DYNAMIS
ENERGY

Site Approval Application Introduction

Dynamis Energy, LLC (Dynamis) is developing and intends to build, own, and operate a renewable energy project in Ada County at the Hidden Springs Landfill. The Ada County WTE Project (the "Project") will be located at 10300 Seaman's Gulch Rd, Ada County, ID, within the existing 2700 acre Ada County Landfill. The proposed Project is subject to 40 CFR Subpart Eb. The proposed project is also subject to the requirements of IDAPA 58.01.06 SOLID WASTE MANAGEMENT RULES. The Project is to be located to the south of the existing Ada County Landfill. The area slated for the proposed facility is an excavated site, which was created by removing earth material used to cap MSW in the landfill. The proposed facility is located at Latitude N43°42.33' and Longitude 116° 16.45'. The Project site will be re-graded to an elevation to accommodate the facility

The Project will consist of a Municipal Solid Waste (MSW) fired steam generator. The heat input of the steam generator is expected to be about 295 million British Thermal Units per hour (MMBtu/hr). The Project will process a maximum of 408 tons per day (tpd) of MSW supplied by Ada County. The Project is designed for a nominal net electrical output of 22 MW for 16 hours per day after supplying steam and electrical power to the facility. The area slated for the proposed facility is on a borrow site within the landfill parcel.

The Project will process MSW, which otherwise would have been deposited in the Ada County landfill. MSW will be diverted from the landfill cell at the landfill gate/scales to the facility and will be placed on tipping floors inside the building. Non-processible waste (primarily bulky appliances and scrap metal) will be returned to Ada for recycling. The nominal, as delivered, higher heating value of MSW is expected to be 5,800 Btu/lb.

Power supplied from the Project will be interconnected with the local 138 kV. The Project will be equipped with a black start generator capable of starting the Project during times when the power grid is out of service. Either this generator or the local grid can be utilized to support operation of the facility if the power plant is out of service.

This Project will not impact the MSW collection activities or recycling programs currently maintained by Ada County, Boise City, Meridian, Etc. Additionally, this Project will not limit recycling activities, goals, or methodology that Ada County, Boise City, Meridian, Star, Eagle, Garden City, and Kuna. may introduce in the future. As such, Dynamis' MSP specifically addresses activities to promote recycling and beneficial re-use of waste once the MSW is delivered to the Project.

The Standards of Performance for Large Municipal Waste Combustors (MWC) for which construction is commenced after September 20, 1994 or for which modification or reconstruction is commenced After June 19, 1996. This New Source Performance Standard (NSPS) applies to "large" MWC units, defined as any MWC unit capable of combusting more than 250 tons per day of municipal solid waste (40 CFR 60.50b(a)).

Pursuant to IDAPA 58.01.06, approval of location restrictions by the Department of Environmental Quality is required of all new non-municipal solid waste management facilities.

This document serves as the Site Approval Application. This document will be made available for public review at the Public Library in Boise and the Public Library in Eagle for a period of at least 30 days after the application is determined complete by DEQ.

1	Site Approval Application
2	Flood Plain Restriction -FEMA Map
3	Endangered or Threatened Species Restriction -Letter from Idaho Dept. of Fish and Game
4	Surface Water Restriction -USGS Topo Map -Storm Water Report
5	Parks, Scenic, or Natural Use Restriction -Eagle Island State Park Map
6	Ground Water -Water Table Contour Map
7	Geologic Restriction -Geotechnical Report
8	Property Line Restriction -Landfill Property Map
9	Ada County Landfill North Ravine Cell Site Certification Report
10	Site Mapping
11	Ada County Authorization
12	Site Pictures

AVERY® READY INDEX® 600086

TAB 1

NON-MUNICIPAL SOLID WASTE MANAGEMENT FACILITY

SITE APPROVAL APPLICATION

(revised 8/26/2008)

REQUIREMENTS

Pursuant to IDAPA 58.01.06, approval of location restrictions by the Department of Environmental Quality (DEQ) is required for all new and lateral expansions of non-municipal solid waste management facilities. Existing facilities are required to comply with the siting requirements within five years from April 26, 2002. Existing facilities may submit existing approvals that demonstrate compliance with applicable siting requirements. Site approval process may occur prior to or concurrently with, design (if applicable), operations, closure and post-closure approval process. Siting approval is required prior to construction of the facility. Approval from local authorities may be required. It is recommended that the applicant contact the county and/or city in which the facility will be located.

PRE-APPLICATION MEETING

It is very important that the applicant meet with the appropriate DEQ regional office staff before the site approval application is submitted. A tour of the site is very helpful in identifying any possible concerns that may need to be addressed in the application. A pre-application meeting with DEQ, the local Health District and local government is recommended. This will help ensure that there is compliance with location restrictions and operational requirements and that sufficient information is submitted to allow for a timely review. It is advantageous to DEQ, the local health district, local government and the applicant to move this process forward as quickly as possible.

REQUIRED SUBMITTALS

Please fill out the attached form, complete the questionnaire, and attach at a minimum, the indicated documentation to support the location restriction determinations. Original and 2 copies of the application and supporting documents should be sent to the appropriate DEQ Regional Office listed below.

INCOMPLETE INFORMATION MAY CAUSE DELAY IN THE APPROVAL PROCESS

DEQ REGIONAL OFFICE CONTACTS:

Please send the original and 2 copies of the completed application and supporting documents to the appropriate regional office below:

Department of Environmental Quality
Coeur d'Alene Regional Office
2110 Ironwood Parkway
Coeur d'Alene, ID 83814
Contact: Gary Gaffney @ 769-1422

Department of Environmental Quality
Lewiston Regional Office
1118 F. Street
Lewiston, ID 83501
Contact: Gayle Westhoff @ 799-4370

Department of Environmental Quality
Boise Regional Office
1445 N. Orchard
Boise, ID 83706
Contact: Jack Gantz @ 373-0550

Department of Environmental Quality
Twin Falls Regional Office
1363 Fillmore
Twin Falls, ID 83301
Contact: Scott Stanton @ 736-2190

Department of Environmental Quality
Pocatello Regional Office
444 Hospital Way, #300
Pocatello, ID 83201
Contact: Tom Mullican @ 236-6160

Department of Environmental Quality
Idaho Falls Regional Office
900 Skyline, Suite B
Idaho Falls, ID 83402
Contact: Christy Swenson @ 528-2650

CENTRAL OFFICE CONTACT:

Solid Waste Program Coordinator
Department of Environmental Quality
Waste Mgmt and Remediation Division
1410 N. Hilton
Boise, ID 83706
Contact: Dean Ehlert @ 373-0416

NON-MUNICIPAL SOLID WASTE SITE APPROVAL APPLICATION

I. GENERAL INFORMATION - Please complete (type or print) the General Information section or attach Preapplication Meeting Information if completed.

Applicant's Name Dynamis Energy, LLC

Applicant's Signature

Application Date July 6th, 2011

Name of Site Ada County Landfill Waste to Energy Plant

Location of Site Hidden Hollow Sanitary Landfill

Total Acreage of Site 2,727 acres for Ada County Landfill/ 6.58 Waste to energy plant

Legal Description See attached map-Appendix 10- C3 Dimension Plan

Property Owner of Record Ada County-See Attached Appendix 11

Address: 200 W Front Street 3rd Floor

Boise, Idaho 83702

Telephone: 208-287-7000

(Attach written approval from owner to use site for stated purpose, if owner is different from applicant)

Operator of Proposed Facility Dynamis Energy, LLC

Address: 776 E Riverside Drive, Ste. 150, Eagle, ID 83616

Telephone: 208-938-2680

Contact Person Regarding This Application

Name: Doyle Pergande

Address: 776 E Riverside Drive, Ste. 150, Eagle, ID 83616

Telephone: 208-462-2640 or 208-941-2848

02/02/12

This application is for a;

__X____ NEW FACILITY

_____ LATERAL EXPANSION of existing facility

This application is for what type of facility?

_____ Construction & Demolition Waste Landfill

_____ Transfer Station

_____ Industrial Landfill

_____ Compost Facility

_____ Septage Disposal Site

_____ Waste Tire Collection Site

_____ Petroleum Contaminated Soils Processing Site

X Other (please specify) _____Thermal Oxidator_____

Proposed Tier Classification

X_____ Tier II

_____ Tier III

What is the composition of the waste material to be managed, processed or disposed?

Material	%
MSW	98.5
Tires	1.5

What is the volume/mass of material received per day?

Volume/Mass	Unit
408 TPD MSW & 12,000 lbs per day Tires	

03/01/12

II. LOCATION RESTRICTIONS - SITE APPROVAL

Instructions: These questions relate directly to the location restrictions for non-municipal solid waste management facilities. Answer the questions below and, in an attached report, include all supporting documents and describe how they were used to make the determinations. This documentation should include, but not be limited to, maps, technical reports, letters, documentation of phone conversations, computer models, on-site investigations, analyses, etc.

01 GENERAL SITING REQUIREMENTS - All Tier II and III solid waste management facilities regulated under the Solid Waste Management Rules, IDAPA 58.01.06 shall comply with items a through d.

a. Flood Plain Restriction

Is the facility located within a one hundred- (100) year floodplain? *Provide a copy of the applicable Federal Emergency Management Agency (FEMA) map showing the location of the disposal site with respect to the 100-year flood boundary. If a map is not available for the proposed site location, a site evaluation report and a letter of confirmation, stamped by a registered professional engineer or registered professional geologist, may be required.*

_____ YES __X__ NO

If yes, provide information demonstrating that the facility will not restrict the flow of the one hundred (100) year flood, reduce the temporary water storage capacity of the floodplain, or result in a washout of solid waste so as to pose a hazard to human health and the environment. *If the site is located within the 100-year floodplain, the demonstration noted above must be prepared and stamped by a registered professional engineer or registered professional geologist, as appropriate.*

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) See attached FEMA Map #16001C0160H

b. Endangered or Threatened Species Restriction

Is the facility located such that it does not cause or contribute to the taking of any endangered or threatened species of plant, fish, or wildlife or result in the destruction or adverse modification of the critical habitat of endangered or threatened species as identified in 50 CFR Part 17?

__X__ YES _____ NO

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) See Letter from Idaho Department of Fish & Game

c. Surface Water Restriction

Is the active portion of the facility located such that the facility shall not cause contamination of surface water, unless such surface water is an integral part of the facility's operation for storm water management and/or leachate management? *Provide at a minimum, a scale map showing the site location and distances to the nearest surface water bodies. A USGS 7.5-minute topographic map is suggested.*

__X__ YES _____ NO

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) _See attached 7.5 Minute Series Topographic map and report from ESC on Retention Pond Design _____

d. Parks, Scenic or Natural Use Restriction.

Is the facility located within one thousand (1,000) feet of any state or National Park, or land reserved or withdrawn for scenic or natural use? *Provide a scaled map showing the disposal site location, and distances to the nearest state or National Park, scenic area, and natural use area.*

_____ YES __X__ NO

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) See attached Google Map closest State park is 6.6 miles from the site.

Note - Items e through g are required for all Tier II facilities and Tier III facilities except Tier II transfer stations, CESQG Management facilities and incinerators. Item h is required for all Tier II and Tier III NMSW landfills.

e. Ground Water

Is the active portion of the facility located, designed or constructed such that the facility shall not cause contamination of a drinking water source or cause contamination of the ground water?

Provide information demonstrating that the facility will not degrade ground water. *Minimum documentation shall include well driller's reports, boring logs, and/or site specific environmental assessment information sufficient to demonstrate depth to the uppermost ground water bearing zone and the composition, thickness and characteristics of overlying soil and rock materials. Depending on the nature of the waste to be disposed at the facility and the vulnerability of the ground water beneath the site, best management practices and best practical methods, including special engineering designs, may be required to ensure protection of the ground water.*

__X__ YES _____ NO

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) Figure 4-21 Water Level Map from North Ravine Cell_

f. Geologic Restriction

Is the NMSW Management Facility's active portion located on land that would threaten the integrity of the design? *Minimum documentation must include a description of the site characteristics sufficient to provide a clear understanding that the site is stable.*

_____ YES __X___ NO

If YES, provide information demonstrating that engineering measures have been incorporated into the NMSW Management Facility design to ensure the integrity of the structural components of the NMSW Management Facility will not be disturbed. Factors to consider can include, but not be limited to, on-site and local soil conditions, on-site or local geologic or geomorphologic features, on-site or local human-made features or events? *A site evaluation and letter of confirmation from a registered professional engineer or registered professional geologist may be required.*

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) _See Strata Report-Geotechnical Engineering Evaluation

g. Property Line Restriction

Is the facility located so that the active portion is any closer than one hundred (100) feet to the property line of adjacent land? *Provide at a minimum, a scaled map showing the active portion of the disposal site, the distance from the active portion to the site property boundaries, and the location and land use of adjacent properties. A 100-foot buffer zone must be maintained between the active portion of the solid waste facility and adjacent properties.*

_____ YES ___X__ NO

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) See Figure 1-1 Master Plan of North Ravine Cell

h. Wetlands

Is the facility located in a wetland? *Documentation may include a copy of the applicable National Wetlands Inventory map and letters from the U. S. Army Corps of Engineers and the National Resource Conservation Service that provide a wetlands determination.*

_____ YES __X___ NO

If YES, provide information demonstrating that:

1. No practical alternative exists?

2. The facility will not cause or contribute to violations of the state water quality standards?

3. The facility will not violate any applicable toxic effluent standards or prohibitions under Section 307 of the federal Clean Water Act?

4. The facility will not cause or contribute to significant degradation of the wetlands?

5. No net loss of wetlands will result?

Information supporting this section of the application can be found in the attached documentation as follows: (document name, page number, etc.) See Ada County Landfill North Ravine Cell Site Certification Report, Section 5.3

02. SITE MAP - In addition to items a through d and any other applicable items e through g, owners and operators shall include as part of the siting application a scaled map. Please attach a scaled map depicting the following:

a. Highways, roads, and adjacent communities;
b. Property boundaries;
c. Total acreage of the site;
d. Off-site and on-site access roads and service roads;
e. Types(s) of land use adjacent to the facility and a description of all facilities on the sites;
f. All water courses, ponds, lakes, reservoirs, canals, irrigation systems, and existing water supplies, within one-quarter (1/4) mile of the proposed facility property lines;
g. High tension power line right-of-way, fuel transmission pipeline right-of-way, and proposed or existing utilities;
h. Proposed or existing fencing;
i. Proposed and existing structures at the facility and within five hundred (500) feet of the facility boundary. This shall include location of employee building and scales (if provided);
j. Direction of prevailing winds.

Professional Engineer or Professional Geologist Certification

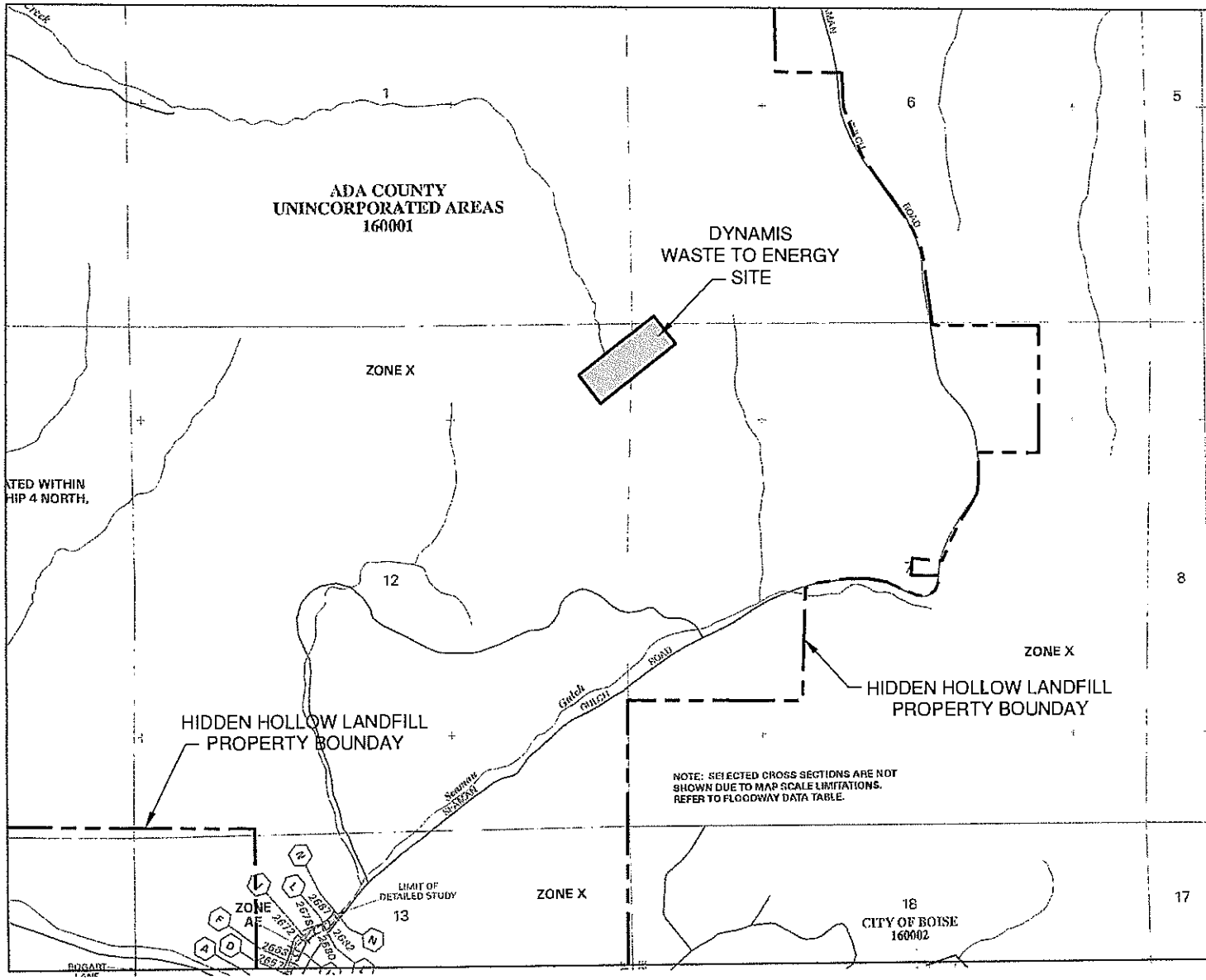
I am a registered professional engineer or registered professional geologist in the State of Idaho and qualified to evaluate and certify compliance with the location criteria for a non-municipal solid waste management facility. I certify that this application was prepared by me or under my direct supervision.

Signature, seal, and date Doyle Pergande 1/9/12

PROFESSIONAL ENGINEER LICENSED P-7141 STATE OF IDAHO DOYLE PERGANDE

02/02/12

TAB 2



P SCALE 1" = 1000'

1000 2000 FEET

METERS

PANEL 0160 H

FIRM

FLOOD INSURANCE RATE MAP

ADA COUNTY,
IDAHO AND
INCORPORATED AREAS

PANEL 160 OF 875

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS

COMMUNITY	NUMBER	PANEL	SUFFIX
ADA COUNTY UNINCORPORATED AREAS	160001	0160	H
BOISE, CITY OF	160002	0160	H

Notice to User: The Map Number shown below should be used when placing map orders; the Community Number shown above should be used on insurance applications for the subject community.

MAP NUMBER
16001C0160 H

MAP REVISED:
FEBRUARY 19, 2003

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

TAB 3

IDAHO DEPARTMENT OF FISH AND GAME
SOUTHWEST REGION
3101 South Powerline Road
Nampa, Idaho 83686

C.L. "Butch" Otter / Governor
Virgil Moore / Director

December 28, 2011

Doyle Pergande
Dynamis Energy, LLC
776 E Riverside Drive, Suite150
Eagle, ID 83616

RE: Sensitive Species Determination for Waste to Energy Plant at Hidden Hollow Landfill

Dear Mr. Pergande,

The Idaho Department of Fish and Game (Department) has reviewed your request for the identification of any federally listed threatened or endangered species, state sensitive species, or critical habitat in the vicinity of the proposed waste to energy plant located at the Hidden Hollow Landfill. The project proposes the construction and operation of a waste to energy electrical generation facility. The Department understands that the site of the proposed facility is within the current operations area and will not result in any additional disturbance to native habitat. The proposed site is located north of Boise, ID in T4N, R1E, Section 12 and T4N, R2E, Section 7.

The Department's records indicate that slickspot peppergrass *(Lepidium papilliferum)*, a plant currently listed as Threatened by the US Fish and Wildlife Service, and Aase's onion *(Allium aaseae)*, a state sensitive species occur adjacent to the current operations area. As proposed, the addition of the waste to energy facility is not expected to impact these species. Please consult with the Department or the US Fish and Wildlife Service if project plans change such that areas of native habitat will be disturbed.

Thank you. Please contact Rick Ward in the Southwest Regional Office at (208) 465-8465, or via e-mail at rick.ward@idfg.idaho.gov if you have any questions.

Sincerely,

Scott Reinecker
Southwest Regional Supervisor

SR/rw
Ecc: Kiefer/ HQ

TAB 4

EAGLE QUADRANGLE
IDAHO-ADA CO.
7.5-MINUTE SERIES (TOPOGRAPHIC)

Scale 1"= 2000'

ENGINEERING
SCIENCE
CONSTRUCTION

Doyle Pergande, P.E.
Dynamis Energy, LLC
776 E Riveside Dr, Suit 160
Eagle, Idaho 83616

Subject: Letter Report
Dynamis WTE Plant Retention Pond Design

Dear Mr. Pergande:

The purpose of this letter is to present our stormwater retention pond calculations and drawings for the Dynamis Waste To Energy (WTE) Plant at the Ada County Landfill. This letter is organized into three sections as follows:

- Background
- Retention Pond Design
- Results

The retention pond calculations and drawings are attached as an appendix.

Background

Engineering Science Construction, P.C. (ESC) was retained by Dynamis Energy, LLC (Dynamis) to produce stormwater retention pond drawings and calculations for the WTE Plant at the Ada County landfill. These calculations and drawings will be available for inclusion in the building permit application.

Retention Pond Design

ESC used the rational method for determining the required retention pond volume for stormwater runoff at the WTE Plant site. The City of Boise Stormwater Design Manual Appendix A (see attached) was referenced for all stormwater retention calculations as recommended by the Ada County Development Services Department. Conservative runoff coefficients from Appendix A – Table 7 of the design manual were used for the analysis and are shown on the attached calculation sheet. The design rainfall intensity for the site is 1 inch per hour (rounded up from 0.9 in/hr) for a 50-year storm event. The required rainfall duration is 1 hour.

The watershed for the project was delineated using a topographical site map. The site includes paved roads and parking, uncovered soil, the roof of the future WTE Plant, and the retention pond as shown on the attached Figure C-01. The total estimated runoff for the design event is 19,073 cubic feet as shown on the attached calculation sheet. An existing depression to the northwest of the site provides an optimal retention pond location. The sides of the depression should be sloped at a minimum 3:1 horizontal to

200 North 4th Street
Suite 300
Boise, ID 83702
208-331-9736 ph
208-331-9739 fax

vertical slope in order to ensure slope stability. With these side slopes, the pond will provide a storage capacity of approximately 27,000 cubic feet.

Results

The pond is located to the northwest of the WTE Plant, adjacent to the roadway intersection as shown in the attached Retention Pond Plan.

If you have any questions regarding the retention pond design or layout, please feel free to contact us.

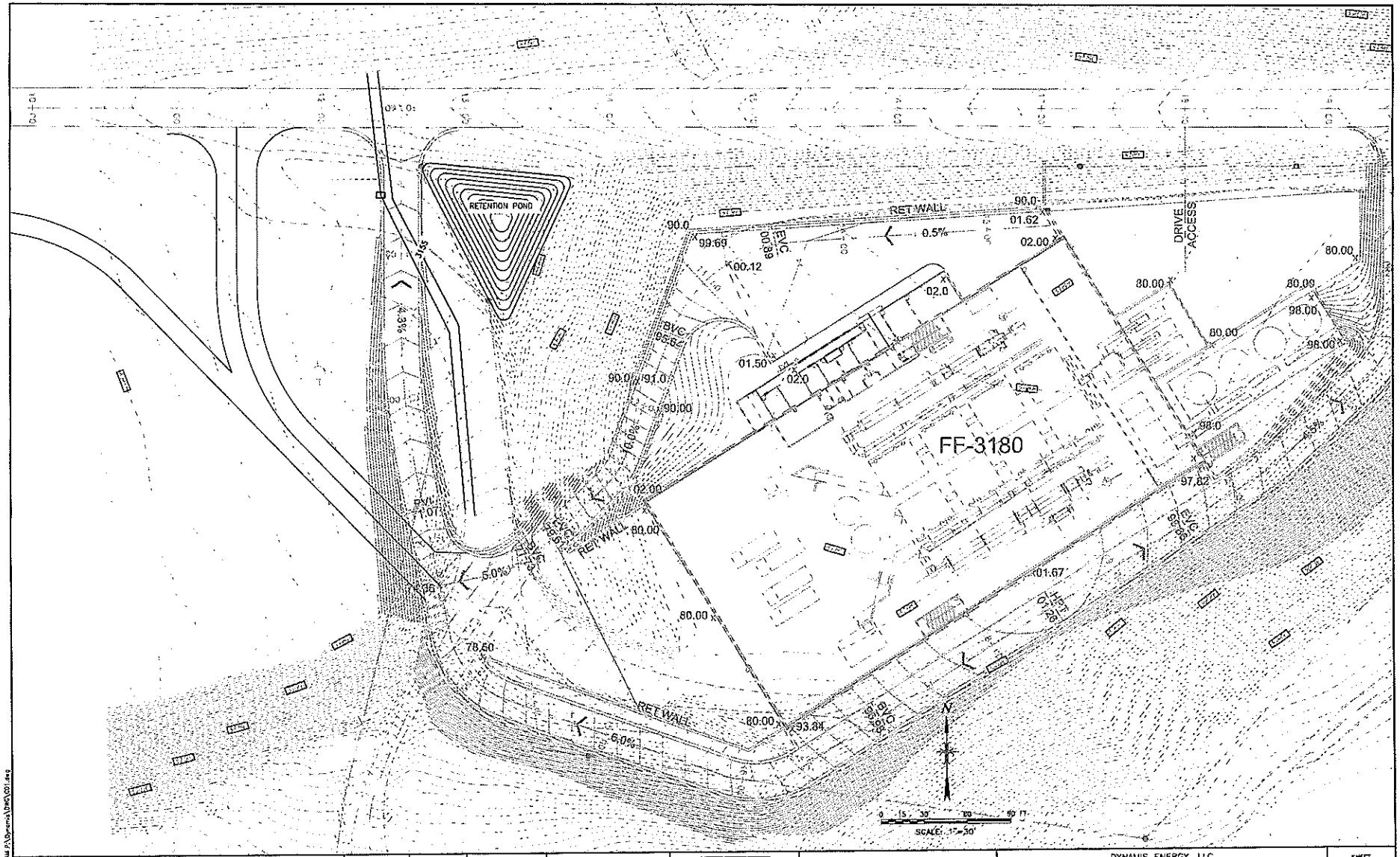
Very truly yours,

Brandon McGoldrick

Brandon McGoldrick
Associate

Si Gilbert, P.E.
Partner

Attachment: Stormwater Calculations
Retention Pond Plan



REV	DATE	BY	DESCRIPTION

SCALE:

WARNING: 0 1/2 — IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE.

DESIGNED: RAC
DRAWN: PDH
CHECKED: BJM

ENGINEERING SCIENCE CONSTRUCTION

DYNAMIS ENERGY, LLC
WASTE TO ENERGY PLANT
RETENTION POND PLAN

SHEET C-01

12/28/2010 11:31 AM P:\Dynamis\DWG\01.dwg

Dynamis WTE Plant - Ada County Landfill
Stormwater Calculations

Design Storm: 50-yr event, 1 hour duration (City of Boise Code)

Design Intensity (i): 1.0 in/hr

Required Retention Storage:

$$V = \Sigma \frac{ciA}{12}$$

Where: A = Area in ft^2
V = Volume in ft^3
c = Runoff Coefficient

Peak Flow:

$$q = ciA$$

Where: A = Area in acres
q = Flow in ft^3/s
c = Runoff Coefficient

C-Values	
Pavement	0.95
Roof	0.95
Exposed Soil	0.15
Parking	0.65
Retention Pond	1.00

Description	Area (ft^2)	Area (acre)	C Value	Contributing Volume (ft^3)	Peak Flow (ft^3/s)
Pavement (Road)	55300	1.27	0.95	4,378	1.21
Roof	77000	1.77	0.95	6,096	1.68
Paved Parking Lot	34400	0.79	0.65	1,863	0.51
Soil	499605	11.47	0.15	6,245	1.72
Retention Pond	5895	0.14	1.00	491	0.14
SUMS	**672,200**	**15.43**		**19,073**	**5.25**

Required Retention Volume ft^3	Peak Flow (cfs)
19,100	5.25

Pond Volume:

A_{top} =	5895 ft^2	
A_{bot} =	172 ft^2	
depth =	9 ft	
Volume =	**27302 ft^3**	> Required Volume; OK

Appendix A

Determining Peak Discharge, Peak Volume, and the Water Quality Volume

Water Quantity

The following is a list of the basic steps to be taken in order to calculate the peak discharge rates from pre- and post-development conditions and the volume of stormwater that must be retained onsite to control for peak discharge rates from specified storms. Two formulas are presented: the Natural Resources Conservation Service (NRCS) TR-55 Method and the Rational Method. Other hydrologic methods may be accepted for determination of runoff rate and volume. However the design professional shall obtain approval from Boise City Public Works Department prior to beginning hydrology studies for the project if an alternate hydrologic method is selected.

The Rational Method should only be used for projects that are less than 10 acres in size. The NRCS TR-55 can be used for projects greater than one acre in size. Many of the steps below have been automated with a computer program and require only data input. For a more detailed account, refer to the NRCS publication, *Urban Hydrology for Small Watersheds.*

A. Steps for Peak Discharge Rate (Rational Method) (calculated for pre- and post-development)

Q_p	=	CiA
A	=	site area (acres)
C	=	dimensionless runoff coefficient
Tc	=	time of concentration
i	=	average rainfall intensity (in./hr) for a duration equal to the time of concentration and for the recurrence interval chosen for design
Q_p	=	peak discharge

1. Determine the contributing drainage (A).

   Use USGS topographic maps, site visits, and other available information.

2. Determine the runoff coefficient (C).

This value is obtained from Table-1 based on pre-development and post-development conditions. For mixed surfaces, determine a weighted coefficient using the following formula:

$$C = [(C1 * A1) + (C2 * A2) + (Cn * An)]/A$$

3. Calculate the time of concentration in minutes (Tc).

The time of concentration, (in. /hr), over a duration equal to the time of concentration for the contributing area can be estimated using the surface flow time curve (Figure-T).

4. Determine the average rainfall intensity (i).

This value obtained from the intensity-duration-frequency curves (Figure-S) based on the time of concentration (Tc) from step #3.

5. Calculate the peak discharge (Q_p).

a) $Q_p = (C) * (i) * (A)$

b) Calculate pre-development Q_p

c) Calculate post-development Q_p

B. Steps to Calculate Onsite Storage Volumes for Control of Peak Discharge Rates (Rational Method)

V_r	=	volume of runoff
C	=	dimensionless runoff coefficient
i	=	average rainfall intensity
T	=	storm duration
A	=	contributing area to site (acres)

1. Calculate the contributing drainage area (A).

Use value from step 1) above.

2. Determine the average rainfall intensity (i).

For the 50 year event, use 1"/hour. For the 100 year event, minimum is 1.1"/hour.

Table-7 Recommended "C" coefficients for "Rational Method Equation"

Modified from ASCE (1972) and the Southeaster Wisconsin Regional Planning Commission

Description of Runoff Area	Runoff Coefficients "C"
Business	
Central business areas	0.70 - 0.95
District and local areas	0.50 - 0.70
Residential	
Single-family	0.35 - 0.45
Multi-family, detached	0.40 - 0.60
Multi-family, attached	0.60 - 0.75
Residential .5 acre lots or larger	0.25 - 0.40
Industrial and Commercial	
Light areas	0.50 - 0.80
Heavy areas	
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.35
Railroad yard areas	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Landscaped areas	0.20

For Impervious Surfaces

Character of Surface	Runoff Coefficient
Streets (asphalt, concrete), Drives and Walks, Roofs	0.90 - 0.95

For Pervious Surfaces

		Runoff Coefficient			
Slope		**A soils**	**B soils**	**C soils**	**D soils**
Flat	0 - 2%	0.04	0.07	0.11	0.15
Average	2 - 6%	0.09	0.12	0.15	0.20
Steep	> 6%	0.13	0.18	0.23	0.28

3. Determine the storm duration (T).

   For this value, use one hour.

4. Determine the runoff coefficient (C).

   Use value from step #2 above.

5. Calculate the volume of runoff (V).

   $V_r = C * (i) * T * A$

C. Steps for the Peak Discharge Rate (TR-55 Method) (calculated for pre- and post- development):

A	=	contributing drainage area to site
RCN	=	runoff curve number
Tc	=	time of concentration
P	=	rainfall from specified event
Ia	=	initial abstraction
Qu	=	unit peak discharge
Q	=	site runoff
Fp	=	pond and wetland adjustment factor
Qp	=	peak discharge

1. Calculate the contributing drainage area.

   Use USGS topographic maps, site visits, and other available information.

2. Calculate the Runoff Curve Number (RCN).

   a) Use NRCS maps and site visits to determine soil types within the drainage area (1).

   b) Determine the Hydrologic Soils Group (HSG) for soils identified in (1).

   c) Determine land use, cover type, treatment, hydrologic condition, % impervious, and % connected/unconnected impervious area ratio.

   d) Develop a composite land use and HSG map from the information in (a)-(c).

e) Select RCNs from appropriate charts (TR-55) (Figure-U).

f) Compute a weighted RCN for the entire drainage area.

3. Calculate the site time of concentration (Tc).

   Determine sheet flow, shallow concentrated flow and channel flow from the most hydraulically distant point in the drainage area to the drainage discharge point at the boundary site.

4. Determine rainfall distribution type.

   Use Type II for Idaho.

5. Determine % of ponds and wetlands in the drainage area.

   Measure from USGS topographic sheet.

6. Select design frequency storms to be evaluated.

7. Determine the 24 hour site rainfall amounts (P) for each design storm.

   Use NOAA Atlas II for Idaho.

8. Determine Initial Abstraction (Ia). Initial abstraction is a representation of interception, initial infiltration, surface depression storage, and evapo-transpiration.

   This value obtained from TR-55 (Figure-U) chart based on the site's RCN.

9. Calculate the Ia/P ratio.

10. Determine the Unit Peak Discharge (Qu).

    This value obtained from TR-55 chart (Figure-U) based on the Ia/P ratio and the Tc.

11. Determine the site runoff (Q).

    This value obtained from TR-55 chart (Figure-U) based on the RCN and the rainfall P.

12. Determine the Pond and Wetland Adjustment Factor (Fp).

This value obtained from TR-55 chart based on percentage of ponds and wetlands.

13. Calculate the final Peak Discharge Rate (Qp) for the site.

a) Q_p = (Qu) * (A) * (Q) * (Fp)

b) Calculate pre-development Q_p

c) Calculate post-development Q_p

D. Steps to Calculate Volume to Store Onsite for Control of Peak Discharge Rates (TR-55 Method)

Qo	=	peak outflow from detention system
Qi	=	peak inflow to detention system
Vs	=	volume of storage for detention system
V_r	=	volume of runoff

1. Calculate the contributing drainage area (1).

   Use value from #1 above.

2. Determine rainfall distribution type.

   Use Type II for Idaho.

3. Select design frequency storms to be evaluated.

4. Determine the peak inflow (Qi) into the water quantity facility (BMP).

   This value is typically the post-development peak discharge rate (Q_p) from #13 above.

5. Determine the peak outflow (Qo) from the water quantity facility (BMP).

   This value is the pre-development peak discharge (Q_p) from #13 above.

6. Calculate the outflow to inflow ratio (Qo/Qi).

   This curve value obtained from the TR-55 graph/chart (Ap-

pendix C) based on the Qo/Qi ratio and the rainfall distribution type. The Vs/V_r ratio will typically be a value between 0.1 and 0.6.

7. Find the volume of storage to volume of runoff (Vs/V_r) ratio.

   This curve value obtained from the TR-55 graph/chart (Appendix C) based on the Qo/Qi ratio and the rainfall distribution type. The Vs/V_r ratio will typically be a value between 0.1 and 0.6.

8. Determine the site runoff (Q).

   Use value from #11 above.

9. Calculate the runoff volume (V_r).

   $V_r = (Q)^* (A)$

10. Calculate the storage volume (Vs) to be allocated for the water quantity facility (BMP).

    This is the volume that must be stored onsite to maintain the peak discharge rates from the specified frequency storm events.

    (a) $Vs = (Vr) * (Vs/V_r)$ [Vs/V_r from #20 above).

11. Repeat steps 7 - 23 to determine the required storage volumes for other required design storm frequencies identified in step#6.

NOTE:

Outlets from detention facilities must be designed to replicate pre-development discharge conditions for all storm events. This may require evaluating additional storm events to properly design pond outlets.

Water Quality

When surface management BMPs are used, they must be designed to treat the Water Quality Volume (V_{wq}). The performance standard for treatment is removal of 80% of the average annual load (post-development conditions) of Total Suspended Solids (TSS). It is presumed that the performance standard for surface management BMPs is met when:

A. Suitable nonstructural practices for source control and pollution prevention are implemented;

B. Stormwater management best management practices (BMPs) are sized to capture the V_{wq} or treat the water quality design flow; and

NOTE:

The application of this standard has been simplified to estimate a site's annual TSS load for compliance with this standard. The calculations have been set up so that every site's annual TSS load entering the first BMP in the system is 1.00 (100%).

C. Stormwater management BMPs are maintained as designed.

1. For each drainage area, list the stormwater BMPs and their order in the engineered system, beginning with the first BMP collecting stormwater from the site. For example, pretreatment and conveyance BMPs will typically precede the removal BMPs. For each drainage area, list the BMPs and their respective order with their estimated TSS removal rates.

2. The TSS removal rates are not additive from one BMP to the next, instead the estimated removal rates must be applied consecutively as the TSS load passes through each BMP. For the purposes of this calculation, represent the estimated annual TSS load as 1.00 (100%).

3. For each drainage area, apply the BMP estimated removal rate in the order that they occur in the stormwater system. The equation for this calculation is:

   Final TSS Removal Rate = (TSS Average Annual Load * BMP 1 Removal Rate) + (Remaining TSS Load After Preceding BMP * BMP 2 Removal Rate) + (Remaining TSS After Preceding BMP * BMP 3 Removal Rate).

4. After all of the BMPs in the initial stormwater system design have been accounted for and their estimated removal rates applied, the Final TSS Removal Rate for each drainage area should be equal to or better than 80% (0.80) or the sliding scale standard, where applicable. If the Final TSS Removal Rate is lower than the Standard for any of the drainage areas, the system should be redesigned to meet the Standards.

Note:

It is imperative to compute the Final TSS Removal Rates for each individual drainage area. Rooftops, if serviced solely by their own BMPs, such as swales, should be considered a separate drainage system.

Water Quality Peak Runoff Rate

Rational Method

The peak runoff rate for the water quality design storm can be calculated using the same steps used for the water quantity storm and substituting the applicable recurrence interval when selecting the value for the average rainfall intensity.

TR-55

The peak runoff rate for the water quality design storm can be calculated using the same steps used for the water quantity storm and substituting the applicable design frequency storms to be evaluated.

Water Quality Volume

Rational Method

$V = CiTA,$

where V_{wq} = water quality volume

C = runoff coefficient for impervious surfaces = 1

i = storm intensity = 0.34"/hour = 0.0283 ft/hr

T = storm duration = 1 hour

A = total impervious area (ft^2)

V_{wq} = #1* (0.0283 ft/hr) * (1 hour) * A = (0.0283 ft2) * A

TR-55

RCN = runoff curve number

Tc = time of concentration

P = rainfall from specified event

Qu = unit peak discharge

Q = site runoff

1. Calculate the contributing drainage area to site (A).

   Use USGS topographic maps, site visits, and other available information.

2. Determine rainfall distribution type.

   Use Type II for Boise City.

3. Select water quality design storm.

   For this value, P= 0.34"/24 hr.

4. Calculate the Runoff Curve Number (RCN) for the impervious areas.

   a) Use NRCS maps and site visits to determine soils and types within A.

   b) Determine the Hydrologic Soils Group (HSG) for soils identified in a).

   c) Determine land use, cover type, treatment, hydrologic condition, % impervious, and % connected unconnected impervious area ratio.

d) Develop a composite land use and HSG map from the information in a)-c).

e) Select RCNs from appropriate charts (TR-55).

f) Compute a weighted RCN for the entire drainage area.

5. Determine the site runoff (Q).

a) This value obtained from TR-55 chart based on the RCN and the rainfall P (inches)

b) Q = [value from 5a)] * A

c) Determine land use, cover type, treatment, hydrologic condition, % impervious, and % connected unconnected impervious area ratio.

d) Develop a composite land use and HSG map from the information in a)- c).

e) Select RCNs from appropriate charts (TR-55).

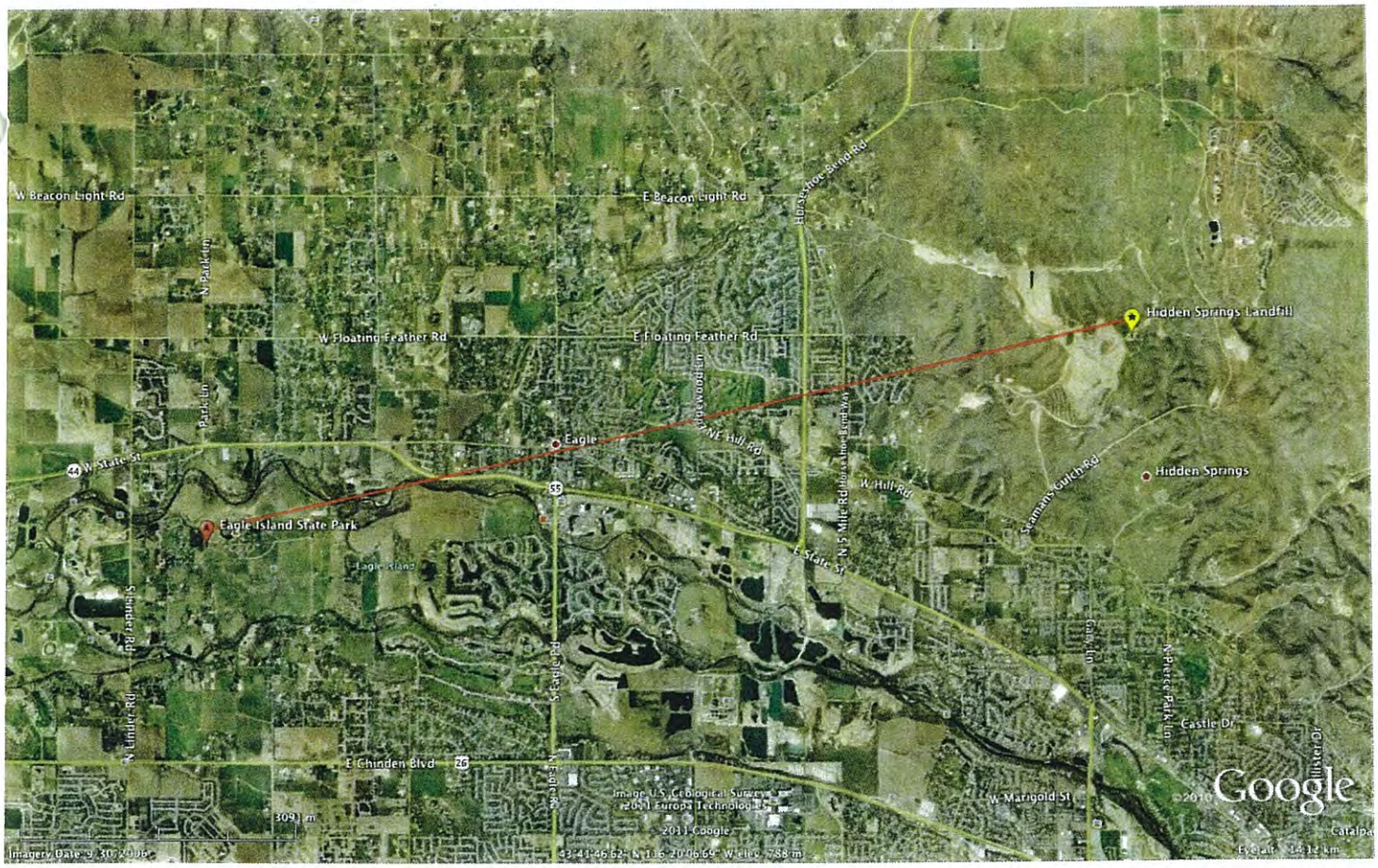
f) Compute a weighted RCN for the entire drainage area.

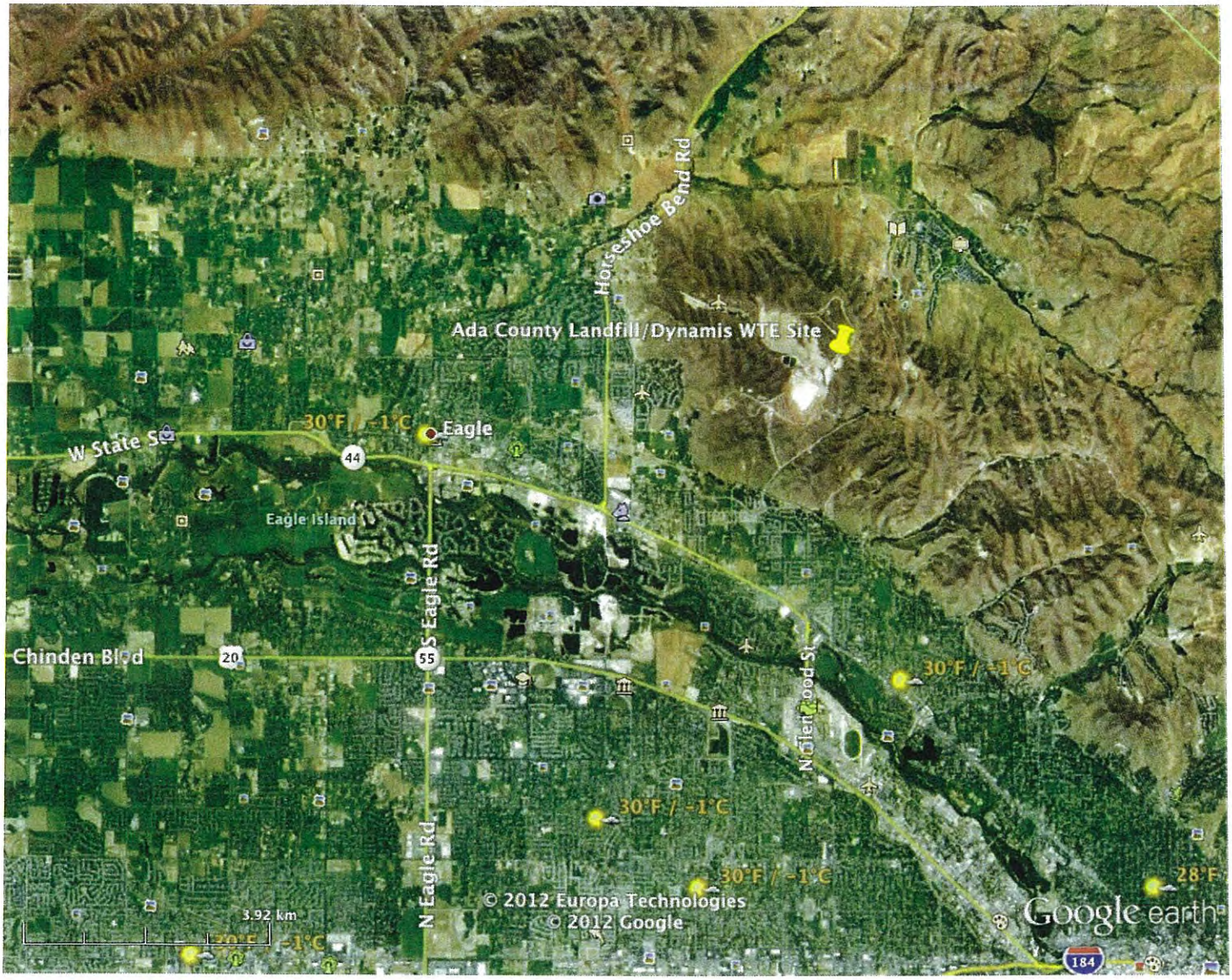
6. Determine the site runoff (Q).

a) This value obtained from TR-55 chart based on the RCN and the rainfall P (inches)

b) Q = [value from 5(a)] * A

TAB 5

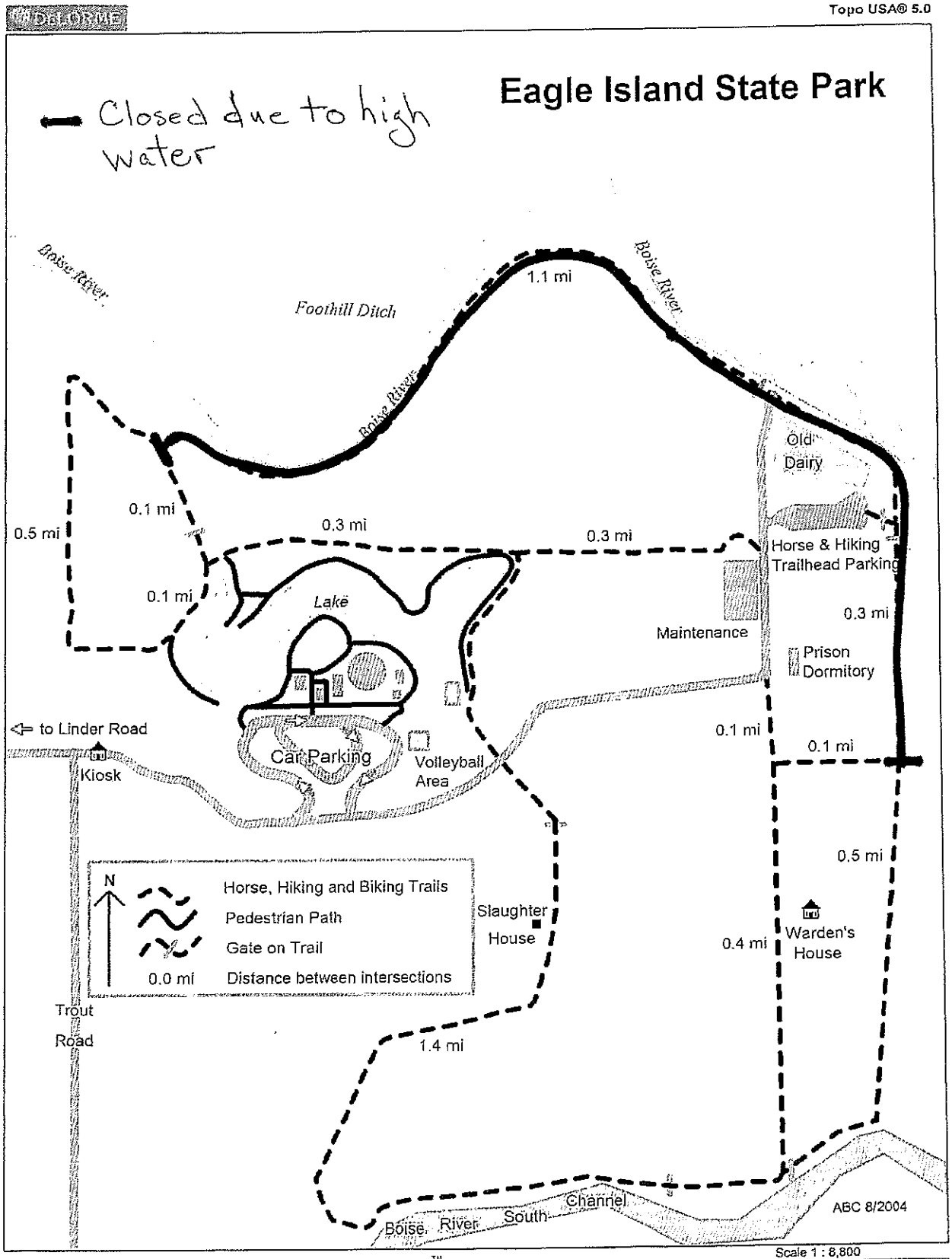

W Beacon Light Rd
E Beacon Light Rd
Hidden Springs Landfill
W Floating Feather Rd
E Floating Feather Rd
Eagle
W State St
Hidden Springs
Eagle Island State Park
W Hull Rd
E State St
Seamans Gulch Rd
Eagle Island
Schuler Rd
N Linder Rd
S Eagle Rd
N Eagle Rd
E Chinden Blvd
W Marigold St
Castle Dr
N Pierce Park Ln
Google



Google earth

miles 6
km 10

Eagle Island State Park



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TN
MN (16.5°E)

Scale 1 : 8,800
1" = 733.3 ft
Data Zoom 14-0

TAB 6

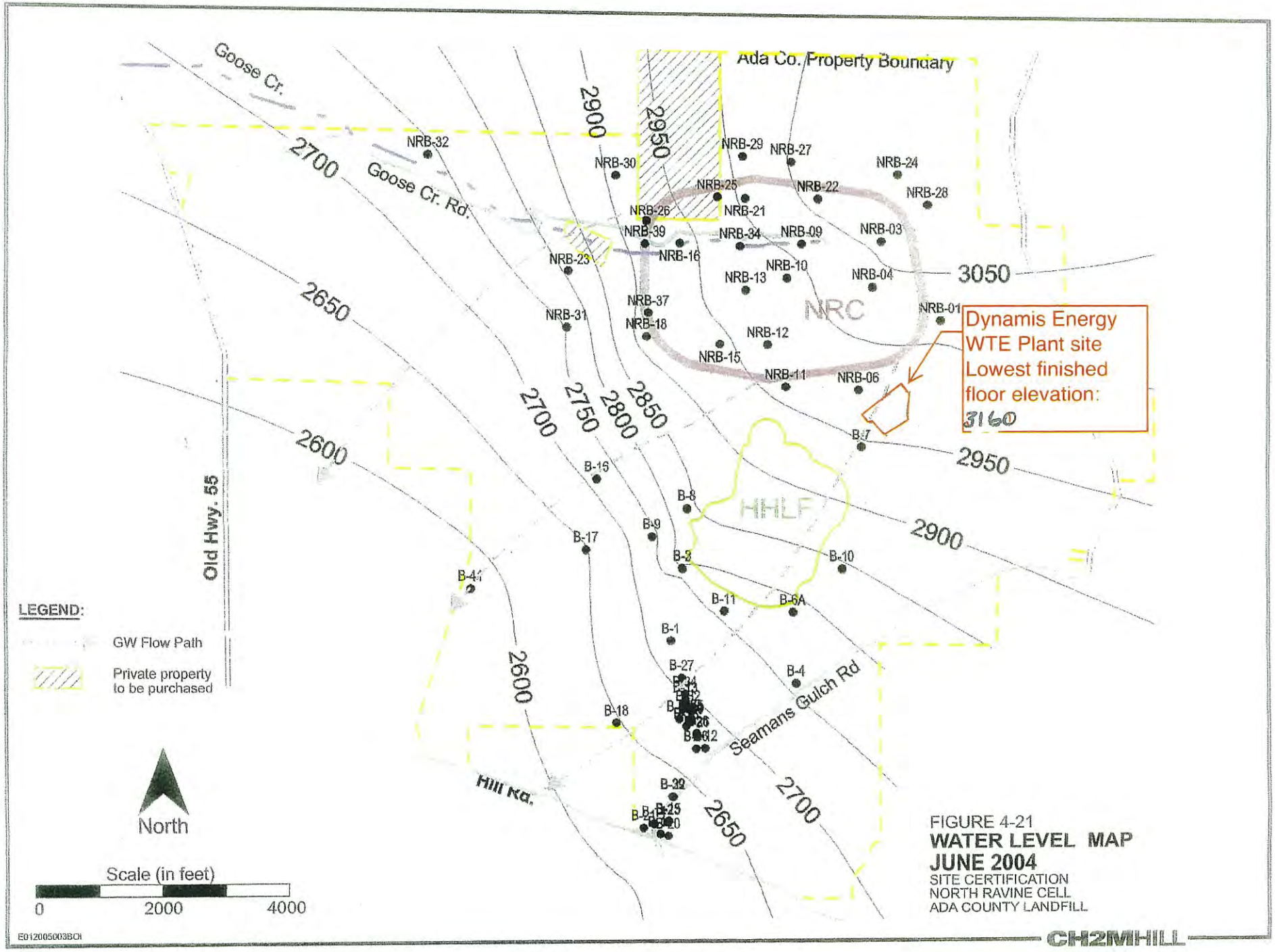

FIGURE 4-21
WATER LEVEL MAP
JUNE 2004
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

E012005003BOI

CH2MHILL

TAB 7

GEOTECHNICAL ENGINEERING EVALUATION
Proposed Ada County Landfill Power Plant
Ada County, Idaho

PREPARED FOR:
Mr. Doyle Pergande
Dynamis Energy, LLC
776 East Riverside Drive, Ste 150
Eagle, Idaho 83616

DYNAMIS
ENERGY

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January 13, 2011

STRATA

A Professional Services Corporation

Integrity from the Ground Up

January 13, 2010
File: DYNENE B10400A

Mr. Doyle Pergande
Dynamis Energy, LLC
776 East Riverside Drive Ste. 150
Eagle, Idaho 83616
doyle@dynamisgroup.com

RE: **Geotechnical Engineering Evaluation**
Proposed Ada County Landfill Power Plant
Ada County, Idaho

Dear Mr. Pergande:

Strata, A Professional Services Corporation, (STRATA) has performed our authorized geotechnical engineering evaluation for the proposed Ada County Landfill Power Plant in Ada County, Idaho. Our geotechnical engineering evaluation's purpose was to explore the subsurface conditions in the proposed area of construction and provide geotechnical recommendations to assist project planning, design and construction. The attached report summarizes our field and laboratory test results and presents our geotechnical engineering opinions and recommendations.

The proposed development area is underlain by a relatively deep deposit of loose to medium dense poorly-graded sand. The following report provides geotechnical recommendations for preparing the site, removing and replacing uncontrolled fill, earthwork activities, foundation design and construction. It is our opinion that geotechnical continuity with the project team throughout construction will help identify uncontrolled fill during earthwork to allow its excavation and recompaction as structural fill below the proposed infrastructure and building footprint.

The project design and construction team must read, understand and implement this report in its entirety. Portions of the report cannot be relied upon individually without the supporting text of remaining sections, appendices and plates. Our opinion is the success of the proposed construction will depend on following the report recommendations, good construction practices, and providing the necessary construction monitoring, testing and consultation to verify that work has been constructed as recommended. We recommend STRATA be retained to provide monitoring, testing, and consultation services to verify our report recommendations are being followed.

We appreciate the opportunity to continue our professional relationship with Dynamis Energy, LLC (Dynamis) and the project design and construction teams. We look forward to our continued involvement on this project throughout construction. Please do not hesitate to contact us if you have any questions or comments.

Sincerely,
STRATA

Kevin P. Ryan, P.E.
Staff Engineer

PROFESSIONAL ENGINEER REGISTERED 10830 1/13/11 STATE OF IDAHO MICHAEL G. WOODWORTH

Michael G. Woodworth, P.E.
Engineering Manager

KPR/nm

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Geotechnical Engineering Evaluation
Proposed Ada County Landfill Power Plant
Ada County, Idaho

INTRODUCTION

Our geotechnical engineering evaluation's purpose was to explore the subsurface soil conditions in the vicinity of the proposed power plant at the Ada County Landfill and to prepare geotechnical recommendations to assist project planning, design and construction. We accomplished our services referencing our authorized geotechnical scope dated December 6, 2010. To accomplish our evaluation, STRATA performed the following services:

1. We reviewed data provided by Dynamis that included preliminary loading information, a site diagram, and general boring logs/cross sections for the Ada County Landfill.

2. We coordinated exploration with the Digline Utility Notification Center to help reduce the potential for damage to existing utilities. We also requested assistance from Dynamis in locating test pits in the field and gaining site access.

3. We coordinated with the Central District Health Department for the evaluation of the on-site soils with regard to the proposed septic system drainfield.

4. We observed 5 exploratory test pits in the proposed power plant and septic system areas outlined to us by Dynamis. Exploratory test pits extended 10 to 15 feet below existing grades. Our staff engineer visually described, classified and logged soil encountered referencing the *Unified Soil Classification System* (USCS). Test pit locations were loosely backfilled with on site soil and their location staked.

5. We accomplished 2 field infiltration tests to evaluate the near surface soils' infiltration characteristics. We also correlated grain size distribution laboratory test results to regional stormwater correlations to help identify the potential stormwater disposal methods.

6. We performed laboratory tests with reference to *ASTM International* (ASTM) procedures including in-situ moisture and density, pH/resistivity and gradation testing. We utilized these laboratory results to help characterize engineering parameters and to correlate soil engineering characteristics used in our design.

7. We performed engineering analyses in order to provide geotechnical design and earthwork construction recommendations. Our engineering analyses provides geotechnical recommendations for:

   - Earthwork
     - Site and subgrade preparation
     - Structural fill criteria
     - Compaction requirements
     - Wet weather construction

- Excavation characteristics
- Recommended unit weight and friction angle of soil
- Corrosivity evaluation of soil

- Conventional foundation design
  - Total and differential settlement estimates
  - Friction coefficients
  - Design factors of safety
  - Allowable bearing pressures
  - Static and seismic lateral earth pressures
  - Soil improvement applications

- Site surface drainage
  - Foundation drainage
  - Stormwater disposal
  - Septic drainfield recommendations
  - Groundwater elevations
  - Recommended infiltration rates
  - Drainage requirements for site grading

- Pavement
  - Recommended asphalt and concrete sections
  - Subgrade preparation requirements

- Concrete slabs-on-grade
  - Minimum support section
  - Vapor retarders
  - Modulus of subgrade reaction

- Additional recommended services
  - Geotechnical design continuity
  - Plan and specification review
  - Geotechnical observation during construction

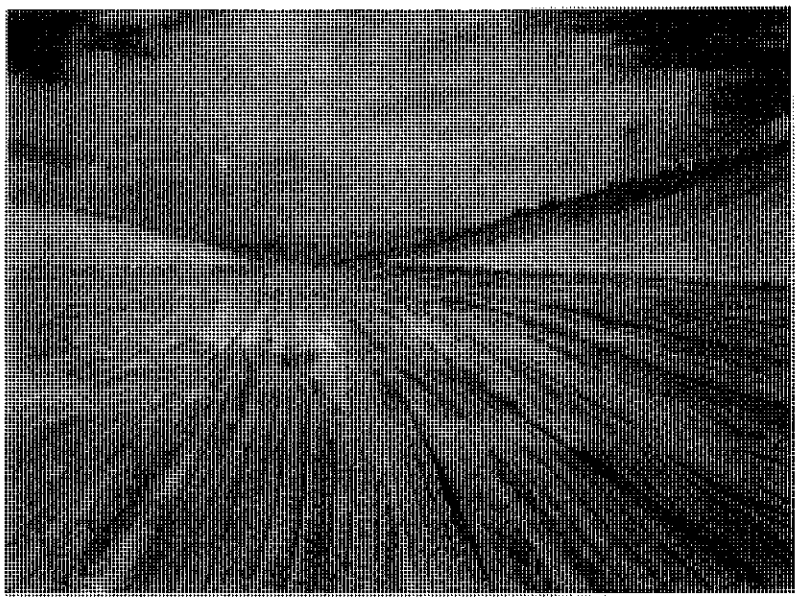
8. Prepared and provided 3 copies of our final report of geotechnical findings and opinions including exploration logs and laboratory test results.

PROJECT UNDERSTANDING

Existing Site Conditions

The proposed power plant site is located within the Ada County Landfill north of the new scale facility in Ada County, Idaho. The project site occupies approximately 3.5 acres, and presently exists as graded vacant parcel. We understand that a prior contractor had excavated an undetermined amount of sand and backfilled the excavation near the center of the proposed power plant.

The project site is relatively flat, with only minor grade changes in the proposed building footprint. However, the site's southern, eastern, and western borders are sloped. The proposed septic system drainfield is located south of the site. The following photograph shows the site's appearance in August of 2010:



Photograph 1: Near the center of the project site, facing north.

In addition to the above photograph, we reviewed the site plan, boring logs, and cross section data provided by Dynamis.

Proposed Construction

We understand Dynamis plans to construct an approximate 20MW power plant at the Ada County Landfill. Dynamis anticipates the power plant will utilize a controlled air gasification process, which thermally converts waste products into combustible gas. The proposed power plant will be capable of utilizing 5 to 325 tons of waste per 12-hour period. The power plant will encompass an approximate 30,000 square foot area, with an additional cooling tower that will be approximately 60 feet by 40 feet.

A foundation loading plan (DWG No. 10-05-784-250) provided by Dynamis Energy indicated equipment loads up to 160 kips. The elevation within the building footprint varies from 3,187 to 3,189 feet. We anticipate that the proposed finished floor elevation will be approximately 3,180 feet and that some filled areas will be at 3,202 feet in elevation. The eastern and western walls could to retain approximately 20 feet of soil.

Dynamis indicated that approximately 50 garbage trucks a day will access the site. A few passenger vehicles, used by employees of the power plant are anticipated. Site stormwater will be directed to below grade seepage beds or surface swales.

SITE GEOLOGIC SETTING

Regional Geology

The site is located on sand of the Pierce Gulch Formation and is described as pale yellow-gray arkosic sand overlain by pebble to cobble gravel. Sand includes cross-bedded and forest-bedded sequences (Othberg & Stanford, 1992). Boring logs and cross section data for Ada County Landfill provided by Dynamis indicate yellow to white coarse-grained sand with occasional interbedded layers of silty to clayey fine sand.

Seismicity and Faulting

Based on our geologic research, 2 faults are mapped within the region. The closest active, major faults are the Squaw Creek fault and the Big Flat-Jakes Creek fault located 30 to 40 miles northwest from the project site (USGS 2002). The following table illustrates fault parameters:

Table 1. Fault Parameters (USGS)

Fault	Characteristic Earthquake Magnitude	Recurrence Interval for Characteristic Magnitude (years)	Recurrence Interval for M≥ 6.5 Event (years)
Squaw Creek	7.1	14,184	4,975
Big Flat-Jakes Creek	6.8	24,213	14,388

We expect the 2006 IBC will be utilized for project structural design. IBC Section 1615.1 outlines the procedure for evaluating site ground motions and design spectral response accelerations. STRATA utilized site soil and geologic data and the project location to establish earthquake-loading criteria at the site referencing IBC Section 1615.1. Based on our field exploration and knowledge of the upper 100 feet of the soil profile, we recommend a Site Class D be utilized as a basis for structural seismic design.

SUBSURFACE EVALUATION PROCEDURES

STRATA observed 5 exploratory test pits on December 22, 2010 in the locations illustrated on Plate 1. Our field engineer visually evaluated the soil encountered in each test pit and logged the soil profile referencing the USCS. We provide a brief USCS explanation in Appendix A to help interpret the terms on the test pit logs. We also provide individual test pit logs in Appendix A. At the conclusion of our subsurface evaluation, test pits were loosely backfilled level with the existing ground surface and we designated test pit locations in the field with labeled stakes for surveying their exact locations.

We performed in-situ infiltration tests to assist in evaluating stormwater disposal infiltration rates. STRATA performed tests in test pits TP-2 and TP-4. We accomplished these tests at 4.5 to 5 feet below the existing ground surface and provide the field infiltration rate measured on the test pit logs.

SUBSURFACE CONDITIONS

Generally, we encountered near surface, poorly-graded sand. We did not observe vegetation and roots at the site. However, very loose sand existed in the upper 6 to 8 inches in each test pit. The poorly graded sand extended to the depth of each test pit. Slightly stained yellow to red bands were observed in each test pit at depths of 4.5 to 8 feet below ground surface. We provide specific soil descriptions and contacts on the test pit logs in Appendix A, *Exploration Logs and USCS*.

We did not encounter groundwater within the depths explored for TP-1, TP-2, TP-3, and TP-5. However, we did encounter groundwater seepage in TP-4 at 3.5 to 4.0 feet below ground surface. Depths below 5 to 6 feet appeared to be dryer and without seeps. At the time of exploration the site was covered with approximately 1 foot of snow. Based on our knowledge of the area, involvement with other landfill projects, and well log review, we anticipate groundwater will typically occur at depths well below 15 to 20 throughout the year. However, local perched aquifers could exist above clay or silt lenses which may result in near surface groundwater.

Test pit exploration only allows us to observe a small portion of the site subsurface conditions. Subsurface variations are possible between test pits and may not be apparent until construction. Where such variations exist, they may impact the opinions and recommendations presented in this report, as well as construction timing and costs. During site exploration, we

did not encounter uncontrolled fill. However, based on our conversations with Mr. Doyle Pergande on December 22, 2011, we understand that some uncontrolled fill may exist under the building footprint.

LABORATORY TESTING

We tested select soil samples obtained during subsurface exploration to assess some soil profile engineering characteristics. Laboratory tests included: in-situ density and moisture content, gradation, and pH/resistivity. We accomplished laboratory testing in reference to ASTM testing procedures and present these results in Appendix B, *Laboratory Test Results* and on exploration logs.

GEOTECHNICAL OPINIONS AND RECOMMENDATIONS

We present the following geotechnical recommendations to assist planning, design and construction of the proposed Ada County Landfill Power Plant, as illustrated on Plate 1 attached to this report. This report provides specific foundation and other geotechnical design criteria for the power plant, which the structural and civil design and construction teams must review to verify the applicability to the planned structure. We base our recommendations on the results of our field evaluation, laboratory testing, our experience with similar soil conditions and our understanding of the proposed construction. If design plans change or if the subsurface conditions encountered during construction vary from those observed during our field evaluation, we must be notified to review the report recommendations and make necessary revisions.

Earthwork

Site Preparation

Although not observed during site exploration, uncontrolled fill may exist on the site as noted by Dynamis. Uncontrolled fill has the potential to settle below new foundations and pavements and such settlement could negatively impact their performance. We recommend STRATA work with the earthwork and general contractors to help identify fill areas.

Following uncontrolled fill removal, excavate the exposed subgrade to the project design elevations and tolerances. Scarify and moisture-condition the finished subgrade to within 3 percent of optimum moisture content to at least 8 inches prior to compaction effort. Moisture-conditioning may include aeration or adding moisture to achieve within 3 percent of optimum moisture content. From exploration, we expect subgrade soil will be poorly-graded sand. To

improve support characteristics, the scarified and moisture conditioned subgrade soil should be compacted to at least 92 percent of ASTM D 1557 (Modified Proctor).

Excavation Characteristics

Based on exploration results, it appears the near surface soil encountered in exploratory test pits may be excavated with conventional equipment. Excavations, especially in sand can cave and slough and must be sloped back in accordance with *Occupational Health and Safety Act* (OSHA) guidelines. Coarse-grained soil and uncontrolled fill is expected to be exposed in excavations throughout the development area and should be temporarily sloped at 1.5H:1V (horizontal to vertical). We recommend earthwork contractors evaluate each excavation configuration specific to OSHA guidelines and to seek appropriate professional guidance to ensure excavation safety and stability. From exploration results and our experience in the area, we do not expect groundwater will be encountered within the depths anticipated for the planned construction unless perched aquifer conditions are encountered. However, surface grading should preclude stormwater from infiltrating excavations and contractors should have contingencies to rapidly remove water that accumulates in excavations. Recommendations for foundation drains and other measures to reduce infiltration at the subgrade are included in the subsequent report text.

Wet Weather/Soil Construction

We recommend earthwork construction take place during dry weather conditions. If construction commences before soil can dry, after precipitation, or during wet periods of the year (November through May), complete earthwork with low pressure, track-mounted equipment that spread and reduce vehicle load. During construction, the earthwork contractor should intersect and divert surface runoff from rainfall or snowmelt to help reduce water ponding on the project site. We recommend that subgrades slope and daylight to help direct water away from subgrades after the end of each construction day or before precipitation.

Structural Fill

All fill placed for the development must be placed as structural fill. The structural fill requirements described in Table 2 below, in general correlate to Idaho Transportation Department (ITD) material specifications. Project structural fill products are described in Table 2 below.

Table 2. Structural Fill Specifications and Allowable Use

Structural Fill Product	Allowable Use	Material Specifications
Granular Structural Fill	General site grading, utilities and fill placement, over-excavations, temporary haul roads, temporary platforms, granular subbase, general structural fill	• Soil must be classified as sand or gravel (GP, GW, SP, or SW) according to the USCS • Soil may not contain particles larger than 6–inches in diameter. • Soil meeting the latest requirements in ITD-SSHC[1] *Section 703.11-Aggregate for Granular Subbase*
Aggregate Base Course[2]	Foundation and slab support, soil improvements, asphalt pavement section aggregate, general structural fill	• Soil must meet granular structural fill requirements • Soil meeting the latest requirements in ITD-SSHC[1] *Section 703.04 – Crushed Aggregate*

[1]Idaho Transportation Department- Standard Specifications for Highway Construction

[2]Includes both 1-1/4-inch and ¾-inch nominal maximum particle size aggregate

All structural fill must be compacted to a minimum of 95 percent of the maximum dry density of the soil referencing ASTM D 1557 (Modified Proctor). Fill placed outside any building or pavement envelope can be placed as non-structural fill (i.e. landscape fill) providing there are no structures (sidewalk, curbs, signs, etc.) planned directly above the landscape fill. We recommend landscape fill be compacted to a minimum of 85 percent of the maximum dry density of the soil according to ASTM D 1557.

Any structural fill products must be moisture-conditioned to near optimum moisture content and placed in maximum 10-inch-thick, loose lifts. The above assumes large, appropriate compaction equipment with drum energy of at least 10 tons or greater is used to attempt compaction. If smaller or lighter compaction equipment is provided, the lift thickness may have to be reduced to meet the compaction requirements presented herein. The site soil is expected to be suitable for reuse as structural fill, providing it can meet the criteria presented in Table 1 above and can be properly moisture-conditioned for compaction.

Utility Trench Construction

Structural fill for backfilling utility trenches and all bedding should conform to section 703 in the ITD-SSHC, except that all trench backfill must be placed and compacted to the structural fill requirements presented herein. Loose soil must be removed from the base of utility trenches prior to placing pipe bedding. If water is encountered, it must be removed from the base of the utility

trench before placing pipe bedding. We recommend utility pipes be placed on at least 4 inches of bedding conforming to 703.02 of the ITD-SSHC placed over undisturbed native soil, structural fill or otherwise supported according to the pipe manufacturer's specifications and ITD-SSHC requirements.

After bedding the pipe, place structural fill and compact it from the pipe invert to 1 foot above the top of the pipe with tamping bars and/or plate compactors to render the backfill in a firm and unyielding condition. Thoroughly place and compact bedding below pipe haunches or the zone between the pipe invert and the spring line. To accomplish backfilling, the distance between the side of the pipe at the spring line and the trench wall should be at least 12 inches. The remainder of the utility trench should be backfilled in accordance with the *Structural Fill* section of this report.

Foundation Design

Bearing Soil

We understand from the Site Plan provided by Dynamis, that the power plant will be constructed near the center of the site and abutting the eastern border. The foundation bearing elevation will occur at approximately 3,175 to 3,180 feet or 10 to 15 feet below existing grade. From explorations, we expect the foundation excavations will expose loose to medium dense native poorly graded sand. We recommend 12 inches of granular soil improvements below foundations to help provide a uniform bearing surface and improve constructability. These improvements must extend 1 foot laterally beyond the foundation edges and 1 foot below the foundation bearing surface as shown on *Plate 2, Soil Improvement Schematic.*

General

We recommend STRATA be retained to observe the foundation system installation including reviewing soil improvements and structural fill compaction prior to placing concrete forms or concrete. Reviewing the soil improvement process and final foundation bearing surfaces helps confirm our allowable bearing pressures and settlement estimates and is an important part of the geotechnical design process.

Exterior footings must extend at least 24 inches below the final exterior ground surface to help protect against frost action. Interior foundations must extend at least 18 inches below final slab-bearing elevations and maintain at least 4 inches of gravel between slabs and the top of the footing to reduce the reflective cracking potential. Foundations must be structurally designed to conform to the latest edition of the *International Building Code* (IBC). The foundation bearing pressures presented below can be increased 30 percent to account for transitory live loads such as

seismic and wind. In our opinion, long-term live loads such as equipment, fixtures, furniture, files, etc. should be considered in the total dead structural loads for the project. Our analysis utilizes a factor of safety against bearing capacity failure of 3.0 or greater. Settlement estimates and other design criteria are unfactored. If loading conditions vary from those presented to STRATA, we recommend that we be consulted to review our recommendations.

Design Criteria

Foundations constructed on granular soil improvements as presented in this report may be designed using a maximum allowable bearing pressure of 3,000 pounds per square foot (psf). Mass concrete placed on soil improvements over compacted subgrades can utilize a friction coefficient (f_s) of 0.45 to resist lateral loads. This coefficient must be reduced by ⅔ if concrete is not cast directly on soil such as for pre-cast panels.

Using good construction practices and constructing during good weather, we estimate foundations bearing on subgrades, prepared as recommended herein, will realize 1 inch of settlement. Our settlement estimates utilize 7-foot-square foundation. Foundation dimensions less than these should be analyzed for changes in settlement potential. Where water accumulates at the foundation elevation, settlement can be in excess of our estimates and the structure tolerances. Therefore, we recommend exterior grading provide adequate drainage away from the building. We recommend subsurface alignment pits be drained with foundation drains as shown on *Plate 3, Foundation/Wall Drain Schematic.*

Corrosivity

Based on our experience in the project vicinity and the laboratory test, the site soil has slightly alkaline pH and high resistivity. Therefore, we anticipate a negligible to slight corrosion environment. As with any below grade construction, careful selection of material for utility piping and other structure materials must account for some potential wall thickness loss due to corrosion. Concrete reinforcing steel should maintain appropriate IBC earth and form clearances at all times. Position reinforcing steel with the maximum available clearance to reduce potential corrosion effects. Our experience in the project vicinity and with regional concrete suppliers indicates site soil will not react with concrete products produced in the area. To the best of our knowledge, silica-alkali reactions have not been observed in the Treasure Valley.

Lateral Earth Pressures

Below grade foundation and wall systems must be designed to resist lateral earth pressures from the retained soil behind the structure and surcharge from equipment, slopes or vehicles adjacent to the walls. For this project, retaining walls must be designed to resist lateral earth pressures. We recommend lateral earth pressures for conventional wall systems be estimated using the following equivalent fluid pressures (EFPs) from Table 3 for native soil or imported structural backfill soil meeting the requirements for general structural fill outlined in this report's *Structural Fill* section.

Table 3. Coulomb Equivalent Fluid Pressures (native soil backfill)[1]

Coulomb Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)
At-rest case (no wall movement)	55 pcf
Active case (wall movement away from soil mass)	35 pcf
Passive case (wall movement toward soil mass)	350 pcf [1]

[1]Has been corrected for 1/2-inch of lateral deflection.

The above values do not take into account the effect of hydrostatic pressure, and below-grade alignment pit walls must be drained according to Plate 3. Lateral surcharge pressures due to equipment, slopes, storage loads, etc. are not included in the above lateral earth pressure recommendations. The lateral earth pressure coefficient of 0.5, acting over the entire wall height, could be used to estimate the lateral earth pressure induced on walls due to adjacent surcharge loads from equipment and the slope behind the structure. We anticipate that the above active and passive EFPs will require horizontal wall deflections of approximately 0.2 percent and 0.6 percent of the final wall height respectively. If horizontal wall deflections required to achieve the active and passive lateral earth pressures are not acceptable, we recommend that at rest EFP be used for design. Additionally, if the basement walls are structurally restrained, at-rest pressures should be used for design.

Dynamic lateral earth pressures are a function of the presence of groundwater, magnitude of ground shaking, soil strength and soil permeability. Dynamic lateral earth pressures are *additive* to the above static lateral earth pressures, but act as an inverted triangle. Table 4 is the equivalent fluid pressures during dynamic loading (excludes static loads) for non-saturated soil. The seismic component of pressure is assumed to have its resultant acting at 0.6 times the wall height measured from the base of the wall.

Table 4. Mononobe-Okabe Dynamic Equivalent Fluid Pressures (submerged conditions)

Coulomb Lateral Earth Pressure Case	Equivalent Fluid Pressure (EFP)
At rest case (no wall movement)	+30 pcf
Active case (wall movement away from soil mass)	+12 pcf
Passive case[2] (wall movement toward soil mass)	-75 pcf

1 – EFP includes the buoyant soil unit weight and excludes the unit weight of water.
2 – Passive resistance has been provided for ½-inch of lateral movement.

Care must be taken in the use of heavy equipment near the face of walls (in a zone extending 5 feet back from the wall) to avoid creating an undesirable degree of over-compaction or lateral wall loading from the soil immediately along the walls and imposing high stresses on the walls. Below-grade walls should be backfilled as described in the *Structural Fill* section of this report.

Concrete Slab-on-Grade Floors

Concrete slab-on-grade floors should be supported by compacted crushed aggregate base course placed on a prepared subgrade, as described in this report's *Site Preparation* section. We recommend concrete slab-on-grade floors exposed to typical pedestrian and light storage loads be underlain by at least 8 inches of crushed aggregate base course to provide a leveling course and capillary break for the slab. Subgrade areas that become soft, wet or disturbed or that cannot be recompacted to structural fill requirements must be over-excavated to firm soil and replaced with granular structural fill prior to placing aggregate base.

Floor slabs must be designed for the anticipated use and equipment or storage loading conditions. Based on correlations to our field and laboratory test results, we recommend concrete slab design utilize a modulus of subgrade reaction (K) of 200 pounds per cubic inch (pci) for poorly-graded sand. To realize the reported modulus of subgrade reaction, drained conditions and the recommended slab support section and subgrades must be provided.

Interior floor slabs may be susceptible to moisture migration caused by capillary action and vapor pressure. If the proposed concrete slabs will receive floor coverings that are sensitive to moisture, additional moisture protection must be considered beneath the concrete slab-on-grade floors. We expect these floor coverings may exist within the retail area where tile, vinyl or other "impervious" coverings may be used. In shop areas where no floor coverings are

expected, a vapor retarder may not be necessary. Where utilized, vapor retarders must consist of a thick, 15-mil, puncture-resistant, polyethylene sheeting placed between a sand cushion and the crushed surfacing. An example of a common vapor retarder is Stego Wrap™, a 15-mil vapor retarder. The sand cushion should be 2 inches of clean, coarse sand conforming to the following gradation:

Table 5. Sand Cushion Gradation

Sieve Size	Percent Passing
No. 4	85-100
No. 8	75-95
No. 16	55-75
No. 50	25-45
No. 200	6-12

The location of the vapor retarder (if a vapor retarder is specified) should be carefully considered by the owner and architect. ASTM E1643 and *American Concrete Institute* (ACI) *Committee 302* are two publications that provide considerations for vapor retarder locations.

Form stakes or other sub-slab penetrations must never be allowed to puncture the vapor retarder. Manufacturer recommendations for proper sealing slab-to-wall connections, plumbing or other penetrations must be strictly followed. Although these recommendations are used, water vapor migration through the concrete floor slab is still possible. Floor covering must be selected accordingly and manufacturer's recommendations strictly followed.

Exterior slabs are susceptible to frost action which can generate substantial frost heave at certain times of the year. The potential for frost heave may not be acceptable at entries, work bays or other critical areas adjacent to the building that will be exposed to weather. One approach to provide partial frost protection requires removing 65 percent of material within the frost depth and replacing it with granular structural fill. If this method is employed, the over-excavated soil must be replaced with aggregate base course as specified in the *Structural Fill* section. Alternatively, if partial frost protection is unacceptable, over-excavation and aggregate base course replacement must be accomplished to the anticipated frost depth (24 inches).

Surface Drainage Conditions

Stormwater Disposal

We performed infiltration testing within native, poorly-graded gravel encountered at depth across the site. We observed an approximate infiltration rate of 20 to 25 inches per hour during testing. We recommend infiltration facilities extend into native, poorly-graded sand soil encountered during our exploration. We recommend all subsurface infiltration facilities be designed using a design infiltration rate of 8 inches per hour. Further, a minimum 3-foot separation between the base of the infiltration facility and estimated seasonal high groundwater must be maintained. We estimate seasonal high groundwater will likely occur below elevation 3,170 feet. However, as noted in this report's *Subsurface Conditions* section, perched groundwater has the potential to exist in portions of the site.

Exterior Grading

We recommend the ground surface outside of any structure be sloped a minimum of 5 percent away for 10 feet to rapidly convey surface water or roof runoff away from foundations. Remaining landscapes should slope at least 2 percent away from structures. Roof downspouts must be provided and connected to a solid pipe placed away from structures and not allowed to infiltrate into the soil underlying the structure. Stormwater should be routed away from disturbed soil areas and should be disposed of in a suitable location as determined by the site civil engineer.

Pavement Subgrade Preparation and Section Design

General

The following flexible asphalt and rigid pavement sections design is provided referencing the *American Association of State Highway and Transportation Officials* (AASHTO) *Guide for Design of Pavement Structures* (1993). STRATA estimated traffic loading and design parameters based on our proposed construction understanding, results from laboratory testing and our understanding of the subsurface conditions. The flexible pavement design is for access roads to and at the site. The rigid pavement design is for the unloading area within the building. We understand that Dynamis intends to utilize concrete additives to increase the wear resistance of the rigid pavements section. We have not provided a concrete recommendation for increased wear effects.

Traffic and Subgrade

The following tables present our traffic loading, geotechnical design parameters and references as well as the resulting flexible pavement and rigid sections design recommendations.

Table 6. Pavement Design Parameters

Design Parameter	Value Used	Reference
Reliability (R)	90%	Assumed
Standard Deviation (S)	0.45 (flexible) 0.35 (rigid)	AASHTO 1993
Flexible Pavement Traffic Loading	2,400,000 ESALS[1] (Heavy-Duty)	See Table 7 below
Rigid Pavement Traffic Loading	9,700,000 ESALS[1] (Heavy Duty)	See Table 7 below
Initial Serviceability (PSI_i)	4.2	Typical regional values
Terminal Serviceability (PSI_z)	2.1	Typical regional values
Design Serviceability Loss	2.1	Typical regional values
Design Life	20 years (flexible) 40 years (rigid)	Assumed
Resilient Modulus (M_r)	18,000 psi[2]	Based on CBR and M_r correlations (see paragraph below)
Asphalt Layer Coefficient (a_1)	0.42	Figure 2.5 AASHTO 1993
Top Course Layer Coefficient (a_2)	0.12	Figure 2.6 AASHTO 1993
Top Course Drainage Coefficient (m_2)	1.0	Table 2.4 AASHTO 1993 for "fair" drainage, 1 to 8 percent saturation
Modulus of Subgrade Reaction (k)	200 pci[3, 4]	Figure 3.3 AASHTO 1993 assuming 6 inches of top course
Concrete Elastic Modulus (E_c)	4.2×10^6 psi	Typical values for 5,600 psi[1] compressive strength concrete
Load Transfer Coefficient (J)	4.2	Table 2.6 AASHTO 1993
Drainage Coefficient (c_d)	1.05	Table 2.5 AASHTO 1993 for "fair" drainage 1 to 5 percent saturation

1 – Equivalent Single Axle Loads (ESALs)
2 – Pounds per square inch (psi)
3 – Pounds per square inch, per inch of subgrade (pounds per cubic inch, pci)
4 – Includes 6 inches new aggregate base above soil subgrade

Table 7. Traffic Loading Assumptions

Pavement Section Area	Traffic Loading Parameters	Frequency[1] or Value Used	EALF[2]	ESALS
Heavy-Duty Section (drive lanes and truck lanes)	Passenger Vehicles	10 trips per day	0.004	355
	Garbage, Trucks (Flexible)	50 trips per day	7.64	2,400,000
	Garbage, Trucks (Rigid)	50 trips per day	12.36	9,700,000
	Annual Growth Factor	2.0%		
	Construction Traffic[3]	**None**		

[1]One trip is one complete pass by the vehicle.
[2]Equivalent Axle Load Factor; Loading by one vehicle trip.
[3]Construction traffic is not included in design.

From correlations to index laboratory testing, we estimate the poorly-graded sand subgrade for proposed asphalt areas will have a CBR value of 12 and a resilient modulus (M_r) correlation of 18,000 pounds per square inch. To help improve subgrade characteristics, the pavement subgrade should be prepared as recommended in this report's *Site Preparation* section. Subgrades must be shaped (crowned) and graded to facilitate positive drainage and inverted crowns must be avoided.

Asphalt, PCC Concrete, and Aggregate Base Materials

Aggregate base courses (¾-inch-minus top and 1¼-inch-minus base course) shall conform to the *Structural Fill* requirements and be placed directly over a properly prepared subgrade. The aggregate base must be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557. We recommend STRATA observe final subgrade preparations, geotextile placement and all aggregate placements.

Portland cement concrete (PCC) pavement shall meet ITD SSHC requirement and have a minimum 5,600 psi compressive strength, 750 psi modulus of rupture, and 4 to 7 percent entrained air. Shrinkage cracks should be controlled by joint spacing not more than 24 times the concrete thickness (in inches), and should be between 1/4 and 1/3 of the slab thickness. Control joints must occur along any line where the concrete thickness changes abruptly and where marked changes in subgrade soil types occur. We recommend a curing compound on all

exterior concrete surfaces, and we recommend the contractor sawcut the concrete as soon as possible following placement to reduce the potential for shrinkage cracking.

Asphalt concrete must be compacted to 92 percent of the maximum density for a Hveem mix design. The final traveling surface of asphalt concrete shall meet ITD SSHC, ¾-inch asphalt mix design requirements. Asphalt and concrete mix designs and all appropriate aggregate source certificates should be accepted by STRATA at least 5 days prior to initiating asphalt paving. Asphalt and concrete pavement construction and final surface smoothness, joints and density should meet ITD specifications. If subgrade conditions appear significantly different during construction, traffic loading conditions change or traffic volumes increase, STRATA should be notified to amend our design accordingly.

Pavement Section Thickness

STRATA evaluated the pavement sections utilizing the AASHTO pavement design methodology, soil-engineering parameters from previous field and laboratory testing and the estimated traffic-loading conditions. Based on subgrades prepared as recommended and the traffic criteria provided, Table 8 provides the recommended asphalt section for the anticipated pavement application. If traffic loading or subgrade conditions change as design is finalized or during construction, STRATA must review our pavement analyses and resulting sections. For clarity, access drives are primary routes into the site where truck, fire vehicles or other heavy traffic is expected.

Table 8. Pavement Design Section

Asphalt Pavement Application	Asphalt Concrete (inches)	Aggregate Base (inches)	Granular Subbase (inches)
Heavy Duty Section – Access Drive	4.0	4	12
Rigid Pavement Application	**Concrete (inches)**	**Aggregate Base (inches)**	**Granular Subbase (inches)**
Heavy Duty Section – Unloading Area	10	12	--

Pavement Maintenance

We recommend crack maintenance be accomplished on all pavement surfaces every 3 to 5 years to reduce the potential for surface water infiltration into the underlying pavement subgrade. Surface and subgrade drainage are extremely important to the performance of the

pavement section. Therefore, we recommend the subgrade, base and asphalt surfaces slope at no less than 2 percent to an appropriate stormwater disposal system or other appropriate location that does not impact adjacent buildings or properties. The pavement's life is dependent on achieving adequate drainage throughout the section and especially at the subgrade. Ponding water at the pavement subgrade surface can induce heaving during the freeze-thaw process.

ADDITIONAL RECOMMENDED SERVICES

Geotechnical Design Continuity

The information contained in this report is based on static loading conditions, preliminary structural loads and development plans. The final floor elevation, floor configuration, loading conditions as well as site geometry can significantly alter our opinions and design recommendations. Specifically, changes in structural design loads and the soil cut and fill geometry may require additional analyses specific to the actual anticipated construction conditions. Therefore, it is critical STRATA provide geotechnical continuity through final planning and design for the planned construction as individual aspects become available during design development phases of the project. It has been our experience that having consultants from the design team review the construction documents prior to bidding helps reduce the potential for errors, and also reduces costly changes to the contract during construction. If we are not provided such opportunities, we cannot be responsible for soil-related design or construction-related errors, omissions, delays or increased costs that are identified during construction.

Construction Observation Monitoring

We recommend STRATA be retained to provide construction monitoring to verify the soil conditions and that report recommendations are incorporated into the actual construction. Such observation is an important part of the geotechnical design process and can help reduce the potential for soil engineering or construction-related errors or omissions. For this project it is especially important to maintain this geotechnical continuity during uncontrolled fill identification, removal and replacement as well as during the subgrading process. If we are not retained to provide the recommended plan review and construction monitoring services, we cannot be responsible for soil engineering-related construction errors or omissions. Further, the selected firm must be required to document in writing to the design team and Dynamis that they have

read and will implement this report and its recommendations in their entirety as the geotechnical engineer of record.

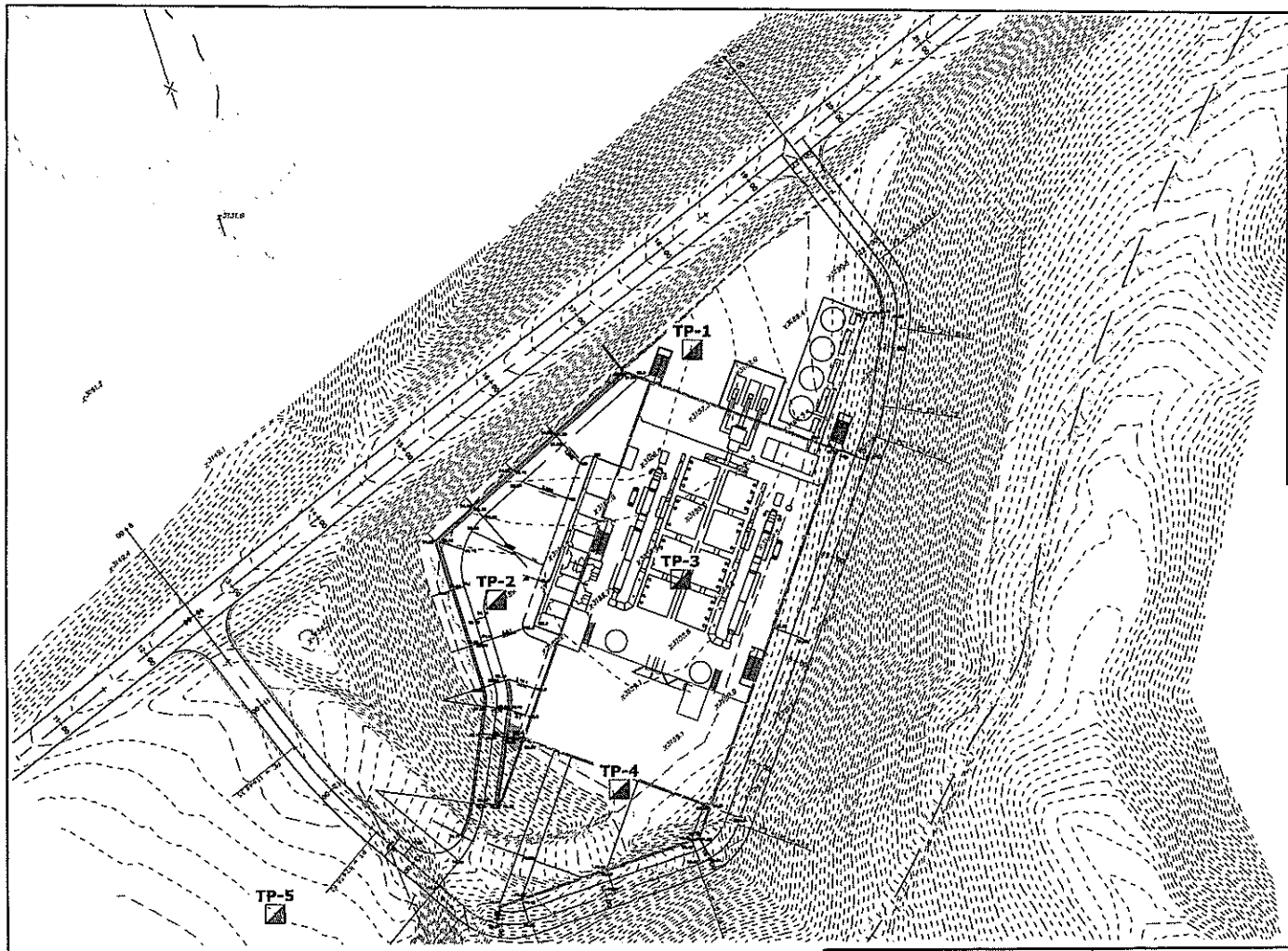
EVALUATION LIMITATIONS

This report has been prepared to assist project planning design and construction of the proposed Dynamis power plant at the Ada County Landfill in Ada County, Idaho. Our geotechnical findings and opinions have been developed based on the authorized subsurface exploration and laboratory testing, as well as our understanding of the project at this time. Our geotechnical design recommendations are specific to the proposed power plant design and infrastructure construction and should not be extrapolated to other future site developments without allowing adequate geotechnical consultation by STRATA.

Our services consist of professional opinions and findings made in accordance with generally accepted geotechnical engineering principles and practices in southwest Idaho at the time of this report. The geotechnical recommendations provided herein are based on the premise that appropriate geotechnical consultation during subsequent design phases is implemented and an adequate program of tests and observations will be conducted by STRATA during construction to verify compliance with our recommendations and to confirm conditions between exploration locations. This acknowledgment is in lieu of all warranties either express or implied.

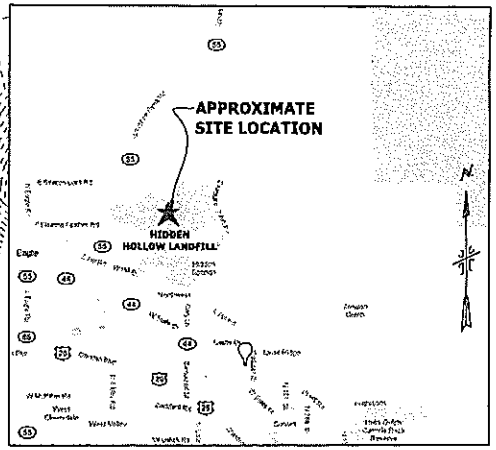
The following plates accompany this report.

Plate 1:	Test Pit Location Plan
Plate 2:	Soil Improvement Schematic
Plate 3:	Foundation/Wall Drainage Schematic
Appendix A:	Unified Soil Classification System (USCS) & Exploration Logs
Appendix B:	Laboratory Test Results



VICINITY MAP

NOT TO SCALE



LEGEND

TP-1 Approximate Test Pit Location Observed by STRATA on December 22, 2010.

THIS PLAN COMPRISES A PORTION OF STRATA'S REPORT AND THE TEXT OF THE REPORT CONTAINS ESSENTIAL INFORMATION; BEFORE UTILIZING THIS PLAN FOR ANY PURPOSE WHATSOEVER, THE REPORT SHOULD BE READ COMPLETELY. THIS PLAN IS INTENDED TO HELP VISUALIZE THE INFORMATION PROVIDED IN THE REPORT. THESE LOCATIONS AND INFORMATION WERE ADDED TO EXISTING PLANS OF THE SITE PREVIOUSLY PREPARED BY OTHERS AND NO CHECK OF ACCURACY, CURRENCY, APPROPRIATENESS, ETC., OF INFORMATION PROVIDED BY OTHERS WAS PERFORMED, SINCE SUCH CHECKS WERE NOT PART OF STRATA'S SCOPE OF SERVICES.

DRAWING DATE: 1-6-2010

DRAWING BY: DMS | CHECKED BY: MGW

TEST PIT LOCATION PLAN
Proposed ADA County Landfill Power Plant
Ada County , Idaho

0 50 100 200

SCALE: 1 Inch = 100 ft

STRATA

A Professional Services Corporation

Intensely from the Ground Up

DYNENE B10400A | PLATE: 1

REFERENCE: Site Plan Provided by Dynamic Energy, Date: 2010.

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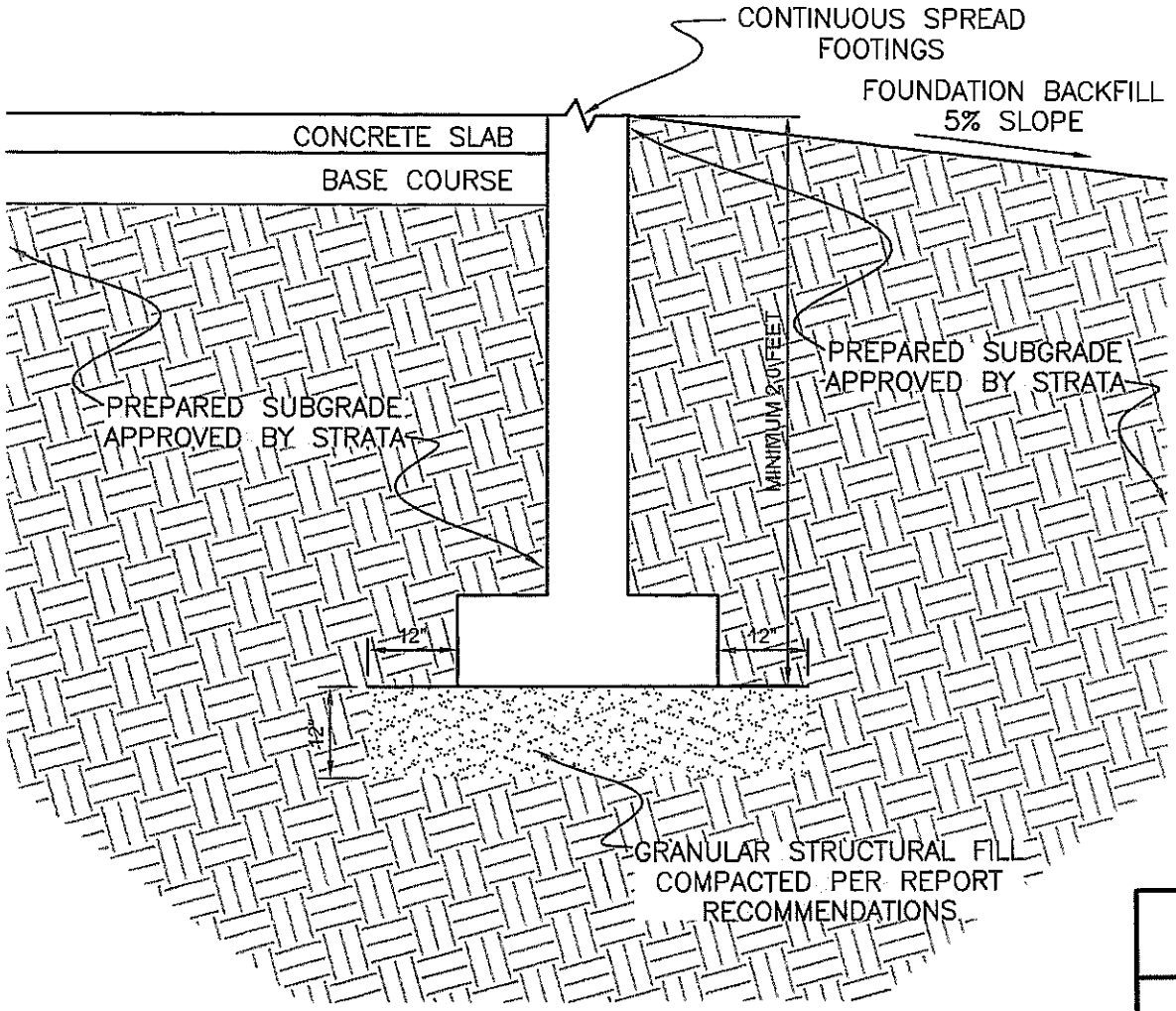
NOTE:
*FOOTING EMBEDMENT MUST EXTEND AT LEAST 2 FEET BELOW EXTERIOR GRADES.



SOIL IMPROVEMENT SCHEMATIC

STRATA
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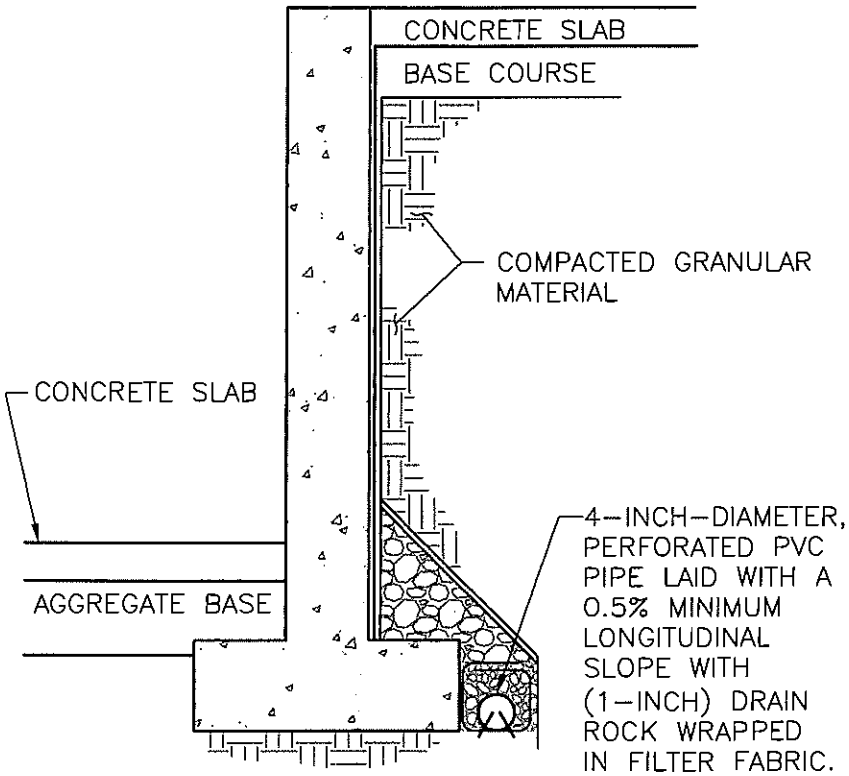
Integrity from the Ground Up

DYNENE B10400A	PLATE: 2

NOTE:
THIS DRAWINGS' CROSS-SECTIONS MAY BE USED FOR DRAINAGE DETAILS. THIS IS NOT A STRUCTURAL DETAIL.

ALIGNMENT PIT DRAINAGE SECTION



INFORMATIONAL ONLY (NOT TO SCALE)

NOTE:
DRAIN PIPE SHALL BE DAY-LIGHTED AND AWAY FROM FOUNDATIONS AND CONCRETE SLABS

FOUNDATION / WALL DRAINAGE SCHEMATIC

STRATA
A PROFESSIONAL SERVICES CORPORATION
Integrity from the Ground Up

DYNENE B10400A	PLATE: 3

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APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS	GRAVELS	CLEAN GRAVELS		GW	Well–Graded Gravel, Gravel–Sand Mixtures.
				GP	Poorly–Graded Gravel, Gravel–Sand Mixtures.
		GRAVELS WITH FINES		GM	Silty Gravel, Gravel–Sand–Silt Mixtures.
				GC	Clayey Gravel, Gravel–Sand–Clay Mixtures.
	SANDS	CLEAN SANDS		SW	Well–Graded Sand, Gravelly Sand.
				SP	Poorly–Graded Sand, Gravelly Sand.
		SANDS WITH FINES		SM	Silty Sand, Sand–Silt Mixtures.
				SC	Clayey Sand, Sand–Clay Mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%			ML	Inorganic Silt, Sandy or Clayey Silt.
				CL	Inorganic Clay of Low to Medium Plasticity, Sandy or Silty Clay.
				OL	Organic Silt and Clay of Low Plasticity.
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%			MH	Inorganic Silt, Micaceous Silt, Plastic Silt.
				CH	Inorganic Clay of High Plasticity, Fat Clay.
				OH	Organic Clay of Medium to High Plasticity.
				PT	Peat, Muck and Other Highly Organic Soils.

BORING LOG SYMBOLS

- Standard 2–Inch OD Split–Spoon Sample
- California Madified 3–Inch OD Split–Spoon Sample
- Rock Core
- Shelby Tube 3–Inch OD Undisturbed Sample

GROUNDWATER SYMBOLS

- Groundwater After 24 Hours
- (7-3-07) Indicates Date of Reading
- Groundwater at Time of Drilling

TEST PIT LOG SYMBOLS

- BG Baggie Sample
- BK Bulk Sample
- RG Ring Sample

Shorthand Notation:
BGS = Below Existing Ground Surface
N.E. = None Encountered

STRATA
A Professional Services Corporation
Integrity from the Ground Up

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	USDA SOIL Textural Classification	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED SAND, (SP) tan, medium dense, moist	0.0				B1					Soil is very loose from 0 to 8 inches BGS.
	2.5									
				BG			98.9	7.0		
	5.0									Red staining observed from 4.5 to 5.0 feet BGS.
	7.5	SP								
	10.0									
	12.5									
	15.0									

Test Pit Terminated at 15.0 Feet.

Client: DYNENE	**Test Pit Number:** TP-1	STRATA A Professional Services Corporation Enterprise from the Ground Up	**EXPLORATORY TEST PIT LOG**
Project: B10400A	**Date Excavated:** 12-22-2010		
Backhoe: CASE 580	**Bucket Width:** 3'		
Depth to Groundwater: N.E.	**Logged By:** KPR		Sheet 1 Of 1

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USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	USDA SOIL Textural Classification	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED SAND, (SP) tan, medium dense, moist	0.0				B1					Soil is very loose from 0 to 8 inches BGS.
	2.5									
				BG		1.7	104.8	5.3		Percolation test performed at 4.5 feet BGS. Infiltration rate = 25 inches per hour measured.
	5.0									
	7.5	SP								Less fines from 4.5 to 10 feet BGS.
	10.0									
				BG						
	12.5									
	15.0									

Test Pit Terminated at 15.0 Feet.

Client: DYNENE	**Test Pit Number:** TP-2	STRATA A Professional Services Corporation Integrity from the Ground Up
Project: B10400A	**Date Excavated:** 12-22-2010	
Backhoe: CASE 580	**Bucket Width:** 3'	
Depth to Groundwater: N.E.	**Logged By:** KPR	

EXPLORATORY TEST PIT LOG

Sheet 1 Of 1

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USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	USDA SOIL Textural Classification	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED SAND, (SP) tan, medium dense, moist	0.0	SP			B1					Soil is very loose from 0 to 8 inches BGS.
	2.5									
				BG		4.8	101.3	8.9		
	5.0									
	7.5									
	10.0									

Test Pit Terminated at 11.0 Feet.

Client: DYNENE	**Test Pit Number:** TP-3	STRATA A Professional Services Corporation Exploring Below the Surface	**EXPLORATORY TEST PIT LOG**
Project: B10400A	**Date Excavated:** 12-22-2010		
Backhoe: CASE 580	**Bucket Width:** 3'		
Depth to Groundwater: N.E.	**Logged By:** KPR		Sheet 1 Of 1

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USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	USDA SOIL Textural Classification	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED SAND, (SP) tan, medium dense, moist	0.0				B1					Soil is very loose from 0 to 8 inches BGS.
	2.5									Seep at 3.5 to 4.0 feet BGS.
	5.0			BG			108.2	11.2		Percolation test performed at 4.5 feet BGS. Infiltration rate = 20-25 inches per hour measured.
	7.5	SP								
	10.0									
	12.5									
	15.0									

Test Pit Terminated at 15.0 Feet.

Client: DYNENE	**Test Pit Number:** TP-4	STRATA A Professional Services Corporation Integrity from the Ground Up	**EXPLORATORY TEST PIT LOG**
Project: B10400A	**Date Excavated:** 12-22-2010		
Backhoe: CASE 580	**Bucket Width:** 3'		
Depth to Groundwater: N.E.	**Logged By:** KPR		Sheet 1 Of 1

STRATA BH / TP / WELL - STRATA.GPJ - 1/13/11 07:02 - R:\GINT\PROJECTS\DYNENE\DYNENE B10400A.GPJ

USCS Description	DEPTH (ft)	U.S.C.S. CLASS	SYMBOL	Sample Type	USDA SOIL Textural Classification	% Passing No. 200 Sieve	Dry Density (pcf)	Moisture Content (%)	Pocket Pen. (tsf)	REMARKS Note: BGS = Below Ground Surface
POORLY GRADED SAND, (SP) tan, medium dense, moist	0.0				B1					Soil is very loose from 0 to 8 inches BGS.
	2.5									
	5.0	SP								
	7.5									
	10.0									
Test Pit Terminated at 10.0 Feet.										

Client: DYNENE	**Test Pit Number:** TP-5	STRATA A Professional Services Corporation Independently from the Ground Up	**EXPLORATORY TEST PIT LOG**
Project: B10400A	**Date Excavated:** 12-22-2010		
Backhoe: CASE 580	**Bucket Width:** 3'		
Depth to Groundwater: N.E.	**Logged By:** KPR		Sheet 1 Of 1

STRATA BH / TP / WELL - STRATA.GPJ - 1/13/11 07:02 - R:\GINT\PROJECTS\DYNENE\DYNENE B10400A.GPJ

APPENDIX B

STRATA

A PROFESSIONAL SERVICES CORPORATION

Integrity from the Ground Up

Summary of Test Results

Project: Ada County Landfill Power Plant
Client: Dynamic Energy

File Name: DYNENE B10400A
Date: 1/12/2011

Test Pit	Depth (feet)	Lab Number	Soil Classification (remarks)	Dry Unit Weight, pcf	In Situ Moisture, %	Passing No. 200,%	Resistivity ohm-cm	pH	Atterberg Limits LL	Atterberg Limits PI	Fines Class.
1	4-4.5	B10L2544A	P.G. Sand		7.0						
2	4-4.5	B	P.G. Sand		5.3	1.7					
3	4.5-5	C	P.G. Sand		8.9	4.8					
4	4.5-5	D	P.G. Sand		11.2		>13340	7.5			

TAB 8

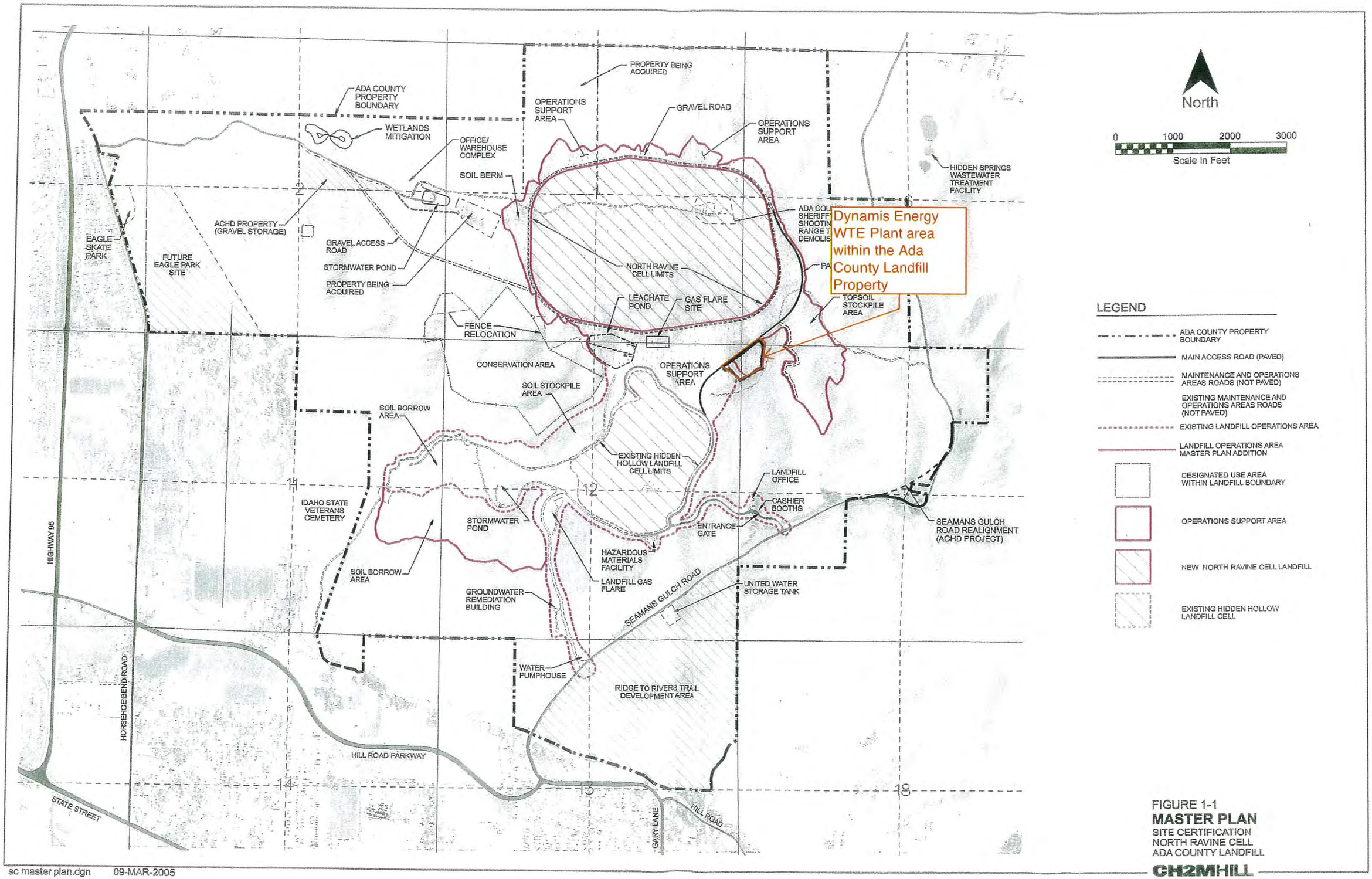

FIGURE 1-1
MASTER PLAN
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

CH2MHILL

sc master plan.dgn 09-MAR-2005

TAB 9

COPY

Report

Ada County Landfill North Ravine Cell Site Certification Report

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Prepared for

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Acronyms

amsl	above mean sea level
ASTM	American Society of Testing and Materials
bgs	below ground surface
Ca	calcium
CFR	Code of Federal Regulations
CH	high plasticity clay
CL	low plasticity clay
cm/sec	centimeters per second
CS	continuous sample
CUTX	consolidated, undrained, triaxial
CZ	confining zone that underlies the uppermost water-bearing zone
DO	dissolved oxygen
DTM	digital terrain map
EPA	U.S. Environmental Protection Agency
ET cover	evapotranspiration cover
FC	field capacity
FEMA	Federal Emergency Management Agency
FIRMs	Flood Insurance Rate Maps
ft/ft	feet per foot
g	gram
GCL	geosynthetic clay layer
g/cm^3	grams per cubic centimeter
gpm	gallons per minute
HCO_3	bicarbonate
HDPE	high-density polyethylene
HHLF Cell	Hidden Hollow Landfill Cell

IAC	Idaho Administrative Code
K	hydraulic conductivity
K_h	horizontal pseudo-static coefficient
ksf	kips per square foot
K_v	vertical hydraulic conductivity
lb/ft^3	pounds per cubic foot
LCRS	leachate collection and removal system
MCL	maximum contaminant level
MDL	method detection limit
mg	milligrams
mg/L	milligrams per kilogram
ml	milliliter
MSW	municipal solid waste
MSWLF	municipal solid waste landfill
Na	sodium
ND	not detected
NMOC	non-methane organic compounds
NPDES	National Pollutant Discharge Elimination System
NRC	North Ravine Cell
OCR	overconsolidation ratios
ohm-cm	ohm-centimeter
pcf	pounds per cubic foot
ppb	parts per billion
PQL	practical quantification limit
psf	pounds per square foot
psi	pounds per square inch
PWP	pore water pressure
PVC	polyvinyl chloride
RC	relative compaction

RCRA	Resource Conservation and Recovery Act
T	township
SC	sandy clay
SM	silty clay
SP	poorly-graded sand
SPT	standard penetrometer test
SS	split spoon
SW	well-graded sand
SWP	stormwater pond
TDS	total dissolved solids
TOC	total organic carbon
μg/L	micrograms per liter
μmhos/cm	micro-mhos per centimeter
USCS	Unified Soil Classification System
USGS	U.S. Geological Survey
UWBZ	uppermost water-bearing zone
VOCs	volatile organic compounds
VWC	volumetric water content

1.0 Introduction

1.1 Background and Overview of the Proposed North Ravine Cell

The Ada County landfill, managed by the Ada County Solid Waste Management Department, provides municipal solid waste (MSW) disposal services for Ada County residences, business, and institutions. Ada County originally opened the Hidden Hollow Landfill (HHLF) Cell on 240 acres of leased property in 1972. In 1984, Ada County purchased the leased property and an additional 2,000 acres (approximately) of surrounding property to provide a buffer zone for HHLF Cell operations and to provide the space necessary for MSW landfill capacity into the future. Additional property purchases since 1984 have added to the Ada County Landfill Property area. Currently the Ada County Landfill Property boundary includes approximately 2,700 acres.

In 2004, the landfill served over 200,000 customer loads. More than two-thirds of the loads were from the public and private sectors. The remainder came from commercial trash haulers. Over 900 million pounds of MSW were landfilled at the HHLF Cell in 2004. In addition to MSW landfilling, Ada County Solid Waste Management Department is responsible for many waste recycling programs that are available to all County residents and closely coordinates solid waste management and recycling programs with various local City programs.

The existing MSW cell, the HHLF Cell, is expected to reach maximum capacity by 2009 or 2010. The ability of the active HHLF Cell to efficiently handle the entire MSW stream generated in Ada County will be reached in 2007. Beyond this time, efficiency will decrease because the landfill's working area will rapidly diminish as the cell is filled closer to the top. In response to the need for more landfill capacity, The Ada County Board of Commissioners (the Board) determined that developing a new MSW cell within the current Ada County Landfill property is the most prudent approach to meet the Ada County's waste disposal needs into the future. The proposed new cell, the North Ravine Cell (NRC), will begin receiving part of the waste stream in 2007, and will receive all MSW by 2010. The current Ada County Landfill property, the HHLF Cell, facilities, supporting infrastructure, and proposed expansion plans are presented in Figure 1-1.

1.2 Project Objectives and Scope

The objective of NRC Site Certification is to evaluate site characteristics and the suitability of the proposed expansion area as a municipal solid waste landfill (MSWLF). The study consisted of the following:

1. An evaluation of the location restrictions of 40 CFR 258 Subpart B and Section 39-7407 (Location Restrictions), Idaho Code to determine the potential effects that the expansion into the NRC may have on the surrounding environment, and the effects that natural

and human-made conditions may have on the performance of the landfill unit. The list of location restrictions under 40 CFR 258 Subpart B and Section 39-7407, Idaho Code is as follows:

- Airport Safety
- Floodplains
- Wetlands
- Fault Areas Seismic Impact Zones
- Unstable areas

2. Characterize site hydrogeology pursuant to 40 CFR 258 Subpart E and Section 39-7407 (Location Restrictions), Idaho Code to identify the following:

   - Lithology, soil types, and stratigraphy
   - Zones of potentially high hydraulic conductivity
   - The presence of confining formations
   - Unpredicted geologic features, such as fault zones, cross-cutting structures, and pinch-out zones
   - Continuity of stratigraphic units
   - Groundwater gradients, velocities, and flow directions
   - Pre-construction groundwater quality
   - Potential receptors of a release

   The compiled information from the hydrogeologic characterization will be used to prepare a groundwater monitoring system for the completed cell pursuant to 40 CFR 258 Subpart E and Sections 39-7409 (Standards for Design) and 39-7410 (Groundwater Monitoring Design), Idaho Code. Therefore, the amount and quality of data collected for the hydrogeologic characterization must be adequate to determine the appropriate number, locations, depths of monitoring wells to yield representative groundwater samples from the uppermost aquifer and ensure early detection of a release.

3. Collect sufficient site geotechnical information to ensure that the cell design will meet the criteria specified in 40 CFR 258 Subpart D and Idaho Code 39-7409 for the following:

   - Composite liner
   - Leachate collection system
   - Point of compliance
   - Leachate discharge

1.3 Conceptual Cell Plan and Profile

A conceptual layout of the proposed NRC is presented to facilitate assessment of the site's suitability in terms of geology, hydrogeology, and topography. The footprint of the proposed cell is shown on the geology and hydrogeology plans and profiles presented in this report to demonstrate that the proposed cell location complies with all siting criteria. The proposed cell capacity is approximately 70,000,000 cubic yards. The conceptual cell layout has been developed based on key geometric criteria as discussed below.

1.3.1 Geometric Criteria

The six geometric criteria used to develop the conceptual layout of the proposed North Ravine cell are as follows:

1. Bottom slopes and elevation of the cell liner
2. Side slopes of the cell liner
3. Elevation of the top of the cell liner
4. Side slopes of the final cover
5. Top elevation of the final cover
6. Horizontal location of cell

Each criterion and the corresponding features of the proposed cell conceptual layout are discussed below.

1.3.1.1 Bottom Slopes and Elevation of the Cell Liner

The slope of the cell bottom liner on the east/west axis (along the bottom of the ravine drainage) is dependent on the slope of groundwater table. The cell bottom liner and leachate drain piping will slope at approximately 4 percent from east to west, generally following the slope of the shallow water table and the surface of the ground. From the north and south edges, the bottom liner and piping will slope at 2 percent to the center of the cell (towards the east-west axis). The resultant (net) slope of the bottom liner will be 4.5 percent. This configuration directs all drainage to the west end of the cell for collection.

The elevation of the bottom liner will be a minimum of 20 feet above the observed water table. This separation between the water table and liner bottom was selected to provide a safety factor to protect the liner against potential local variations in ground water elevations.

1.3.1.2 Side Slopes of the Cell Liner

The side slopes of the proposed liner will be 3H:1V (horizontal : vertical) or flatter. Liner installation on slopes steeper than 3H:1V become substantially more difficult and costly. Construction experience with the sand and silty sand soils at the HHLF Cell has also shown that construction on side slopes as steep as 3H:1V can be reasonably accomplished with the right equipment and care.

The proposed cell side slopes include a 50-foot-wide bench at 50 feet of elevation gain above the cell floor to avoid hazards and construction difficulties that can occur with very long uninterrupted side slopes. This bench will also provide the vertical interface between the cell build-out phases. A bench width of 50 feet was selected to provide adequate width for

equipment operation (for example, a scraper requires a minimum of 40 feet to turn around). Combining 3H:1V side slopes and the 50-foot-wide bench, results in an overall approximate 4H:1V side slope geometry. The bench will be longitudinally sloped a minimum of 2 percent for leachate collection piping drainage, with all leachate draining to the west end of the cell.

1.3.1.3 Elevation of the Top (Rim) of the Cell Liner

Setting the elevation of the top of the cell liner around the perimeter rim is a balance between two factors: 1) reducing excavation and embankment fill quantities across the ravines and ridges to minimize initial construction costs; and, 2) maximizing net excavation quantities for use as daily cover soil. Lower elevations will also reduce the capacity of the cell unnecessarily. Higher elevations will increase the capacity but create unbalanced earthwork and increase cost by the extra excavation required to obtain the embankment material needed.

The proposed cell top liner elevation varies between 3,180 and 3,200 feet above mean sea level (amsl) along the southern rim and between 3,160 and 3,180 feet amsl along the northern rim. This configuration provides approximately 11 million cubic yards of excess excavation that will be available for use as daily cover, and the excavation and embankment fill quantities in the initial construction area will be approximately balanced to lower initial construction costs.

1.3.1.4 Side Slopes of the Final Cover

Side slopes of the final cover will be 5H:1V, the same as the HHLF Cell finished side slopes. Experience in current HHLF Cell operations and the completed Phase 1 evapotranspiration (ET) cover construction has found that 5H:1V final cover slopes are the steepest practical long-term slopes for the landfill waste.

1.3.1.5 Top Elevation of Final Cover

The elevation of the cell final cover edge is matched to the top elevation of the surrounding ridge tops, which are at roughly 3,250 feet. Keeping the final cover below 3,250 feet will blend the completed cell with the adjacent foothill ridge lines and keep landfill operations mostly hidden from view of the surrounding communities. The final landfill height at the cell center will be approximately 50 feet higher (at 3,300 feet) to provide adequate slope to promote stormwater runoff and drainage of gas collection piping.

1.3.1.6 Horizontal Location of Cell

The proposed cell will be located horizontally within the North Ravine site using the same criteria as the top rim elevation of the cell liner, namely, a balance of 1) reducing and balancing excavation and embankment fill quantities to lower initial construction costs and 2) providing sufficient excavation volume to meet daily cover soil needs. The volume of excavation minus embankment is the volume of excess soil material that will be available for daily cover from cell construction. The model indicates that a total of approximately 9.5 million cubic yards of in-place daily cover will be available from the excavation. Ultimately a total of 21 million cubic yards of daily cover soil will be required over the life of the landfill. The remaining 10 million cubic yards will be obtained from borrow areas adjacent to the site. These additional areas are to be identified in the NRC Conditional Use Permit (in preparation).

1.3.2 Leachate and Gas Management

The conceptual leachate management plan is to direct leachate drainage to the west end of the proposed landfill cell. Leachate will be collected in sumps and pumped to leachate holding basins outside of the cell. Leachate disposal will be accomplished through a combination of evaporation, landfill operations dust control within the lined limits of the cell, and re-injection into the landfill.

The conceptual landfill gas management plan is collection of gas using horizontal collector pipes installed as waste lifts are placed. The long-term management plan for the gas from the NRC is ultimately to combine with gas from the HHLF Cell at a common flare station.

1.3.3 Stormwater Management

A proposed stormwater basin will provide storage for stormwater runoff from the landfill surface. This stormwater will be collected and allowed to evaporate and infiltrate. By containing potentially impacted stormwater (water that has run off the cell surface) onsite, the County is exempt from the U.S. EPA National Pollutant Discharge Elimination System (NPDES) stormwater permit and monitoring requirements. This cost-effective strategy is currently used for existing HHLF Cell stormwater runoff, where all potentially impacted stormwater is contained in a pond.

The conceptual plan for the NRC is to route most of the stormwater runoff from the landfill surface to multiple intermediate containment areas, such as operations areas outside the limits of the liner, for infiltration and evaporation. The concept is to provide local area drainage and containment that will result in a significant reduction of the required storage volume of the stormwater basin. Because of either slope configuration or the absence of suitable adjacent stormwater containment areas, some areas of the landfill surface will be collected in the stormwater basin to the west of the cell. The conceptual plan shows a basin with approximately 20 acre-feet of capacity.

A system of diversion berms and other control features will minimize the generation of potentially impacted runoff. A stormwater diversion channel will convey non-impacted runoff (hillside drainage) around the stormwater basin and into the natural drainage channel below the project, preventing run-on. Stormwater flows in the natural drainage channel currently infiltrate into the subsurface soils. Landfill personnel have not observed overland stormwater flow at the mouth of the drainage during the past 25 years. Separating impacted and non-impacted waters will significantly reduce the required stormwater pond storage capacity and associated costs.

2.0 Site Description

2.1 Introduction

Ada County is proposing to expand the Ada County Landfill by constructing a new landfill cell within the Goose Creek drainage. The new cell will be directly north of the existing HHLF Cell on Ada County Landfill property. The proposed cell is referred to as the North Ravine Cell or NRC. A description of the NRC is provided below.

2.2 Location

The Ada County Landfill is located in Section 12, Township 4 North, Range 1 East, off Seamans Gulch and Hill Roads in northwest Boise, Idaho (Figure 1-1). The existing HHLF Cell encompasses an area of approximately 110 acres. The proposed NRC will occupy an additional 260 acres within the County-owned property. County-owned property surrounding the landfill includes approximately 2,640 acres as shown in Figure 1-1. Ada County is currently in the process of purchasing an additional 87 acres of property to accommodate the new cell and to ensure that adequate buffer property will surround the NRC. This additional property is identified in Figure 1-1 and will increase the size of the Ada County property at the landfill to 2,727 acres.

2.3 Previous Investigations

From 1989 to the present, CH2M HILL has completed a series of field investigations of the HHLF Cell to characterize the uppermost aquifer and to determine appropriate monitoring well placement and construction. Key characterization reports that have been prepared for the existing HHLF Cell and are pertinent to the characterization and construction of the NRC are as follows:

- *Ada County Hidden Hollow Landfill Subtitle D Groundwater Monitoring Report* (CH2M HILL, November 1989)
- *Report on Phase I and Phase II Field Investigation at the Ada County Hidden Hollow Landfill Boise, Idaho* (CH2M HILL, May 1991)
- *Site Characterization and Groundwater Monitoring Program for Ada County Hidden Hollow Landfill* (CH2M HILL, March 1995)
- *Area of Impact of Groundwater Contamination at the Ada County Hidden Hollow Landfill* (CH2M HILL, March 1995)
- *Nature and Extent of Groundwater Contamination at the Ada County Hidden Hollow Landfill* (CH2M HILL, June 1995)
- *Fault Investigation—Ada County Hidden Hollow Landfill* (CH2M HILL, September 1995)

- *Hydrogeologic Characterization of the Groundwater Remediation Site, Ada County Hidden Hollow Landfill* (CH2M HILL, July 1996)
- *Well Construction and Hydrogeologic Characterization of the Groundwater Remediation Site, Ada County Hidden Hollow Landfill* (CH2M HILL, March 1998)
- *Well Drilling, Construction, Development, and Groundwater Sampling of Extraction Wells EW-6, EW-7, EW-8, EW-9, and EW-10 Ada County Hidden Hollow Landfill Boise, Idaho* (CH2M HILL, December 2000)

Additionally, numerous routine site reports (quarterly remediation monitoring reports, semi-annual assessment/detection monitoring reports) have also been prepared for the HHLF Cell, but are not referenced here. All reports are available upon request.

2.4 Regional Setting

2.4.1 Physiography

The study area is located in the northern Rocky Mountain province (Savage, 1958) on the northwest margin of a 20,000-square-mile physiographic unit known as the Snake River Plain. The plain extends from the vicinity of Ashton, Idaho, to north of Ontario, Oregon. The Snake River Plain is approximately 350 miles in length and varies in width from 25 to 75 miles. The Boise Valley is a broad alluvial, terraced plain located within the Snake River drainage.

The study area is located on the west end of the Boise foothills in northeastern Ada County. In this area, the foothills extend from the Boise River alluvial plain at an elevation of approximately 2,700 feet above mean sea level to slightly more than 3,200 feet. The foothills are generally treeless and are moderately incised by westward- to southwestward-trending drainages with 200 to 500 feet of topographic relief from drainage bottom to ridge top. The foothills in the vicinity of the study area merge to the northeast with the irregular peaks of Boise Ridge, which have elevations in excess of 7,000 feet.

The NRC will occupy the eastern extent of the Goose Creek drainage between 2,900 and 3,200 feet amsl in elevation (Figure 1-1). The Goose Creek drainage trends west until it ends at Highway 55. No culvert or drainage exists for drainage past Highway 55. Drainage within the basin evaporates or infiltrates into the highly permeable basin soils. The Goose Creek drainage appears to be an ephemeral drainage which flows only in direct response to precipitation.

2.4.2 Climate

Ada County is characterized by relatively mild year-round temperatures and low precipitation. Located some 300 miles from the Pacific Ocean, the area is influenced by prevailing westerly maritime winds via the Columbia and Snake River Valleys. This influence is especially noticeable during the winter months, when most of the precipitation occurs. Snowfall is light in the valleys, often accompanied by rapid melting. For the period of record, 1940 to 2004, average seasonal snowfall is 20 inches. The greatest snow depth at any one time was 10 inches. On the average, 11 days have at least 1 inch of snow on the ground, but the number of such days varies greatly from year to year. In the higher

elevations of Boise Ridge, snow accumulation covers the ground for most of the winter. The summer months tend to be dry, sunny, and warm. The Boise Valley has an annual precipitation of 12 inches. Of this, approximately 4 inches (33 percent) usually falls in April through September.

The mean January temperature range for Boise is 22°F to 37°F, although extreme temperatures of -10°F to -25°F have been recorded. The average daily maximum temperature during the summer months is 84°F, and the minimum is 53°F. Maximum temperatures recorded have been as high as 111°F.

Prevailing winds are from the southeast for 8 months of the year (September through April), and from the northwest during the summer months. Small dust storms can occur during warmer periods of the summer. This partially explains why the region is mantled with a variable thickness of loess (Savage, 1958).

2.4.3 Regional Geology and Water-Bearing Characteristics

The Boise area is located along the northeast margin of a downwarped and faulted graben of the western Snake River Plain. A lake formed within this large depression in the plain. Fine silts and clays settled to the floor of the lake, while coarser sediments from upland streams were deposited along the streams and on the edge of the lake. Up to 6,000 feet of lake, river, and alluvial fan sediments have accumulated in the basin over the past 10 million years (Squires et al., 1992). These sediments have been collectively referred to as the Glenns Ferry Formation of the Upper Idaho Group (Baker, 1991). The sediments have been laid down over a thick Miocene rhyolite volcanic sequence, which is faulted against Cretaceous rocks of the Idaho batholith. Holocene basalt flows of the Snake River Group form the present day surface along with younger alluvium, terrace gravels, and eolian deposits.

Jointed and fractured zones in the silicic volcanic rocks form geothermal aquifers in the northeastern part of the Boise Valley at depths of 1,000 to 2,000 feet below ground surface (bgs). A thick sequence of relatively impermeable clay sediments and lava flows separates the geothermal waters from overlying water-bearing sediments of the Idaho Group. These sediments and the fractured zones of the Snake River Basalts form the cold water aquifers in the region (Squires et al., 1992).

The lake-river-alluvial fan facies in the Boise River Valley have changed many times over the past 10 million years as a result of shifting lake shorelines, riverbeds, and floodplains. Thus, a vertical section of any one point contains a strata of alternating clay, silt, sand, and gravel sequences, depending on the energy of the depositional environment. Most cold water wells in the Boise area produce from the saturated alternating sand/gravel layers. The cold water aquifers in the Idaho Group sediments are limited in areal extent by the relatively impermeable boundary of crystalline rocks to the north and basin truncation faults to the south. The thick sequence of clay sediments and lava flows underlying the cold water aquifers, combined with shifting depositional layering patterns, form hydraulic barriers with depth.

2.4.4 Local Geology

The Ada County Landfill is underlain by lacustrine and lake-margin sediments of the Tertiary Period Upper Idaho Group. The age of the sediments is considered to be late Pliocene Epoch (Othberg and Stanford, 1992). The formation is comprised of nonindurated, complexly interbedded, continental deposits of clay, silt, sand, and fine gravel. Some volcanic ash and local lava flows of olivine basalt occur elsewhere within the formation (Dion, 1972), but are not present in the vicinity of the landfill. Total thickness of the Upper Idaho Group sediments in the Boise area is approximately 2,000 feet. The sediments are well exposed along the Boise front. Evidence of minor faulting and tilting within the formation has been documented (Savage, 1958). A northwest-trending fault line scarp is apparent immediately north of Boise. The fault line separates the floor of the Boise valley from the Boise front and generally follows Hill Road to the south of the HHLF. This fault is part of the Boise Front Fault system, and is estimated to be late Tertiary in age (Breckenridge, et al, 2003).

The Upper Idaho Group sediments in the area of the landfill consist of two distinct facies. Work by Othberg and Burnham (1990), Othberg (1994), and Burnham and Wood, (1992) uses local formation names for these sediments to distinguish them separately. The lower facies is primarily lacustrine (lake) sediments deposited in deep, low-energy, offshore waters, and is referred to as the Terteling Springs Formation. Beneath the study area, the lacustrine sediments are characterized by thick silts and clays interbedded with thin layers of fine sand. The lacustrine sediments are typically dark gray or blue in color because they were deposited in oxygen-deficient water. For ease of reference, the term "blue clay" will be used when referring to these sediments; however, the actual sediments consist of silts, silty clays, and dense, plastic clays.

Overlying the lacustrine sediments is a fluvial facies consisting of fine- to coarse-grained arkosic sand and fine gravel, thinly bedded silt, and some minor clay deposited at the lake margins and in river deltas. The fluvial facies has been identified as the Pierce Gulch Formation (Othberg and Burnham, 1990; Othberg, 1994). The sand includes cross-bedded and forest-bedded sequences. This was a much higher energy, oxygen-rich environment; thus, the fluvial sediments are typically coarse-grained and tan to light brown in color with occasional iron staining.

2.4.5 Stratigraphy

Well logs and cores from boreholes and monitoring wells onsite indicate a similar lithology throughout the study area. Fine- to coarse-grained arkosic sand with occasional fine gravel and/or silty interval is underlain by interbedded layers of silty to clayey fine sand and sandy silt which is then underlain by thick blue clays and silts interbedded with thin layers of fine sand. Thin intermittent layers of cemented sand, compact silts, and clay are evident in most of the shallow borings. The local water table in the shallow borings at the NRC was encountered between 1 and 180 feet bgs, depending on the elevation of the boring location. The base of the uppermost water-bearing unit is generally defined by the lacustrine facies (Terteling Springs Formation). This massive confining layer of dense, blue-gray sandy and silty clay is relatively impermeable compared to the overlying fluvial sediments. The saturated thickness of the fluvial sediments at the shallow well locations varies between 4 and 52 feet. The depth to water and saturated thickness appears to be a function of both

the land surface topography and the subsurface expression of the top of the blue clay layer. The surface of the confining clay layer slopes 2.1 to 11.3 degrees to the south-southwest below the study area.

2.4.6 Uppermost Water-Bearing Unit

The first fully saturated sediments were found within the fine-to-coarse-grained sand beds stratigraphically above the thick and continuous lacustrine blue clay. The blue clay is an important hydrogeologic horizon that underlies the entire landfill and surrounding area. It acts as a definitive confining unit with a very low permeability upon which the shallow groundwater lies. Depth to this layer varies somewhat as a result of preferential erosion of overlying sediments and the variability of its surface slope.

Depth to water and the thickness of the uppermost aquifer is largely dependent on the attitude and topographic relief of the underlying blue clay. The blue clay surface slopes predominantly to the southwest. In general, the groundwater flow direction is also to the southwest.

The top of the uppermost aquifer is defined by the water table or top of saturation (potentiometric surface). No significant clay layers exist between the ground surface and the top of the groundwater to act as an upper confining bed; therefore, the groundwater appears to be unconfined.

2.4.7 Groundwater Quality

Groundwater quality samples have been collected at the HHLF Cell since April, 1994. Groundwater samples downgradient of the existing HHLF Cell have tested positive for volatile organic compounds (VOCs). In response, Ada County has developed both a detection monitoring program and an assessment monitoring program. The HHLF Cell detection monitoring program started in September 1994 and the HHLF Cell assessment monitoring program started in January 1995. Both of these programs are ongoing. The concentration of VOCs in the HHLF Cell monitoring wells that have been effected are holding steady or are showing a decline. Both monitoring programs indicate that the proposed new NRC is upgradient of the HHLF Cell.

In addition to the HHLF Cell detection and assessment monitoring program described above, Ada County has installed and operated a groundwater extraction system designed to capture and treat contaminated groundwater downgradient of the HHLF Cell. As part of the groundwater remediation program, quarterly monitoring of selected extraction wells and remediation system monitoring wells is ongoing. These sample results show that the VOC concentrations in the remediation system monitoring wells downgradient from the extraction wells are decreasing which indicates that the system is working.

3.0 Investigative Methods

3.1 Site Survey, Surface Digital Terrain Modeling, and North Ravine Cell Siting Investigation

A topographic survey of the North Ravine and surrounding area was performed under the supervision of a CH2M HILL Idaho-licensed land surveyor. Geographic coordinate and elevation data were gathered through precise aerial stereo-photography methods throughout the North Ravine to accurately represent the field conditions. The aerial photography was tied to a surveyed local datum established at the facility by use of targets set on the ground at known coordinates and elevations. An electronic three-dimensional digital terrain map (DTM) with an accuracy of ± 1.0 foot was produced. Ground surveys were then performed to validate the DTM during the process of surveying wells and boring locations. These methods are well established in topographic surveying practice, and are the normal methods used to produce topographic maps for large civil engineering projects. Based on the DTM, the three-dimensional footprint of the NRC will be developed. The six major features of the cell footprint will be as follows:

- Bottom slopes and elevation of the cell liner
- Elevation of the top of the cell liner
- Side slopes of the cell liner
- Side slopes of the final cover
- Top elevation of the final cover
- Horizontal location of the cell

3.2 Site Geology/Hydrogeology Investigation

3.2.1 Soil Borehole Drilling Program

3.2.1.1 Soil Boring Locations and Rationale

Forty soil borings (NRB-1 to NRB-40) were drilled at strategic locations throughout the NRC under the observation of CH2M HILL engineers and hydrogeologists (Figure 3-1). These borings were drilled to depths ranging from 8.5 to 200.5 feet bgs. The rationale for the selected boring locations was to characterize NRC lithology and hydrogeology, install monitoring wells, and collect the necessary geotechnical samples for the design of the new cell. Boring locations included the perimeter and interior of the NRC, the proposed wetlands mitigation area, and other selected strategic locations.

The soil borings were drilled in three stages. During the first phase of drilling, between August 25 and September 23, 2003, borings NRB-1 through NRB-20b were installed in the south half of the Goose Creek drainage because the proposed expansion of the Ada County Landfill at the NRC was limited to south half of the Goose Creek drainage at that time. During the second phase of drilling, between April 5 and May 6, 2004, additional borings were drilled at NRB-21, NRB-22, and NRB24 through NRB-35 because the proposed NRC was expanded to include the north half of the Goose Creek drainage. During the third phase

phase of drilling, between July 26 and July 30, 2004, borings NRB-23 and NRB-36 through NRB-40 were installed to investigate anomalous stratigraphic features identified during the initial two phases of drilling. All borings were installed using a CME 850 track-mounted drill rig or CME 75 or BK-81 truck-mounted drill rigs operated by Haz-Tech Drilling, Inc., of Meridian, Idaho. Each drill rig was equipped with a 4.5-inch-inside-diameter (8-inch-outside-diameter) hollow-stem auger. After completion of each soil boring, either a monitoring well was constructed in the borehole, or the borehole was backfilled with bentonite chips. Section 3.2.3 describes monitoring well construction in detail. Figure 3-1 identifies the boreholes that were completed as monitoring wells.

3.2.1.2 Borehole Survey

Ground elevations and coordinate data for each borehole were surveyed by a CH2M HILL Idaho-licensed land surveyor. The geographic coordinates and ground elevations of each borehole were surveyed into a local datum established for the Ada County Landfill to within an accuracy of plus or minus 1.0 foot. Vertical elevations of the borings and wells were surveyed to an accuracy of 0.1 foot.

3.2.1.3 Soil Sampling, Soil Logging, and Field Tests

The soil borings described above were used to characterize the stratigraphy, geotechnical conditions, and soil properties at the NRC. At each borehole, geotechnical soil samples were initially collected at a minimum interval of every 10 feet utilizing a standard 2-inch-outside-diameter split-spoon sampler, in accordance with the requirements of the American Society for Testing and Materials (ASTM) procedures *ASTM D1586—Penetration Test and Split-Barrel Sampling of Soils*. At depth, relatively undisturbed samples were collected from the soil borings using a 5-foot continuous sampler with 3.5-inch outside-diameter Lexan liners. Both ends of each liner were covered with plastic caps and sealed with tape to preserve the sample and prevent moisture loss.

Standard Penetration Tests (SPTs) were performed by driving the split-spoon sampler with a 140-pound hammer free-falling 30 inches. The drill rigs used by Haz-Tech Drilling, Inc., were equipped with a hydraulic auto-hammer with an equivalent efficiency of approximately 80 percent. The number of blows to drive the sampler for each 6-inch increment of an 18-inch drive was recorded, with the sum of the blows of the last 12 inches recorded as the standard penetration resistance, or N-value. Refusal is noted as the condition in which the sampler is driven less than 6 inches after 50 blows, or greater than 6 inches but less than 12 inches after 100 blows. After the sampler was driven and the blow counts were recorded, the sampler was withdrawn from the boring and a soil sample (disturbed) was recovered.

Lithologies were logged based on soil samples collected utilizing cuttings, split-spoon and continuous core samplers. Representative soils from each split-spoon sample were placed in zip-lock baggies or glass sample jars to document the soils encountered at each borehole. Lithologies of the different strata encountered were recorded on drilling logs and classified in accordance with ASTM soil classification procedures (*ASTM D2488—Description and Identification of Soils (Visual—Manual Procedure)*. These classifications were reviewed and modified in the soil boring logs, as appropriate, following the completion of the laboratory testing program. The drilling logs provided in Appendix A show lithology and split spoon/continuous core sample depths.

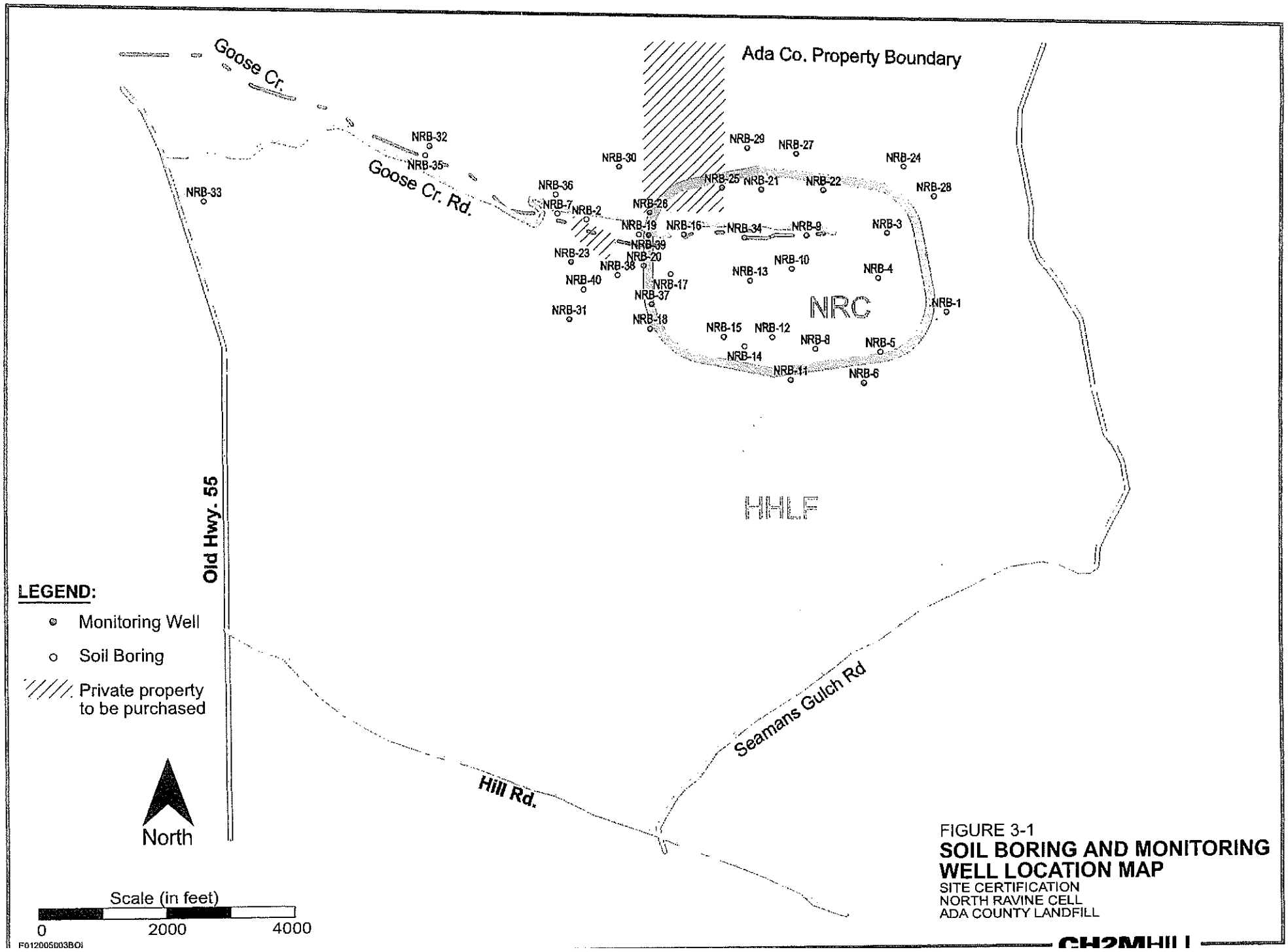

FIGURE 3-1
SOIL BORING AND MONITORING WELL LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

CH2MHILL

3.2.1.4 Open-Hole Percolation Testing

Open-hole percolation testing was performed in shallow boreholes NRB-2, NRB-7, and NRB-19 in accordance with *USBR 7310—Constant Head Hydraulic Conductivity Tests in Single Drill Holes* (U.S. Bureau of Reclamation, 1989). This test procedure consists of determining a test zone within a borehole, and measuring the volume of water that infiltrates into the subsurface through the borehole walls within that zone over a short period of time. In a borehole drilled with hollow-stem augers, the augers are retracted from the bottom of the hole until the lowermost auger is at the top of the targeted test zone. Then water is fed by gravity-flow into the open borehole by tremie pipe, until the head within the borehole reaches a level (usually near the ground surface) that can be easily monitored. Once this head level is attained, the flow is regulated to maintain the head at that position for the duration of the test. The volume of water added during the test is measured by a flow meter at the ground surface. The total volume is measured at timed intervals, usually 1-minute, until the flow rate stabilizes. Open-hole percolation test locations are shown in Figure 3-2.

3.2.1.5 Geotechnical Laboratory Testing

A laboratory testing program was performed on representative soil samples collected in the field. Index property testing was performed to determine the basic characteristics of the soils. Index properties include in situ moisture content, in situ density, gradation, Atterberg limits (as applicable), and classification. Soil strength and deformation tests were also performed to evaluate the behavior of the soils under the expected conditions associated with the proposed landfill construction. Corrosion properties were characterized by testing for resistivity and pH.

Geotechnical laboratory testing for the project was performed by American Geotechnics and Strata, Inc., both of Boise, Idaho. The laboratory testing methods are listed in Table 3-1.

TABLE 3-1
Geotechnical Laboratory Testing Methods

Test	Methods
Moisture Content	ASTM D2216—Laboratory Determination of Water (Moisture) Content of Soil and Rock
Atterberg Limits	ASTM D4318—Liquid Limit, Plastic Limit, and Plasticity Index of Soils
Particle-Size Analysis (Gradation)	ASTM D422—Particle-Size Analysis of Soils
Percent Fines	ASTM D1140—Amount of Material in Soils Finer Than the No. 200 (75-μ) Sieve
Consolidated Undrained Triaxial Compression	ASTM D4767—Consolidated-Undrained Triaxial Compression Test for Cohesive Soils.
One-Dimensional Consolidation	ASTM D2435—One-Dimensional Consolidation Properties of Soils
Hydraulic Conductivity – Flexible Wall Permeameter (Constant Volume)	ASTM D5084—Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter
Hydraulic Conductivity – Rigid Wall Permeameter (Constant Head)	ASTM D5856—Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter

TABLE 3-1
Geotechnical Laboratory Testing Methods

Test	Methods
Moisture-Density Relationship	ASTM D1557—Laboratory Compaction Characteristics of Soil Using Modified Effort (55,000 ft-lbf/ft3 (2,700 kN-m/m3))
Drained Direct Shear	ASTM D3080—Direct Shear Test of Soils Under Consolidated Drained Conditions
Resistivity	ASTM WK3025—Measurement of Soil Resistivity Using the Two-Electrode Soil Box Method
pH	D4972—pH of Soils

3.2.2 Surface Evapotranspiration Soils Assessment Program

3.2.2.1 Soil Test Pit Locations and Rationale

Ten test pits (HH-NR-TP1 through TP10) were excavated to observe soil profiles, collect bulk samples, and evaluate the suitability of the underlying soils in the NRC site for use in constructing the final ET cover. Ten test pits were excavated to depths between 7 and 10 feet bgs by Haile Laser Leveling of Parma, Idaho, on April 13, 2004, using a rubber-tired John Deere 310C backhoe. A CH2M HILL engineering geologist observed the excavation, logged the soils, and collected bulk samples from the test pits. Table 3-2 presents the test pit locations and depths. Soil Test pit locations are shown in Figure 3-3. Test pit logs are included in Appendix B.

TABLE 3-2
ET Test Pit Locations and Depths

Test Pit ID	Easting	Northing	Elevation (feet)	Test Pit Depth (feet)	Topsoil Thickness (feet)
HH-NR-TP1	361476.54	744377.42	3,210	9.8	4.0
HH-NR-TP2	360582.25	744309.18	3,195	7.0	2.5
HH-NR-TP3	361329.53	745563.19	3,077	8.0	3.0
HH-NR-TP4	360215.87	745749.04	3,065	8.0	1.5
HH-NR-TP5	359364.51	744505.89	3,120	6.0	3.5
HH-NR-TP6	359756.70	746488.42	3,048	8.0	1.5
HH-NR-TP7	361884.55	746319.78	3,178	8.0	2.0
HH-NR-TP8	358647.11	744509.22	3,110	7.0	1.0
HH-NR-TP9	358109.46	745183.9	3,025	7.0	2.5
HH-NR-TP10	359130.89	745759.93	2,995	8.5	2.8

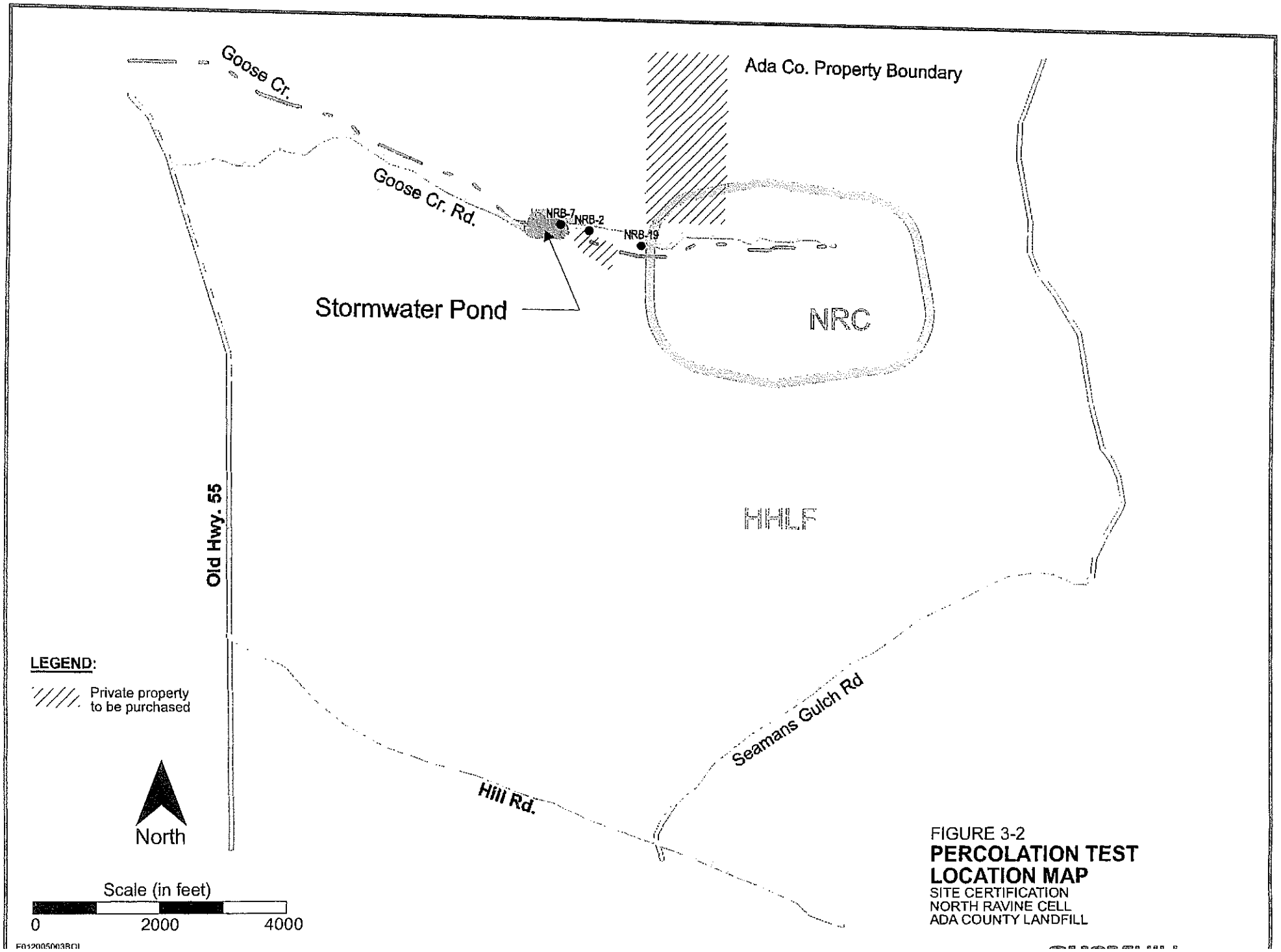

FIGURE 3-2
PERCOLATION TEST LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

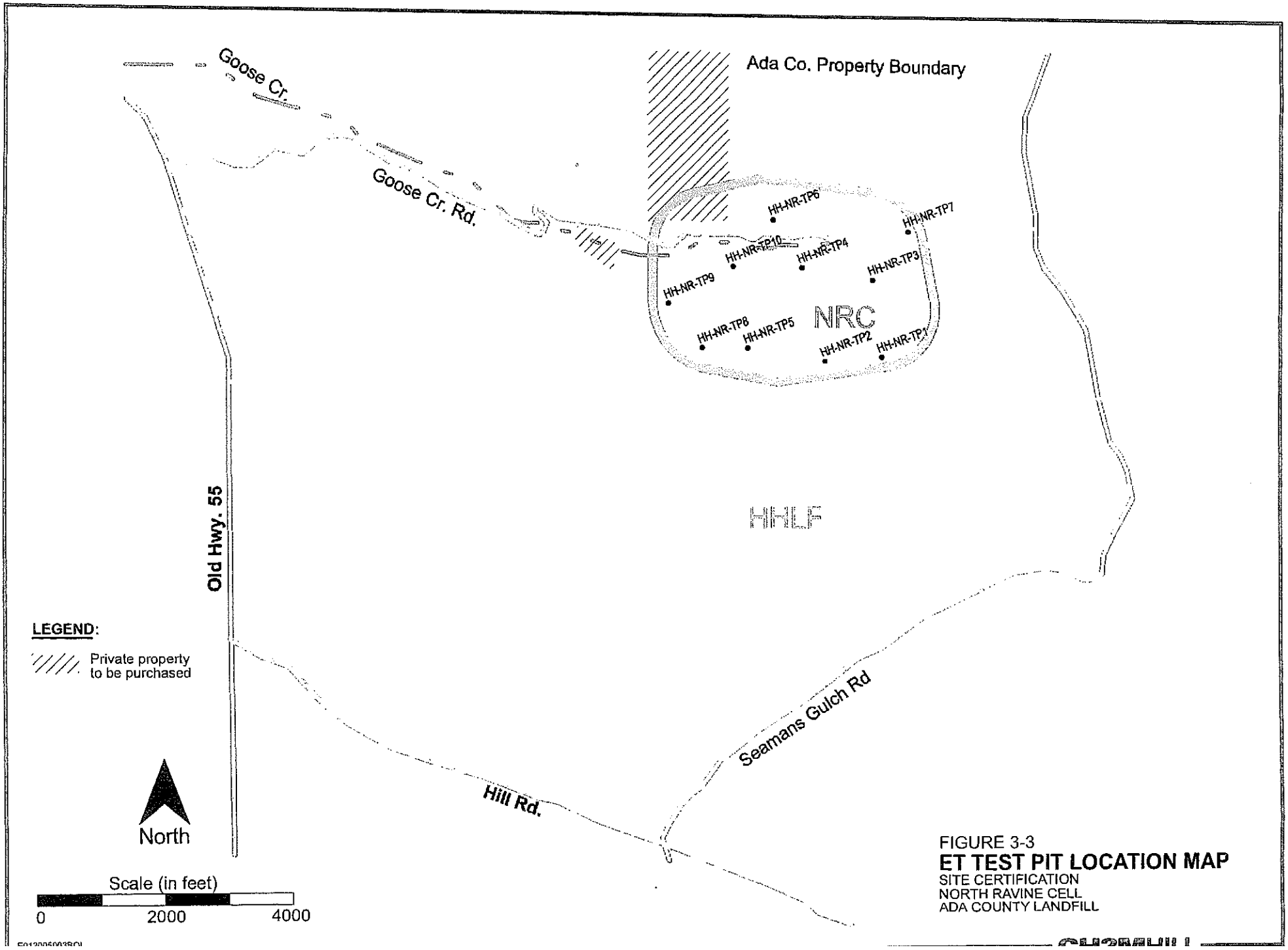

FIGURE 3-3
ET TEST PIT LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

The test pits were excavated at locations spread across the North Ravine site. The pits were excavated generally on or near ridge tops because material from the ridge tops, including the topsoil, will be excavated and stockpiled for use in the final ET cover when the NRC is closed. In addition, the test pits were useful in determining the thickness of topsoil for volumetric estimates and requirements for topsoil stripping during construction of the NRC.

3.2.2.2 Soils Characterization for Evapotranspiration Cover

Composite bulk topsoil samples were collected from the ten test pits. The composite samples represent a range of topsoil properties. Laboratory testing on three composite topsoil samples was performed by Daniel B. Stephens and Associates, Inc., at their hydrologic testing laboratory in Albuquerque, New Mexico. Each test sample was remolded to 85 percent of the maximum dry density at the optimum moisture content determined in accordance with ASTM D1557. The topsoil testing program yielded measurements of the following:

- Initial moisture content
- Bulk dry density
- Calculated porosity
- Saturated hydraulic conductivity (constant-head)
- Moisture retention by hanging column, pressure plate, water activity meter, and relative humidity box methods.
- Calculated unsaturated hydraulic properties

These test measurements provide data to develop the soil characteristic curves relating moisture content, pressure head, and hydraulic conductivity. The moisture characteristics tests and methods are listed in Table 3-3.

TABLE 3-3
Moisture Characteristics Tests and Methods

Test	Method*
Dry Bulk Density	ASTM D4531—Bulk Density of Peat and Peat Products
Moisture Content	ASTM D2216—Laboratory Determination of Water (Moisture) Content of Soil and Rock
Calculated Porosity	Klute, A., 1986. *Porosity*. Chap. 18-2.1, pp. 444-445, in A. Klute (ed.) Methods of Soil Analysis, American Society of Agronomy, Madison, Wisconsin
Saturated Hydraulic Conductivity	ASTM D2434—Permeability of Granular Soils (Constant Head)
Water Potential Method	Rawlins, S. L., and G. S. Campbell, 1986. Water Potential: Thermocouple Psychrometry. Chap. 24, pp. 597-619, in A. Klute (ed.) Methods of Soil Analysis, American Society of Agronomy, Madison, Wisconsin
Relative Humidity Box	Karathanasis and Hajek, 1982. Quantitative Evaluation of Water Adsorption on Soil Clays. Soil Science Society of America Journal 46:1321-1325
Calculated Unsaturated Hydraulic Conductivity	Soil Science Society of America Journal. 1980 44:892-898

* As reported by Daniel B. Stephens & Associates, Inc.

3.2.3 Groundwater Monitoring Program

3.2.3.1 Predesign Groundwater Monitoring Water Well Locations and Rationale

Twenty-eight predesign monitoring wells were installed to characterize groundwater flow and water quality of the uppermost water-bearing unit at the NRC. The predesign groundwater monitoring wells were constructed at strategic locations throughout the NRC study area including the proposed wetlands mitigation area. Monitoring well locations are included in Figure 3-1.

3.2.3.2 Predesign Groundwater Monitoring Well Construction

Monitoring wells were constructed in three phases in conjunction with the soil boring drilling program. The first phase of monitoring well installation occurred on August 25 through September 23, 2003, during which wells NRB-1, NRB-3, NRB-4, NRB-6, NRB-9, NRB-10, NRB-11, NRB-12, NRB-13, NRB-15, NRB-16, and NRB-18 were constructed. The second phase of monitoring well installation occurred on April 5 through May 6, 2004, during which wells NRB-21, NRB-22, NRB-24, NRB-25, NRB-26, NRB-27, NRB-28, NRB-29, NRB-30, NRB-31, NRB-32, NRB-34 and NRB-35 were constructed. The third phase of monitoring well installation occurred on July 29 through July 30, 2004, during which wells NRB-23, NRB-37, and NRB-39 were constructed.

The monitoring wells were constructed using flush-threaded, 2-inch-inside-diameter Schedule 40 PVC casing. The manufactured screen was positioned inside the hollow stem auger at the water table (uppermost water-bearing zone). Washed silica sand was placed over the entire screened section, followed by a bentonite seal to within 2 feet of the ground surface. The sand and dry bentonite were slowly poured into the hole while the hollow stem auger was slowly retracted, ensuring proper placement of the filter pack and seal. With the exception of NRB-33, an aboveground locking steel monument and well-casing cap completed the construction of these wells to discourage and prevent unauthorized access and tampering. At NRB-33, the well was completed with a flush-mount monument set in concrete. The casing at NRB-33 was capped with a locking J-plug. Detailed completion diagrams for these wells are presented in Appendix C.

3.2.3.3 Predesign Groundwater Monitoring Well Survey

Ground elevations, measuring points, and coordinate data for each groundwater monitoring well were surveyed by a licensed Idaho land surveyor. The geographic coordinates of each well were surveyed into a local datum established for the Ada County Landfill to within an accuracy of plus or minus 1.0 foot. The measuring point elevations for each well was established to an accuracy of plus or minus 0.01 foot.

3.2.3.4 Predesign Groundwater Monitoring Well Development

The groundwater wells were first developed by hand bailing using a 1.5-inch by 3-foot bailer to remove sand from inside the casing. When the bailing operation was complete, a 2-inch Grunfos Rediflo 2 submersible pump was used to remove the remaining fines from inside the casing and to develop the filter pack. Each well was pumped until the water was clear and no turbidity was noticeable, if possible. Wells that did not produce adequate water for pumping were hand-bailed until the water was as clear as possible. During development, three or more boring volumes were removed, if possible. A minimum of one

volume of water was removed from poor producing wells. During development, water levels and volumes were recorded to estimate the yield of each well. A well development summary for each well is shown in well construction diagrams in Appendix C.

3.2.3.5 Water Level Measurements

Water level measurements were collected at NRC monitoring wells on June 11, 2004, August 11, 2004, and December 20, 2004. Water level measurements were also collected at HHLF Cell monitoring wells to compare NRC water levels with area-wide water levels. Water levels were measured and recorded to the nearest 0.01 foot using an electronic water level sounder.

3.2.3.6 Groundwater Quality Sampling

Three rounds of groundwater sampling were performed at NRC monitoring wells. The first groundwater monitoring event was conducted on November 5 through November 10, 2003. During the November 2003 groundwater monitoring event, groundwater samples were collected from a selected set of available monitoring wells: NRB-3, NRB-6, NRB-11, NRB-12, NRB-15, and NRB-18. These samples were analyzed for VOCs using EPA Method 8260.

The second groundwater monitoring event was conducted on May 4 through May 19, 2004. During the May 2004 groundwater monitoring event, groundwater samples were collected from all available NRC monitoring wells: NRB-1, NRB-3, NRB-4, NRB-6, NRB-9 through NRB-13, NRB-15, NRB-16, NRB-18, NRB-21, NRB-22, NRB-24 through NRB-32, and NRB-34. Additionally, groundwater samples were collected from two local deep drinking water supply wells: the Equestrian Well located west of the proposed NRC Cell and the Ada County Sheriff's shooting range well located within the proposed NRC Cell's boundary.

The third groundwater monitoring event was conducted on August 17 through August 27, 2004. During the August 2004 groundwater monitoring event, groundwater samples were collected from a selected set of monitoring wells including NRB-1, NRB-3, NRB-4, NRB-9, NRB-12, NRB-13, NRB-15, NRB-21, NRB-23, NRB-24, NRB-25, NRB-26, NRB-28, NRB-30, NRB-32, NRB-34, NRB-37, and NRB-39.

The May and August 2004 samples were analyzed for VOCs using EPA Method 8260, unfiltered Resource Conservation and Recovery Act (RCRA) metals (antimony, arsenic, barium, beryllium, cadmium, chromium, cobalt, copper, lead, nickel, selenium, silver, thallium, vanadium, and zinc) using EPA Method 200.7/200.8, common ions (sodium, calcium, potassium, magnesium, bicarbonate, chloride, sulfate, and nitrate) using EPA Methods 200.7, 300.0, 310.1, and total organic carbon (TOC) using EPA Method 415.1.

Purging was performed using a bailer or a Grunfos Rediflo 2 submersible pump. Purging generally involved the removal of three well-bore volumes at a low rate of flow, approximately 0.1 to 1.5 gallons per minute (gpm), depending on the productivity of the well. If a well pumped or bailed to dryness, it was allowed to recover before sampling.

Field parameters (pH, temperature, turbidity, specific conductivity, and dissolved oxygen) were generally measured at a frequency of one well-bore-volume increments until they stabilized to within 10 percent over three consecutive well-bore volumes. If a submersible pump was used to sample a well, the flow was reduced to approximately 0.5 gpm or less to collect the sample. If a bailer was used to sample a well, the sample was collected directly

from the bailer drain spout. Field parameters were measured immediately following sample collection to confirm that field parameters had remained stable.

Well purge and sample results were documented in a field notebook and on individual groundwater sample forms for each well. Groundwater samples were immediately stored in a cooler with ice and hand-delivered to Alchem Laboratories of Boise, Idaho, under standard chain-of-custody practices.

3.2.3.7 Aquifer Testing

Twelve single-well aquifer tests were performed in the uppermost water-bearing unit in September 2004 and three single-well aquifer tests were performed in local deep water supply wells in February 2005. The purpose of the single-well aquifer tests was to determine the range and distribution of shallow and deep aquifer hydraulic properties within the NRC study area. Aquifer test locations are shown in Figure 3-4.

A total of seven slug tests were performed in the uppermost water-bearing unit at wells NRB-3, NRB-4, NRB-6, NRB-9, NRB-13, NRB-26, and NRB-31 and were comprised of both injection and withdrawal slug tests. Slug tests were performed using a pre-filled (distilled water) stainless steel bailer. To perform the slug tests, the pre-filled bailer was quickly lowered into the well, then removed after water levels had fully recovered. The water level recoveries were monitored and recorded for a minimum of 1 hour to a maximum of 114 hours using a 30-pound-per-square-inch (psi) In-Situ Incorporated miniTROLL data logger placed near the bottom of the well.

Five single well pumping tests were performed in the uppermost water-bearing unit at wells NRB-B-18, NRB-B-22, NRB-B-23, NRB-B-34, and NRB-B-37. Pumping tests were performed using a Grunfos Rediflo II submersible pump equipped with a one-way check valve to prevent backflow into the well. During drawdown, groundwater discharge was measured by timing the filling of a 5-gallon bucket. Drawdown continued until near-steady-state conditions were achieved. When drawdown approached steady-state, pumping was discontinued and groundwater was allowed to recover. Water levels during drawdown and recovery were monitored utilizing a 30-psi In-Situ Incorporated miniTROLL data logger placed just above the pump.

Data from the aquifer tests described above were stored on the miniTROLL until it was transferred to a personal computer. Analysis of field-scale aquifer properties of each well are discussed in Section 4.2.5. The tests were conducted in accordance with ASTM standard D4043-91, *Standard Guide for Selection of Aquifer-Test Method in Determining Hydraulic Properties by Well Techniques.*

A total of three aquifer tests were performed at three existing deep water supply wells in February 2005 to evaluate the hydraulic conductivity of the confining blue clay/silt unit that underlies the study site. The three deep wells included the Sheriff's Shooting Range well, the Terteling #2 well (T-2), and the Equestrian well. The aquifer tests at the Sheriff's Shooting Range and the Equestrian wells were performed using the existing submersible water supply pumps. During drawdown, groundwater discharge was measured by timing the filling of a 5-gallon bucket. Drawdown continued until near-steady-state conditions were achieved. When drawdown approached steady-state, pumping was discontinued and groundwater was allowed to recover. The aquifer test at the Terteling #2 well was

performed using a Grunfos Rediflo II submersible pump. Previous testing at the Terteling #2 well had indicated that this well readily draws down and is very slow to recover. Consequently, testing at this well was performed using a modified bail method consisting of rapid drawdown (approximately 10 feet) using the Grunfos pump, followed by a measured recovery. Water levels during drawdown and recovery at all three deep wells were monitored utilizing an electronic water level sensor. The locations of the three deep wells are included in Figure 3-4.

Note that Figure 3-4 includes HHLF Cell wells B-6, B-9, B-10, and B-23. Single well aquifer tests were performed at these wells as part of the HHLF characterization (CH2M HILL, 1996). The results from these previous HHLF aquifer tests are included in aquifer test discussion provided in Section 4.2.6.

3.2.3.8 Laboratory Permeability Testing

A laboratory permeability testing program was performed on representative continuous core (undisturbed) soil samples collected from the confining zone beneath the uppermost water bearing unit. Laboratory permeability testing measures the vertical hydraulic conductivity of a sample. The targeted confining zone underlies both the uppermost water-bearing unit and the proposed NRC liner.

Samples tested for vertical hydraulic permeability are listed in Table 3-4 and permeability sample test locations are shown in Figure 3-5. Seventeen confining zone samples were tested for vertical hydraulic permeability. Based on field logs, the texture of the confining zone material generally consisted of gray to blue to brown, silty lean clay to fat clay. Four additional samples from soils that overlie the confining zone were also tested to assess the vertical hydraulic conductivity within the vadose and saturated zones above the confining zone. The texture of these soils was highly variable, ranging from poorly graded sand to sandy silt to sandy clay.

Laboratory permeability testing was performed by American Geotechnics of Boise, Idaho. Testing methods included ASTM D5084—Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter and ASTM D5856—Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter. An effective stress ranging from 10 psi to 50 psi was applied to several undisturbed (continuous core) samples to evaluate how the soils respond to unloading (removal of overburden during the construction phase of the landfill) and loading (filling the landfill with solid waste). Additionally, several of the undisturbed samples were remolded to test how sample disturbance during sampling may have effected permeability measurements. Disturbance to samples (overcompaction of the sample or stress fracturing) often occurs during normal drilling/sampling activities and the degree of disturbance is controlled by the depth of drilling and properties of the material such as the degree of compaction and consolidation, texture, moisture content, cohesiveness, etc.

TABLE 3-4
Permeability Testing Sample Locations

Borehole ID	Sample ID	Depth	Zone	Field Description
NRB-1	SS-20	199-200.5	CZ	Dark gray, lean to fat clay
NRB-3	CS-16A	84.5-89.0	Overlies the CZ	Gray-brown sandy silt
NRB-3	CS-16B	87.5	Overlies the CZ	Gray-brown sandy silt
NRB-3	SS-21	109.5-111.0	CZ	Blue-gray lean clay with sand to fat clay
NRB-15	SS-11	108.5-110	CZ	Blue-gray lean clay
NRB-18	CS-15	123.5-128.5	CZ	Blue-gray lean clay with sand
NRB-22	CS-11C	82.7	CZ	Tan brown clayey silt
NRB-22	CS-12C	88.5	CZ	Blue silty clay with some sand
NRB-24	CS-4B	32	Overlies the CZ	Olive sandy lean clay
NRB-24	CS-6B	43.5	Overlies the CZ	Olive silty sand
NRB-29	CS-15B	131.5	CZ	Blue-gray sandy clay
NRB-30	CS-20A	185	CZ	Blue-gray lean clay
NRB-32	CS-11B	52.4	CZ	Blue-gray lean clay
NRB-32	CS-11B	53	CZ	Blue-gray lean clay
NRB-33	CS-11B	103	CZ	Blue-gray lean clay
NRB-33	CS-11B	103.6	CZ	Blue-gray lean clay
NRB-33	CS-12B	109.2	CZ	Blue-gray lean clay
NRB-34	CS-5B	43.5	CZ	Blue-gray lean clay
NRB-34	CS-5B	43.5	CZ	Blue-gray lean clay
NRB-35	CS-2B	12.6	UWBZ	Poorly graded sand
NRB-40	CS-14A	127	CZ	Gray-blue silty clay
NRB-40	CS-15A	132	CZ	Gray-blue silty clay

CZ = confining zone that underlies the uppermost water-bearing zone
UWBZ = Uppermost water-bearing zone

3.2.3.9 Goose Creek Channel Investigation

Test Pit and Piezometer Locations and Rationale. Eleven test pits (PGC-1 through PGC-11) were excavated along the Goose Creek channel at the NRC. Test pits PGC-1 through PGC-4 were excavated on April 14, 2004 and test pits PGC-5 through PGC-11 were excavated on April 20, 2004. The purpose of the test pit locations was to characterize shallow native soils and the depth of groundwater along the channel bottom of the Goose Creek drainage for the design of the groundwater capture/diversion and wetlands mitigation program at NRC. The test pits were excavated with a backhoe operated by Haile Laser Leveling. A CH2M HILL hydrogeologist observed the excavations, logged the soils and water levels, and installed piezometers in selected pits (see below). The location of the test pits are shown in Figure 3-5 and Table 3-5 summarizes the location and depth information of each test pit.

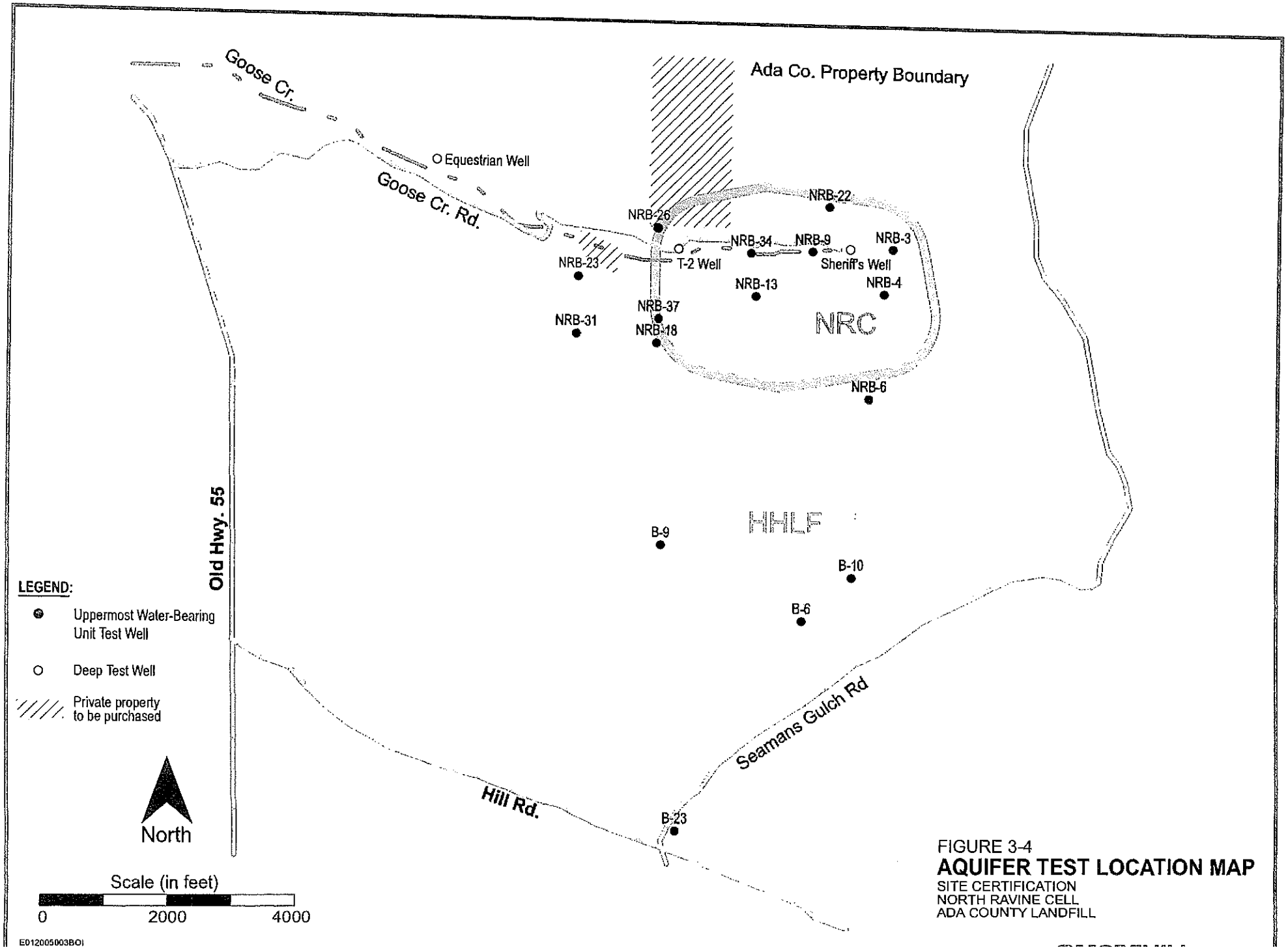

FIGURE 3-4
AQUIFER TEST LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

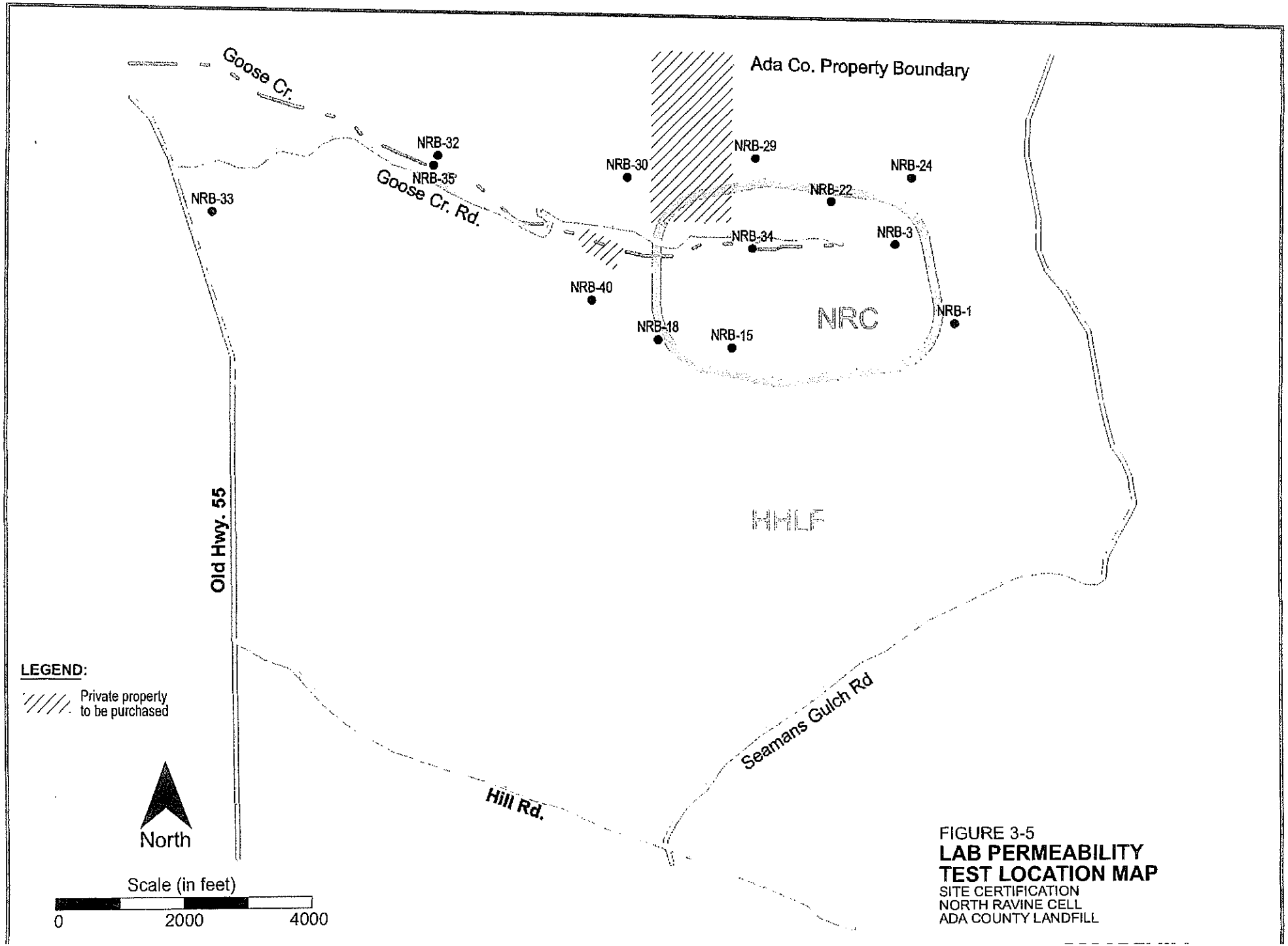

FIGURE 3-5
LAB PERMEABILITY TEST LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

TABLE 3-5
Goose Creek Test Pit Locations and Depths

Test Pit ID	Easting	Northing	Elevation (feet)	Test Pit Depth (feet)	Water Level[a] (feet bgs)
PGC-01	360610.91	746096.27	3,035	7.0	5.0
PGC-02	359565.64	746077.28	2,992	8.2	8.0
PGC-03	Not surveyed	Not surveyed	Not surveyed	9.5	dry
PGC-04	Not surveyed	Not surveyed	Not surveyed	9.5	dry
PGC-05[b]	360316.55	746123.37	3,020	7.0	6.71
PGC-06[b]	360039.47	746143.78	3,010	6.0	6.05
PGC-07[b]	359804.83	746100.46	3,000	6.5	6.89
PGC-08	359199.19	746106.3	2,968	11.0	dry
PGC-09[b]	359012.14	746090.63	2,958	10.0	5.52
PGC-10[b]	358683.06	746115.83	2,945	11.0	3.64
PGC-11[b]	358122.47	746014.36	2,930	6.5	6.02

[a] Water levels in PGC-5, PGC-6, PGC-7, PGC-9, PGC-10, and PGC-11 are based on water levels measured in the piezometers on December 23, 2004. Water levels at PGC-01, PGC-2, PGC-3, PGC-04, and PGC-8 are based on observed water levels in pit at the time of excavation.
[b] Test pit was completed as a shallow piezometer.

Piezometer Construction, Development, and Water Level Measurement. Shallow piezometers were installed at 6 of the 11 test pits along the Goose Creek channel at PGC-5, PGC-6, PGC-7, PGC-9, PGC-10, and PGC-11. The piezometers were constructed using flush-threaded, 2-inch-inside-diameter Schedule 40 PVC screen. The manufactured screen was positioned inside the test pit at the water table (first water-bearing zone). The test pit was then backfilled with native soils. Detailed completion diagrams for these piezometers are presented in Appendix C. The location of the piezometers is shown in Figure 3-6.

The piezometers were first developed by hand bailing using a 1.5-inch by 3-foot bailer to remove sand from inside the casing. When the bailing operation was complete, a Grunfos suction pump was utilized in piezometers with suitable yield to remove the remaining fines from inside the casing and to develop a natural filter pack. Each piezometer was pumped until the water was clear, if possible.

3.2.4 Soil Vapor Monitoring Program

During the course of characterization activities, soil vapor samples were collected on June 24, 2004, from methane monitoring well M-1, and ground water monitoring wells NRB-15, NRB-18, NRB-31 (groundwater wells with screens completed above the water table) to evaluate the potential for landfill gas migration from the HHLF Cell to the NRC. The ground water wells from which soil gas samples were collected are located north of the existing HHLF Cell. The soil gas samples were analyzed by EPA Method 8260B for VOCs which provides a screening level of analysis VOC vapors (detection concentrations in the part per million volume basis, ppmv, range). The analytical results from these screening

samples indicated VOCs were below detection in the soil gas samples from groundwater monitoring wells (NRB-15, NRB-18, and NRB-31), and two VOCs (refrigerants) were detected in M-1.

However, methane sampling at Methane Well M-1 (a former landfill perimeter soil gas detection well located north of the existing HHLF Cell) indicated that a spike in methane levels had occurred compared to historical data. The data indicated the possibility of changed landfill gas migration conditions in the vicinity of M-1 and possibly into the area defined by the proposed footprint of the NRC. To characterize the nature and extent of landfill gas in soils, a limited soil vapor monitoring program was performed as described in following text.

3.2.4.1 Soil Vapor Monitoring Well Locations and Rationale

To supplement vapor data collected from Methane Well M-1, three additional vapor monitoring wells, M-5, M-6, and M-7 were constructed at strategic locations along the south side of the NRC, adjacent to the existing HHLF Cell. The new soil vapor wells were completed within the vadose zone at depths intended to characterize the extent and nature of vapor migration from the existing HHLF Cell into the NRC. The locations of the soil vapor monitoring wells are shown in Figure 3-7.

3.2.4.2 Soil Vapor Monitoring Well Construction

Monitoring wells were constructed in conjunction with the third phase of the soil boring drilling program. Vapor wells M-5, M-6, and M-7 were constructed between August 2 and August 4, 2004, by Haz-Tech, Inc. of Meridian, Idaho, using a CME 850 track-mounted drill rig. The vapor monitoring wells were constructed using flush-threaded, 2-inch-inside-diameter Schedule 40 PVC casing. The manufactured screen was positioned inside the hollow stem auger at selected depths above the water table. Washed silica sand was placed over the entire screened section, followed by a bentonite seal to within 2 feet of the ground surface. The sand and dry bentonite were slowly poured into the hole while the hollow-stem auger was slowly retracted, ensuring proper placement of the filter pack and seal. An aboveground locking steel monument and well-casing cap completed the construction of these wells to discourage and prevent unauthorized access and tampering. Detailed completion diagrams for these wells are presented in Appendix C.

3.2.4.3 Soil Vapor Well Survey

Vertical elevations and coordinate data for vapor well M-1 was surveyed by a CH2M HILL Idaho-licensed land surveyor. The geographic coordinates and ground elevations were surveyed into a local datum established for the Ada County Landfill to within an accuracy of plus or minus 1.0 foot. The vertical elevation was surveyed to an accuracy of 0.1 foot.

Coordinate data for each of the new vapor monitoring wells (M-5, M-6, and M-7) were surveyed by a CH2M HILL field scientist using a Garmin Geko 201 GPS field unit. The GPS geographic coordinates were converted into the State Plane NAD 27 Idaho West-1103 coordinate system to be consistent with the local datum established for the Ada County Landfill. The coordinates were surveyed to within an accuracy of plus or minus 20.0 feet. Vertical elevations of the borings and wells at M-5, M-6, and M-7 were determined from the topographic survey described in Section 3.1 and are assumed to be within an accuracy of plus or minus 5 feet.

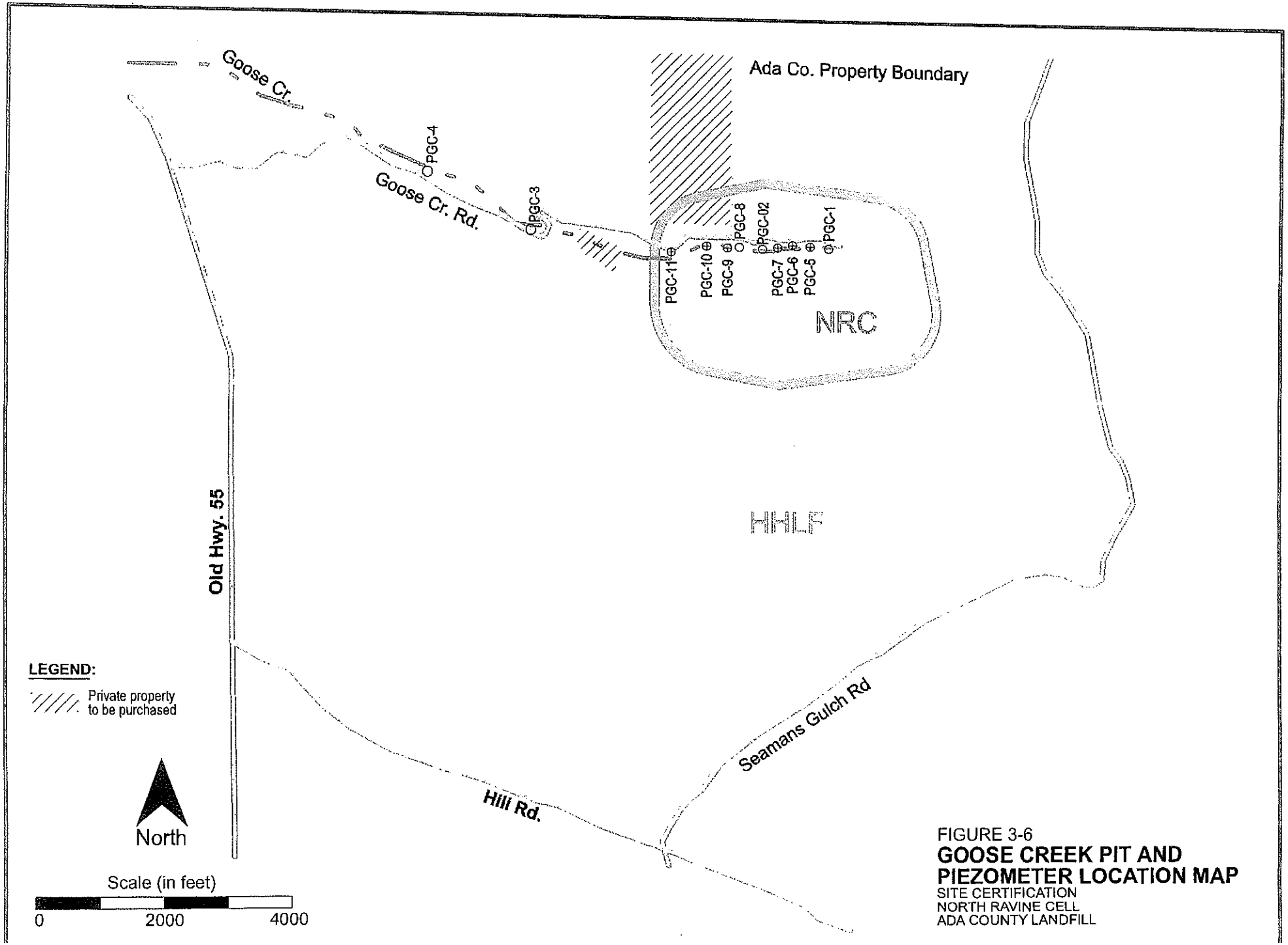

FIGURE 3-6
GOOSE CREEK PIT AND PIEZOMETER LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

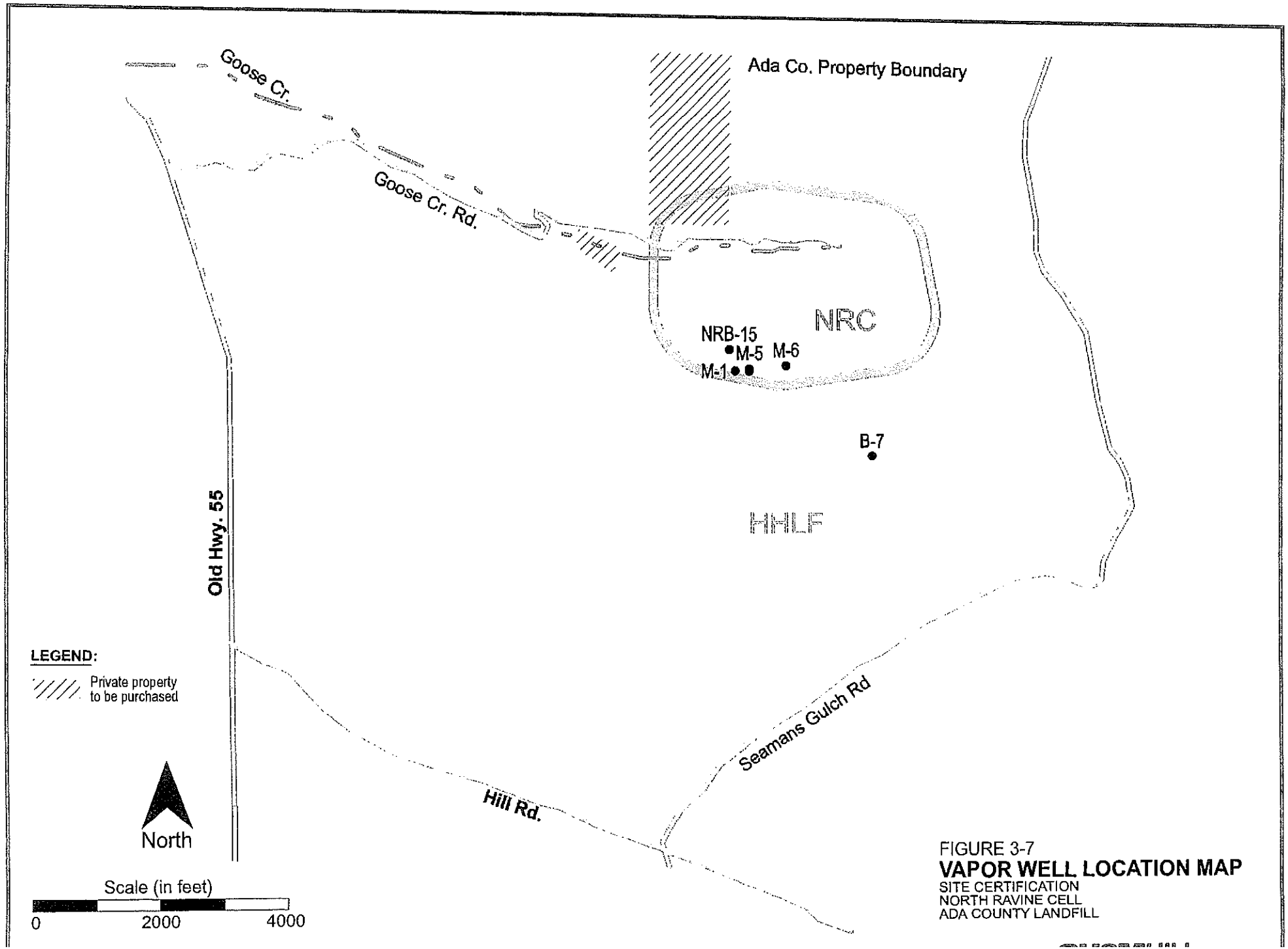

FIGURE 3-7
VAPOR WELL LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

3.2.4.4 Soil Vapor Sampling

After approximately 8 weeks, on October 6, 2004 (time for soil gas conditions to recover from well installation disturbances), soil gas samples were collected from vapor monitoring wells M-1, M-5, M-6, and M-7. Soil gas samples were also collected from nearby groundwater monitoring wells NRB-15 and HHLF Cell monitoring well B-7. These groundwater monitoring wells include well screen above the water table allowing the sampling of soil gas.

To collect a vapor sample, sample tubing was attached to the wellhead and the wellhead was sealed to prevent the leakage of ambient air into the casing. The well was then purged of a minimum of three casing volumes of air prior to collecting the sample. The wells were purged using an Air Dimensions eccentric vacuum pump at a rate of 20 to 28 liters per minute. The flow rates were monitored with an inline rotometer. The samples were collected into 1-liter Tedlar bags provided by the analytical laboratory.

During purging, fixed gases (CO_2, O_2, and methane) were monitored using a Landtec GEM 2000 landfill gas monitor. At the end of purging, vapor samples were collected for laboratory analysis by Method TO-15. Method TO-15 includes the significant VOCs potentially present in landfill gas and has analytical detection levels at very low concentrations (part per billion volume, ppbv, range). The samples were shipped overnight to Data Chem Laboratories of Salt Lake, Utah, for analysis.

3.3 Location Restrictions Investigation

Location restrictions for the NRC were investigated in accordance with 40 CFR 258, Subpart B, Criteria for Municipal Solid Waste Landfills. The location criteria of 40 CFR 258, Subpart B address the following criteria for siting an MSW landfill:

- Airport safety
- Floodplains
- Wetlands
- Fault Areas
- Seismic impact zones
- Unstable areas

The methods used to evaluate each of the criteria listed above relative to the siting of the NRC are summarized in the following text. In accordance with 40 CFR 258, Subpart B, the employed methods were designed to assess the effect that the proposed MSW landfill unit may have on the environment and the effects that natural and human-made conditions may have on the performance of the landfill unit.

3.3.1 Airport Safety Investigation

In accordance with 40 CFR 258.10, distances to the runway part of nearby public airports were investigated using existing maps that show the relationship of existing runways to the NRC. All public airport runways within a 5,000-foot radius of the NRC and used by piston aircraft or within a 10,000-foot radius of the NRC and used by turbojet aircraft were identified.

3.3.2 Floodplain Investigation

In accordance with 40 CFR 258.11, the NRC was evaluated for the possibility that it is within a 100-year floodplain. The investigation consisted of a field reconnaissance along Goose Creek and its tributaries to identify overbank deposits and a review of available Flood Insurance Rate Maps (FIRMS) and the Federal Emergency Management Agency (FEMA) floodway maps.

3.3.3 Wetlands Investigation

In accordance with 40 CFR 258.12, the NRC was evaluated for the presence of jurisdictional wetlands. The indicators assessed for the presence of jurisdictional wetlands, as defined in the *Corps of Engineers Wetland Delineation Manual* (U.S. Army Corps of Engineers, 1987), were hydrophytic vegetation, hydric soils, and wetland hydrology.

Preliminary site visits, topographic maps, and field indicators were used to evaluate potential wetland areas within the proposed NRC. Plant keys were used to identify unknown plant species on wetland sites. The boundaries of wetlands were defined in the field using routine determination methods referenced in the *Corps of Engineers Wetland Delineation Manual* (U.S. Army Corps of Engineers, 1987). Dominant vegetation was identified and percent cover estimated. Hydric soils were identified using field observation for gleying, mottling, oxidized root zones, sulfuric odor, moisture status, and other hydric soil indicators. A Munsell Soil Color Chart was used to determine soil color. Wetland areas were staked in the field and clearly marked. A survey crew then mapped the staked areas.

Formal delineation procedures consisted of the following:

- Digging soil pit
- Measuring and defining soil color, texture, and depth for each soil horizon
- Recording vegetation and estimating canopy cover for each specie
- Recording hydrologic characteristics
- Completing Wetland Field forms
- Staking wetland boundaries with clearly labeled and flagged survey stakes

3.3.4 Fault Area Investigation

3.3.4.1 Holocene Faulting

In accordance with 40 CFR 258.13, new MSW landfill units shall not be located within 200 feet of a Holocene fault. A Holocene fault is defined in 40 CFR 258.13 as having displacement within Holocene time (within the last 10,000 years, or the Holocene Epoch).

In order to evaluate whether any Holocene and/or pre-Holocene faults were in the vicinity of the proposed NRC, available geologic information was compiled and evaluated. This information included the following:

- **Published geologic maps of the area.** A literature review was conducted to evaluate the presence of Holocene faults within a 3,000 foot radius of the site. Geologic maps were used to identify any mapped faults in the project area, in particular faults that were suspected to be Holocene.

- **Soil boring data.** Subsurface data from the 40 soil borings advanced in the vicinity of the proposed NRC, as well as available soil borings from the existing HHLF Cell were evaluated to determine if any offsets of stratigraphic units were present that may indicate faulting.
- **Aerial photograph analysis.** Aerial photographs of the area were analyzed for any surficial trace of faults, surface offsets, scarps, or other suspicious lineations that might indicate a Holocene fault.
- **On-site geologic mapping.** During the course of the field investigations, field geologists searched for any evidence of surface faulting, including scarps, abrupt changes in stratigraphy, and offset geologic contacts.
- **Exploratory trenching.** Exploratory trenches and test pits were excavated where surface observations, primarily stratigraphic changes, and subsurface data indicated a possible fault trace. The trenches were logged, photographed and examined for evidence of Holocene faulting.

3.3.5 Seismic Impact Zone Evaluation

The NRC site was evaluated to determine whether it is located within a Seismic Impact Zone, as defined by 40 CFR 258.14. The seismic impact investigation was completed by: 1) researching literature that shows probabilistic earthquake accelerations; 2) examining the site vicinity for indications of past seismic impacts; and 3) assessing existing seismic vulnerabilities that could be adversely impacted by a future seismic event.

Indicators of past seismic impacts include earthquake-induced features including sand boils, stream offsets, surface ruptures, and subsidence. Evidence of these features was searched for during the field reconnaissance portion of the site investigation.

Indicators of potential seismic vulnerabilities include the presence of Holocene faults, potentially unstable slopes, and liquefaction-susceptible soil deposits. The possible presence of these features were investigated by geological field reconnaissance, aerial photograph review, literature review, soil borings, test pit excavations, groundwater level measurements, SPTs, and laboratory testing of the soil gradations.

3.2.5.1 Predicted Ground Acceleration

The predicted ground acceleration for the North Ravine site was investigated by researching probabilistic ground motion values published by the U.S. Geological Survey (USGS). The maximum firm-rock ground acceleration with 10 percent or greater probability of exceedence in 250 years was determined using National Seismic Hazard Mapping Project data (USGS, 2002). The USGS provides ground motion data in terms of 2 percent probability of exceedence in 50 years. These data are approximately equivalent to the data for 10 percent probability of exceedence in 250 years. The return periods for these two exceedence probabilities are 2,476 years and 2,373 years, respectively. Based upon typical relations between ground motion and return period, the corresponding ground motions differ by 1 percent or less (USGS, 2004).

3.2.5.2 Liquefaction Potential

Liquefaction is a temporary loss of soil strength and stiffness because of lateral spreading or flow failure that occurs during seismic shaking. Liquefaction occurs under three conditions: the sediments must be saturated with ground water, the sediments must be composed of sand- or silt-sized particles, and the sediments must be compacted very loosely. The possible presence of liquefaction-susceptible soil deposits was investigated by geological field reconnaissance, literature review, soil borings, test pit excavations, groundwater level measurements, SPTs, and laboratory testing of the soil gradations.

3.2.5.3 Geologic Structure

Geologic structures that pertain to seismic impacts include primarily Holocene faults and liquefaction potential. Field geologic mapping and exploratory trenching were used to evaluate the geologic structure and its potential seismic impact in the project vicinity. Field mapping was conducted primarily to locate the contacts between the geologic units outside of the areas where drilling was conducted. Also, field mapping in conjunction with exploratory trenching was used to evaluate the presence of surface faults.

3.3.6 Unstable Areas Investigation

The NRC site was evaluated for the potential for unstable areas, as defined by 40 CFR 258.15. Site stability investigations with respect to seismic impacts focused on searching for typical indicators of unstable sites, including weak foundation conditions, foundation conditions susceptible to excessive settlement or subsidence, evidence of past slope instability, and past mass wasting and movement that may be influenced by seismic activity. Methods of investigations for stability included geological field reconnaissance, literature review, soil borings, aerial photograph reconnaissance, test pit excavations, groundwater level measurements, SPTs, and laboratory testing for strength and deformation properties of the subsurface soils.

3.3.6.1 Foundation Assessment

The site was evaluated for weak foundation conditions, foundation conditions susceptible to excessive settlement or subsidence, and compressibility of foundation soils. Methods of investigation included geological field reconnaissance, literature review, soil borings, test pit excavations, groundwater level measurements, SPTs, and laboratory testing for strength and deformation properties of the subsurface soils.

3.3.6.2 Settlement

The site was assessed to determine whether the site is stable from the perspective of compressibility. Estimates for the magnitude of total and differential settlements that may occur were calculated, and the impacts of those settlements on liner and leachate drainage systems were evaluated.

The settlement analysis included consolidation tests performed on undisturbed samples of the blue clay and thin overlying brown clays and silts from the site, and a one-dimensional consolidation analysis to estimate the magnitude and rate of consolidation settlement at two locations including the center of the landfill and the location with the maximum waste thickness. In addition, immediate compression settlement was also estimated using linear

strain analysis and added to consolidation settlement for a conservative estimate of the total settlement that could occur under the NRC.

3.3.6.3 Slope Stability

Analyses of potential slope failure on circular and planar (block sliding) surfaces were performed for the NRC. The purpose of these analyses is to determine if the site is stable prior to waste placement, during operations, and after final closure. The analysis evaluated veneer, block (planar), and circular sliding surfaces.

Global analyses were performed with the STABL5M software program in the STEDwin version 2.70 editor (Van Aller, 2002). This program uses three methods of general limit equilibrium analysis: simplified Janbu, modified Bishop, and Spencer. The simplified Janbu method, used by STEDwin in computing factors of safety when searching for the critical failure surface in block sliding, is known to compute conservative (lower than actual) factors of safety because it assumes no inter-slice forces. Spencer's method, a method which considers vertical force, horizontal force, and moment equilibrium, was used to re-analyze a critical block sliding surface identified by the simplified Janbu method. The modified Bishop method, used by STEDwin to search for and analyze circular failure surfaces, is generally satisfactory.

3.3.6.4 Karst Terrain

The NRC site was evaluated for presence of karst terrain, which includes sinkholes, sinking streams, caves, large springs, or blind valleys, and underlying bedrock such as limestone, dolomite, or other soluble rocks. Field reconnaissance included borehole logging (see Section 3.2) and windshield survey. Aerial photo reconnaissance and a literature review were also performed to evaluate the presence of Karst terrain.

3.3.6.5 Mass Wasting and Movement

Literature searches and geologic reconnaissance were conducted at the NRC site for evidence of mass wasting or mass movement, including landslides, avalanches, debris slides, debris flows, soil fluction, block sliding, rock fall, or other features of structural instability in the site geology.

3.4 Well Receptor Inventory

In a letter dated December 23, 2004, CH2M HILL, on behalf of Ada County, submitted a well inventory search area proposal for the NRC to DEQ. A map showing the proposed well inventory search area is shown in Figure 3-8. On January 19, 2005, DEQ provided written approval of the proposed well inventory search boundary, contingent on the findings of this Site Certification Report. Available Well Driller's Reports (well logs) were obtained from the Idaho Department of Water Resources to identify any water supply wells within the approved search area shown in Figure 3-8. Copies of available Well Driller's Reports are provided in Appendix E.

4.0 Data Analysis

4.1 Surface DTM and NRC Siting Footprint

A three-dimensional DTM of the footprint of the NRC is provided in Figure 4-1. The NRC siting footprint is intrinsic to the completion of an acceptability analysis for the NRC because the cell footprint defines the area of potential impact. The major features of the NRC footprint are as follows:

- Bottom slopes and elevation of the cell liner
- Elevation of the top of the cell liner
- Side slopes of the cell liner
- Side slopes of the final cover
- Top elevation of the final cover
- Horizontal location of the cell

The results of data collected to characterize the site are presented below. An acceptability analysis for the NRC is presented in Chapter 5.

4.2 Site Geology/Hydrogeology

4.2.1 Geotechnical Data

Geotechnical soil samples were collected from every borehole to facilitate the design, construction, operation, and closure of a cost-effective new landfill cell (the NRC) that is protective of both the public health and the environment. The geotechnical soil samples were collected utilizing a split-spoon sampler or a continuous sampler. Table 4-1 lists pertinent information for boreholes drilled at the NRC. The locations of the NRC boreholes are shown on Figure 4-2. Copies of geotechnical laboratory reports are included in Appendix F. The results of the geotechnical sampling program are provided below.

4.2.1.1 Field Geotechnical Testing

Field geotechnical testing during site characterization consisted primarily of SPTs and pocket penetrometer testing. SPTs provide an estimate of the relative density of granular soils and consistency of fine-grained cohesive soils. These data are primarily related to soil strength and will be used in evaluating slope stability and settlement potential at the proposed NRC.

Because of the widely variable grain size of the soils across the site, SPT blow counts also had a wide range. The results of the SPTs (blow counts) are shown as the N-values on the soil boring logs, which are included in Appendix A.

Pocket penetrometer tests were conducted on relatively undisturbed fine-grained soil samples collected from the split spoons. The pocket penetrometer provides an estimate of the unconfined compressive strength of cohesive soils and was used primarily in samples of

the blue clay. The pocket penetrometer test results indicated that the unconfined compressive strength of the blue clay was typically greater than 4.5 tons per square foot, which is the maximum range of the pocket penetrometer. Complete pocket penetrometer test results are shown on the "Comments" column of the soil boring logs (see Appendix A).

4.2.1.2 Open-Hole Percolation Testing

Open-hole percolation testing was performed in shallow Boreholes NRB-2, NRB-7, and NRB-19. Table 4-2 presents the results of the open-hole percolation testing.

The test results provide a range of in-situ permeability values for the native soils along the bottom of the Goose Creek drainage, and to the west of the NRC footprint (see Table 4-2). These data will be used in estimating infiltration rates and storage losses through the bottom of the proposed stormwater pond and other surface water control features downstream of the NRC (Figure 3-2). The range of percolation rates was 1.5E-05 centimeters per second (cm/sec) to 4.0E-03 cm/sec which is consistent for published values for silty and sandy soils tested (Freeze and Cherry, 1979).

TABLE 4-2
Open-Hole Percolation Test Results

Borehole	Test Interval (feet bgs)	Test Head (feet)	Hydraulic Conductivity (cm/sec)	Material
NRB-19	5 to 8.6	3.6	3.4E-04	Silty sand
NRB-19	5 to 8.8	8.6	4.7E-04	Silty sand
NRB-19	10 to 18.7	10	3.1E-05	Silty sand
NRB-2	5 to 10	5.2	1.5E-05	Silty sand
NRB-2	10.5 to 19	9	8.1E-04	Silty sand
NRB-7	0.4 to 8.4	8	2.0E-04	Silty sand
NRB-7	10.3 to 18.8	9.2	1.1E-03	Silty sand
NRB-7	10.3 to 18.8	12.1	4.0E-03	Silty sand

4.2.1.3 Summary of Geotechnical Laboratory Testing

A comprehensive geotechnical laboratory testing program was undertaken to evaluate the properties of the soils in the vicinity of the proposed NRC. These data will be used in evaluating soil properties including classification, strength, compressibility, compaction requirements, and corrosivity. The results of the laboratory testing are summarized in Tables 4-3a and 4-3b. The following is a brief discussion of the results of the laboratory testing performed.

4.2.1.3.1 Index Test Results. Index testing, including in-situ moisture content, gradation, percent fines (passing #200 sieve), and Atterberg Limits, was performed on most of the samples listed in Table 4-3. These tests are used to determine the basic properties and Unified Soil Classification System (USCS) classification of the soil samples.

In-Situ Moisture. The in-situ moisture gives an indication of the moisture content of the soils in their natural state. The in-situ moisture content can be compared to optimum moisture requirements for compacted soils. The moisture content is typically higher in fine-grained soils (silts and clays) and lower in granular soils (sands).

The results of the moisture content testing show that the moisture content in sandy soils ranged from 2.6 to 20.5 percent. Among these, the higher moisture contents occurred in the silty sands and clayey sands. Moisture contents of the silty soils ranged from 20.8 to 36.0 percent. Moisture contents of the clayey soils ranged from 26.6 to 38.0 percent in the fat clays (clays with relatively high plasticity) and from 22.6 to 41.7 percent in the lean clays (clays with relatively low plasticity).

Gradation and Percent Fines. The gradation of a soil sample shows the grain-size distribution and whether the soils are primarily fine or coarse grained. The coefficient of uniformity and the coefficient of curvature computed from the grain-size distribution curve determine whether granular soils are well-graded or poorly graded. The percent fines (percent passing the #200 sieve, or "P200") gives an indication of the percentage of the soil by weight that falls into the silt and clay category. Gradations and P200s were run on numerous soil samples to provide an accurate classification.

Gradation analyses showed that the sandy soils at the proposed NRC site include poorly-graded silty fine sands, clayey sands, poorly-graded to well-graded silty sands, and well-graded sands. The gradations and P200s also showed that the silty soils contained between 9 and 34 percent sand, and the clayey soils contained between 2.6 and 49 percent sand. Therefore, the fine-grained soils are typically fine sandy silts to fine sandy clays.

Atterberg Limits. Atterberg Limits use the Liquid Limit (water content of an unconsolidated material where it passes from a plastic solid to a turbid liquid) and Plasticity Index (the numerical difference between the liquid limit and the plastic limit of a soil) to evaluate the plasticity of soils, and, therefore, determine if soils classify as silts or clays based on the Plasticity Chart (ASTM D2487). Atterberg Limits were performed on several of the fine-grained soils, and on some of the sandy clays that appeared to have appreciable plasticity.

Based on the Atterberg Limits, the fine-grained soils classified as silt, lean clay, and fat clay. The liquid limits of the silts ranged from 35 to 43, and plasticity indexes ranged from 7 to 16. The liquid limits of the lean clays ranged from 35 to 49, and plasticity indexes ranged from 11 to 29. The liquid limits of the fat clays ranged from 50 to 73, and plasticity indexes ranged from 24 to 44.

4.2.1.3.2 Compaction Test Results. Compaction tests were performed on composite topsoil samples and bulk coarse sand samples collected from the 10 ET test pits. The compaction tests determine the relationship between gravimetric moisture content and dry bulk density of the soils using a specified compaction effort. These data will be used to determine the compaction characteristics including maximum density and optimum moisture content.

The results of the compaction tests show that the topsoil samples had maximum densities between 117.5 pounds per cubic foot (lb/ft^3) at 12. 5 percent moisture and 129.0 lb/ft^3 at 8.0 percent moisture. The coarse sandy samples had maximum densities between 126.5 lb/ft^3 at 9.0 percent moisture and 132.0 lb/ft^3 at 7.5 percent moisture. Compaction test results are also discussed under *Compaction Characteristics* in Section 4.2.3.

4.2.1.3.3 Shear Strength Results. Soil strength and deformation tests were performed on selected samples. These data will be used to evaluate the behavior of the soils under the expected conditions associated with the proposed landfill construction. These tests included consolidated undrained triaxial tests with pore pressure measurements and drained direct shear tests. These tests yield the Mohr-Coulomb parameters that define the strength of soils, which include the angle of internal friction and cohesion. The undrained triaxial shear strength tests were performed on relatively undisturbed silty and clayey samples retrieved using the continuous sampler. The drained direct shear tests were performed on silty sands from split-spoon samples, and silt from the continuous sampler.

The triaxial tests showed that under lower normal stresses (zero to 16,000 pounds per square foot (psf)) the effective peak friction angle ranged from 23 to 36 degrees, and the effective peak cohesion ranged from zero to 4,000 psf. However, under increased normal stresses above 16,000 psf, the effective peak friction angle in some of the samples decreased to between 16 and 18 degrees, whereas the cohesion increased to between 6,000 and 6,800 psf. The drained direct shear tests showed total friction angles in the silty sands and silts range from 34 to 39 degrees, with cohesion values that ranged from 300 to 700 psf.

4.2.1.3.4 Corrosion Test Results. Corrosion tests were performed on selected samples. These data will be used in evaluating the potential for corrosion of steel and concrete. Corrosion properties include testing for resistivity and pH. In general, sandier soils have a higher resistivity and fine-grained soils have a lower resistivity. The results of the corrosion tests indicate that the resistivity values ranged from 900 ohm-centimeter (ohm-cm) for the blue clay to 7,340 ohm-cm for well-graded sand. The pH of the soils was relatively neutral and ranged from 6.0 to 8.0.

4.2.1.3.5 Consolidation Test Results. Consolidation tests were performed on selected samples. These data will be used in evaluating time-dependent deformation and settlement rate of the fine-grained soils. Immediate and primary consolidation settlement properties of the site soils were evaluated using one-dimensional consolidation tests. These tests were performed primarily on silty clays and clayey fine sands.

The laboratory test results indicate the cohesive foundation soils are over-consolidated, with preconsolidation pressures ranging from 13,000 to 31,500 psf, and overconsolidation ratios (OCRs) ranging from 1.6 to 34. These evaluations confirm the field observations that the cohesionless soils are dense to very dense, and the cohesive soils are hard and overconsolidated. The overconsolidation is attributable to past loading caused by overlying overburden materials, such as the sediments of the Pierce Gulch formation.

4.2.2 Site Geology

Well logs and cores from boreholes and monitoring wells onsite indicate a consistent lithology throughout the study area. In general, fine- to coarse-grained arkosic sand with occasional fine gravel and/or silty seams overlie interbedded layers of silty to clayey fine sand and sandy silt which then overlie a massive blue clay/silt unit with occasional thin layers of fine sand. A detailed discussion of the site geology is presented in the following text.

4.2.2.1 Geologic Formations

The geology of the site consists of three geologic units that include stream (Quaternary age) alluvium, the Pierce Gulch Formation, and the Terteling Springs Formation. A general description of each unit follows. Additional information on the Pierce Gulch and Terteling Springs formations based on soil boring information and geologic mapping are included under Sections 4.2.2.2, 4.2.2.3, and 4.2.2.4.

4.2.2.1.1 Quaternary Alluvium. Quaternary-age stream alluvium is found primarily in the bottom of the Goose Creek drainage and in some of the tributary ravines. The stream alluvium is primarily comprised of brown to gray, fine to coarse grained, interbedded sandy to silty sand material derived from surrounding hillsides and redeposited in the valley bottoms. Based on drilling information, the alluvium is up to 30 feet thick in some areas of the Goose Creek drainage.

The portions of the proposed cell that intersect drainage bottoms will probably be constructed within Quaternary-age stream alluvium. The relationship of the cell bottom to Quaternary-age stream alluvium is illustrated on the geologic cross sections presented in Section 4.2.2.3.2.

4.2.2.1.2 Pierce Gulch Formation. The Pierce Gulch formation was formerly called a member of the upper Idaho Group. The formation is Tertiary in age and is generally characterized as a medium to coarse-grained arkosic sand, subangular, light gray to tan in color, compacted and not cemented (Beck, 1989; Burnham et al., 1992; and Othberg, 1992). The Pierce Gulch Formation lies conformably above the Terteling Springs Formation.

Throughout the NRC study area, the observed textures of the Pierce Gulch Formation and its contact with the underlying Terteling Springs Formation were consistent with previous studies of the upper members of Idaho Group. These studies describe the upper members of Idaho Group as sands, claystones, and siltstones resulting from fluvial, deltaic, and lacustrine depositional environments (Castelin and Petrich, 2005). Throughout the study area, the Pierce Gulch Formation sands were observed to form the upper erosional surface in the present topography (with the exception of quaternary alluvium at the drainage bottoms). The Pierce Gulch sands ranged in thickness from less than a few feet along the drainage bottom at Goose Creek to more than 315 feet along the ridge tops surrounding the NRC.

The lower-most part of the Pierce Gulch formation consists primarily of olive to light gray-brown, interbedded very fine silty sand, sandy silt, and silty clay, with oxidized seams and laminar bedding. The thickness of this unit ranges from zero up to 300 feet. This unit represents sediments deposited in a transitional fluvial to lacustrine environment. For the purposes of this report, this unit is included within the Pierce Gulch Formation.

The vast majority of the proposed cell will be constructed within the Pierce Gulch Formation. The remainder of the cell will be constructed within Quaternary-age stream alluvium, as previously described. The relationship of the cell bottom to the Pierce Gulch Formation is illustrated on the geologic cross sections in Section 4.2.2.3.2.

4.2.2.1.3 Terteling Springs Formation. The Terteling Springs Formation was formerly called a member of the lower Idaho Group. The formation is Tertiary in age and is generally characterized as tan to gray in color, poorly to well indurated clays, silts, and fine to coarse sands (Beck, 1989). The claystone and siltstone members contain thin sand layers. The Terteling Springs Formation conformably underlies the Pierce Gulch Formation. For the

purposes of this report, the lacustrine blue clay is inferred to represent the upper boundary of the Terteling Springs Formation.

Throughout the NRC study area, the observed textures of the Terteling Springs Formation were consistent with previous descriptions of the lower Idaho Group. These studies describe the lower Idaho Group as sands, claystones, and siltstones resulting from deltaic and lacustrine depositional environments, and are more fine grained than the upper Idaho Group (Castelin and Petrich, 2005).

The Terteling Springs formation is often locally referred to as "blue clay." Soil boring information at the NRC indicates that the blue clay member of the Terteling Springs Formation consists of bluish-grey silts and clays. The blue clay was observed at-depth throughout the entire NRC study area. The thickness and depth to the bottom of the formation is not known, but well logs indicate the blue clay member of the formation is more than 600 feet thick in the vicinity of the proposed landfill. Further west, within the Maryglenn subdivision (Township 5N, Range 1E, Section 35), well logs indicate the blue clay member of the formation is more than 1,000 feet thick (see Appendix E, Driller's Reports for wells 5N1E35_15 and 5N1E35_03). The blue clay appears to be a massive fine grained lacustrine deposit that has been chemically reduced to a bluish-grey color. At some locations, the lacustrine deposits were directly overlain by a thin veneer of brown clay. This brown clay also appears to be lacustrine in origin—although it does not appear to have been subject to the same reducing environment as the blue clays.

The presence of these lacustrine beds is significant to the construction of the NRC, because they form an underlying low-permeability structural barrier that controls the horizontal and vertical flow of local groundwater in the vicinity of the NRC. The relationship of the cell bottom to the fine grained lacustrine deposits of the Terteling Springs Formation is illustrated on the geologic cross sections presented in Section 4.2.2.3.2.

4.2.2.1.4 Occurrence of Groundwater. The first fully saturated sediments were found within the fine-to-coarse-grained sand beds stratigraphically above the thick and continuous lacustrine blue clay of the Terteling Springs Formation. At the time of drilling, the depth to groundwater ranged from approximately 1 foot along the Goose Creek channel (NRB-16) to up to 180 feet along the ridge tops (NRB-1) that surround the NRC. Within the NRC study area, the saturated thickness of the uppermost aquifer ranged from approximately 4 feet (NRB-31) to 52 feet (NRB-4). The blue clay is an important hydrogeologic horizon that underlies the entire landfill and surrounding area. It acts as a definitive confining unit with a very low permeability upon which the groundwater lies. Depth to this layer varies somewhat as a result of preferential erosion of overlying sediments and the variability of its surface angle.

The top of the uppermost aquifer is defined by the water table or top of saturation (potentiometric surface). No significant clay layers were encountered between the ground surface and the top of the groundwater to act as an upper confining bed; therefore, the groundwater appears to be unconfined. No perched water was encountered while drilling. Depth to water and the thickness of the uppermost aquifer is largely dependent on the attitude and topographic relief of the underlying blue clay.

Figure 4-3 shows the saturated thickness of the uppermost water-bearing unit beneath the NRC and the HHLF Cell based on the difference in elevations between the water table (June

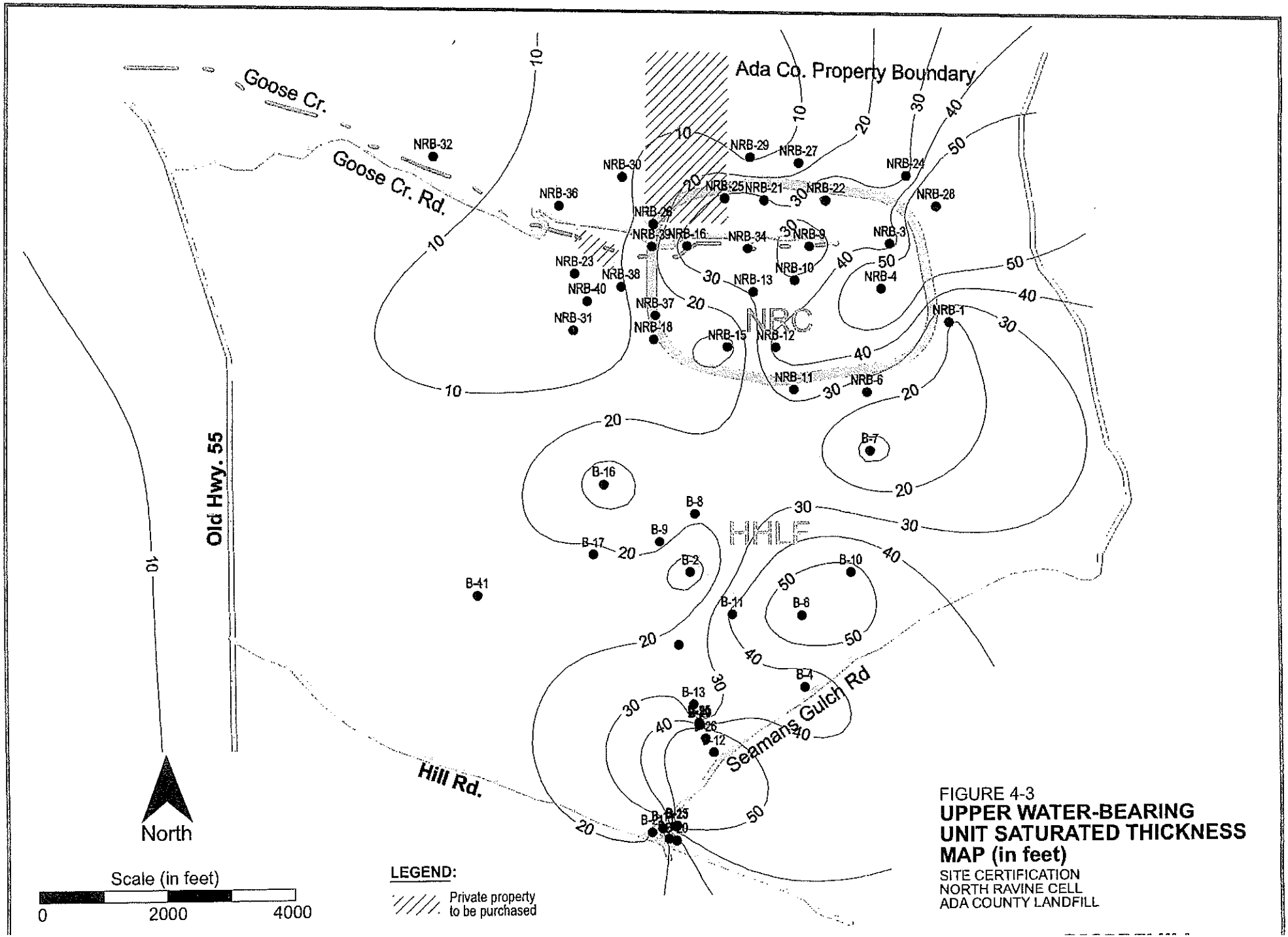

FIGURE 4-3
UPPER WATER-BEARING UNIT SATURATED THICKNESS MAP (in feet)
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

2004 water level measurements) and the underlying blue clay. The saturated thickness of the uppermost water-bearing unit beneath the NRC is greatest along the east side of the NRC (greater than 50 feet in thickness at NRB-4 and NRB-28) and generally thins out to approximately 10 feet in thickness to the west and southwest of the NRC. South of the NRC and the HHLF Cell, the saturated zone does not thin out, but remains comparatively thick (40 to 50 feet thick) along Seamans Gulch Road. This latter zone is coincidental with the zone of VOC contamination downgradient of the HHLF Cell.

4.2.2.2 Geologic Structure

The geologic structure of the site generally consists of west- to southwest-dipping bedding and pre-Holocene faults. The strike and dip of exposed bedding planes of the Pierce Gulch Formation were measured at several locations within the NRC study area. The strike of these beds is north-south. The dip of these beds ranges from 20 to 30 degrees west.

No exposures of the Terteling Springs Formation were observed within the NRC study area. In order to evaluate the structure of the blue clays of the Terteling Springs Formation, elevations of the blue clay were compiled from boreholes drilled at the NRC, HHLF Cell, and from local water supply wells (Table 4-4). These data were used to construct a blue clay structural contour map beneath the NRC and adjoining areas (Figures 4-4a and 4-4b). Appendixes A and E include copies of the drillers' logs for each borehole and water supply wells included in Table 4-4.

TABLE 4-4
Blue Clay Elevations and Coordinates

Well ID	Easting	Northing	Blue Clay Elevation (ft amsl)
NRC Monitoring Wells			
NRB-01	362571.32	744924.58	2992
NRB-02	356880.07	746309.52	2847
NRB-03	361628.83	746149.44	3021
NRB-04	361493.75	745441.37	2993
NRB-06	361273.35	743826.06	2943
NRB-07	356423.55	746398.60	2829
NRB-09	360360.82	746098.86	2998
NRB-10	360129.76	745568.55	2945
NRB-11	360116.93	743864.65	2928
NRB-12	359825.44	744520.37	2931
NRB-13	359471.94	745382.26	2942
NRB-15	359061.94	744522.32	2935
NRB-16	358423.98	746091.29	2899
NRB-17	358215.70	745470.71	2938
NRB-18	357894.74	744634.67	2914
NRB-19	357712.43	746084.92	2879
NRB-20a	357792.91	745595.98	2924
NRB-21	359461.28	746807.80	2977
NRB-22	360618.32	746811.95	3019

TABLE 4-4
Blue Clay Elevations and Coordinates

Well ID	Easting	Northing	Blue Clay Elevation (ft amsl)
NRB-23	356641.03	745646.99	2829
NRB-24	361884.95	747203.00	3065
NRB-25	359018.78	746832.27	2942
NRB-26	357887.96	746426.80	2922
NRB-27	360189.61	747386.27	3039
NRB-28	362361.59	746733.55	3019
NRB-29	359416.69	747465.93	3004
NRB-30	357391.44	747145.14	2907
NRB-31	356620.55	744768.71	2749
NRB-32	354402.12	747436.78	2728
NRB-33	350879.71	746538.16	2639
NRB-34	359382.83	746055.74	2942
NRB-36	356393.30	746698.08	2840
NRB-37	357920.54	745008.24	2922
NRB-38	357378.06	745448.94	2892
NRB-39	357864.00	746076.81	2890
NRB-40	356842.70	745226.12	2786
HHLF Cell Monitoring Wells			
B-2	358,470.71	741,015.98	2753
B-4	360,273.72	739,230.04	2725
B-6A	360,229.38	740,349.09	2721
B-7	361,321.01	742,911.21	2942
B-8	358,546.25	741,939.81	2821
B-9	357,986.03	741,500.24	2704
B-10	361,013.86	741,020.53	2790
B-11	359,132.90	740,364.59	2713
B-13	358,504.66	738,968.08	2669
B-14	357,989.55	737,036.30	2613
B-16	357,105.80	742,383.51	2677
B-17	356,932.57	741,296.39	2629
B-18	357,401.58	738,613.26	2610
B-19	358,087.73	736,877.32	2581
B-20	358,211.51	736,847.63	2580
B-21	357,821.07	736,977.07	2600
B-23	358,211.51	737,069.33	2552
B-24	358,590.20	738,652.80	2626
B-25	358,592.10	738,688.90	2682
B-26	358,688.40	738,436.30	2625
B-29	358,605.20	738,647.40	2625
B-40	355984.00	741664.00	2601
B-41	355,094.53	740,707.01	2563

TABLE 4-4
Blue Clay Elevations and Coordinates

Well ID	Easting	Northing	Blue Clay Elevation (ft amsl)
Residences/Water Supply Wells			
EW-4	358499.60	738978.70	2689
E-1	361892.90	740994.50	2897
E-2	361890.00	740984.00	2895
E-3	358842.00	738219.00	2641
E-4	358497.00	741036.00	2734
Thomas Residence	352233.70	743318.10	2527
Chapin Residence	351532.90	748137.40	2675
Equestrian	354211.60	747305.00	2715
Everett Residence	350180.00	748353.80	2615
Guilfoyle Residence	348764.80	747971.20	2538
Kleimon Residence	348493.40	748705.70	2525
Massie Residence	352775.90	743730.30	2545
Terteling #4	365497.00	740606.00	2929
Ketner Well	363139.00	741098.00	2920
Dry Creek Cemetery	353664.00	739741.00	2503
Duckert Residence	352040.00	749850.00	2743
Tradewinds Bed and Breakfast	352640.00	750650.00	2838
Howell Residence	351823.00	748848.00	2720
Terteling #2	357978.00	746041.00	2906
Sheriff's Shooting Range	360940.00	746135.00	3037

The blue clay contour map shows that the surface of the blue clay slopes toward the southwest at a regional dip of about 3 degrees. Elevations of the surface of the blue clay in the immediate vicinity of the project area range from 3,065 feet in NRB-24 near the northeast portion of the proposed NRC, to 2,750 feet in NRB-31 just southwest of the proposed NRC. A zone of higher dip in the blue clay surface is present just west of the proposed NRC between borings NRB-20A and NRB-31. The dip of the blue clay in this area is approximately 8 degrees to the west-southwest. The origin of the steepened dip of the surface of the blue clay at this location is unknown. Probable causes include localized shoreline formational/erosional processes during the transgression and/or regression of ancient Lake Idaho or the presence of a north trending, pre-Holocene fault offset.

Numerous small-offset high-angle normal faults were observed in exposures in and around the project area. These faults are interpreted to be pre-Holocene faults that formed due to regional tectonic movements or soft-sediment deformation of the deposits. A more detailed discussion of these faults and their significance to construction of the NRC is presented in Section 4.2.3.

4.2.2.3 Site-Specific Stratigraphy

Based on the geologic information obtained from the soils borings, test pits, exploratory trenches, and water wells drilled in the vicinity, a generalized subsurface stratigraphic profile was constructed. The general subsurface profile consists of fine to coarse sands (units SP and SW) overlying interbedded fine sands, silts, and clays (unit ML), which overlie massive brown and blue silty clay (units CL and CH). Alluvium (Qal) is found in the drainages, and interbedded alluvial fans (unit Qaf) have been shed from the P.G. formation. This stratigraphic sequence is described in detail in Section 4.2.2.4.

4.2.2.4 Description of Geologic Cross Sections

Nine geologic cross sections were constructed to graphically show the thickness and lateral distribution of the geologic units encountered in the vicinity of the NRC study area. A plan view of the locations of the cross sections is provided in Figure 4-5 and the cross sections are shown in Figures 4-6 through 4-14. The subsurface stratigraphy was divided into seven general units that are shown on the cross sections. Because of the horizontal scale and vertical exaggeration on the sections, other thin discontinuous lenses of materials encountered at the site may not be shown on the sections. The contacts between the stratigraphic units were based primarily on split-spoon and continuous core sample descriptions, field mapping, and changes observed during drilling.

Descriptions of the seven major stratigraphic units shown on the cross sections are provided in the following text.

Unit SW. This unit consists primarily of medium to coarse-grained, white to yellow arkosic sand to clayey sand with a large percentage of coarse quartz grains. Zones of clayey coarse sand were also observed within this unit. Clayey zones appear to result from weathering of feldspar minerals to clay. This unit is interpreted to be the Pierce Gulch Formation and underlies nearly all of the proposed NRC. This unit consists primarily of fluvial and deltaic sediments deposited in a relatively high-energy, near shore environment. The thickness of this unit varies highly because of surface erosion at the site and ranges from 0 to more than 315 feet.

Unit SP. This unit consists primarily of fine-grained, white to yellow sand to silty sand, with oxidized seams and laminar bedding. This unit underlies the project area generally west of the proposed cell footprint and appears to transition laterally from the coarse-grained sands of Unit SW. This unit appears to be primarily fluvial to lacustrine sediments deposited in pro-delta, offshore environments and is interpreted to be a part of the Pierce Gulch Formation. The thickness of this unit is highly variable due to erosion at the site, and ranges from 0 to more than 300 feet.

Unit ML. This unit consists primarily of olive to light gray-brown, interbedded very fine silty sand, sandy silt, and silty clay, with oxidized seams and laminar bedding. This unit underlies the SW and SP units across much of the site. The thickness of this unit ranges from zero up to 95 feet. This unit represents sediments deposited in a transitional fluvial to lacustrine environment. As discussed in Section 4.2.2.1.2, this material is interpreted to be a part of the Pierce Gulch Formation.

FIGURE 4-4A
WATER LEVEL MAP
AUGUST 2004
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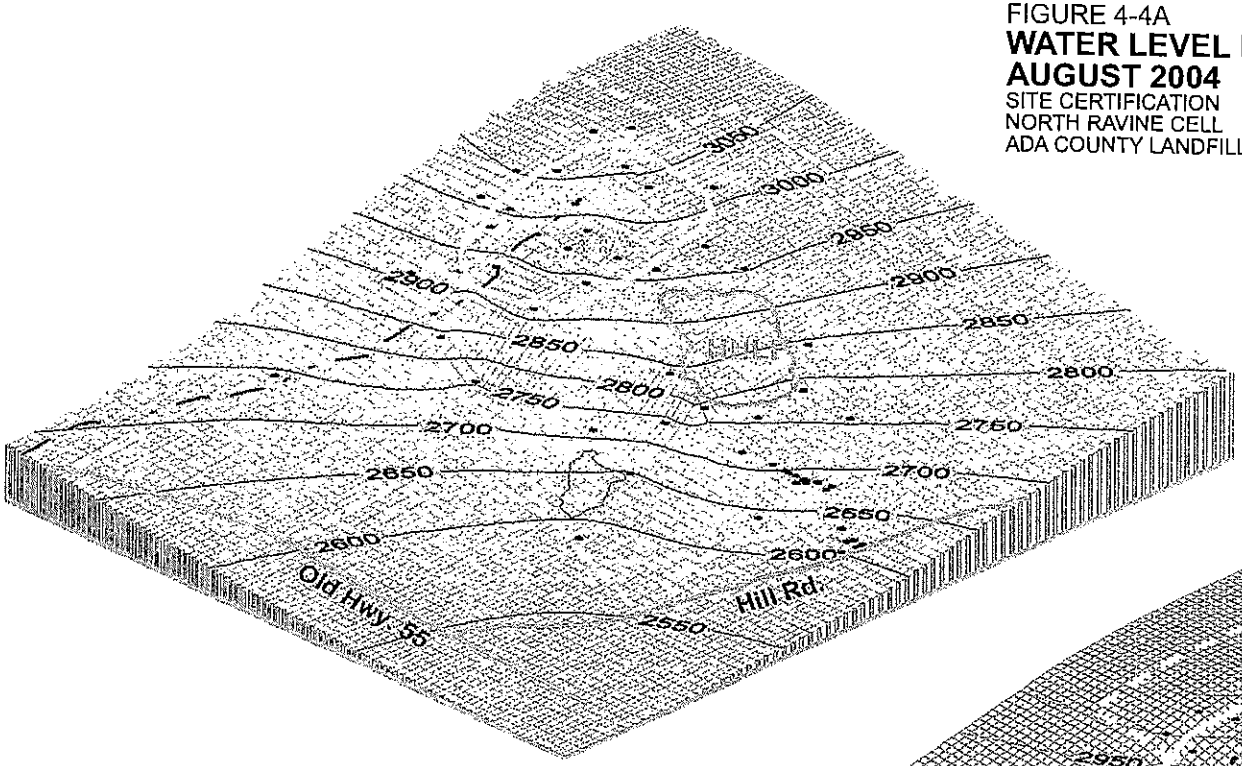

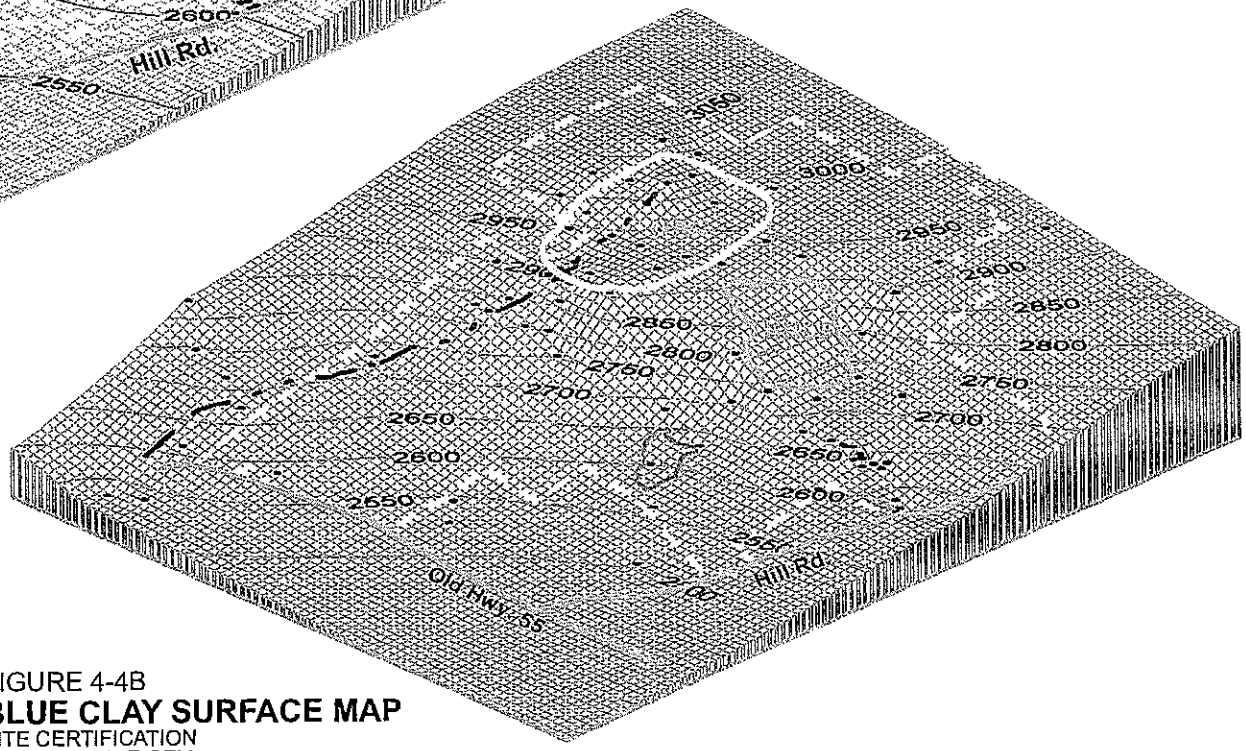

FIGURE 4-4B
BLUE CLAY SURFACE MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

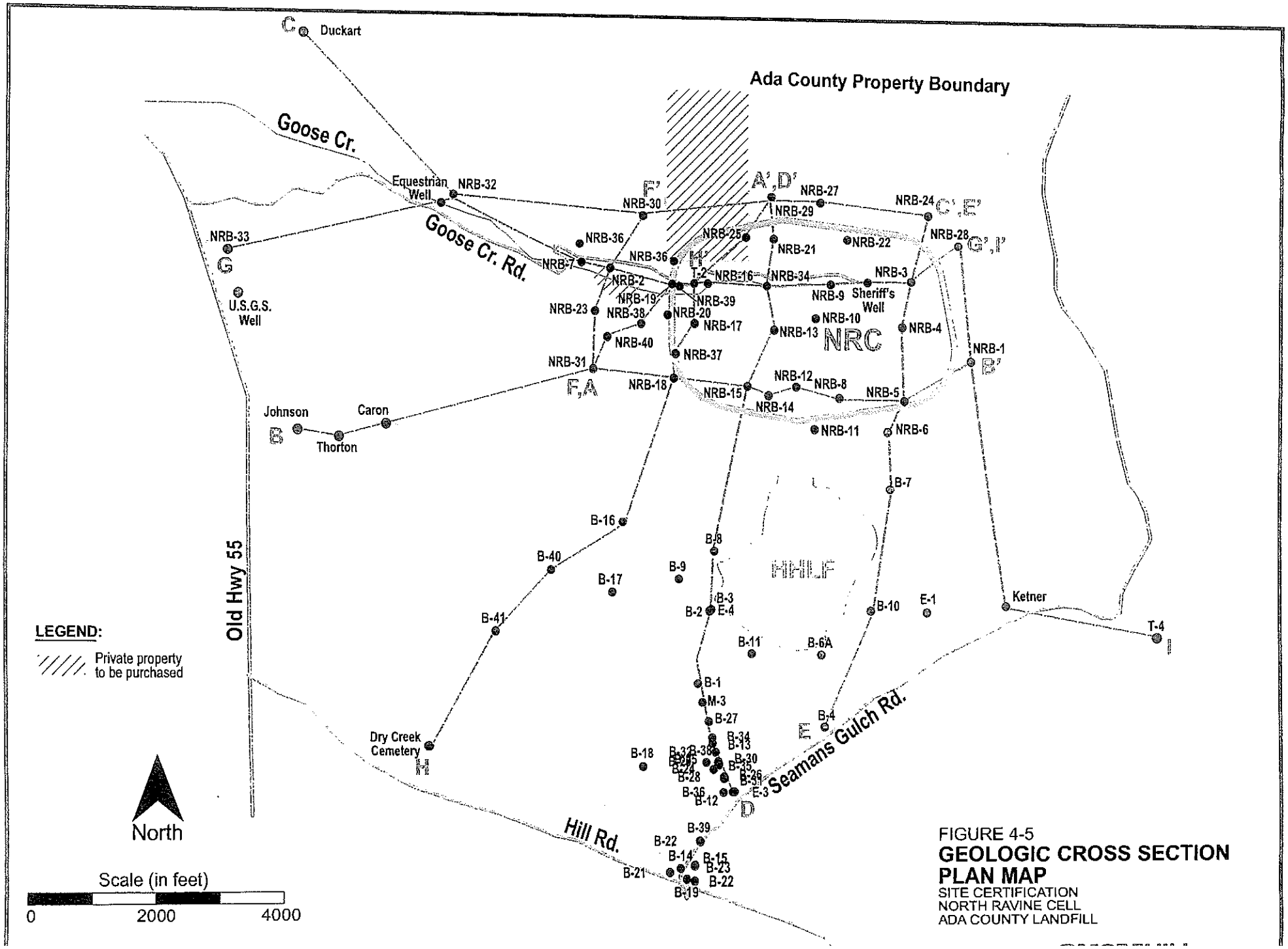

FIGURE 4-5
GEOLOGIC CROSS SECTION PLAN MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

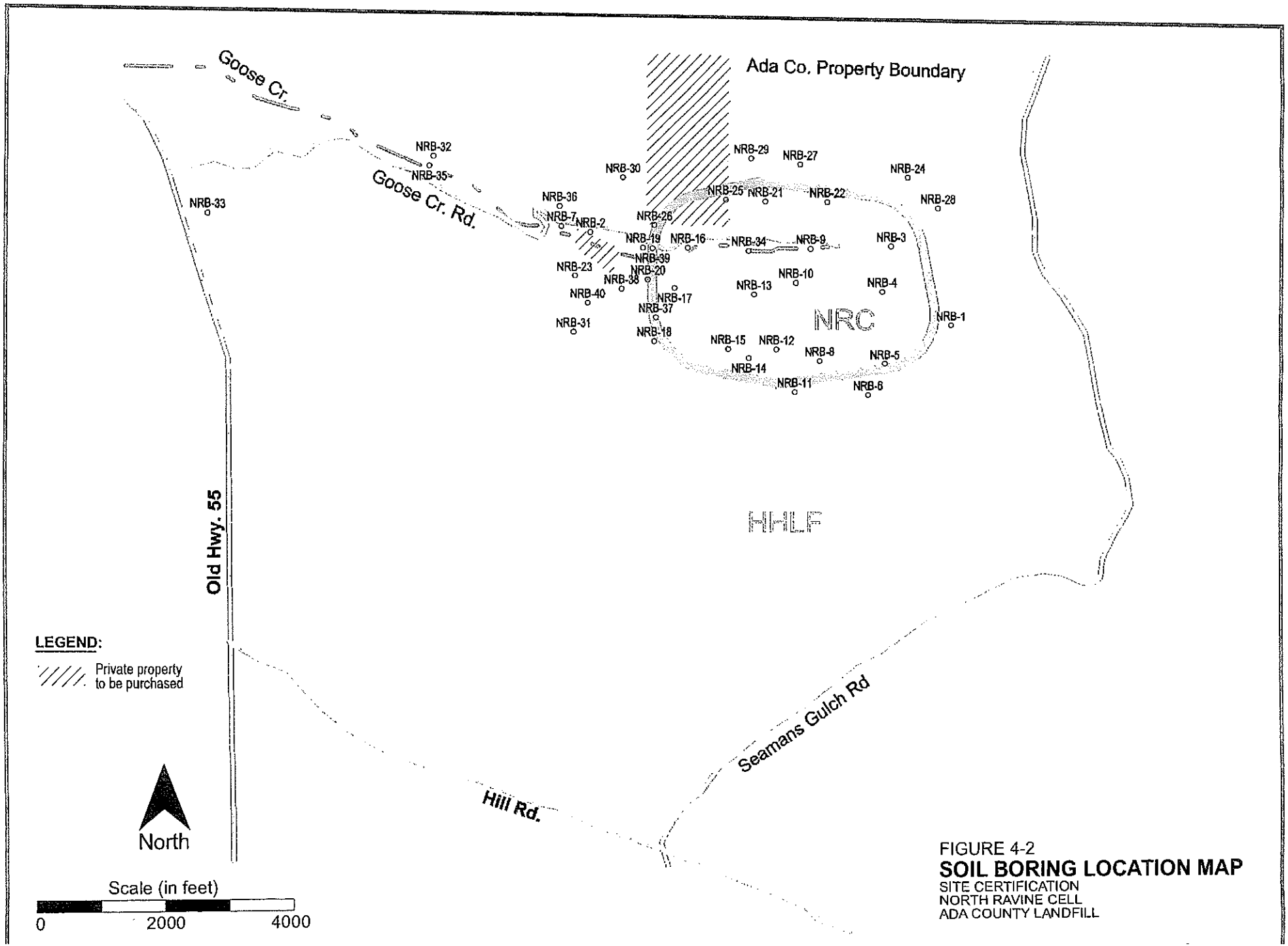

FIGURE 4-2
SOIL BORING LOCATION MAP
SITE CERTIFICATION
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Unit CL. This unit consists primarily of brown to olive brown, low-plasticity clay to silty clay. This unit was found in some of the borings and may be the oxidized equivalent to the blue clay (Unit CH). This unit ranges from zero to 25 feet in thickness and was deposited in a very low-energy lacustrine environment.

Unit CH. This unit consists primarily of dark gray to blue-gray clay to silty clay with sandy seams, and is known locally as the blue clay. This unit is found throughout the entire project study area and is interpreted to be the Terteling Springs Formation. The thickness of the blue clay in the project area is more than 600 feet, based on soil borings and well drillers' logs. The surface of the blue clay is dipping and eroded, and the bottom of the blue clay has not been penetrated within the study area.

Unit Qal. Quaternary-age stream alluvium is primarily comprised of brown to gray, fine to coarse grained, interbedded sandy material derived from surrounding hillsides and redeposited in the valley bottoms. Based on drilling information, the alluvium is up to 30 feet thick in some areas of the Goose Creek drainage.

Unit Qaf. This unit is Quaternary-age alluvial fan material, which consists primarily of medium to coarse sand interbedded with silty fine sand and silt (Othberg, 1992). This material was re-worked and redeposited from the Tertiary sediments (Pierce Gulch Formation). This unit was essentially shed from the foothills onto the lower-elevation river floodplain, and is mapped primarily in the west and southwest portion of the study area in the vicinity of Old Horseshoe Bend road. Based on drillers' logs, this material consists of interbedded, tan- to brown-colored sands, sandy clay, and clay, with occasional interbedded gravels. The gravels likely represent the alluvial fan materials interfingering with alluvial terrace gravels.

4.2.2.5 Details of Cross Sections

Nine geologic cross sections were constructed. Cross Sections *D-D'*, *E-E'*, *F-F'*, *H-H'*, and *J-J'* run north-south. Cross Sections *B-B'*, C-C', and *G-G'* run east-west. Cross Section *A-A'* runs northeast-southwest. Descriptions of the stratigraphy and geology are shown in the following text. The bottom liner elevation of the proposed NRC is shown on each of the following cross sections that cross the NRC footprint.

4.2.2.5.1 Cross Section *A-A'*

Section Location/Trend. Cross Section *A-A'* runs southwest-northeast near the western side of the proposed NRC (Figure 4-6). This section shows subsurface data in the following: NRC borings NRB-31, NRB-40, NRB-38, NRB-20A, NRB-16, NRB-25, and NRB- 29; an abandoned irrigation well at Terteling #2 (T-2); and Test Pit PGC-11.

General Geology. The geology of this section shows fine to coarse sands of the Pierce Gulch Formation overlying interbedded olive-brown fine sands, silts and clays, and also the blue clay of the Terteling Springs Formation. The middle part of the section depicts where Goose Creek has eroded through the formations and the drainage has backfilled with stream alluvium. The underlying blue clay appears to be continuous across the entire section, and is at least 600 feet thick.

Stratigraphy/Structure. This section illustrates the westward transition from the coarse-grained to fine-grained facies of the Pierce Gulch Formation. The surface of the blue clay is

dominated by a southwesterly dip, dipping more steeply to the southwest between borings NRB-38 and NRB-40.

Groundwater. The groundwater flows from northeast to southwest. The groundwater surface appears to be interrupted by a high point in the blue clay in the vicinity of Boring NRB-20A. Borings NRB-40 and NRB-38 appeared to be dry at the time of drilling and therefore no monitoring wells were installed in these borings. NRB-31 exhibited low saturated thickness.

4.2.2.5.2 Cross Section *B-B'*

Section Location/Trend. Cross Section *B-B'* runs east-west along the southern side of the proposed NRC (Figure 4-7). This section shows subsurface data in the following: NRC borings NRB-31, NRB-18, NRB-15, NRB-14, NRB-12, NRB-8, NRB-6, NRB-5, and NRB-1; Test Trench 3A; and three representative borings from Sage Acres Subdivision (Johnson, Thorton, and Caron Wells). NRC borings NRB-5, NRB-8, and NRB-14 were drilled in ridge tops to evaluate material properties for daily cover. Because these borings did not penetrate the water table, no monitoring wells were installed.

General Geology. The geology of this section shows a thick sequence of fine-grained to coarse-grained deltaic sands of the Pierce Gulch Formation overlying interbedded olive-brown fine sands, silts and clays, and also the blue clay of the Terteling Springs Formation. The western part of the section, where the domestic wells are drilled, shows the alluvial fan materials shed from the foothills.

Stratigraphy/Structure. This section illustrates the westward transition from the coarse-grained to fine-grained facies of the Pierce Gulch Formation between NRB-18 and NRB-31. This section also shows the inferred westward transition from the fine-grained Pierce Gulch sands to the interbedded alluvial fan materials, between NRB-31 and the Caron well. The domestic wells may also include Pierce Gulch formation sediments. Brown clay, which may be an oxidized portion of the blue clay, was encountered in boring NRB-12. The underlying blue clay appears to be continuous across the entire section. Because the section line is not along the true dip of the blue clay, the blue clay appears relatively flat lying across much of the section. The blue clay dips more steeply to the west-southwest between borings NRB-38 and NRB-40.

Groundwater. The groundwater in the uppermost water-bearing unit flows from northeast to southwest through the area. Between NRB-1 and NRB-18, the groundwater gradient appears very flat in this section because the orientation of the section is not-parallel to the maximum gradient. The groundwater gradient is considerably steeper between borings NRB-18 and NRB-31, which corresponds with the steeper dip of the blue clay surface. Water level information is not available for shallow groundwater west of NRB-31. The domestic wells are all completed within the blue clay, and thus groundwater elevations shown represent potentiometric heads within the screened portion of the blue clay.

4.2.2.5.3 Cross Section *C-C'*

Section Location/Trend. Cross Section C-C' runs east-west to the north of the proposed NRC (Figure 4-8). This section starts near the eastern property line for the Ada County Landfill, continues along the northern margin of the proposed NRC and crosses the foothills into the Dry Creek drainage at the western end of the section. This section shows subsurface data in

the following: NRC borings NRB-32, NRB-30, NRB-29, NRB-27, and NRB-24; and a representative downgradient domestic well (Duckart) in the Maryglenn Subdivision.

General Geology. The geology of this section shows a thick sequence of fine-grained to coarse-grained deltaic sands of the Pierce Gulch Formation overlying interbedded olive-brown fine sands, silts and clays, and also the blue clay of the Terteling Springs Formation. Alluvium is shown at NRB-32 where the section crosses through the Goose Creek drainage. The underlying blue clay appears to be continuous across the entire section, and is at least 600 feet thick beneath the NRC. Note that well logs for other nearby domestic wells within the Maryglenn Subdivision (Township 5N, Range 1E, Section 35) indicate the blue clay member of the Terteling Springs formation is more than 1,000 feet thick west of the Ada County Landfill Property Boundary (see Appendix E, Driller's Reports for wells 5N1E35_15 and 5N1E35_03).

Stratigraphy/Structure. This section illustrates the westward transition from the coarse-grained to fine-grained facies of the Pierce Gulch Formation between NRB-29 and NRB-30. Brown clay, which may be an oxidized portion of the blue clay, was encountered in boring NRB-29, NRB-27, and NRB-24. The blue clay appears to be continuous across the entire section. The blue clay appears relatively flat lying across eastern part of this section (NRB-27 to NRB-24); however, the section line is not along the dip of the blue clay. The blue clay dips more steeply to the west-southwest between borings NRB-27, NRB-30, and NRB-32. The blue clay appears flat between NRB-32 and the Duckart well because the section bends to the northwest, and generally follows the strike of the blue clay between these wells.

Groundwater. The groundwater flows from northeast to southwest through the area. The groundwater gradient appears to dip to the west, with a marked steepening west of NRB-27, which corresponds with the steeper dip of the blue clay surface. The groundwater gradient flattens to the west of NRB-32, which corresponds with the flattening dip of the blue clay surface. The Duckart well is completed within the blue clay and therefore the groundwater level in this well is the potentiometric head within the screened portion of the blue clay.

4.2.2.5.4 Cross Section *D-D'*

Section Location/Trend. Cross Section *D-D'* runs in a north-south direction through the middle of the proposed NRC (Figure 4-9), starting just north of the proposed NRC, going west of the existing HHLF Cell, and ending near Seamans Gulch Road. This section shows subsurface data in the following borings: NRB-29, NRB-21, NRB-34, NRB-13, and NRB-15; HHLF Cell borings B-8, B-2, B-1, B-13, and B-12; methane monitoring well M-3; and former (abandoned) HHLF production wells E-3, and E-4.

General Geology. The geology of this section shows a thick sequence of fine-grained to coarse-grained deltaic sands of the Pierce Gulch Formation overlying interbedded olive-brown fine sands, silts and clays, which overlies the blue clay of the Terteling Springs Formation.

Stratigraphy/Structure. This section illustrates the southward trend of the facies of the Pierce Gulch Formation between NRB-29 and E-3, where the coarse, sandy, near-shore deltaic deposits transition to finer-grained, off-shore lacustrine deposits. Brown clay was encountered only in NRB-13. One boring and subsequent well (E-4) was advanced nearly 300 feet into the blue clay; this well encountered one sand seam. Another boring and

subsequent well (E-3) was advanced nearly one hundred feet into the blue clay; this well encountered two small sand seams. The blue clay appears to be continuous across the entire section. Data are insufficient to evaluate the continuity of the deep sandy lenses encountered in E-3 and E-4 and whether they are connected. The southward dip of the blue clay is evident in this section.

Groundwater. The groundwater flows from northeast to southwest through the area. The groundwater gradient appears to dip to the south, with a marked steeping in the gradient to the south of NRB-15, which corresponds with the steeper dip of the blue clay surface. Generally the surface of the groundwater mimics the surface of the blue clay contact.

4.2.2.5.5 Cross Section *E-E'*

Section Location/Trend. Cross Section *E-E'* runs in a north-south direction along the eastern side of the proposed NRC (Figure 4-10). This section starts near the north Ada County Landfill property boundary, continues along the eastern part of the proposed NRC, continues to the east of the HHLF Cell, and terminates near Seamans Gulch road. This section shows subsurface data in the following: NRC borings NRB-24, NRB-3, NRB-4, NRB-5, and NRB-6; and HHLF Cell borings B-7, B-10, B-4, and HHLF production well E-1.

General Geology. The geology of this section shows coarse deltaic sands of the Pierce Gulch Formation overlying and transitionally interbedded with olive-brown fine sands, silts and clays. These deposits overlie the blue clay of the Terteling Springs Formation. Fine sands and silts are generally absent south of NRB-4.

Stratigraphy/Structure. Much of this section is dominated by a thick sequence of deltaic sands directly overlying the blue clay; the transitional fine sand and silt deposits are generally absent, except in the northern portion of the section. Borings NRB-3 and NRB-24 encountered relatively thick fine-grained, interbedded lacustrine deposits. The blue clay appears to be continuous across the entire section. The surface of the blue clay dips southward along this section.

Groundwater. The groundwater flows from northeast to southwest through the area. The groundwater gradient is to the south on this section. The groundwater saturated thickness in the immediate vicinity of the NRC is greatest in boring NRB-4 (54 feet). Borings NRB-4 and NRB-6 could not be advanced to the blue clay because of the loose, saturated sands.

4.2.2.5.6 Cross Section *F-F'*

Section Location/Trend. Cross Section *F-F'* runs in a north-south direction along the west side of the proposed NRC (Figure 4-11). This section shows subsurface data in the following: NRC borings NRB-30, NRB-2, NRB-23, and NRB-31. No well was installed in the Qal unit near NRB-2.

General Geology. The geology of this section shows a thick sequence of fine-grained deltaic to lacustrine sands of the Pierce Gulch Formation overlying the blue clay of the Terteling Springs Formation.

Stratigraphy/Structure. This section illustrates the fine-grained sandy facies of the Pierce Gulch Formation to the west of the NRC footprint. The blue clay appears to be continuous across the entire section. The blue clay dips more steeply to the south between NRB-23 and NRB-31.

Groundwater. The groundwater flows from northeast to southwest through the area. The groundwater gradient appears to mimic the blue clay contact. Saturated thickness is very small in this area.

4.2.2.5.7 Cross Section *G-G'*

Section Location/Trend. Cross Section *G-G'* runs west to east near the center of the NRC (Figure 4-12). This section starts near the western Ada County Landfill property line and runs along the valley of Goose Creek, through the proposed NRC, and terminates near the eastern Ada County Landfill property line. This section shows subsurface data in the following: NRC borings NRB-33, NRB-32, NRB-7, NRB-2, NRB-19, NRB-39, NRB-16, NRB-34, NRB-9, NRB-3, NRB-28; and the deep wells at the Skateboard Park (USGS Well), Equestrian Park, Terteling #2, and Ada County Sheriff's Shooting Range.

General Geology. The geology of this section shows fine-grained and coarse-grained deltaic sands of the Pierce Gulch Formation, sandy alluvium of Goose Creek, and interbedded olive-brown fine sands, silts and clays, all overlying the blue clay of the Terteling Springs Formation.

Stratigraphy/Structure. This section illustrates the continuity of the blue clay throughout the area and its relatively consistent western-trending surface gradient. The eastern portion of the section shows coarse-grained deltaic sands overlying interbedded fine-grained materials overlying the blue clay. The center of the section shows how Goose Creek has eroded through the overlying materials to the blue clay, and the valley has become backfilled with alluvial sediments. The western portion of the section shows fine sands overlying silts and brown clays which in turn overlie the blue clay. NRB-33 and the USGS well are located at the transition between the Pierce Gulch formation and alluvial fan deposits, and probably penetrate both units. The blue clay surface slopes in a southwesterly direction along this section with a relatively constant slope angle. The thickness of the blue clay shown in this section is more than 600 feet thick at Terteling #2. Note: The Equestrian Well and the USGS well both penetrated sandy lenses within the blue clay. However, the data are insufficient to evaluate the continuity of these lenses and whether they are connected.

Groundwater. The groundwater flows from east to west across the section. The groundwater gradient varies somewhat on this section. The area along the bottom of the Goose Creek drainage (wells NRB-16 and NRB-34) exhibits very shallow groundwater conditions; these correspond with mapped wetland areas. The saturated thickness varies across the site with the greatest saturated thickness occurring in upgradient well NRB-28. Monitoring well NRB-33 is dry. It appears that the groundwater table is lower in elevation than this well.

4.2.2.5.8 Cross Section *H-H'*

Section Location/Trend. Cross Section *H-H'* runs north-south/southwest along the western side of the proposed NRC to the Dry Creek Cemetery (Figure 4-13). This section shows subsurface data in the following: NRC borings NRB-17, NRB-37, NRB-18; HHLF cell borings B-16, B-40, and B-41; and the deep water supply wells at Terteling #2 and the Dry Creek Cemetery.

General Geology. The geology of this section shows fine-grained and coarse-grained deltaic sands of the Pierce Gulch Formation, sandy alluvium of Goose Creek, and interbedded olive-brown fine sands, silts and clays, all overlying the blue clay of the Terteling Springs

Formation. The southern part of the section shows inferred alluvial fan deposits in the vicinity of Dry Creek Cemetery.

Stratigraphy/Structure. This section illustrates the southward transition from deltaic deposits to lacustrine deposits. The northern portion of this section is dominated by a thick sequence of coarse-grained deltaic sands directly overlying the blue clay. The transitional fine sand and silt deposits are absent in Borings NRB-18, NRB-37, and NRB-17. In contrast, HHLF Cell borings B-16, B-40, and B-41 encountered relatively thick fine-grained interbedded deposits. The blue clay appears to be continuous across the entire section and is at least 600 feet thick in the vicinity of Terteling #2. The surface of the blue clay dips to the south, with a steeper dip between NRB-18 and B-16. The Dry Creek well is completed near the transition between the foothills and the Boise River floodplain. Sediments in the upper 100 feet of this well are interpreted to be alluvial fan materials with interbedded fluvial gravels.

Groundwater. The groundwater flows from northeast to southwest through the area. The groundwater gradient appears to dip to the south, with a marked steeping in the gradient to the west south of NRB-18, which corresponds with the steeper dip of the blue clay surface. Generally the surface of the groundwater mimics the blue clay contact. The Dry Creek Cemetery well is completed above and within the blue clay.

4.2.2.5.9 Cross Section *I-I'*

Section Location/Trend. Cross Section *I-I'* runs north-south along the eastern side of the proposed NRC (Figure 4-14). This section shows subsurface data in the following: NRC borings NRB-28 and NRB-1; and deep wells for Ketner and Terteling #4.

General Geology. The geology of this section shows fine-grained and coarse-grained deltaic sands of the Pierce Gulch Formation and interbedded olive-brown fine sands, silts and clays, all overlying the blue clay of the Terteling Springs Formation.

Stratigraphy/Structure. The northern portion of this section is dominated by a thick sequence of coarse sands overlying the transitional fine sand and silt deposits overlying blue clay. In contrast, the Ketner and Terteling #4 borings encountered relatively thick fine sand deposits and not the transitional fine sand and silt bedding. The blue clay appears to be continuous across the entire section and dips to the south.

Groundwater. The groundwater flows from northeast to southwest through the area. The groundwater gradient is to the south on this section. The groundwater saturated thickness is greatest in boring NRB-28—more so than in other parts of the study area.

4.2.3 North Ravine Cell Fault Investigation

The original drilling program at NRC boreholes NRB-1 through NRB-35 suggested that a fault was possible in the area immediately west of the footprint of the NRC. The presence of a fault was suspected in this area based on the following:

- A lateral change in Pierce Gulch lithology (coarse sand to fine sand) was observed during initial drilling and geologic mapping of the area
- A north-trending, west-sloping linear feature in the blue clay was observed to underlie the lithology change in the Pierce Gulch

Consequently, additional drilling and exploratory trenching were conducted in this area to determine if a fault offset is present in the blue clay and overlying deposits. If the fault offset is present, data from the drilling and trenching were used to determine whether the fault offsets Holocene deposits. The results of the additional drilling and exploratory trenching programs are discussed below.

4.2.3.1 Additional Drilling Results

Based on the blue clay contours generated using data from existing borings, a linear north-trending structural feature appeared to be present in the surface of the blue clay just west of the NRC footprint. This linear feature was initially identified by NRC borings NRB-18, NRB-31, NRB-20A and appeared to extend northward from the vicinity of the HHLF Cell groundwater remediation area. To more clearly define the blue clay structure and this linear feature, five additional boreholes (NRB-23, NRB-37, NRB-38, NRB-39, and NRB-40) were advanced west of the NRC footprint in July 2004 as shown in Figure 4-2. Borings NRB-38 and NRB-40 were drilled between NRB-31 and NRB-20A to better define the slope of the blue clay; NRB-36 was drilled on the north side of Goose Creek to determine if the linear feature extended to the north side of Goose Creek; NRB-23 was drilled south of Goose Creek between NRB-40 and NRB-36; and NRB-38 was drilled between NRB-18 and NRB-20A to determine if the high point in the blue clay in the vicinity of NRB-20A extends southward.

The additional data points indicated that the north-trending linear feature does not continue north of Goose Creek, and appears to fade out between NRB-23 and NRB-40. The blue clay surface contours in this vicinity are irregular, but show a general west to southwest slope. The irregularity of the blue clay may be the result of faulting and folding during past tectonic activity, erosion of the blue clay surface prior to deposition of younger sediments on the blue clay surface, or downwarping resulting from sediment loading.

In conclusion, the additional drilling did not positively identify a fault trace. The linear feature under investigation was determined to fade out or bend to the northwest on the south side of Goose Creek. The more closely-spaced borings enabled revision of the blue clay contours and show that the surface of the blue clay is irregular but in general west- to southwest-dipping.

4.2.3.2 Exploratory Trenching

Exploratory trenches were excavated to evaluate the nature of the lateral change in Pierce Gulch lithology (the contact between the coarse sands (Unit SW) and fine sands (Unit SP)) west of the NRC footprint. Specifically, the purpose of the trenches was to determine if the contact between these units is depositional or a fault contact, and if it is a fault contact, determine if there is evidence of Holocene activity.

The location of the trenches were based on available soil boring and field reconnaissance data. Borings NRB-8, NRB-26, and NRB-37 showed predominantly coarse deltaic sands (Unit SW) overlying blue clay. West of these, borings NRB-28, NRB-30, and NRB-31 showed fine sands (Unit SP) overlying the blue clay. Field reconnaissance mapping also revealed a similar change in the surface stratigraphy of the area. Exposures of the contact were poor, but by digging for contacts, examining exposures along road cuts, and inspecting "cuttings" from badger burrows, field personnel determined that the contact was likely to be located along the ridge immediately west of borings NRB-18 and NRB-37.

Five trenches and four test pits were excavated in the ridge and adjacent slopes to observe stratigraphic relationships in the near-surface sediments. Figure 4-15 shows the locations of the trenches. Figures 4-16 through 4-20 show detailed maps of the trench walls that exposed conclusive stratigraphic relationships. Logs of the test trenches and test pits that revealed no conclusive stratigraphic relationships are included in Appendix G. The following is a summary of the trench findings.

Trench 1A and 1B. Trench 1A was excavated from west to east, and exposed white, west-dipping, very fine sandy silt (Unit SP) overlain by brown, cross-bedded fluvial sands and gravels. The coarse sand unit (Unit SW) was not encountered in this trench. Contacts between the exposed units were depositional. No fault contacts were observed. A detailed map of Trench 1A was not prepared because the contact between SP and SW was not exposed. A log of the north wall is included in Appendix G.

Trench 1B exposed west-dipping coarse, deltaic sand beds (Unit SW) at its eastern end and west-dipping interbedded, laminated sands and silts (Unit SP) at its western end. The surficial soil horizon followed the present ground surface, which truncated the dipping beds. Throughout the middle of the trench was a large alluvial channel fill deposit of massive clayey sand with occasional rounded gravel. Unfortunately, this channel-fill deposit obscured the contact between Units SW and SP and the actual contact could not be observed. However, no fault offset was seen within the channel-fill deposit and no surface offset was present that would indicate the presence of a Holocene fault. Figure 4-16 shows a detailed map of the western end of the north wall of Trench 1B.

Trench 2. Trench 2 was excavated in the western side of the ridge. This trench exposed west-dipping fine silty sands to sandy silts (Unit SP) overlying medium- to coarse-grained, cross-bedded, fluvial sands (Unit SW). The contact between the units was depositional. The surficial soil horizon followed the present ground surface. No fault contacts or offsets were observed. Figure 4-17 shows a detailed map of the north wall of Trench 2.

Trench 3A. Trench 3A exposed west-dipping beds that included cross-bedded, medium- to coarse-grained sands (Unit SW), laminated silty sands to sandy silts (Unit SP), and massive sands and clayey sands deposited in alluvial channel fills. The surficial topsoil followed the present ground surface and truncated the dipping beds. The contact between the coarse sands and fine sand was depositional. No fault contacts were observed. Figure 4-18 shows a detailed map of the south wall of Trench 3A.

Test Pits 3B, 3C, and 3D. Test pits 3B, 3C, and 3D were excavated in a line to the west of Trench 3A in an attempt to reveal stratigraphic units that might show the contact between the coarse sands and fine sands. These pits were excavated between 9 and 14 feet in depth and exposed massive brown silty sands to clayey sands, with no easily recognizable stratigraphy. In contrast with other trenches which showed west-dipping white to yellow sands in depositional contact, these pits did not show conclusive stratigraphic relationships. No fault contacts or surface offsets were observed in these trenches. These pits were not mapped in detail because of a lack of recognizable stratigraphy; logs of these trenches are included in Appendix G.

Trench 4A and 4B. Trench 4A was excavated in the east-facing slope west of the ridge where the other trenches were excavated to evaluate whether the sediments farther to the west consisted of fine-grained sands (Unit SP), or if the sediments gradually transitioned westward from Unit SP and SW. In addition, because this trench was excavated along an

east-facing slope underlain by west-dipping beds, and therefore a thicker stratigraphic section could be exposed without digging an excessively deep trench.

Trench 4A exposed west-dipping, interbedded, laminar white sands and silts with orange oxidized seams. In addition, two high-angle, down-to-the-west dip-slip faults were observed offsetting the beds. Fault 1 (see trench log) has several splays and offsets a silt bed. The offset on this fault is approximately 2 feet. This fault trends approximately 11 degrees north and dips 85 degrees west. Fault 2 is north-trending, and offsets thick sandy beds. No marker beds were present to measure the fault offset. Both faults become unrecognizable near the surface and do not offset the surface or the surficial soil horizon. Figure 4-19 shows a detailed map of the north wall of Trench 4A.

In order to examine Fault 1 in detail and closely evaluate its near-surface characteristics, a second shallow trench (Trench 4B) was excavated along the strike of the fault immediately north of Trench 4A. The general fault trace revealed in this trench was steep and had several splays, which is common in faults. The trace of the fault in this trench offsets sandy beds and upon detailed examination, the fault trace does not offset surface soils or the ground surface. The sand in the fault zone is sheared and very loose, therefore rootlets have grown in the fault zone and animals have burrowed in between the fault traces. The fault planes themselves were slightly cemented, most likely with calcium carbonate. This cementation occurs as a result of fluid movement along the fault traces. Figure 4-20 shows a detailed map of the south wall of Trench 4B.

The faults observed in trenches 4A and 4B are inferred to be pre-Holocene based on the following geologic evidence:

1. Lack of offset of surficial soils—a Holocene fault would offset surface soils formed on the slope.
2. Lack of a surface scarp—when an active fault ruptures the ground surface, a distinctive linear scarp is formed. Studies performed on age-dating scarps indicated that it can require tens of thousands of years to degrade and erode a scarp (Nash, 1980).
3. The existing ridge-and-ravine landscape evolved after the last movement of this fault, and the present topography has been stable long enough for surface soils to form on the present landscape.

These faults likely formed during pre-Holocene tectonic movements or as a result of soft-sediment deformation and stress relief of the lacustrine and fluvial deposits. Numerous other small fault offsets have been observed and mapped in the Boise Foothills and around the existing HHLF Cell, and likely many more exist that have never been observed. None of these faults are considered active or have been mapped as active (see *Holocene Faulting* discussion in Section 5.4).

4.2.4 Surface Evapotranspiration (ET) Soils Analytical Tests

Three composite samples were created by blending individual topsoil samples collected from 10 test pits at the NRC site (Figure 3-3). These composite samples, representing the range of topsoil types observed across the NRC site, were tested in the laboratory for compaction characteristics, gradation analysis, Atterberg Limits, moisture characteristics, bulk density, porosity, water retention and unsaturated flow parameters, fertility, and hydraulic conductivity. The results of these tests are presented below.

4.2.4.1 Compaction Characteristics

The laboratory performed compaction testing on the three composite topsoil samples. The testing was performed in accordance with ASTM D1557, *Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft³(2,700 kN-m/m³))*, sometimes called the modified Proctor test, to determine the relationship between gravimetric moisture content and dry bulk density of the soils using a specified compaction effort. Copies of the laboratory reports for these tests are presented in Appendix F. Table 4-5 presents the maximum dry density and the optimum moisture content for each sample.

TABLE 4-5
Moisture-Density Relationships

Sample ID	Description	Maximum Dry Density* (pcf)	Optimum Moisture Content (%)
Topsoil Composite 1	Sandy clay (SC)	129.0	8.0
Topsoil Composite 2	Sandy lean clay (CL)	125.0	10.5
Topsoil Composite 3	Sandy lean clay (CL)	117.5	12.5

*ASTM D1557 in pounds per cubic foot

4.2.4.2 Gradation Analysis

The testing laboratory performed gradation analyses on the three composite samples. The analytical method used followed ASTM D422, *Standard Test Method for Particle-Size Analysis of Soils*. Copies of the laboratory reports for these tests are presented in Appendix F. Table 4-6 summarizes the percent fines and percent fines plus fine sand of the composite samples.

TABLE 4-6
Percent Passing 200 Mesh Sieve (Fines) and 60 Mesh Sieve (Percent Fines plus Fine Sand)

Sample ID	Description	Percent Passing No. 60 Sieve	Percent Passing No. 200 Sieve
Topsoil Composite 1	Sandy clay (SC)	49.7	41.2
Topsoil Composite 2	Sandy lean clay (CL)	59.9	51.1
Topsoil Composite 3	Sandy lean clay (CL)	61.2	53.3

4.2.4.3 Atterberg Limits

The testing laboratory performed Atterberg Limit analyses on the three composite samples. The analytical method used followed ASTM D4318, *Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. This method analyzes the liquid limit and plastic limit of the samples. Copies of the laboratory reports for these tests are presented in Appendix F. Table 4-7 summarizes the results of these analyses.

TABLE 4-7
Atterberg Limits

Sample ID	Description	Liquid Limit	Plasticity Index	Plastic Limit
Topsoil Composite 1	Sandy clay (SC)	26	17	9
Topsoil Composite 2	Sandy lean clay (CL)	31	18	13
Topsoil Composite 3	Sandy lean clay (CL)	35	15	20

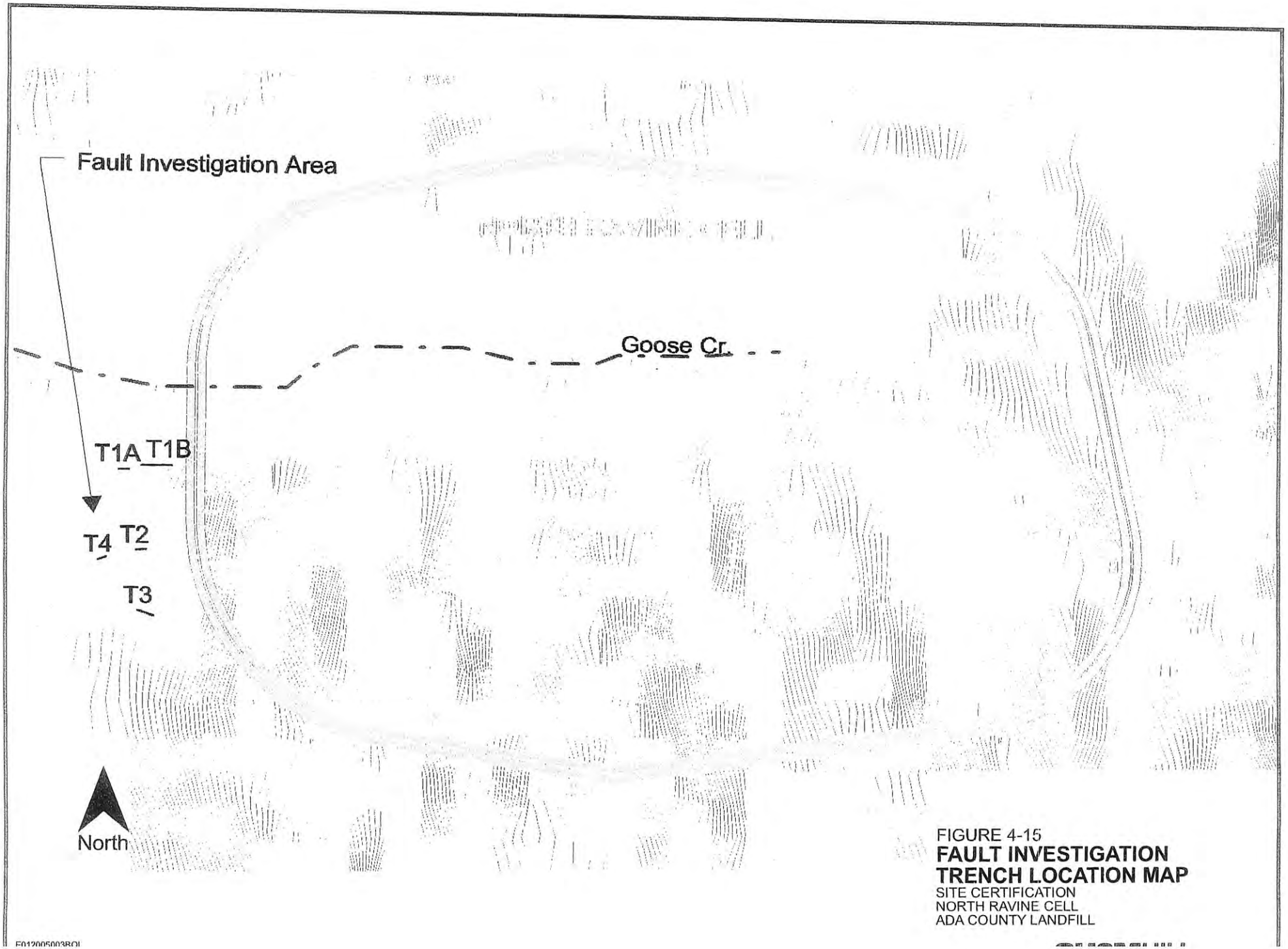
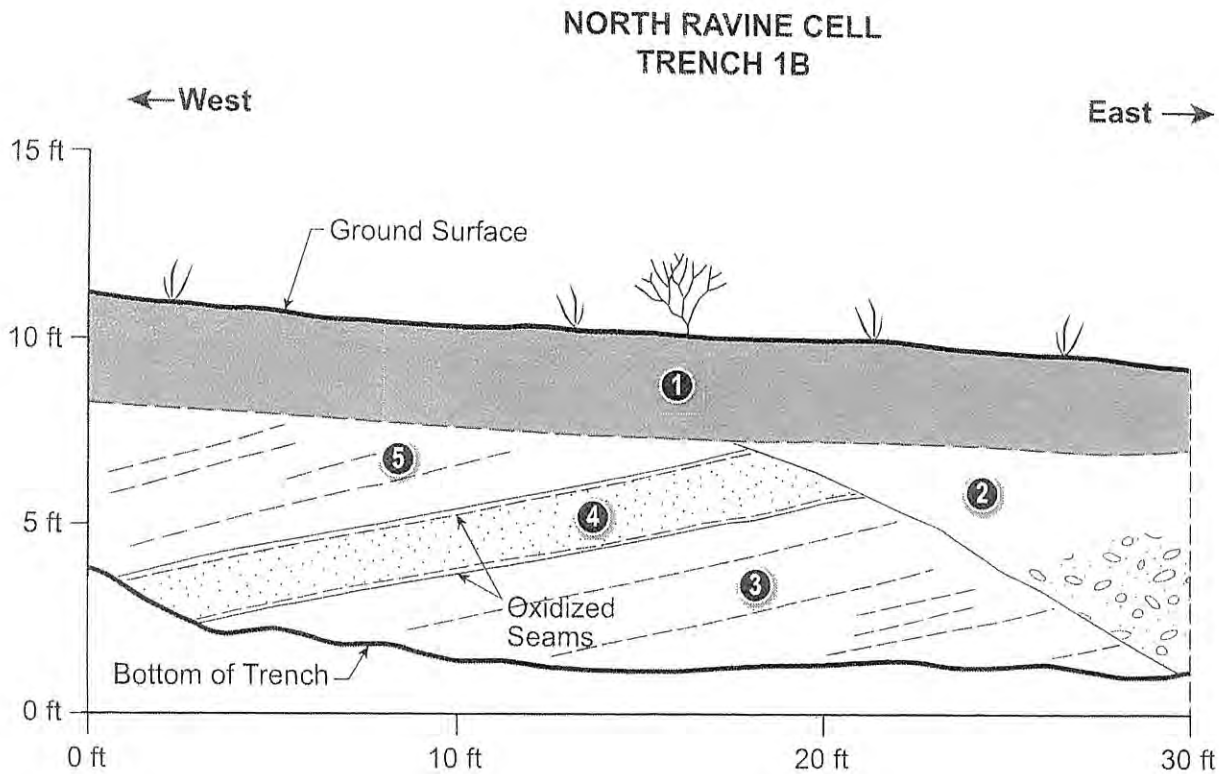


FIGURE 4-15
FAULT INVESTIGATION TRENCH LOCATION MAP
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

F012005003BOI



Stratigraphic Units:

1. Topsoil, brown, silty
2. Sand to clayey sand with occasional round gravel, brown, massive and unstratified, old channel fill deposit
3. Sandy silt to silty sand interbedded, yellow to white, laminar bedding, oxidized seams
4. Sand, yellow-tan, medium to coarse grained, bedded, oxidized
5. Silt to silty sand, white, fine grained, laminar bedding

Explanation:

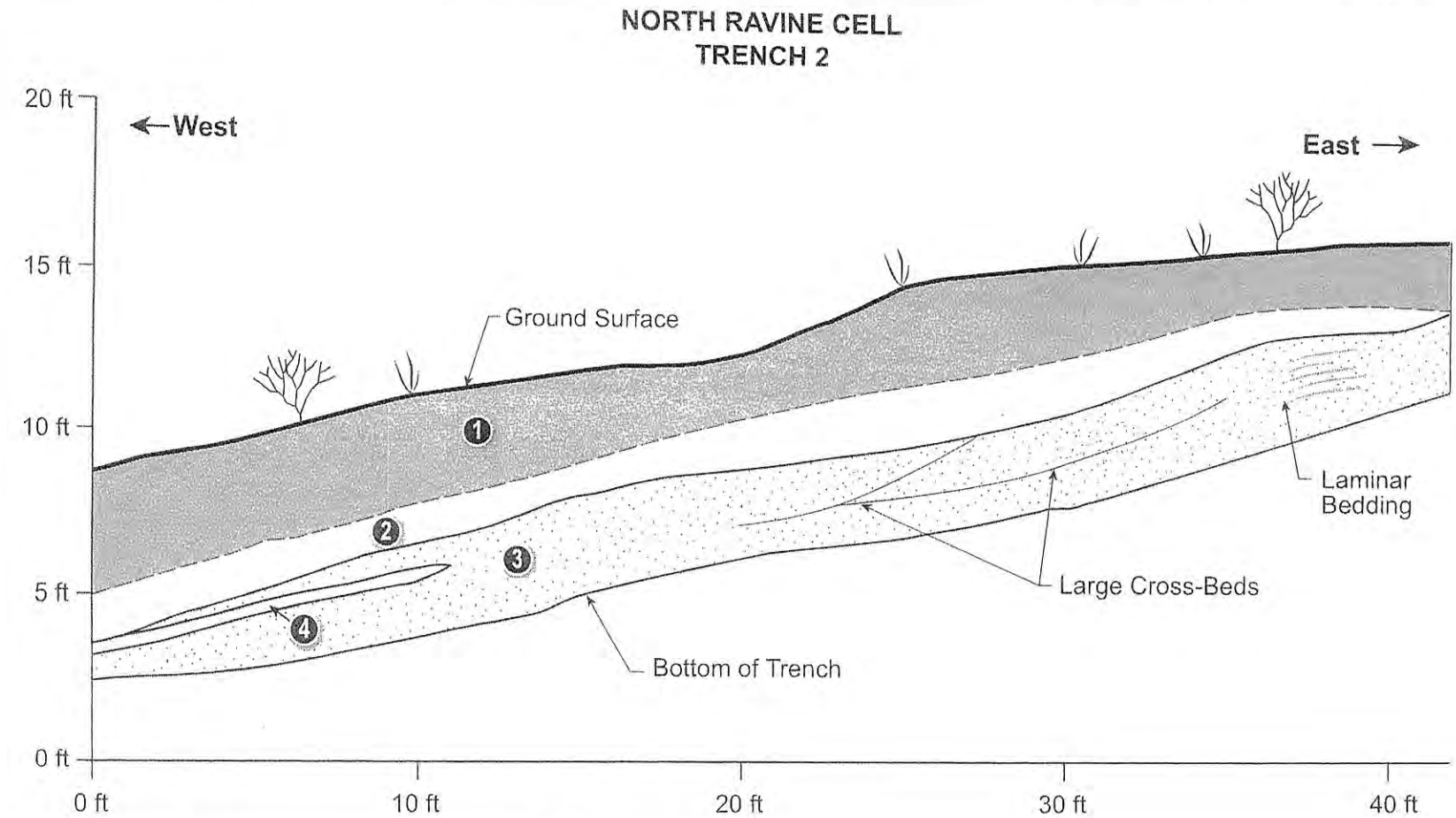
Contacts between stratigraphic units, dashed where gradational or inferred

Faults, arrows show direction of movement

FIGURE 4-16
TRENCH 1B
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

E012005003BOI

CH2MHILL



Stratigraphic Units:

1. Topsoil, silty to sandy, light brown to brown
2. Silty sand to sandy silt, white
3. Sand, orange to yellow, coarse-grained, cross-bedded
4. Silt, white, hard

Explanation:

Contacts between stratigraphic units, dashed where gradational or inferred

Faults, arrows show direction of movement

FIGURE 4-17
TRENCH 2
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

E012005003BOI

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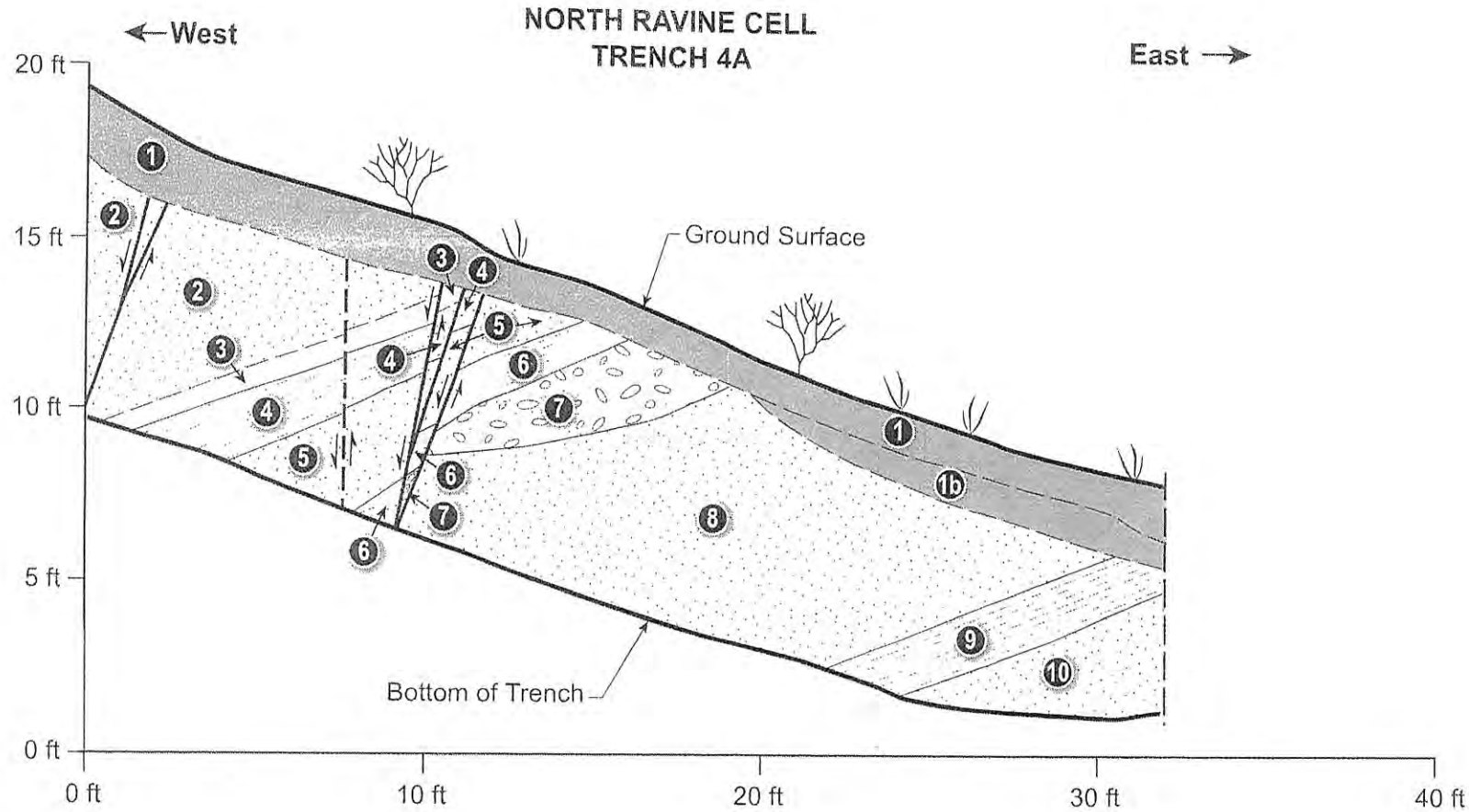

Stratigraphic Units:

1. Topsoil, brown, silty, unit 1B is colluvial topsoil
2. Sand, yellow-tan, medium to coarse grained, bedded with cross beds
3. Silt to sand, orange, transitional unit
4. Silt #1, white, massive, hard
5. Sand, orange to white, medium to coarse grained, laminated to cross-bedded
6. Silt #2, white, massive, hard
7. Sand lens, tan, coarse-grained with fine gravel
8. Sand, tan, medium to coarse grained, bedded
9. Silt #3, white, massive, hard
10. Sand, orange, fine to medium grained, bedded

Explanation:

Contacts between stratigraphic units, dashed where gradational or inferred

Faults, arrows show direction of movement

FIGURE 4-19
TRENCH 4A
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

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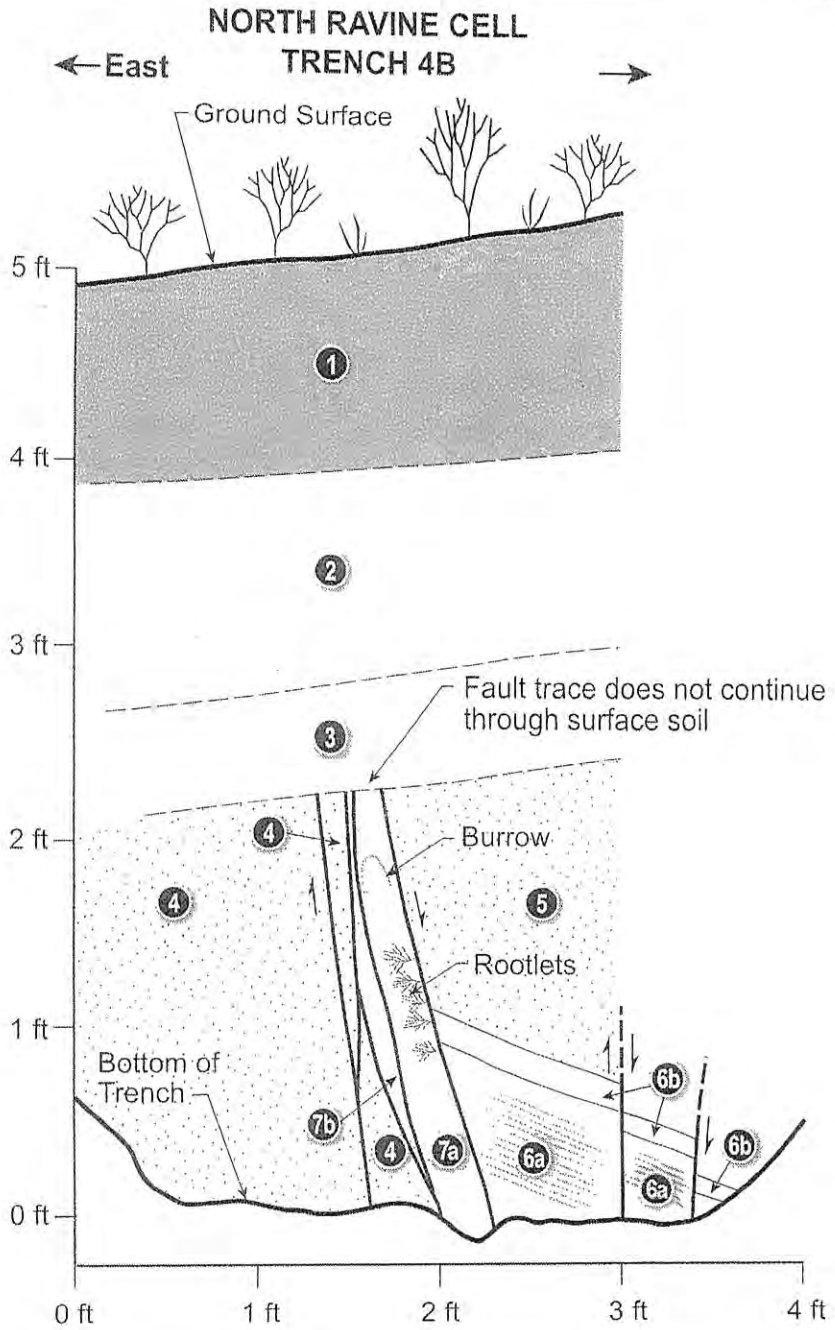


Explanation:

Contacts between stratigraphic units, dashed where gradational or inferred

Faults, arrows show direction of movement

Stratigraphic Units:

- 1 Topsoil, dark brown, silty
- 2 Clayey sand, dark brown, medium-grained, very hard, cemented
- 3 Clayey sand, medium brown, medium-grained, very hard, cemented
- 4 Sand, yellow-tan, medium-grained, slightly cemented
- 5 Sand, light brown, medium-grained, relatively loose
- 6a Sand, white to orange, laminar bedded
- 6b Orange, oxidized sandy seams
- 7a Sand, white, medium-grained, very loose, burrowed, rootlets in fault zone
- 7b Calcium-cemented sand

FIGURE 4-20
TRENCH 4B
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

E0120050036OI

CH2MHILL

4.2.4.4 Initial Soil Parameters

Daniel B. Stephens & Associates measured initial soil parameters including moisture content, bulk density, and a calculated value for porosity after compacting each sample to 85 percent relative compaction (based on maximum dry density). Moisture content was measured on both a gravimetric (weight) basis and volumetric basis. Bulk density tests were conducted for both dry and wet specimens of the sample. Finally, a value for porosity was calculated. The calculations of porosity and bulk density are made with the assumption of a soil particle specific gravity of 2.65 grams/cm^3. Copies of the laboratory reports for these tests are presented in Appendix F. Table 4-8 summarizes the results of these analyses.

TABLE 4-8
Summary of Initial Soil Properties

Sample ID	Description	Moisture Content, Gravimetric (%)	Moisture Content, Volumetric (%)	Dry Bulk Density (g/cm^3)*	Wet Bulk Density (g/cm^3)*	Calculated Porosity (%)
Topsoil Composite 1	Sandy clay (SC)	6.9	12.2	1.76	1.88	33.6
Topsoil Composite 2	Sandy lean clay (CL)	9.4	16.1	1.70	1.86	35.8
Topsoil Composite 3	Sandy lean clay (CL)	11.4	18.3	1.60	1.78	39.7

* Grams per cubic centimeter

4.2.4.5 Constant Head Hydraulic Conductivity

The saturated hydraulic conductivity of the three composite samples was tested using a constant-head laboratory method. The hydraulic conductivity was measured at two remolded densities: 85 percent and 95 percent relative compaction (based on maximum dry density). Table 4-9 presents the results of the hydraulic conductivity analyses.

TABLE 4-9
Saturated Hydraulic Conductivity of Topsoil

Sample ID	Description	K_{sat} at 95% RC[a, c] (cm/sec)	K_{sat} at 85% RC[b, c] (cm/sec)
Topsoil Composite 1	Sandy clay (SC)	3.72E-05	1.4E-05
Topsoil Composite 2	Sandy lean clay (CL)	2.00E-05	1.5E-06
Topsoil Composite 3	Sandy lean clay (CL)	5.44E-05	1.4E-06

[a] ASTM D5084
[b] ASTM D2434
[c] RC = relative compaction

The test results presented in Table 4-9 indicate the hydraulic conductivity is actually lower for the less-densely-compacted test specimens. These data are counter-intuitive and appear to be the result of the different testing laboratories performing the two sets of tests with different apparatus and different methods. Copies of the laboratory reports for these tests are presented in Appendix F.

4.2.4.6 Moisture Characteristics

Daniel B. Stephens & Associates analyzed the moisture characteristics of the three composite samples to determine the relationship between soil water content and soil water pressure

head. Four apparatuses were used to cover the entire pressure head range for these soils. The four apparatuses were: hanging column, pressure plate, water activity meter, and relative humidity box. A hanging column allows measurements up to approximately minus 200 centimeters of pressure head. The pressure plate apparatus covers from minus 200 centimeters to approximately minus 2,000 centimeters (minus 2 bar) of pressure head. The water activity meter covers from minus 2 bars to approximately minus 20 bars of pressure head. The relative humidity box covers greater than minus 20 bars of pressure head. Copies of the laboratory reports for these tests are presented in Appendix F. Table 4-10 summarizes the field capacity and permanent wilting point results.

TABLE 4-10
Water Retention/Moisture Characteristics

Sample ID	Description	VWC[a] at PWP[b] (cm^3/cm^3)	SWP[c] at PWP (-Bars)	VWC at FC[d] (cm^3/cm^3)	SWP[c] at FC[d] (cm^3/cm^3)
Topsoil Composite 1	Sandy clay (SC)	0.095	20	0.215	0.33
Topsoil Composite 2	Sandy lean clay (CL)	0.135	20	0.275	0.33
Topsoil Composite 3	Sandy lean clay (CL)	0.13	20	0.33	0.33

[a] VWC = Volumetric water content
[b] PWP = Permanent wilting point—water content at which plants cease transpiring and die
[c] SWP = Soil water pressure
[d] FC = Field capacity—water content at which all significant drainage ceases

4.2.4.7 Water Retention and Unsaturated Flow Model Parameters

Daniel B. Stephens & Associates fit the van Genuchten (1980) water retention and unsaturated hydraulic conductivity functions to the measured moisture characteristics and saturated hydraulic conductivity data. The fitting parameters used to define these models are α and N. The parameter α is approximately equal to the inverse of the air-entry pressure head at which the largest soil pores drain. The parameter N reflects the slope of the hydraulic conductivity and water content versus pressure head relations. Copies of the laboratory reports for these tests are presented in Appendix F. The tabulated fitting parameters for each sample are summarized in Table 4-11.

TABLE 4-11
Unsaturated Flow Model Fitting Parameters

Sample ID	Description	α (cm^{-1})	N (dimensionless)
Topsoil Composite 1	Sandy clay (SC)	0.0187	1.2015
Topsoil Composite 2	Sandy lean clay (CL)	0.0053	1.1911
Topsoil Composite 3	Sandy lean clay (CL)	0.0023	1.3923

4.2.4.8 Topsoil Fertility Analysis

Topsoil chemistry and fertility was not evaluated for this report because topsoil chemistry and fertility can change with time. A soil fertility analysis will be performed on the stockpiled topsoil just prior to the time of the proposed ET cover construction. The analysis will dictate the soil nutrients and amendments required to establish and support a thriving stand of native vegetation.

4.2.5 Groundwater Monitoring

4.2.5.1 Water Levels

Groundwater levels and elevations for the first water-bearing zone (uppermost water-bearing unit) for measurements collected in June, August, and December 2004 are presented in Table 4-12 and the resultant groundwater elevation contour maps are presented in Figures 4-21, 4-22, and 4-23, respectively. As previously noted, water level measurements were collected at all available NRC and HHLF Cell monitoring wells to develop a comprehensive map of groundwater flow beneath the NRC and local adjoining areas. As a result, Figures 4-21, 4-22, and 4-23 depict the water table surface of the uppermost aquifer over an extensive area that includes the NRC and adjoining properties.

TABLE 4-12
NRC Groundwater Elevations, June 2004 to December 2004

Well/ Boring ID	Northing	Easting	M.P. Elevation (ft amsl)[a]	Depth to Water June 2004 (ft bmp)[b,c]	Depth to Water August 2004 (ft bmp)	Depth to Water December 2004 (ft bmp)	Water Table Elevation, June 2004 (ft amsl)	Water Table Elevation, August 2004 (ft amsl)	Water Table Elevation, December 2004 (ft amsl)
North Ravine Monitoring Wells									
NRB-1	744924.58	362571.32	3189.99	180.56	180.53	180.43	3009.43	3009.46	3009.56
NRB-3	746149.44	361628.83	3132.21	78.41	78.52	78.56	3053.80	3053.69	3053.65
NRB-4	745441.37	361493.75	3086.19	40.93	41.38	41.46	3045.26	3044.81	3044.73
NRB-6	743826.06	361273.35	3146.26	178.9	178.84	178.90	2967.36	2967.42	2967.36
NRB-9	746098.86	360360.82	3028.24	10.47	11.86	11.64	3017.77	3016.38	3016.60
NRB-10	745568.55	360129.76	3038.46	62.65	63.1	62.93	2975.81	2975.36	2975.53
NRB-11	743864.65	360116.93	3080.05	118.59	118.74	118.76	2961.46	2961.31	2961.29
NRB-12	744520.37	359825.44	3044.19	68.01	68.07	56.94	2976.18	2976.12	2987.25
NRB-13	745382.26	359471.94	3009.77	36.84	36.97	37.02	2972.93	2972.80	2972.75
NRB-15	744522.32	359061.94	3038.25	99.16	99.23	99.31	2939.09	2939.02	2938.94
NRB-16	746091.29	358422.98	2937.81	2.32	3.74	1.76	2935.49	2934.07	2936.05
NRB-18	744634.67	357894.74	3039.80	112.25	112.25	112.32	2927.55	2927.55	2927.48
NRB-21	746807.80	359641.28	3070.38	59.36	59.49	59.57	3011.02	3010.89	3010.81
NRB-22	746811.95	360618.32	3108.22	45.12	45.38	45.41	3063.10	3062.84	3062.81
NRB-23	745646.99	356641.03	2876.8	32.74	32.74	33.57	2844.06	2844.06	2843.23
NRB-24	747203.00	361883.95	3120.3	30.65	31.07	31.24	3089.65	3089.23	3089.06
NRB-25	746832.27	359018.78	3121.62	147.16	147.34	147.48	2974.46	2974.28	2974.14
NRB-26	746426.80	357887.96	2965.36	30.92	31.01	31.03	2934.44	2934.35	2934.33
NRB-27	747386.27	360189.61	3180.66	127.96	128	128.02	3052.70	3052.66	3052.64
NRB-28	746733.55	362361.59	3206.67	103.31	103.62	103.89	3103.36	3103.05	3102.78
NRB-29	747465.93	359416.69	3136.12	124.86	125.13	125.46	3011.26	3010.99	3010.66
NRB-30	747145.14	357391.44	3085.19	174.17	174.18	174.16	2911.02	2911.01	2911.03
NRB-31	744768.71	356620.55	2918.02	170.58	170.53	170.45	2747.44	2747.49	2747.57
NRB-32	747436.79	354402.12	2777.48	36.25	36.56	36.69	2741.23	2740.92	2740.79
NRB-33	746538.16	350879.71	2734.1	dry	dry	dry	---	---	---
NRB-34	746055.74	359382.83	2985.21	4.81	5.78	5.53	2980.40	2979.43	2979.68
NRB-37	745008.24	357920.54	2995.95	---	68.87	68.92	---	2927.08	2927.03
NRB-39	746076.81	357864.00	2921.4	---	19.28	19.38	---	2902.12	2902.02
Hidden Hollow Landfill Monitoring Wells									
B-1	358,279.81	739,895.84	2,816.17	91.47	91.42	91.26	2724.70	2,724.75	2,724.91
B-2	358,470.71	741,015.98	2,869.59	116.03	115.96	115.96	2753.56	2,753.63	2,753.63
B-3	358,468.71	741,025.63	2,870.53	41.24	41.29	41.71	2829.29	2,829.24	2,828.82
B-4	360,273.72	739,230.04	2,774.59	17.09	17.32	17.16	2757.50	2,757.27	2,757.43
B-6A	360,229.38	740,349.09	3,024.14	240.41	240.33	240.40	2783.73	2,783.81	2,783.74
B-7	361,321.01	742,911.21	3,219.34	271.75	264.80	264.74	2947.59	2,954.54	2,954.60

TABLE 4-12
NRC Groundwater Elevations, June 2004 to December 2004

Well/ Boring ID	Northing	Easting	M.P. Elevation (ft amsl)[a]	Depth to Water June 2004 (ft bmp)[b,c]	Depth to Water August 2004 (ft bmp)	Depth to Water December 2004 (ft bmp)	Water Table Elevation, June 2004 (ft amsl)	Water Table Elevation, August 2004 (ft amsl)	Water Table Elevation, December 2004 (ft amsl)
B-8	358,546.25	741,939.81	2,970.58	128.35	128.25	128.21	2842.23	2,842.33	2,842.37
B-9	357,986.03	741,500.24	2,934.68	206.17	201.90	205.46	2728.51	2,732.78	2,729.22
B-10	361,013.86	741,020.53	2,979.69	136.33	136.32	136.27	2843.36	2,843.37	2,843.42
B-11	359,132.90	740,364.59	3,000.31	244.45	244.34	244.32	2755.86	2,755.97	2,755.99
B-12	358,815.64	738,216.56	2,713.07	29.20	29.69	29.98	2683.87	2,683.38	2,683.09
B-13	358,504.66	738,968.08	2,765.03	67.84	68.46	66.97	2697.19	2,696.57	2,698.06
B-14	357,989.55	737,036.30	2,662.49	38.74	37.54	38.06	2623.75	2,624.95	2,624.43
B-15	358,216.99	737,082.70	2,671.91	64.15	61.60	63.30	2607.76	2,610.31	2,608.61
B-16	357,105.80	742,383.51	2,836.70	123.62	123.26	121.22	2713.08	2,713.44	2,715.48
B-17	356,932.57	741,296.39	2,846.64	200.42	198.84	198.82	2646.22	2,647.80	2,647.82
B-18	357,401.58	738,613.26	2,790.51	(No Access)	(No Access)	(No Access)	---	---	---
B-19	358,087.73	736,877.32	2,652.93	52.62	41.52	50.41	2600.31	2,611.41	2,602.52
B-20	358,211.51	736,847.63	2,657.21	52.22	43.19	52.44	2604.99	2,614.02	2,604.77
B-21	357,821.07	736,977.07	2,649.78	26.56	50.90	27.28	2623.22	2,598.88	2,622.50
B-22	358,297.98	737,451.44	2,683.34	50.01	50.01	(Dry)	2633.33	2,633.33	---
B-23	358,211.51	737,069.33	2,671.39	61.25	59.75	61.82	2610.14	2,611.64	2,609.57
B-24	358,590.20	738,652.80	2,745.78	55.17	53.77	53.94	2690.61	2,692.01	2,691.84
B-25	358,592.10	738,688.90	2,748.15	53.23	53.22	53.09	2694.92	2,694.93	2,695.06
B-26	358,688.40	738,436.30	2,731.83	43.35	43.47	43.63	2688.48	2,688.36	2,688.20
B-27	358,449.80	739,311.00	2,786.03	(Dry)	(Dry)	(Dry)	---	---	---
B-28	358,680.90	738,444.50	2,732.88	29.16	29.91	30.40	2703.72	2,702.97	2,702.48
B-29	358,605.20	738,647.40	2,745.13	66.25	66.28	66.41	2678.88	2,678.85	2,678.72
B-30	358,600.11	738,637.01	2,745.37	56.61	56.75	57.01	2688.76	2,688.62	2,688.36
B-31	358,684.07	738,420.11	2,731.19	46.01	46.16	46.43	2685.18	2,685.03	2,684.76
B-32	358,548.69	738,833.96	2,758.39	63.92	64.34	63.31	2694.47	2,694.05	2,695.08
B-34	358,500.86	739,058.25	2,770.79	70.38	70.59	69.84	2700.41	2,700.20	2,700.95
B-35	358,515.56	738,559.88	2,734.63	39.44	39.70	39.75	2695.19	2,694.93	2,694.88
B-36	358,671.34	738,207.17	2,713.28	31.00	31.30	31.29	2682.28	2,681.98	2,681.99
B-38	358,402.72	738,673.85	2,762.70	(Dry)	(Dry)	(Dry)	---	---	---
B-39	358,287.52	737,458.04	2,683.13	70.83	69.80	71.93	2612.30	2,613.33	2,611.20
B-41	355,094.53	740,707.01	2,710.20	125.85	126.07	124.34	2584.35	2,584.13	2,585.86
Goose Creek Piezometers									
PGC-5	360316.55	746123.37	3020.0	4.01	---	6.71	3015.99	---	3013.29
PGC-6	360039.47	746143.78	3010.0	3.20	---	6.05	3006.80	---	3003.95
PGC-7	359804.83	746100.46	3000.0	4.95	---	6.89	2995.05	---	2993.11
PGC-9	359012.14	746090.63	2958.0	6.92	---	5.52	2951.08	---	2952.48
PGC-10	358683.06	746115.83	2945.0	3.95	---	3.64	2941.05	---	2941.36
PGC-11	358122.47	746014.36	2930.0	5.70	---	6.02	2924.30	---	2923.98

[a] amsl = above mean sea level
[b] bmp = below measuring point
[c] PGC Piezometers measured in May 2004

Based on the water table contour maps, the local groundwater flow direction beneath the NRC and local adjoining areas was toward the southwest. The local hydraulic gradient ranged from 0.0357 foot per foot (ft/ft) to 0.0952 ft/ft. Little change in hydraulic gradient was observed between the three monitoring events.

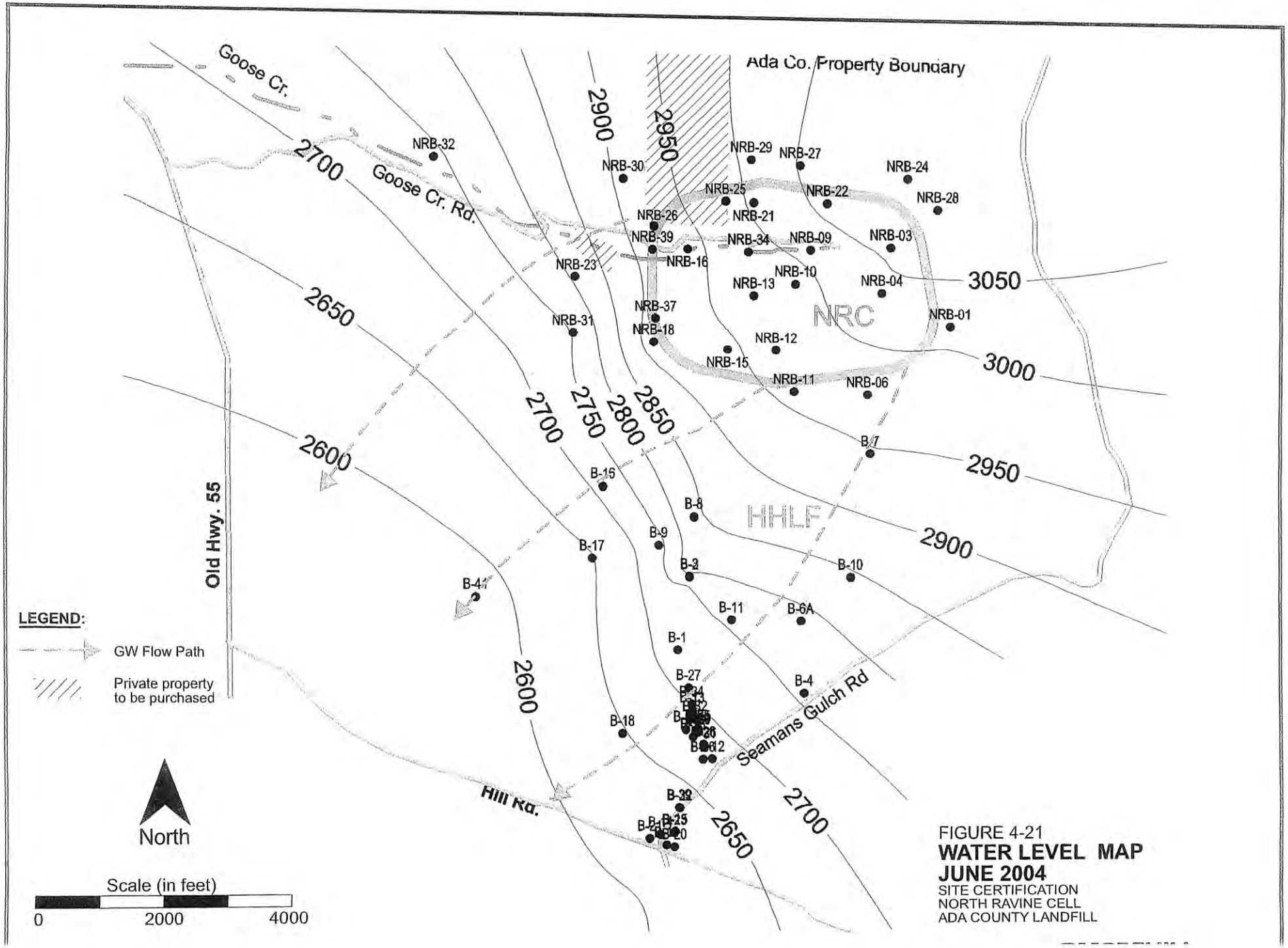


FIGURE 4-21
WATER LEVEL MAP
JUNE 2004
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

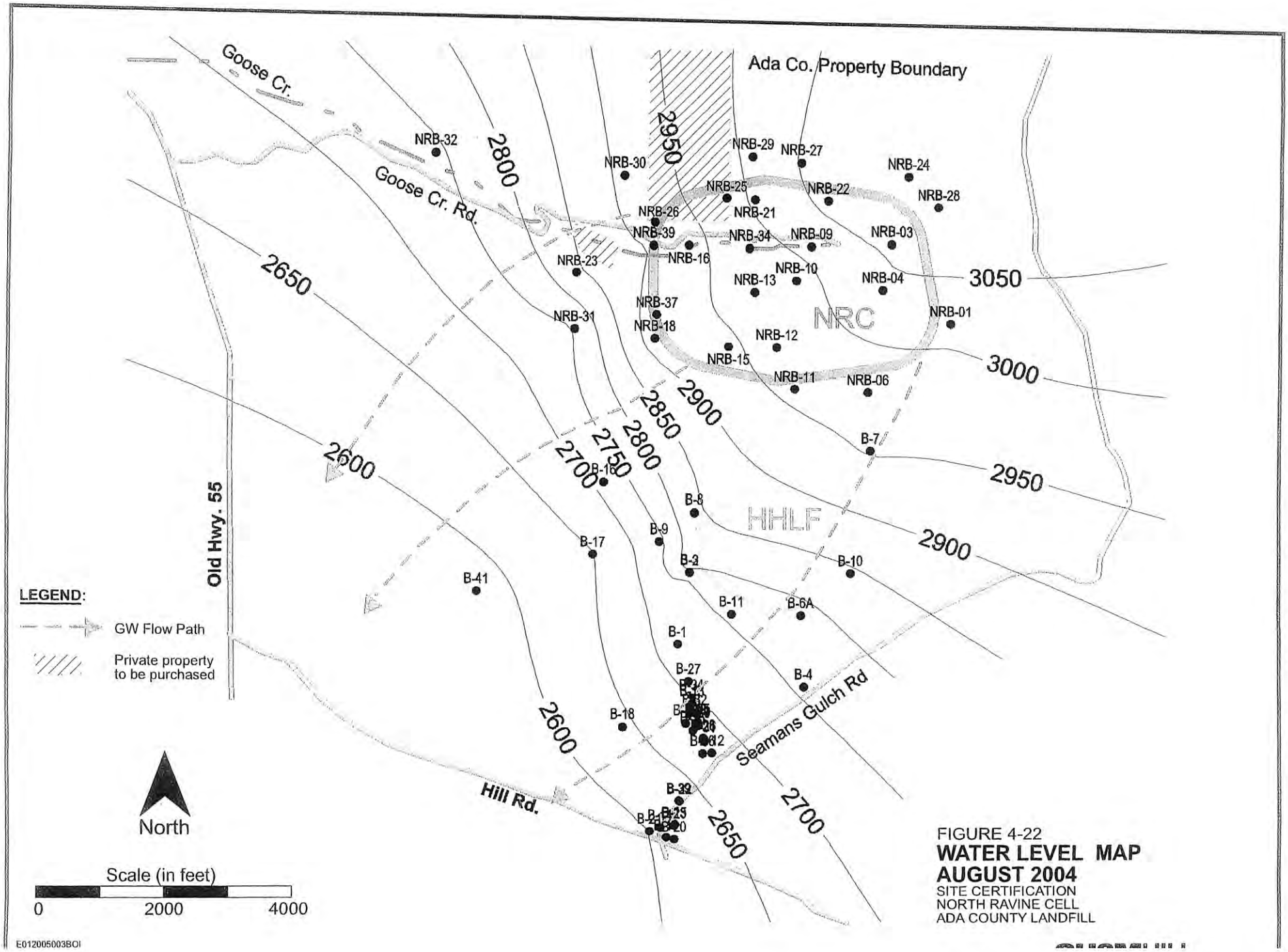


FIGURE 4-22
WATER LEVEL MAP
AUGUST 2004
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

E012005003BOI

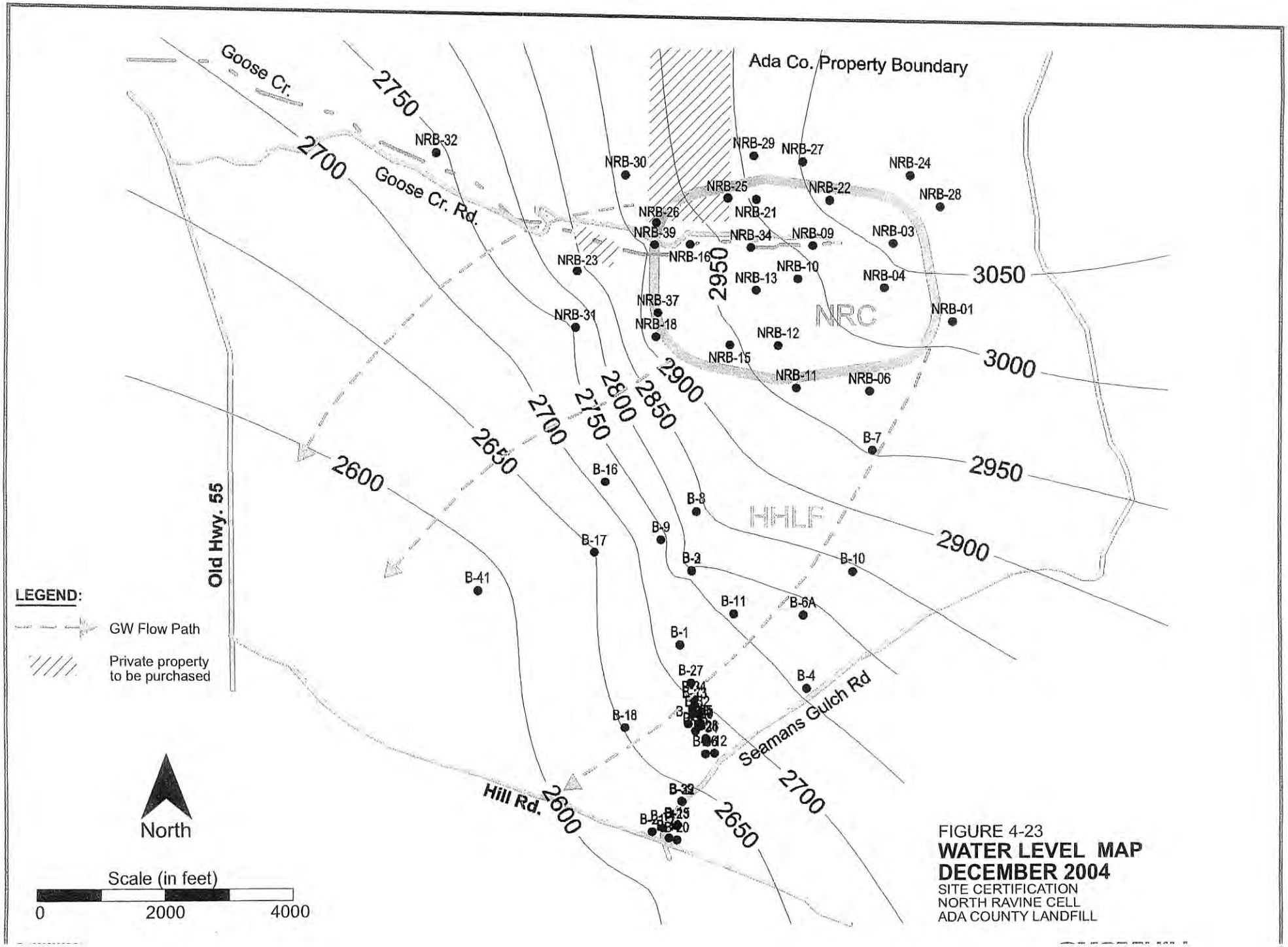


FIGURE 4-23
WATER LEVEL MAP
DECEMBER 2004
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

As previously discussed, the zone of saturation was typically encountered within the fluvial/deltaic sediments of the Pierce Gulch Formation. The bottom of zone of saturation was often defined by the underlying contact with the fine grained lacustrine blue clay/silt sediments of the Terteling Springs Formation. In all cases, the underlying fine-grained lacustrine sediments appeared to be unsaturated, indicating they form a low-permeable confining zone at the base of the water table aquifer. Three-dimensional contour maps depicting the local water table (August 2004) and the surface of the lacustrine deposits are presented in Figures 4-4a and 4-4b. The similarity of the two surface maps illustrates the extent to which the uppermost water-bearing unit is structurally controlled by the underlying, low-permeable lacustrine deposits.

4.2.5.2 Water Quality

Background (pre-cell construction) water quality conditions for the NRC were evaluated based on groundwater samples collected in November 2003, June 2004, and August 2004. Sample parameters included VOCs, RCRA metals, total organic carbon (TOC), major ions, and field parameters (temperature, pH, specific conductivity, and DO). Copies of the analytical laboratory reports for all samples collected are included in Appendix H. Specific water quality results are discussed in the following sections.

4.2.5.2.1 Field Parameters. Table 4-13 summarizes the background field parameter (temperature, pH, specific conductance, and DO) data for each monitoring well at the NRC.

TABLE 4-13
Temperature, pH, Specific Conductance, and Dissolved Oxygen (DO) Values for NRC Groundwater

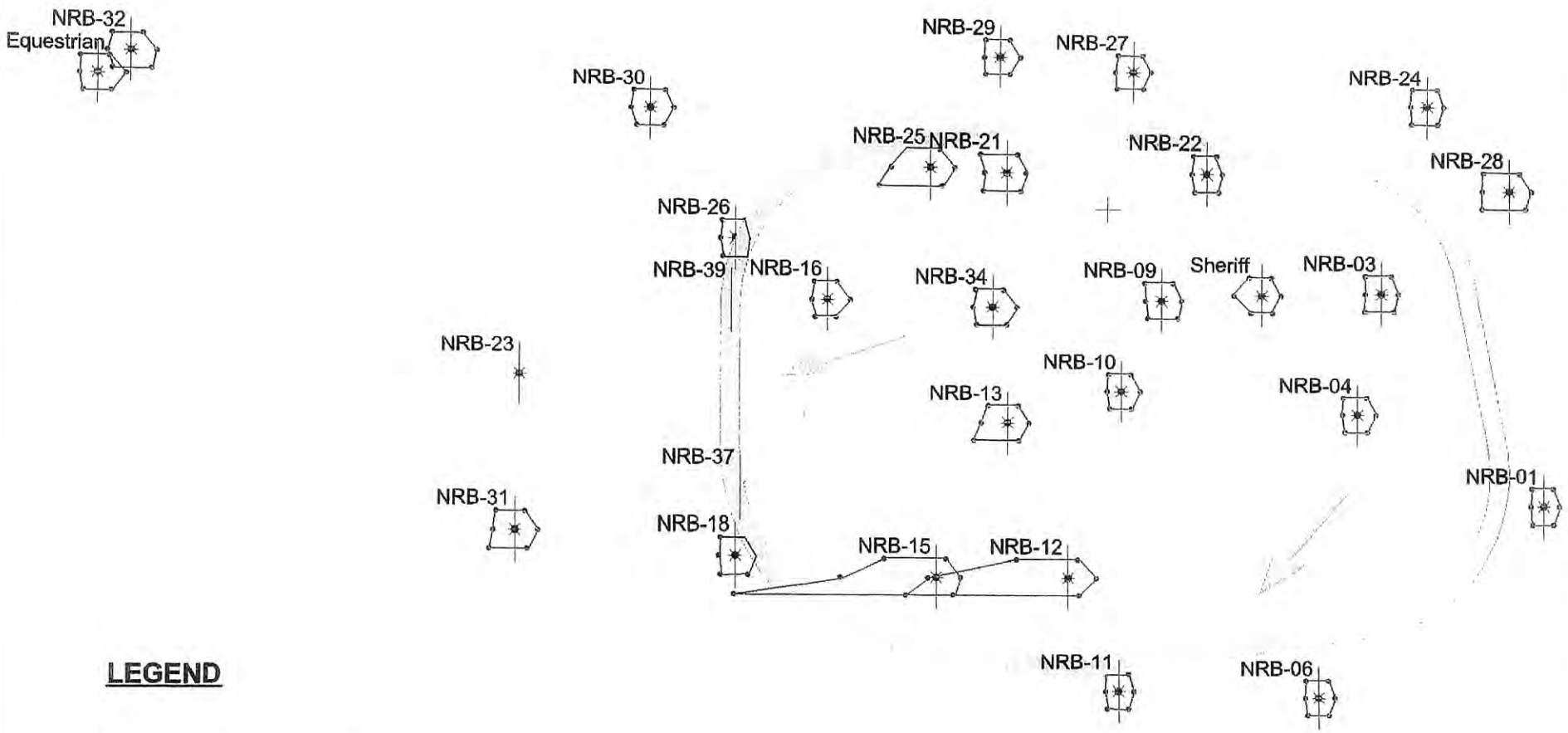
Well	Date	pH (Standard Units)	Temperature (degrees Celsius)	Specific Conductance (μmhos/cm)[a]	Dissolved Oxygen (mg/L)[b]
NRB-1	05/13/04	6.93	19.9	99	4.76
	08/27/04	6.77	17.3	98	9.48
NRB-3	11/10/03	7.03	15.0	183	—
	05/07/04	7.30	21.5	120	3.10
	08/23/04	7.39	19.1	118	5.93
NRB-4	05/10/04	7.13	19.0	128	5.06
NRB-6	11/10/03	9.01	17.4	95	—
	05/13/04	7.10	18.3	110	3.97
NRB-9	05/05/04	6.94	15.5	229	10.56
	08/17/04	7.12	18.6	195	12.27
NRB-10	05/10/04	6.99	15.8	134	1.63
NRB-11	11/05/03	8.10	14.4	86	—
	05/13/04	7.15	16.9	101	10.66
NRB-12	11/06/03	8.00	13.6	296	—
	05/14/04	7.45	17.4	140	4.12
	08/25/04	7.45	16.3	136	7.58
NRB-13	05/12/04	7.08	16.1	199	8.28
	08/17/04	7.21	17.8	184	8.77
NRB-15	11/06/03	8.40	11.9	161	—
	05/20/04	7.30	15.2	162	3.82
	08/23/04	7.36	15.6	122	9.18
NRB-16	05/04/04	6.78	14.2	164	12.4

TABLE 4-13
Temperature, pH, Specific Conductance, and Dissolved Oxygen (DO) Values for NRC Groundwater

Well	Date	pH (Standard Units)	Temperature (degrees Celsius)	Specific Conductance (μmhos/cm)[a]	Dissolved Oxygen (mg/L)[b]
NRB-18	11/06/03	6.80	16.7	163	—
	05/14/04	6.50	16.4	176	0.75
NRB-21	05/19/04	8.01	16.2	216	4.51
	08/20/04	7.58	18.2	141	10.74
NRB-22	05/05/04	6.88	16.1	152	11.25
NRB-23	08/17/04	6.76	15.1	294	9.73
NRB-24	05/05/04	7.58	17.4	122	8.46
	08/20/04	7.53	17.3	93	8.72
NRB-25	05/19/04	7.73	16.6	161	0.45
	08/26/04	6.69	17.2	127	6.49
NRB-26	05/06/04	6.95	18.9	143	9.79
	08/20/04	6.69	19.4	131	9.57
NRB-27	05/18/04	7.36	17.2	110	5.93
NRB-28	05/07/04	7.59	17.6	250	7.40
	08/24/04	7.67	17.3	169	6.77
NRB-29	05/06/04	7.19	19.3	157	7.10
NRB-30	05/06/04	7.60	17.8	232	11.47
	08/26/04	7.58	15.9	206	10.33
NRB-31	05/19/04	7.36	18.3	187	7.75
NRB-32	05/17/04	6.78	16.2	330	1.54
	08/24/04	6.99	17.1	332	7.78
NRB-34	05/17/04	6.49	16.4	241	3.35
	08/16/04	5.91	19.5	199	9.50
NRB-37	08/17/04	6.93	17.0	114	10.16
NRB-39	08/23/04	6.85	16.6	165	7.48
Maximum		9.01	21.5	332	12.40
Minimum		5.91	11.9	86	0.45
Median		7.15	17.1	161	7.75
Average		7.23	17.0	167	7.28

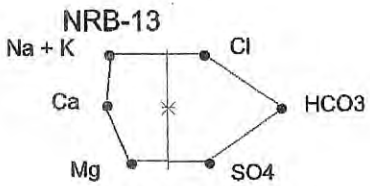
Notes:
[a]μmhos/cm = micro-mhos per centimeter
[b]mg/L = milligrams per liter
Blank cell indicates a parameter was not measured

4.2.5.2.2 General Water Chemistry Analysis (Common Ions). Background common ion concentrations (Table 4-14) of sodium, calcium, magnesium, potassium, chloride, bicarbonate, and sulfate were plotted and evaluated to compare general ionic chemistry across the site. Stiff diagrams for the uppermost aquifer based on June and August 2004 common ion data are presented in Figures 4-24 and 4-25, respectively. The Stiff diagrams in Figures 4-24 and 4-25 illustrate the distribution of common ions in the uppermost aquifer. The composition of groundwater depends on the rock type through which the water flows.



LEGEND

GW Flow Direction



Stiff Diagram

NRB-4

Well Not Sampled

FIGURE 4-24
GROUNDWATER CHEMISTRY/ STIFF DIAGRAMS
JUNE 2004
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

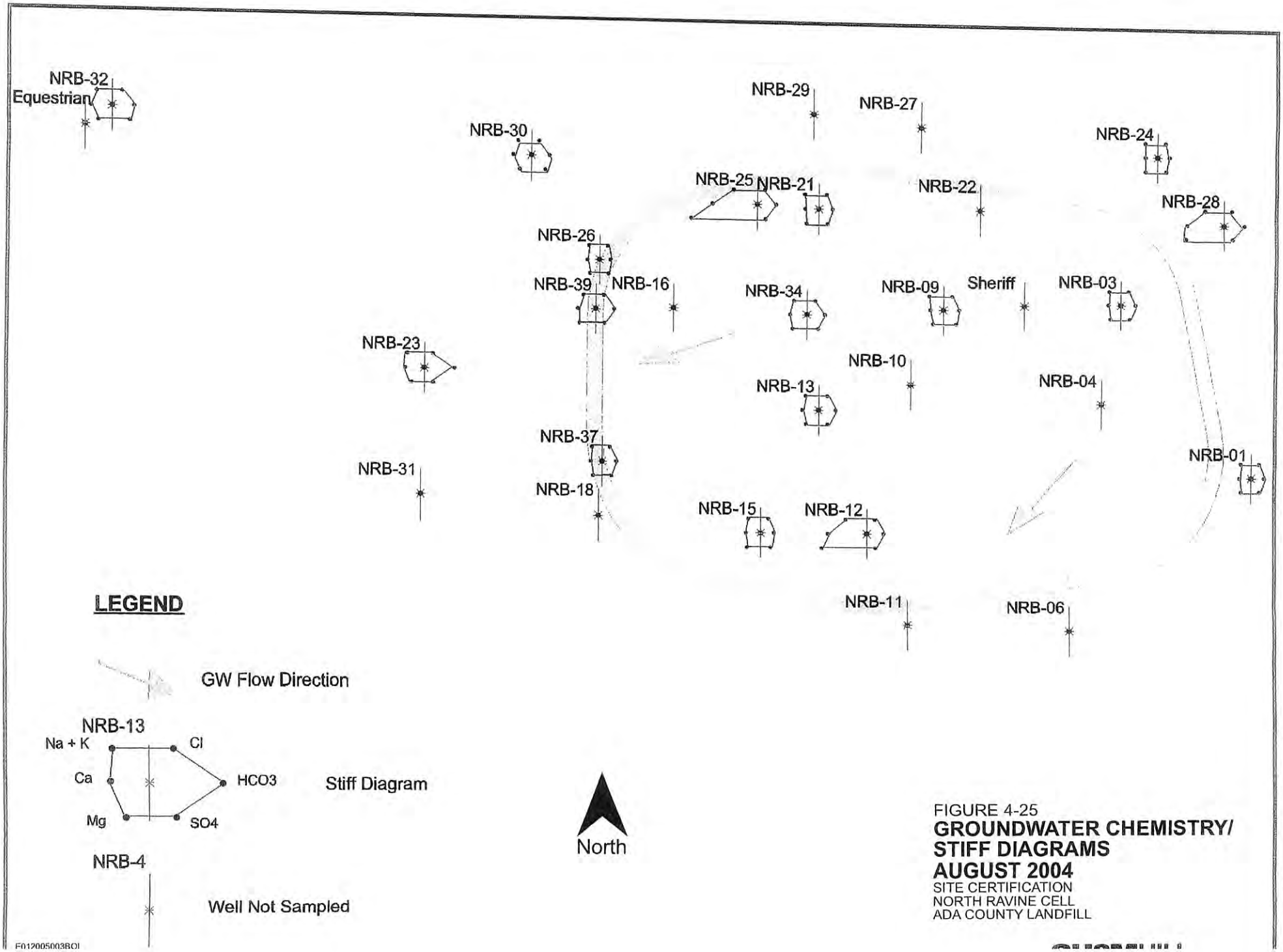


FIGURE 4-25
GROUNDWATER CHEMISTRY/ STIFF DIAGRAMS AUGUST 2004
SITE CERTIFICATION
NORTH RAVINE CELL
ADA COUNTY LANDFILL

In this case, the rock type is typically sands, claystones, and siltstones resulting from fluvial, deltaic, and lacustrine depositional environments. The parent rock of the sediments is generally of igneous origin (granitic and basaltic).

As shown in Figures 4-24 and 4-25, the background major ion composition does not appear to vary significantly across the site, with the possible exceptions of NRB-12, NRB-15, and NRB-25. The composition of upgradient groundwater is predominantly a Ca-HCO_3 (NRB-1, NRB-22, NRB-26, NRB-30) to a Ca-Mg-HCO_3 (NRB-24, NRB-25, NRB-27, NRB-28, NRB-29, and NRB-39) water. Along the groundwater flow path, ion exchange appears to increase the concentrations of Mg and Na compared to Ca. As a result, the major ion composition downgradient of the NRC trends towards a Mg-Ca-HCO_3 to Ca-Na-HCO_3 water.

As previously noted, the major ion compositions at NRB-12, NRB-15, and NRB-25 appeared to vary from the composition of the rest of the site, especially in the June 2004 sample set. In each case, the major ion composition of the water was Ca-Mg-HCO_3, but the TDS value was substantially higher resulting in the comparatively large Stiff diagrams shown on Figure 4-24. These wells were re-sampled in August 2004 to re-evaluate their major ion chemistry. Based on the August 2004, the TDS concentrations at NRB-12 and NRB-15 decreased substantially and their major ion compositions tended toward the average for the site. No significant change in composition was apparent for NRB-25. It is important to note the error balance (Table 4-14) for both samples at NRB-12, NRB-15, and NRB-25 were high, indicating sample or laboratory error may be an issue at these locations.

4.2.5.2.3 VOCs. Laboratory analytical results for VOCs detected in NRC groundwater monitoring well samples between November 2003 and August 2004 are summarized in Table 4-15. Trace concentrations (less than 9 micrograms per liter [μg/L]) of four Landfill Appendix I Regulated VOCs were detected in the uppermost aquifer, including acetone, carbon disulfide, toluene, and xylenes. No Landfill Appendix II Regulated VOCs or other unregulated VOCs were detected in the uppermost aquifer. Additionally, a trace concentration of carbon disulfide was also detected in the Sheriff's Well. No VOCs exceeded Maximum Concentration Levels (MCLs).

TABLE 4-14
Background Common Ion Concentrations, in milligrams per liter (mg/L)

Well ID	Date	Calcium	Sodium	Magnesium	Potassium	Chloride	Sulfate	Bicarbonate as $CaCO^3$	Bicarbonate as HCO^3	Water Type	Error Balance
NRB-NRB-01	05/14/04	10.2	7.6	3.2	2.5	2.7	6.6	38.7	47.2	Ca-HCO_3	1.9%
	08/27/04	14.0	8.9	5.5	4.7	3.2	7.4	42.1	51.3	Ca-HCO_3	15.7%
NRB-NRB-03	05/07/04	17.3	15.1	10.9	8.3	4.4	21.3	47.3	57.7	Mg-Ca-HCO_3	26.5%
	08/23/04	15.6	12.4	5.8	5.2	3.8	14.3	54.8	66.8	Ca-HCO_3	11.9%
NRB-NRB-04	05/10/04	14.8	12.2	5.1	4.7	2.2	7.1	55.9	68.2	Ca-HCO_3	9.1%
NRB-NRB-06	05/13/04	10.7	9.62	3.4	2.8	3.6	8.4	43.0	52.4	Ca-Na-HCO_3	1.2%
NRB-NRB-09	05/05/04	17.6	21.3	7.6	2.4	9.9	41.2	64.5	78.6	Na-Ca-HCO_3-SO_4	1.3%
	08/17/04	17.7	22.4	5.9	1.9	8.6	40.1	62.9	76.7	Na-Ca-HCO_3-SO_4	1.0%
NRB-NRB-10	05/11/04	12.8	10.3	4.2	1.8	4.0	6.7	60.2	73.4	Ca-Na-HCO_3	4.3%
NRB-NRB-11	05/13/04	10.3	9.40	4.1	2.5	4.1	6.1	36.6	44.6	Ca-Na-HCO_3	6.1%
NRB-NRB-12	05/14/04	306.0	31.7	217.0	141.0	3.9	12.5	114.0	139.0	Mg-Ca-HCO_3	86.5%
	08/25/04	92.2	24.0	66.9	40.3	4.2	10.7	68.0	82.9	Mg-Ca-HCO_3	74.5%
NRB-NRB-13	05/12/04	41.8	16.1	35.4	21.5	7.9	14.2	73.1	89.1	Mg-Ca-HCO_3	45.7%
	08/17/04	27.2	17.5	9.8	4.9	7.9	13.5	73.1	89.1	Ca-HCO_3	17.5%
NRB-NRB-15	05/20/04	204.0	27.6	274.0	151.0	3.3	43.5	89.3	108.9	Mg-HCO_3-SO_4	85.9%
	08/23/04	22.2	13.0	11.6	7.7	4.3	20.9	44.7	54.5	Ca-Mg-HCO_3	31.1%
NRB-NRB-16	05/04/04	16.0	10.8	5.1	3.2	3.8	0.8	83.9	102.3	Ca-HCO_3	1.0%
NRB-NRB-18	05/14/04	18.8	14.3	9.3	4.5	5.1	21.7	68.8	83.9	Ca-Mg-HCO_3	8.9%
NRB-NRB-21	05/19/04	32.1	38.7	21.0	15.5	8.3	28.6	60.9	74.3	Mg-Na-HCO_3	38.7%
	08/20/04	19.0	19.2	8.9	6.4	4.3	13.3	47.7	58.2	Ca-Na-HCO_3	26.8%
NRB-NRB-22	05/05/04	14.8	10.5	4.8	2.7	4.4	17.3	38.7	47.2	Ca-HCO_3	5.2%
NRB-NRB-23	08/17/04	34.7	33.1	10.1	1.8	7.4	13.6	164.0	200.0	Ca-Na-HCO_3	3.2%
NRB-NRB-24	05/05/04	18.0	11.3	9.7	7.6	6.1	11.5	45.2	55.1	Ca-Mg-HCO_3	23.2%
	08/20/04	15.0	14.9	8.8	6.6	3.3	10.8	34.5	42.1	Ca-Mg-HCO_3	34.4%

TABLE 4-14
Background Common Ion Concentrations, in milligrams per liter (mg/L)

Well ID	Date	Calcium	Sodium	Magnesium	Potassium	Chloride	Sulfate	Bicarbonate as $CaCO^3$	Bicarbonate as HCO^3	Water Type	Error Balance
NRB-NRB-25	05/19/04	72.0	14.7	60.0	46.2	2.9	15.4	89.3	108.9	Mg-Ca-HCO_3	63.8%
	08/26/04	109.0	16.3	103.0	65.7	3.7	9.4	87.3	106.4	Mg-Ca-HCO_3	76.7%
NRB-NRB-26	05/06/04	14.6	10.0	5.7	4.8	4.4	12.6	28.0	34.1	Ca-HCO_3	3.6%
	08/20/04	15.1	9.7	4.4	4.2	4.6	13.9	27.4	33.4	Ca-HCO_3	3.7%
NRB-NRB-27	05/18/04	21.8	9.4	15.9	11.0	2.0	8.8	50.8	61.9	Ca-Mg-HCO_3	33.3%
NRB-NRB-28	05/17/04	42.7	35.5	26.1	16.4	7.6	42.6	77.4	94.4	Mg-Ca-HCO_3-SO_4	38.3%
	08/24/04	85.7	23.1	53.9	27.4	4.4	11.4	97.4	118.8	Mg-Ca-HCO_3	62.9%
NRB-NRB-29	05/06/04	16.5	14.3	8.1	4.2	4.1	7.8	68.8	83.9	Ca-Mg-HCO_3	8.9%
NRB-NRB-30	05/06/04	24.9	17.6	6.9	7.0	24.9	27.8	86.0	104.9	Ca-HCO_3	5.8%
	08/26/04	32	17.9	7.5	4.9	3.6	49.2	79.2	96.6	Ca-HCO_3-SO_4	5.1%
NRB-NRB-31	05/19/04	30.5	17.9	24.1	15.7	4.6	18.5	81.2	99.0	Mg-Ca-HCO_3	36.9%
NRB-NRB-32	05/17/04	34.7	22.9	13.9	6.5	13.4	65.9	98.9	120.6	Ca-HCO_3-SO_4	3.6%
	08/24/04	40.1	24.9	11.8	4.2	14.1	74.2	106.0	129.2	Ca-HCO_3-SO_4	0.4%
NRB-NRB-34	05/17/04	26.1	19.1	10.7	7.2	8.0	29.4	90.3	110.1	Ca-HCO_3	9.5%
	08/16/04	28.3	15.6	12.8	7.6	6.4	29.1	79.2	96.6	Ca-Mg-HCO_3	16.8%
NRB-NRB-37	08/17/04	13.1	9.4	3.82	3.4	2.8	15.3	50.8	61.9	Ca-HCO_3	1.0%
NRB-NRB-39	08/23/04	30.7	12.4	16.3	9.4	4.3	13.5	81.2	99.0	Ca-Mg-HCO_3	28.1%
Equestrian	05/19/04	21.4	21.7	7.8	2.4	12.2	28.5	115.0	140.2	Ca-Na-HCO_3	9.2%
Sheriff's Shooting Range	05/26/04	45.1	8.8	3.2	3.1	2.4	10.5	60.9	74.3	Ca-HCO_3	32.1%

TABLE 4-15
Volatile Organic Concentrations, in micrograms per liter (μg/l)

Well ID	Date	VOC	Toluene	Acetone	Carbon disulfide	Total Xylenes
NRB-01	05/14/04	ND	—	—	—	—
	08/27/04	ND	—	—	—	—
NRB-03	11/10/03	ND	—	—	—	—
	05/07/04	ND	—	—	—	—
	08/23/04	—	4.9	—	—	—
NRB-04	05/10/04	ND	—	—	—	—
NRB-06	11/10/03	ND	—	—	—	—
	05/13/04	ND	—	—	—	—
NRB-09	05/05/04	ND	—	—	—	—
	08/17/04	ND	—	—	—	—
NRB-10	05/11/04	ND	—	—	—	—
NRB-11	11/05/03	—	—	—	2.2	—
	05/13/04	ND	—	—	—	—
NRB-12	11/06/03	ND	—	—	—	—
	05/14/04	—	1.3	—	—	—
	08/25/04	—	3.6	—	—	—
NRB-13	05/12/04	ND	—	—	—	—
	08/17/04	—	1.8	—	—	—
NRB-15	11/06/03	—	1.1	9.0	—	—
	05/20/04	ND	—	—	—	—
	08/23/04	—	—	—	7.0	—
NRB-16	05/04/04	ND	—	—	—	—
NRB-18	11/06/03	—	—	—	2.6	—
	05/14/04	ND	—	—	—	—
NRB-21	05/19/04	ND	—	—	—	—
	08/20/04	ND	—	—	—	—
NRB-22	05/05/04	ND	—	—	—	—
NRB-23	08/17/04	ND	—	—	—	—
NRB-24	05/05/04	ND	—	—	—	—
	08/20/04	ND	—	—	—	—
NRB-25	05/19/04	—	2.9	—	—	2.5
	08/26/04	ND	—	—	—	—
NRB-26	05/06/04	ND	—	—	—	—
	08/20/04	ND	—	—	—	—
NRB-27	05/18/04	ND	—	—	—	—
NRB-28	05/17/04	ND	—	—	—	—
	08/24/04	—	1.1	—	—	—
NRB-29	05/06/04	ND	—	—	—	—
NRB-30	05/06/04	—	—	8.1	—	—
	08/26/04	ND	—	—	—	—
NRB-31	05/19/04	ND	—	—	—	—
NRB-32	05/17/04	ND	—	—	—	—
	08/24/04	ND	—	—	—	—
NRB-34	05/17/04	ND	—	—	—	—
	08/16/04	ND	—	—	—	—
NRB-37	08/17/04	ND	—	—	—	—

TABLE 4-15
Volatile Organic Concentrations, in micrograms per liter (μg/l)

Well ID	Date	VOC	Toluene	Acetone	Carbon disulfide	Total Xylenes
NRB-39	08/23/04	ND	—	—	—	—
Equestrian	05/19/04	ND	—	—	—	—
Sheriff's Shooting Range	05/26/04	—	—	—	1.8	—
Notes:						
MCL (Maximum Concentration Limit)			1000	NA	NA	10000
ND = Non-detect						

Acetone was detected at concentrations of 9.0 μg/L and 8.1 μg/L at wells NRB-15 and NRB-30, respectively. A possible source of dissolved acetone in NRB-15 is equilibrium partitioning with landfill gas that is emanating from the HHLF Cell (see Section 4.4). Landfill gas, including several VOC vapors, is known to be present in several of the vapor monitoring wells located near to NRB-15. This includes acetone vapors in vapor monitoring wells M-6, B-7, and NRB-15 (NRB-15 functions as both a groundwater and vapor monitoring well). The source of the trace concentration of acetone in NRB-30 is unknown. Landfill gas migration and groundwater flow from the HHLF Cell are not likely to impact NRB-30 because NRB-30 is hydraulically oblique to the HHLF Cell. Also, the Goose Creek drainage should act as a physical barrier to vapor transport between the HHLF Cell and NRB-30. Another possible source of the acetone in NRB-15 and NRB-30 is laboratory error because acetone is a common laboratory contaminant.

Carbon disulfide was detected at concentrations ranging from 1.8 μg/L to 7.0 μg/L at wells NRB-11, NRB-15, NRB-18, and in the Sheriff's Shooting Range well. Wells NRB-11, NRB-15, NRB-18 are located adjacent to the HHLF Cell. Similar to acetone, a possible source of dissolved carbon disulfide in NRB-11, NRB-15, and NRB-18 is equilibrium partitioning with landfill gas from the HHLF Cell (see Section 4.4). Landfill gas is known to be present in several nearby vapor monitoring wells, including carbon disulfide vapors in vapor monitoring wells M-1, M-5, M-6, NRB-15, and B-7. Landfill gas is an unlikely source for the trace of dissolved carbon disulfide detected in the Sheriff's Shooting Range well because the Sheriff's well is a deep well completed within the blue clay and carbon disulfide was not detectable in any intervening well including NRB-4, NRB-10, NRB-13, and NRB-12. The probable source of carbon disulfide in the Sheriff's Well is anaerobic biodegradation. Carbon disulfide is a common, natural breakdown product from the microbial reduction of sulfates in soil. Other possible sources of the carbon disulfide in the Sheriff's well are laboratory or sample error.

Toluene was detected at concentrations ranging from 1.1 μg/L to 4.9 μg/L at wells NRB-3, NRB-12, NRB-13, NRB-15, NRB-25, and NRB-28. Wells NRB-12, NRB-13, NRB-15 are located near to the HHLF Cell. Similar to carbon disulfide and acetone, a possible source of dissolved toluene in NRB-12, NRB-13, NRB-15 is equilibrium partitioning with landfill gas that is emanating from the HHLF Cell (see Section 4.4). Landfill gas is known to be present in several nearby vapor monitoring wells, including toluene vapors in vapor monitoring wells M-1, M-6, M-7, NRB-15, and B-7. Landfill gas is an unlikely source for the trace concentration of dissolved toluene detected in NRB-3, NRB-25, and NRB-28 because NRB-3,

NRB-25, and NRB-28 are hydraulically upgradient of the HHLF Cell and the Goose Creek drainage should form a physical barrier to vapor flow between NRB-3, NRB-25, and NRB-28 and the HHLF Cell. Possible sources of the toluene detections at NRB-3, NRB-25, and NRB-28 are laboratory or sample error or oil from the drill rig.

Xylenes were detected at 2.5 $\mu g/L$ at well NRB-25. Landfill gas is an unlikely source for the trace of dissolved xylenes detected in NRB-25 because NRB-25 is hydraulically upgradient of the HHLF Cell and the Goose Creek drainage should form a physical barrier to vapor flow between NRB-25 and the HHLF Cell. Possible sources of the xylenes detection at NRB-25 are oil from the drill rig or laboratory or sample error.

4.2.5.2.4 Inorganic Chemicals. Table 4-16 summarizes the background concentrations of inorganic chemicals (RCRA metals and nitrate) at the NRC study area. As shown in Table 4-16, the background concentrations of several metals exceeded MCLs, including antimony, arsenic, barium, beryllium, cadmium, chromium, copper, lead, selenium, and thallium. Metals exceeded MCLs across the entire study site, including several upgradient locations, mid-gradient, and downgradient locations. Given the probable absence of an artificial (human-made) source, the exceedences represent natural background conditions and result from the collection of unfiltered samples from unproductive wells. Many wells were unproductive because they were completed within fine-grained sediments. These wells were difficult to pump and often could not be fully developed resulting in substantial amounts of suspended sediments in the water samples. Additionally, pursuant to 40 CFR 258.53, groundwater samples were not field-filtered prior to laboratory analysis. Given the low productivity and the resultant difficulty in developing the wells, the occurrence of elevated metal concentrations in the unfiltered samples is not surprising because the analysis would include concentrations of the absorbed fraction (metals absorbed to particles greater than 40 microns in size) in addition to the dissolved fraction (metals absorbed to particles less than 40 microns in size). These data indicate that the NRC design should incorporate techniques to control sediment in groundwater samples and a filter study to correct for the presence of sediments in compliance (sentinel) monitoring wells.

4.2.5.2.5 Total Inorganic Carbon (TOC). Table 4-17 summarizes the background concentrations of TOC at the NRC study area. Given the probable absence of an artificial (human-made) source, the observed TOC levels (less than 10 mg/L) are likely to be from the natural infiltration of precipitation and soil water.

TABLE 4-17
Background Total Organic Carbon (TOC) Concentrations, in milligrams per liter (mg/l)

Well ID	Date	TOC
NRB-NRB-01	05/14/04	<1.0
	08/27/04	<1.0
NRB-NRB-03	05/07/04	<1.0
	08/23/04	1.61
NRB-NRB-04	05/10/04	1.16
NRB-NRB-06	05/13/04	<1.0
NRB-NRB-09	05/05/04	1.17
	08/17/04	1.13
NRB-NRB-10	05/11/04	<1.0
NRB-NRB-11	05/13/04	<1.0
NRB-NRB-12	05/14/04	9.31
	08/25/04	1.36
NRB-NRB-13	05/12/04	<1.0
	08/17/04	<1.0
NRB-NRB-15	05/20/04	4.12
	08/23/04	1.43
NRB-NRB-16	05/04/04	1.56
NRB-NRB-18	05/14/04	<1.0
NRB-NRB-21	05/19/04	1.43
	08/20/04	2.93
NRB-NRB-22	05/05/04	1.08
NRB-NRB-23	08/17/04	1.35
NRB-NRB-24	05/05/04	<1.0
	08/20/04	<1.0
NRB-NRB-25	05/19/04	<2.88
	08/26/04	<1.00
NRB-NRB-26	05/06/04	<1.0
	08/20/04	<1.0
NRB-NRB-27	05/18/04	<1.0
NRB-NRB-28	05/17/04	1.82
	08/24/04	<1.0
NRB-NRB-29	05/06/04	1.24
NRB-NRB-30	05/06/04	4.41
	08/26/04	2.79
NRB-NRB-31	05/19/04	<1.0
NRB-NRB-32	05/17/04	1.31
	08/24/04	1.12
NRB-NRB-34	05/17/04	2.14
	08/16/04	2.3
NRB-NRB-37	08/17/04	<1.0
NRB-NRB-39	08/23/04	1.11
Equestrian	05/19/04	1.01
Sheriff's Shooting Range	05/26/04	<1.01
Maximum		**9.31**
Minimum		**<1.0**
Average		**1.65**

4.2.6 Aquifer Testing/Hydraulic Conductivity Measurements

4.2.6.1 Horizontal Conductivity Measurements

Two different field-scale aquifer test methods were used for the NRC: slug testing and pump testing. Selected wells were tested using one of these two methods based on field observations during their development and subsequent recovery. For each test, data was recorded during both the stressing and the removal of stress to the aquifer to determine the field-scale aquifer properties. Analysis of field-scale aquifer properties of each well are discussed below. The locations of the aquifer tests are shown in Figure 3-4.

4.2.6.1.1 Slug Testing. Seven uppermost water-bearing unit wells and one deep well were selected for slug testing and subsequent bail testing. The uppermost water-bearing unit wells included NRB-3, NRB-4, NRB-6, NRB-9, NRB-13, NRB-26, NRB-31, and the deep well was the Terteling #2 (T-2) well. Slug tests were analyzed using the Bouwer and Rice (1976) solution in Aqtesolv™. Because these uppermost water-bearing wells were developed and have a sand filter pack, the mathematical solution may overstate the hydraulic conductivity because of the sand pack around the well. Because wells NRB-26, NRB-31, and T-2 did not have water levels above the screen interval during the entire test (see Table 4-18), the solution requires use of effective radius (borehole radius opposed to piezometer radius) in its mathematical solution.

TABLE 4-18
NRC Water Levels During Slug Testing To Screen Interval

Well ID	Screen Interval (feet bgs)	Water Table Range During Test (feet bgs)	Water Level Above Screen Interval During Testing
NRB-3	95 to 105	75.12 to 78.52	Yes
NRB-4	52 to 62	40.86 to 43.55	Yes
NRB-6	178 to 88	175.74 to 176.44	Yes
NRB-9	20 to 30	7.04 to 9.98	Yes
NRB-13	48.7 to 68.7	32.49 to 36.02	Yes
NRB-26	19.2 to 39.2	26.05 to 28.22	No
NRB-31	160.5 to 170.5	165.25 to 167.43	No
Terteling #2	24.0 to 615.0*	32.40 to 42.60	No

*Open hole (no well screen)

The slug test solutions are summarized in Table 4-19. The calculated range in hydraulic conductivity for the uppermost water-bearing unit wells was 5.70E-05 cm/sec to 2.43E-01 cm/sec. The hydraulic conductivity for the deep Terteling #2 well was 4.17E-07 cm/sec. The calculated hydraulic conductivities were generally consistent with literature values for the type of lithology described in the boring logs. Graphs of the solutions are presented in Appendix I. Aquifer test drawdown and elapsed time data is available upon request.

TABLE 4-19
NRC Slug Test Results

Well ID	Test	Hydraulic Conductivity (cm/sec)	Notes
NRB-3	Slug	5.70E-05	Value similar to silty sands or fine sands
	Bail	1.70E-04	Value similar to silty sands or fine sands
NRB-4	Slug	1.32E-03	Value similar to silty sands or fine sands
	Bail	4.20E-04	Value similar to silty sands or fine sands
NRB-6	Slug	6.69E-02	Value similar to clean sands
	Bail	1.34E-01	Value similar to clean sands
NRB-9	Slug	3.63E-04	Value similar to silty sands or fine sands
	Bail	7.45E-05	Value similar to silty sands or fine sands
NRB-13	Slug	2.20E-03	Value similar to silty sands or fine sands
	Bail	1.42E-03	Value similar to silty sands or fine sands
NRB-26	Slug	7.66E-05	Value similar to silty sands or fine sands
	Bail	1.78E-04	Value similar to silty sands or fine sands
NRB-31	Slug	2.43E-01	Value similar to well sorted sands
	Bail	6.49E-05	Value similar to silty sands or fine sands
Terteling #2	Bail	4.17E-07	Value for clays and silts

4.2.6.1.2 Pump (Residual Drawdown) Testing. Five uppermost water-bearing unit wells and two deep wells were selected for single well pump test recovery (residual drawdown). The uppermost water-bearing unit wells included NRB-18, NRB-22, NRB-23, NRB-34, and NRB-37, and the deep wells were the Equestrian and Sheriff's Shooting Range wells.

The Jacobs Straight Line Method (Jacobs, 1950) was used to determine a value for K from single well pump test recovery (residual drawdown) data. The Jacob Straight Line Method was selected because in a single well test, residual recovery data are generally more reliable than drawdown data (the recovery occurs at a constant rate, whereas constant discharge pumping is often difficult to achieve in the field).

The results of the analysis are summarized in Table 4-20. The calculated range in hydraulic conductivity for the uppermost water-bearing unit wells was 7.8E-04 cm/sec to 3.8E-02 cm/sec. The calculated range in hydraulic conductivity for the deep wells was 6.0E-05 cm/sec to 7.6E-05 cm/sec. The calculated hydraulic conductivities were generally consistent with literature values for the type of lithology described in the boring logs. Graphs of the solutions are presented in Appendix J. Aquifer test drawdown and elapsed time data is available upon request.

TABLE 4-20
NRC Pump Test (Residual Drawdown) Results

Well ID	Q (gpm)	T (cm^2/sec)	B (cm)	Hydraulic Conductivity (cm/sec)	Notes
NRB-18	2.0	0.61	701	8.7E-04	Value similar to silty sands or fine sands
NRB-22	2.0	NA	—	NA	Pumped well went dry
NRB-23	2.0	1.95	579	3.4E-03	Value similar to clean sands
NRB-34	2.0	NA	—	NA	Pumped well went dry
NRB-37	2.0	6.90	183	3.8E-02	Value similar to clean sands
Equestrian	2.6	0.04	671	6.0E-05	Value similar to silty sands
Sheriff's Shooting Range	2.3	0.0139	183	7.6E-5	Value similar to silty sands

NA = Not Applicable, see "Notes" column.

Note that the hydraulic conductivity values listed in Tables 4-19 and 4-20 for the uppermost water-bearing unit wells are consistent with previously reported values of hydraulic conductivity for the uppermost water bearing unit at the HHLF Cell; 8.15E-03 cm/sec, 2.82E-04 cm/sec, 2.30E-03 cm/sec, and 2.43E-03 cm/sec for HHLF Cell wells at B-6, B-9, B-10, and B-23, respectively (CH2M HILL, 1996a).

The low hydraulic conductivity values listed in Tables 4-19 and 4-20 for the deep wells are consistent with silt/clay units with limited water production capabilities. The Driller's Reports indicate each of these deep wells were completed within a massive clay/silt unit (Terteling Springs Formation) containing occasional thin sandy interbeds. The aquifer test and Driller's Report data indicate the sand seams that are intersected by these wells are not very productive and are probably discontinuous. In the extreme case, the Terteling #2 well log shows no detectable water-bearing strata along the entire section of its 615 feet of open hole. This observation is consistent with its very low hydraulic conductivity value of 4.17E-07 cm/sec. In the case of the Sheriff's Shooting Range well, the Driller's Report indicates that this 400 feet deep well has an estimated 6 feet of water producing sandy clays. Furthermore, according to the Driller's Report, the well can be entirely dewatered within one hour by pumping the well at 10 gpm. For the Equestrian well, the Driller's Report indicates that this 150 feet deep well has about 22 feet of water producing fine sands. The low calculated hydraulic conductivity value (6.0E-05 cm/sec) for the Equestrian well is similar to the hydraulic conductivity of the water producing sandy clays of the Sheriff's Shooting Range.

4.2.6.2 Laboratory Permeability Testing for Vertical Hydraulic Conductivity

Table 4-21 summarizes the results of laboratory permeability testing on selected vadose zone, saturated zone, and the confining zone soils at the NRC. These test results represent the vertical hydraulic conductivity (K_v) of the samples. As shown in Table 4-21, the two available undisturbed vadose zone samples ranged in K_v from 1.57E-05 to 5.25E-06 cm/sec.

The three available undisturbed saturated zone (uppermost aquifer) samples ranged in K_v from 5.91E-05 to 2.71E-07 cm/sec. The 17 available undisturbed confining zone samples ranged in K_v from 4.91E-05 to 2.84E-09 cm/sec. These ranges in K_v are consistent with published hydraulic conductivity values for similar lithologies (Freeze and Cherry, 1979).

The low permeability of the blue clay forms a no-flow boundary to the overlying sands of the uppermost water-bearing unit. Note that the hydraulic conductivity of the majority (9 of 17 samples) of blue clay samples was lower than 5E-06 cm/sec. Also note that hydraulic conductivities approaching 1E-07 cm/sec and smaller are characterized as low-permeability material (Dominico and Schwart, 1990). The low permeability of the confining zone blue clays is of critical importance to the construction of the NRC. The NRC will be constructed with substantial safeguards to prevent a leachate release to the environment, including a double layer of synthetic liners and leachate collection system. Should the liner system fail, the low permeability of the underlying blue clay will form a no-flow boundary that will prevent downward vertical migration of leachate into deeper regional aquifer systems. In other words, the low permeability of the blue clay will confine leachate migration to the uppermost aquifer where it can be detected by the compliance monitoring system.

To evaluate the potential effect that loading or unloading may have on the K_v of the blue clay, staged variable loading of 15 psi, 35 psi, and 50 psi was applied by the laboratory to six of the blue clay samples (see Table 4-21). The load (effective stress) applied by the laboratory is important because it can have a direct effect on the predicted K_v of the sample. The load should have an inverse relationship to K_v (as the load increases, the K_v gets smaller). Consequently, the loading applied to the confining zone soil samples should be similar to in-situ loading that will occur at the NRC. The observed effect of variable loading on the six blue clay confining zone samples from the NRC study area is illustrated in Figure 4-26. In each case, the K_v decreased as the load increased.

The effective stress of the overburden on the confining zone beneath the NRC is anticipated to vary both temporally and spatially as construction of the NRC progresses. The effective stress (in psi) on the confining zone beneath the NRC is calculated based on the equation:

$$E = \frac{(S_h \times S_b) + (T_h \times T_b) - (W_h \times W_b)}{144\left(in^2 / ft^2\right)} \qquad \textit{Equation 1}$$

where

E = Effective Stress, psi
S_h = height of the soil column above the blue clay, in ft
S_b = bulk density of soil, in ft
W_h = height of the water column above blue clay, in ft
W_b = bulk density of water, in lbs/ft^3
T_h = height of the trash column above blue clay, in ft
T_b = bulk density of trash, in lbs/ft^3

Table 4-22 summarizes the probable range of effective stress on the surface of the confining zone at the NRC as calculated using Equation 1. Currently, pre-construction effective stress at the surface of the confining zone is estimated to range from approximately 7 psi (NRB-20) to 162 psi (NRB-1). The low end of the range (7 psi) represents the thin layer of overburden

along the bottom of the Goose Creek drainage whereas the upper end of the range represents the effective stress along existing upland portions of the site. Upon construction of the liner system (before waste disposal), the effective stress of the overburden on the surface of the confining zone is estimated to range from 12 psi to 54 psi. Upon completion of the landfill (after waste disposal), the effective stress of the overburden on the confining zone is estimated to range from 150 to 161 psi. From this analysis, the K_v values based on 15 psi of loading appear to be conservative estimates of K_v for the confining blue clays at the NRC.

TABLE 4-22
Calculated Range in Effective Stress on Confining Zone Beneath the NRC

	Range of Depth of Soil (ft)	Range of Depth of Landfill Waste (ft)	Depth of Groundwater (ft)	Effective Stress at Blue Clay (ft)
Current Conditions				
Minimum	14	0	10	7
Maximum	200	0	10	162
Pre-Disposal Conditions				
Minimum	20	0	10	12
Maximum	70	0	10	54
Post-Disposal Conditions				
Minimum	70	130	10	150
Maximum	20	200	10	161

Assumptions:
Moist Density of soil = 120 lb/ft^3
Moist Density of landfill waste = 107 lb/ft^3
Density of groundwater = 62.4 lb/ft^3

In addition to the staged laboratory permeability tests described above, laboratory permeability tests were also performed on several remolded core samples. The objective of the remold permeability tests was to develop a database to compare remold results to undisturbed results. The remold permeability test results are included in Table 4-21 along with the undisturbed permeability test results. This database will be used to calibrate remold results to undisturbed results if future permeability testing on stockpiled split-spoon (disturbed) samples collected from the NRC is deemed necessary; (that is, supplemental K_v data for the blue clay can be obtained from stockpiled split-spoon samples by remolding them and testing for K_v). It is important to note that, to date, there appears to be sufficient undisturbed K_v data to adequately characterize the K_v of the blue clay, thus additional remold tests have not been run on stockpiled split-spoon sample material.

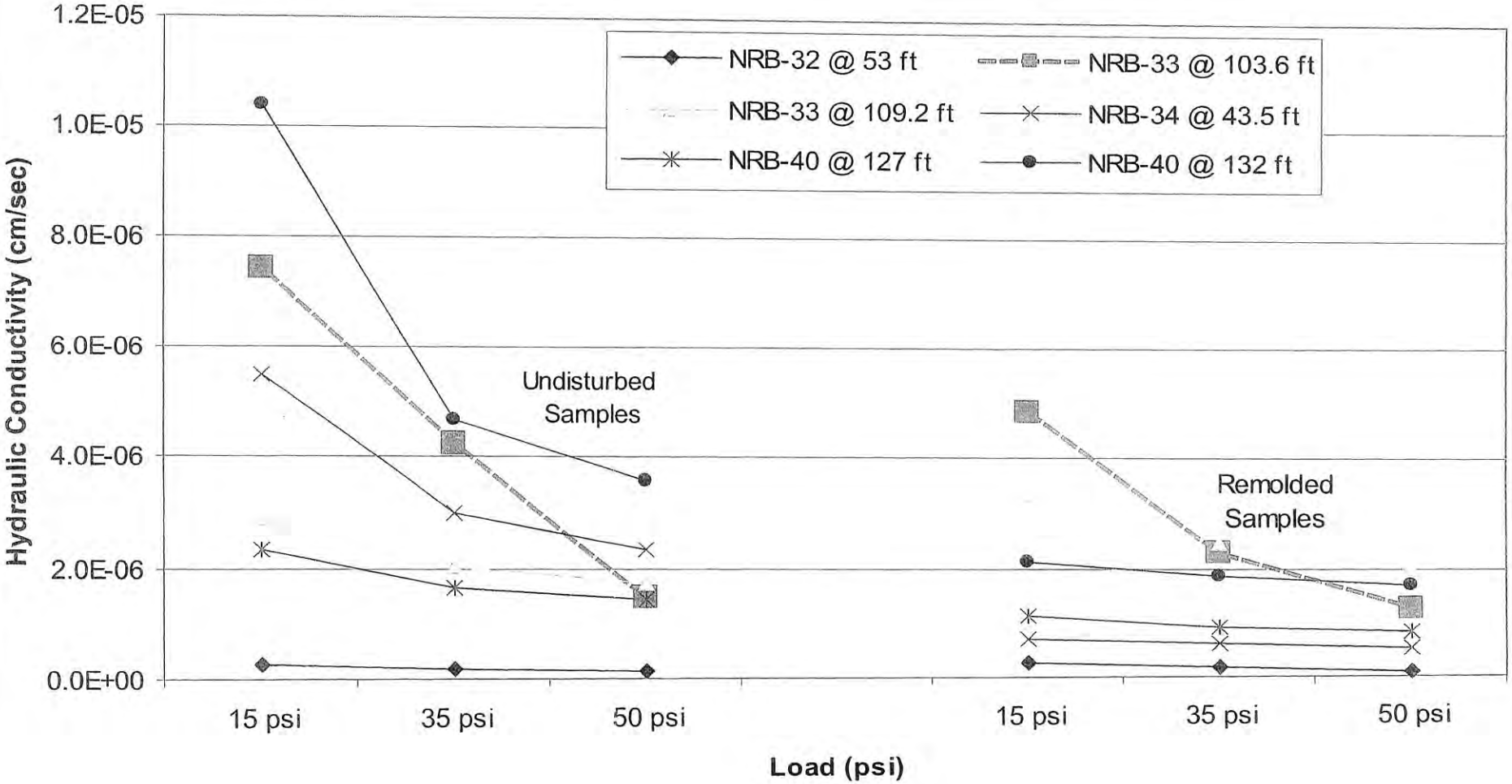


FIGURE 4-26
STAGED LABORATORY PERMEABILITY TESTS
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4.2.6.3 Groundwater Velocities and Travel Times

4.2.6.3.1 Horizontal Groundwater Flow Within the Uppermost Water-Bearing Unit. Horizontal groundwater velocities within the uppermost water-bearing unit were calculated based on the Darcy equation. The Darcy equation provides the following relationship for groundwater velocity:

$$v = KI / n_e \qquad \textit{Equation 2}$$

where

v= average linear velocity, ft/yr
K = Hydraulic Conductivity, ft/yr
I= hydraulic gradient, ft/yr
n_e = effective porosity of the media (dimensionless)

The rate of horizontal groundwater flow beneath the NRC is dependent on the hydraulic conductivity (K), hydraulic gradient (I), and effective porosity (n_e) of the uppermost water-bearing unit. Table 4-23 summarizes the hydraulic gradient, hydraulic conductivity, effective porosity, and the calculated flow velocities of the uppermost water-bearing unit. Note that in Table 4-23, the uppermost water-bearing unit is separated into two distinct physiographic areas: the area within the NRC footprint and the area downgradient of the NRC. The area downgradient of the NRC is further separated into two distinct sub-areas: an area west of the NRC (toward Highway 55 and the City of Eagle) and an area south of the NRC (toward Hill Road and the City of Boise). Each of these areas is shown in Figure 4-27. The downgradient areas to the west and south of the NRC are considered separately in Table 4-23 because available data indicate that the two areas have distinctly different groundwater regimes. Groundwater flow to the south of the NRC appears to be predominantly within higher-permeability coarse sands that overlie the blue clay confining zone, whereas groundwater flow to the west of the NRC appears to be to be predominantly within the lower-permeability fine sand and silt interbeds that overlie the blue clay. Furthermore, the area to the south is known to be impacted by the HHLF Cell and is being remediated and closely monitored.

In Table 4-23, the hydraulic gradient values are the average gradients observed in June, August, and December 2004, and the effective porosity is the average measured porosity (n) based on the laboratory tests provided in Appendix F. The hydraulic conductivity values are based on the aquifer test results presented in Section 4.2.5. The median K values were selected to represent the K for the designated sub-areas. The median is the number in the middle of a set of numbers (that is, half the numbers have values that are greater than the median, and half have values that are less). The median value for K was selected rather than the average, because a wide range of observed K values (up to three orders of magnitude) was measured in each downgradient sub-area. Had the average K-value been used for the analyses, this value would have been skewed by a single exceptionally high or low K-value.

TABLE 4-23
Groundwater Flow Information for the Uppermost Aquifer at the NRC

Location	Aquifer Test Location	Average Hydraulic Gradient (ft/ft)	Range in K (cm/sec)	Median K (cm/sec)	Effective Porosity*	Average Linear Velocity (ft/year)*
NRC	NRB-3, NRB-4, NRB-9, & NRB-13	0.0478	2.20E-03 to 5.70E-05	3.92E-04	0.35	55
Downgradient of NRC						
West of NRC	NRB-18, NRB-23, NRB-26, NRB-31, NRB-37	0.0674	2.43E-01 to 6.49E-05	7.80E-04	0.35	155
South of NRC	NRB-6, HHLF B-6, HHLF B-9, HHLF B-10, HHLF B-23	0.0511	1.34E-01 to 3.53E-04	5.31E-03	0.35	802

* The effective porosity was assumed to be equal to the average total porosity of the uppermost water-bearing unit. The average total porosity of the uppermost water-bearing zone was calculated from available analytical laboratory porosity measurements (Appendix F).

Stratigraphic units within the upper water-bearing unit are not laterally continuous as shown by the borehole log data and the type of depositional environment, where the coarse-grained higher K units typically pinch out and interfinger. Groundwater travel time will be controlled by the low-permeability units that surround and enclose the higher-permeability units. The available subsurface data indicate that an extensive, continuous high-permeability zone, such as a sandy stream-channel deposit, is not present at the site.

Based on the calculated groundwater velocities shown in Table 4-23, travel times within the uppermost water-bearing unit from the NRC to the nearest receptors can be estimated. West of the NRC, the nearest receptors are assumed to be the domestic wells in the Sage Acres Subdivision. The distance from the NRC to Sage Acres is approximately 4,000 feet. Given a groundwater velocity of 155 feet per year, a release from the NRC would take approximately 25.8 years to reach Sage Acres. It is important to note that wells in the Sage Acres subdivision are typically completed within the blue clay or overlying Quaternary-age alluvial fan material and not within the uppermost water-bearing unit of the Pierce Gulch sands.

South of the NRC, the nearest receptors are assumed to be the domestic wells along Hill Road. The distance from the NRC to Hill Road is 6,800 feet. Given a groundwater velocity of 802 feet per year, a release from the NRC would take approximately 8.5 years to reach Hill Road.

It is important to note that in the unlikely event of a release from the NRC, the release would first be detected by the compliance (sentinel) monitoring wells located along the downgradient edge of the landfill, as required in 40 CFR 258 Subpart E. Corrective action would then be initiated pursuant to 40 CFR 258 Subpart E to prevent contaminants from leaving the Ada County Landfill property. The downgradient travel times shown in Table 4-23 are for advective transport without remediation (after the contaminants would have been detected by the compliance wells). The travel times shown in Table 4-23 do not account for the mitigating processes of hydrodynamic dispersion or chemical retardation of

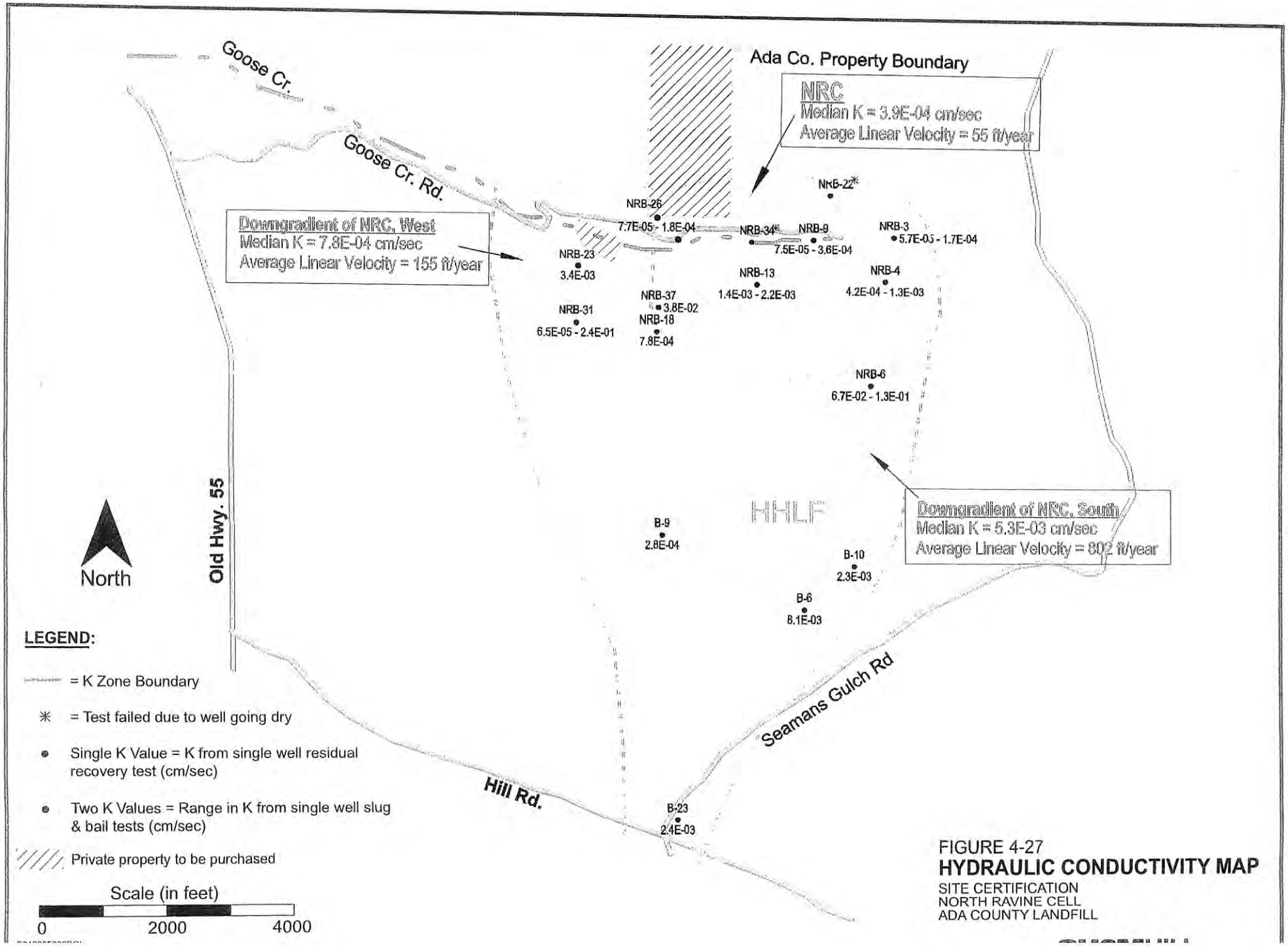

FIGURE 4-27
HYDRAULIC CONDUCTIVITY MAP
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the potential contaminants. Hydrodynamic dispersion is caused by thermo-kinetic diffusion and the transverse and longitudinal spreading (mechanical mixing) of the of the contaminant particles which results in the natural dilution of the solute. Contaminant retardation is caused by chemical interactions with geologic materials (phase partitioning). These interactions cause most dissolved solutes in groundwater to become dilute and to migrate at velocities that are substantially slower than the groundwater itself. Consequently, advection is generally not sufficient for predicting the migration of contaminants. Neglecting the effects of dispersion and retardation results in an overly pessimistic evaluation of the potential for contaminate transport. Thus, the travel times shown in Table 4-23 are conservative.

4.2.6.3.2 Groundwater Flow Within the Confining Blue Clays/Silts. Groundwater flow within the clays/silts of the confining zone includes two components of flow, vertical and horizontal flow. The vertical (downward) component is through the low permeable blue clays/silts that directly underlie the uppermost water-bearing unit. This rate of flow is based on the assumption that groundwater from the uppermost water-bearing unit will leak into the underlying confining clays/silts in accordance with the Darcy equation, Equation 2. Based on this equation, the vertical groundwater flow rate into the blue clay is dependent on the K_v and effective porosity of the blue clay and the vertical hydraulic gradient between the uppermost water-bearing unit and saturated sand seams within the blue clay. Table 4-24 summarizes vertical gradient, hydraulic conductivity, effective porosity, and the calculated flow velocities at three locations within the NRC study area.

The effective porosity shown in Table 4-24 is the calculated average measured porosity based on the laboratory tests provided in Appendix F. The hydraulic gradients in Table 4-24 were calculated from water level measurements collected from selected well groupings on February 1, 2005. The vertical distances between well pairs were calculated based on the difference between the mean screen or open hole elevations. Based on the well groupings listed in Table 4-24, a downward vertical hydraulic gradient is indicated for the blue clay directly beneath the NRC. Beneath the NRC, the downward hydraulic gradient was calculated to range from 0.31 ft/ft at the east side of the NRC (Sheriff's Shooting Range) to 0.11 ft/ft at the west side of the NRC (Terteling #2). This range in downward vertical hydraulic gradients is consistent with previous measurements at the HHLF Cell (CH2M HILL, 1996a). The resultant downward vertical groundwater velocities in the blue clay beneath the NRC are calculated to range from 1.9 to 4.7 ft/year.

West of the NRC, an upward vertical hydraulic gradient of 0.39 ft/ft was calculated at the Equestrian Well on February 1, 2005, indicating groundwater in the blue clay is under confined pressure at this location. The conclusion that groundwater at the Equestrian Well is under confining pressure is consistent with the original well driller's report on file at IDWR (Appendix E).

The trend in observed vertical hydraulic gradients across the NRC study area is illustrated in Figure 4-28. From east to west across the NRC (along the general groundwater flow path), the downward vertical hydraulic gradient decreases from 0.39 ft/ft at the Sheriff's Shooting Range to 0.11 ft/ft at Terteling #2. West of the NRC, the vertical hydraulic gradient switches to an upward gradient of 0.38 ft/ft at the Equestrian Well. These data suggest the upland portion of the Goose Creek drainage is a groundwater recharge area, whereas, the lowland portion of the Goose Creek drainage may be a groundwater discharge area.

TABLE 4-24
Vertical Groundwater Velocity Calculations for Confining Blue Clay/Silts

Deep Well	Shallow Well	Vertical Gradient (ft/ft)[a]	K_v (cm/sec)[b]	K_v (ft/year)[b]	Effective Porosity[c]	Avg. Linear Velocity (ft/year)[a]
Sheriff's Shooting Range Well	NRB-3/9/22	-0.31	7.0E-06	7.2	0.47	-4.7
Terteling Well 4	NRB-16/26/39	-0.11	7.0E-06	7.2	0.47	-1.9
Equestrian Well	NRB-32	+0.39	7.0E-06	7.2	0.47	+6.0

[a] A negative sign indicates a downward hydraulic gradient/velocity and a positive sign indicates an upward vertical gradient/velocity.
[b] Average laboratory K_v based on loading of 15 psi (see Table 4-21)
[c] Average total porosity of confining zone samples based on analytical laboratory measurements. The effective porosity was assumed to be equal to the total porosity.

The horizontal groundwater flow component within the low permeable blue clays/silts is based on the assumption that groundwater from the uppermost water-bearing unit will leak downward into the underlying confining clays/silts at the velocities shown in Table 4-24, and then, at depth, will move horizontally through the more permeable sand lenses. In accordance with Equation 2, the horizontal groundwater flow velocity within the sand seams is dependent on the hydraulic conductivity, effective porosity, and horizontal hydraulic gradient of the saturated sand seams. Table 4-25 summarizes horizontal gradient, hydraulic conductivity, effective porosity, and the calculated flow velocities for two flow paths; the flow path between the Sheriff's Shooting well and the Terteling #2 well (beneath the NRC footprint), and the flow path between the Terteling #2 well and the Equestrian well (downgradient of the NRC). The effective porosity shown in Table 4-25 is assumed based on literature values for fine sands (Dominico and Schwartz, 1990) and the hydraulic gradients were calculated from water level measurements collected from the deep wells on February 1, 2005. Based on this analysis, the deep groundwater in the sand seams (within the massive blue clays/silts) generally flows west at calculated average linear velocities in the range of 3.2 to 5.8 feet per year.

TABLE 4-25
Horizontal Groundwater Velocity Calculations for Confining Blue Clay/Silts

Zone	Horizontal Gradient[a] (ft/ft)	K (cm/sec)	K (ft/year)	Effective Porosity[b]	Avg. Linear Velocity (ft/year)
Sheriff's Shooting Range Well to Terteling #2 Well	0.014	7.6E-05	78.6	0.35	3.2
Terteling #2 Well to Equestrian Well	0.033	6.0E-05	62.1	0.35	5.8

[a] Note that the hydraulic gradient is based on the solution between two wells, this is only an estimate of the gradient.
[b] Average total porosity of confining zone samples based on analytical laboratory measurements. The effective porosity was assumed to be equal to the total porosity.

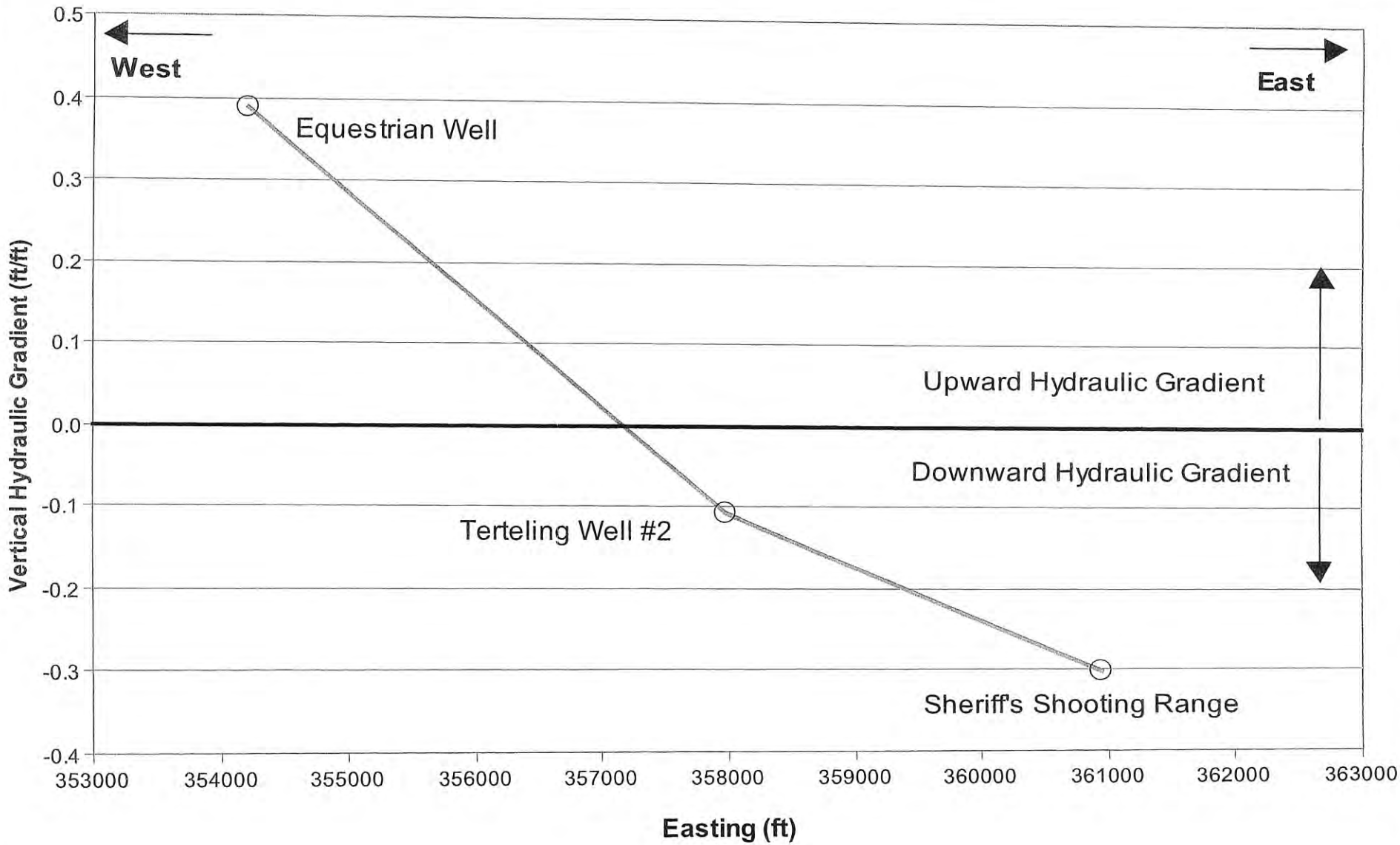

FIGURE 4-28
VERTICAL HYDRAULIC GRADIENT PROFILE
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Based on the calculated groundwater velocities shown in Table 4-25, travel times within the massive clay/silt unit from the NRC to the nearest receptors can be estimated. West of the NRC, the nearest receptors are again assumed to be the domestic wells in the Sage Acres Subdivision. The distance from the western edge of the NRC to Sage Acres is approximately 4,000 feet. Given a groundwater velocity of 5.8 feet per year, a release from the NRC would take approximately 690 years to reach Sage Acres, assuming the impacted groundwater could penetrate the confining blue clay/silt unit and intercept a continuous sand seam. Again, this travel time estimate is based solely on advective transport and does not account for the mitigating processes of hydrodynamic dispersion or chemical retardation of potential contaminants. Additionally, this travel time estimate does not account for the fact that a release from the landfill would be detected in the uppermost water-bearing unit compliance (sentinel) wells long before it could penetrate the confining clays/silts and reach a potential sand seam; therefore, corrective action would be initiated pursuant to 40 CFR 258 Subpart E to prevent contaminants from leaving the Ada County Landfill property.

4.3 Receptor Survey

A survey of the area around the NRC was conducted to identify wells that could be at risk because of their location with respect to the proposed cell. Figure 3-8 shows the sections that were surveyed along with the location of the proposed cell. Idaho Department of Water Resources files were searched to obtain all of the driller reports for each of the sections that are close to the proposed cell. Electronic copies of these well driller reports are included in Appendix E along with a summary of the wells by section. Some of these sections are upgradient of the proposed cell but were included because of the possible radius of influence from these wells. In all, there were 151 domestic, 5 irrigation, 54 monitoring, 4 industrial, 1 commercial, 1 test, 1 exploration, 1 livestock, 1 municipal, and 6 unknown wells. All of the monitoring wells that are in the area are associated with the existing HHLF and North Ravine Cells.

Based on the flow paths shown in Figures 4-21, 4-22, and 4-23, the wells within the Sage Acres subdivision are the closest potential receptors identified within the area of the receptor survey. As previously noted, the distance from the NRC to the Sage Acres subdivision is approximately 4,000 feet and the travel time, based on advective transport alone, is calculated to be 25.8 years. This travel time is the amount of time required for a contaminant to travel to Sage Acres after detection by the NRC Cell compliance (sentinel) wells. This travel time is conservative because it does not take into consideration the mitigating effects of chemical retardation and remediation. It is important to note that wells in the Sage Acres subdivision are typically completed within the blue clay or overlying Quaternary-age alluvial fan material and not within the uppermost water-bearing unit of the Pierce Gulch sands.

4.4 Soil Vapor Monitoring Results

Landfill gas migration north of the existing HHLF Cell was characterized as part of NRC site certification/characterization activities. The landfill gas data indicate that VOCs are present in soil gas at the southwestern portion of the NRC site and the area adjacent to the

northern/northeastern boundary of the HHLF Cell. Analytical laboratory reports for the vapor samples are included in Appendix H.

Three soil gas monitoring wells (M-5, M-6, and M-7) were installed in the vicinity of existing HHLF Cell methane monitoring well M-1 during first week of August 2004 (see Figure 3-7). After allowing approximately 8 weeks time for soil gas conditions to return to equilibrium following disturbance from well drilling and installation, soil gas samples were collected from methane monitoring wells M-1, M-5, M-6, and M-7. Soil gas samples were also collected from nearby groundwater monitoring wells NRB-15 and HHLF Cell well B-7. These ground water monitoring wells are screened above the water table allowing for the sampling of soil gas. The methane/fixed gas results for the October 6, 2004, sampling event are summarized in Table 4-26.

TABLE 4-26
Methane/Fixed Gas Concentration Results—October 6, 2004

Well ID	Methane Concentration (% by Volume)	CO_2 Percent by Volume	O_2 Percent by Volume
MW-1	7.7	7.3	0.3
MW-5	12.0	18.6	0.0
MW-6	Less than 0.1	0.0	18.1
MW-7	1.0	5.8	8.2
NRB-15	Less Than 0.1	0.0	16.9
HHLF Cell B-7	Less Than 0.1	0.0	17.0

The soil gas sample results indicate landfill gas has migrated from the existing HHLF Cell to the southern (adjacent) portion of the NRC. Because of the migration distance and low volume of gas, landfill gas VOCs emanating from the current HHLF Cell appear to decrease to very low concentrations within the NRC as exhibited at M-1, M-5, M-6, and M-7, NRC groundwater monitoring well NRB-15, and HHLF Cell groundwater detection monitoring well B-7 (see Table 4-27). The observed decrease in VOC concentrations associated with increased migration distance is consistent with diffusion controlled landfill gas migration and natural attenuation processes. Landfill gas VOCs and non-methane organic compounds (NMOCs) are attenuated and soil gas concentrations decrease as the distance increases from the source.

TABLE 4-27
Gas Concentration Results—October 6, 2004

CAS Number	Analyte Name	Laboratory Analytical Results (ppbv [parts per billion volume])						
		Soil Gas Monitoring Wells				Ground Water Monitoring Wells		
		M-1	M-5	M-6	M-7	NRB-15	HHLF B-7	Notes
75-71-8	Dichlorodifluoromethane	11000	11000	20	5700	230	170	—
74-87-3	Chloromethane	ND	ND	ND	ND	ND	ND	—
76-14-2	Freon 114	780	920	ND	240	11	8.0	—
75-01-4	Vinyl Chloride	71	570	ND	93	ND	ND	—
74-83-9	Bromomethane	ND	ND	ND	ND	ND	ND	—
75-00-3	Chloroethane	ND	37	ND	ND	ND	ND	—
75-69-4	Freon 11	2400	2700	12	910	18	14	—
156-59-2	cis-1,2-Dichloroethene	7.0	6.4	4.6	ND	5.0	ND	—
75-15-0	Carbon disulfide	17	21	16	16	18	9.9	—
76-13-1	Freon 113	500	580	4.0	140	7.1	4.8	—
67-64-1	Acetone	ND	ND	58	ND	60.	34	—
75-09-2	Methylene chloride	23	33	24	28	28	22	—
156-60-5	trans-1,2-dichloroethene	ND	7.0	ND	ND	ND	ND	—
75-34-3	1,1-Dichloroethane	ND	ND	ND	ND	ND	ND	—
108-05-4	Vinyl acetate	ND	ND	ND	ND	ND	ND	—
75-35-4	1,1-Dichloroethene	19	95	ND	28	ND	ND	—
78-93-3	2-Butanone	ND	ND	ND	ND	ND	ND	—
67-66-3	Chloroform	ND	ND	ND	ND	ND	ND	—
71-55-6	1,1,1-Trichloroethane	23	160	4.0	32	ND	ND	—
56-23-5	Carbon tetrachloride	ND	ND	ND	ND	ND	ND	—
71-43-2	Benzene	11	12	7.9	7.0	7.8	9.7	—
107-06-2	1,2-Dichloroethane	ND	ND	ND	ND	ND	ND	—
79-01-6	Trichloroethene	25	150	23	21	21	8.6	—
78-87-5	1,2-Dichloropropane	ND	ND	ND	ND	ND	ND	—
75-27-4	Bromodichloromethane	ND	ND	ND	ND	ND	ND	—
10061-01-5	cis-1,3-Dichloropropene	ND	ND	ND	ND	ND	ND	—
108-10-1	4-Methyl-2-pentanone	ND	ND	ND	ND	ND	ND	—
108-88-3	Toluene	170	ND	130	120	130	53	—
10061-02-6	trans-1,3-Dichloropropene	ND	ND	ND	ND	ND	ND	—

TABLE 4-27
Gas Concentration Results—October 6, 2004

CAS Number	Analyte Name	Laboratory Analytical Results (ppbv [parts per billion volume])						
		Soil Gas Monitoring Wells				Ground Water Monitoring Wells		
		M-1	M-5	M-6	M-7	NRB-15	HHLF B-7	Notes
79-00-5	1,1,2-Trichloroethane	ND	ND	ND	ND	ND	ND	—
127-18-4	Tetrachloroethene	56	510	95	130	78	27	—
591-78-6	2-Hexanone	ND	ND	ND	ND	ND	ND	—
124-48-1	Dibromochloromethane	ND	ND	ND	ND	ND	ND	—
106-93-4	1,2-Dibromoethane	ND	ND	ND	ND	ND	ND	—
108-90-7	Chlorobenzene	ND	ND	ND	ND	ND	ND	—
100-41-4	Ethylbenzene	31	28	37	22	30	14	—
136777-61-2	m,p-Xylene	41	35	55	34	42	30	—
95-47-6	o-Xylene	12	12	16	11	12	9.1	—
100-42-5	Styrene	4.5	4.5	4.8	4.0	4.2	ND	—
75-25-2	Bromoform	ND	ND	ND	ND	ND	ND	—
79-34-5	1,1,2,2-Tetrachloroethane	ND	ND	ND	ND	ND	ND	—
100-44-7	Benzyl chloride	ND	ND	ND	ND	ND	ND	—
622-96-8	4-Ethyl toluene	ND	ND	ND	ND	ND	ND	—
108-67-8	1,3,5-Trimethylbenzene	ND	ND	ND	ND	ND	ND	—
95-63-6	1,2,4-Trimethylbenzene	ND	ND	ND	ND	ND	ND	—
541-73-1	1,3-Dichlorobenzene	ND	ND	ND	ND	ND	ND	—
106-46-7	1,4-Dichlorobenzene	ND	ND	ND	ND	ND	ND	—
95-50-1	1,2-Dichlorobenzene	ND	ND	ND	ND	ND	ND	—
120-82-1	1,2,4-Trichlorobenzene	ND	ND	ND	ND	ND	ND	—
87-68-3	Hexachlorobutadiene	ND	ND	ND	ND	ND	ND	—
1634-04-4	Methyl t-butyl ether	ND	ND	ND	ND	ND	ND	—
	alpha-Pinene	160	160	240	150	11	69	J*
	Isobutane	1400	2000	—	—	—	—	J
	Butane	140	—	—	99	—	—	J
	Butane, 2-methyl-	420	480	—	390	—	—	J
109-66-0	Pentane	250	250	—	150	—	—	J
	Pentane, 2-methyl-	230	260	—	350	64	—	J
	Pentane, 3-methyl-	250	240	—	230	—	—	J

TABLE 4-27
Gas Concentration Results—October 6, 2004

CAS Number	Analyte Name	Laboratory Analytical Results (ppbv [parts per billion volume])						
		Soil Gas Monitoring Wells				Ground Water Monitoring Wells		Notes
		M-1	M-5	M-6	M-7	NRB-15	HHLF B-7	
	Cyclopentane, methyl-	280	290	—	260	—	—	J
	Hexane, 2-methyl-	220	210	—	110	—	—	J
	Pentane, 2,3-dimethyl-	280	270	—	160	—	—	J
	Hexane, 3-methyl-	350	270	—	210	—	—	J
	Heptane	300	250	70	5.6	68	—	J
	Cyclohexane, methyl-	520	540	81	110	87	—	J
	Hexane, 2,4-dimethyl-	160	190	—	80	—	—	J
	Pentane, 2,3,4-trimethyl-	550	610	—	230	—	—	J
	Hexane, 2,5-dimethyl-	280	—	—	—	—	—	J
	Heptane, 2-methyl-	—	280	—	—	—	—	J
	Heptane, 3-methyl-	210	180	—	75	—	—	J
	Hexane, 2,2,5-trimethyl-	640	—	—	270	100	—	J
	Hexane, 2,2,4-trimethyl-	—	640	100	—	—	—	J
	Cyclohexane, 1,2-dimethyl-	190	—	—	—	—	—	J
	Cyclohexane, 1,1,3-trimethyl-	240	310	53	130	53	—	J
	Cyclohexane, 1,2,4-trimethyl-	140	210	—	—	—	—	J
	Methane, dichloroflouro-	—	160	—	—	—	—	J
	Pentane, 2,3,3-trimethyl-	—	660	—	—	—	—	J
	Nonane	—	—	54	—	47	—	J
	C10 Terpene	—	—	110	73	83	—	J
	Camphene	—	—	76	—	210	—	J
	C10 Hydrocarbon	—	—	130	—	80	—	J
	Cyclohexane, 1,1,2,3-tetrameth	—	—	43	—	44	—	J
	C11 Hydrocarbon	—	—	60	74	41	—	J
	Cyclopentane, 1,2-dimethyl-	—	—	—	74	—	—	J
	1-Octene	—	—	—	91	—	—	J

* Indicates an estimated value between the method detection limit (MDL) and the practical quantification limit (PQL) for tentatively identified compounds.

To date, available data indicate that landfill gas migration through soils from the HHLF Cell north to the NRC site has not resulted in significant detectable concentrations of VOCs in groundwater beneath the NRC site. This is illustrated by the results of long-term sampling at HHLF Cell detection monitoring well B-7. HHLF Cell well B-7 is an upgradient detection monitoring well for the HHLF Cell and is located between the HHLF Cell and the NRC. HHLF Cell well B-7 was first sampled and tested in 1991 and has been sampled and tested quarterly or semiannually for groundwater VOCs since 1994. Ten years of quarterly/semiannual groundwater sample results for HHLF Cell well B-7 indicate groundwater VOCs are below detection (detection levels typically less than 1 part per billion [ppb]). The absence of detectable VOCs in groundwater at HHLF Cell well B-7 is consistent with the trace VOC concentrations detected in the soil gas.

It is important to note that the detection of VOCs in the soil gas within the NRC and the potential for existing trace concentrations of dissolved VOCs within the NRC as discussed in Section 4.2.5.2.3, indicates there is a potential for future landfill gas migration through soils from the HHLF Cell that may:

- Cause detectable groundwater VOC concentrations at the HHLF Cell upgradient detection monitoring well B-7.
- Cause detectable groundwater VOC concentrations between the NRC and HHLF Cell where NRC detection monitoring wells will be located.
- Cause groundwater VOC contamination beneath the NRC.

The soil gas levels observed are the result of 30 years of operations at the HHLF Cell. Since startup of the HHLF Cell landfill gas collection system in May 2004, the system appears to be collecting a significant portion of the landfill gas being generated by the HHLF Cell. It is anticipated that over time the landfill gas collection system will result in a significant decrease in groundwater VOCs beneath the HHLF Cell. Whether the existing landfill gas collection system is impacting the rate of migration of landfill gas through soils on the north side is presently unknown. Ada County is currently evaluating the possibility of expanding the landfill gas collection system to include the north side of the HHLF Cell. Furthermore, it is unknown whether the installation of the NRC liner will impede or facilitate the migration of VOC vapors into the area of the NRC because of the capping effect of the liner. Expansion of the landfill gas collection system to the north of the HHLF Cell is probably the best method to prevent further migration of soil gas into the NRC.

5.0 Location Acceptability Analysis

40 CFR 258 Subpart B includes location restrictions to address both the potential effects that a MSWLF unit may have on the surrounding environment, and the effects that natural and human-made conditions may have on the performance of the landfill unit. The location criteria of Subpart B cover the following:

- Airport Safety
- Floodplains
- Wetlands
- Fault Areas Seismic Impact Zones
- Unstable areas

The floodplain, fault area, seismic impact zone, and unstable area restrictions in 40 CFR 258 Subpart B address conditions that may have adverse effects on landfill performance and could lead to releases to the environment or disruptions of natural functions. Airport safety, floodplain, and wetlands criteria are intended to restrict MSWLF units where sensitive natural environments and/or the public may be adversely affected. The following discussion addresses each of the location criteria of 40 CFR 258 Subpart B and demonstrates that each of the criteria have been met with respect to the construction and operation of a MSWLF at the North Ravine site.

5.1 Airport Safety

The purpose of this analysis is to demonstrate that the NRC does not pose a bird hazard to aircraft pursuant to 40 CFR 258.10 which specifies a minimum distance to an airport runway of 10,000 feet for an airport used by turbojet aircraft or 5,000 feet from any airport used by piston-type aircraft. The nearest airports to the project site are the Boise Airport and the Idaho Air National Guard base at Gowen Field. These facilities are used by both piston and turbojet aircraft. The NRC site is more than 9.4 miles from either airport facility. Therefore, the proposed NRC site does not pose a bird hazard to aircraft.

5.2 Floodplains

The purpose of this analysis is to demonstrate that the NRC will not restrict the flow of a 100-year flood nor reduce the water storage capacity, and will not result in a wash-out of solid waste pursuant to 40 CFR 258.11.

Based on a review of electronic Federal Emergency Management Agency (FEMA) maps (www.HazardMaps.gov) no floodplains are located within the North Ravine site. Floodplains are located north and west of the site in Dry Creek and Dry Creek Valley. The vicinity of the NRC is classified as Zone X (www.fema.gov) which corresponds to: 1) areas outside the 100-year floodplains; 2) areas of 100-year sheet flow flooding where average depths are less than 1 foot; 3) areas of 100-year stream flooding where the contributing drainage area is less than 1 square mile; or 4) areas protected from the 100-year flood by levees.

5.3 Wetlands

The purpose of this analysis is to demonstrate that the NRC unit will not cause or contribute to a significant degradation of wetlands and that appropriate steps will be taken to achieve no net loss of wetlands pursuant to 40 CFR 258.12.

The NRC was evaluated for the presence of jurisdictional wetlands and the results of the study are presented in the report *Ada County Hidden Hollow Sanitary Landfill Expansion Project Wetland Delineation Report* (CH2M HILL, 2004a). A copy of this report is attached as Appendix D. In a letter dated July 20, 2004, the U. S. Army Corp of Engineers commented on the wetlands delineation report. In that letter, the Corps concluded ...."the proposed expansion of the Hidden Hollow Sanitary Landfill in Ada County, Idaho [the NRC] does not contain waters of the United States that are regulated under Section 404 of the Clean Water Act." Thus, the proposed NRC does not contain jurisdictional wetlands of the United States. A copy of the letter is included in Appendix D.

It is important to note that a wetlands mitigation plan will be developed for the non-jurisdictional wetlands at the NRC in accordance with Ada County requirements. The wetlands mitigation plan will be submitted to Ada County as part of the Conditional Use Permit for the NRC in accordance Ada County's planning and zoning requirements.

5.4 Fault Areas

The purpose of this analysis is to demonstrate that a Holocene fault is not present within 200 feet of the NRC pursuant to 40 CFR 258.13.

Based on the results of the fault investigation conducted at the NRC, no Holocene (active) faults were observed within 200 feet of the NRC footprint (Section 4.2.2.4). The Holocene Epoch is the most recent epoch of the Quaternary period, extending from the end of the Pleistocene Epoch to the present. The Pleistocene Epoch ended approximately 10,000 years ago; therefore, Holocene implies an age of less than 10,000 years.

The results of the fault investigation can be summarized as follows:

- No Holocene faults are shown in published literature or in the USGS fault index. Quaternary Fault and Fold Database maps by the USGS (2003) show no Holocene faults within 3,000 feet of the site. The Quaternary fault system closest to the North Ravine site is the Western Snake River Plain fault system, located approximately 10 miles south of the site. However, this fault system is much older than Holocene. The closest known Holocene-age fault is the Squaw Creek fault located approximately 13 miles northwest of the North Ravine site.
- No Holocene faults were identified on aerial photographs of the site.
- No Holocene faults were identified during geologic mapping and field reconnaissance.
- A fault investigation performed in an area of possible faulting west of the NRC footprint showed no evidence of Holocene faults. A fault interpreted to be pre-Holocene was observed in one of the exploratory trenches and is discussed in detail in Chapter 4, Section 4.2.3.

5.5 Seismic Impact Zone

The purpose of this analysis is to demonstrate that the structural components of the NRC are designed to resist the maximum horizontal acceleration in lithified earth material at the site pursuant to 40 CFR 258.14.

5.5.1 Predicted Ground Acceleration

The NRC site is located in a Seismic Impact Zone, defined by 40 CFR 258.14 as "an area with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years." "Lithified earth material" is defined as rock.

The maximum firm-rock ground acceleration with 10 percent or greater probability of exceedence in 250 years was determined from probabilistic ground motion values published by the USGS. The USGS presents ground motion data in terms of 2 percent probability of exceedence in 50 years. These are approximately equivalent to the 10 percent probability of exceedence in 250 years. The return periods for these two exceedence probabilities are 2,476 years and 2,373 years, respectively. Based upon typical relations between ground motion and return period, the corresponding ground motions should differ by 1 percent or less (USGS, 2004). The maximum firm-rock ground motions for the North Ravine site with 2 percent probability of exceedence in 50 years are (USGS, 2002):

- Peak ground acceleration —0.13g
- 0.2 second spectral acceleration—0.32g
- 1.0 second spectral acceleration—0.11g

This peak ground acceleration is greater than the regulatory threshold criterion of 0.10 g; therefore, the site is classified as a Seismic Impact Zone. As such, all proposed containment structures, including liners, leachate collection systems, and surface water control systems, are required to be designed to resist seismic ground motions. The final construction drawings and technical specifications will demonstrate seismic-resistant design.

5.5.2 Liquefaction Potential

An evaluation of the liquefaction potential of the site soils under current (in situ) conditions and future completed cell conditions is included in the site characterization and site certification processes for the North Ravine site. Liquefaction is a temporary loss of soil strength and stiffness because of the generation of high pore-water pressures during a seismic event. Liquefaction can lead to lateral spreading or flow failure. The purpose of this evaluation is to demonstrate that the site foundation is stable, in accordance with the requirements of the Idaho Solid Waste Facilities Act (Idaho Administrative Code [IAC] Title 39, Chapter 74), and to determine what engineering measures, if any, are needed to ensure the integrity of the facility. A detailed description of the analysis of the potential for liquefaction at the NRC is presented below.

5.5.2.1 Method of Analysis

This evaluation follows the procedure detailed in Section 5 of the U.S. EPA design guidance document titled *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities* by Richardson et al (1995). The evaluation method began with five initial

screening criteria. If at least three criteria were met, the liquefaction potential is considered to be small. If this initial screening indicates the potential for liquefaction exists, a more rigorous assessment is made using the SPT method of evaluation developed originally by Seed and Idriss (1982). Updates, modifications, and revisions by others which reflect current state-of-the-art understanding are incorporated into the assessment, as detailed by Richardson et al. (1995).

In this assessment, the site characteristics were evaluated using the five initial screening criteria. The more rigorous assessment was made for the non-clayey soils located below the existing groundwater table and above the blue clay unit. This more rigorous assessment was made for existing (in situ) conditions and for final completed cell conditions.

5.5.2.2 Design Criteria and Assumptions

Design criteria and assumptions were as follows:

- The design peak free-field acceleration is 0.18 g. Free-field acceleration is the amplified acceleration through the soil profile.
- A magnitude 7.1 seismic event was used in this analysis. The maximum credible earthquake magnitude was estimated using statistical correlations between fault length and magnitude by Bonilla et al (1984). The length of the Squaw Creek Fault (the longest Quaternary fault in the region) is 37 kilometers. Using an ordinary-least-squares regression model, Bonilla et al (1984) estimated the corresponding maximum credible earthquake magnitude is 7.1. In comparison, the 1983 Borah Peak Earthquake was magnitude 7.3.
- The groundwater levels used in the evaluation were measured on May 25, 2004, during the wet spring season for the site. Therefore, the groundwater levels used in the analysis were presumed to be at or very near the seasonal high. The site is on a local groundwater divide (in other words, a recharge area). After the cell is constructed, the groundwater levels are expected to drop because recharge will be restricted by the overlying lined cell. However, for the purposes of this analysis, these pre-development groundwater levels were assumed to remain unchanged throughout the life of the cell.
- To simplify the evaluation, a relative density of 50 percent was assumed for all soils with effective overburden stress greater than 12 kips per square foot (ksf) [one kip equals 1,000 pounds] and for all soils in future operations and final completion conditions. In these future scenarios, the change in relative density from the current (existing) relative density resulting from waste and fill placement cannot be accurately predicted. This is a very conservative assumption, because in reality these soils are at depths of 100 feet or more and are not expected to be loose.
- Although the evaluation examined all non-clayey soils below the groundwater table regardless of depth, soils deeper than 75 feet are not expected to liquefy. Moreover, soils deeper than 50 feet are generally not expected to liquefy. However, an alternate method was used to re-examine liquefaction potential if the calculated factor of safety against liquefaction initially approached 1.0.

5.5.2.3 Discussion

The initial screening indicates the potential for liquefaction is generally low to non-existent for the blue clay and the overlying sands. Detailed analysis was made of all other saturated, non-clayey soils where SPT data exists:

Existing (In Situ Pre-cell) Conditions. A rigorous assessment using the SPT procedure was made using all SPT data collected in sandy and silty (non-clay) soils regardless of depth. It clearly indicated that the potential for liquefaction exists at borehole NRB-16. Several other locations also indicated liquefaction potential, but these were at depths of 71, 111, and 161 feet. The SPT method only can be reliably applied when the depth is less than 75 feet (or less than 50 feet, according to Seed and Idriss [1982]), so the analyses in the marginal cases were checked using an alternate analysis method (a proprietary CH2M HILL spreadsheet-based program). The results of the analysis are provided in Appendix K.

This alternate method was used to check the calculated results for borings NRB-4, NRB-10, NRB-16, NRB-21, NRB-25, and NRB-28, and comparison of the results indicated that the original analysis results are reasonable and conservative. With the exception of NRB-16, the alternate method indicated that liquefaction is not expected to occur in any of the soil and subsurface conditions encountered in the exploration locations. In each case, the factor of safety against liquefaction calculated by the alternate method was greater than the factor of safety for the corresponding case in the original analysis. This leads to the conclusion that the original analyses are conservative. Much of this conservatism is likely attributable to the assumption of 50 percent relative density for the deeper soils and to the alternative method accounting for the soil classification and percent fines.

Finally, the conditions at NRB-16 were checked using the alternate method and it confirmed that there is potential for liquefaction at NRB-16 to a depth of approximately 29 feet.

Operations Conditions (End of Construction). Analysis of this scenario was not necessary because of the low probability that the design seismic event will occur after the cell area is excavated and lined, but prior to any waste placement. However, the scenario was analyzed to see if unloading and loading in cut and fill areas, respectively, of the cell would alter the areas that the In Situ Conditions analysis indicated were susceptible to liquefaction. The only additional location (in addition to those identified in the In Situ analysis) of liquefaction susceptibility identified is NRB-26.

It was not possible to use the alternate method of analysis to check the conditions after cut and fill because the change in relative density of the soil cannot be predicted accurately. In addition, as relative density changes, so will the SPT blow counts. It is not possible to predict the magnitude of the blow count change. Consequently, the conservatism confirmed for the In Situ Conditions analyses is reasonably extended to the Operations Conditions as well.

Final Completed Cell Conditions. The final completed cell conditions were analyzed using the same SPT-based method. Results of the analysis indicate the same areas of initial concern as the In Situ Conditions analysis. Discounting soils located more than 100 feet below existing ground surface, only the soils at borehole NRB-16 were again identified as potentially liquefiable.

Checking with the alternate method of analysis was not feasible for these conditions (as explained above), but the conservatism seen in the In Situ Conditions analysis can be reasonably extended and assumed to apply to the Final Completed Cell Conditions.

5.5.2.4 Liquefaction Summary and Conclusions

Overall, the site has very low liquefaction potential. The potential for loose, saturated cohesionless soils which may exist at depths greater than 100 feet to liquefy during the design seismic event is considered to be very low.

There is risk of liquefaction, however, in the vicinity of borehole NRB-16. Borehole NRB-16 is located in the bottom wetlands of the Goose Creek drainage and has loose, saturated soils in the upper 30 feet (approximately). Some of this loose soil will be overexcavated and replaced with compacted backfill during the wetlands mitigation. In addition, the groundwater from the wetlands area will be collected and piped further to the west to the new wetlands area which will be constructed for the mitigation. Any remaining loose, saturated soils in the NRB-16 vicinity will require densification, overexcavation, replacement with compacted fill, or drainage to improve the stability of the soils.

Because it was not feasible to explore and test all subsurface soils in all potential wetlands areas, it is reasonable to extrapolate the conclusions for the borehole NRB-16 soil profile to other similar soil profiles within the site. Therefore, all loose, saturated soils along the bottom of the Goose Creek drainage are potentially liquefiable and all should be stabilized using one or more of the methods noted above.

If isolated pockets of liquefaction-susceptible soils remain after soil improvement efforts, they are not expected to create instability of the completed landfill because they will be very limited in extent. The overall landfill footprint is very large and the compacted, compressed waste is expected to behave as a large, dense mass that can temporarily bridge over isolated pockets of liquefied soils until the excess pore pressures dissipate and soil strength is regained.

The soils located above the groundwater table and the saturated clays at the site are not susceptible to liquefaction. Also, the wetlands mitigation conceptual plan calls for collecting subsurface water beneath the NRC using a French drain collection system (gravel-filled trench) and piping the collected water to the mitigation area. This drain should lower the water table in the existing wetland areas along the Goose Creek drainage. The remaining saturated sands at the wetlands site require overexcavation, compaction, vibro-densification, or drainage to eliminate the liquefaction potential. A geotechnical engineer should oversee this work to ensure the depth and areal extent of soil improvement is sufficient. This work can be phased into the overall site development plan and does not need to be implemented until the staged expansion of the landfill cell reaches the wetlands.

5.5.3 Geologic Structure

The NRC site is underlain by unconsolidated geologic deposits. Therefore, the only geologic structures that would limit location acceptability are Holocene (active) faults. The potential for Holocene faults was investigated in detail, and the results are described in Sections 4.2.3 and 5.4. No Holocene faults were observed during the site investigation and literature review.

5.6 Unstable Areas

The purpose of this analysis is to demonstrate the structural integrity of the NRC unit pursuant to 40 CFR 258.15.

5.6.1 Slope Stability Analysis

Analyses of potential slope failure on circular and planar (block sliding) surfaces have been performed in support of the site characterization effort for the NRC. The purpose of this analysis is to demonstrate the site is stable prior to waste placement, during operations, and after final closure. The analysis evaluated veneer, block (planar), and circular sliding surfaces.

This narrative summarizes the geotechnical design criteria, material properties, analyses, and conclusions of the slope stability evaluations. The potential impacts of seismic loading on stability were evaluated. The results of these analyses demonstrate that the liner system and leachate collection and removal system (LCRS) will be stable under seismic loading (40 CFR 258.15).

5.6.1.1 Conceptual Design Criteria and Basis

The conceptual lining system for the side slope is comprised of, in ascending order, a reinforced geosynthetic clay liner (GCL) with non-woven geotextiles on both sides, a textured (both sides) high-density polyethylene (HDPE) geomembrane, and an 18-inch layer of drain sand.

The slope stability evaluation examines both veneer stability and global stability. The veneer stability evaluation is an analysis of potential sliding of the drain sand layer on the liner system, sliding between components of the liner system, and sliding of the liner system on the sloped subgrade before any waste is placed. The global stability evaluation is an analysis of potential sliding of waste on a failure surface passing through the waste, along the lining system, through native subgrade soils, or a combination of the three. The minimum required factors of safety against slope failure for the various cases and conditions analyzed are listed in Table 5-1 following the guidance of Richardson et al (1995).

TABLE 5-1
Stability Criteria

Case	Required Factor of Safety
Veneer Stability	
Dead load (self-weight of lining system)	1.5
Dead load plus equipment live load	1.3
Dead load plus seepage in the drain layer	1.3
Dead load plus equipment live load plus seepage in the drain layer	1.2
Global Stability	
Static, with bottom liner peak strength and side slope liner residual strength	1.5
Static, with residual strengths only	1.0 to 1.2*
Seismic (pseudo-static), with bottom liner peak strength and side slope liner residual strength	1.3
Seismic (pseudo-static), with residual strengths only	1.0

* The factor of safety for residual strength analyses should be in excess of 1.0 as recommended by Gilbert (2001). In addition, Filz et al (2001) recommend the typical minimum required factor of safety (FS = 1.5) be reduced using a factor of 0.82, which is based on peak GCL internal and residual GCL internal strengths. The adjusted minimum required factor of safety is 1.2.

Only static veneer stability analysis is warranted for the liner system on the NRC side slopes because of the low probability of the design earthquake occurring during the relatively short period the liner system is exposed. For global stability of the completed landfill, both static and seismic (pseudo-static) conditions were analyzed. Conditions applied to the evaluation of material properties are as follows:

- The typical bottom liner side slope is no steeper than 3:1 (horizontal : vertical).
- The maximum uninterrupted bottom liner side slope height is 100 feet with a slope length of 350 feet (conservative).
- The final cover slopes are 5:1.
- The maximum uninterrupted final cover slope height is 210 feet, with a maximum cover elevation of 3,250 feet.
- A pseudo-static coefficient of 0.09 was selected for the seismic stability analyses. This value is two-thirds of 0.13 g, the site peak ground acceleration with 2 percent probability of exceedence in 50 years. The basis for the site peak ground acceleration value is described in Section 5.5.

5.6.1.2 Material Properties

All analyses were performed using total strength parameters. Unique material properties were selected for each type of analysis. This is appropriate because the range of normal stresses that will be experienced by the liner system before waste is placed (the veneer stability conditions) are substantially lower than those that will be imposed by the full-height landfill at completion (the global stability conditions). Varying the shear strength according to normal stress enables modeling of the non-linear shape of the Mohr-Coulomb strength envelopes and selection of appropriate shear strength properties for the geosynthetic interfaces.

The weakest, or controlling, interface shear strength or material internal shear strength varies depending upon location and normal stress. Linear regression analysis of a CH2M HILL database of geosynthetic shear strengths was used in selecting the geosynthetic internal and interface properties. The geosynthetic interfaces considered were as follows:

- Textured HDPE geomembrane/drain sand
- GCL/textured HDPE geomembrane
- Native soil/GCL

The interface strengths between geosynthetics and soils were selected using engineering judgment. Additional components may be incorporated into the bottom liner system during the design phase, but the geosynthetic materials and interfaces listed above will govern stability.

Veneer Stability Analysis. In the veneer stability analyses, the interface and internal strength values were selected based on data for very low normal stresses, which are expected for the liner system prior to placing waste. Low normal stress for the veneer stability analysis was selected as less than 600 psf, which is equivalent to less than 5 or 6 feet of soil. Table 5-2 lists the veneer stability material properties used and considered in the analysis.

At the low confining pressures used in the veneer stability analysis, the relative effect of cohesion is large. Compare, for example, the textured HDPE/GCL residual interface strength to the GCL residual internal strength. At 600 psf overburden stress, phi of 30 degrees gives a shear strength of 346 psf; phi of 28 degrees plus 176 psf cohesion gives a shear strength of 495 psf. As the overburden stress decreases, the effect of the cohesion increases. Therefore the critical interface is the textured HDPE/GCL interface with a residual friction angle of 30 degrees.

TABLE 5-2
Material Properties Used and Considered in Veneer Stability Analyses

Material	Peak Interface Friction / Cohesion	Residual Interface Friction / Cohesion	Peak Internal Friction / Cohesion	Residual Internal Friction / Cohesion	γ_{moist} (pcf*)	γ_{sat} (pcf*)	Reference
Textured HDPE/drain sand at <600 psf normal stress	30° 0 psf	30° 0 psf	—	—	—	—	Engineering judgment based on drain sand internal friction
HDPE (textured)/GCL at <600 psf normal stress	46° 0 psf	30° 0 psf	—	—	—	—	CH2M HILL (2003)
GCL (needle-punched) at <600 psf normal stress	—	—	30° 280 psf	28° 176 psf	60	70	CH2M HILL (2003)
GCL/native sand subgrade	30° 0 psf	30° 0 psf	—	—	—	—	Engineering judgment based on native soil internal friction
Drain sand	—	—	34° 0 psf	34° 0 psf	120	130	Laboratory testing
In situ sand subgrade	—	—	34° 0 psf	34° 0 psf	120	130	Estimated from SPT

* pounds per cubic foot

Global Stability Analysis. The material properties considered and used in the global stability analysis are provided in Table 5-3. A division of strength properties at a normal stress of 2,000 psf was selected for the global stability analysis. To facilitate modeling of the entire landfill, shear strengths for normal stresses less than 2,000 psf were considered but not used because the vast majority of the liner system will experience normal stresses in excess of 2,000 psf. This simplifying assumption lends conservatism to the analysis.

TABLE 5-3
Material Properties Used and Considered in Global Stability Analyses

Material	Peak Interface Friction / Cohesion	Residual Interface Friction / Cohesion	Peak Internal Friction / Cohesion	Residual Internal Friction / Cohesion	γ_{moist} (pcf)	γ_{sat} (pcf)	Reference
Waste	—	—	20° 200 psf	20° 200 psf	107	107	Engineering judgment and CH2M HILL (2004b)
GCL/HDPE (textured)							
<2,000 psf normal stress	18° 262 psf	13° 135 psf	—	—	—	—	CH2M HILL (2003)
>2,000 psf normal stress	18° 612 psf	9° 409 psf	—	—	—	—	CH2M HILL (2003)
GCL (needle-punched)							
<2,000 psf normal stress	—	—	30° 280 psf	28° 176 psf	60	70	CH2M HILL (2003)
>2,000 psf normal stress	—	—	15° 1775 psf	6° 405 psf	60	70	CH2M HILL (2003)
GCL/Native Sand	30° 0 psf	30° 0 psf	—	—	—	—	Engineering judgment based on in situ sand internal friction
Textured HDPE/ Drain Sand	30° 0 psf	30° 0 psf	—	—	—	—	Engineering judgment based on drain sand internal friction
Drain Sand and Embankment Fill	—	—	34° 0 psf	34° 0 psf	120	125	Laboratory testing
In Situ Sand	—	—	34° 0 psf	34° 0 psf	120	125	Estimated from SPT
Blue Clay	—	—	31° 1,900 psf	31° 400 psf	120	125	Laboratory testing

The decision of whether to use peak shear strengths or residual shear strengths to analyze the stability of the landfill is complex. Gilbert (2001) and Thiel (2001) recommend analysis with properly selected residual strengths and designing for a factor of safety of at least 1.0. Filz et al (2001) note that the actual strength mobilized is less than peak, but somewhat greater than residual.

Under normal stress greater than 2,000 psf, the weakest peak strength is the HDPE/GCL interface strength and the weakest residual strength is the GCL internal strength. However, Gilbert states that the residual strength for the system should be the residual strength of the component with the lowest peak strength because the residual strength can only be mobilized if the peak strength is exceeded. Residual (large displacement) strengths were

used in the veneer analysis. Using the weakest residual strength for the entire system analysis is conservative.

5.6.1.3 Analyses

The NRC was analyzed for veneer and global stability. Veneer stability evaluates planar sliding conditions of the sides slopes prior to waste placement. Global stability evaluates the overall cell stability prior to waste placement, during operations when waste is placed within the cell without any buttressing at the toe of the waste slope, and after the landfill cell is completed with the final cover in place.

Veneer Stability. The veneer stability analysis evaluated failure planes along the side slope geosynthetics prior to waste placement. One section was analyzed for several loading conditions:

- Dead load
- Dead load plus equipment load
- Dead load plus seepage load
- Dead load plus equipment load plus seepage load

An equipment live load of 1,100 pounds per linear foot (7.6 psi) was selected based on typical tracked earthwork equipment ground pressures. A 9-inch saturated thickness within the drain sand was selected for the seepage load alone and a 9-inch saturated thickness when combined with the equipment load.

A modified sliding block method was used to analyze veneer stability. The modification accounts for the buttressing force applied by the drain sand layer at the slope toe. This method, by Druschel and Underwood (1993), has been incorporated into a proprietary spreadsheet-based program called SLOPBASE. Because the length of the slope section analyzed is much greater than the drain sand layer thickness, the relative effect of the toe buttress force is very small. A sensitivity run for a slope of near-infinite length demonstrated that the veneer stability factor of safety is applicable to slopes of 350 ft in length or greater.

Global Stability. The global stability analysis evaluated the landfill prior to waste placement (at the end of construction), during active waste placement (operations phase), and after waste placement (as a completed cell with final cover). The operations and final conditions are analyzed when waste heights are the greatest. Shearing can occur through any of the waste, lining, or foundation materials and along the bottom liner material interfaces. Sections analyzed for global stability are as follows:

- **End of Construction.** This analysis section (Appendix L) is located on the south side where the side slope height is the greatest. Sliding occurs towards the north. Sliding along the liner system is addressed for this condition by the veneer stability analysis.

- **Operations.** This analysis section (Appendix L) extends from the south side (where the side slope height is the greatest) to Goose Creek. Sliding occurs to the north.

- **Final.** This analysis section (Appendix L) is cut in the east-west direction, following the LCRS invert, with sliding occurring to the west.

Each of these cross-sections was analyzed for static conditions considering circular and sliding block failure surfaces. The operations and final configurations are also analyzed for seismic (pseudo-static) conditions, considering circular and sliding block failure surfaces. The end-of-construction case does not warrant a seismic analysis because the probability of the design seismic event occurring in this brief window of time is considered to be very low.

To aid in the analysis of slip surfaces along the bottom liner for global stability after waste placement begins, the bottom liner was modeled with a thickness of 4 feet. Cover materials were not modeled separately from the waste.

Global analyses were performed with the STABL5M software program in the STEDwin version 2.70 editor (Van Aller, 2002) using three methods of general limit equilibrium analysis: simplified Janbu, modified Bishop, and Spencer. The simplified Janbu method, used by STEDwin in computing factors of safety when searching for the critical failure surface in block sliding, is known to compute conservative (lower than actual) factors of safety because it assumes no inter-slice forces. Spencer's method, a method which considers vertical force, horizontal force, and moment equilibrium, was used to re-analyze a critical block sliding surface identified by the simplified Janbu method. The modified Bishop method, used by STEDwin to search for and analyze circular failure surfaces, is generally accurate and critical circular failure surfaces were not re-analyzed using Spencer's method.

A stepped approach was used in the global analysis. The initial analyses were performed with the weakest residual strength (the residual GCL internal shear strength) assigned to the entire liner system (sides and bottom). This conservative approach was sufficient in the analyses of all conditions except one—the factor of safety for the Operations Phase under seismic (pseudo-static) conditions in block failure mode did not meet the required minimum, so a more refined approach was used. Spencer's method was used to calculate a more accurate factor of safety, but in this particular case the computed factor of safety was still below the required minimum.

Block failure mode for the Operations Phase cross-section was then analyzed again for seismic conditions using a combination of strengths. The section was re-analyzed with the weakest peak shear strength (the HDPE/GCL peak shear strength) assigned to the bottom liner and the weakest residual shear strength (the residual GCL internal shear strength) assigned to the side slope liner. Research by Filz et al (2001) indicates that the lining system will experience differing shear displacements because of varying loadings, and strengths will vary between peak and residual along the potential failure surfaces. This combination of peak and residual strengths models strain-compatibility in the assigned strengths. By doing so, the calculated factor of safety increased from 1.0 to 1.8, which is much greater than the required minimum.

5.6.1.4 Conclusions

The NRC is expected to be stable at the end of construction, during operations, and after completion (final). The veneer stability analysis demonstrates that the side slopes of any length and slope grade of up to 33.3 percent (3:1) are stable. No anchorage forces in the geosynthetics are needed to achieve stability. The global stability analysis demonstrates the landfill is stable under static and seismic loading conditions. The key materials governing stability are the GCL and the HDPE geomembrane, and the critical failure surfaces are located within the waste and along the bottom liner.

The following should be considered to further enhance slope stability:

- Avoid horizontal GCL seams on the side slopes to the extent possible. Although typical GCL panels are manufactured in 150-foot lengths, at least one GCL manufacturer can produce panels up to 200 feet in length for no additional cost. In limited areas, a 200-foot panel will still be too short to avoid seams on the side slopes. Where seams are unavoidable, the seam should be a simple overlap, with the seam as close to the toe as possible. Overlapped panels should be shingled, with the upslope panel overlying the downslope panel.
- Although the veneer stability analysis indicates the liner system during construction is stable under the combined equipment live load and seepage conditions, equipment should avoid working directly on the lined slope during or immediately following rain or melting snow conditions if possible. After the infiltrated moisture has drained sufficiently such that the sand drain layer is no longer saturated, equipment tracking on the slope can resume.
- The waste should be placed in horizontal lifts, taking care not to damage the liner system.
- The veneer stability findings are for HDPE geomembrane textured on both sides. Smooth panel edges for ease of seaming are acceptable.

The critical failure surfaces and the corresponding factors of safety from the stability analyses are listed in Table 5-4. Copies of the STEDwin output plots and SLOPBASE spreadsheets are provided in Appendix L.

TABLE 5-4
Slope Stability Analyses Results

Case	Slip Surface	Analysis Method	Required Factor of Safety	Computed Factor of Safety
Veneer Stability–End of Construction				
Static Dead Load	Planar	Modified Sliding Block	1.5	1.7
Static Dead Load—Infinite Slope	Planar	Modified Sliding Block	1.5	1.7
Static Dead Load + Equipment Load	Planar	Modified Sliding Block	1.3	1.7
Dead Load + 9-inch Seepage Load	Planar	Modified Sliding Block	1.3	1.3
Dead Load + Equipment Load + 9-inch Seepage Load	Planar	Modified Sliding Block	1.2	1.3
Global Stability—End of Construction				
Static	Circular	Modified Bishop	1.5	2.4
Static	Block	Simplified Janbu	1.5	2.4
Global Stability—Operations				
Static with residual strength	Circular	Modified Bishop	1.0 to 1.2	1.8
Static with residual strength	Block	Simplified Janbu	1.0 to 1.2	1.7
Seismic with residual liner strength on side slope and peak liner strength on bottom	Block	Simplified Janbu	1.3	1.8
Seismic with residual strength	Circular	Modified Bishop	1.0	1.3

TABLE 5-4
Slope Stability Analyses Results

Case	Slip Surface	Analysis Method	Required Factor of Safety	Computed Factor of Safety
Global Stability—Final				
Static with residual strength	Circular	Modified Bishop	1.0 to 1.2	2.2
Static with residual strength	Block	Simplified Janbu	1.0 to 1.2	2.5
Seismic with residual strength	Circular	Modified Bishop	1.0	1.5
Seismic with residual strength	Block	Simplified Janbu	1.0	1.7

Note: Seismic analyses used a horizontal pseudo-static coefficient of 0.09

5.6.3 Foundation Assessment—Compressibility and Settlement

5.6.3.1 Introduction

The purpose of this assessment is to determine whether the North Ravine site is stable from a foundation compressibility perspective. (Ref. 40 CFR 258.15 and IAC Title 39, Chapter 74, Paragraph 39-7407.2.k) This analysis also estimates the magnitude of total and differential settlements that may occur and the impacts of those settlements on liner and leachate drainage systems.

The foundation conditions for the NRC site can be generally classified as dense sand overlying a continuous, massive, hard clay/silt layer, the surface of which can be approximated as a gently dipping plane. The existing ridges within the cell limits are comprised of dense sand and will be excavated during construction of the cell, leaving only a thin layer of dense sand overlying the hard clay/silt. The clay/silt unit is a lacustrine clay deposited in a reducing environment.

5.6.3.2 Method of Analysis

Consolidation tests were performed on undisturbed samples of the blue clay unit and the thin overlying brown clays and silts from the North Ravine site. A one-dimensional consolidation analysis was used to estimate the magnitude and rate of consolidation settlement at two locations: the center of the landfill and the location with the maximum waste thickness. Stress distribution was ignored because the load is applied over a very large area. The full waste load was applied.

Secondary compression (creep) settlement is typically very small in over-consolidated soils. It not expected to be large enough to be a concern considering the relatively short post-closure monitoring period. Therefore, creep magnitude was not estimated.

Immediate compression settlement was also estimated. Immediate settlement was estimated by linear strain analysis. This was added to consolidation settlement for a conservative estimate of the total settlement that could occur under the NRC. This total settlement magnitude was used to estimate the strain in the liner system and the impact on the slope of the leachate drainage systems.

5.6.3.3 Design Criteria and Assumptions

The following conservative criteria and assumptions were applied to the analysis:

- Average waste density of 107 pounds per cubic foot (pcf) was assumed for the completed cell, based on an estimate developed in a separate analysis.
- Average soil density of 125 pcf was assumed.
- The thickness of the blue clay unit is at least 600 feet. Although the actual layer thickness may be greater than 600 feet, 100 feet is a conservative assumption of the portion of the soil column that has soil-like compressibility properties. Deeper material (greater than 100 feet) is expected to exhibit relatively incompressible properties similar to shale (rock).
- Consolidation properties obtained through laboratory testing of the uppermost portions of the blue clay were assumed representative of the entire fine-grained soil column to a depth of approximately 100 feet.

5.6.3.4 Analyses

Consolidation parameters were selected for use in the analysis based on laboratory testing data on representative samples of the soils (see Table 5-5). Calculations of the 1-D consolidation magnitudes and time-rates were performed. Calculations were also performed to estimate immediate settlement, liner strain magnitudes, and slope change.

TABLE 5-5
Consolidation Parameters Used in Analysis

Parameter	Selected Analysis Value
Compression Ratio, $C_{c\varepsilon}$	0.13
Recompression Ratio, $C_{r\varepsilon}$	0.01
Coefficient of Consolidation, C_v (ft^2/day)	1.5
Effective Preconsolidation Pressure, P_c' (psf)	39,700

5.6.3.5 Discussion

The laboratory test results confirm the cohesive foundation soils are over-consolidated, with preconsolidation pressures ranging from 13 to 35 ksf, and OCRs ranging from 1.6 to 34). The field logs and SPT blow counts indicate that the blue clay is hard.

Much of the consolidation settlement that is expected to occur will be recompression. Virgin compression will occur only after the preconsolidation pressure is exceeded.

The consolidation settlement magnitudes estimated to occur in the foundation soils because of the weight of the waste in the completed landfill will be potentially up to 1.5 feet under the maximum waste height. From the time-rate analyses, the consolidation is expected to occur very rapidly. From laboratory testing performed on these soils, the soils are shown to have very low hydraulic conductivity and one would tend to assume the rate of consolidation would be very slow. Interestingly, the opposite appears to be the case. For

example, a 50-foot thick layer (a drainage path of 25 feet) appears to achieve 90 percent of primary consolidation in 1 year. Comparison to typical values of the coefficient of consolidation, however, indicates that the design value selected is reasonable.

The assumption that the blue clay consolidation test properties obtained from the uppermost zones of the blue clay unit are representative of blue clay at depths of up to 100 feet is conservative. At some depth, it is expected the blue clay will begin to behave less like compressible fine-grained soil and more like soft sedimentary rock (such as shale or claystone). Rock-like properties were observed in some shallow samples, and logs for deep wells completed at the site in the past by others occasionally describe the deeper blue clay material as rock.

Stress distribution is ignored for the settlement magnitude analyses. For foundation soils located close to the center of the landfill, the landfill plan area is sufficiently large that it approximates a load of infinite area. The full load of the waste column was applied to calculate settlement magnitudes. These magnitudes are conservative and can be considered as upper bounds which are not expected to be exceeded.

In the strain and slope change analyses, stress distribution was again conservatively ignored by assuming settlement at the edge of the landfill cell is zero. This gives very conservative estimates of liner strain and slope change. In reality, the foundation soils will feel varying percentages of the applied waste load depending on where the soils are located. Even soils close to but outside of the cell footprint are expected to experience some stress increase because of the waste load. These stresses will induce settlement in the foundation soils that will vary from a maximum directly under the maximum waste load near the cell center to zero some distance beyond the cell limits.

A conservative estimate of the total (consolidation plus immediate) settlement that may occur beneath the maximum waste load is 3.5 feet. When differential settlement is conservatively estimated to occur over a distance of 1,000 feet (from the cell edge to the cell center), the liner system strain created by this settlement is estimated to be less than a maximum of 0.01 percent. This is within the acceptable range of liner material strains. The estimated change in liner slope is 0.2 percent (from landfill edge to center along the east-west axis). This is an acceptable change.

5.6.3.6 Conclusions

Immediate and primary consolidation settlement properties of the site soils were evaluated based on laboratory testing of samples of site soils. These evaluations confirmed the field observations that the cohesionless soils are dense to very dense, and the cohesive soils are hard and overconsolidated. These evaluations also demonstrated that poor foundation conditions, highly compressible conditions, and the potential for large differential settlement do not exist. (40 CFR 258.15)

Total and differential settlement magnitudes are estimated and the resulting liner strain is very small. No adverse impact to liner materials is expected. The leachate drainage system slope will not be decreased by more than 0.2 percent. Therefore, the potential settlement impacts are manageable and reasonable.

5.6.4 Karst Terrane

The NRC site is not located within karst terrane. No physiographic features characteristic of karst terrane exist at the site (for example, such as sinkholes, sinking streams, caves, large springs, or blind valleys). No subsurface materials and/or geologic units characteristic of karst terrane exist at the site (for example, limestone, dolomite, or other soluble rock types).

5.6.5 Mass Movement and Geomorphology

Detailed geologic reconnaissance of the site by a registered professional geologists and engineers found no evidence of onsite or local geologic or geomorphologic features which could compromise the structural integrity of the proposed NRC. They found no evidence at the site of mass wasting or mass movement, including landslides, avalanches, debris slides, debris flows, soil fluction, block sliding, rock fall, or other features of structural instability in the site geology. Each of these phenomena is described, using definitions by the American Geological Institute (1984), and discussed as follows:

- A landslide is a general term describing a variety of processes involving downslope movement of soil and rock. The landslide potential is addressed in terms of slope stability in Section 5.6.1.
- An avalanche is a mass of snow, ice, soil, or rock, or a combination of these materials, falling, sliding, or flowing rapidly downslope. Snow accumulation at the site average less than 1 foot per year; therefore, the potential for avalanches is remote.
- A debris flow is a moving mass of rock, soil, and mud, with more than half of the particles larger than sand. Because the site soils are predominantly comprised of sand-size or smaller particles, the potential for debris flows is remote. A detailed analysis of potential soil slope failure is described in Section 5.6.1.
- Soil fluction, also termed solifluction, is the downward creep of waterlogged soils underlain by frozen, saturated ground (typically permafrost). The gravity-induced movement is described as flowing and slipping. Soil fluction will not occur at the site because of the absence of permafrost or other saturated frozen soils at the site. In addition, the free-draining characteristics of the surficial site sands generally prevent formation of a waterlogged soils.
- Block sliding is downslope movement of a generally intact soil and/or rock mass along a planar surface. Block sliding potential is discussed in greater detail in Section 5.6.1.
- Rock fall is the free falling of a newly detached mass of bedrock of any size from a cliff, steep slope, cave, or arch. Rock fall will not occur because there are no bedrock outcrops on site.

There no indication of these mass movement phenomena at the North Ravine site. There is also no indication of areas or soils which could be susceptible to these mass movement phenomena due to natural events which could reasonably be expected to occur during the design life of the proposed landfill. Although slope instability could be created by over-steepening cuts and embankments, such man-induced instability will be prevented by proper engineering design of slope cuts and embankment fill slopes.

5.7 Surface Evapotranspiration (ET) Soils

The purpose of this section is to demonstrate that an adequate volume of appropriate on-site materials to construct the final cover system is present at the site. Although not required under the Location Criteria of 40 CFR 258 Subpart B, a key site characteristic under the Closure and Post-Closure Care requirements of 40 CFR 258 Subpart F is the ability to install a final cover system designed to minimize infiltration and erosion upon site closure. Therefore, a description of the proposed final cover material is included in this Certification Report.

The proposed ET cover for the NRC is identical to the ET cover for the HHLF Cell as described in detail in the report *Evapotranspiration Cover Design Report* (CH2M HILL, 2002). The proposed NRC final cover system is an ET soil cover. An ET cover utilizes the water holding capacity of the soil to retain precipitation that has infiltrated the surface. The retained water is then removed through evaporation from the soil and transpiration through the cover vegetation. An ET cover utilizes soils with appropriate properties and sufficient depth, combined with native vegetative plant cover, to provide a system conforming to the alternate final cover design criteria cited in federal regulations (40 CFR 258.60(b)).

The proposed cover will be constructed of silty sand topsoil that will be stripped from the NRC site and stockpiled during initial construction. Available data indicate that approximately 2.4 million cubic yards of ET cover material are available onsite. This volume is more than adequate to provide a minimum of 4 feet of ET soil cover over the entire area of the landfill cell, adjacent roads, and staging areas.

Characterization testing indicates the on-site soil has the proper water-retention characteristics and soil properties to construct an ET cover in accordance with regulatory requirements. A properly designed ET cover constructed with the site topsoils will provide sufficient soil depth to store precipitation infiltration, significantly limit infiltration into the waste, and support a vigorous growth of native arid vegetation to prevent erosion and minimize long-term maintenance. It is expected that the cover will be constructed in phases as the landfill is filled and sufficient portions of surface area are finished to final grade.

6.0 Conclusions

6.1 Location Acceptability Analysis

The location restrictions of 40 CFR 258 Subpart B and Section 39-7407, Idaho Code (Location Restrictions—Site Certification) were evaluated to determine the potential effects that a MSWLF unit may have on the surrounding environment, and the effects that natural and human-made conditions may have on the performance of the landfill unit. The list of location restrictions under 40 CFR 258 Subpart B include:

- Airport Safety
- Floodplains
- Wetlands
- Fault Areas Seismic Impact Zones
- Unstable areas

Available data demonstrate that each of the restriction criteria have been met with respect to the construction and operation of a MSWLF at the North Ravine site. The floodplain, fault area, seismic impact zone, and unstable area restrictions of 40 CFR 258 Subpart B and Section 39-7407, Idaho Code were determined to have no potential adverse effects on landfill performance at the NRC. Available data indicate floodplain, fault area, and seismic impact zone conditions will not lead to releases to the environment or disruptions of natural functions. The only unstable condition identified in the study was the potential for liquefaction during a seismic event (earthquake) at NRB-16 and possibly other loose, saturated areas along the Goose Creek drainage bottom. This condition will be addressed in the design of the NRC. The anticipated corrective design will incorporate the removal of potentially unstable soils along the Goose Creek channel, backfilling and compaction with appropriate materials, and installation of a subsurface drain system to dewater shallow groundwater along the Goose Creek channel. Additional liquefaction mitigation measures will be employed to ensure the safety of the NRC, as necessary. Airport safety, floodplain, and wetlands criteria indicate that sensitive natural environments and/or the public will not be adversely affected by the construction and operation of a MSWLF at the North Ravine site.

6.2 Hydrogeologic Characterization

A comprehensive hydrogeological study was conducted at the NRC. Adequate site-specific data has been collected to determine the following:

- The lateral and vertical extent of the uppermost aquifer
- The lateral and vertical extent of the underlying confining layer
- The geology (for example, stratigraphy, lithology, and structural setting) of the NRC
- The chemical properties of the uppermost aquifer
- The location of potential downgradient receptor welds

- Groundwater flow, including:
  - The vertical and horizontal direction of groundwater flow in the uppermost aquifer
  - The hydraulic conductivities of the uppermost aquifer and it confining units
  - The average linear horizontal and vertical velocities of groundwater in the uppermost aquifer and the underlying confining zone

Adequate hydrogeological data has been collected at the NRC to prepare a comprehensive groundwater monitoring plan in accordance with 40 CFR 258 Subpart E and Sections 39-7409 (Standards for Design) and 39-7410 (Groundwater Monitoring Design), Idaho Code which establish the criteria for groundwater monitoring at all MSW landfills. The compiled data from the NRC will facilitate the installation and sampling of a representative detection (sentinel) monitoring well network, as follows:

- The installation of the appropriate number, spacing, and depths of detection groundwater monitoring wells to ensure early detection of a release. Early detection will allow sufficient time for corrective measures to be developed and implemented before sensitive receptors are impacted.
- The design and location of detection groundwater monitoring wells to address both existing conditions and anticipated facility development, as well as potential changes in groundwater flow in the uppermost water-bearing unit.
- The design and location of detection groundwater monitoring wells to detect natural water level fluctuations in the uppermost water bearing unit.
- The collection of background and downgradient groundwater samples from the uppermost aquifer.
- The expansion of the HHLF Cell landfill gas collection system to protect groundwater beneath the NRC.

It is important to note that the existing NRC predesign monitoring wells were specifically installed to conduct the NRC Site Certification study and <u>will not</u> be incorporated into the NRC compliance monitoring network. Instead, the existing NRC predesign monitoring wells will either be abandoned in accordance with Idaho Department of Water Resources rules and regulations or converted into water level piezometers for the NRC. The locations, construction details, and placement of the final NRC compliance monitoring network will be submitted in future NRC design documents in accordance with 40 CFR 258 Subparts D and E.

6.3 Geotechnical Characterization

The geotechnical characteristics of the NRC site were studied and evaluated through field exploration, soil sampling, field and laboratory testing, and engineering analysis. The geotechnical engineering evaluations conclude that a MSW landfill can be designed for the North Ravine site in accordance with the design criteria specified in 40 CFR 258 Subpart D and Idaho Code 39-7409 for:

- Composite liner
- Leachate collection system
- Point of compliance
- Leachate discharge

6.4 Statement of Certification

Pursuant to Section 39-7408, Idaho Code (Site Certification Procedure), this Site Certification Application has been prepared by Idaho qualified professionals to address each of the criteria set forth in Section 39-7407, Idaho Code (Location Restrictions—Site Certification).

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DEPARTMENT OF THE ARMY
WALLA WALLA DISTRICT, CORPS OF ENGINEERS
201 NORTH THIRD AVENUE
WALLA WALLA, WASHINGTON 99362-1876

REPLY TO
ATTENTION OF

July 20, 2004

Regulatory Division

SUBJECT: NWW No. 042100102

Mr. Dave Logan
Ada County
650 Main Street
Boise, Idaho 83702-5960

Dear Mr. Logan:

Our approved jurisdictional determination indicates that the North Ravine Cell project area for your proposed expansion of the Hidden Hollow Sanitary Land Fill in Ada County, Idaho does not contain waters of the United States that are regulated under Section 404 of the Clean Water Act. The boundary of your project is shown on Figure 2 of your wetland delineation report entitled "Ada County Hidden Hollow Sanitary Landfill Expansion Project", June 2004. This jurisdictional determination is valid for a period of 5 years from the date of this letter unless new information warrants revision of the determination before the expiration date.

We are enclosing an appeals form that explains the options you have if you do not agree with this approved jurisdictional determination. If you decide to appeal this determination, you need to send the form to the Division Engineer, Northwestern Division, so that he receives it within 60 days of this letter. If you have new information you want us to consider, you may send it to the Regulatory Division, Walla Walla District, at the letterhead address before you file the appeal.

If you have any questions, please contact Mr. Gregory Martinez at 208-345-2154. A copy of this letter is being sent to Mr. John Olson, Environmental Protection Agency, Idaho Operations Office, 1435 North Orchard Street, Boise, Idaho 83706; and Mr. Denny Mengel, CH2MHill, 700 Clearwater Lane, Boise, Idaho 83712.

Sincerely,

A. Bradley Daly
Chief, Regulatory Division

Enclosures

Delineation of
Waters of the United States

Upper North Ravine

Ada County, Idaho

February 9, 2012

Prepared for:
Dynamis Energy
776 E. Riverside Drive, Suite 150
Eagle, Idaho 83616
(208) 938-2684

RESOURCE SYSTEMS INC
CONSULTING ENGINEERS
BIOLOGISTS & SCIENTISTS
10306 Harvester Drive
Boise, Idaho 83709
(208) 376-5264

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Introduction

The delineation area is approximately 12 acres and encompasses a short 390-foot reach of the upper portion of a ravine called North Ravine, by landfill operators. The feature is unnamed on the USGS topographic map. The project vicinity map is Figure 1. The feature map is Figure 2.

Purpose

The purpose of this report is to delineate jurisdictional Waters of the United States (which includes wetland) as defined by the Clean Water Act and regulated by the U.S. Army Corps of Engineers for the discharge of fill. This delineation utilizes the Regional Supplement to the Corps of Engineers Wetland Delineation Manual Arid West Region (U.S. Army Corps of Engineers, 2008).

Evaluators

Karl Gebhardt - Hydrologist and Environmental Engineer. He has conducted water quality and wetland research, and has instructed courses on wetland hydrology throughout the western United States and has authored a number of technical publications on wetland management and classification. He has taken classes in wetland soil and soil-water physics and has worked closely with professional soil scientists, wetland ecologists, and botanists in the development of the Bureau of Land Management's Proper Functioning Condition procedures which he is an author.

Joyce Gebhardt - Certified Wildlife Biologist. She has completed the Interagency Wetland Delineation Training Course, and has worked on a number of wetland delineations in the Boise River system. She was a member of the team that conducted the Boise River Fish and Wildlife Study (Resource Systems, Inc. 1983) that served as the basis for the Boise River Plan.

Features and Concerns

The delineation area is approximately 12 acres (< 0.2 square miles) and encompasses a short 390-foot reach of the upper portion of a ravine called North Ravine, by landfill operators. The feature is unnamed on the USGS topographic map. The area is rangeland and is currently part of the Ada County Sanitary Landfill. Much of the site has been previously graded. The road currently blocks the ravine and as a result, no runoff moves downstream towards Goose Creek. This fact alone is probably sufficient to remove the area from consideration as a jurisdictional Water of the United States.

Methods

The routine determination described in Environmental Laboratory (1987) as modified by the U.S. Corps of Engineers (2008) was generally followed. The Ada County Soil Survey (Soil Conservation

Service 1980) was used to obtain general soil mapping boundaries.

Findings

Climate - Average annual precipitation in the area is about 12 inches, of which about 70 percent falls during the non-growing season resulting in the desert environment. The growing season is considered the end of April to early October (SCS, 1980).

WETS Station : BOISE WSFO AIRPORT, ID1022 Creation Date: 01/07/2003
Latitude: 4334 Longitude: 11613 Elevation: 02840
State FIPS/County(FIPS): 16001 County Name: Ada
Start yr. - 1971 End yr. - 2000

Month	Temperature (Degrees F.) avg daily max	avg daily min	avg	Precipitation (Inches) avg	30% chance will have less than	more than	avg # of days w/.1 or more	avg total snow fall
January	37.0	22.3	29.6	1.39	1.00	1.75	4	4.9
February	44.8	27.7	36.2	1.14	0.71	1.35	4	3.3
March	53.8	32.7	43.2	1.42	0.88	1.82	5	1.5
April	62.1	37.8	49.9	1.27	0.85	1.59	4	0.3
May	71.1	44.5	57.8	1.27	0.59	1.55	3	0.1
June	80.9	51.7	66.3	0.74	0.34	0.87	2	0.0
July	89.9	57.6	73.8	0.39	0.09	0.45	1	0.0
August	88.8	57.2	73.0	0.30	0.06	0.29	0	0.0
September	77.8	48.7	63.3	0.76	0.21	1.08	2	0.0
October	64.9	39.1	52.0	0.76	0.37	0.98	2	0.1
November	48.0	30.3	39.1	1.38	0.92	1.76	5	2.9
December	37.6	22.3	30.0	1.38	0.73	1.84	4	6.5
Annual	-----	-----	-----	------	10.76	13.35	--	----
Average	63.0	39.3	51.2	------	------	------	--	----
Total	-----	-----	-----	12.19	------	------	36	19.5

Source: http://www.wcc.nrcs.usda.gov/ftpref/support/climate/wetlands/id/16001.txt

Soils

Soils are mapped in SCS (1980), however, the data does not appear in the current online soil information website supported by the NRCS. The soil on the site is mapped as Payette-Quincy-, 30 to 65 percent slopes. This soil is a complex of Payette sandy loam and Quincy fine gravelly loamy coarse sand. The soil is not listed as a hydric soil. The soil has high permeability which is typical of granitic soil. Runoff from the area would be expected to quickly infiltrate into the soil where slopes are shallow as is seen upstream from the road crossing on the site.

Hydrology

The contributing watershed is quite small being less than 0.2 square miles, and therefore has not potential to provide any permanent water on site. Therefore, wetland hydrology is considered impossible to occur on the property under natural conditions. The drainage is considered ephemeral and would only have water during for short periods during precipitation and snowmelt events.

Vegetation

The uplands can be considered a 10 - 12 inch precipitation loamy/sandy range site whose potential natural community would consist of bluebunch wheatgrass, indian ricegrass, bitterbrush, sagebrush, among other typical rangeland vegetation. All of the vegetation on the property is considered upland.

Delineation Summary

There are no areas of potential wetland on the property.
The drainage is blocked from downstream movement and would most likely be considered non-jurisdictional by the U.S. Army Corps of Engineers.

References and Bibliography

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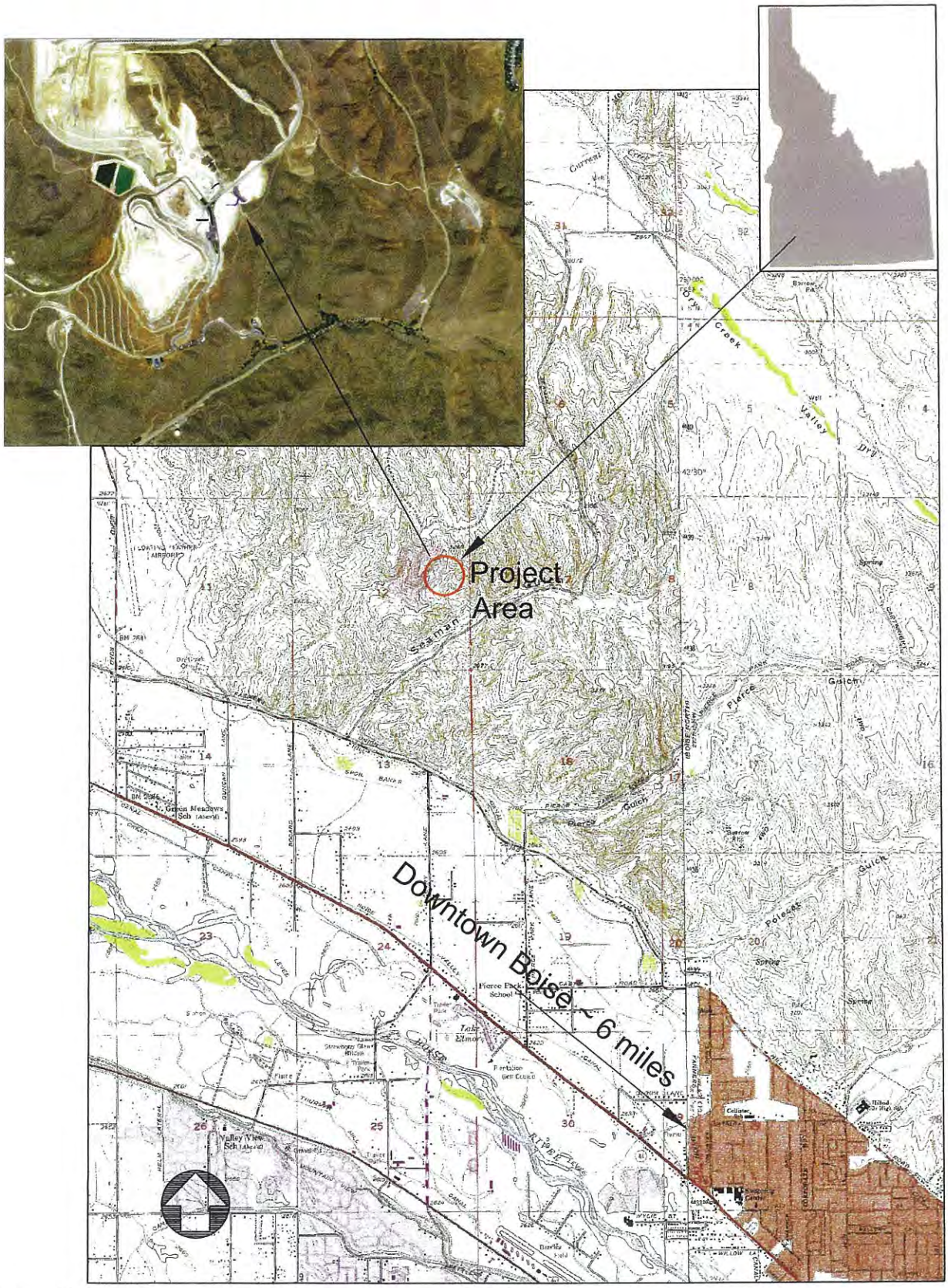
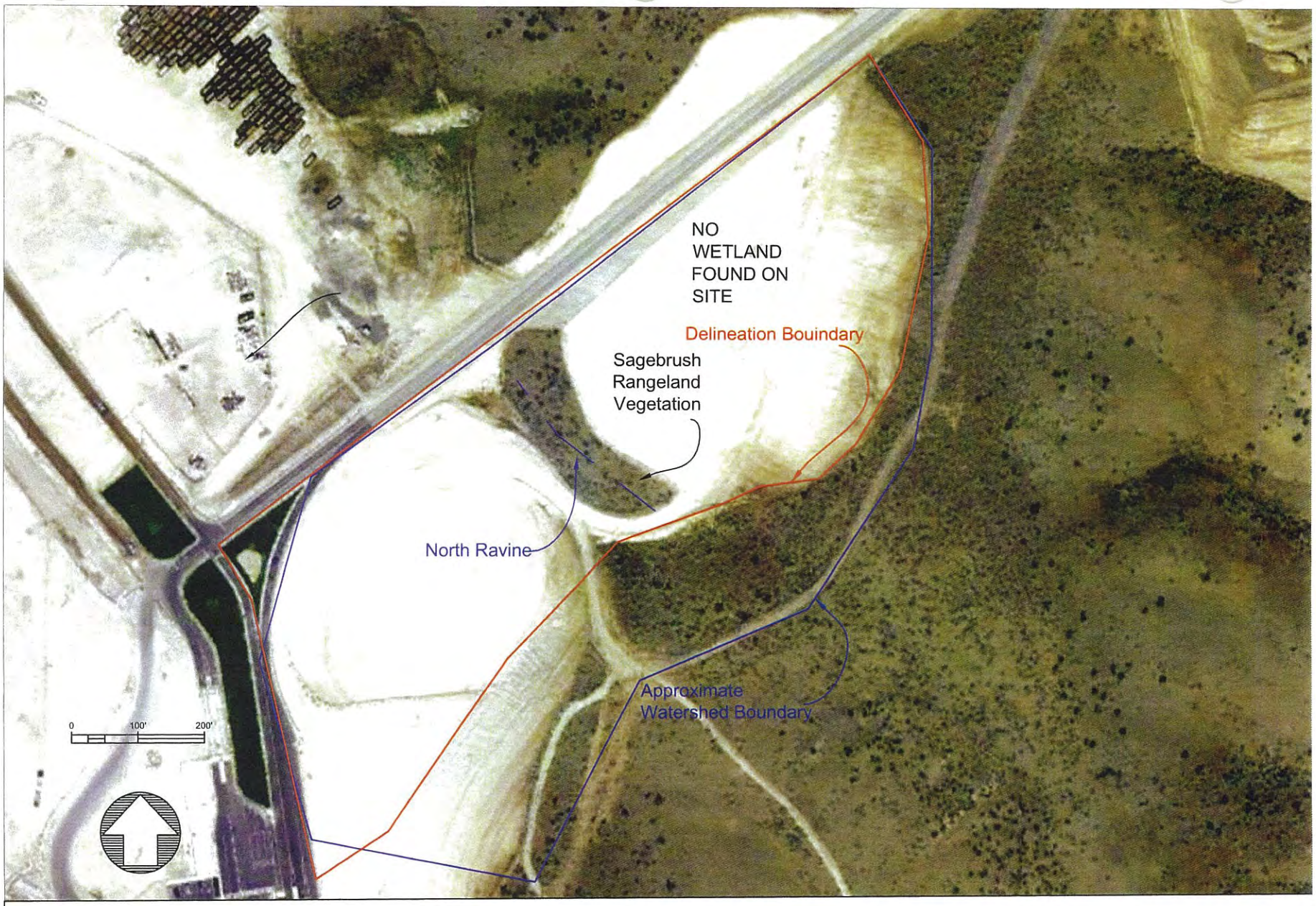


Prepared by:
Resource Systems, Inc.
Boise, Idaho

Dynamis Energy, LLC
Hidden Hollow Waste to Energy Plant
Vicinity

Figure 1



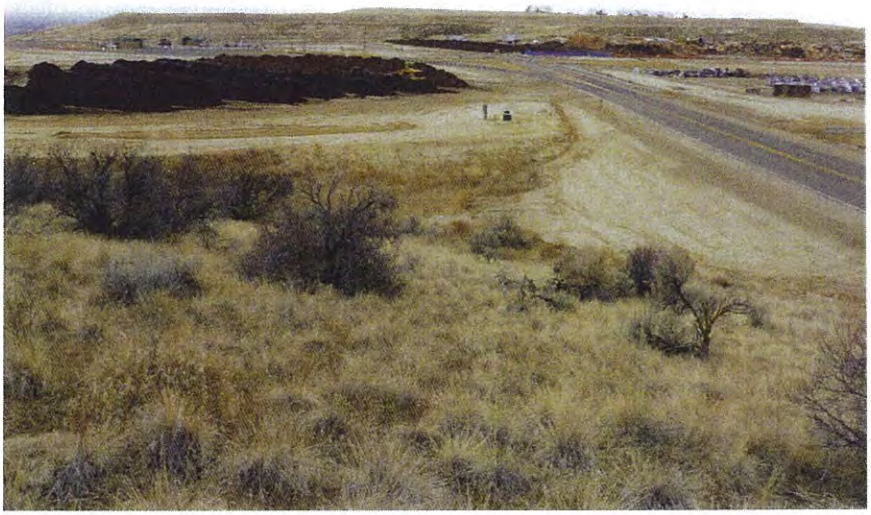
Prepared by:
Resource Systems, Inc.
Boise, Idaho

Dynamis Energy, LLC
Hidden Hollow Waste to Energy Plant
Wetland Delineation

Figure 2

Site Photos

1. Lower part of ravine at main road looking west.



2. Mid distance at ravine looking southwest.

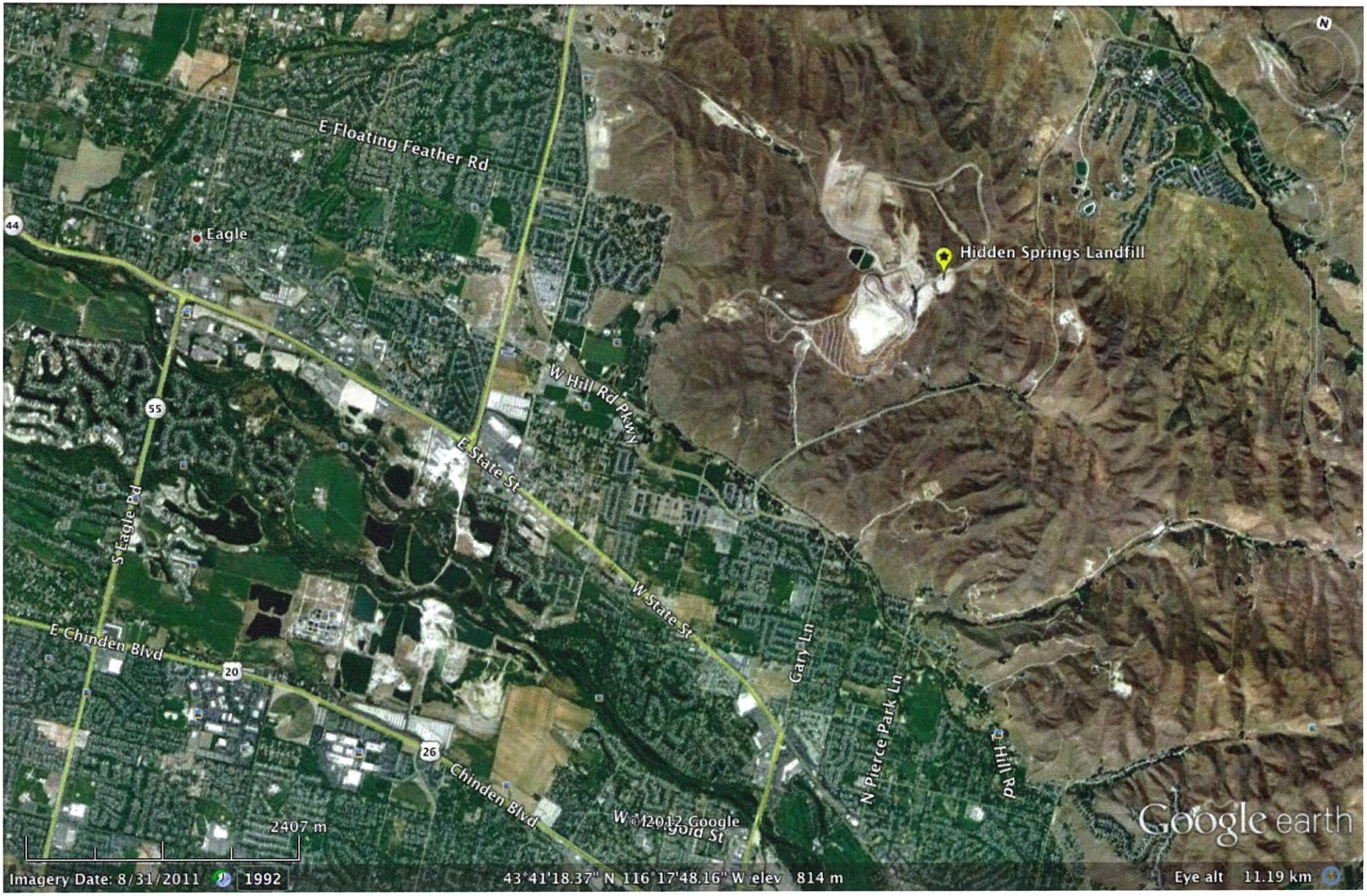
3. Ravine, looking west.

4 . Upper part of ravine looking south.

5 - Culvert upstream from main road.

6 - Area graded north of ravine.

TAB 10


E Floating Feather Rd
Eagle
Hidden Springs Landfill
W Hill Rd Pkwy
E State St
S Eagle Rd
W State St
E Chinden Blvd
Gary Ln
N Pierce Park Ln
E Hill Rd
Chinden Blvd
W Joid St
2407 m
© 2012 Google
Google earth
Imagery Date: 8/31/2011
1992
43°41'18.37" N 116°17'48.16" W elev 814 m
Eye alt 11.19 km

HIDDEN HOLLOW LANDFILL WASTE TO ENERGY PLANT

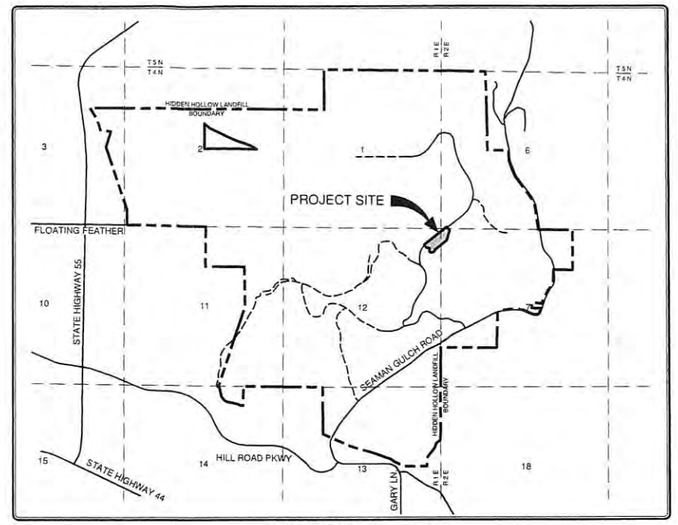
SITUATED WITHIN THE NORTHEAST QUARTER OF THE NORTHEAST QUARTER (NE4 NE4) OF SECTION 12, TOWNSHIP 4 NORTH, RANGE 1 EAST, BOISE MERIDIAN, AND GOVERNMENTLOT 1 OF SECTION 7 AND GOVERNMENT LOT 7 OF SECTION 6, TOWNSHIP 4 NORTH, RANGE 2 EAST, BOISE MERIDIAN, ADA COUNTY IDAHO

ADA COUNTY, IDAHO

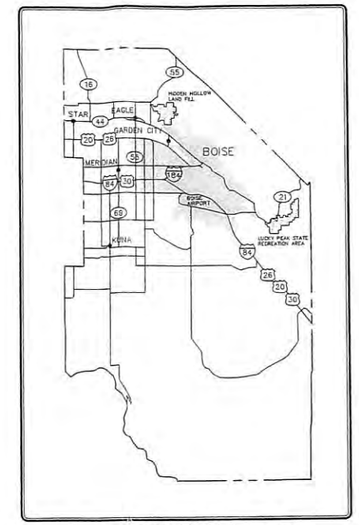
prepared by:

DYNAMIS ENERGY, LLC

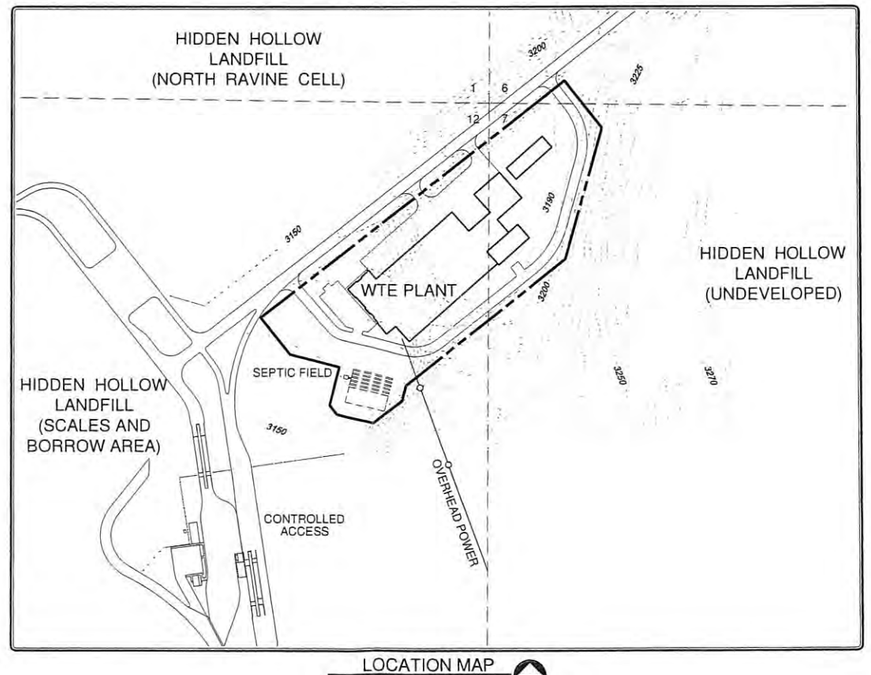
776 EAST RIVERSIDE DRIVE, SUITE 150
EAGLE, IDAHO 83616
CONTACT: DOYLE PERGANDE, P.E.
PHONE: (208) 938-2680



VICINTY MAP
1"=2000'



COUNTY MAP
NOT TO SCALE



LOCATION MAP
1"=150'

BASIS OF BEARING

THE WEST LINE OF THE NORTHWEST QUARTER OF SECTION 7, TOWNSHIP 4 NORTH, RANGE 2 EAST, BOISE MERIDIAN, BEING A FOUND A ⅝ INCH REBAR WITH CAP, MARKED PLS 5461, FROM WHICH A FOUND 3 ¼ INCH BRASS CAP MONUMENT MARKING THE WEST QUARTER CORNER OF SAID SECTION 7 BEARS SOUTH 02°13'17" WEST, 2611.35 FEET.

BASIS OF ELEVATION

HORIZONTAL DATUM: NAD83/99, IDAHO STATE PLANE COORDINATES, IDAHO WEST ZONE 1103, ADJUSTED FOR SCALE AND ELEVATION BY A COMBINED FACTOR OF 1.0001832259.

VERTICAL DATUM: NAVD88

UNITS: U.S. SURVEY FEET

CONTROL: N.G.S. STATION "IDTD", PID OH1488
N.G.S. STATION "BOIS A", PID AH9750

PER "SURVEY CONTROL DIAGRAM, HIDDEN HOLLOW LANDFILL" PREPARED BY DAVID S. SHORT, P.L.S. (2009).

SHEET INDEX

SHEET NO.	PAGE	SHEET NAME
C1	1 OF 6	COVER SHEET
C2	2 OF 6	TYPICAL SECTIONS AND LEGEND
C3	3 OF 6	DIMENSIONING PLAN
C4	4 OF 6	GRADING PLAN
C5	5 OF 6	UTILITY PLAN
C6	6 OF 6	SITE CROSS SECTIONS

PROFESSIONAL ENGINEER REGISTERED 14049 STATE OF IDAHO DOYLE E. PERGANDE 1-15-12

REVISIONS:

DATE: DECEMBER 27, 2011
DESIGNED BY: STAFF
DRAWN BY: STAFF
REVIEWED BY:
PROJECT:
SCALE:

COPYRIGHT ALL RIGHTS, LEVELS, FEATURES, AND PARTS, GRAPHICS, DESIGNS/DRAWINGS, LOCATIONS, DIMENSIONS AND PROCEDURES HEREIN ARE THE PROPERTY OF DYNAMIS ENERGY, LLC. ALL RIGHTS RESERVED. REPRODUCTION OR USE WITHOUT PERMISSION IS PROHIBITED.

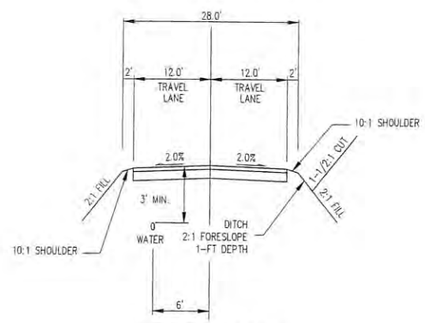
SEAL

DYNAMIS ENERGY, LLC
ADA COUNTY LANDFILL WASTE TO ENERGY PLANT
COVER SHEET

DYNAMIS ENERGY

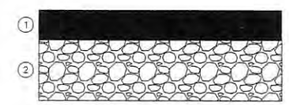
C1
SHEET 1 OF 6
V. 8 R._

path: C:\0-Scott\0dynamis Landfill\03_Design\ file name: WTE-COVER.dwg | plot date: December 27, 2011 | plotted by: sanders



ACCESS LOOP

N.T.S.



1. 4" ASPHALTIC TOP COURSE
2. 4" AGGREGATE BASE COURSE

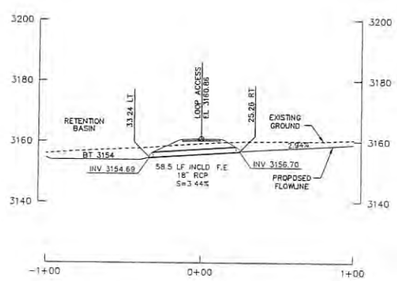
PER SITE SPECIFIC GEOTECHNICAL REPORT PREPARED BY STRATA, INC., DATED JANUARY 13, 2011

PAVEMENT SECTION

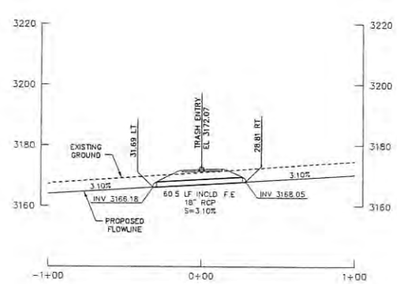
N.T.S.

LEGEND

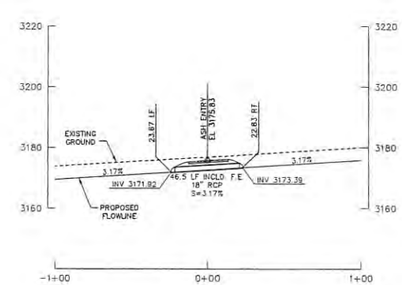
CENTERLINE	PROPOSED CURB AND GUTTER
RIGHT-OF-WAY	PROPOSED SANITARY SEWER
PROPERTY LINE	PROPOSED STORM DRAIN
EASEMENT	PROPOSED RETAINING WALL
EXISTING CURB AND GUTTER	PROPOSED WATER
EXISTING ELECTRICAL	SET NAIL
EXISTING GAS	SET REBAR
EXISTING SANITARY SEWER	SET CHISELED "X"
EXISTING STORM DRAIN	SET MONUMENT
EXISTING TELEPHONE	SET BRASS CAP IN HANDHOLE
EXISTING WATER	HORIZONTAL CONTROL POINT
EXISTING FENCE	PROPOSED MANHOLE
FOUND NAIL	PROPOSED STORM DRAIN
FOUND REBAR	PROPOSED CATCH BASIN WITH WING
FOUND CHISELED "X"	PROPOSED DOUBLE CHECK VALVE
FOUND BRASS CAP	PROPOSED FIRE HYDRANT
FOUND BRASS CAP IN HANDHOLE	PROPOSED WATER METER
EXISTING STREET LIGHT	PROPOSED WATER VALVE
EXISTING POWER POLE	PROPOSED AIR RELEASE VALVE
EXISTING MANHOLE	GRADE BREAK/GRADE CHANGE/PVI
EXISTING CLEANOUT	GRADE BREAK/GRADE CHANGE
EXISTING CATCH BASIN	CONCRETE
EXISTING CATCH BASIN	SAFETY POST
EXISTING DOUBLE CHECK VALVE	
EXISTING FIRE HYDRANT	



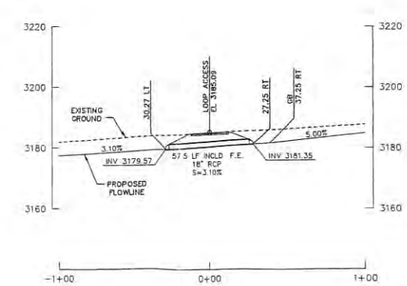
S1/C4 SOUTH LOOP ENTRY STORM CULVERT



S2/C4 TRASH ENTRY STORM CULVERT



S3/C4 ASH ENTRY STORM CULVERT



S4/C4 NORTH LOOP ENTRY STORM CULVERT

0 20 40 80 120
SCALE: 1"=40'
2X VERTICAL EXAGGERATION

PROFESSIONAL ENGINEER REGISTERED STATE OF IDAHO 07141 12/5/12 DALLE E. PERGANDE

SEAL

REVISIONS

DATE:	DECEMBER 27, 2011
DESIGNED BY:	STAFF
DRAWN BY:	STAFF
REVIEWED BY:	
PROJECT:	
CODE:	

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DYNAMIS ENERGY, LLC

ADA COUNTY LANDFILL WASTE TO ENERGY PLANT

DETAIL SHEET

TYPICAL SECTIONS AND LEGEND

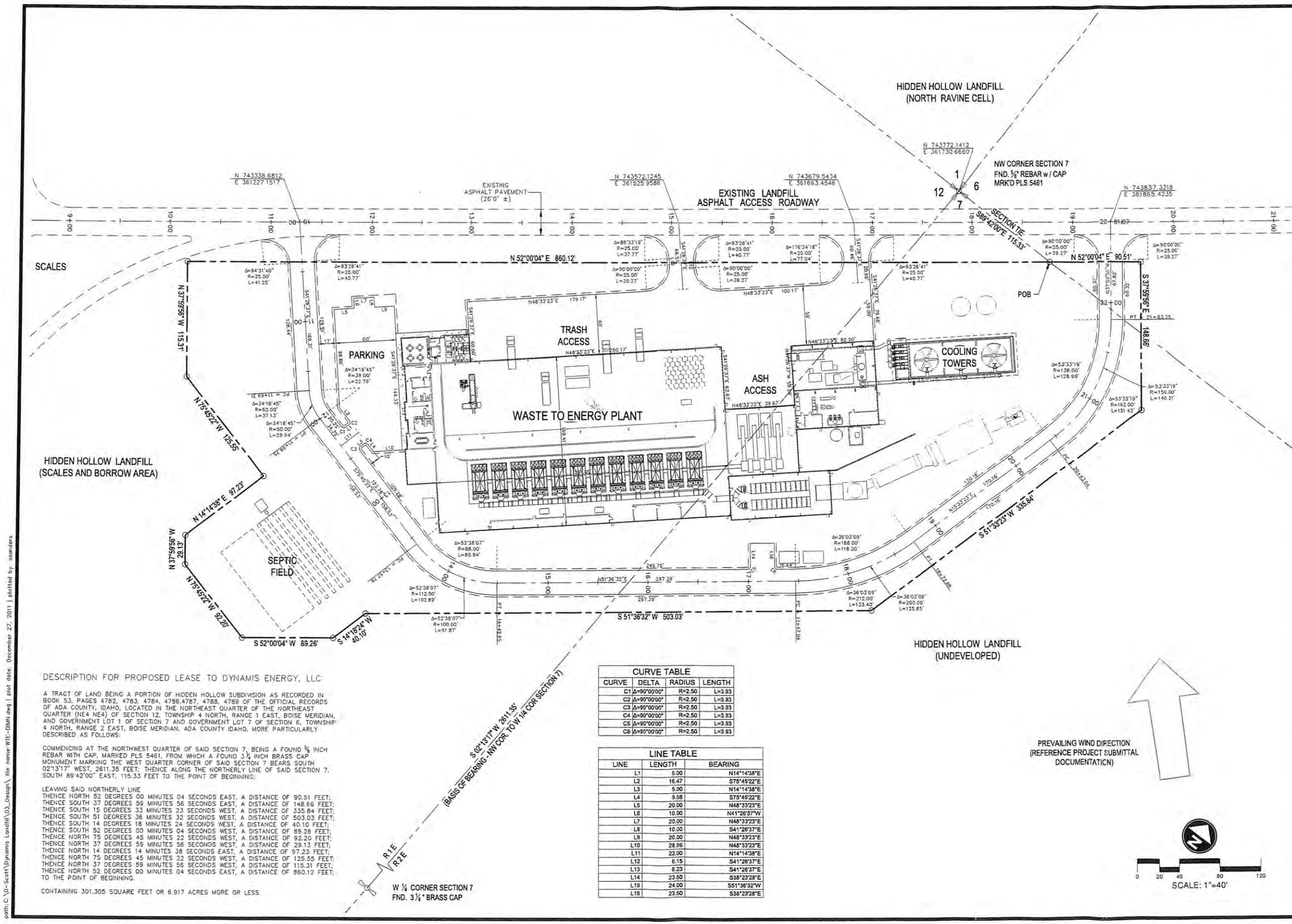
DYNAMIS ENERGY

C2

SHEET 2 OF 6

V. 8 R._

path: C:\0-Scott\Dynamis Landfill\03_Design\ file name: WTE-NOTES.dwg | plot date: December 27, 2011 | plotted by: ssanders



DESCRIPTION FOR PROPOSED LEASE TO DYNAMIS ENERGY, LLC

A TRACT OF LAND BEING A PORTION OF HIDDEN HOLLOW SUBDIVISION AS RECORDED IN BOOK 53, PAGES 4782, 4783, 4784, 4786,4787, 4788, 4789 OF THE OFFICIAL RECORDS OF ADA COUNTY, IDAHO, LOCATED IN THE NORTHEAST QUARTER OF THE NORTHEAST QUARTER (NE4 NE4) OF SECTION 12, TOWNSHIP 4 NORTH, RANGE 1 EAST, BOISE MERIDIAN, AND GOVERNMENT LOT 1 OF SECTION 7 AND GOVERNMENT LOT 7 OF SECTION 6, TOWNSHIP 4 NORTH, RANGE 2 EAST, BOISE MERIDIAN, ADA COUNTY IDAHO, MORE PARTICULARLY DESCRIBED AS FOLLOWS:

COMMENCING AT THE NORTHWEST QUARTER OF SAID SECTION 7, BEING A FOUND 5/8 INCH REBAR WITH CAP, MARKED PLS 5461, FROM WHICH A FOUND 3 1/4 INCH BRASS CAP MONUMENT MARKING THE WEST QUARTER CORNER OF SAID SECTION 7 BEARS SOUTH 02°13'17" WEST, 2611.35 FEET; THENCE ALONG THE NORTHERLY LINE OF SAID SECTION 7, SOUTH 89°42'00" EAST, 115.33 FEET TO THE POINT OF BEGINNING;

LEAVING SAID NORTHERLY LINE
THENCE NORTH 52 DEGREES 00 MINUTES 04 SECONDS EAST, A DISTANCE OF 90.51 FEET;
THENCE SOUTH 37 DEGREES 59 MINUTES 56 SECONDS EAST, A DISTANCE OF 148.66 FEET;
THENCE SOUTH 15 DEGREES 33 MINUTES 23 SECONDS WEST, A DISTANCE OF 335.84 FEET;
THENCE SOUTH 51 DEGREES 36 MINUTES 32 SECONDS WEST, A DISTANCE OF 503.03 FEET;
THENCE SOUTH 14 DEGREES 18 MINUTES 24 SECONDS WEST, A DISTANCE OF 40.10 FEET;
THENCE SOUTH 52 DEGREES 00 MINUTES 04 SECONDS WEST, A DISTANCE OF 89.26 FEET;
THENCE NORTH 75 DEGREES 45 MINUTES 22 SECONDS WEST, A DISTANCE OF 92.20 FEET;
THENCE NORTH 37 DEGREES 59 MINUTES 56 SECONDS WEST, A DISTANCE OF 29.13 FEET;
THENCE NORTH 14 DEGREES 14 MINUTES 38 SECONDS EAST, A DISTANCE OF 97.23 FEET;
THENCE NORTH 75 DEGREES 59 MINUTES 45 MINUTES 22 SECONDS WEST, A DISTANCE OF 125.55 FEET;
THENCE NORTH 37 DEGREES 59 MINUTES 56 SECONDS WEST, A DISTANCE OF 115.31 FEET;
THENCE NORTH 52 DEGREES 00 MINUTES 04 SECONDS EAST, A DISTANCE OF 860.12 FEET;
TO THE POINT OF BEGINNING.

CONTAINING 301,305 SQUARE FEET OR 6.917 ACRES MORE OR LESS.

CURVE TABLE

CURVE	DELTA	RADIUS	LENGTH
C1	Δ=90°00'00"	R=2.50	L=3.93
C2	Δ=90°00'00"	R=2.50	L=3.93
C3	Δ=90°00'00"	R=2.50	L=3.93
C4	Δ=90°00'00"	R=2.50	L=3.93
C5	Δ=90°00'00"	R=2.50	L=3.93
C6	Δ=90°00'00"	R=2.50	L=3.93

LINE TABLE

LINE	LENGTH	BEARING
L1	5.00	N14°14'38"E
L2	16.47	S75°45'22"E
L3	5.00	N14°14'38"E
L4	9.58	S75°45'22"E
L5	20.00	N48°33'23"E
L6	10.00	N41°26'37"W
L7	20.00	N48°33'23"E
L8	10.00	S41°26'37"E
L9	20.00	N48°33'23"E
L10	28.96	N48°33'23"E
L11	22.00	N14°14'38"E
L12	6.15	S41°26'37"E
L13	6.23	S41°26'37"E
L14	23.50	S38°23'28"E
L15	24.00	S51°36'32"W
L16	23.50	S38°23'28"E

PROFESSIONAL ENGINEER
STATE OF IDAHO
DOUGLAS E. PERGANDE
10/15/11

REVISIONS	
DATE:	DECEMBER 27, 2011
DESIGNED BY:	STAFF
DRAWN BY:	STAFF
REVIEWED BY:	
PROJECT:	
SCALE:	

SEAL

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DYNAMIS ENERGY, LLC

ADA COUNTY LANDFILL WASTE TO ENERGY PLANT

PLAN SHEET

DIMENSIONING PLAN

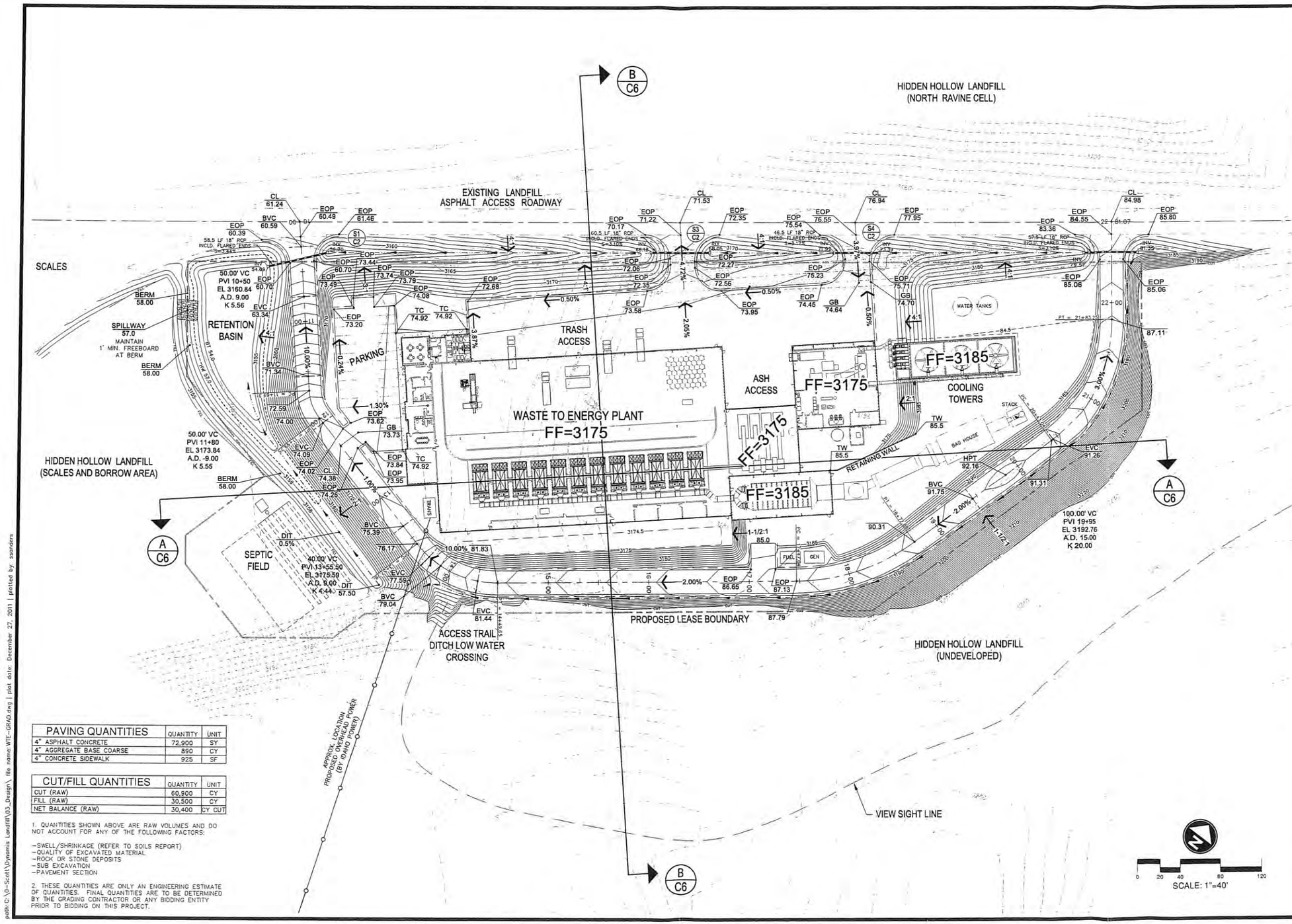
DYNAMIS ENERGY

C3

SHEET 3 OF 6

V. 8 R._

path: C:\JG-Scott\Dynamis Landfill\03_Design\ file name: WTE-DIMN.dwg | plot date: December 27, 2011 | plotted by: ssanders



PAVING QUANTITIES	QUANTITY	UNIT
4" ASPHALT CONCRETE	72,900	SY
4" AGGREGATE BASE COARSE	890	CY
4" CONCRETE SIDEWALK	925	SF

CUT/FILL QUANTITIES	QUANTITY	UNIT
CUT (RAW)	60,900	CY
FILL (RAW)	30,500	CY
NET BALANCE (RAW)	30,400	CY CUT

1. QUANTITIES SHOWN ABOVE ARE RAW VOLUMES AND DO NOT ACCOUNT FOR ANY OF THE FOLLOWING FACTORS:

- SWELL/SHRINKAGE (REFER TO SOILS REPORT)
- QUALITY OF EXCAVATED MATERIAL
- ROCK OR STONE DEPOSITS
- SUB EXCAVATION
- PAVEMENT SECTION

2. THESE QUANTITIES ARE ONLY AN ENGINEERING ESTIMATE OF QUANTITIES. FINAL QUANTITIES ARE TO BE DETERMINED BY THE GRADING CONTRACTOR OR ANY BIDDING ENTITY PRIOR TO BIDDING ON THIS PROJECT.

path: C:\D-Scott\Dynamis Landfill\03_Design\ file name: WTE-GRAD.dwg | plot date: December 27, 2011 | plotted by: ssanders

PROFESSIONAL ENGINEER
10107141
STATE OF IDAHO
DOUGLAS E. PERGAULT

REVISIONS

DATE: DECEMBER 22, 2011
DESIGNED BY: SWF
DRAWN BY: SWF
REVIEWED BY:
PROJECT:
ISSUE:

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DYNAMIS ENERGY, LLC

HIDDEN HOLLOW LANDFILL WASTE TO ENERGY PLANT
PLAN SHEET

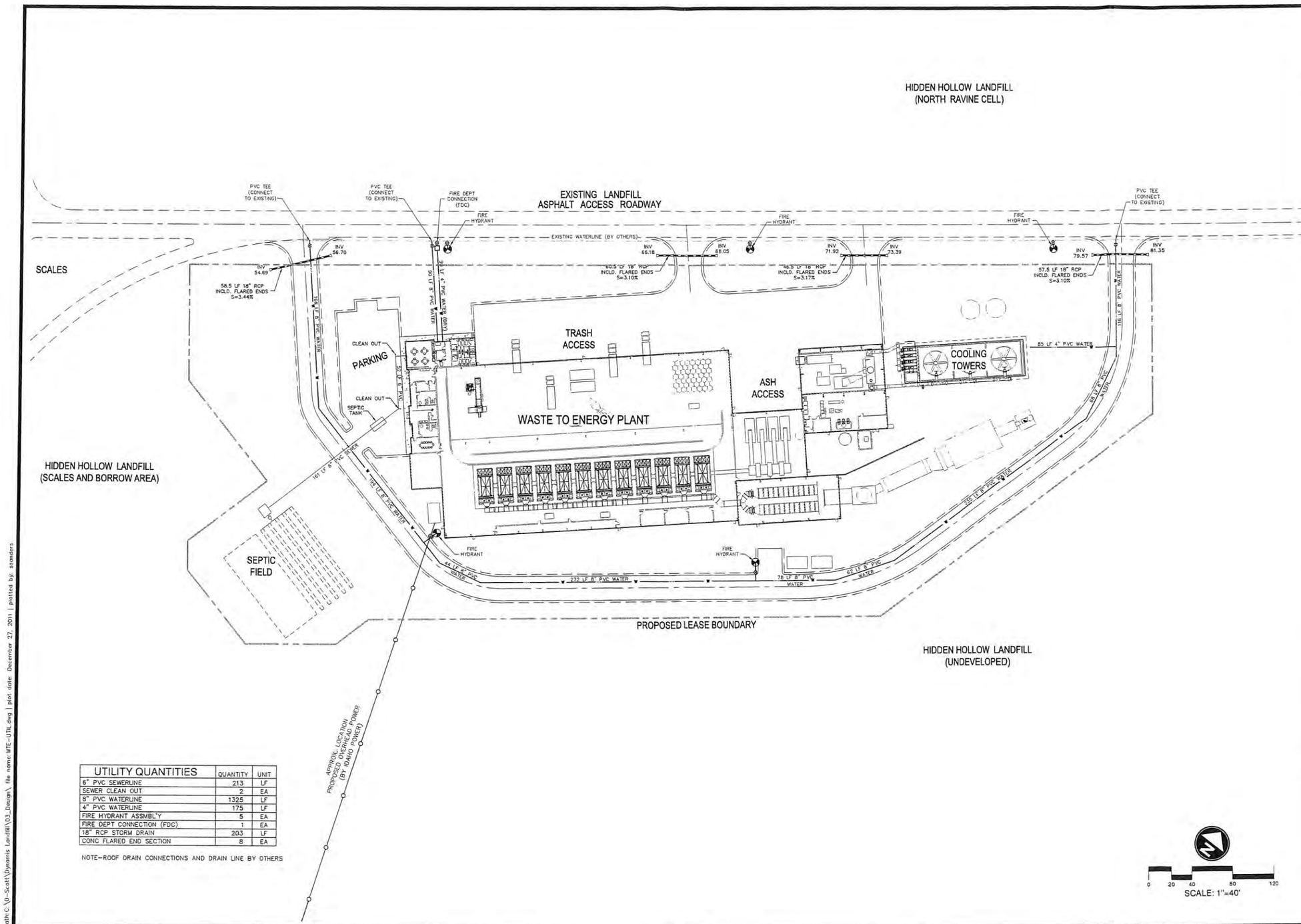
GRADING PLAN

DYNAMIS ENERGY

C4

SHEET 4 OF 6

V. 8 R._



UTILITY QUANTITIES	QUANTITY	UNIT
6" PVC SEWERLINE	213	LF
SEWER CLEAN OUT	2	EA
8" PVC WATERLINE	1325	LF
4" PVC WATERLINE	175	LF
FIRE HYDRANT ASSMBL'Y	5	EA
FIRE DEPT CONNECTION (FDC)	1	EA
18" RCP STORM DRAIN	203	LF
CONC FLARED END SECTION	8	EA

NOTE–ROOF DRAIN CONNECTIONS AND DRAIN LINE BY OTHERS

path: C:\Jo-Scott\Dynamis Landfill\03_Design\ file name: WTE-UTIL.dwg | plot date: December 27, 2011 | plotted by: ssanders

PROFESSIONAL ENGINEER
07-07141
STATE OF IDAHO
JOSHUA E. PERGANDE
12/15/11

REVISIONS	
DATE:	DECEMBER 27, 2011
DESIGNED BY:	STAFF
DRAWN BY:	STAFF
REVIEWED BY:	
PROJECT:	
ISSUE:	

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DYNAMIS ENERGY, LLC

ADA COUNTY LANDFILL WASTE TO ENERGY PLANT

PLAN SHEET

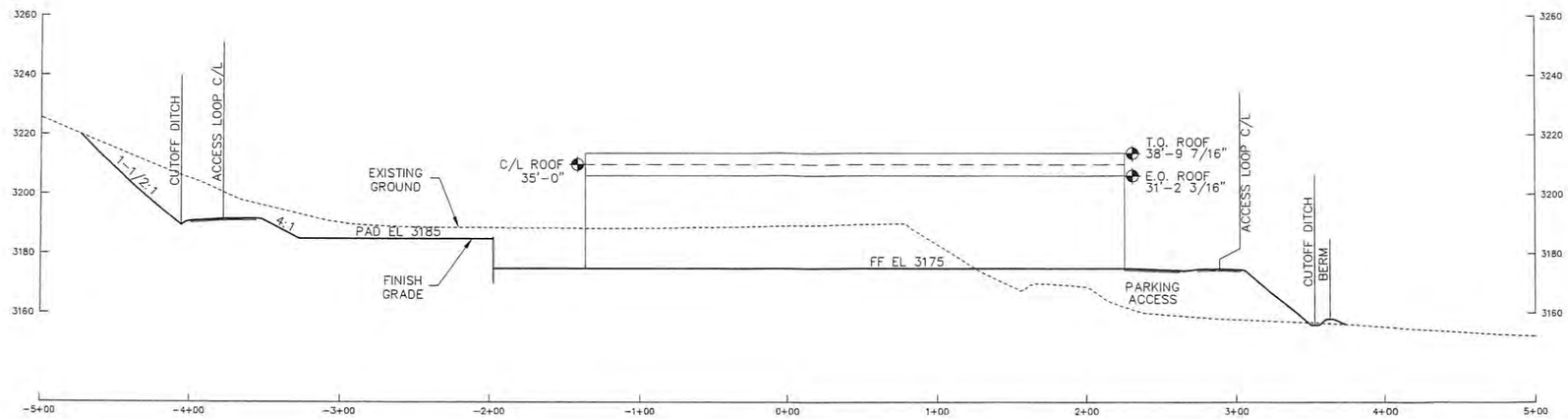
UTILITY PLAN

DYNAMIS ENERGY

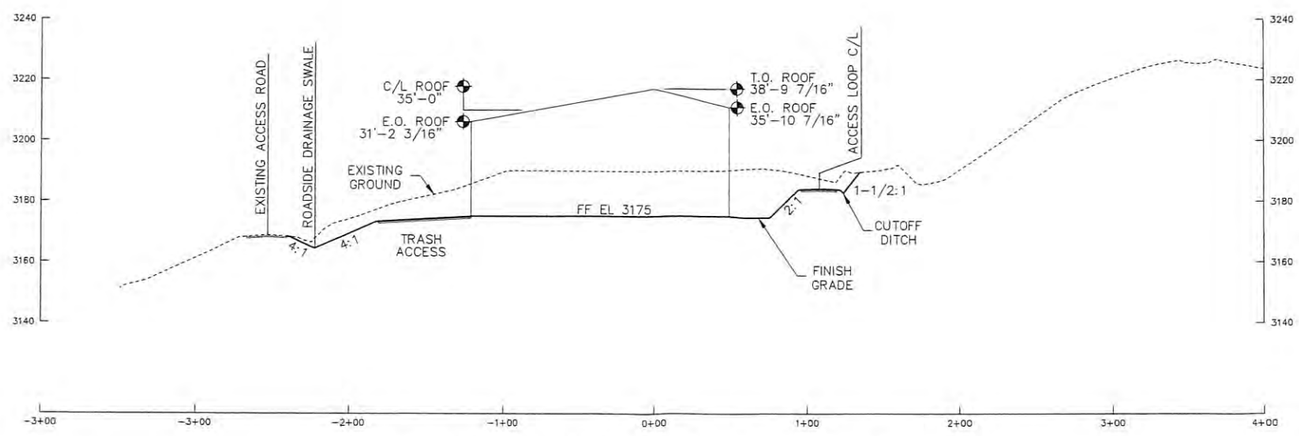
C5

SHEET 5 OF 6

V. 8 R._



A/C4 SITE CROSS SECTION (LOOKING SOUTHEAST)



B/C4 SITE CROSS SECTION (LOOKING NORHTWEST)

0 20 40 80 120
SCALE: 1"=40'
2X VERTICAL EXAGGERATION

PROFESSIONAL ENGINEER REGISTERED PE 07141 STATE OF IDAHO DOUGLAS E. PERGANDE 1/5/12

SEAL

DATE:	DECEMBER 27, 2011
DESIGNED BY:	SYAFF
DRAWN BY:	SYAFF
REVIEWED BY:	
PROJECT:	
ISSUE:	

REVISIONS

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DYNAMIS ENERGY, LLC

ADA COUNTY LANDFILL WASTE TO ENERGY PLANT
SECTION SHEET

SITE CROSS SECTIONS

DYNAMIS ENERGY

C6

SHEET 6 OF 6

V. 8 R._

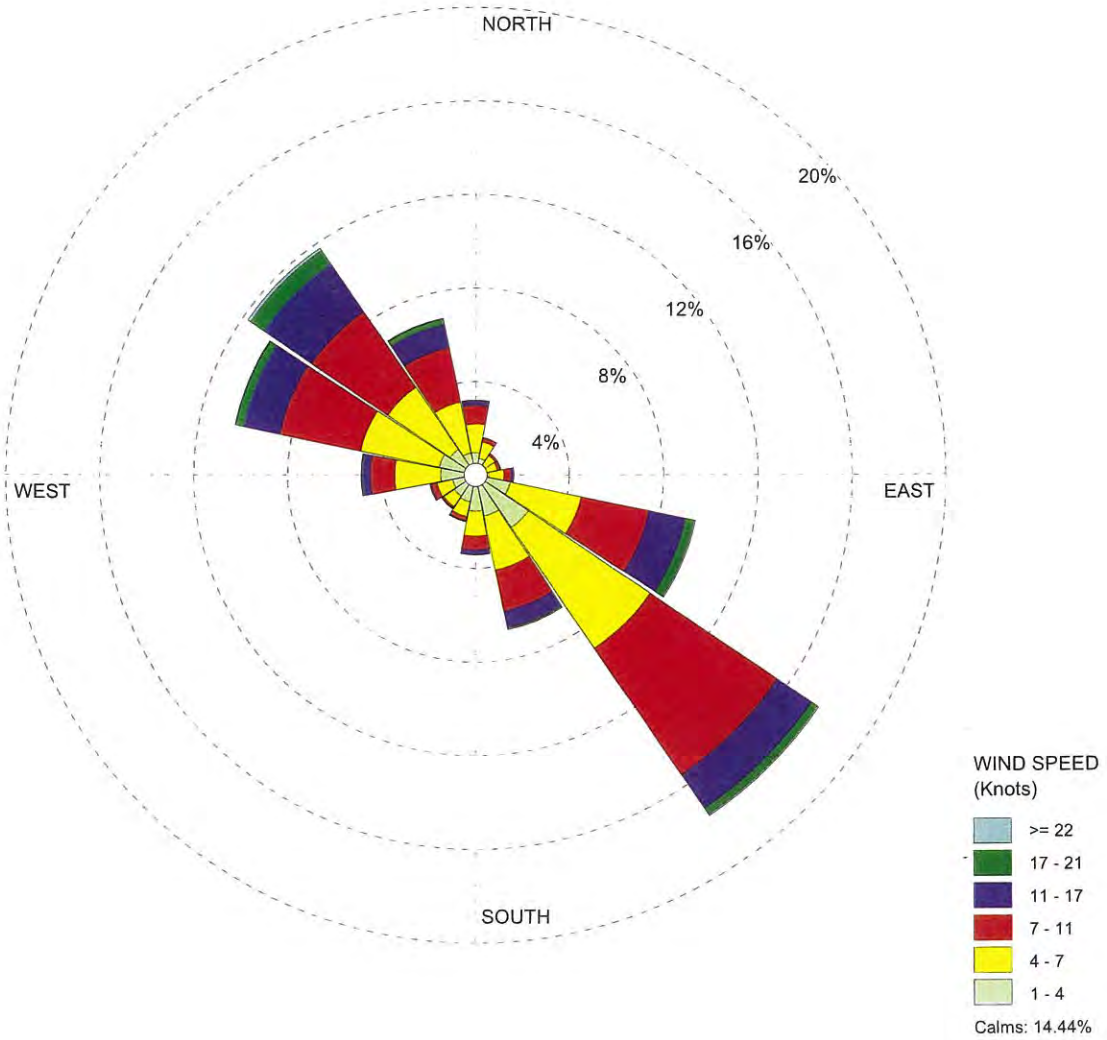
path: C:\0-Scott\Dynamis Landfill\03_Design\ file name: WTE-SECT.dwg | plot date: December 27, 2011 | plotted by: ssanders

WIND ROSE PLOT:

Dynamis WTE Facility - Hidden Hollow Landfill
Boise Met Data

DISPLAY:

Wind Speed
Direction (blowing from)



COMMENTS:	DATA PERIOD:	COMPANY NAME:	
	2001-2005 **Jan 1 - Dec 31** **00:00 - 23:00**	MODELER:	
	CALM WINDS: **14.44%**	TOTAL COUNT: **41530 hrs.**	
	AVG. WIND SPEED: **6.53 Knots**	DATE: **1/21/2011**	PROJECT NO.:

WRPLOT View - Lakes Environmental Software

COMMISSIONERS
OFFICE
200 W. Front Street 3rd Floor
Boise, Idaho 83702
208: 287-7000
Fax: 208: 287-7009
boc@adaweb.net
www.adaweb.net

ADA COUNTY

June 30, 2010

Mr. C. Lloyd Mahaffey
Chairman and CEO
Dynamis Energy, LLC
842 Winding Creek Drive
Eagle, ID 83616

RE: Landfill Waste to Energy Project

Dear Mr. Mahaffey:

The Board of Ada County Commissioners provides this preliminary commitment to grant a franchise to Dynamis Energy, LLC (the "Franchisee) for the purpose of conducting a 250 ton per day waste to energy operation (the "Franchise") at the Hidden Hollow Sanitary Landfill. The material business terms are outlined below.

1. Ada County will grant Franchisee a five-year non-exclusive right to divert 250 tons of waste from the North Ravine Cell for the purposes of operating a waste to energy plant (the "Facility") upon mutually agreeable terms.

2. Ada County and Franchisee may renew the Franchise contemplated herein at such times and on such terms as they may find to be mutually beneficial and in compliance with applicable statutes, codes, laws, rules and regulations.

3. Ada County will lease Franchisee a site located in the Ada County Renewable Technology Industrial Complex of sufficient size to operate the Facility for twenty (20) years at the rate of $1 per year (the "Lease") on a form satisfactory to the Board of Ada County Commissioners.

4. Ada County will provide any waste tires delivered to the Hidden Hollow Sanitary Landfill to Franchisee at no cost to Franchisee during the term of the Franchise. Franchisee may charge a tipping fee not to exceed $5.00 per tire. Further negotiations on the mechanism to accomplish this goal will be conducted in good faith.

5. Ada County will accept ash by-products from the Facility at no charge to Franchisee on condition that such by-product meets or exceeds any applicable regulatory requirements. Franchisee will be responsible for transportation costs.

TAB 11

This serves to confirm that Dynamis is responsible under Section II.B.4 of the Franchise Agreement between Dynamis and Ada County (a copy of the relevant provision is attached) for obtaining all air quality and construction permits necessary to build the facility.

Dynamis Energy, LLC
776 E Riverside Dr. Ste 150
Eagle, Idaho 83616
208-938-2680 (p)

AGREEMENT NO. 9385

FRANCHISE AGREEMENT

BETWEEN ADA COUNTY AND DYNAMIS ENERGY, LLC

HIDDEN HOLLOW WASTE TO ENERGY PROJECT

PROJECT NO. _____

LANDFILL WASTE TO ENERGY FRANCHISE AGREEMENT

THIS FRANCHISE AGREEMENT, dated as of this 1st day of November, 2011 ("Franchise Agreement"), is by and between Dynamis Energy LLC, an Idaho limited liability company (the "Franchisee"), and Ada County, Idaho, a duly organized and existing county under the laws and constitution of the State of Idaho(the "County").

RECITALS AND FINDINGS

A. County owns and operates County's Facility at which County collects, places, compresses, and covers Municipal Solid Waste.

B. County provided an opportunity to the private sector to provide new technologies applicable to recycling, waste reduction and energy production from Municipal Solid Waste and to that end solicited and received proposals.

C. Franchisee submitted a proposal to design, construct, operate, and maintain a waste to energy facility located at County's Facility, which was accepted by County.

D. The Board determined to enter into negotiations with Franchisee, which has resulted in this Franchise Agreement.

E. As part of that negotiation the Board determined to pay Franchisee to design a waste to energy facility, which design, rights, and intellectual property the County would own, subject to purchase by Franchisee.

F. During the period of this Franchise Agreement Franchisee has agreed to take a minimum average of 408 tons of Municipal Solid Waste per calendar day delivered to its waste to energy facility at no charge to County for the purpose of securing a long-term supply of fuel for electric power generation. County and Franchisee have agreed that Franchisee may be allowed to take more.

G. Franchisee has made arrangements and will be responsible for constructing, operating, and maintaining the waste to energy facility.

H. The Board has determined that significant benefits to all Ada County residents can be obtained if the amount of waste being accepted and compacted in its landfills can be reduced and a steady electric power supply provided.

I. County and Franchisee have mutually agreed that Franchisee's Facilities will fit upon and shall be located on the Premises.

IN WITNESS WHEREOF, the parties hereto have caused this Franchise Agreement to be executed by their duly authorized undersigned officers as of the date first set forth above.

Board of Ada County Commissioners

By: _____
Rick Yzaguirre, Chairman

By: _____
Sharon M. Ullman, Commissioner

By: Approved VLB
Vernon L. Bisterfeldt, Commissioner

ATTEST:

_____
Christopher D. Rich, Ada County Clerk

FRANCHISEE – DYNAMIS ENERGY, LLC

10-28-11
Date executed

By: _____
Title: Chairman & CEO
Address: 776 E. Riverside Dr., Suite 150
Eagle, ID 83616

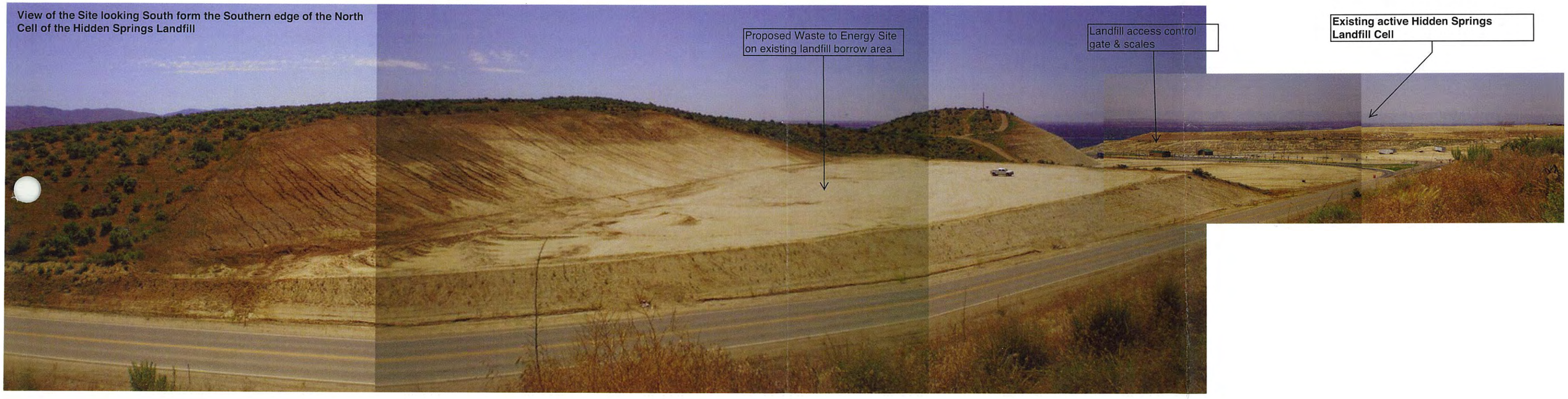
STATE OF IDAHO)
) ss.
County of Ada)

On this 28th day of October, 2011, before me, a Notary Public, personally appeared C. Lloyd Mahaffey, known or identified to me to be the CEO of the limited liability company that executed the instrument or the person who executed the instrument on behalf of said company, and acknowledged to me that such company executed the same.

CRYSTAL A. HINKLE NOTARY PUBLIC STATE OF IDAHO

_____
Notary Public for Idaho
Commission Expires November 24, 2014

TAB 12


View of the Site looking South form the Southern edge of the North Cell of the Hidden Springs Landfill
Proposed Waste to Energy Site on existing landfill borrow area
Landfill access control gate & scales
Existing active Hidden Springs Landfill Cell