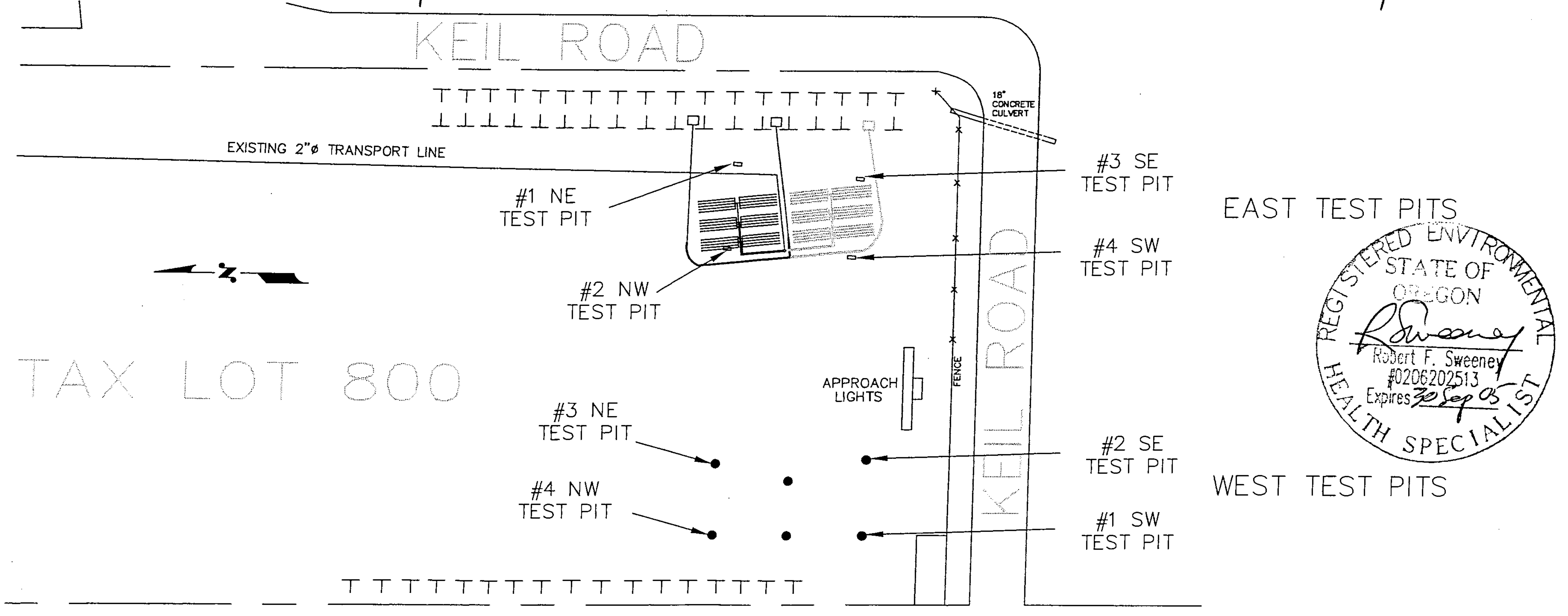
4080 SE International Way, Suite #B-112, Milwaukie, OR 97222-8867 (503) 353-9691 fax: (503) 353-9695 (Vancouver, WA 360-735-1109)

APPLICANT: HD AVIATION
PREVIOUS EVALUATIONS: Y / N | ID#:
ADDRESS OF SITE: 1400 KEIL RD, AURORA, OR 97002
Sub Division:
LEGAL DESCRIPTION: TOWNSHIP: 2S RANGE: 1W SECTION: 2, 11 PARCEL/TAX LOT: 800
other:

SOIL PROFILES: DRAINFIELD AREA - WEST SET OF HOLES

			Redoximorphic Features						
TEST HOLE	DEPTH (IN)	TEXTURE	MATRIX COLOR	Oxidized COLOR	Reduced Color	Coarse Fragments	ROOTS	STRUCTURE & Other Comments:	APPL. RATE w/ Pretreatment
#1 SW	0" - 10"	SiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	10" - 21"	SiL	7.5YR 3/2	--	--	--	cf	abk	45 LFt/150g
	21" - 25"	gSiL	7.5YR 3/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles	50 LFt/150g
Slope= 1% W	25" - 60"	SiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
#2 SE	0" - 15"	SiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	15" - 19"	gSiL	7.5YR 3/2	--	--	--	cf	sbk	45 LFt/150g
	19" - 28"	SiCL	7.5YR 3/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles	50 LFt/150g
Slope= 1% W	28" - 60"	SiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
#3 NE	0" - 15"	SiL	7.5YR 3/3	--	--	--	mf	sbk	45 LFt/150g
	15" - 20"	gSiL	7.5YR 3/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles / abk	50 LFt/150g
	20" - 60"	SiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
Slope= 1% W									
#4 NW	0" - 16"	SiL	7.5YR 3/3	--	--	--	mf	abk	45 LFt/150g
	16" - 22"	gSiL	7.5YR 3/2	7.5YR 4/6	7.5YR 3/2	--	ff	Distinct Mottles / abk	50 LFt/150g
	22" - 60"	SiCL	7.5YR 4/2	7.5YR 4/6	7.5YR 3/2	--	vff	Somewhat Massive	50 LFt/150g
Slope= 1% W									

WEATHER CONDITIONS: (X) CLEAR ( ) CLOUDY ( ) PRECIPITATION ( ) EXTREME DRYNESS ( ) OTHER:
LANDFORMS: ( ) FLOODPLAIN (X) TERRACE (X) UPLAND ( ) SIDESLOPE ( ) OPEN ( ) OTHER:
VEGETATION: (X) PASTURED ( ) FORESTED (X) CLEARED ( ) WETLAND ( ) OTHER:
EFFECTIVE SOIL DEPTH ~ 20-25" OTHER: SURFACE LAYERS INCLUDE OLD FILL FROM AIRSTRIP CONSTRUCTION.
MAJOR LIMITING FACTOR(S): (X) WATER TABLE / (X) SOIL SUITABILITY / ( ) SET BACKS / ( ) LOT SIZE
Soil Survey Mapping: Marion County Sheet #6= Amity Silt Loam
SOIL TYPE: **SiL / SiCL** PROPOSED SYSTEM: **RGF to Pressure Drainfield**
Center Test Pits Similar to Corners w/ Redox Features starting at >15 inches below ground surface
ADDITIONAL COMMENTS: **Soil Pits Observed with Randy Trox, RS of DEQ-Western Region.**
EVALUATOR'S NAME: ROBERT F. SWEENEY, RS
EVALUATORS SIGNATURE: R Sweeney
DATE: 14 July 1999



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Expires 30 Sep 05

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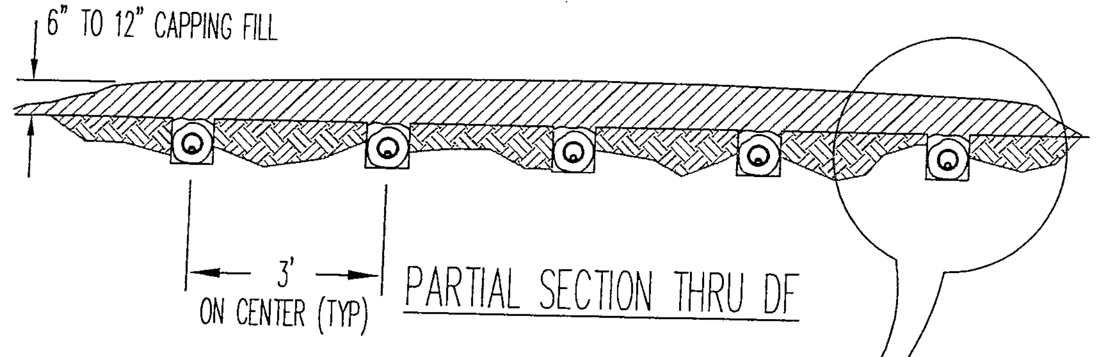
CHECKED BY:
DRAWN BY:

H.D. AVIATION & SOUTHEND AIRPARK

T.4S, R.1W, SEC. 2D & 11A, T.L. 200, 203, 400, 401, 1600

SOIL EVALUATIONS

PAGE 4
11 JAN 2005
SCALE: NONE



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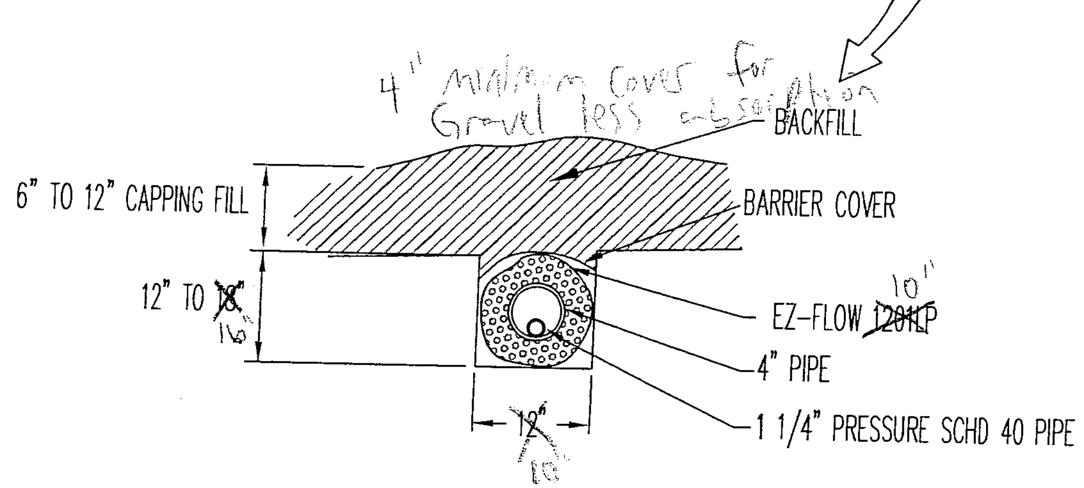
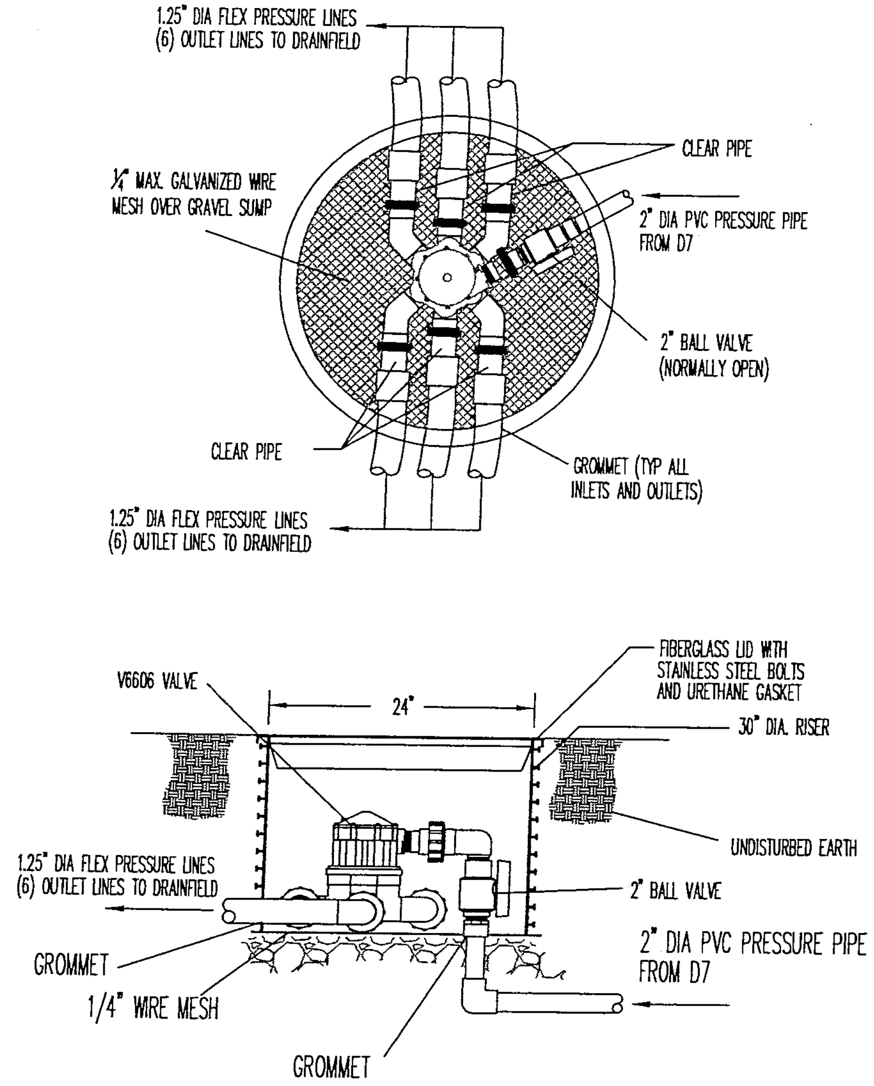
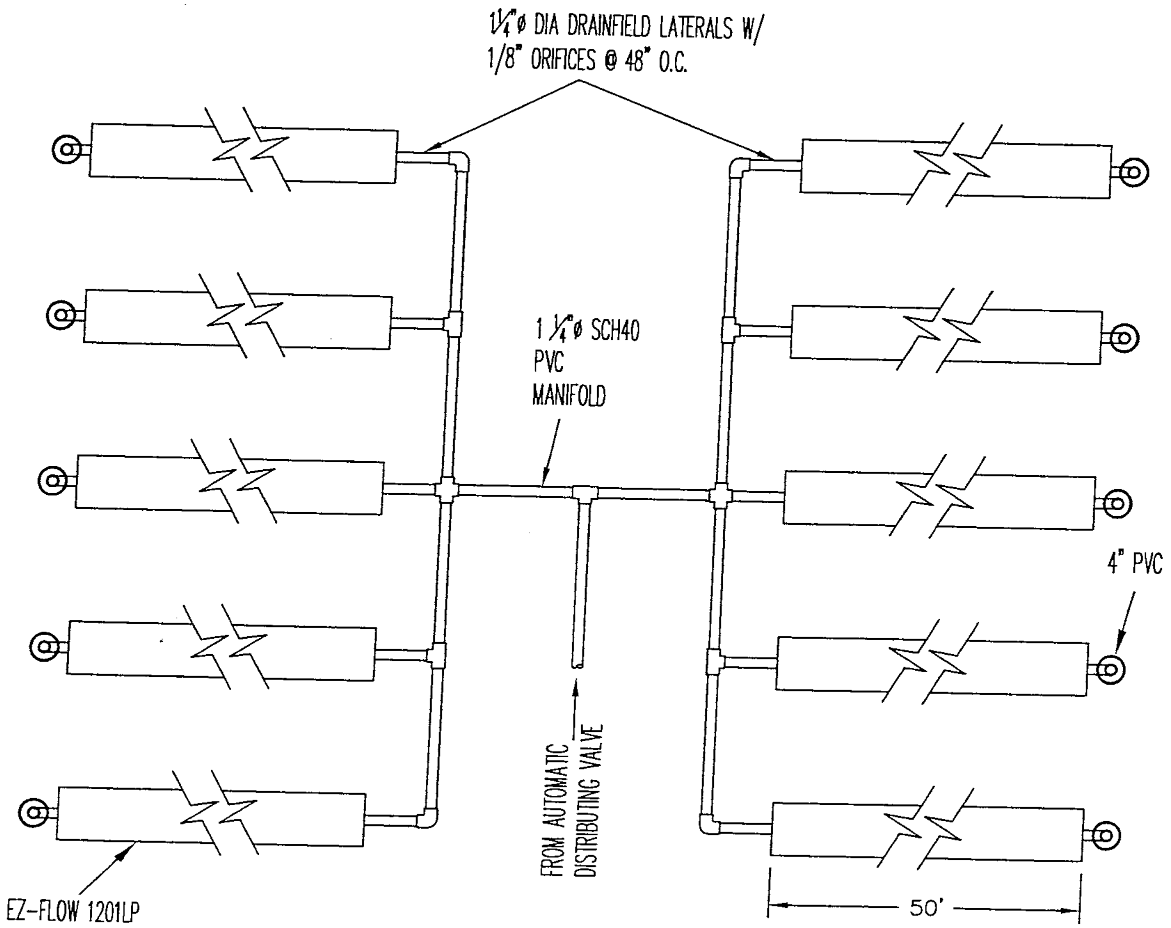


4" minimum cover for Gravel less absorption



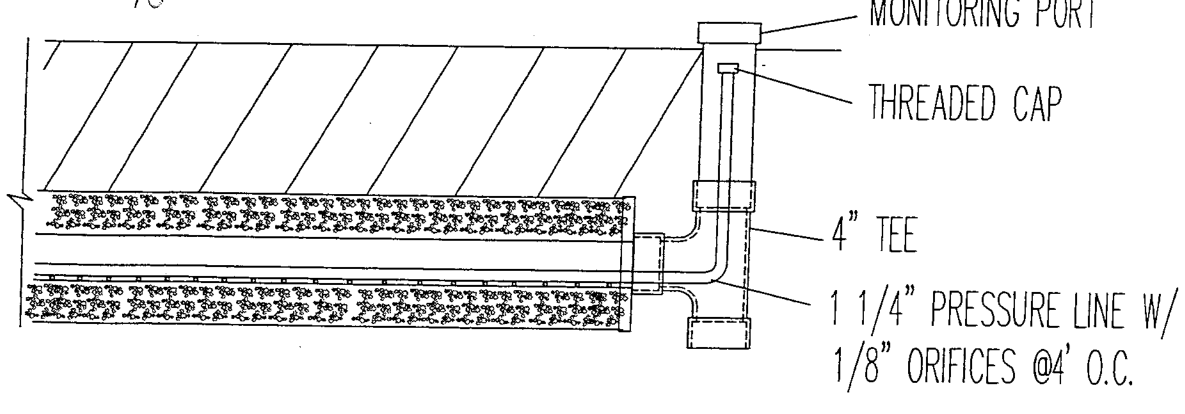
V6606 DISTRIBUTING VALVE

Robert F. Sweeney
020G202513
Expires 30Sep05



DRAINFIELD CELL

~~12"~~ 10" EZFLOW W/ 4" DISTRIBUTION PIPE, HOLES DOWN



MONITORING PORT @ END OF EACH LINE

Environmental
Management
Systems Inc.
4080 SE International Way, Ste. B112
Milwaukie, OREGON 97222
OR (503)353-9691 WA (360)735-1109
FAX (503)353-9695

CHECKED BY:
DRAWN BY:

H.D. AVIATION & SOUTHEND AIRPARK

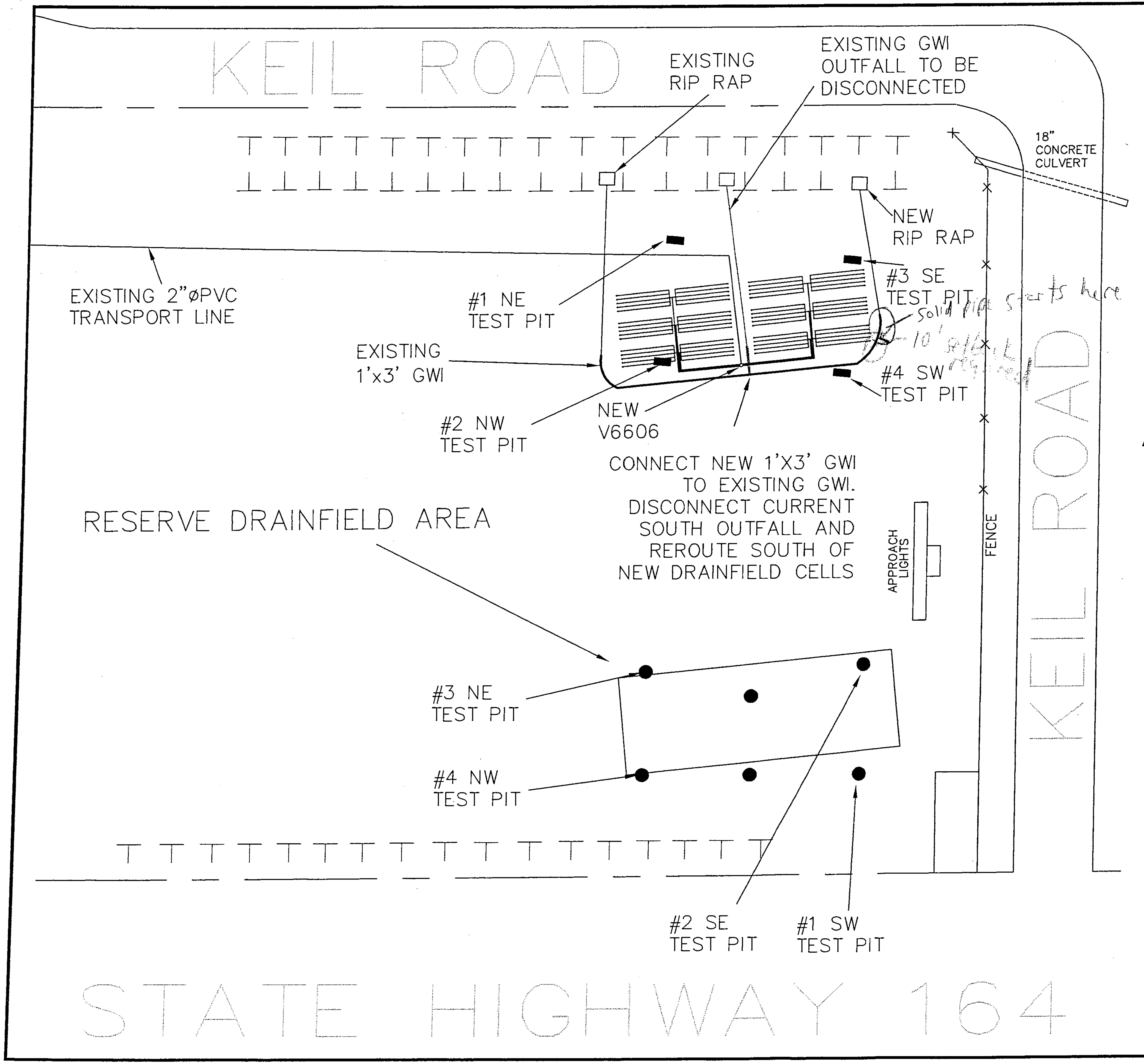
T.4S, R.1W, SEC. 2D & 11A, T.L. 200, 203, 400, 401, 1600

DRAINFIELD DETAILS-1

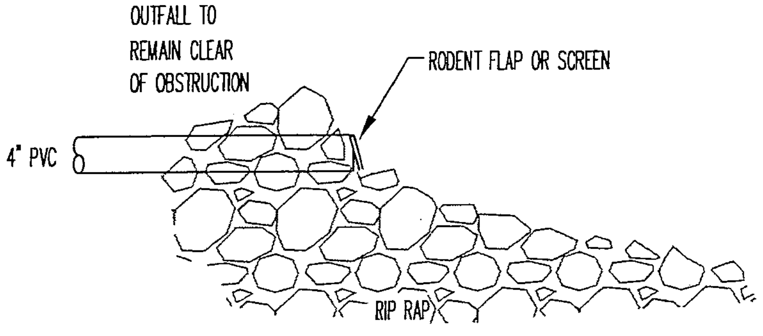
PAGE 23

11 JAN 2005

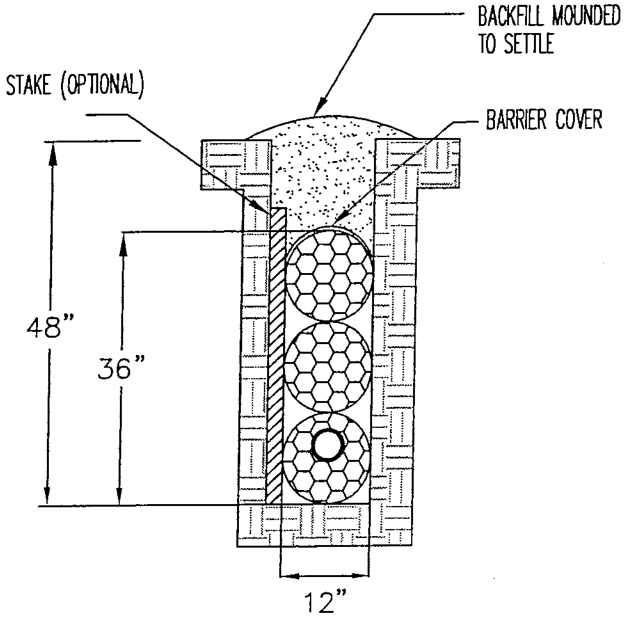
SCALE: NONE



REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
STATE OF OREGON
R Sweeney
Robert F. Sweeney
#0206202513
Expires 30 Sep 05



RIP RAP DETAIL



EZ1203V

GWI DETAIL

PAGE 24
11 JAN 2005
SCALE: NONE

H.D. AVIATION & SOUTHEND AIRPARK
T.4S, R.1W, SEC. 2D & 11A, T.L. 200, 203, 400, 401, 1600
DRAINFIELD DETAILS-2

CHECKED BY:
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OR (503)353-9691 WA (360)735-1109
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Exhibit 2

faegre@earthlink.net

From:	BEACH Anthony <Anthony.BEACH@odav.oregon.gov>
Sent:	Monday, February 7, 2022 8:06 AM
To:	'Helbling, Tony'; 'Aron Faegre'; 'Michelle DaRosa'
Cc:	STANSBURY Betty; 'Ted Millar'; 'Martha Meeker'
Subject:	RE: HDSE drainfield expansion area at UAO

Hi all, I've pasted the questions and request for additional information from our consultants below.

Thanks for getting the response back from NV5 and Aron. We have looked through what was sent over and still have questions and information needed. We also still need a copy of the report that opens & displays all the figures (this was stated as included but it was not one of the attachments). Since other questions are focused on details of the proposed improvements we have not received, we have responded to NV5's answers in that area in orange below:

- Materials/Construction Proposed
    - o What materials specification is to be used (ODOT, proprietary, etc.) for the aggregate?

      Per the GeoWeb Manufacturer the infill material should consist of one third pulverized topsoil and two thirds crushed aggregate. The aggregate portion should be crushed rock that has a particle size range from 0.375 to 1.0 inches with a D50 of 0.5 inches and a 30 percent void space. The engineered fill should lightly be compacted to allow vegetation growth.

      What are the assumed properties of these materials if there are not more specifics as to what might be used? What is "light compaction"? Is there a minimum void space requirement that should be met? Performance spec for infiltration?

    - o What compaction specifications and test methods are proposed to achieve the proposed Geoweb strengths?

      After the cells have been filled the prepared ground surface should be proofrolled with a fully loaded dump truck. Some rutting and deflection is acceptable considering that the FAA specifies the upper 4-inches of subgrade consist loose uncompacted soil over 12-inches of compacted subgrade.

      Again, what is the density intended for these layers? We are not analyzing the rest of the RSA and we need to know how much rutting or deflection is being assumed to be "acceptable". We are concerned with what is being proposed and whether it can support aircraft and vehicle loading.

    - o What compaction specifications and test methods are proposed for soil layers to be placed along with the Geoweb?

      The only other soil that will be placed is the washed gravel or drain rock in the drainage trenches. We recommend only light compaction of this material until it is well keyed. Even at this level of compaction we believe its load bearing characteristics will be superior to the soil that exists in the RSA. Over compacting this material will inhibit its drainage characteristics

      What are the assumed properties of these materials if there are not more specifics as to what might be used? What is "light compaction"? Is there a minimum void space requirement that should be met? Performance spec for infiltration? What load bearing characteristics will these yield? Will these layers retain their characteristics when the grass is mowed or a vehicle passes over the top of them?

o What subgrade compaction specifications and test methods are proposed for the expanded drain field areas?

See our response to the two prior questions.

Same.

o What materials are proposed for use in the rest of the elements of the drain field system (pipes, manifolds, perf spec., etc.)?

To be addressed by others. [[Note: Attachment 6 added by Aron Faegre to this memo for providing this information to Tony Beach.]

Attachment Six does not provide enough detail about the weight rating for proposed elements (structures/pipes/manifolds/etc) or even the proposed cross section in any of the different areas with the geoweb installed. The 2005 design also does not address grading in the proposed drainfield area, but shows a "capping fill" which would not meet RSA grading standards. Please provide a detailed design that includes structure weight ratings and grading plans that meet FAA RSA grading standards. Also, please provide proposed typical sections showing the pipes/structures/geoweb/etc.. Include layer depths, typical surface grades, and detail where the proposed sections will intersect proposed drain field structures/drainage elements.

Please let me know if you have any questions about the above, and we look forward to your response,

Tony Beach
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
OFFICE 503-378-2523 **CELL** 503-302-5455
M-F 7:30am – 4pm

From: BEACH Anthony
Sent: Tuesday, February 1, 2022 4:36 PM
To: Helbling, Tony <helbling@wilsonconst.com>; Aron Faegre <faegre@earthlink.net>; 'Michelle DaRosa' <mdarosa@landandcondolaw.com>
Cc: STANSBURY Betty <Betty.STANSBURY@odav.oregon.gov>; 'Ted Millar' <tmillar@tlmholdingsllc.com>; 'Martha Meeker' <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Hi Tony, I did check in with our consultants and they said they need some additional information. I pressed them earlier today, they are putting together their clarifying questions and I will forward them as soon as I receive them.

Tony Beach
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
OFFICE 503-378-2523 **CELL** 503-302-5455
M-F 7:30am – 4pm

From: Helbling, Tony <helbling@wilsonconst.com>
Sent: Tuesday, February 1, 2022 3:56 PM
To: BEACH Anthony <Anthony.BEACH@odav.oregon.gov>; Aron Faegre <faegre@earthlink.net>; 'Michelle DaRosa' <mdarosa@landandcondolaw.com>

Cc: STANSBURY Betty <Betty.STANSBURY@odav.oregon.gov>; 'Ted Millar' <tmillar@tlmholdingsllc.com>; 'Martha Meeker' <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Tony,

We're two weeks out since last update – could you please poke the consultants and get info to us?

Tony Helbling
Logistics Manager
Wilson Construction Company
1190 NW 3rd Ave
Canby, OR 97013
Cell: 503-519-6059
Office: 503-263-6882
helbling@wilsonconst.com
www.wilsonconst.com

From: BEACH Anthony <Anthony.BEACH@odav.oregon.gov>
Sent: Tuesday, January 18, 2022 2:05 PM
To: Aron Faegre <faegre@earthlink.net>; 'Michelle DaRosa' <mdarosa@landandcondolaw.com>; Helbling, Tony <helbling@wilsonconst.com>
Cc: STANSBURY Betty <Betty.STANSBURY@odav.oregon.gov>; 'Ted Millar' <tmillar@tlmholdingsllc.com>; 'Martha Meeker' <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Hi Aron, Happy New Year.

Our consultants are still reviewing the information you provided. I will get an update and see if your geotech consultants can provide any assistance.

I'll keep you updated as soon as I get more information, thanks for your patience!

Tony Beach
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
OFFICE 503-378-2523 **CELL** 503-302-5455
M-F 7:30am – 4pm

From: Aron Faegre <faegre@earthlink.net>
Sent: Monday, January 17, 2022 12:23 PM
To: BEACH Anthony <Anthony.BEACH@odav.oregon.gov>; 'Michelle DaRosa' <mdarosa@landandcondolaw.com>; 'Tony Helbling' <helbling@wilsonconst.com>
Cc: STANSBURY Betty <Betty.STANSBURY@odav.oregon.gov>; 'Ted Millar' <tmillar@tlmholdingsllc.com>; 'Martha Meeker' <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Hi Tony,

Hope your holidays went well.

Would it help to have our geotech consultant meet with your geotech consultant to get this resolved? We have provided detailed information for each of your questions, showing that the runway safety area complies with FAA standards. The standards acknowledge that utility systems can be in runway safety areas, and this is an important utility system for the airport.

Aron

Aron Faegre, AIA, PE, ASLA
Aron Faegre Architect
13200 Fielding Road
Lake Oswego, Oregon 97034
503-880-1469
faegre@earthlink.net
www.faegre.org

From: Aron Faegre <faegre@earthlink.net>
Sent: Tuesday, December 21, 2021 4:14 PM
To: 'BEACH Anthony' <Anthony.BEACH@aviation.state.or.us>; 'Michelle DaRosa' <mdarosa@landandcondolaw.com>; 'Tony Helbling' <helbling@wilsonconst.com>
Cc: 'STANSBURY Betty' <Betty.STANSBURY@aviation.state.or.us>; 'Ted Millar' <tmillar@tlmholdingsllc.com>; 'Martha Meeker (MeekerMA92@msn.com)' <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Hi Tony

One last thing. I should have added a note to your question about whether infiltration testing was done. The testing for a drainfield is quite different than for normal stormwater infiltration testing. In fact, too rapid of an infiltration requires a more complicated septic drainfield piping design. Our septic processing system and drainfield designs are approved directly through State of Oregon DEQ.

Aron

Aron Faegre, AIA, PE, ASLA
Aron Faegre Architect
13200 Fielding Road
Lake Oswego, Oregon 97034
503-880-1469
faegre@earthlink.net
www.faegre.org

From: Aron Faegre <faegre@earthlink.net>
Sent: Tuesday, December 21, 2021 3:07 PM
To: 'BEACH Anthony' <Anthony.BEACH@aviation.state.or.us>; 'Michelle DaRosa' <mdarosa@landandcondolaw.com>;

'Tony Helbling' <helbling@wilsonconst.com>
Cc: 'STANSBURY Betty' <Betty.STANSBURY@aviation.state.or.us>; 'Ted Millar' <tmillar@tlmholdingsllc.com>; 'Martha Meeker (MeekerMA92@msn.com)' <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Hi Tony,

Attached are the answers to your detailed questions. Does this provide the information you need to approve our proposal?

Aron

Aron Faegre, AIA, PE, ASLA
Aron Faegre Architect
13200 Fielding Road
Lake Oswego, Oregon 97034
503-880-1469
faegre@earthlink.net
www.faegre.org

From: BEACH Anthony <Anthony.BEACH@aviation.state.or.us>
Sent: Thursday, December 9, 2021 4:20 PM
To: Michelle DaRosa <mdarosa@landandcondolaw.com>; Tony Helbling <helbling@wilsonconst.com>
Cc: STANSBURY Betty <Betty.STANSBURY@aviation.state.or.us>; Ted Millar <tmillar@tlmholdingsllc.com>; Aron Faegre - Aron Faegre & Associates (faegre@earthlink.net) <faegre@earthlink.net>; Martha Meeker (MeekerMA92@msn.com) <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Hi Michelle, thank you for your patience as we look into the information you have provided.

Our consultants have taken a first pass through the report along with their Geotech GRI, and they came up with the following list of questions/clarifications/additional information needed:

GRI requests the additional data listed below based on reviewing the November 8, 2021 report “Report of Geotechnical Engineering Services: Aurora State Airport Septic Drain Field Improvements for HDSE Sewer System.” [*HDSE drainfield expansion Geotech Study AronFA-2-01-110821-geor.pdf*]

- Field Data Collection
  - Date of soil sampling
  - Were any logs prepared to describe the bulk sampling results?
  - Was a sieve analysis and/or Atterberg Limits test performed to validate the Silt visual classification?
  - Was infiltration testing performed? If not, why?

- As-builts or other construction documents pertaining to the existing drain field

- Report references
  - Geoweb design procedure
  - Provide addition discussion on how the 6-inch geoweb, with 2/3 aggregate and 1/3 topsoil, replaces 12 inches of compacted soil.
  - Equivalent Single Wheel Load source
  - Source identifying the critical aircraft type

- Report figures
    - Figure A-1: graphic does not show up in the provided pdf
    - Figure A-2: graphic does not show up in the provided pdf

- “Such stringent compaction is not permitted in the soil cover of drain fields”
    - Where does this statement come from?

In addition to the list above, we will also need specifics on the proposed Geoweb reinforced drain field construction.

- Materials/Construction Proposed
    - What materials specification is to be used (ODOT, proprietary, etc.) for the aggregate?
    - What compaction specifications and test methods are proposed to achieve the proposed Geoweb strengths?
    - What compaction specifications and test methods are proposed for soil layers to be placed along with the Geoweb?
    - What subgrade compaction specifications and test methods are proposed for the expanded drain field areas?
    - What materials are proposed for use in the rest of the elements of the drain field system (pipes, manifolds, perf spec., etc.)?

Could you please provide this information so I may forward it to our consultants for review?

Thank you,

Tony Beach
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
OFFICE 503-378-2523 **CELL** 503-302-5455
M-F 7:30am – 4pm

From: Michelle DaRosa <mdarosa@landandcondolaw.com>
Sent: Monday, November 15, 2021 3:51 PM
To: BEACH Anthony <Anthony.BEACH@aviation.state.or.us>; Tony Helbling <helbling@wilsonconst.com>
Cc: STANSBURY Betty <Betty.STANSBURY@aviation.state.or.us>; Ted Millar <tmillar@tlmholdingsllc.com>; Aron Faegre - Aron Faegre & Associates (faegre@earthlink.net) <faegre@earthlink.net>; Martha Meeker (MeekerMA92@msn.com) <meekerma92@msn.com>
Subject: RE: HDSE drainfield expansion area at UAO

Re-sending to include Ms. Martha Meeker.

Michelle D. Da Rosa
Attorney at Law
205 SE Spokane Street, Suite 300
Portland, OR 97202
Office: (503) 220-2891
Direct: (971) 600-6307
www.landandcondolaw.com

MICHELLE D. DA ROSA 2021
RECOGNIZED BY
Best Lawyers

From: Michelle DaRosa
Sent: Monday, November 15, 2021 3:25 PM
To: Tony Beach (anthony.beach@aviation.state.or.us) <Anthony.BEACH@aviation.state.or.us>; Tony Helbling <helbling@wilsonconst.com>
Cc: Betty Stansbury (betty.stansbury@aviation.state.or.us) <Betty.STANSBURY@aviation.state.or.us>; Ted Millar <tmillar@tlmholdingsllc.com>; Aron Faegre - Aron Faegre & Associates (faegre@earthlink.net) <faegre@earthlink.net>
Subject: FW: HDSE drainfield expansion area at UAO

Dear Betty and Anthony,

This missive from me, in my capacity as the attorney for TLM Holdings LLC and from Tony Helbling, as a director of HDSE Sewer System Owners Association and Chairperson of the Southend Corporate Airpark Condominium Owners Association, requests that you (i) rescind your denial of HDSE's plans to expand the HDSE drainfield on UAO property, (ii) retract ODA's stated intention to not renew HDSE's drainfield lease in 2024, and (iii) issue an approval of the expansion plans as previously submitted earlier this year. The attached study and our explanations below respond to the concerns ODA cited as the reason for its decisions.

The denial of the proposed expansion was sent to me in the email from Anthony dated July 30, 2021 in the email string below. ODA's expansion denial and threat to terminate the drainfield located on the Aurora State Airport that serves HDSE users (all buildings at Southend) sent concerned shock-waves through the Southend Airpark community because of the vital importance of the drainfield to the HDSE Sewer System, and the HDSE Sewer System to the continued operation of all of the property at Southend. The threat to "not renew" was made notwithstanding that the Non-Commercial Site Lease provides HDSE with two 5-year options and that the Utility Easement recorded as Instrument No. 2020-00001957 on January 13, 2020 is perpetual.

The attached geotechnical study by NV5 (formerly known as GeoDesign), dated November 8, 2021 demonstrates through detailed soil analysis that the drainfield areas already are likely 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" [AC 150/5300-13A, p. 61]. The area is also free of objects, is drained by grading and a perimeter drain system to avoid accumulation of water, and has no ruts, humps, depressions or other surface variations, as required by the FAA's design standards for RSA's.

We propose resolution of this issue by:

a. Making no changes to the existing drainfields as they have been in the RSA for around 20 years now, with no problems occurring, and the gravel filled drainfield trenches already demonstrating regular supporting of tractors for mowing and thus physically demonstrating meeting the RSA vehicle support requirements.

b. For the new expansion drainfields use the addition of the 6 inch geo-fabric in the top layer, which then results in gaining of 95% compaction (in fact with a 1.5 safety factor bearing capacity over that).

In addition, we note as mitigating factors that:

- To promote the functionality of Aurora Airport as a resiliency resource following a major earthquake, the septic system will allow the airport to seamlessly continue operation following an earthquake, whereas those airports relying on urban sanitary systems will generally require from one month to a year to become functional after the earthquake – thus the HDSE's septic system is an advantage to promote at Aurora Airport.

- The existing and proposed drainfields are approximately 150 feet or more to the side of the runway centerline, and thus they are areas that are least likely to be needed for emergency use.

- Many existing areas of the RSA do not currently meet the 95% compaction requirement (as shown in the geotech study).

Sincerely yours,

Tony Helbling
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From: BEACH Anthony <Anthony.BEACH@aviation.state.or.us>
Sent: Friday, July 30, 2021 10:20 AM
To: Michelle DaRosa <mdarosa@landandcondolaw.com>
Subject: RE: HDSE drainfield expansion area at UAO

Good morning Ms. DaRosa,

I am writing to follow up on your request for 103,104 square feet of additional drain field and reserve area lease space at the Aurora State Airport. We understand your client, HDSE Sewer System Owners Association, already has 61,375 square feet of premises leased for a drain field, reserve area, and piping. We are also aware that the existing lease was entered into with a general understanding that additional space would be needed, and that additional space would be made available by the Oregon Department of Aviation. Though both drain field use and leasing within Runway Safety Areas are unusual in my experience, I have been working to honor that arrangement with the intent of accommodating the expansion.

In initiating the Pen and Ink change to our Airport Layout Plan for this expansion, some concerns were raised by the FAA regarding compatibility of drain fields and Runway Safety Areas (RSA). The RSA enhances the safety of aircraft which undershoot, overrun, or veer off the runway, and it provides greater accessibility for firefighting and rescue equipment during such incidents. There are four requirements that our RSAs must meet, those include being:

1. cleared and graded and have no potentially hazardous ruts, humps, depressions, or other surface variations;
2. drained by grading or storm sewers to prevent water accumulation;
3. capable, under dry conditions, of supporting snow removal equipment, Aircraft Rescue and Fire Fighting (ARFF) equipment, and the occasional passage of aircraft without causing damage to the aircraft; and
4. free of objects, except for objects that need to be located in the RSA because of their function…

To address these concerns we closely evaluated the information you provided, and we analyzed what impacts, if any, a drain field would have on meeting the RSA's design standards. What we have found is that generally leach field soils are not compacted to the densities needed to support vehicle loads. The effluent from the waste stream has to be able to move into the pores of the soil around the drain tiles for the leach field to function. This increases the moisture content of the soils and further reduces their ability to support loads. At best, we are concerned that vehicle loading (including mowers) will reduce the porosity of the leach field soil (resulting in slower infiltration over time) or, at worst, cause damage to the shallow drain tiles and manifolds resulting in surface failures. It is our conclusion that drain fields in the RSA present a potential hazard to aircraft forced to roll out in the RSA. They are especially hazardous for heavier aircraft or those with higher tire pressures.

Due to the decreased soil strength and increased water accumulation caused by a drain field's function, we are unable to expand your client's drain field and reserve areas. Further, because the existing drain field and reserve area are not compatible within the RSA, we will not be able to renew the lease once the current term expires August 30th, 2024. At that time, all pipes and associated equipment will need to be removed by the Lessee, and the site will need to be returned to its original condition.

I am sorry I don't have a better answer for you, please let me know if you have any questions,

Anthony Beach, C.M., ACE
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
M-F 7:30am – 4pm

OFFICE 503-378-2523 **CELL** 503-302-5455

EMAIL Anthony.Beach@aviation.state.or.us

3040 25TH STREET SE, SALEM, OR 97302

WWW.OREGON.GOV/AVIATION

Exhibit 3

faegre@earthlink.net

Subject:	UAO HDSE Drainfield Discussion
Location:	Microsoft Teams Meeting
Start:	Wed 2/16/2022 10:00 AM
End:	Wed 2/16/2022 11:00 AM
Recurrence:	(none)
Meeting Status:	Accepted
Organizer:	BEACH Anthony

Hi all, let's get together and talk about the HDSE Drainfield at UAO. This is the only time that works for us, CWE, and GRI. We could push this meeting to start at 11am same day, otherwise we'd need to find sometime the following week. Let me know if this doesn't work for all of you.

Link is below.

Tony Beach
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
OFFICE 503-378-2523 **CELL** 503-302-5455
M-F 7:30am – 4pm

Microsoft Teams meeting

Join on your computer or mobile app
Click here to join the meeting

Or call in (audio only)
+1 971-277-1965,,945506483# United States, Portland
Phone Conference ID: 945 506 483#
Find a local number | Reset PIN

Learn More | Meeting options

Exhibit 4

faegre@earthlink.net

From: BEACH Anthony <Anthony.BEACH@odav.oregon.gov>
Sent: Wednesday, February 16, 2022 12:56 PM
To: STANSBURY Betty; 'James Kirby'; Helbling, Tony; Ted Millar (tmillar@southendairpark.com); Aron Faegre
Cc: PECK Heather; Ted Millar; Lindsi Hammond; Wes Spang; Wes Spang
Subject: UAO HDSE Drainfield Discussion

Hi everyone,

Thanks again for meeting today and going over the details we'll need to see for us to agree to keeping the existing drainfield, and leasing additional land for a new drainfield in our Runway Safety Area (RSA). Here's a quick recap.

Conditions we need the RSA to meet

1. Advisory Circular Standards
    a. cleared and graded and have no potentially hazardous ruts, humps, depressions, or other surface variations;
    b. drained by grading or storm sewers to prevent water accumulation;
    c. capable, under dry conditions, of supporting snow removal equipment, Aircraft Rescue and Fire Fighting (ARFF) equipment, and the occasional passage of aircraft without causing damage to the aircraft; and
    d. free of objects, except for objects that need to be located in the RSA because of their function
2. We further discussed the practical requirements for drainfields in the RSA:
    a. Supporting weight of Critical Design Aircraft, emergency response vehicles, and maintenance vehicles with regular mowing without compromising the drainfield's function
    b. Remaining clear of objects (signs, vents, posts), and wildlife attractants
    c. Minimal/no impacts to aircraft operations for serviceability (no equipment or potentially hazardous ruts, humps, depressions, or other surface variations in the RSA to service/repair the drainfield)
    d. Runway extension – no potential to reduce the lifespan of airport infrastructure (runway/taxiway pavement, subbase erosion, etc.)

We need the above demonstrated in detail in stamped engineering plans that we can review before we can agree.

We also discussed HDSE's communication and coordination with ODA's consultants and subs, please continue to communicate directly with me. We will be happy to answer any questions you have while you work through design for these improvements, and to review your plans.

After the meeting we discussed a couple potential alternatives internally. Have you considered locating the drainfields on the new Aurora Airport Business Center (AABC) property, or have you tried reaching out to HTS?

Thanks again, let me know if you have any questions,

Tony Beach
OREGON DEPARTMENT OF AVIATION
STATE AIRPORTS MANAGER
OFFICE 503-378-2523 **CELL** 503-302-5455
M-F 7:30am – 4pm

Exhibit 5

faegre@earthlink.net

From: STANSBURY Betty <Betty.STANSBURY@odav.oregon.gov>
Sent: Thursday, May 26, 2022 2:56 PM
To: Helbling, Tony; Ted Millar; Aron Faegre
Cc: Martha Meeker; BEACH Anthony; PECK Heather
Subject: Aurora drain field update

I haven't forgotten about my IOU on the drain field paper, but there has been a couple of developments I wanted to share with you.

1) Wastewater Treatment plant – I toured the Columbia Helicopters facility on Monday, and looked at their wastewater treatment plant. It is a state of the art, 15,000 gallon per day capacity "Membrane Bio-reactor' facility, currently running about 3-5,000 gallons per day. They are willing to discuss the possibility of allowing other airport buildings onto their system, so I have asked our engineers to do a preliminary feasibility review about the potential of having the CHI treatment plant handle all of the wastewater being served by the seven on-airport drain fields. This is conceptual at this point, and I do not have any further details. There are several hurtles to get over, but it is a potential solution worth evaluating. When I asked CHI's staff if they thought it could handle 1500 people (the number you gave me for airport employment), they thought it could. (And that is before subtracting HTS, which would stay on its own system, or adding visitors, which would probably bring it back up to around 1500 total, ballpark.)

2) FAA position on drain fields in runway safety areas – I asked the FAA's Seattle Airports District Office for guidance on whether a "modification to standards" (which requires their approval) would be needed for an expansion of the drain field in the runway safety area. Their response is below. Given the limited likelihood of success, I would prefer to focus our efforts (and our engineers time) to the possibility of tying into CHI's system. However, if you still wish to pursue attempting to design a system that would meet the RSA requirements (and with the understanding that you would be responsible for the cost of permitting, construction and installation), I am still willing to review it for consideration, and will commit up to eight hours of our engineers time to review your proposal. (And the proposal to move the location to the sides of the runway (email dated May 10th) shows they are still within the runway and taxiway safety areas, so that doesn't help.) And I agree to your proposed decision date of no later than the end of September (four months from now) so I'd like to know your intentions by mid-June if possible.

FAA response to question about a mod to standards

Hi Betty:

Thank you for your question. I hope that my response is clear and concise and that it helps you as you move forward at UAO with regards to the septic fields in the RSA (and other airports in OR that might have septic (drainage) fields in RSAs).

Please reject future proposed septic (drainage) fields under Aurora State Airport's safety areas and *take action to remove existing septic drainage fields under the airport's safety areas at your earliest opportunity.*

The safety area must remain, "capable, under dry conditions, of supporting snow removal equipment, ARFF equipment, and the occasional passage of aircraft without causing major damage to the aircraft." Septic (drainage) fields risk compromising this requirement by:

- ✈ Including elements structurally incapable of supporting these loads either initially or over the length of time the drainage fields remain under the safety area;

- Supersaturating the subsurface, undermining the surrounding soil’s load bearing capacity.

We allow a temporary reduction in load bearing capacity due to natural precipitation. We will not allow artificial saturation of the subsoil to compromise the safety area’s load bearing capacity.

If a drainage field Engineer is somehow able to provide documented evidence that the drainage field will not compromise the safety area’s load bearing capacity over the length of time the drainage fields remain under the safety area, we *may* consider it acceptable, but this would be considered a nonstandard condition and not a Modification of Standards (MOS) in this case because the drainage fields were not federally funded. In addition, we will not approve MOS requests in any case that will diminish the safety area’s ability to perform its function or located within the RSA. Please keep in mind that this would be a long process that would require HQ involvement and potentially might not result in the allowance of the septic fields to remain even if the drainage field Engineer is able to shown that the drainage field would not compromise the safety area.

Luckily this issue is being brought up now as I think that finding a solution can be one of the items in the ongoing master planning effort.

Exhibit 6

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

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

FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

Printed By: mmiller | Print Date: 11/3/2021 8:29:54 AM
File Name: J:\A-D\AronFA\AronFA-2\AronFA-2-01\Figures\CAD\AronFA-2-01-VM01.dwg | Layout: FIGURE 1

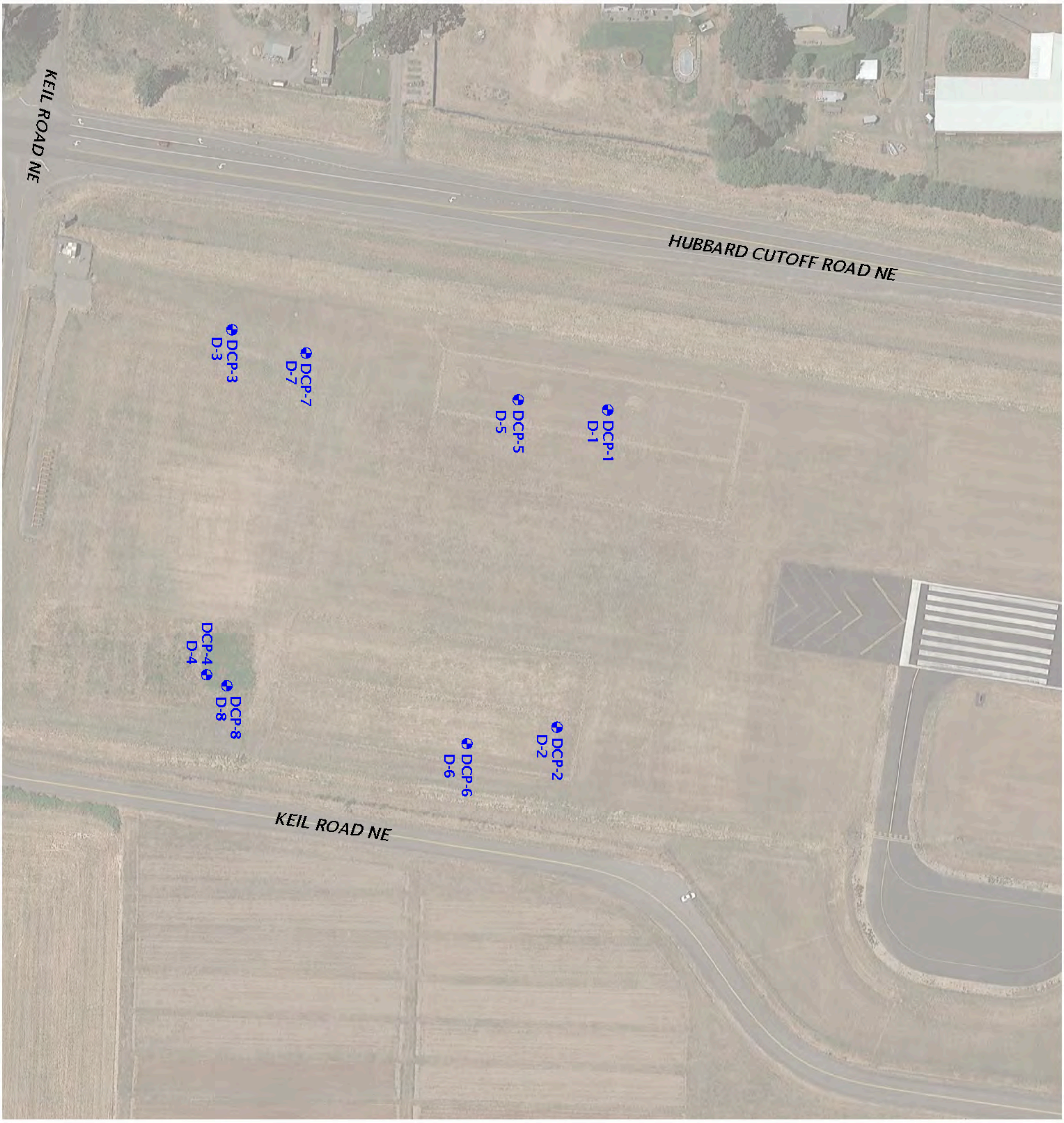
ARONFA-2-01

NOVEMBER 2021

VICINITY MAP

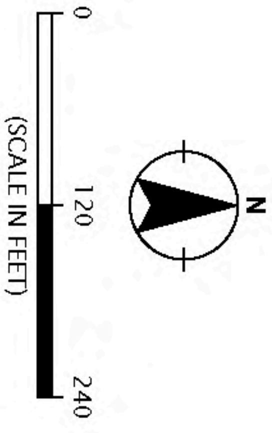
SEPTIC DRAIN FIELD IMPROVEMENTS
AURORA, OR

FIGURE 1



LEGEND:

DCP-1 ● D-1	DCP TEST AND DENSITY MEASUREMENT



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

NV5

ARONFA-2-01	SITE PLAN	
NOVEMBER 2021	SEPTIC DRAIN FIELD IMPROVEMENTS AURORA, OR	FIGURE 2

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:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	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

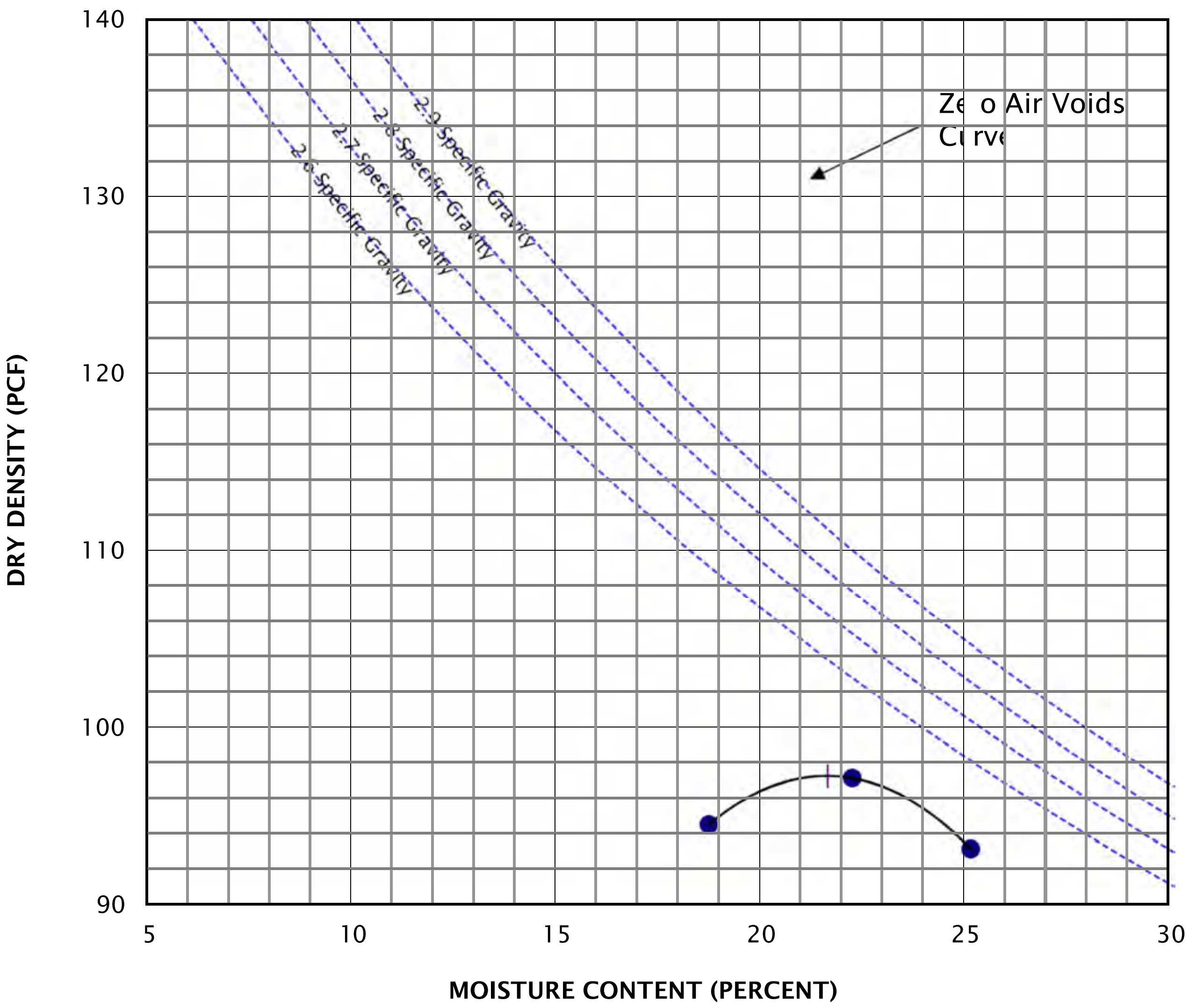
FIGURE 3

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**

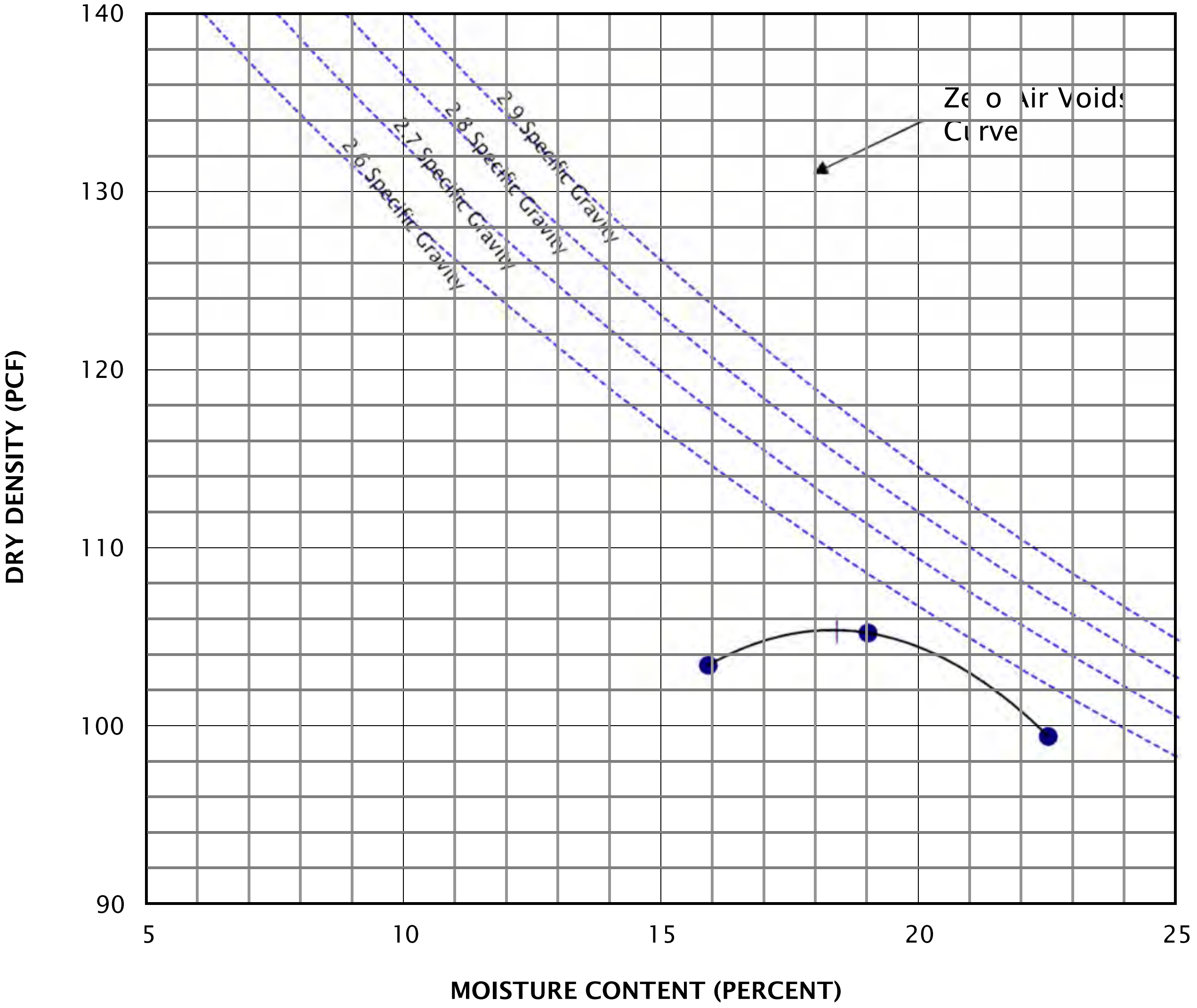
ARONFA-2-01

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/ft³	**105.4**
OPTIMUM WATER CONTENT, %	**18.4**

OVERSIZE CORRECTION

MAXIMUM DRY DENSITY, lb/ft³	**NA**
OPTIMUM WATER CONTENT, %	**NA**

AronFA-2-01-FA1_A2.docx Print Date: 11/2/21

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

APPENDIX B

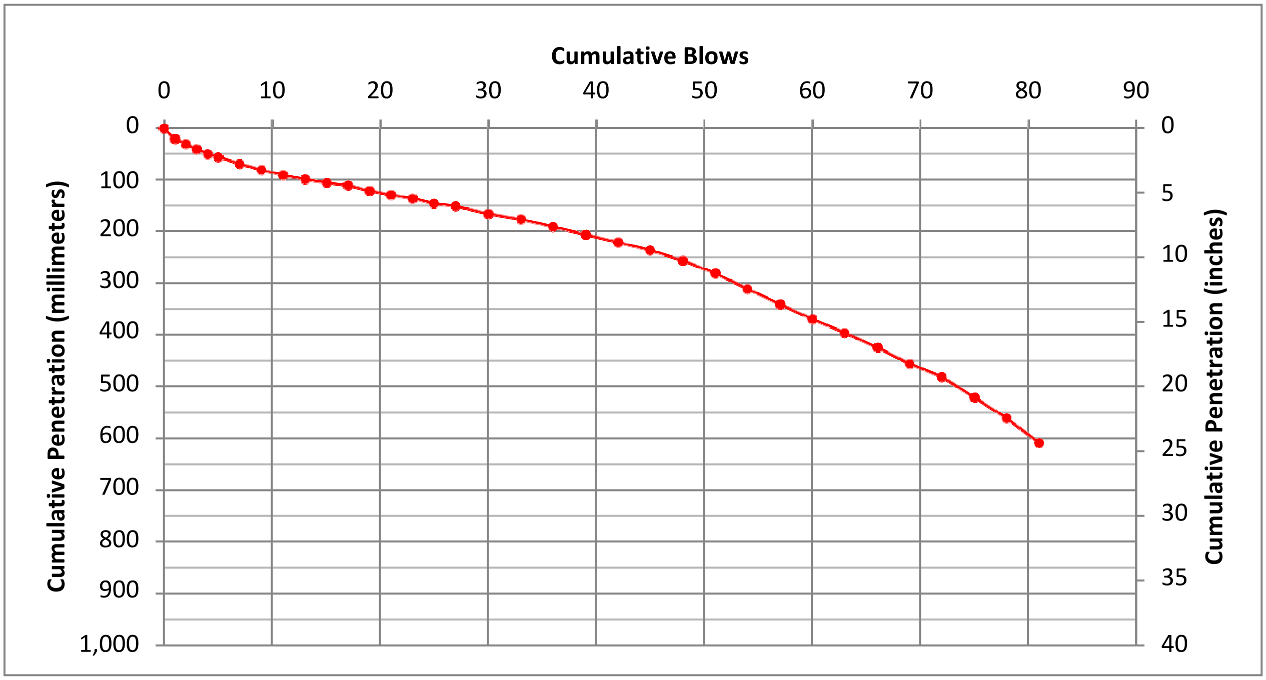
APPENDIX B

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 = Slope (mm/blow)	17.6 pounds 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

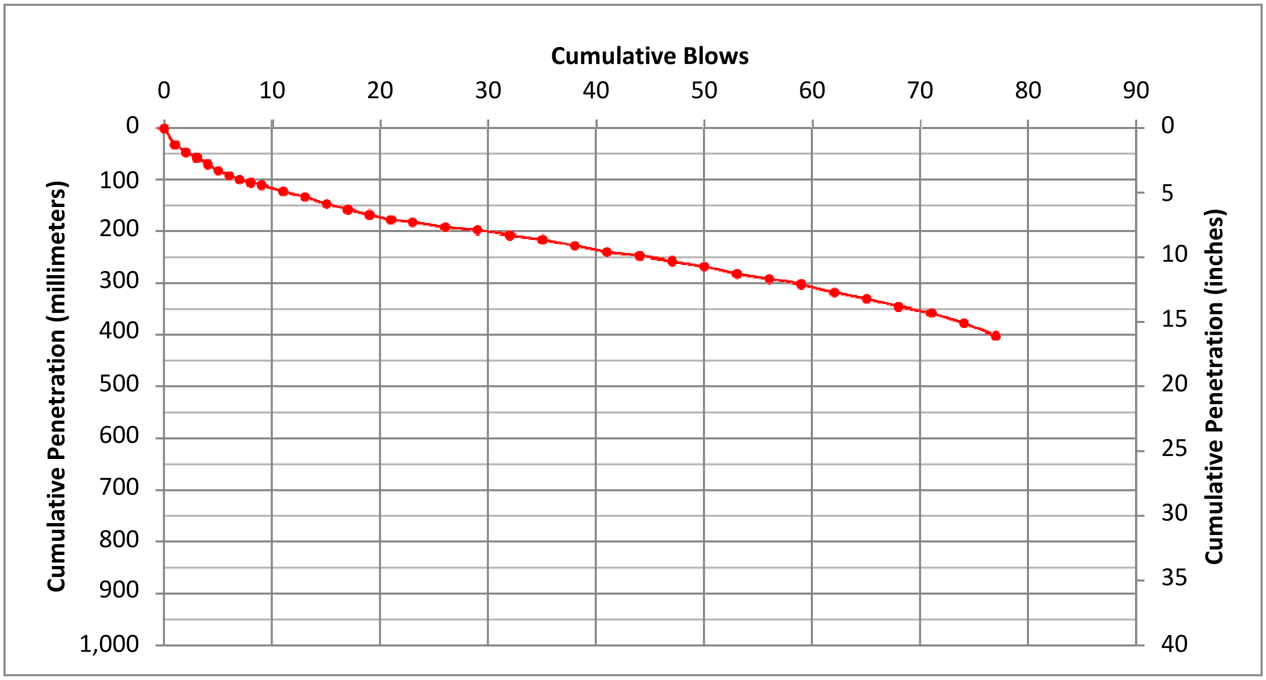
References:

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 × $S^{-0.7168}$; soil not CL, CBR < 10 or not CH

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

M_R = 108206 × $S^{-0.64}$; 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

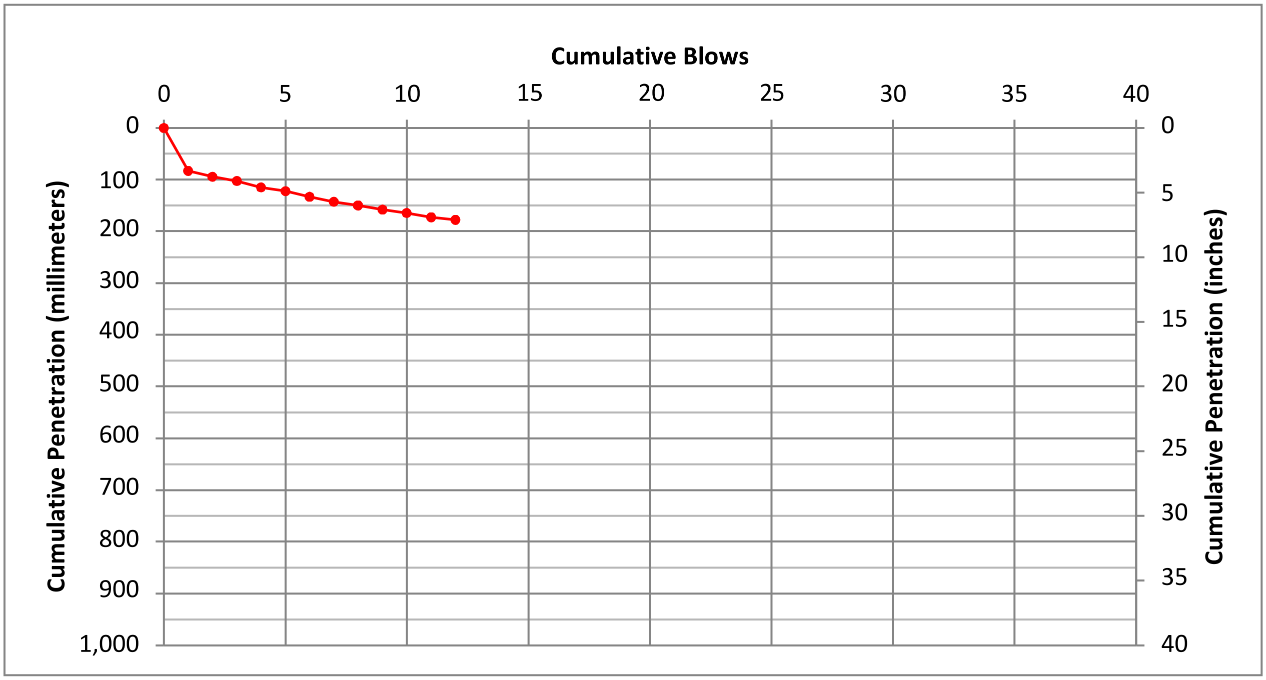
References:

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

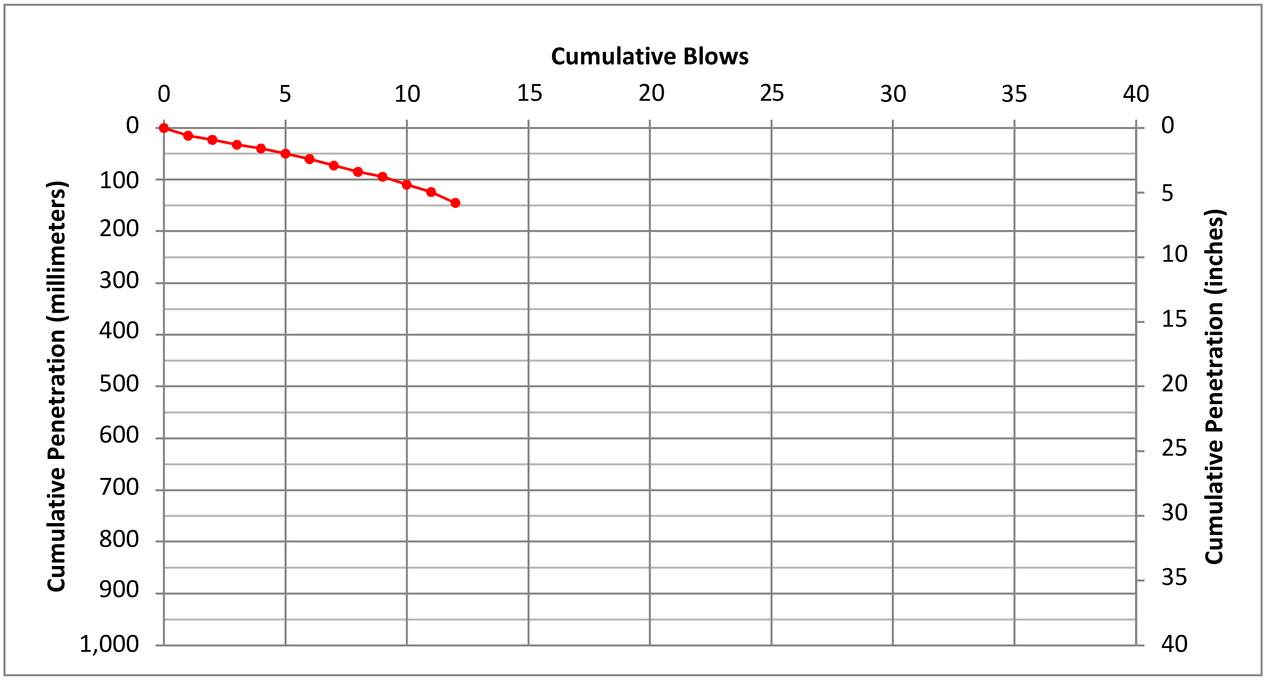
References:

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

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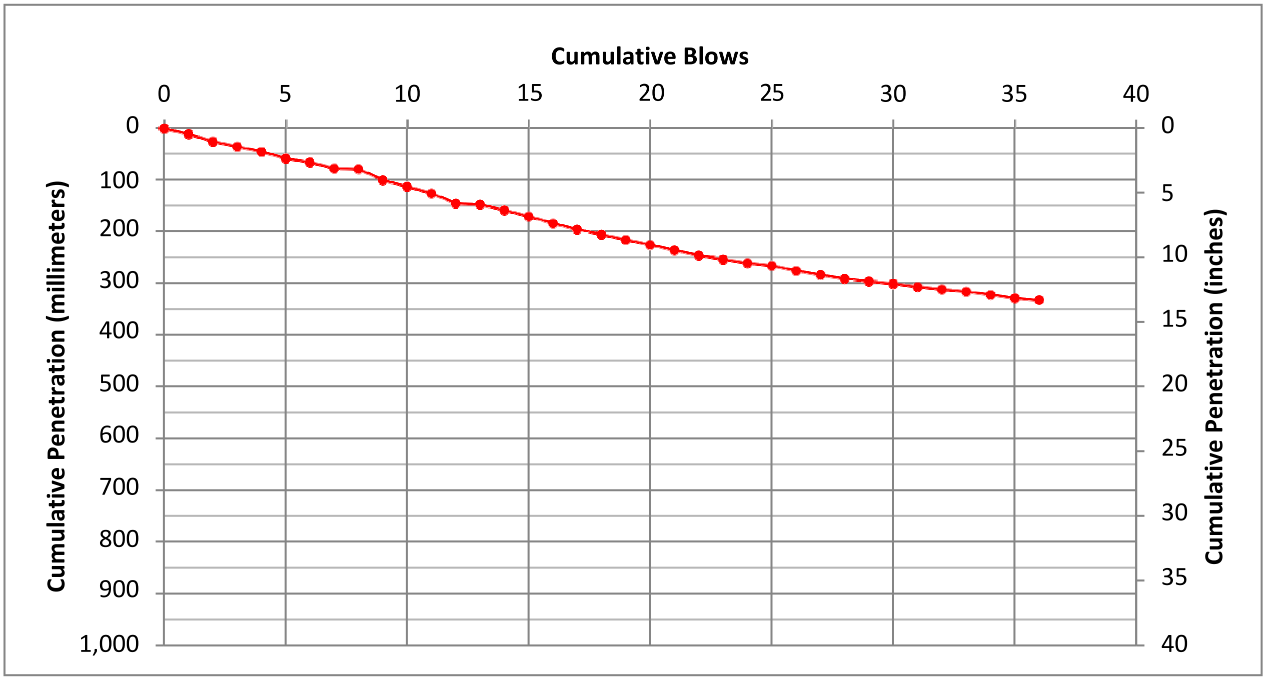
ASTM D 6951, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.

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

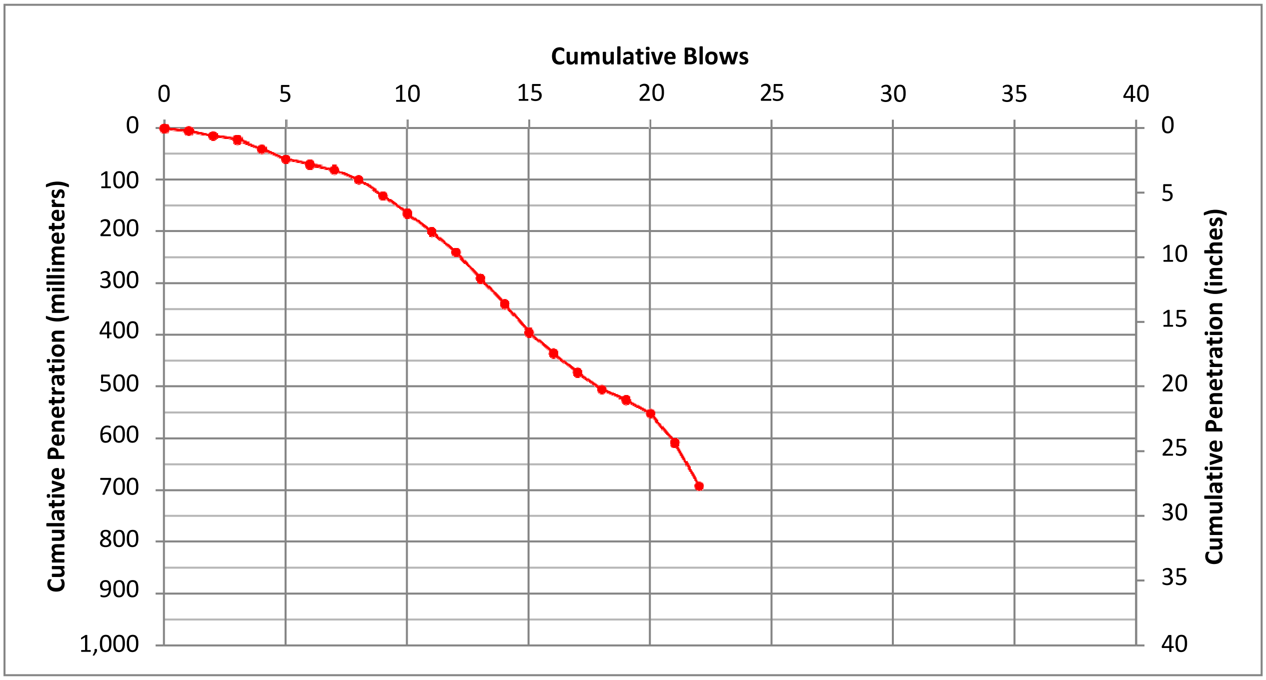
References:

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

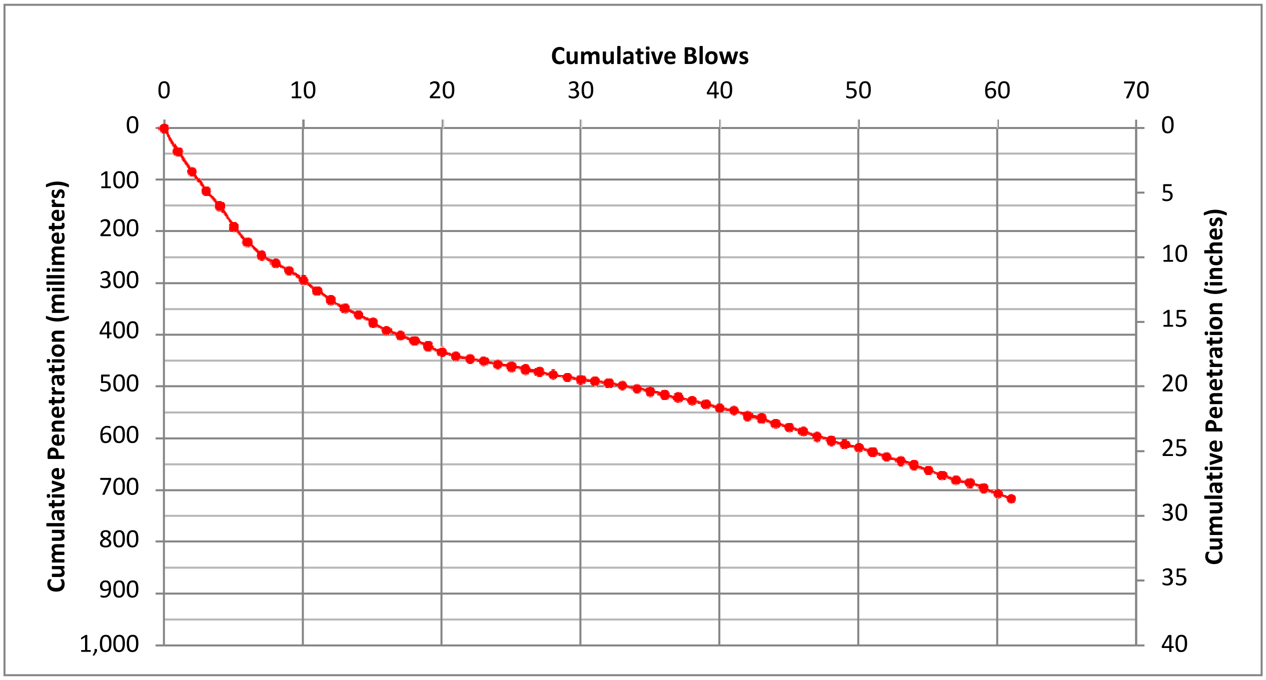
References:

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.

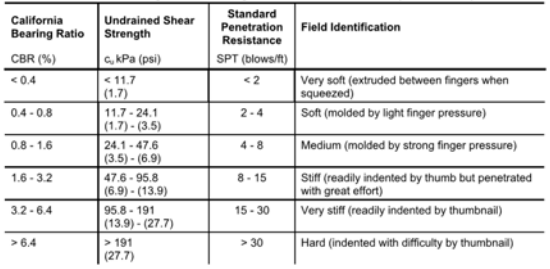
AASHTO H20

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
radius of loaded area	R	7.1
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

$$q_a = N_c c_u$$

where R = Radius of loaded area (i.e. effective radius of single or dual tires)

$$R = \sqrt{\frac{P}{p\pi}}$$

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

$$\sigma_{ht} = K_a \sigma_{vt}$$

$$\sigma_{hb} = K_a \sigma_{vb}$$

$$\sigma_{avge} = \frac{(\sigma_{ht} + \sigma_{hb})}{2}$$

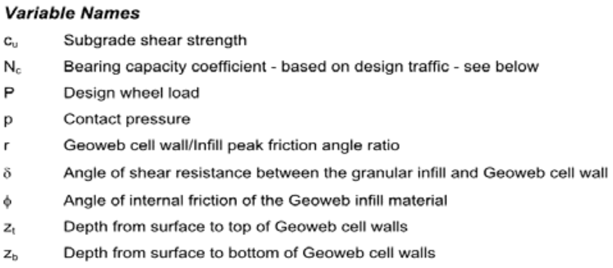
$$\sigma_r = 2\left(\frac{H}{D}\right)\sigma_{avge}\ \tan\delta$$

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
Zb	9
H (in.)geoweb depth	8
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

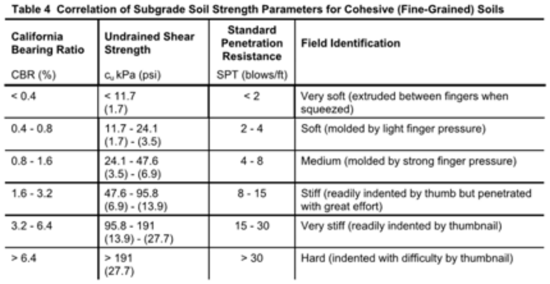


Table 4 Correlation of Subgrade Soil Strength Parameters for Cohesive (Fine-Grained) Soils

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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)
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> 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
radius of loaded area	R	6.9
vertical stress top of geoweb	σvt	199.4
vertical stress bottom of geoweb	σvb	100.8
Active earth pressure coefficient	Ka	0.4
horizontal stress top of geoweb	σht	72.0
horizontal stress bottom of geoweb	σhb	36.4
average horizontal stress	σave	54.2
stress reduction beneath loaded area	σr	45.7

$$q_a = N_c c_u$$

where R = Radius of loaded area (i.e. effective radius of single or dual tires)

$$R = \sqrt{\frac{P}{p\pi}}$$

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

$$\sigma_{ht} = K_a \sigma_{vt}$$

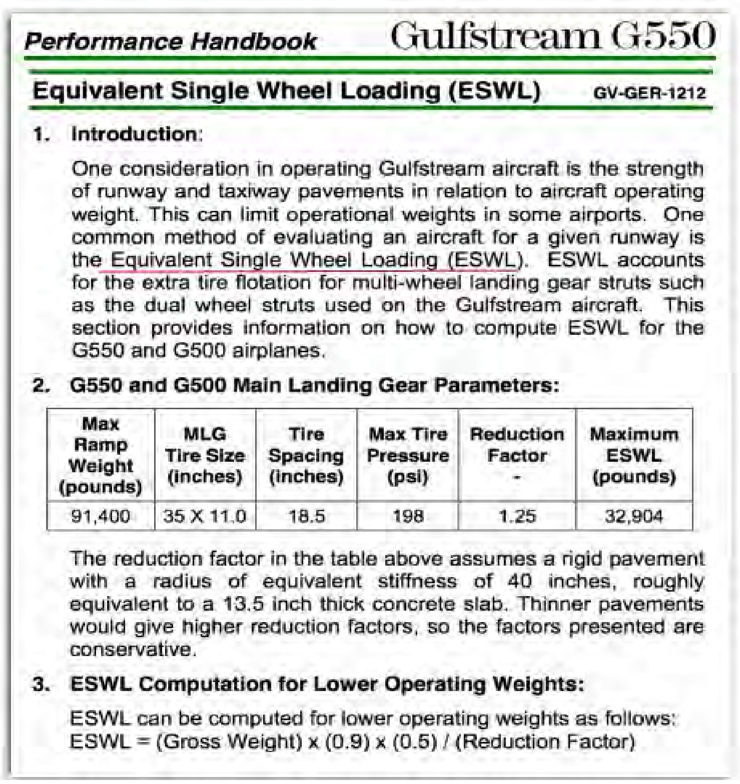
$$\sigma_{hb} = K_a \sigma_{vb}$$

$$\sigma_{avge} = \frac{(\sigma_{ht} + \sigma_{hb})}{2}$$

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

Allowable Stress on Subgrade	71.61
Stress on Subgrade	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:

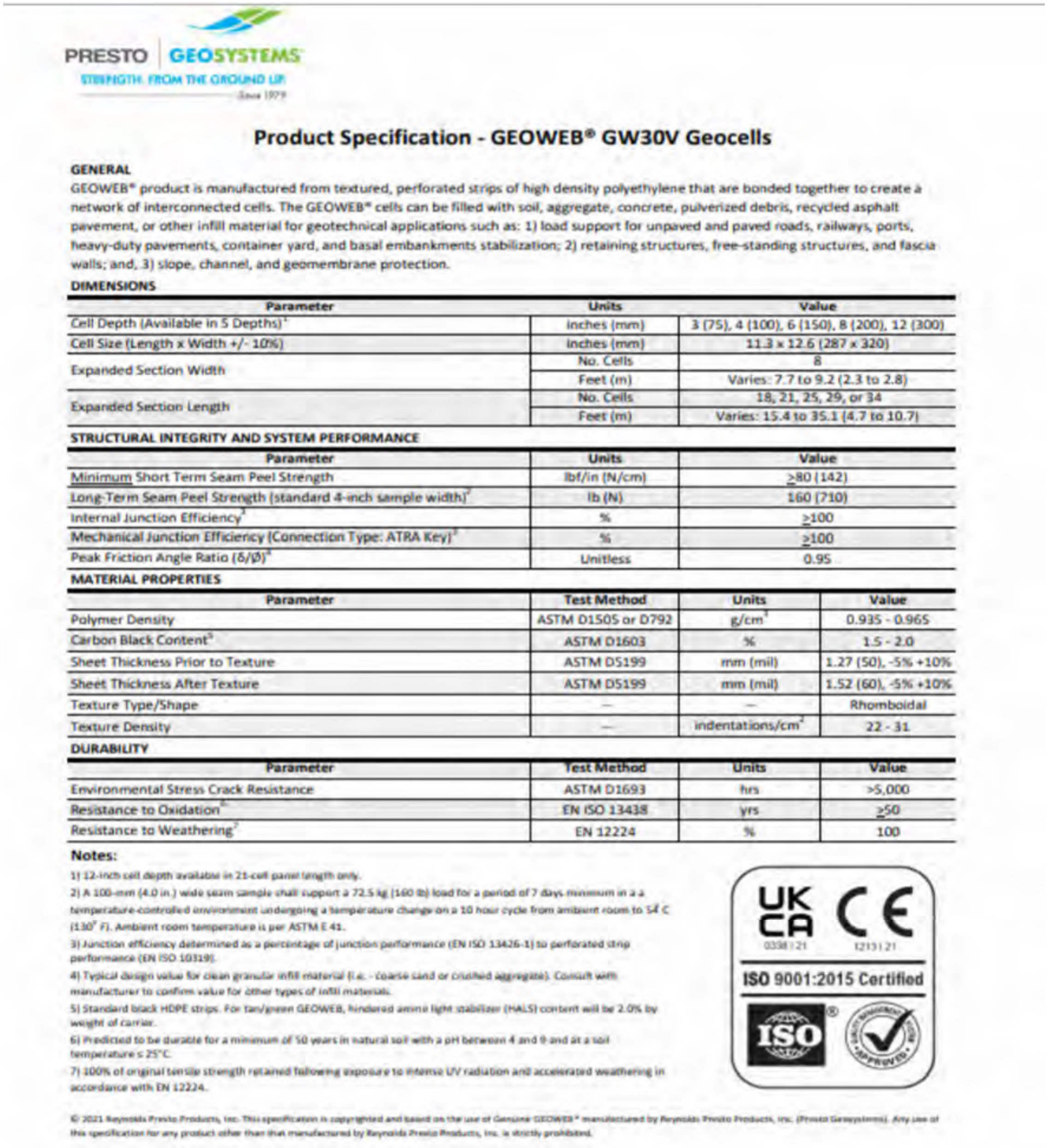
Max Ramp Weight (pounds)	MLG Tire Size (inches)	Tire Spacing (inches)	Max Tire Pressure (psi)	Reduction Factor -	Maximum ESWL (pounds)
91,400	35 X 11.0	18.5	198	1.25	32,904

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 isallowed
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/cm³	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/cm²	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 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.

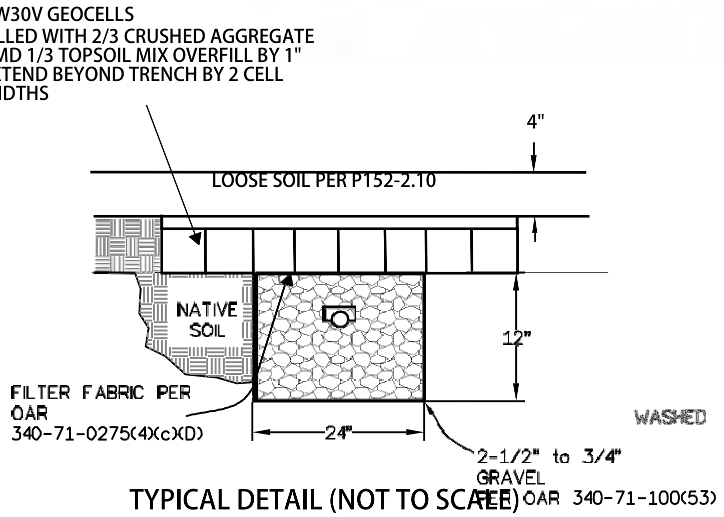
5) Standard black HDPE strips. For tan/green GEOWEB, hindered amine light stabilizer (HALS) content will be 2.0% by weight of carrier.

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 0338 | 21 CE 1213 | 21

ISO 9001:2015 Certified



TYPICAL DETAIL (NOT TO SCALE)

NV5

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Improving Lives

Exhibit 7

ENVIRONMENTAL
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OR: 503-353-9691
FAX: 503-353-9695
WA: 360-735-1109

www.envmgtsys.com

4080 SE International Way
Suite B-112
Milwaukie, OR 97222

25 February 2025

Aron Faegre, AIA, PE

13200 Fielding Road

Lake Oswego, Oregon 97034

RE: Suitability of Proposed Modifications to the Onsite Wastewater Treatment System Drainfield at Aurora State Airport

Dear Mr. Faegre,

I am writing in response to your request regarding the proposed modifications to the onsite wastewater treatment (septic) system drainfield serving HDSE at the Aurora State Airport. Specifically, the inquiry concerns whether the addition of geotextile grid reinforcement over the existing drainfield—intended to enhance support for emergency vehicle access while maintaining compliance with wastewater treatment regulations—meets the requirements set forth by the Oregon Department of Environmental Quality (DEQ).

As a professional with over 47 years of direct experience in onsite wastewater treatment system design and regulatory compliance, I have been continuously involved in the design, installation, and oversight of this specific wastewater system since the initial feasibility evaluation in 1999. I personally conducted the original soil assessment in coordination with DEQ and have overseen all subsequent system modifications, each of which received DEQ approval.

At no time during my 25 years of direct involvement with this site has the drainfield posed any risk to safety, wastewater performance, or compliance. In response to concerns regarding the placement of a drainfield near an active airfield, I have recommended the installation of Presto GeoSystems' Geoweb system. This geotextile cellular confinement system is designed to improve load distribution while protecting the drainfield's function, making it well-suited for incidental vehicle travel, including emergency response access.

Regulatory Compliance and Engineering Considerations

Under **Oregon Administrative Rule 340-071-0520**, large onsite wastewater treatment systems must comply with specific design and impact assessment requirements, including:

- **Impact Analysis**: Any system modifications must be accompanied by a written assessment of potential impacts on public health and water quality, prepared by a qualified professional as specified in ORS 672.535. The NV5 geotechnical report satisfies this requirement by demonstrating compliance with DEQ design standards.
- **Structural Integrity**: DEQ regulations require a minimum of six inches of soil cover over drainfield trenches, with additional cover allowed where engineering supports such design. There is no regulatory limit on maximum cover depth, provided system function is maintained.
- **Load-Bearing Design**: The proposed geotextile grid reinforcement includes a blend of 3/8-inch to 1-inch nominal crushed aggregate to distribute loads in accordance with **AASHTO H20 and Gulfstream ESWL standards**, ensuring adequate safety margins for incidental vehicular travel.
- **Permeability & Drainage Protection**: The selected geoweb design includes a **D50 grain size of 0.5 inches** with an approximate **30% void space**, preventing compaction while facilitating proper aeration and infiltration within the drainfield. The system would extend laterally beyond each drainage trench to mitigate concentrated loading effects.
- **Surface Preparation**: Following manufacturer specifications, the geoweb would be covered with at least **one inch of selected fill** to support vegetative cover, and a **4-inch layer of loose, uncompacted soil** will be applied to satisfy **FAA Item P-152-2.6** seedbed requirements.

Conclusion

Based on my expertise and long-term familiarity with this system, the proposed modifications meet DEQ requirements for drainfields while aligning with FAA safety guidelines for runway safety areas. The use of geotextile grid reinforcement offers a sound engineering solution that enhances load distribution without compromising wastewater treatment performance.

I am a **Registered Environmental Health Specialist** in Oregon and Nevada and a **Professional Onsite Wastewater Treatment System Designer** in Washington. My experience includes serving as **President of the Washington On-Site Sewage**

Association, a **Board Member of the Oregon Onsite Wastewater Association,** and a **Technical Practices Committee Member for the National Onsite Wastewater Recycling Association (NOWRA).** My **resume and aviation specific addendum** is attached for reference.

If you require further clarification or supporting documentation, please do not hesitate to contact me at 503-313-3942.

Sincerely,

REGISTERED ENVIRONMENTAL HEALTH SPECIALIST
STATE OF OREGON
R Sweeney
Robert F. Sweeney
EH-S-202513
Expires 30 SEP 25

Robert F. Sweeney, MS, REHS

President

Environmental Management Systems, Inc.

EMS
ENVIRONMENTAL
MANAGEMENT
SYSTEMS, INC.

OR: 503-353-9691
FAX: 503-353-9695
WA: 360-735-1109

www.envmgtsys.com

4080 SE International Way
Suite B-112
Milwaukie, OR 97222

Robert F. Sweeney, MS, REHS

February 2025

SUMMARY

47 years, providing expertise in Public Health & Environmental Protection programs, ranging in scope from Disaster Preparedness, Epidemiology, Food Protection, Water Quality, Solid Waste Management, Public Health Education, Erosion and Sediment Control, Wastewater Treatment System Design, Wetlands Delineation, Water System Evaluations, Environmental Assessments, Biological Assessments, Treatment System Maintenance and Performance Monitoring.

CERTIFICATIONS

Registered Environmental Health Specialist (Sanitarian), Nevada & Oregon.
OR REHS #EH-S-202513 since June 1977. Nevada REHS #807 since Feb 2024
Professional Onsite Wastewater Treatment System Designer, Washington. Department of Licensing #5100154 since July 2001.
Wetlands Delineation & Management Training, RCET, Inc./ USACE 2001
Wetlands Rating System, Washington Department of Ecology, 2008
Streamflow Duration Assessment Method, USEPA / USACE / ODSL 2009
Certified Erosion & Sediment Control Lead, WA#23320 Feb2007

EXPERIENCE

Consulting Firm Principal – 1997-Present. President of Environmental Management Systems, Inc. EMS specializes in assisting property owners to achieve their development due diligence, environmental design and regulatory compliance goals. Areas of performance include: On-Site Waste-Water Treatment, Site & Soils Evaluation, Wetland Delineation & Mitigation, Drainage and a wide range of environmental consulting services. Focus is on: Site Evaluation, Soil Profiling, Design of Treatment Systems, Inspection, Certification and Performance Monitoring. Over the past 26+ years the firm has completed over 3,000 projects, ranging from Environmental Site Assessments, Geologic Hazard Studies, Wastewater Treatment Systems treating from 250 to over 75,000 gallons/day. System designs include innovative technology such as subsurface drip-fields, textile filters, aerobic treatment units, sequence batch reactors and telemetry. Conducted Evaluation of status of Local Health Department Disaster / Bio-Terrorism Preparedness and Capabilities. Environmental Health Specialists, Professional Soil Scientists, Engineers, Geologists, Surveyors, Biologists, Wetland Professionals and Engineering Technicians are on staff & through partner firms.

Disaster Management Program Coordinator – 1990 – June 2002 Disaster Management Program Coordinator for a Civil Affairs Brigade, US Army Reserve. (Lieutenant Colonel) Coordinated the

Disaster Preparedness Program, with teams assisting nations throughout the Pacific Rim and beyond. Developed the inter-disciplinary, civilian-military program and personally conducted training and exercises for Disaster Preparedness activities in the Marshall Islands, Palau, Bosnia, Samoa and Niue. Developed a Geographic Information System and database driven disaster management program in conjunction with US and international Civilian and Military components.

Chief Environmental Engineer - Stabilization Force, Bosnia-Herzegovina. 1997. Served as the Chief Environmental Engineer for the Civil Military Task Force. Assisted civil authorities to evaluate, plan and repair war-torn environmental infrastructure for water, sewage and solid waste facilities. Supervised teams of engineers in restoring water, sewage, housing, utilities, transportation and communication systems. Conducted Emergency Response Training for Civilian and Multi-National Military Personnel. Awarded Defense Meritorious Service Medal.

Environmental Health Program Manager 1989 to 1997 - Served 8 years as Manager of the Liquid Waste and Land-Use Department for Southwest Washington Health District, a 3-County / 12- Municipality, Regional Health District in Washington State. Developed programs for On-Site Liquid waste regulation involving Water Quality studies, Professional Development, Septic System maintenance and Low Interest Loans for repairs of failed on-site sewage disposal systems. Developed the first comprehensive On-Site Sewage System Maintenance Program for the Southwest Washington Health District, one of the first in Washington.

Environmental Health Specialist (Sanitarian) 1977 – 1989. Deschutes County Health Dept (5 yrs), Multnomah County Health Department (7 yrs): Food, Wastewater, Water, Swimming Pools, Tourist Facilities, Housing Sanitation, Solid Waste, Epidemiologic Investigations of Food-borne outbreaks. Primary author of Multnomah County's first Food Handler Test.

EDUCATION

Master Business Administration, Veteran Entrepreneurial Training & Resource Network Apr23
Command & General Staff College, US Army, 1993
Civil Affairs Officers Advanced Course, US Army, 1992
Master of Science in Management, Marylhurst University 1987
Medical Service Officers Advanced Course, US Army, 1985
Tactical Intelligence Officers Course, US Army, 1983
Certificate in Public Health, Portland State University, 1977
Bachelor of Science in General Science, Portland State University, 1977

CURRENT MEMBERSHIP AND COMMITTEES

National Onsite Wastewater Recycling Association – Board Member / Tech Practices
Oregon On-site Wastewater Association (Former Board Member)
Washington On-Site Sewage Association (President 2003-04)

PREVIOUS MEMBERSHIP AND COMMITTEES (partial)

WA Department of Health Sewage Tank Rule Revision Committee (2010)
WA Department of Health Large Onsite Sewage System Rule Committee (2010)
Clackamas County Citizens Involvement Committee (2004-2005)
METRO Technical Advisory Committee (2004-2005)
Oregon Governor's Task Force on Wastewater Re-Use Committee (May – Dec 2004)
WA Department of Licensing Working Groups on: Sewage Tank Rules & Licensing Onsite Wastewater Treatment System Designers (1996-2000)

EMS
ENVIRONMENTAL
MANAGEMENT
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Robert F. Sweeney, MS, REHS

February 2025

AVIATION RELATED ADDENDUM

Mr. Sweeney has been continuously involved with Safety from 1967 to present as a Water Safety Instructor, Aviation Technician & Environmental Professional in programs, including Training and Experience in Disaster Preparedness with Aviation Related Emergency Scenarios:

Consulting Firm Principal – 1997-Present. President of Environmental Management Systems, Inc. EMS specializes in assisting property owners to achieve development and wastewater treatment goals, including Airports & Aviation Related Projects as follows:

- Aurora State Airport, Aurora. OR (1999 – Present)
  - Van's Aircraft HD Aviation – Wastewater Treatment System New Construction
  - HDSE Aviation – Wastewater Treatment System Expansion
  - Columbia Aviation Association – Wastewater Treatment System -
  - Helicopter Transport Systems– Wastewater Treatment System
  - AABC – Wastewater Treatment System
  - Wylee Condo Association – Wastewater Treatment System
- Scappoose Airport, OR Hangar Addition - – Wastewater Treatment System
- Private Airports - Wastewater Treatment System in Clackamas & Washington Counties

US Army Disaster Management Program Coordinator – 1990 – June 2002 Disaster Management Program Coordinator for a Civil Affairs Brigade. Lieutenant Colonel Sweeney Coordinated the Disaster Preparedness Program, with teams assisting nations throughout the Pacific Rim and beyond. Developed and personally conducted training and exercises including Emergency Preparation for Assessing: Threats, Vulnerability, Resources & Organization for Action including Aviation Crash Scenarios in the Marshall Islands, Palau, Bosnia, Samoa and Niue.

Chief Environmental Engineer - Stabilization Force, Bosnia-Herzegovina. (1997). Served as the Chief Environmental Engineer for the Civil Military Task Force. Conducted Emergency Response Training for Civilian and Multi-National Military Personnel. Awarded Defense Meritorious Service Medal.

US Navy: 1969 – 73 Petty Officer Sweeney served as an Aviation Fire Control / Electronics & Computer Technician with Training and Experience in Aviation Fuel Fire Response:

Whidbey Island Naval Air Station, WA; Annual Training in Aviation Fuel Fire Fighting (70-71).

USS Kitty Hawk Aircraft Carrier (CVA 63) – Safety Petty Officer and Team Leader for 2 Bomb Cooling Crews in Yankee Station - Gulf of Tonkin Viet Nam. (Dec 1971-Dec 72)