SCS ENGINEERS

Williamson County Recycling and Disposal Facility

Notice of Deficiency Response

Site Development Plan Permit Modification

Landfill Phasing and Groundwater Monitoring System

Presented to:

Williamson County

710 Main Street, Suite 101
Georgetown, Texas 78626
(512) 943-1550

STATE OF TEXAS
JAMES R. MURRAY III
73860
REGISTERED PROFESSIONAL ENGINEER
12/23/11

Presented by:

SCS ENGINEERS
Texas Board of Professional Engineers Registration No. F-3407
12651 Briar Forest, Suite 205
Houston, Texas 77077
(281) 397-6747

December 2011
File No. 16208501.04 and 16208501.07

Environmental Consultants
and Contractors

12651 Briar Forest Drive
Suite 205
Houston, Texas 77077-2397

281 397-6747
FAX 281 293-7878
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SCS ENGINEERS

December 20, 2011

P. Hunt Prompuntagorn
Project Manager
Municipal Solid Waste Permits Section
Waste Permits Division
MC-124
Texas Commission on Environmental Quality
P.O. Box 13087
Austin, Texas 78711-3087

Subject: Williamson County Recycling and Disposal Facility – Williamson County Municipal Solid Waste (MSW) – Permit No. 1405B
Permit Modification Request – Landfill Phasing and Groundwater Monitoring System
Response to Notice of Deficiency
Tracking No. 14984728; CN600897888/RN100225754

Dear Mr. Prompuntagorn:

On behalf of Williamson County, SCS Engineers (SCS) is pleased to submit this response to the Notice of Deficiency you issued December 5, 2011 regarding the revisions to the Part III, Site Development Plan (SDP) of the approved permit 1405B for the Williamson County Recycling and Disposal Facility (WCRDF) in Williamson County, Texas.

Specifically, the following comments were offered accompanied by our written response.

Comment No. 1

In accordance with 30 TAC §330.57(i), the applicant must provide a complete copy of any application that requires public notice, including all revisions and supplements to the application, on a publicly accessible internet Web site, and provide the commission with the Web address link for the application materials. In Section B of the Part I Application Form, the Web address link for the referenced application (original submittal and all subsequent revisions) is not provided. Please provide the Web address link.

Response No. 1

The original permit modification application and the first notice of deficiency response were posted to http://www0.wm.com/wm/texas/permits.asp. The web address link has been included in Section B of the Part I Application.

Comment No. 2

The Site Layout Plan, Figure III-1.1, is revised to change the landfill development phasing. Please note that arrows in cells 2C and 6A through 6H seem to point incorrectly. Please re-evaluate these arrows and make adjustments as necessary.

Response No. 2

The Site Layout Plan, Figure III-1.1, has been revised to correct the arrows in cells 2C and 6A through 6I.

Comment No. 3

Revisions to the SDP make a few references to information contained in different attachments. Please make sure that these references provide specific locations. For example, the fifth paragraph of Section 9.1 on page III-A4-67 of the redlined version states "See Attachment 5 for a description of the groundwater monitoring system". A specific section in the referred Attachment 5 should have been provided.

Response No. 3

The revisions to revised sections of the Site Development Plan were reviewed and references have been updated to provide specific sections.

Comment No. 4

Attachments 4, 5, and 11 are substantively revised, including rule citations update. However, some rule citations in Sections 1 through 9 of Attachment 4 still show old rules prior to the March 2006, 30 TAC Chapter 330 rules revision. Please update all rule citations in Attachment 4 to the current 30 TAC Chapter 330.

Response No. 4

Attachment 4, Section 9, Groundwater Monitoring System has been revised to include all appropriate current rule citations. Some minor revisions to other sections in Attachment 4 have been made where appropriate.

Many of the sections 1 through 8 of Attachment 4 were prepared and approved in accordance with rules that were valid and applicable at that time. This has not changed. We believe that the requested rule substitutions would create misleading statements indicating the old documents were prepared in accordance with the new rules. Due to the substantial revisions in the new rules, it is often invalid to simply substitute a new rule citation for an old rule citation, because the new rules are not word-for-word equivalent to the old rules. Therefore, SCS has only cited the new rules for those sections that are the subject of this Permit Modification, which were prepared in accordance with the new rules. All rule citations in Section 9 have been updated.

Comment No. 5

Please correct a typographical error (cinfirmation) in Section 5.3(1) of Attachment 11.

Response No. 5

The typographical error (cinfirmation) was corrected in Section 5.3(1) of Attachment 11.

Revisions to Sections of Part III, Site Development Plan include the following:

Section	Title	Description
Updated Part I Form	Permit or Registration Application for Municipal Solid Waste Facility	Updated Part I Form to reflect Notice of Deficiency submittal.
Revision of Part III	Site Development Plan	Revised Cover Sheet and Table of Contents.
Revision of Part III, Attachment 1	Site Layout Plan	Revised Cover Sheet, Table of Contents and Figure III-1.1, Site Layout Plan.
Revision of Part III, Attachment 4	Geology Report	Revised Cover Sheet, Table of Contents and text.
Revision of Part III, Attachment 5	Groundwater Sampling and Analysis Plan	Revised Cover Sheet, Table of Contents and Section 5.3, Page 14.

The certification statement required by 30 TAC §305.44 is included in the permit application as part of the enclosed Part I Form.

As required by 30 TAC §330.219(a) of the TCEQ rules, please be advised that this letter with enclosures is being placed in the operating record for the subject facility in accordance with the requirements of 30 TAC §330.219(a) and/or (b). Also as required, an original, two unmarked copies, and a redline/strikeout copy of the revised permit modification are being submitted. The original and one copy are being submitted to you and another copy is being submitted directly to Mr. Barry Kalda, Waste Program Manager, TCEQ Regional 11 office.

I trust that this submittal is complete and will lead towards approval of this Permit Modification. If you have any questions or comments concerning this submittal, please contact me at (281) 397-6747.

Sincerely,

J. Roy Murray, P.E.
Vice President
SCS ENGINEERS

Enclosures

cc: Honorable Judge Dan A. Gattis, Williamson County
Barry Kalda, TCEQ
Steve Jacobs, WMTX
Tim Champagne, WMTX

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December 2011
File No. 16208501.04 and 16208501.07

PART 1 FORM

Texas Commission on Environmental Quality

Permit or Registration Application for Municipal Solid Waste Facility

Part I

A. General Information

Facility Name:	Williamson County Recycling and Disposal Facility			
Physical or Street Address (if available):	600 Landfill Road			
(City) (County)(State)(Zip Code):	Hutto	Williamson	TX	78634
(Area Code) Telephone Number:	(512)759-8881			
Charter Number:	Not Applicable			

If the application is submitted on behalf of a corporation, provide the Charter Number as recorded with the Office of the Secretary of State for Texas.

Operator Name[1]:	Williamson County			
Mailing Address:	710 Main Street, Suite 101			
(City) (County)(State)(Zip Code):	Georgetown	Williamson	TX	78626
(Area Code) Telephone Number:	(512) 943-1550			
(Area Code) FAX Number:	(512) 943-1662			
Charter Number:	Not Applicable			

If the permittee is the same as the operator, type "Same as Operator".

Permittee Name:	Same as Operator			
Physical or Street Address (if available):				
(City) (County)(State)(Zip Code):				
(Area Code) Telephone Number:				
Charter Number:				

If the application is submitted by a corporation or by a person residing out of state, the applicant must register an Agent in Service or Agent of Service with the Texas Secretary of State's office and provide a complete mailing address for the agent. The agent must be a Texas resident.

Agent Name:				
Mailing Address:				
(City) (County)(State)(Zip Code):				
(Area Code) Telephone Number:				
(Area Code) FAX Number:				

Application Type:

☒	Permit	☐	Major Amendment	☐	Minor Amendment
☐	Registration	☒	Modification	☐	Temporary Authorization
		☒	w/Public Notice		
		☐	w/out Public Notice	☒	Notice of Deficiency Response

[1] The operator has the duty to submit an application if the facility is owned by one person and operated by another [30 TAC 305.43(b)]. The permit will specify the operator and the owner who is listed on this application [Section 361.087 Texas Health and Safety Code].

Facility Classification:

☒	Type I	☐	Type IV	☐	Type V	☐	Type IX
☐	Type I AE	☐	Type IV AE	☐	Type VI		

Activities covered by this application (check all that apply):

☐	Storage	☐	Processing	☒	Disposal

Waste management units covered by this application (check all that apply):

☐	Containers	☐	Tanks	☐	Surface Impoundments	☒	Landfills
☐	Incinerators	☐	Composting	☐	Type IV Demonstration Unit	☐	Type IX Energy/Material Recovery
☐	Other (Specify)			☐	Other (Specify)		
☐	Other (Specify)			☐	Other (Specify)		

Is this submittal part of a Consolidated Permit Processing request, in accordance with 30 TAC Chapter 33?

☐ Yes ☒ No

If yes, state the other TCEQ program authorizations requested.

N/A

Provide a brief description of the portion of the facility covered by this application. For amendments, modifications, and temporary authorizations, provide a brief description of the exact changes to the permit or registration conditions and supporting documents referenced by the permit or registration. Also, provide an explanation of why the amendment, modification, or temporary authorization is requested.

Notice of Deficiency response to permit modification to revise the Site Development Plan to revise the sequence of landfill development and update the groundwater monitoring system to comply with the 30 TAC 330, Subchapter J requirements.

Does the application contain confidential Material? ☐ Yes ☒ No

If yes, cross-reference the confidential material *throughout the application* and submit as a separate document or binder conspicuously marked "CONFIDENTIAL."

Alternative Language Notice Instructions

For certain permit applications, public notice in an alternate language is required. If an elementary school or middle school nearest to the facility offers a bilingual program, notice may be required to be published in an alternative language. The Texas Education Code, upon which the TCEQ alternative language notice requirements are based, trigger a bilingual education program to apply to an entire school district should the requisite alternative language speaking student population exist. However, there may not exist any bilingual students at a particular school within a district which is required to offer the bilingual education program. For this reason, the requirement to publish notice in an alternative language is triggered if the nearest elementary or middle school, as a part of a larger school district, is required to make a bilingual education program available to qualifying students and either the school has students enrolled at such a program on-site, or has students who attend such a program at another location in satisfaction of the school's obligation to provide such a program as a member of a triggered district.

If it is determined that an alternative language notice is required, the applicant is responsible for ensuring

that the publication in the alternate language is complete and accurate in that language. Electronic versions of the Spanish template examples are available from the TCEQ to help the applicant complete the publication in the alternative language.

Alternative Language Notice Application Form:

Alternative language notice confirmation for this application:

1. Is a bilingual program required by the Texas Education Code in the school district where the facility is located? ☐ YES ☒ NO

   (If NO, alternative language notice publication not required)

2. If YES to question 1, are students enrolled in a bilingual education program at either the elementary school or the middle school nearest to the facility? ☐ YES ☐ NO

(IF YES to questions 1 and 2, alternative language publication is required; If NO to question 2, then consider the next question)

3. If YES to question 1, are there students enrolled at either the elementary school or the middle school nearest to the facility who attend a bilingual education program at another location? ☐ YES ☐ NO

(If Yes to questions 1 and 3, alternative language publication is required; If NO to question 3, then consider the next question)

4. If YES to question 1, would either the elementary school or the middle school nearest to the facility be required to provide a bilingual education program but for the fact that it secured a waiver from this requirement, as available under 19 TAC §89.1205(g)? ☐ YES ☐ NO

(If Yes to questions 1 and 4, alternative language publication is required; If NO to question 4, alternative language notice publication not required)

If a bilingual education program(s) is provided by either the elementary school or the middle school nearest to the facility, which language(s) is required by the bilingual program?

Note: Applicants for new permits and major amendments must make a copy of the administratively complete application available at a public place in the county where the facility is, or will be, located for review and copying by the public.

Public place where administratively complete permit application will be located.				
Public Place (e.g., public library, county court house, city hall, etc.):	N/A			
Mailing Address:				
(City) (County)(State)(Zip Code):				
(Area Code) Telephone Number:				

B. Facility Location

Except for Type I AE and Type IV AE landfill facilities, for permits, registrations, amendments, and modifications requiring public notice, provide the URL address of a publicly accessible internet web site where the application and all revisions to that application will be posted.
http://www0.wm.com/wm/texas/permits.asp

Local Government Jurisdiction:	Williamson County
Within City Limits of:	N/A
Within Extraterritorial Jurisdiction of City of:	N/A
Is the proposed municipal or industrial solid waste disposal or processing facility located in an area in which the governing body of the municipality or county has prohibited the disposal or processing of municipal or industrial solid waste? (If YES, provide a copy of the ordinance or order):	
☐ YES ☒ NO	

Provide a description of the location of the facility with respect to known or easily identifiable landmarks.
1 mile north of the blue Jonah Water Supply Corp. water tower

Detail the access routes from the nearest United States or state highway to the facility.
5 miles north of State Highway 79 on FM 1660

Provide the latitudinal and longitudinal geographic coordinates of the facility.

Latitude	**N 30° 36' 15"**
Longitude	**W 97° 33' 12"**
Elevation (above msl)	724.157 ft

Is the facility within the Coastal Management Program boundary?	☐ Yes	☒ No

Texas Department of Transportation District Location:

TXDOT District Name & Number:	Austin District No. 14			
District Engineer's Name:	Carlos Lopez, P.E.			
Street or P. O. Box:	P.O. Drawer 15426			
(City) (County)(State)(Zip Code):	Austin	Travis	TX	78761
(Area Code) Telephone Number:	(512) 832-7000			
(Area Code) FAX Number:	(512) 478-8243			

The local governmental authority or agency responsible for road maintenance:

Agency Name	Williamson County Road Department			
Contact Person's Name:	Greg Bergeron			
Street or P. O. Box:	3151 S.E. Inner Loop, Suite B			
(City) (County)(State)(Zip Code):	Georgetown	Williamson	TX	78626
(Area Code) Telephone Number:	(512) 943-3330			
(Area Code) FAX Number:	(512) 943-3335			

State Representative:

District Number:	52			
State Representative's Name:	Larry Gonzales			
District Office Address:	P.O. Box 2501			
(City) (County)(State)(Zip Code):	Round Rock	Williamson	TX	78680
(Area Code) Telephone Number:	(512) 463-0670			
(Area Code) FAX Number:	(512) 463-1469			

State Senator:

District Number:	5			
State Senator's Name:	Steve Ogden			
District Office Address:	P.O. Box 12068			
(City) (County)(State)(Zip Code):	Austin	Travis	TX	78711
(Area Code) Telephone Number:	(512) 463-0105			
(Area Code) FAX Number:	(512) 463-5713			

Council of Government (COG) Information:

COG Name:	Capital Area Council of Governments			
COG Representative's Name:	Kelly Freeman			
COG Representative's Title:	Solid Waste Program Coordinator			
Street or P. O. Box:	6800 Burleson Road, Building 310, Suite 165			
(City) (County)(State)(Zip Code):	Austin	TX	Travis	78744
(Area Code) Telephone Number:	(512) 916-6040			
(Area Code) FAX Number:	(512) 916-6001			

River Basin Information:

River Authority:	Brazos River Authority			
Contact Person's Name:	Phil Ford, General Manager and CEO			
Watershed Sub-Basin Name:	Central Brazos River Basin			
Street or P. O. Box:	P.O. Box 7555			
(City) (County)(State)(Zip Code):	Waco	McLennan	TX	76714
(Area Code) Telephone Number:	(254) 761-3100			
(Area Code) FAX Number:	(254) 761-3207			

This site is located in the following District of the U.S. Army Corps of Engineers:			
☐ Albuquerque, NM	☒ Ft. Worth, TX	☐ Galveston, TX	☐ Tulsa, OK

C. Maps

General

For permits, registrations, and amendments only, submit a topographic map, ownership map, county highway map, or a map prepared by a registered professional engineer or a registered surveyor which shows the facility and each of its intake and discharge structures and any other structure or location regarding the regulated facility and associated activities. Maps must be of material suitable for a permanent record, and shall be on sheets 8-1/2 inches by 14 inches or folded to that size, and shall be on a scale of not less than one inch equals one mile. The map shall depict the approximate boundaries of the tract of land owned or to be used by the applicant and shall extend at least one mile beyond the tract boundaries sufficient to show the following:

each well, spring, and surface water body or other water in the state within the map area;

the general character of the areas adjacent to the facility, including public roads, towns and the nature of development of adjacent lands such as residential, commercial, agricultural, recreational, undeveloped, etc;

the location of any waste disposal activities conducted on the tract not included in the application; and

the ownership of tracts of land adjacent to the facility and within a reasonable distance from the proposed point or points of discharge, deposit, injection, or other place of disposal or activity.

General location maps

For permits, registrations, and amendments only, submit at least one general location map at a scale of one-half inch equals one mile. This map shall be all or a portion of a county map prepared by Texas Department of Transportation (TxDOT). If TxDOT publishes more detailed maps of the proposed facility area, the more detailed maps shall also be included in Part I. Use the latest revision of all maps.

Land ownership map

Provide a map that locates the property owned by adjacent and potentially affected landowners. The maps should show all property ownership within 1/4 mile of the facility, on-site facility easement holders, and all mineral interest ownership under the facility.

Landowners list

Provide the adjacent and potentially affected landowners' list, keyed to the land ownership map with each property owner's name and mailing address. The list shall include all property owners within 1/4 mile of the facility, easement holders, and all mineral interest ownership under the facility. Provide the property, easement holders', and mineral interest owners' names and mailing addresses derived from the real property appraisal records as listed on the date that the application is filed. Provide the list in electronic form, as well.

D. Property owner information

For permits, registrations, amendments, and modifications that change the legal description, a change in owner, or a change in operator only, provide the following:

(1) the legal description of the facility;

(A) the abstract number as maintained by the Texas General Land Office for the surveyed tract of land;

(B) the legal description of the property and the county, book, and page number or other generally accepted identifying reference of the current ownership record;

(C) for property that is platted, the county, book, and page number or other generally accepted identifying reference of the final plat record that includes the acreage encompassed in the application and a copy of the final plat, in addition to a written legal description;

(D) a boundary metes and bounds description of the facility signed and sealed by a registered professional land surveyor;

(E) on-site easements at the facility, and

(F) drawings of the boundary metes and bounds description; and

(2) a property owner affidavit signed by the owner.

E. Legal authority

Provide verification of the legal status of the owner and operator, such as a one-page certificate of incorporation issued by the secretary of state. List all persons having over a 20% ownership in the proposed facility.

Indicate Ownership status of the facility:									
☐	Private	☐	Corporation	☐	Partnership	☐	Proprietorship	☐	Non-Profit Organization
☐	Public	☐	Federal	☐	Military	☐	State	☐	Regional
☒	County	☐	Municipal	☐	Other (Specify)				

Does the operator own the facility units and the facility property?	☐ Yes ☒ No

If "No," for permits, registrations, amendments, and modifications that changes the legal description, a change in owner, or a change in operators submit a copy of the lease for the use of or the option to buy the facility units or facility property, as appropriate, and identify:	
Owner Name:	
Street or P. O. Box:	
(City) (County)(State)(Zip Code):	
(Area Code) Telephone Number:	
(Area Code) FAX Number:	
Charter Number:	

F. Evidence of competency

For permits, registrations, amendments, and modifications that change the legal description, a change in owner, or a change in operators submit a list of all Texas solid waste sites that the owner and operator have owned or operated within the last ten years.

Site Name	Site Type	Permit/Reg. No.	County	Dates of Operation

Submit a list of all solid waste sites in all states, territories, or countries in which the owner and operator have a direct financial interest.

Site Name	Location	Dates of Operation	Regulatory Agency (Name & Address)

A licensed solid waste facility supervisor, as defined in 30 TAC Chapter 30, Occupational Licenses and Registrations will be employed before commencing facility operation.

Provide the names of the principals and supervisors of the owner's and operator's organization, together with previous affiliations with other organizations engaged in solid waste activities.

Name	Previous Affiliation	Other Organization

For landfill permit applications only, evidence of competency to operate the facility shall also include landfilling and earthmoving experience if applicable, and other pertinent experience, or licenses as described in 30 TAC Chapter 30 possessed by key personnel. The number and size of each type of equipment to be dedicated to facility operation will be specified in greater detail on Part IV of the application within the site operating plan.

Landfilling/Earthmoving Equipment Types	Personnel Experience or Licenses

For mobile liquid waste processing units, submit a list of all solid waste, liquid waste, or mobile waste units that the owner and operator have owned or operated within the past five years. Submit a list of any final enforcement orders, court judgments, consent decrees, and criminal convictions of this state and the federal government within the last five years relating to compliance with applicable legal requirements relating to the handling of solid or liquid waste under the jurisdiction of the commission or the United States Environmental Protection Agency. Applicable legal requirement means an environmental law, regulation, permit, order, consent decree, or other requirement.

Solid waste, liquid waste, or mobile waste units owned or operated within past 5 years	Texas and federal final enforcement orders, court judgments, consent decrees, and criminal convictions

G. Appointments

Provide documentation that the person signing the application meets the requirements of 30 TAC §305.44, Signatories to Applications. If the authority has been delegated, provide a copy of the document issued by the governing body of the owner or operator authorizing the person that signed the application to act as agent for the owner or operator.

H. Application Fees

For a new permit, registration, amendment, modification, or temporary authorization, submit a $150 application fee.

For authorization to construct an enclosed structure over an old, closed municipal solid waste landfill in accordance with 30 TAC 330 Subchapter T, submit a $2,500 application fee.

If paying by check, send payment to:

Texas Commission on Environmental Quality
Financial Administration Division, MC 214
P. O. Box 13087
Austin, Texas 78711-3087

Payment maybe made online using TCEQ e-pay at www.tceq.state.tx.us/e-services/	
E-pay confirmation number	

PROPERTY OWNER AFFIDAVIT

"I, Dan A. Gattis, Williamson County Judge,
(property owner)
acknowledge that the State of Texas may hold me either jointly or severally responsible for the operation, maintenance, and closure and post-closure care of the facility. For a facility where waste will remain after closure, I acknowledge that I have a responsibility to file with the county deed records an affidavit to the public advising that the land will be used for a solid waste facility prior to the time that the facility actually begins operating as a municipal solid waste landfill facility, and to file a final recording upon completion of disposal operations and closure of the landfill units in accordance with Title 30 Texas Administrative Code §330.19, Deed Recordation. I further acknowledge that I or the operator and the State of Texas shall have access to the property during the active life and post-closure care period, if required, after closure for the purpose of inspection and maintenance."

(Owner signature) 01-13-2012 (Date)

Signature Page

I, Dan A. Gattis (Operator), Williamson County Judge (Title),

certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

Signature: _____ Date: 01-17-2012

TO BE COMPLETED BY THE OPERATOR IF THE APPLICATION IS SIGNED BY AN AUTHORIZED REPRESENTATIVE FOR THE OPERATOR

I, _____ (Print or Type Operator Name), hereby designate _____ (Print or Type Representative Name)

as my representative and hereby authorize said representative to sign any application, submit additional information as may be requested by the Commission; and/or appear for me at any hearing or before the Texas Commission on Environmental Quality in conjunction with this request for a Texas Water Code or Texas Solid Waste Disposal Act permit. I further understand that I am responsible for the contents of this application, for oral statements given by my authorized representative in support of the application, and for compliance with the terms and conditions of any permit which might be issued based upon this application.

_____
Printed or Typed Name of Operator or Principal Executive Officer

_____
Signature

SUBSCRIBED AND SWORN to before me by the said Dan A. Gattis

On this 13th day of January, 2012

My commission expires on the 18th day of August, 2012

PEGGY VASQUEZ
NOTARY PUBLIC
STATE OF TEXAS
MY COMM. EXP. 8/18/12

_____
Notary Public in and for

Williamson County, Texas

(Note: Application Must Bear Signature & Seal of Notary Public)

SITE DEVELOPMENT PLAN

- PART III, SITE DEVELOPMENT PLAN (MAIN TEXT)
- PART III, ATTACHMENT 1, SITE LAYOUT PLAN
- PART III, ATTACHMENT 4, GEOLOGY REPORT
- PART III, ATTACHMENT 11, GROUNDWATER SAMPLING AND ANALYSIS PLAN

WILLIAMSON COUNTY
RECYCLING & DISPOSAL FACILITY
PERMIT AMENDMENT APPLICATION MSW-1405B
WILLIAMSON COUNTY, TEXAS

SITE DEVELOPMENT PLAN

PART III

Prepared for:

WILLIAMSON COUNTY
301 S.E. Inner Loop, Suite 109
Georgetown, Texas 78626

and

WASTE MANAGEMENT OF TEXAS, INC.
9900 Giles Road
Austin, Texas 78754

Prepared by:

SCS ENGINEERS
Texas Board of Professional Engineers Registration No. F-3407
12651 Briar Forest Drive, Suite 205
Houston, Texas 77077

MAY 2005
TECHNICALLY COMPLETE: FEBRUARY 2006
REVISION I: AUGUST 2009
REVISION 2: OCTOBER 2009
REVISION 3: OCTOBER 2011
REVISION 4: DECEMBER 2011

TABLE OF CONTENTS

1.0 SOLID WASTE DATA §330.54(3) 1
2.0 DESIGN DATA §330.54(4) 1
2.1 Landfill Method §330.55(a)(1) 2
2.2 All-weather Operations §330.55(a)(2) 2
2.3 Access Control §330.55(a)(3) 3
2.4 Site Life §330.55(a)(4) 4
3.0 DRINKING WATER PROTECTION §330.54(4), §330.55(a)(5), §330.200-206 4
3.1 Liner Design §330.200; §330.202 4
3.2 Leachate Collection System §330.201 5
3.3 Liner Design Constraints §330.203 6
3.4 Faults §330.204 6
3.5 Soil and Liner Quality Control Plan §330.205; §330.206 6
4.0 SURFACE WATER PROTECTION §330.55(b)(1)-(8) 7
4.1 Discharge of Pollutants §330.55(b)(1) 7
4.2 Handling Contaminated Water §330.55(b)(3)&(6) 8
4.3 Natural Drainage Patterns §330.55(b)(5)(D) 9
4.4 Site Drainage Design §330.55(b)(5)(A)-(C) 9
4.5 Drainage Impact §330.55(b)(5)(D) 10
4.6 Floodplain Considerations §330.55(b)(7) 11
4.7 Final Cover Erosion Protection §330.55(b)(4),(5)(E),&(8) 12
5.0 PROTECTION OF ENDANGERED SPECIES §330.55(b)(9) 13
6.0 LANDFILL MARKERS §330.55(b)(10)(A-H) and (J) 14
6.1 Site Markers §330.55(b)(10) 14
6.1.1 Easement And R.O.W. Markers §330.55(b)(10)(A),(B)&(E) 14
6.1.2 Site Grid System Markers §330.55(b)(10)(A),(B)&(F) 14
6.1.3 SLER or GLER Area Markers §330.55(b)(10)(A),(B),&(G) 15
6.1.4 100 Year Flood Limit Protection Markers §330.55(b)(10)(A),(B)&(H) 15
6.1.5 Site Boundary Markers §330.55(b)(10)(A)(B)&(C) 15
6.1.6 Buffer Zone Markers §330.55(b)(10)(A)(B)&(D) 15
6.1.7 Permanent Benchmark §330.55(b)(10)(J) 15

APPENDICES

A Estimated Rate of Waste Deposition and Operating Life of Site

LIST OF ATTACHMENTS

III-1 Site Layout Plan (Revised December 2011)
III-2 Fill Cross-Section
III-3 Existing Contour Map
III-4 Geology Report (Revised December 2011)
III-5 Groundwater Characterization Report (Revised December 2011)
III-6 Groundwater and Surface Water Protection Plan & Drainage Plan (Revised July 2010)
III-7 Final Contour Map
III-8 Closure and Post-Closure Cost Estimate (Revised August 2009)
III-9 Applicant's Statement
III-10 Soil and Liner Quality Control Plan
III-11 Groundwater Sampling and Analysis Plan (Revised December 2011)
III-12 Closure Plan
III-13 Post-Closure Plan
III-14 Landfill Gas Management Plan (Revised October 2011)
III-15 Leachate and Contaminated Water Plan

WILLIAMSON COUNTY RECYCLING & DISPOSAL FACILITY PERMIT AMENDMENT APPLICATION MSW-1405B WILLIAMSON COUNTY, TEXAS

SITE LAYOUT PLAN

PART III, ATTACHMENT 1

Prepared for:

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and

WASTE MANAGEMENT OF TEXAS, INC.
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Prepared by:

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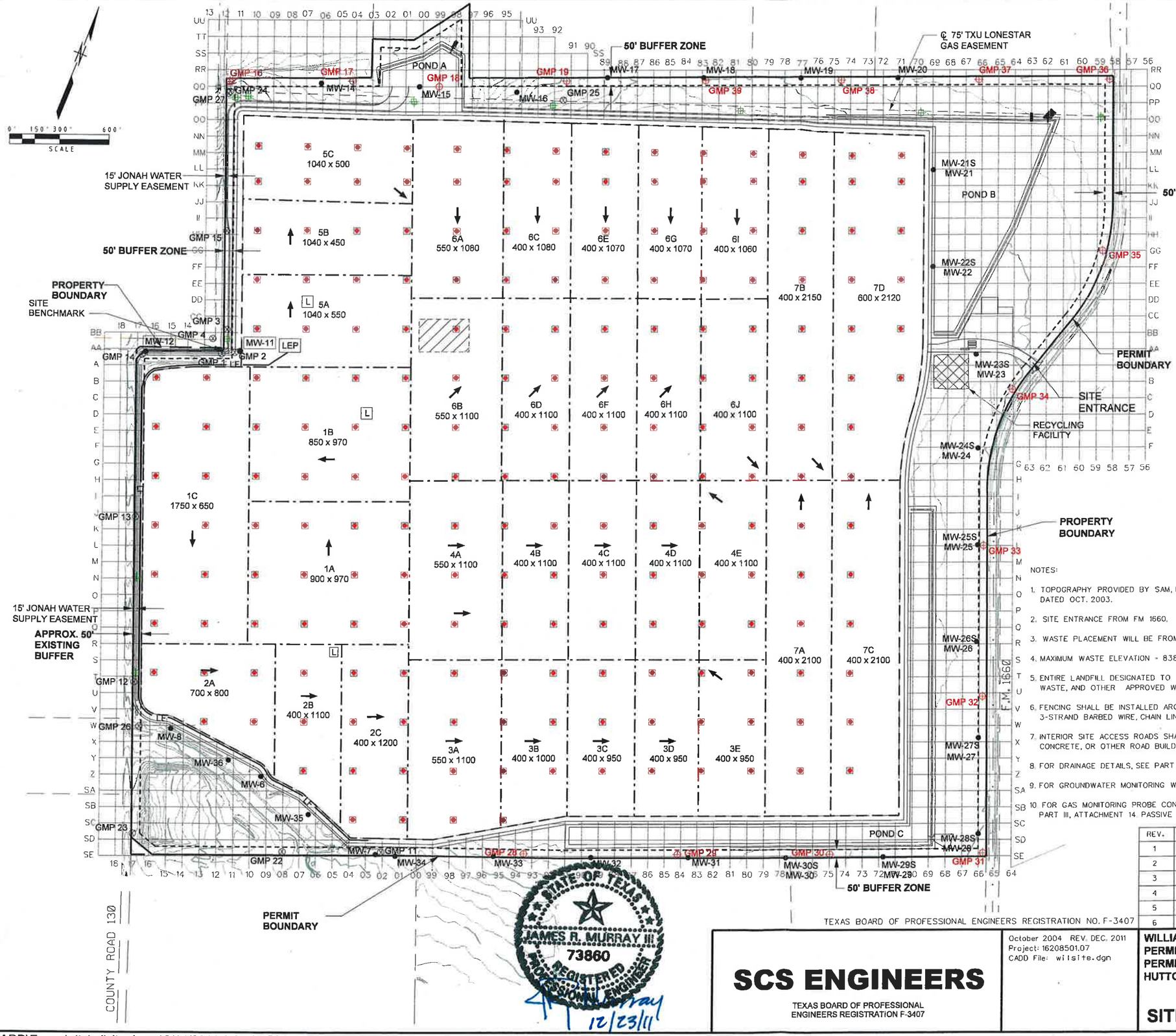
MAY 2005
TECHNICALLY COMPLETE: FEBRUARY 2006
REVISION 1: MAY 2009
REVISION 2: OCTOBER 2009
REVISION 3: OCTOBER 2011
REVISION 4: DECEMBER 2011

TABLE OF CONTENTS

FIGURES

III - 1.1 Site Layout Plan (Revised December 2011)
III - 1.2A Existing Facility Entrance Plan (Revised October 2009)
III - 1.2B Proposed Facility Entrance Plan (Revised October 2009)
III - 1.3 Road Details
III - 1.4 Landfill Development Phasing - Initial Phases (Revised July 2011)
III - 1.5 Landfill Development Phasing - Intermediate Phases (Revised July 2011)
III - 1.5A Landfill Development Phasing - Intermediate Phases (Added July 2011)
III - 1.5B Landfill Development Phasing - Intermediate Phases (Added July 2011)
III - 1.5C Landfill Development Phasing - Intermediate Phases (Added July 2011)
III - 1.6 Landfill Development Phasing - Final Phases (Revised July 2011)
III - 1.7 Shingle and C&D Recycling Facility Layout (Added May 2009)
III - 1.8 Asphalt Shingle Processing Unit General Facility Arrangement (Added May 2009)
III - 1.9 C&D Recycling Facility General Arrangement (Added May 2009)
III - 1.10 Recycling Facility Site Plan (Added October 2009)



LEGEND

- PROPERTY BOUNDARY
- PERMIT BOUNDARY
- 4C — CELL DESIGNATION & GENERAL DIRECTION OF PROGRESSION TO THE NEXT CELL
- CELL LIMIT
- 50 FOOT BUFFER ZONE
- F / 56 — SITE GRID SYSTEM
- 730 — EXISTING CONTOUR
- POND C — PROPOSED PONDS
- PROPOSED ROADS
- GMP 39 — PROPOSED GAS MONITORING PROBE
- GMP 24 — EXISTING GAS MONITORING PROBE
- PASSIVE LANDFILL GAS VENT
- UNDERGROUND UTILITY GAS VENT
- MW-12 — BOX INDICATES BACKGROUND MONITORING WELL
- MW-4 — GROUNDWATER MONITORING WELL
- SITE BENCHMARK N 10193605.909 E 3170469.313 EL.=724.157
- L — LIQUID STABILIZATION BASIN (TEMPORARY)
- LEP — LEACHATE EVAPORATION POND (TEMPORARY)
- LF — LEACHATE FORCE MAIN
- APPROXIMATE LOCATION OF ASPHALT SHINGLE PROCESSING UNIT AND CONSTRUCTION AND DEMOLITION WASTE RECYCLING FACILTY
- RECYCLING FACILTY

NOTES:

1. TOPOGRAPHY PROVIDED BY SAM, INC. DATED OCT. 2003.
2. SITE ENTRANCE FROM FM 1660.
3. WASTE PLACEMENT WILL BE FROM WEST TO EAST.
4. MAXIMUM WASTE ELEVATION = 838.0 FEET msl. MAXIMUM FINAL COVER ELEVATION = 840.0 FEET msl.
5. ENTIRE LANDFILL DESIGNATED TO RECEIVE MUNICIPAL SOLID WASTE, CONSTRUCTION AND DEMOLITION WASTE, AND OTHER APPROVED WASTES. NO SPECIAL TRENCH DESIGNATIONS ARE INCLUDED.
6. FENCING SHALL BE INSTALLED AROUND THE ENTIRE PERMIT BOUNDARY. FENCING SHALL BE AT LEAST 3-STRAND BARBED WIRE, CHAIN LINK, OR EQUIVALENT.
7. INTERIOR SITE ACCESS ROADS SHALL BE EITHER COMPACTED SOIL, GRAVEL, CRUSHED STONE, APHALT, CONCRETE, OR OTHER ROAD BUILDING MATERIAL. SITE TO MAINTAIN ROADS TO PROVIDE ACCESS TO LANDFILL.
8. FOR DRAINAGE DETAILS, SEE PART III, ATTACHMENT 6.
9. FOR GROUNDWATER MONITORING WELL CONSTRUCTION SCHEDULE, SEE PART III, ATTACHMENTS 4 AND 5.
10. FOR GAS MONITORING PROBE CONSTRUCTION SCHEDULE AND DETAILS OF PASSIVE LANDFILL GAS VENT, SEE PART III, ATTACHMENT 14. PASSIVE LANDFILL GAS VENTS MAY BE REPLACED WITH ACTIVE GAS CONTROL SYSTEM.

REV.	DATE	DESCRIPTION	DRN BY	DES BY	CHK BY	APP BY
1	7/05	TECH NOD 1	JG	JRM	JW	JKR
2	8/07	CORRECT CELL DIMENSIONS	JG	BF	JRM	JKR
3	5/09	ADD ASPU and CDWRF	JG	PJ	JRM	JKR
4	9/09	REV: RECYCLING, MAINT. & OFFICE	JG	PJ	JRM	JKR
5	10/11	ADD LEACHATE POND & SOLID BASIN	JG	PJ	JRM	JKR
6	12/11	REVISED SEQUENCE OF DEVELOP & GW WELLS	JG	JRM	RD	JKR

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REGISTERED PROFESSIONAL ENGINEER
12/23/11

SCS ENGINEERS

TEXAS BOARD OF PROFESSIONAL ENGINEERS REGISTRATION F-3407

October 2004 REV. DEC. 2011
Project: 16208501.07
CADD File: wilsite.dgn

WILLIAMSON COUNTY RECYCLING & DISPOSAL FACILITY
PERMIT MODIFICATION
PERMIT NO. MSW-1405B
HUTTO, WILLIAMSON COUNTY, TEXAS

SITE LAYOUT PLAN

FIGURE III-1.1

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WILLIAMSON COUNTY
RECYCLING & DISPOSAL FACILITY
PERMIT AMENDMENT APPLICATION MSW 1405B
WILLIAMSON COUNTY, TEXAS

GEOLOGY REPORT

PART III, ATTACHMENT 4

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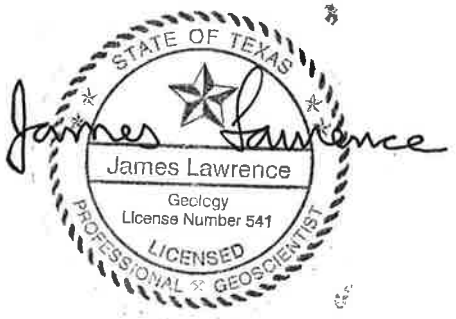


12·19·2011

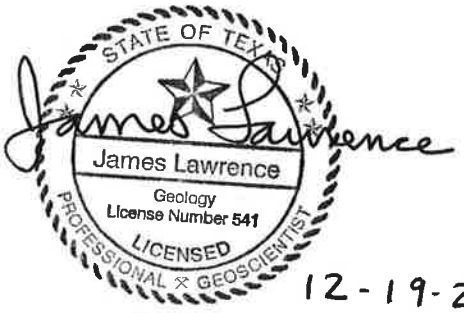
ATTACHMENT 4
GEOLOGY REPORT

As the supervising hydrogeologist, I have reviewed modified portions of this report (as revised July 2011) and found it to be prepared by qualified groundwater scientists in accordance with the applicable regulations outlined in 30 TAC §330.63(e) and relevant portions of 30 TAC 330.401 through .421. The accompanying modified documents "Attachment 5 Groundwater Characterization Report" and "Attachment 11 Groundwater Sampling and Analysis Plan" also address certain portions of 30 TAC §330.63(e) and 30 TAC 330.401 through .421.

James Lawrence

James Lawrence, P.G.
Principal Hydrogeologist
Professional Geologist (TX #541)

Date: 12-19-2011



12-19-2011

TABLE OF CONTENTS

Section			Page
1.0		**INTRODUCTION**	**1**
	1.1	Background	1
2.0		**PHYSIOGRAPHY AND TOPOGRAPHY**	**3**
	2.1	Regional Physiographic Setting	3
	2.2	Topography	4
	2.3	Surface Water Drainage Features	4
3.0		**REGIONAL GEOLOGY**	**5**
	3.1	Stratigraphy and Lithology	5
	3.2	Regional Structural Geology	9
4.0		**ACTIVE GEOLOGICAL PROCESSES**	**10**
	4.1	Holocene Faults and Seismic Impact	10
		4.1.1 Holocene Faults	10
		4.1.2 Seismic Impact Zones	13
	4.2	Subsidence	14
	4.3	Unstable Areas	14
		4.3.1 Karst Terrain	15
		4.3.2 Poor Foundation Conditions	15
		4.3.3 Areas Susceptible to Mass Movement	16
	4.4	Erosion	17
	4.5	Wetlands	17
5.0		**REGIONAL HYDROGEOLOGY**	**19**
	5.1	Regional Aquifers	19
		5.1.1 Edwards Aquifer	20
		5.1.2 Taylor Group and Austin Chalk	21
		5.1.3 Travis Peak Formation	22
	5.2	Regional Water Quality	22
	5.3	Regional Water Use	24
6.0		**SUBSURFACE INVESTIGATION**	**25**
	6.1	Overview	25
		6.1.1 Previous Subsurface Investigation Reports	25
		6.1.2 Subsurface Investigation Scope of Work	27
	6.2	Field Activities	27
		6.1.1 Survey and Ground Control	27
		6.2.1 Aerial Photograph and Map Interpretation	28
		6.2.2 Drilling Program	28
		6.2.3.1 Drilling and Soil Sampling	28
		6.2.3.2 Borehole Abandonment	30
		6.2.3.3 Soil Classification and Logging	31
		6.2.4 Piezometer Installations	31
		6.2.4.1 PVC Piezometer Installations	31
		6.2.4.2 Piezometer Development	33
	6.3	Results	33
		6.3.1 Site Stratigraphy	34
		6.3.1.1 Surficial Clay	34

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6.3.1.2 Claystone 35
6.3.1.3 Limestone 36
7.0 GEOTECHNICAL REPORT 37
7.1 Geotechnical Soil Sampling Procedures 37
7.2 Laboratory Testing Program and Summary of Geotechnical Test Results 37
7.2.1 Percent Fines (ASTM D 1140) 39
7.2.2 Soil pH (ASTM D 4972) 39
7.2.3 Organic Content (ASTM D 2974) 39
7.2.4 Sieve and Hydrometer (ASTM D 422) 40
7.2.5 Atterberg Limit Determinations (ASTM D 4318) 40
7.2.6 Moisture Content Determinations (ASTM D 2216) 40
7.2.7 Density (ASTM D 698) 41
7.2.8 Recompacted Permeability (ASTM D 5084) 41
7.2.9 Permeability (ASTM D 5084 and ASTM D 4525) 42
7.2.10 Unconfined Compressive Test (ASTM D 2166) & Uniaxial Compression Test (ASTM D 2938) 43
7.2.11 Specific Gravity (ASTM D 854) 43
7.2.12 Porosity and Void Ratio 44
7.2.13 Rock Quality Designation (RQD) 44
7.3 Geotechnical Analyses 44
7.3.1 Classification 44
7.3.1.1 Soil Classification 44
7.3.1.2 Rock Classification 46
7.3.2 Stability 47
7.3.3 Liner Soils 47
7.3.4 Excavation Considerations 47
7.4 Geotechnical Conclusions 48
8.0 GROUNDWATER INVESTIGATION 48
8.1 Purpose 48
8.2 Summary of Activities 49
8.3 Methodology 50
8.3.1 Subsurface Investigation 50
8.3.2 Geotechnical Investigation 53
8.3.3 Groundwater Investigation 54
8.3.3.1 Piezometer Installation 54
8.3.3.2 Groundwater Elevation Monitoring 55
8.3.3.3 In-Situ Permeability Tests 55
8.4 Investigation Results 57
8.4.1 Geologic Setting 57
8.4.2 Groundwater Occurrence 58
8.4.3 Groundwater Quality 60
8.4.4 Recharge/Discharge 60
8.4.5 Groundwater Level Fluctuation 62
8.4.6 Hydraulic Conductivity 62
8.4.6.1 Slug Tests 62
8.4.6.2 Laboratory Permeability 63
8.4.7 Hydraulic Head Relationships 63
8.4.7.1 Horizontal Hydraulic Gradient 63

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Geology
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LICENSED PROFESSIONAL GEOSCIENTIST
12-19-2011

8.4.7.2 Vertical Hydraulic Gradient 64
8.4.8 Average Linear Flow Velocity 64
8.4.9 Potential Migration Pathways 65
9.0 GROUNDWATER MONITORING SYSTEM 66
9.1 Proposed Groundwater Monitoring Program 66
9.2 Monitoring Well Construction 69
9.3 Background Monitoring 70
9.4 Detection Monitoring 71
10.0 REFERENCES 72
11.0 STANDARD OF CARE 74

TABLES

Table III-4.1 Regional Geologic Formations
Table III-4.2 Regional Hydrogeologic Units
Table III-4.3 Summary of Borings
Table III-4.4 Summary of Previous Monitoring Well and Piezometer Construction Details
Table III-4.5 Summary of Present Monitoring Well and Piezometer Construction Details
Table III-4.6 Summary of Site Stratigraphic Units
Table III-4.7 Summary of Geotechnical Testing Results for Surficial Clay
Table III-4.8 Summary of Geotechnical Testing Results for Claystone
Table III-4.9 Summary of Geotechnical Testing Results for Limestone
Table III-4.10 Summary of Surficial Clay/Claystone Groundwater Elevation Data
Table III-4.11 Summary of Claystone/Limestone Groundwater Elevation Data
Table III-4.12 Summary of In-Situ Hydraulic Conductivity Test Results
Table III-4.13 Summary of Horizontal Hydraulic Conductivity Gradient Calculations
Table III-4.14 Summary of Vertical Hydraulic Conductivity Gradient Calculations
Table III-4.15 Summary of Average Linear Flow Velocity Calculations
Table III-4.16 Recommended Groundwater Monitoring Well Network

FIGURES

12-19-2011

Title and Index
Figure III-4.1 Site Location Map
Figure III-4.2 Topographic Map
Figure III-4.3 2002 Aerial Photograph
Figure III-4.4 Central Texas Physiographic Province Map
Figure III-4.5 Regional Geological Map
Figure III-4.6 Regional Cross-Section of Geologic/Hydrogeologic Units
Figure III-4.7 Stratigraphic Profile of Edwards Limestone
Figure III-4.8 Regional Structure Map
Figure III-4.9 Seismic Impact Zones Map, Peak Acceleration (% g) with 2% Probability of Exceedance in 50 Years
Figure III-4.10 Wetland Delineation Map

FIGURES (continued)

Figure III-4.11 Approximate Altitude of Water Levels in Edwards Aquifer, 1997-98
Figure III-4.12 Monitoring Well, Piezometer, and Boring Location Map
Figure III-4.13 Typical Piezometer Detail
Figure III-4.14 Cross Section Location Map
Figure III-4.15 Cross Section Legend
Figure III-4.16 Cross Section A-A'
Figure III-4.17 Cross Section B-B'
Figure III-4.18 Cross Section C-C'
Figure III-4.19 Surficial Clay Isopach Map
Figure III-4.20 Coarse Grained Isopach Map
Figure III-4.21 Top of Claystone
Figure III-4.22 Claystone Isopach Map
Figure III-4.23 Top of Limestone
Figure III-4.24 Surficial Clay/Claystone Potentiometric Surface (Shallow)
Figure III-4.25 Claystone/Limestone Potentiometric Surface (Deep)
Figure III-4.26 Historical High Groundwater Map of Surficial Clay/Claystone (Shallow)
Figure III-4.27 Historical High Groundwater Map of Claystone/Limestone (Deep)
Figure III-4.28 Recommended Groundwater Monitoring System

APPENDICES

Appendix III-4A Aerial Photographs
Appendix III-4B Water Well Reports
Appendix III-4C Boring Logs
Appendix III-4D Abandoned Piezometers and Monitoring Wells
Appendix III-4E Texas Plugging Reports
Appendix III-4F Current Piezometer and Monitoring Well Construction Diagrams
Appendix III-4G Geotechnical Laboratory Test Results
Appendix III-4H Slug Test Plots
Appendix III-4I Documentation of Certification

12-19-2011

1.0 INTRODUCTION

The Texas Commission on Environmental Quality (TCEQ) Municipal Solid Waste Management Regulations (MSWMR) require that the owner or operator of a municipal solid waste (MSW) facility that disposes solid waste resulting from or incidental to municipal, community, commercial, institutional, and recreational activities, including garbage, rubbish, ashes, street cleanings, dead animals, abandoned automobiles, and all other solid waste other than industrial solid waste submit a Geology Report, Attachment 4 as part of a Permit Application submittal package.

Waste Management of Texas, Inc. (WMTX) is submitting a Permit Amendment Application for Williamson County Recycling and Disposal Facility (Site), a Type I MSW facility. This report represents Attachment 4 of the permit submittal package as required by Title 30 of the Texas Administrative Code (TAC) Chapter (§)330.56(d). This report summarizes the regional and Site-specific geology and hydrogeology and includes the Subsurface Investigation, Geotechnical, and Groundwater Investigation reports. This report includes portions of Attachment 11 of the Permit Amendment Application No. 1405A that was submitted in September 1994 to the TCEQ (formerly, the Texas Natural Resource Conservation Commission) by Cook-Joyce, Inc. (CJI) and McCulley, Frick & Gilman, Inc. (MFG) on behalf of Williamson County. The attachment was revised on February 17, 1995 for final submittal to the TCEQ.

1.1 Background

The Site is a Type I MSW landfill under authority of Permit No. 1405A issued by the TCEQ on August 4, 1995. The Site is wholly owned by Williamson County and is located northeast of the city of Hutto in Williamson County, Texas. The existing landfill portion of the Site is composed of approximately 202 acres as shown on Figure III-4.1. Access to the Site is from the east via Farm to Market Road (FM) 1660. The Site serves Williamson County and surrounding areas with an equivalent population of approximately 360,000 persons.

The Site encompasses a total of 572 acres of which 202 acres are presently permitted for landfill operations. The permitted portion of the Site includes 80 acres of undeveloped acreage to the north of landfill operations. The remaining property outside the permitted landfill boundaries includes expansion acreage to the east (370 acres) that includes the area evaluated during this investigation. The Site is located within several tracts of land to include: J. F. Ferguson Survey (201.392 acres), J. Dykes Survey (121.190 acres, 60.0 acres, and 201.54 acres), and J. Thomas Survey (1.448 acres), Williamson County, Texas (WCAD, 2004). A general Site location map is included as Figure III-4.1.

The Site accepts municipal solid waste as defined by 30 TAC §330.2 to include "solid waste resulting from or incidental to municipal, community, commercial, institutional, and recreational activities, including garbage, rubbish, ashes, street cleanings, dead animals, abandoned automobiles, and all other solid waste other than industrial solid waste." The Site also accepts Class 2 and Class 3 industrial waste as defined in 30 TAC §330.2, in accordance with 30 TAC §330.137. Regulated industrial hazardous waste, Class 1 industrial waste, and radioactive waste are not accepted for disposal at the Site with the exception of Class 1 industrial solid waste that is Class 1 only because of asbestos content. On an average, the Site receives approximately 1,000 tons of municipal solid waste per day.

Williamson County initiated waste disposal activities at the Site in 1983. In 1984, the County chose to contract with a private company for the daily operations of the landfill. In 1987, the day-to-day operation of the Site was transferred to WMTX by a contract to lease and operate the Site. WMTX continues to operate the Site.

Williamson County began the operations of the Site as a Pre-Subtitle D facility. In-situ clay and/or recompacted clay liners were constructed from the initial cell development through the implementation of the Subtitle D requirements for composite liners with leachate collection systems. The Site is equipped with a groundwater monitoring system, a perimeter gas monitoring system, and gas utilities venting systems.

The Site has received numerous Class I modifications from the TCEQ. These modifications have ranged from revision of existing plans to the addition of certain operations (composting and mulching, scales, wind turbines) to changes to the base grading plan, final cover design, and sequence of operations.

Under this application, Williamson County proposes to expand the Site laterally and vertically. The lateral expansion will increase the actual limit of waste from approximately 160 acres to approximately 500 acres. The permitted area will increase from approximately 202 aces to approximately 575 acres as shown on Figures III-4.2 and III-4.3. The remaining acreage of the permitted area will be utilized for buffer zones, perimeter access roads, drainage facilities, miscellaneous equipment/supplies storage, daily and final cover stockpiling, and other operational uses.

The vertical expansion will increase the maximum height of the landfill from its current permitted height of 766 feet mean sea level (msl) to approximately 840 feet msl. The landfill will incorporate four horizontal to one vertical (4:1) sideslopes along the outer slopes of the existing landfill, transitioning to 5:1 sideslopes for the expansion area. The expanded landfill will have a four percent topslope.

The western portion of the current landfill footprint has been filled with waste and the southern portion of the existing constructed landfill is presently receiving waste.

2.0 PHYSIOGRAPHY AND TOPOGRAPHY

This section provides information on regional physiography, topography, and surface-water drainage under 30 TAC §330.56(d)(1).

2.1 Regional Physiographic Setting

The Site is located in south-central Williamson County and is situated in the Grand Prairie Physiographic Province. The Grand Prairie Physiographic Province (a.k.a., Rolling Prairie) is the easternmost portion of the Edwards Plateau and the northeast-trending Balcones Fault Zone, separating the Blackland Prairie Physiographic Province to the east and the Jollyville Plateau to the west (Senger et al, 1990). The Blackland Prairie, the westernmost portion of the West Gulf Coastal Plain Physiographic Province and the Jollyville Plateau, represents the undissected portion of the Edwards Plateau of south-central and southwest Texas. Typical topographic expression of the area surrounding the Site is characterized by rolling to gently rolling hills. A physiographic province map of the Central Texas area, along with the general location of the Site, is shown on Figure III-4.4.

The Grand Prairie Physiographic Province forms an elongated area of grassland country that is underlain by thin, stony, gently sloping to sloping soils (USDA SCS, 1983). These soils formed in limestone or limestone and marl of upper Cretaceous rocks, which outcrop in the region. The soils are typically fertile and have a high water capacity; therefore, the soils are suitable for agricultural uses (CJI and MFG, 1995). The Lower Cretaceous (Comanchean) "hard" limestones with interbedded marly clays are in contrast with the "soft" rocks and calcareous clays underlying the Blackland Prairie to the east. The landscape features of the Grand Prairie are similar to those of the Blackland Prairie, although the surface features of the former are sharper and more acute in outline than the typically smooth-featured, roundly rolling to undulating topography of the Blackland Prairie (Senger et al, 1990). The Grand Prairie Physiographic Province is cut across by streams with meandering channels, which were acquired during a previous cycle of topographic development and are rather deeply incised below the regional level of the area. The strata at the surface dip perceptibly eastward.

The drainage divide between the Colorado River to the south and the Brazos River to the north coincides with the Travis-Williamson county line. Therefore, only a relatively narrow recharge zone exists on the Jollyville Plateau for groundwater that flows toward the Colorado River. The major creeks in the

northern segment of the Edwards aquifer (northern Travis and Williamson counties) form a low-gradient drainage network within the Brazos River drainage basin (Senger et al, 1990).

2.2 Topography

The topography of the area surrounding the Site is dominated by gently sloping valleys trending from a topographic high on the western permit boundary near the northwest corner of the original 122-acre landfill. The valleys slope to the north, east, west, and south. The natural ground elevations range from a low of approximately 670 feet msl on the northwestern expansion property and 672 feet msl on the southwestern boundary to a high of approximately 730 feet msl on the northern half of the west boundary, as shown on Figure III-4.2. The surface slopes range from approximately 0.009 feet per foot (ft/ft) to approximately 0.03 ft/ft in various directions across the Site, with an overall surface dip to the east and north (CJI and MFG, 1995).

2.3 Surface Water Drainage Features

Williamson County is drained in the center and south by the San Gabriel River, which is the only river in the county. The San Gabriel River, which includes the North, Russell, Middle, and South forks, is part of the Brazos River drainage basin and is located approximately three miles north of the Site, as shown on Figures III-4.1 and III-4.3. The river flows from Burnet County eastward through Williamson County to its confluence with the Little River in Milam County, for a total length of 120 miles. There are two large dams on the San Gabriel River; one that is eight miles west of Granger and approximately 16 miles northeast of the Site, forms Granger Lake, which has a pool elevation of 512 feet msl. The other dam, four miles upstream from Georgetown and approximately 10 miles northwest of the Site, forms Lake Georgetown, which has a pool elevation of 791 feet msl. Major tributaries of the San Gabriel are Brushy, Alligator, Opossum, Berry, Pecan, Little, and Oatmeal creeks (Texas Online, September 2004).

Surface water drainage features closest to the Site include Brushy Creek, Mustang Creek, and an unmanned tributary of the San Gabriel River. Brushy Creek in Williamson County rises three miles northwest of Leander in southwestern Williamson County and flows east for sixty-nine miles, through southern Williamson County and southwestern Milam County, to its mouth on the San Gabriel River, five miles north of Rockdale and approximately 33 miles to the northeast of the Site. Brushy Creek flows southeast approximately four miles south of the Site along the Williamson – Travis county line. Mustang Creek, one of the tributaries of Brushy Creek, rises four miles northwest of Hutto in south central Williamson County (at 30°36' N, 97°34' W) and runs southeast for twenty miles to its mouth on Brushy

Creek, six miles southeast of Taylor (at 30°32' N, 97°18' W). Its banks are wooded in places with mesquite, oak, and juniper trees, and the creek flows through flat to rolling terrain surfaced by clayey soils used predominantly to grow crops (Texas Online, September 2004). Mustang Creek, is an intermittent stream that flow from northwest to southeast across the southern portion of the permitted landfill.

The Site overlies the drainage divide between Mustang Creek and an unnamed tributary to the San Gabriel River, which is located in the northern portion of the permitted landfill. Surface drainage from more than three-fourths of the Site is southwest toward Mustang Creek, and the remainder of the drainage is to the north, toward the unnamed tributary of the San Gabriel River (Hydro-Search and MFG, 1990). These features can be seen on the 2002 aerial photograph included as Figure III-4-3.

3.0 REGIONAL GEOLOGY

This section describes the regional and structural geology surrounding the Site and provides information as required under 30 TAC §330.56(d)(2). The information has been compiled from review of available published literature and university research.

3.1 Stratigraphy and Lithology

Per 30 TAC §330.56(d)(2)(B), information pertaining to geologic age, lithology, variations in lithology, thickness, depth, geometry, hydraulic conductivity, and depositional history are presented for the Site, based upon available geologic information, for each geologic unit from the base of the lowermost aquifer capable of providing usable groundwater, or from a depth of 1,000 feet, whichever is less, to the land surface. Data provided on the various geologic units are based on regional studies and may not specifically reflect localized information obtained from subsurface investigations conducted at the Site.

Regional geology of the Williamson County area consists of Quaternary-age terrace deposits, Upper Cretaceous limestones, marls and chalks, and Lower Cretaceous limestones and dolomites. Figure III-4.5 is a regional geological map showing the location of the Site with respect to the surface exposure of major stratigraphic units, which outcrop in the area. A general vertical profile of geologic strata is illustrated in the regional cross section in Figure III-4.6.

The Site is located at the western limits of Quaternary System, Pleistocene-age gravel deposits also defined as Older Alluvium. These gravel deposits are thin sheets of gravel and sand from old river terraces, usually 20 feet or less in thickness, and are often mixed with the underlying clays of the Upper

Cretaceous Taylor and Navarro Groups by plowing. The Older Alluvium is composed of gravel, sand, clay, well-rounded pebble- to cobble-sized gravel, few boulders, mostly chert and limestone, some quartz and igneous and metamorphic rock detritus (BEG, 1997). The alluvium has moderate to high infiltration capacity allowing for adequate drainage of groundwater most of the time, except during extended wet periods when the unit may become saturated (Garner and Young, 1976). Thicknesses of the Quaternary deposits in Williamson County are as great as 36 feet and may be thicker in some locations (Senger et al, 1990). At the Site, the Quaternary deposits have an average thickness of 31 feet.

According to the Geologic Map of Hutto Area (1974), alluvium of more recent age is located in the southwestern portion of the Site along the banks of Mustang Creek. The alluvium consists of limestone and chert gravel, sand, silt, and clay and the deposits are thin and spatially narrow (Marks, 1950; Garner and Young, 1976).

Below the veneer of alluvium is an eastward thickening wedge of progressively younger rocks that dip gently southeast toward the Gulf Coast. The uppermost geologic unit in the vicinity of the Site is the Cretaceous-age Lower Taylor Marl, locally referred to in part as the Sprinkle Formation or Sprinkle Claystone. The Lower Taylor Marl also has been classified as the Lower Taylor Group, the Pecan Gap Chalk, or the Ozan Formation (BEG, 1974). The Lower Taylor Marl consists of marly montmorillonitic clay and claystone, with some glauconite, phosphate pellets, and hematite and pyrite nodules (BEG, 1974). The calcareous content decreases upward through the approximate 600 feet of total thickness (BEG, 1974). The Lower Taylor Marl contains megafossils in the area of the Site to include Exogyra ponderosa, Exogyra ponderosa var. erraticostata, Ostrea falcate, Anomia argentaria, and Hamulus onyx . The Lower Taylor Marl also contains microfossils to include Pseudoclavulina clavata, Nodosaria amphioxys, and Planulina taylorensis (Marks, 1950). The Lower Taylor Marl has a low infiltration capacity that generally does not transport groundwater through the unit unless fractures are aligned. At the Site, the Lower Taylor Marl has an average thickness of 24 feet.

The Lower Taylor Marl conformably overlies strata of the Austin Chalk or Austin Group. The Austin Group has a reported total thickness of 325 to 420 feet. The Austin Group is approximately 390 feet thick at the Site, based on water well logs within one mile of the Site. Frank G. Bryant and Associates, Inc. (1985) tentatively identified the strata underlying the Lower Taylor Marl at the Site as the equivalent to the Pflugerville Formation, which is the uppermost (youngest) formation of the Austin Group. In the Austin area, the Pflugerville Formation is described as a chalky and clayey limestone approximately 40 feet thick (Garner and Young, 1976). Young and Woodruff (1985) describe this formation as consisting

of slightly clayey to clayey limestone. Several other formations of the Austin Group occur below the Pflugerville Formation. These include the Burditt, Dessau, Jonah, Vinson, and Atco formations (BEG, 1997). All of these formations, or their stratigraphic equivalent facies, are included in the Austin Group with the Pflugerville Formation. These Austin Group formations are collectively referred to and mapped as the Austin Chalk in Williamson County. They are composed of thin assemblages of chalk, limestone, and marl, bentonite seams locally recessive, medium gray with pyrite nodules common (BEG, 1974). According to the Geologic Map of Hutto Quadrangle (1997), the Austin Group occurs on the surface in the northwestern portion of the Site along the banks of the unnamed tributary of the San Gabriel River. The Austin Chalk contains fossils in the area of the Site to include Exogyra ponderosa, Exogyra tigrina, Exogyra laeviuscula, Ostrea travisana, Hemiaster texanus, Inoceramus undulatoplicatus, Gryphaea wratheri, and Inoceramus subquadratus (Marks, 1950). The Austin Group has a low to moderate infiltration capacity that generally does not transport groundwater through the unit unless fractures are aligned. Matrix permeability is low (0.03 to 1.27 millidarcys) (Hovorka, 1998).

Immediately below the Austin Group is the Eagle Ford Group, representing the oldest rocks of the Upper Cretaceous System. The Eagle Ford consists of massive, calcareous shale with thin interbeds of silty, sandy, and flaggy limestone (Brune and Duffin, 1983). The unit primarily contains montmorillonitic clay and is nearly impermeable. The Eagle Ford Group only has a low infiltration capacity if open joints exist. Several thin (0.4 to 3 inches) bentonite beds also may occur in the middle part of the unit (Garner and Young, 1976). Its maximum thickness is approximately 65 feet in Williamson County (BEG, 1997). The Eagle Ford is approximately 70 feet thick at the Site, based on water well logs within one mile of the Site.

The Eagle Ford Group overlies rocks of the Washita Group, the youngest rocks of the Lower Cretaceous. The Washita Group consists of the Buda Limestone, the Del Rio Clay, and the Georgetown Formation. The Buda Formation in Williamson County consists of a lower, slightly glauconitic, fossiliferous limestone and an upper, hard, resistant, burrowed, fossiliferous, shell-fragment limestone (Senger et al, 1990) that has a low to high infiltration capacity (Garner and Young, 1976). The formation thins northward across the area from approximately 30 feet to less than 3 feet. Buda limestone is absent at several places north of the Site and the San Gabriel River and is interpreted where Buda strata are absent as being structurally high or interpreted as pre-Eagle Ford erosion of the unit (Senger et al, 1990). Undivided Quaternary surficial material covers much of the area, so it is also possible that the Buda was eroded from the area during the Quaternary (Senger et al, 1990). The Buda is approximately 30 feet thick at the Site, based on water well logs within one mile of the Site.

The Del Rio Clay is mostly fossiliferous clay and marl, with some gypsum, pyrite, siltstone, and sandstone. The fossil, Exogyra arietina, is abundant in the clay. Unweathered Del Rio Clay is composed of kaolinite, illite, and subordinate amounts of montmorillonite. During weathering, illite alters to montmorillonite. Weathered Del Rio Clay contains only small quantities of illite and greater amounts of montmorillonite and generally has a low infiltration capacity (Garner and Young, 1976). The Del Rio Clay is approximately 70 feet thick at the Site, based on water well logs within one mile of the Site, and serves as the confining bed for the Edwards aquifer.

The Georgetown Formation consists of richly fossiliferous limestones and marls, and has an approximate maximum thickness of 100 feet (Brune and Duffin, 1983) and a low to moderate infiltration capacity (Garner and Young, 1976). The Georgetown limestones contain marine megafossils that include Kingena wacoensis and Gryphaea washitaensis. Small vugs may occur within some beds but generally are not common. The contact between the Georgetown Formation and overlying Del Rio Formation is gradational (Senger et al, 1990). The Georgetown Formation is considered the uppermost member of a group of rocks referred to as Edwards and associated limestones. The Georgetown Formation is approximately 90 feet thick at the Site, based on water well logs within one mile of the Site.

The Edwards and associated limestones represent the upper portion of the Fredricksburg Group and the lower portion of the Washita Group. Collectively these limestones are regarded as the Edwards aquifer, the principal aquifer for a large region of Central Texas. The formations of the Edwards aquifer are the Georgetown Limestone, the Edwards Limestone, the Comanche Peak Limestone, and the Walnut Formation. These rocks are generally limestone, but also include marl, clay, dolomite, gypsum, and anhydrite that have infiltration capacities ranging from low to high. The approximate depth to the top of the Edwards aquifer in the general area of the Site is 600 to 700 feet (Senger et al, 1990). Based on water well logs within one mile of the Site, the Edwards Group occurs at depths ranging from 680 to 710 feet.

The water-bearing limestones that compose the Edwards aquifer in the vicinity of the Site lie within the Georgetown (see above), Edwards, and Comanche Peak strata; however, the Edwards Limestone contains most of the aquifer. The contacts between the Georgetown, Edwards, and Comanche Peak formations do not reflect major lithologic changes. The Georgetown Formation unconformably overlies Edwards strata.

The Edwards Limestone consists of massive- to thin-bedded limestones and dolostones with reef material consisting of shell fragments and rudistid reefs and exhibits moderate to high infiltration capacities. The sequence is about 300 feet thick in the Austin area and thins northward to about 100 feet in southern Bell

County, north of Williamson County, as seen on Figure III-4.7 (Senger et al, 1990). The Edwards is a transgressive facies representing a reef-lagoon complex.

The Comanche Peak Formation, a nodular limestone and marl sequence, is about 40 feet thick in southern Williamson County. The Comanche Peak has transitional contacts with the Walnut and Edwards sequences. Comanche Peak lithology and fauna indicate a widespread, uniform, marine depositional environment (Senger et al, 1990).

The Walnut Formation, composed of limestone and marl, underlies the Edwards aquifer strata. The formation is a transgressive facies deposited in a shallow-marine environment. The Walnut sequence in the area of the Site has been subdivided into six members: the Bull Creek limestone, Bee Cave marl, Cedar Park limestone, Whitestone limestone, Keys Valley marl, and the upper marl (Senger et al, 1990). The Whitestone and Keys Valley Members overlie these strata in southern Williamson County. Toward the north, in Williamson and Bell Counties, only the Keys Valley marl overlies the three older members. Individual Walnut members are approximately 30 to 50 feet thick. The Walnut sequence is generally not a water-bearing unit (Senger et al, 1990).

3.2 Regional Structural Geology

The Site lies approximately 15 miles northwest of the boundary between rocks of the Cenozoic and Mesozoic Eras (specifically rocks of the Tertiary and Cretaceous Systems, Table III-4.1. This geologic boundary is coincident with the physiographic boundary between the Edwards Plateau to the west and the Blackland Prairie to the east. The boundary between the Cretaceous and Tertiary rocks is structurally controlled by the Balcones fault zone, which trends north-northeast through central Texas and the eastern edge of which is located five miles to the west of the Site. In Cenozoic time, readjustment of previously deposited sediments resulted in the extensive faulting within the Balcones fault zone.

The Edwards aquifer straddles the Balcones Fault Zone. The fault zone appears to have a more complex pattern in the vicinity of Austin and a less complex pattern northward toward Williamson County, as seen on Figure III-4.8 (Senger et al, 1990). The major faults generally strike 000° to 040° and dip between 40° and 80°. The net throw across the fault zone is down toward the east, although faults dip both eastward and westward. Cross faults that strike sub-perpendicular to the northward-striking structures have been mapped in the Austin area, but were not recognized in the Georgetown area, possibly because of limited exposures. They may exist in the Georgetown area because some minor faults strike westward (Senger et al, 1990). Near Austin, the fault zone is generally composed of one large fault (Mount Bonnell fault) that

has a throw of about 715 feet and many smaller faults. Eastward, the largest fault in Austin other than the Mount Bonnell fault has a maximum throw of about 300 feet, and most mapped faults have displacements of less than 50 feet (Woodruff and Collins, 2001). The Mount Bonnell fault trends along the west margin of Austin, across south and central Travis County. North of the Mount Bonnell fault, in north Travis County, the main fault of the en echelon fault system has a throw of approximately 260 feet. Breccia zones, nearly vertical dipping strata, and abundant minor faults and joints occur adjacent to the major fault (Woodruff and Collins, 2001). Northward in Williamson County, the maximum displacement of individual faults is about 150 feet, and displacements on most of the major faults are estimated to be less than 40 feet (Woodruff and Collins, 2001). Gentle flexures that dip less than 10 degrees eastward also occur in the area. Minor faults with throws less than seven feet appear to occur most abundantly near major faults and flexures (Senger et al, 1990).

4.0 ACTIVE GEOLOGICAL PROCESSES

This section summarizes and describes the active geologic processes in the vicinity of the Site in accordance with 30 TAC §330.56(d)(3). This description of geologic processes presented below includes information on (1) faulting and seismic impacts, (2) unstable areas, (3) erosion potential, (4) wetlands, and (5) differential subsidence as they relate to development of the Site.

4.1 Holocene Faults and Seismic Impact

In accordance with 30 TAC §330.303(a), "*New municipal solid waste landfill (MSWLF) units and lateral expansions shall not be located within 200 feet of a fault that has had displacement in Holocene time unless the owner or operator demonstrates to the executive director that an alternative setback distance of less than 200 feet will prevent damage to the structural integrity of the MSWLF unit and will be protective of human health and the environment.*" The Holocene Epoch, extending from the end of the Pleistocene Epoch to the present represents the most recent 11,000 years of the Quaternary Period.

4.1.1 Holocene Faults

In order to address the proximity of any potential Holocene faults to the Site, the following resources were investigated: 1) published information on the structural and seismic history of the Williamson County area; 2) documented locations of seismic epicenters in recent times; 3) evidence of displacement of surficial deposits observed during field reconnaissance; and 4) evaluation of lineaments on aerial photographs and topographic maps.

The Austin Chalk, intensively fractured and faulted to the west of the Site, is a prominent rock unit along the coastward portion of the Balcones/Ouachita trend of central Texas (Figure III-4.8). Both the surface and subsurface trends follow the western and northern margins of the Gulf Coast Embayment and its outcrop is coincident with the eastern thrust zone of the buried Ouachita structural belt (Young and Woodruff, 1985). This fault system parallels the structural grain of the Paleozoic Ouachita deformed belt, which is exposed to the northwest in the Llano Uplift and Texas Hill Country. It may be inferred, but cannot be proved, that the location and trend of the Balcones Fault system is the result of an existing weak zone in the underlying Ouachita basement rocks. Regardless of any basement control, the general style of faults in the Balcones Fault system indicates that they formed in response to down-to-the-coast extension of the sedimentary basin (Corbett, et al, 1991). In this respect, they are similar to other, younger faults in the Gulf Coast region. The age of Balcones system faulting is not tightly constrained, as no crosscutting stratigraphic relationships have been preserved. All sediments younger than Cretaceous and older than Quaternary have been removed by erosion. Therefore, the age is established by analogy with the tectonically similar subsurface Luling, Mexia, and Talco fault zones (Figure III-4.8). These zones affect rocks as young as Oligocene but apparently do not offset Miocene strata (Corbett, et al, 1991). It is generally accepted that the Balcones faulting occurred in the Miocene Epoch, because of the presence, a few miles east of Austin, of Miocene-age conglomerates, containing clay, sand and pebbles derived from the initial fault scarp. Erosion acts at an accelerated pace on fault scarps, explaining the origin of the conglomerates by which Balcones faulting was dated. All the evidence indicates that Balcones faulting has ceased (Young et al., 1977).

Normal faults of the Balcones Fault Zone are abundant in the Austin/Williamson County area. Most of these faults strike northward to northeastward and dip toward the east and southeast, although some dip westward. Faults have broken the relatively narrow (<1.5-mile wide) outcrop belt of these strata into a series of fault-bound blocks, producing a regional-scale outcrop pattern that steps northwestward (Figure III-4.5). Most blocks are bound by southeastward-dipping faults, although some grabens formed by northwest-dipping faults also exist. Geologic maps illustrate that relatively closely spaced faults also are very common within the blocks of the Upper Cretaceous Del Rio-Buda-Eagle Ford strata (Woodruff and Collins, 2001).

The closest documented fault is found on a geologic map of the Jonah quadrangle prepared by Marks (1950). The surface trace of the inactive Jonah Fault trends southwest-northeast about 1,750 feet northwest of the northern permitted landfill property. No faults were mapped by Marks (1950) in the immediate vicinity of the Site.

An inactive, north-northwest trending, high angle normal fault is shown through the southeast corner of the existing landfill permit area on regional geologic maps (Geologic Atlas of Texas - Austin Sheet) and also shown on a top-of-Edwards aquifer structure map (Senger et al, 1990) (see Figure III-4.5). The vertical displacement indicated on the map is relatively small (less than 30 feet). Field observation and examination of aerial photographs showed no surficial evidence of faulting or subsidence. The Texas Department of Health (TDH) ran resistivity profiles and four additional boreholes were drilled by Frank G. Bryant and Associates, Inc. in late 1981 to investigate the occurrence of this fault within the landfill property. The evaluation of the resistivity data was reported to be inconclusive and the evaluation of the borehole data concluded that there was no evidence of a fault. Stratigraphic data collected from earlier studies were integrated and evaluated to assess the possible existence of the fault. Again, no evidence of fault displacement was observed (CJI and MFG, 1995). The disparity between the mapped locations of this fault may be related to the small scale of the Austin Sheet and the top-of-Edwards structure relative to the more detailed work by Marks (1950). In summary, although shown on some regional geologic maps, field investigations have shown that a fault does not exist at the Site (CJI and MFG, 1995). Additionally, no faults since the Miocene Epoch are known to have been active in the vicinity of the Site or along the Balcones Fault Zone, the eastern edge of which is located five miles to the west of the Site.

Existing borehole data, field observation, and previous examination of aerial photographs showed no evidence of faulting at the existing Site. A recent review of aerial photographs was conducted as part of the fault study in September 2004. Faults may be manifested on aerial photographs as linear features (photo-linears) resulting from differences in color tone or shading caused by natural features, such as water bodies or changes in vegetation, associated with the fault scarp. A review of black and white aerial photographs of the Site for the years 1941, 1953, 1974, 1988, 1997, 2002, and 2004 showed **no** photo-linears in the area of the Site. Appendix III-4A contains the aerial photographs reviewed.

Based on previous fault studies and a recent review of additional references, **no** surface expressions or differential subsidence that has had displacement in Holocene time were identified within 200 feet of the Site. As stated in Attachment 11 of the June 1995 Permit Amendment Application, the Site is located in a geologically stable area that is not subject to active geological faulting. The Site is underlain by rocks of Upper and Lower Cretaceous age (at least 65 million years old) and has not been subject to active faulting since Miocene time (five to 24 million years ago) (CJI and MFG, 1995).

4.1.2 Seismic Impact Zones

In accordance with 30 TAC §330.304, *"New municipal solid waste landfill units and lateral expansions shall not be located in seismic impact zones, unless the owner or operator demonstrates to the executive director that all containment structures, including liners, leachate collection systems, and surface water control systems are designed to resist the maximum horizontal acceleration in lithified earth material for the Site."* As defined in 30 TAC §330.304, *"a seismic impact zone is an area with a 10 percent or greater probability that the maximum horizontal acceleration in rock, expressed as a percentage of the earth's gravitational pull, will exceed 0.10g in 250 years. Maximum horizontal acceleration is defined as the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90% or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment."*

A search of documented seismic epicenters located within a 100-mile (161 kilometer) radius of the Site was conducted using the USGS electronic bulletin board system, operated by the National Earthquake Information Center in Denver, Colorado. Data were researched in the Preliminary Determination of Epicenters, an electronic database containing seismic epicenters recorded from 1534 through 2004. Twelve earthquake centers were identified within the 100-mile search radius. Nine of the earthquake epicenters occurred between the years of 1847 through 1914 and three were recorded in 1982, 1992, and 1995. All epicenters were located greater than 17 miles west of the Site and had magnitudes ranging from 2.7 to 3.6 on the Richter scale.

The seismic risk map for the United States and the seismic history of central Texas suggests a very low risk of an earthquake occurrence of a sufficient magnitude to present a problem at the Site. Central Texas is located within one of the least seismically active areas in the United States. According to the 250 year, 90 percent non-exceedance probabilistic seismic risk map (Algermissen et al, 1990), the Site lies within the 0.04g zone (Figure III-4.9).

Available data from the USGS Seismic Hazard Mapping Project (Frankel et al, 1996) indicates that the maximum horizontal acceleration anticipated at the Site (2 percent probability of exceedance in 50 years) is approximately 0.0375 g based on the center latitude and longitude coordinates of the Site. Therefore, the Site is **not** considered to be located within a seismic impact zone (Figure III-4.9).

4.2 Subsidence

In accordance with 30 TAC§330.303(b), *"Applications submitted for the operation of sites located within areas that may be subject to differential subsidence or active geologic faulting shall include fault studies. When an active fault is known to exist within one-half mile of the site, the site shall be investigated for unknown faults. Areas experiencing withdrawal of crude oil, natural gas, sulfur, etc., or significant amounts of groundwater shall be investigated in detail for the possibility of differential subsidence or faulting that could adversely affect the integrity of landfill liners. "* As documented in Section 4.1.1 of this report, the Site is **not** located within five miles of an active geologic fault; only inactive faults have been recognized but not confirmed in close proximity to the Site.

The Edwards Aquifer Homepage states that the limestones of the Edwards aquifer appear to have sufficient bearing capacity so that subsidence will not be a major problem (Eckhardt, 2004). If groundwater withdrawals are uncontrolled, it is unknown whether underground caverns would collapse or flow patterns would change should the level be drawn lower than ever before (Eckhardt, 2004). Based on land subsidence associated with other aquifers, the Edwards Aquifer Authority (EAA) has set a cap on groundwater withdrawal from the Edwards aquifer at 450,000 acre feet per year until 2008, when the cap will be reduced to 400,000 acre feet per year (Eckhardt, 2004).

As stated in Attachment 11 of the June 1995 Permit Amendment Application, Site is located in a geologically stable area that is not subject to differential subsidence. The Site is underlain by rocks of Upper and Lower Cretaceous age (at least 65 million years old) and has not been subject to active faulting since Miocene time (five to 24 million years ago). Existing borehole data, field observation, and examination of aerial photographs showed no evidence of subsidence at the Site.

4.3 Unstable Areas

The location criterion in 30 TAC §330.305 states that *"Owners or operators of new municipal solid waste landfill (MSWLF) units, existing MSWLF units, and lateral expansions located in an unstable area shall demonstrate that engineering measures have been incorporated into the MSWLF unit's design to ensure that the integrity of the structural components of the MSWLF unit will not be disrupted."*

Unstable areas, by definition, are areas susceptible to natural or human-induced events or forces that are capable of impairing the integrity of some or all structural components (i.e. liners, leachate collection systems, final covers, etc.) of a disposal unit. Unstable areas can include karst terrains, poor foundation

conditions, and areas susceptible to mass movement. An evaluation of each of these types of unstable conditions for the Site is presented in the following sections.

4.3.1 Karst Terrain

Development of karst terrain requires the subsurface dissolution of some soluble rock, typically limestone or dolomite. The Site is located at the western edge of Pleistocene-age fluviatile gravel/sand terrace deposits and the eastern edge of the outcropping of the Upper Cretaceous-age Austin Chalk. The uppermost geologic unit in the vicinity of the Site include the Upper Cretaceous-age Lower Taylor Marl and Austin Chalk. The Lower Taylor Marl primarily consists of marly montmorillonitic clays and claystone while the Austin Chalk consists of chalk, limestone and calcareous claystone. Site-specific geology consists of surficial clay ranging in thickness from 4.5 to 60 feet. Underlying this clay is a calcareous claystone, belonging to the Lower Taylor Marl, ranging in thickness from 5.5 to 49.4 feet. A very hard, fossiliferous limestone, belonging to the Austin Chalk, is encountered at depths ranging from 11 to 75 feet below ground surface (bgs) and between 607 and 679 feet MSL.

Although potentially soluble rock (calcareous claystone, limestone) is present, no developed karst features are evident (CJI and MFG, 1995). No indications of surface manifestation of karst topography, such as sinkholes, closed depressions, or other features indicating that karst features have developed, were observed on topographic maps, aerial photographs or during site reconnaissance and exploratory drilling. Available information does not indicate that carbonate and/or calcareous dissolution zones exist in the subsurface that would cause instability at the surface. Therefore, there is **no** potential of excessive differential settlement or foundation failure resulting from the collapse of underground solution voids or the formation of karst terrain.

4.3.2 Poor Foundation Conditions

"Poor Foundation Conditions" by definition in 30 TAC §330.2, are *"areas where features exist which indicate that a natural or man-induced event may result in inadequate foundation support for the structural components of an MSWLF unit."*

To evaluate the presence or absence of poor foundation conditions, visual inspection of the subsurface core was performed along with a review of available subsurface data (i.e., boring logs and geotechnical results for unconfined compressive strength). Based upon this review and field observations, there were **no** features or geotechnical data that would indicate the presence of poor foundation conditions. Also, no evidence of features was discovered that would result in any foundation inadequacies related to the

structural components of the proposed landfill expansion unit or were features discovered that could be considered a feature/catalyst for the potential failure of the structural components of the landfill.

The foundation conditions at the Site are suitable for support of the structural components of the Site. No fill has been identified at the proposed base grade elevations at the Site. The in-situ soils consist of high to moderately plastic clays underlain by a calcareous claystone. The base grades of the disposal areas are founded in the claystone strata.

Differential settling of these soils is not a concern for the following reasons:

- The claystone is an indurated clay, therefore long-term consolidation should be minimal;
- The Site will have a mechanically reworked and compacted clay liner that covers the base grade. Any settlement in the liner from overburden would be expected to be minimal; and,
- The Site stratigraphy is horizontally bedded, continuous, and uniform.

In addition, the Site has been in operation for 20 years, and during that time, there has been no evidence of unstable conditions such as differential settlement.

4.3.3 Areas Susceptible to Mass Movement

Mass movement is a function of the slope of the terrain, soil mass, soil moisture, and soil composition. Based upon a review of the available data, the Site is not located in an area susceptible to mass movement. The Site is not considered to be susceptible to mass movement because of the relatively low relief of the topography, the cohesive nature and consistency of the subsurface sediments, and the stability of the saturated groundwater zone. These conditions combine to make the probability that the landfill will experience damage due to land movement low. The cohesive soils at the Site enable typical cut and/or fill slopes to be constructed at the Site without risk of slope failure. The Site is considered non-susceptible to excessive groundwater drawdown that could result in excessive settlement or bearing capacity failure of the foundation soils (Section 4.2). Refer to the Geotechnical Report for stability and settlement analysis (Section 7.0).

These conclusions, in conjunction with the conclusions presented previously in Sections 4.3.1 and 4.3.2, support our premise that the Site is **not**, nor will it be, subjected to any natural or man-induced events that will result in the development of poor foundation conditions for the Site.

4.4 Erosion

As required by 30 TAC §330.56(d)(3)(B), this section includes a discussion of the potential for erosion due to surface water processes such as overland flow, channeling, and fluvial processes such as meandering streams and undercut banks. Erosion control measures for the Site are discussed in Part III, Attachment 6, Groundwater and Surface Water Protection Plan and Drainage Plan.

The Site has a low potential for erosion due to surface water processes such as overland flow, channeling, gullying, and fluvial processes associated with meandering stream channels (channel migration and undercut banks). The Site is located on a broad, relatively level drainage divide at the headwaters of Mustang Creek, a small ephemeral creek, and one unnamed drainageway of the San Gabriel River. In this physiographic setting, surface water processes have less erosive effects than in downstream areas with larger drainage catchments. Cutbank erosion is not a feature prevalent in the small, ephemeral drainage (Mustang Creek and the unnamed tributary of the San Gabriel River) at the Site. In addition, the gentle slopes and relatively competent soil (high clay content) and rock (claystone) present at or near the surface are not readily erodible.

4.5 Wetlands

As required by 30 TAC §330.53(b)(12)(B), this section includes a discussion of wetlands. Three wetland studies have been conducted at the Site in accordance with the 1987 United States Army Corps of Engineers (USACE) Wetland Delineation Manual (Technical Report Y-87-1). The first study was conducted by MEP Environmental, Inc. (MEP) on the permitted landfill in July 1991. Findings identified six areas at the Site, which appeared capable of demonstrating the three mandatory criteria of hydrophytic vegetation, hydric soils, and wetland hydrology. Area 1 was identified as a depressed section of the northern acreage, approximately 1.9 acres in size, and included a normally dry stream bed which drained the surrounding acreage northward to an existing stock pond. The pond was determined to be exempt from Section 404 regulation and was excluded from the detailed study. Areas 2 through 6 were identified in the southern one-third of the permitted Site, along Mustang Creek. Area 2 (2.1 acres), Area 3 (0.3 acres) and Area 6 (0.1 acres) exhibited the mandatory criteria and qualified as jurisdictional wetlands. Area 4 (0.2 acres) and Area 5 (0.7 acres) did not exhibit all criteria and did not qualify as jurisdictional under Section 404. Area 1 was defined as a wetland, which likely existed because of the construction of the stock pond adjacent to its northern extent. MEP stated that the stock pond had interrupted the natural drainage and created a hydrologic regime which facilitated the development of wetland criteria characteristics. MEP also stated that elimination of the pond and restoration of previous drainage patterns

could result in a natural conversion to upland characteristics. Areas 2, 3, and 6 were stated as being influenced by periodic flooding of Mustang Creek. These areas were defined as riparian woodlands; however, their extents were too small to offer significant function or value as regulatory wildlife habitat. MEP concluded that these areas, along with Area 1 would be subject to approvals if they are to be affected by the discharge of fill as part of the landfill operations. Mustang Creek within- the ordinary high water mark was defined as likely "Waters of the United States" and, as such, is subject to regulation under Section 404.

A second wetlands delineation of the Site was completed in September 1994 by Rust Lichliter/Jameson (Rust) on the existing permitted landfill, as part of the permit amendment application for MSW Permit No. 1405A, The wetland delineation investigation consisted of a review of available aerial photographs and soil surveys, followed by a field reconnaissance and wetland delineation. Based on an evaluation of the presence of all three wetlands criteria (specifically, hydrophytic (water loving) vegetation, hydric soils, and wetland hydrology) and the definitions of "Waters of the United States", as found at 33 CFR Part 328, two areas studied in the wetland delineation were determined to be jurisdictional waters of the United States under Section 404 of the Clean Water Act (Figure III-4.10). These two areas consist of 0.3856 acres (Area A) of jurisdictional waters associated with Mustang Creek in the southwest corner of the Site and 0.09 acres (Area C) associated with an unnamed drainage pathway/tributary located along the northern property boundary of the existing permitted landfill property (Rust, 1994). Rust stated that filling these areas up to ten (10.00) acres in size could be accomplished under Nationwide Permit (NWP) Number 26 (33 CFR Part 330, Appendix A, Section 8.26, Headwaters and Isolated Waters Discharges) under Section 404 of the Clean Water Act, with a Pre-Discharge Notification (PDN). If less than one acre is filled, as is the case at the Site, the activity can be authorized under NWP 26 without a PDN (Rust, 1994).

A third wetlands study was performed on the proposed eastern expansion property by SWCA, Inc. Environmental Consultants (SWCA) in October 2002. SWCA identified Mustang Creek as a jurisdictional water of the United States, as considered by the USACE. Mustang Creek did not contain flowing water at the time of the field visit and was considered to be ephemeral. Dominant vegetation along the creek was goldenrod (*Solidago riddellii*). The reach of Mustang Creek that runs through the permitted landfill is approximately 185 feet long. The creek channel is largely un-vegetated and contained within banks that are approximately four feet deep and four to 15 feet apart. This creek is considered likely to qualify as a jurisdictional water of the U.S. because it possesses a well-incised channel (SWCA, 2002).

No wetlands were identified on the eastern expansion property that is presently used for agricultural purposes. The two previously defined wetland areas on the permitted landfill (Rust, 1994) are located outside the existing and future permitted disposal area boundaries and will most likely not be affected by landfill operations. No landfill unit or lateral expansion will be located in identified wetlands, as regulated in 30 TAC §330.302.

5.0 REGIONAL HYDROGEOLOGY

This section describes the regional hydrogeology of the area surrounding the Site. This information is provided in accordance with 30 TAC §330.56 (d)(4) and is taken from published and open-file sources, as available. A summary of the regional hydrogeologic stratigraphy is also presented in Table III-4.2. The following sections describe the major hydro stratigraphic units beneath the Site and the associated geologic units, general water quality for each unit, and their hydrogeologic properties.

5.1 Regional Aquifers

The Edwards aquifer is the most important source of groundwater in central Williamson County. In the vicinity of the Site, the upper Edwards aquifer units are at a depth of approximately 615 feet, based on water well logs within one mile of the Site. Small quantities of water are produced in the region from the Taylor Group of the Ozan Formation, the Austin Chalk, and the Hosston Member of the Travis Peak Formation (Lower Trinity aquifer). The Eagle Ford Group, the Buda Limestone, and the Del Rio Clay are not known to yield water to wells in the region (CJI and MFG, 1995). At the Site, groundwater is encountered in the Lower Taylor Group and top of Austin Chalk at an average elevation of 684 feet MSL and from coarse material in the Older Alluvium at an average elevation of 689 feet MSL.

The Edwards-Trinity aquifer system is in carbonate and clastic rocks of Cretaceous age in a 77,000-square-mile area that extends from southeastern Oklahoma to western Texas. The aquifer system consists of three complexly interrelated aquifers-the Edwards-Trinity, the Edwards, and the Trinity aquifers. The Edwards-Trinity and the Trinity aquifers are stratigraphically equivalent in part and are hydraulically connected in some places (Ryder, 1996). The Edwards aquifer overlies the Trinity aquifer in the Balcones Fault zone, 5 miles to the west of the Site, and the two aquifers are hydraulically connected where no confining unit separates them. In Williamson County, the Walnut Formation, composed of limestone and marl, underlies the Edwards aquifer strata and separates the Edwards aquifer from the underlying Trinity aquifer (Senger et al, 1990).

5.1.1 Edwards Aquifer

The width of the Edwards aquifer ranges from about four miles at the Colorado River in Austin to about 30 miles in Medina and Williamson counties. The updip boundary of the aquifer in most places is the farthest updip extent of the Edwards Group, except in the westernmost area where the updip boundary is determined by a decreased incidence of faulting. From Kinney County eastward and northward to the Colorado River in Austin, the updip boundary generally coincides with the Balcones Escarpment. The downdip boundary of the aquifer is largely fault controlled (Ryder, 1996).

The northern segment of the Edwards aquifer in Travis, Williamson, and Bell counties is the northernmost extension of the Edwards aquifer along the Balcones Fault Zone (Figure III-4.6). Cretaceous strata dip gently eastward and are locally overlain by terrace deposits and alluvium. Several major normal faults, downthrown to the east, strike northward across the area. Gentle flexures, possibly related to faulting, parallel the faults. Minor normal faults and joints are most abundant in areas adjacent to major faults and flexures. These fractured-strata zones probably parallel the length of the faults or flexure axes and may be as wide as one mile. The potentiometric surfaces for conditions of high flow and low flow indicate a regional flow system that is noticeably affected by Balcones faults along the north and south edges of the northern segment of the Edwards aquifer. Near the Site, the potentiometric surface of the Edwards aquifer varies from approximately 550 feet MSL during low flow to 625 feet MSL during high flow (Senger et al, 1990). The average potentiometric surface of the Edwards aquifer, in close proximity to the Site, is approximately 587 feet MSL (Senger et al, 1990), which is 102 feet below the average potentiometric surface of the surficial clay/upper claystone (Older Alluvium) and 97 feet below the average potentiometric surface of the lower claystone/upper limestone (Taylor Group/Austin Chalk).

In the Georgetown area, northwest of the Site, where faults are less abundant, regional groundwater generally flows eastward, following approximately the dip of the stratigraphic units. Groundwater flow circulation is faster in the western part of the aquifer than in the eastern part. Main discharge for the western, fast-flowing system occurs along fractures through springs and seeps at the major creeks and rivers in the Georgetown area. Some recharge water moves farther downdip past these springs, where water levels show large seasonal variations due to reduced transmissivity and storativity of the aquifer. In the confined portion of the aquifer, large fluctuations in the potentiometric surface cause seasonal reversals in the vertical flow component. These thin shale layers within the Edwards aquifer strata represent large surface areas for vertical flow (Senger et al, 1990).

Recharge to the Edwards aquifer occurs through infiltration of rainfall and seepage from streams crossing outcrop areas in west-central Williamson County (generally west of Interstate 35). Water occurs primarily in solution and solution-collapse zones throughout the aquifer. Water occurs under both confined and unconfined conditions in the Edwards aquifer, depending on the location of the aquifer relative to the outcrop. In central Williamson County, the Edwards aquifer is typically confined. Groundwater flow is primarily to the east and southeast in the general area of the Site as shown in Figure 4-11 (Senger et al, 1990).

Water in the Edwards aquifer moves primarily by turbulent flow in underground channels, and thus, travels relatively fast (Brune and Duffin, 1983). Solution features, such as honeycombing, sinkholes, and caverns allow for rapid infiltration of water at the outcrop, as well as for rapid movement within the aquifer (Ridgeway and Petrini, 1999). The intense faulting at the outcrop is important because it has allowed many of the solution features to develop (Senger et al, 1990).

As a result of the dominance of secondary permeability features (e.g., solution features), the Edwards aquifer exhibits considerable anisotropy and heterogeneity. Hydraulic conductivities range from 8.7 to 877 gallons/day/feet^2 (4.1×10^{-4} to 4.1×10^{-2} centimeters per second (cm/sec)) for the Edwards aquifer (Brune and Duffin, 1983). Transmissivity values are estimated to range from 400 to 300,000 gallons/day/foot (0.6 to 430 cm^2/sec) due to the extreme range in hydraulic conductivity and substantial variations in aquifer thickness. Estimates of effective porosity and storativity range from four to six percent and 0.00025 to 0.00045, respectively, (Brune and Duffin, 1983).

5.1.2 Taylor Group and Austin Chalk

Water usually occurs in the upper, weathered outcrop portions of the Taylor Group and Austin Chalk and is typically under water-table (unconfined) conditions. Large cracks may develop in the exposed clays of the Taylor Group during dry periods, thus allowing recharge to permeable units within the Taylor Group. The Austin Chalk contains numerous fractures and joints that are saturated and in addition, water can occur in the softer marls throughout the unit. Some thin beds of sand and gravel found in the Taylor Group and Austin Chalk may contain significant amounts of localized water. All of these units are recharged by rainfall on outcrops, by streams that cross the outcrops, and by lakes or ponds, which may lose water to them (CJI and MFG, 1995). At the Site, groundwater is encountered at the base of the Taylor Group and top of Austin Chalk. This interval is currently monitored for groundwater detection on the permitted landfill.

The Austin Group consists of seven formations, which in ascending order are Atco Formation, Vinson Chalk, Jonah Formation, Dessau Chalk, Burditt Marl, Pflugerville Formation, and Sprinkle Claystone. The lower six formations are carbonate and the uppermost is a montmorillonite. claystone. The McKown Limestone is a grainstone facies of the upper part of the Dessau, the Burditt, and the Pflugerville formations (Young and Woodruff, 1985).

The Atco, Vinson, Jonah, and probably the Dessau remain carbonate to the northeast into the East Texas Embayment. The Sprinkle Claystone (Lower Taylor Marl) is a tongue of the Ozan that extends southwestward over the Burditt and Pflugerville formations, which are platformal facies of the lower part of the Ozan Formation of the East Texas Embayment.

5.1.3 Travis Peak Formation

In eastern Williamson County, the Hosston Member of the Travis Peak Formation (at the base of the Trinity Group) is a sandstone and provides fresh to slightly brackish water from approximately 1,500 feet beneath the base of the Edwards aquifer. Reported well yields are greatest in eastern Williamson County; 1,089 gallons per minute (gpm) at Taylor and 228 gpm at Bartlett (Senger et al, 1990). Transmissibility values as high as 32,700 gallons per day per feet (gpd/ft) have been reported in Williamson County (Brune and Duffin, 1983). The Hensell Sand in the Upper Trinity is an important source of water in western Williamson County. Because of the great depth to the Trinity Group sands and the presence of the Edwards aquifer at a much shallower depth, the Trinity Group is not used for water supply in central Williamson County (CJI and MFG, 1995).

5.2 Regional Water Quality

Water from the Taylor and Austin rocks is mainly a calcium-carbonate type water, hard, and usually fresh (Brune and Duffin, 1983). The low total dissolved solids (TDS) content probably reflects the recharge of meteoric water to these shallow water-bearing zones.

Water in the Edwards aquifer may be described as a calcium bicarbonate type water, generally becoming sodium sulfate to sodium chloride type water downdip (to the east). The water becomes progressively more mineralized downdip. Within about four to 10 miles of the outcrop, Edwards aquifer water is generally within the recommended TDS limits for drinking water established by the TDH (Senger et al, 1990). Concentrations of TDS in water vary considerably in different geological regions of the Edwards aquifer, resulting from the degree of faulting and differences in the solubilities of minerals that make up the aquifer. As a result of faulting, the chemical quality of the water in the Edwards aquifer can change

abruptly in a very short distance across a zone often referred to as the "bad-water line." Along this line, the water is fresh on the upthrown side of a fault, containing less than 1,000 milligrams per liter (mg/L) of TDS, and very saline (usually a sodium- or calcium-sulfate type water) on the downthrown side. Generally, TDS concentrations increase from the recharge zone (1,500 square mile area where highly faulted and fractured Edwards limestones outcrop at the land surface) to the artesian zone (confined between two relatively impermeable formations - the Glen Rose Formation below and the Del Rio Clay on top). Intensive faulting in the Edwards aquifer has created barriers for groundwater flow to the east resulting in higher TDS values in this area (BSEACD, accessed 11/04).

There are several noticeable trends in the change in TDS content in relation to fluctuations in water levels and rainfall. Generally, as water levels increase after major rain events, TDS concentrations tend to decrease. This suggests that rainwater, which is less mineralized, recharges the aquifer and can dilute high concentrations of organics, carbonic acid, nutrients and other ions that exist in the aquifer system, which eventually results in lower TDS values. A drop in TDS content after a major rain event may represent significant hydraulic connectivity between the surface and the well at a specific monitoring site. Another trend is that TDS content and water level increase after a rain event, which may be the result of water recharging the aquifer and moving groundwater from low permeability portions of the aquifer into areas with higher TDS into fractures and conduits that flow towards the well. These two types of responses to a single rain event reveal the complexity of the aquifer system. Rainwater can enter the system through several different pathways, either directly through recharge features or by diffuse routes, which both can influence TDS concentrations within a well (BSEACD, accessed 11/04).

TDS concentrations in the Edwards aquifer, in close proximity to the Site, range between 1,526 and 2,445 mg/L (Senger et al, 1990).

Unlike the Edwards, the Trinity aquifer recharges very slowly. Only four to five percent of water that falls as rain over the area ends up recharging the Aquifer, and water also moves through the Trinity much more slowly than through the Edwards. In addition, the Trinity contributes a significant amount of water as recharge for the Edwards. Recharge to the Edwards can occur where the layers are juxtaposed by faults or, where the Trinity underlies the Edwards, by upwelling. (Edwards Aquifer Homepage, accessed 9/04). In the Hill County, the Travis Peak Formation is made up mostly of sands, silts, conglomerates, and limestones, and is subdivided into six members. Most of the lower members of the Travis Peak Formation, including the Hosston member have not been extensively used. Water quality in the Trinity Aquifers is generally much lower than in the Edwards and it is also more variable. Groundwater to the

west of I-35, within the Trinity Group aquifer, is a calcium-magnesium-carbonate type water. The water becomes a sodium-sulfate or sodium-chloride type water downdip and the quality of water degenerates downdip, to the south and southeast, as indicated by the increased levels of TDS. In north-central Texas, waters in the Glen Rose (Upper Trinity) are highly mineralized and are a source of contamination for wells drilled into the underlying Twin Mountain Formation; but towards the south in the Hill Country, the Glen Rose can yield moderate quantities of fresh water.

5.3 Regional Water Use

Regional water use from the Austin Chalk and the Taylor Group is almost exclusively by individuals for domestic, livestock, and/or domestic irrigation purposes (CJI and MFG, 1995). Water withdrawn from the Edwards-Trinity aquifer is largely used for irrigation in the northwestern part of the aquifer (including Williamson County) where soil conditions are particularly favorable for farming (Ryder, 1996).

A TCEQ water well records search was performed for this amendment application and included in Appendix III-4B. The search revealed six water wells (Map ID 2, 5, and 6) within a one-mile radius of the proposed permit boundary of the Williamson County RDF (Appendix III-4B). Five of the wells are completed for domestic use and one well is unused. The two nearest water wells identified in the water well records search (Map ID 5), are located approximately 4,700 feet north of the Site as shown in Appendix III-4B. The depths of the five domestic wells range from 640 to 820 feet bgs and are completed in the Edwards aquifer. The unused well, drilled to a depth of 2,919 feet bgs, most likely is completed in the Trinity aquifer. In addition to the wells identified in the water-well records search, there are four wells identified on the 1987 United States Geological Survey (USGS) Hutto, Tex. quadrangle map within one mile of the Site and two hand-dug wells. One of the wells is located near the abandoned house located in the southeast corner of the Site and one well is located along the northern perimeter of the Site. The hand-dug wells, located within the property boundary, will be plugged within 30 days of permit issuance in accordance with TCEQ regulations.

The rural area around the Site is served by the Jonah Water Supply Corporation (JWSC), which obtains its water supply from six deep wells completed in the Edwards aquifer. The closest JWSC well to the Site is located 3.5 miles to the west. All of the JWSC wells are located upgradient of the Site and the water distribution lines are located along the western and eastern perimeter boundaries of the Site (CJI and MFG, 1995).

The City of Hutto has three wells completed in the Edwards aquifer, but only two are in use. The three Hutto wells are all within, or immediately adjacent to, the Hutto city limits, or about 3 miles south of the Site. Water obtained from the Manville Water Supply Corporation in Coupland (located 10 miles southeast of Hutto) is blended with water from the City of Hutto wells (CJI and MFG, 1995).

6.0 SUBSURFACE INVESTIGATION

The following sections describe the field activities performed for the subsurface investigation of the eastern landfill expansion area conducted by Earth Tech, Inc. (Earth Tech) at Williamson County RDF from April 13 through July 27, 2004. Also presented is a discussion of previous subsurface investigations that were completed from 1980 through 1991. A discussion of the stratigraphy beneath the Site is presented based on data obtained from soil borings and piezometers installed during this subsurface investigation and previously collected data. The terminology used to classify the Site stratigraphy is derived from the previous investigations. Information is provided in accordance with 30 TAC §330.56(d)(5).

6.1 Overview

An overview has been prepared to discuss the previous subsurface studies and the scope of work completed at the Site.

6.1.1 Previous Subsurface Investigation Reports

Several subsurface investigations were conducted at the Site, west of the expansion property. The locations of borings and piezometers are shown on Figure III-4.12. Site geologic data for each boring are summarized in Table III-4.3 and piezometer construction data are summarized in Table III-4.4. Lithologic boring logs are presented in Appendix III-4C and piezometer construction data are presented in Appendix III-4D.

Twenty-eight borings were drilled at the Site from 1980 through 1985. These earlier boreholes were usually advanced by the hollow stem auger (HSA) method. Thin walled samplers were used to obtain undisturbed samples either at discrete intervals or continuously in the surficial clays, to depths of 10 to 20 feet bgs. Continuous sampling in more consolidated strata below the clay was performed by double tube coring (rotary hydraulic). Hole depths were usually between 30 and 50 feet bgs. Only one hole was drilled deeper than 50 feet, to 60 feet bgs.

The initial subsurface and geotechnical investigation was completed by American Soils Limited in March 1980. Six borings B-1 through B-6) were advanced to depths ranging from 27 and 45 feet bgs. In October 1980, Frank G. Bryant & Associates, Inc. advanced 10 borings (D-12, D-18, D-27, F-1, J-7, J-23, O-5, O-18, O-27, and Q-12) with depths ranging from 34 to 60 feet bgs. Both investigations were conducted as part of the preliminary assessment of the land prior to landfill development and as the soils report for the original permit application. In October 1981, Frank G. Bryant & Associates, Inc. advanced four borings (B-1 through B-4) to 50 feet bgs and in August 1985, advanced eight borings (B-20 through B-27) to 30 feet bgs to conduct additional geotechnical and field permeability testing data for the permitting process.

From December 5 through December 8, 1989, Hydro-Search, Inc. and MFG performed a Phase II study to describe and correlate the strata underlying the Site. The scope of work included the collection of soil samples for geotechnical testing, determining in-situ hydraulic conductivity values within potential water-bearing strata, obtaining water-level data from the installed piezometers, and determining hydraulic gradients. Four soil borings (P-1 through P-4) were advanced within the footprint of the existing permitted area to depths ranging from 33 to 50 feet bgs to establish a geologic profile for the Site. In addition, nine piezometers (P-1D/S, P-2D/S, P-3D/S/I, and P-4D/S) were installed to depths ranging from six to 37 feet bgs. During the investigation to support the groundwater monitoring system design for the then existing landfill, continuous samples were collected for classification of lithology, investigation of groundwater conditions, and collection of geotechnical samples. Selected subsurface samples were analyzed in the laboratory for Atterberg limits and particle size analysis.

An additional subsurface investigation for the northern acreage of the Site was conducted by CJI, as part of the Permit Amendment Application No. 1405-A. The investigation was conducted in two phases. Phase I was conducted in February 1991 to obtain a general description of shallow lithology and the occurrence of shallow water. The Phase I site investigation included drilling and sampling of three lithologic borings (OSB-1 through OSB-3), advanced to depths ranging from 29 to 44 feet bgs, and the installation of four piezometers (OSB-1D/S, OSB-2D, and OSB-3D). Phase II was performed in May 1991 and was conducted to generate data to support development of a hydrogeologic model of the Site. The Phase II boring program was determined in consultation with the TDH in light of previously-developed data on both the existing and proposed expansion areas and approved in a July 17, 1991 letter to the Site operator, WMTX. Continuous core samples were retrieved to the total depth of each boring. The Phase II field investigation for the northern expansion area consisted of the drilling of 20 borings (A-1 through A-18, OSB-2S, and OSB-3S) and the installation of 10 piezometers (A-3D/S, A-14D/S, A-

16D/S, A-18D/S, OSB-2S, and OSB-3S). The borings were continuously sampled to total depth, ranging from 36 to 56.5 feet bgs.

In October 1991, four additional piezometers (A-8D, A-11D, A-12D, A-17D) were installed adjacent to pre-existing borehole locations where groundwater was encountered during drilling activities. Depths of the piezometers ranged from 32 to 36.5 feet bgs. The previous borings, piezometers and monitoring wells are listed in Tables III-4.3 and III-4.4 and are included in Appendices III-4C through III-4F.

6.1.2 Subsurface Investigation Scope of Work

The recent subsurface investigation at the Site was completed April 13 through July 27, 2004, in accordance with the Boring Plan that was approved by the TCEQ MSW Permitting division on February 27, 2004. The purpose of this investigation is to provide the necessary data to analyze the hydrogeological and geotechnical feasibility of a vertical expansion of the Site and a lateral expansion of 373 acres to the east.

During this investigation, 44 borings were installed; 22 of these were converted to piezometers. Of the 44 borings, 17 of the borings were completed at least five feet below the deepest excavation planned and 18 were completed a minimum of 30 feet below the deepest excavation planned to comply with 30 TAC §330.56(d)(5) (A)(ii) of the TCEQ. The locations of the boring can be seen on Figure III-4.12 and a summary of the borings can be found in Tables III-4.3 and III-4.6.

6.2 Field Activities

During 2004, field activities at the Site consisted of establishing ground control; reviewing aerial photographs; interpreting data and photos; implementing the TCEQ-approved boring plan; drilling; logging; sampling; and installing and developing piezometers.

6.1.1 Survey and Ground Control

Boring locations and piezometers installed during this investigation were surveyed by SAM, Inc. of Austin, Texas. Prior to commencement of the field work, air-photo grid locations were surveyed for development of a Site topographic map for establishment of horizontal and vertical control. The proposed locations were submitted to and approved by the TCEQ MSW division prior to conducting the field investigation. After completion, the borings and piezometer installations were surveyed to provide accurate locations and elevations for groundwater level measurements. Standard ground control methods were utilized to conduct the survey. At each location, northing and easting values were established; as

well as, vertical control elevations for: 1) the top of the protective casing, 2) the top of the PVC piezometer casing, 3) the top of the concrete pad (at marker), and 4) the ground surface. Locations were referenced to the Texas State Plane Coordinate System. Elevations were established relative to the United States Coast and Geodetic Survey (USC&GS) Mean Sea Level datum. Figure III-4.12 illustrates the surveyed locations of borings and piezometers installed at the eastern landfill expansion area during this field investigation.

6.2.1 Aerial Photograph and Map Interpretation

Aerial photographs and topographic maps of the Site and surrounding area were used to:

- Identify topographic controls;
- Locate surface water features that could influence the shallow groundwater system and/or impact drilling activities; and,
- Identify lineaments suggestive of regional faulting.

The locations of topographic relief, underground petroleum pipeline, and acreage of the proposed landfill area were criteria used for the selection of piezometer locations. Topographic relief, the elevation of the proposed depth of the landfill subgrade, and the occurrence of groundwater encountered during drilling operations were controlling factors in selecting the target depths of the piezometer screen intervals and borehole depths. Maps and aerial photos covering five decades were inspected to identify lineaments that might be indicative of fractures in the subsurface. No features were identified that were indicative of subsurface discontinuities. Aerial photographs are located in Appendix III-4A and a discussion of features observed on the aerial photographs is found in Section 4.1.1.

6.2.2 Drilling Program

Drilling services were subcontracted by Earth Tech, Inc. to Core Tech Drilling, Inc. of San Marcos, Texas. One boring/piezometer completion was performed by TWF Drilling of Tyler, Texas in the initial mobilization to the Site. The subcontractors provided equipment and personnel for drilling, sampling, piezometer installation, and borehole abandonment. Earth Tech staff were present on-site during the entire drilling program to provide supervision and technical oversight of operations.

6.2.3.1 Drilling and Soil Sampling

In compliance with 30 TAC §56(d)(5)(A)(ii), a site-specific Boring Plan was approved by the TCEQ MSW Permitting division on February 27, 2004. Based on discussions with the TCEQ, the investigation

area totaled 373 acres, which included only the eastern landfill expansion footprint, since the permitted landfill had been thoroughly studied during previous subsurface investigations. Completion of this investigation satisfied the TCEQ requirements that specific geologic and hydrogeologic data be collected prior to permitting and construction of any landfill expansion area pursuant to 30 TAC §330 - Subchapter E - Municipal Solid Waste (MSW) Permit Procedures.

Specifically, the subsurface investigation consisted of drilling 44 borings at 35 different locations for lithologic description, determining stratigraphic continuity, and collecting samples for geotechnical testing. To comply with the mandates of the TAC, 17 soil borings (E-20 through E-22, E-23S, E-24S, E-25 through E-33, E-34S, E-35, and E-36S) were advanced on the expansion area to a depth of at least five feet below the deepest planned excavation and 18 borings (E-1D through E-9D and E-10 through E-18) were drilled to a depth of at least 30 feet below the deepest planned excavation (Figure III-4.12). In addition, nine shallow borings (E-1S through E-9S) were advanced and completed as piezometers, adjacent to deeper completed piezometers (E-1D through E-9D). These clusters of two piezometers were installed for water-level elevation determinations in the uppermost aquifer and potentially in a hydraulically, interconnected, underlying aquifer and to perform a series of hydraulic conductivity tests as part of the groundwater investigation. In addition, four borings (E-23S, E-24S, E-34S and E-36S) were converted to piezometers for water elevations in the uppermost aquifer.

TWF Drilling and a geologist from Earth Tech initiated installation of the borings on April 13, 2004. Operations were shutdown on April 16 and commenced on April 26, 2004 with Core Tech Drilling, Inc. and a geologist from Earth Tech to completion of the field effort. Each boring was installed using HSA and rotary wash drilling techniques. A Mobil B-57 truck-mounted drill rig was initially utilized at the Site; replaced with a Mobil B-59 truck-mounted drill rig that was used to drill the borings until designated target depths were reached. The shallowest boring, E-24S was drilled to a depth of approximately 23 feet bgs and the deepest boring, E-13 was drilled to 118 feet bgs. Groundwater was not encountered while drilling, with the exception of borings E-1D, E-1S, E-5D, E-5S, E-7D, E-7S, E-12, E-15, E-18, E-23S, E-27, E-30, E-32, E-34S, E-35 where groundwater was encountered ranging from approximately 20 to 39 feet bgs.

All of the borings were continuously sampled using a five-foot CME sampler and a five-foot NQ rock core sampler until the desired depth was achieved. In the unconsolidated soil, sampling was conducted using a 3.25-inch outer diameter (O.D.) CME sampler. If a piezometer was installed in the surficial clay/upper claystone unit, the boring was advanced using a six-inch diameter HSA, except for E-23S that

was advanced using an eight-inch diameter HSA. Once the boring was advanced to the desired depth, the piezometer was installed.

The piezometers installed in the base of claystone/top of limestone unit were advanced through the unconsolidated material using a six-inch or eight-inch diameter HSA. When consolidated material was encountered, sampling was conducted using a 3.25-inch O.D. NQ rock core barrel. Water was required during the rock coring procedures and obtained from the on-site water tank supplied by city water from JWSC. When the boring was advanced to the desired depth, the NQ rock coring equipment was removed and the piezometer was installed. Only consolidated portions of five borings (E-1D, E-2D, E-3D, E-4D, and E-7D) were reamed to a five-inch diameter.

If groundwater was encountered, drilling was stopped and water-level measurements were noted on drilling logs. Drilling was then resumed and the borings were augured to the desired depth.

Samples collected during the drilling program were either stored in labeled waxed, cardboard core boxes or properly labeled and submitted to Great Lakes Soil & Environmental Consultants, Inc. in Burr Ridge, Illinois for geotechnical analyses. Site cross sections assisted in determining the depth at which each geotechnical sample was collected. Undisturbed representative soil samples were collected using a five-foot long CME sampler, and undisturbed rock core samples were collected using a five-foot long NQ rock core sampler. The cohesive soil and rock samples were wrapped with a base layer of plastic wrap and an outer layer of aluminum foil, sealed with duct tape, labeled with the location identification and depth interval, and packed in bubble wrap. Non-cohesive soil samples were collected in plastic Ziploc bags and labeled with the location identification and depth interval. Large quantity soil samples were collected in the form of composites and placed in five-gallon plastic buckets with lids, sealed with packing tape, and labeled with the location identification and depth interval. Drilling and sample collection were performed in general accordance with applicable American Society for Testing & Materials (ASTM) standards.

6.2.3.2 Borehole Abandonment

Forty-four soil borings were advanced during the subsurface investigation. Twenty-two borings were converted to piezometers and the remaining 22 borings were sealed using bentonite chips. The bentonite chips were poured into the boring at a rate slow enough to allow the chips to reach the bottom. The borings were filled with bentonite chips to approximately two feet bgs. The bentonite chips were hydrated with the water that was used during the drilling process. If additional water was required to

hydrate the bentonite chips, it was obtained from the onsite water supply. Texas Plugging Reports are located in Appendix III-4E.

6.2.3.3 Soil Classification and Logging

Geologic logs were maintained in the field during coring and sampling activities. Descriptions were made in accordance with 30 TAC §330.56(d)(5)(A), in general accordance with the Unified Soil Classification System (USCS), and procedures described in "Standard Practice for Description and Identification of Soils Visual-Manual Procedure" (ASTM, 1994). Soil samples were logged, classified, and geologically interpreted in the field. Soil descriptions include unit, consistency or density, matrix color and secondary color (classified using the Munsell® soil color charts, 1994), USCS {using procedures described in ASTM D2487 and D 2488}, field moisture, plasticity, cohesiveness, primary sedimentary structures, observed secondary features, and weathering zone abbreviations (Hallberg et al, 1978).

Also, in accordance with 30 TAC §330.56(d)(5)(C)(i), the level at which groundwater was encountered is presented on the boring logs. Water levels included on the borings logs are recorded in feet bgs.

Field logs were reviewed for technical accuracy, consistency, and format. The final boring logs can be found in Appendix III-4C.

6.2.4 Piezometer Installations

A total of 22 piezometers were installed during the subsurface investigation. The piezometers were installed immediately upon the completion of drilling, to minimize the possibility of borehole collapse. The installation of the piezometers was done in accordance with current Texas regulations: 30 TAC §330.242 and the Texas Water Well Drillers Rules 16 TAC Part 4 §76.1006.

Figure III-4.13 provides typical piezometer construction specifications used for this investigation. Details regarding piezometric installations are described in Sections 6.2.4.1 and 6.2.4.2. A summary of piezometer construction details is presented in Table III-4.5. Detailed piezometer construction summaries are also presented in Appendix III-4F.

6.2.4.1 PVC Piezometer Installations

From April 16 to July 13, 2004, 22 piezometers were installed during the subsurface investigation. Thirteen piezometers were installed in the surficial clay/upper claystone unit, and nine piezometers were

installed in the base of claystone/top of limestone unit. Locations of each piezometer can be found on Figure III-4.12 and a summary of present monitoring well and piezometer construction details are noted in Table III-4.5. Current piezometer and monitoring well construction diagrams are found in Appendix III-4F.

All piezometers in the surficial clay/upper claystone unit were installed directly into six- or eight-inch diameter boreholes once the augers were removed. All piezometers in the base of claystone/top of limestone unit were installed into four- or five-inch diameter boreholes. The unconsolidated portions of those borings were augured using six- or eight-inch diameter HSA. Each piezometer was constructed with two-inch inner diameter (I.D.), flush-threaded, Schedule 40 polyvinyl chloride (PVC) screen and casing. The screens were generally 10 or 15 feet in length with 0.010-inch machined slots. The filter packs are constructed of Unimin 12/20 silica sand and were installed from the bottom of the borehole to approximately two feet above the PVC screens. The remaining annular space was sealed using 3/8-inch bentonite chips. The bentonite chips were placed on top of the filter pack to approximately two feet bgs and hydrated with water.

The surface completion for each piezometer consists of a two-foot by two-foot square by six-inch or four-foot by four-foot square by six-inch thick concrete pad, reinforced with rebar; five-foot long, four-inch diameter anodized aluminum protective cover with lock (approximately 3.5 to four feet above the concrete pad); four steel guard posts (one at each corner of the concrete pad, filled with concrete, and painted yellow); and pea gravel was placed in between the PVC well and protective casing. A 3/8-inch diameter weep hole is located through the protective casing (approximately twoinches above the top of the concrete pad) to prevent moisture buildup inside the protective casing. A brass marker is located on the concrete pad and is used for the surveyed vertical elevation of the concrete pad at each piezometer.

The piezometers were constructed to monitoring well specifications established by the TCEQ MSW division (30 TAC §330.242) except for 13 concrete pad completions located within the proposed landfill footprint. The size of the concrete pads was based on the location of the piezometer either within the landfill footprint (two-foot by two-foot square) or along the perimeter (four-foot by four-foot square), where the piezometer can be converted to a detection monitoring well during future landfill expansion, if deemed appropriate.

6.2.4.2 Piezometer Development

Eight piezometers were developed in accordance with 30 TAC §330.242(c). Development took place at a minimum of 24 hours after piezometer installation and construction. Development was conducted in order to remove drilling residues from the piezometer, breakdown the borehole smear resulting from augering activities, and stimulate the flow of fresh groundwater throughout the screened interval of the piezometer. The development procedure was designed to affect the entire screened interval of the piezometer. Development was accomplished by surging and over-pumping each piezometer utilizing a ProActive Pump Monsoon® 4-stage, 12 volt submersible pump. The indicator parameters of pH, conductivity, temperature, and color were collected from each well volume removed, as the development proceeded. This procedure was performed to ensure that new formation water was entering the well and drilling residues were being removed. Development continued until the following criteria were satisfied:

- Discharged water was significantly free of sediment;
- Field measurements of pH, specific conductance, and temperature had stabilized; and,
- A minimum of five well volumes had been removed.

All attempts were made to fully develop the wells based on the criteria established above; however, poor well yields prevented 18 of the 22 piezometers to meet all of these criteria. Between five and 11 well volumes were removed from four piezometers (E-1S, E-5S, E-7S, and E-23S) to complete development. Eighteen of the piezometers were considered 'dry' and development ceased after only recovering one well volume. Four piezometers (E-8S, E-9S, E-24S, and E-36S) had no water in their screened interval and were not developed.

The equipment used to develop the piezometers was properly decontaminated using a low-phosphate detergent and distilled water wash and rinse. The purged groundwater was dispersed onto the ground surface away from the piezometer location

6.3 Results

This section describes the soil stratigraphic conditions encountered at the Site. The locations of three geologic cross sections are shown on Figure III-4.14 and the legend for the cross sections is shown on Figure III-4.15. Subsurface geologic conditions, piezometer construction details, and groundwater level data across the Site are shown on Figures III-4.16 through III-4.18.

6.3.1 Site Stratigraphy

In general, the investigated Site stratigraphy consists of Quaternary-age terrace deposits, Upper Cretaceous limestones, marls and chalks, and Lower Cretaceous limestones and dolomites (CJI and MFG, 1995). The Site is located at the western limits of Quaternary System, Pleistocene-age gravel deposits also defined as Older Alluvium. These gravel deposits are thin sheets of gravel and sand from old river terraces, usually 20 feet or less in thickness. The Older Alluvium is composed of gravel, sand, clay, well-rounded pebble- to cobble-sized gravel, few boulders, mostly chert and limestone, some quartz and igneous and metamorphic rock detritus (BEG, 1997). Alluvium of more recent age is located in the southwestern portion of the Site along the banks of Mustang Creek (BEG, 1997). The alluvium consists of limestone and chert gravel, sand, silt, and clay and the deposits are thin and spatially narrow (Marks, 1950; Garner and Young, 1976). Below the veneer of alluvium is an eastward thickening wedge of progressively younger rocks that dip gently southeast toward the Gulf Coast. The uppermost geologic unit in the vicinity of the Site is the Cretaceous-age Lower Taylor Marl, which consists of marly montmorillonitic clay and claystone, with some glauconite, phosphate pellets, and hematite and pyrite nodules (BEG, 1974). Below this formation is the Austin Group, consisting of slightly clayey to clayey limestone (Young and Woodruff, 1985).

The stratigraphy encountered during this investigation is consistent with previous investigations conducted at the Site. In general, the units beneath the Site consist of one hydrostatic unit, with three stratigraphic units (surficial clay, claystone, and limestone) down to approximately 592 feet msl, as seen in Table III-4.6. All three units are confined and act as one aquitard unit to the Edwards aquifer, approximately 600 to 700 feet bgs (100 to zero feet msl). Technically, the three units are unconfined; however, when analyzed for field permeability tests may show semi-confined or confined conditions. Across the Site, the surficial clay generally occurs between 717 and 655 feet msl, the claystone generally occurs between 705 and 635 feet msl, and the limestone generally occurs between 668 and 592 feet msl. Cross sections illustrating the lithology and units beneath the Site are shown in Figures III-4.16 through III-4.18.

6.3.1.1 Surficial Clay

The surficial clay unit is 8.6 to 60 feet thick on the Site expansion property, with an average thickness of 31 feet. The surficial clay unit generally increases in thickness to the east across the Site. Figure III-4.19 is an isopach map that shows the thickest surficial clay deposits in the northern portion of the expansion property. The unit mainly classifies as lean clays (CL) in the USCS. Lean clay with sand or gravel,

gravely lean clay, sandy lean clay, gravely lean clay with sand, fat clay, and fat clay with sand are also included to a lesser extent. The surficial clay unit is black to very dark gray at the surface, but transitions to pale yellow with depth. Generally, the soils found in the surficial clay unit are very soft to very dense, slightly moist to wet, low to medium plasticity, cohesive, and massive. Fine to coarse chert sand and gravel can be found within the upper portion of the surficial clay unit. In the lower portion of this unit fine to coarse iron nodules and iron staining are sometimes present. Occasionally, trace black staining and trace fossil shell fragments are also present. Throughout the surficial clay unit, fine to coarse caliche nodules and fine caliche layers are present, but tend to increase in occurrence within the lower portion.

Occasionally within the lower portion of the surficial clay unit, a coarse-grained unit is also present (E-1D/1S, E-5D/5S, E-7D/7S, E10, E-12, E-15, E-18, E-23S, E-27, E-30, E-32, E-33, E-34S, and E-35). The coarse-grained material is one to 20 feet thick, with an average thickness of 8.2 feet, and typically occurs between elevations 679.01 to 651.33 feet msl on the expansion property, with an average elevation of 668.94 feet MSL. Figure III-4.20 is an isopach map of the coarse-grained unit that shows the thickest deposits along the northern portion of the eastern landfill expansion area, trending east-west, and within the northern expansion area of the permitted landfill. Generally the coarse-grained material classifies as poorly graded gravel with clay and sand (GP-GC). Well and poorly graded gravel, poorly graded sand with clay, clayey gravel with sand, and silty sand are also included to a lesser extent. Generally, the coarse-grained materials found in the lower portion of the surficial clay unit are very loose to medium dense, moist to wet, nonplastic, noncohesive, and massive.

The surficial clay unit will be completely excavated during construction of the lateral expansion and base grades will lie below the bottom of the unit. Geotechnical data for representative samples of the surficial clay unit are summarized on Table III-4.7 and included in Appendix III-4G.

6.3.1.2 Claystone

The claystone unit stratigraphically occurs below the surficial clay unit and typically occurs between elevations 705.07 to 629.83 feet msl on the expansion property, with an average elevation of 664.53 feet msl. Figure III-4.21 depicts the top of claystone elevations. On the expansion property, thicknesses range typically from 5.5 to 49.4 feet, with an average thickness of 24 feet. Figure III-4.22 is an isopach map of the claystone unit that shows the thickest deposits along the western portion of the expansion area and the northwestern corner of the permitted landfill. The claystone layer is typically pale yellow to light yellowish brown. Generally, the claystone is weak to moderate, dry to moist, massive, aphanitic, fresh to slightly decomposed, competent to slightly disintegrated, and unfractured to moderately fractured.

Brownish yellow iron staining is found throughout the claystone unit while trace fine to coarse iron nodules are found sporadically throughout the unit. Fossil and fossil shell fragments are found throughout various portions of the claystone unit. Generally, the fractures in the claystone unit are horizontal to vertical, extremely narrow to moderately narrow, and totally healed to moderately healed. The fractures are healed with cohesive sediment, calcite mineralization, or a combination of both cohesive sediment and calcite mineralization. Typically, black discoloration along the fracture planes and iron staining along some fractures occurs throughout the unit. The fracture faces in this unit are smooth but slickensided surfaces are also present.

Within the expansion area, the claystone unit will be partially excavated during construction and base grades will lie within the unit. Geotechnical data for representative samples of the claystone unit are summarized on Table III-4.8 and included in Appendix III-4G.

Occasionally in the lower portion of the claystone unit, a transition zone is also present (E-6D, E-15, E-22, E-23S, E-24S, E-26, E-27, E-30, E-31, E-34S, and E-35). The transition zone typically is encountered between 40.8 and 60.8 feet bgs and at elevations ranging from 669.88 to 629.84 feet msl, with an average elevation of 646.09 feet msl. The thickness of the transition zone is 1.2 to 22.1 feet with an average thickness of 7.09 feet. Typically, the transition zone alternates between layers of claystone and limestone, with the thickness of the layers varying with location.

6.3.1.3 Limestone

The limestone unit stratigraphically occurs below the claystone unit and typically is encountered between 42 and 75.2 feet bgs. Elevations range from 667.79 to 606.77 feet msl on the expansion property, with an average elevation of 640.82 feet msl. Figure III-4.23 depicts the top of limestone elevations. The limestone unit is light gray to very dark gray and generally weak to strong, dry to moist, massive, aphanitic, fresh to slightly decomposed, competent to slightly disintegrated, and unfractured to slightly fractured. Generally, the fractures in the limestone unit are horizontal to vertical, extremely narrow to very narrow, and are predominately totally healed, but some fractures are moderately healed. The fractures are healed with cohesive sediment, calcite mineralization, or a combination of both cohesive sediment and calcite mineralization. Occasionally iron staining occurs along some fractures throughout this unit. The fracture faces in this unit are smooth, but slickensided surfaces are also present. Fossil and fossil shell fragments are found throughout various portions of the limestone unit with increased fossilized zones also being present. Trace very fine-grained pyrite, with occasional fine to coarse pyrite nodules, is present throughout the limestone unit.

Geotechnical data for representative samples of the limestone are summarized on Table III-4.9 and included in Appendix III-4G.

7.0 GEOTECHNICAL REPORT

This geotechnical report discusses the soil sampling procedure, laboratory testing program, and geotechnical test results in order to characterize the subsoils at the Site in terms of soil classification, gradation, consistency, permeability, shear strength, compressibility, and consolidation. Based on the information provided by the field investigation and the laboratory testing, geotechnical analyses are provided for the suitabilities of the subsoils to support the landfill foundation and to be utilized for construction of the compacted soil liner.

7.1 Geotechnical Soil Sampling Procedures

The geotechnical investigation was conducted in accordance with 30 TAC §330.56(d)(5)(B)(i)-(ii). Forty-four borings were drilled during the field investigation. As the borings advanced, soil samples were collected by a five-foot long CME sampler until the subsurface became hard. The sampling method was then converted to coring using a five-foot NQ rock core sampler, which continued until total depth of the borings was achieved.

Soil samples were collected during drilling activities for geotechnical testing. The cohesive soil and rock samples were wrapped with a base layer of plastic wrap and an outer layer of aluminum foil, sealed with duct tape, labeled with the location identification and depth interval, and packed in bubble wrap. Non-cohesive soil samples were collected in plastic Ziploc bags and labeled with the location identification and depth interval. Large quantity soil samples were also collected in the form of composites and placed in five-gallon plastic buckets with lids, sealed with packing tape, and labeled with the location identification and depth interval. Samples collected during the drilling program for geotechnical testing were properly labeled and submitted to Great Lakes Soil & Environmental Consultants, Inc. in Burr Ridge, Illinois for geotechnical analyses. Drilling and sample collection were performed in general accordance with applicable ASTM standards.

7.2 Laboratory Testing Program and Summary of Geotechnical Test Results

Testing on the soil and rock samples obtained during the field investigation was conducted according to the ASTM standards. The laboratory tests were performed in order to classify and characterize the

subsurface materials. The testing program included various laboratory tests as discussed in detail in the subsequent sections. The ASTM standards used for testing are summarized below:

- D 1140 - Standard Test Methods for Amount of Material in Soils Finer Than the No. 200 (75-µm) Sieve
- D 4972 - Standard Test Method for pH of Soils
- D 2974 - Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils
- D 422 - Standard Test Method for Particle-Size Analysis of Soils
- D 4318 - Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- D 2216 - Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- D 698 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft3 (600 kN-m/m3))
- D 5084 - Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter
- D 2166 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
- D 854 - Standard Test Method for Specific Gravity of Soil
- D 4525 - Standard Test Method for Permeability of Rocks by Flowing Air
- D 2938 - Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens

According to 30 TAC §330.56(d)(5)(B)(ii)(I), "Constant head with back pressure per Appendix VII of Corps of Engineers Manual EM1110-2-1906, "Laboratory Soils Testing;" ASTM D5084 "Saturated Porous Materials Using a Flexible Wall Permeameter" shall be used to calculate the final coefficient of permeability test results for each sample tested. The Appendix VII of Corps of Engineers Manual EM1110-2-1906 is for "Permeability Tests", including two test types 1) Constant-head test and 2) falling head test. For the constant-head test method, it states, "For fine-grained soils, Q (water quantity) is small and may be difficult to measure accurately. Therefore, the constant-head test is used principally for coarse-grained soils (clean sands and gravels) with hydraulic conductivity (K) values greater than about 10×10^{-4} centimeters per second (cm/sec)." The undisturbed soil samples collected during the Site subsurface investigation were all very fine-grained soil samples with K values much smaller than 10×10^{-4} cm/sec. Therefore, the constant-head method was not applicable for this Site investigation.

The standard test methods for one-dimensional consolidation properties of soils (ASTM D 2435) and for consolidated undrained triaxial compression test for cohesive soils (ASTM D 4767) were planned. However, due to the extremely impermeable nature of the soils, the soil laboratory abandoned these tests.

The detailed laboratory test results are presented in Tables III-4.7 through III-4.9. Copies of all pertinent laboratory testing reports are provided in Appendix III-4G.

7.2.1 Percent Fines (ASTM D 1140)

Percent Fines were performed with the aim of simplifying the soil classification. In this test, the control sieve is No. 200 and Percent Fines are defined by the percentage of fine soils passing the No. 200 sieve (75 micrometers (μm)). Soils retained by the No. 200 sieve are considered granular and soils passing the sieve are considered as cohesive.

The testing results indicate that the surficial soil layer consists of predominantly cohesive soils with seams of sand and gravel. The percent fines of the cohesive soil (greater than 50%) in the surficial soil layer range from approximately 61 to 97 percent. The sands and gravels encountered at some boring locations have percent fines as low as eight percent. One Percent fines test was performed on a claystone sample and the test result indicates that the sample has 72.5% percent fines. Refer to Table III-4.7 for detailed percent fine test results.

7.2.2 Soil pH (ASTM D 4972)

Soil pH was performed on the soil samples near the ground surface. The test results indicate that pH values range from 7.6 to 8.9 for the soil samples tested in water and range from 7.3 to 7.8 for the soil samples tested in 0.01 Molar (M) calcium chloride solution. Refer to Table III-4.7 for detailed soil pH test results.

7.2.3 Organic Content (ASTM D 2974)

This test measures the organic content in soil. Normally, inorganic soil should have very low organic contents (less than five to six percent). The soil samples near the ground surface were collected and tested for organic content. The test results indicate that the organic content in the soils near the ground surface range from 1.07 to 3.69 percent. Refer to Table III 4.6 for detailed organic content test results.

7.2.4 Sieve and Hydrometer (ASTM D 422)

Sieve and hydrometer tests were performed to classify the soils in detail to obtain the percentages of gravel, sand, silt and clay. The sieve test (particle size or gradation) includes most sieve sizes from three-inch to No. 200. When the particle size below the No. 200 sieve is desired, a hydrometer test is performed. Based on the grain size, soils can be classified as clay, silt, sand, gravel and cobbles in accordance with the Unified Soil Classification System (USCS).

The sieve and hydrometer test results indicate that the surficial soil and the claystone layers consist of predominantly silt and clay. The average silt and clay contents are approximately 93 percent for the surficial soil and 89 percent for the claystone. Refer to Tables III-4.7 and III-4.8 for detailed sieve and hydrometer test results.

7.2.5 Atterberg Limit Determinations (ASTM D 4318)

Atterberg Limits (i.e., liquid limit (LL), plastic limit (PL), and plasticity index (PI)) were performed to evaluate soil plasticity characteristics. The PI is the range of moisture content over which a soil deforms as a plastic material. It is bracketed by the LL, is the highest moisture content at which a soil becomes sufficiently wet to flow as a heavy viscous fluid, and the PL the lowest moisture content at which a soil is sufficiently plastic to be manually rolled into threads of 1/8-inch in diameter. Soils can be classified based on the plasticity in accordance with the Unified Soil classification System (USCS).

Based on the Atterberg Limits test results, the unit classification and LL & PI ranges are summarized below:

- Surficial Soils classified CL (LL range: 24 - 47 and PI range: 9 - 32) and CH (LL range: 54 - 85 and PI range: 39 - 61), with occasionally thick layers of ML, SM, GC or GP.
- Claystone classified predominantly CL (LL range: 35 - 49 and PI range: 21 - 32) and CH (LL range: 70 - 77 and PI range: 47 - 53).

Refer to Tables III-4.7 and III-4.8 for detailed Atterberg Limits test results.

7.2.6 Moisture Content Determinations (ASTM D 2216)

Soil samples were selected for moisture content determinations. Samples were weighed, then placed in an oven set to 110 degrees Celsius to dry. The soil samples remained in the oven until no additional weight loss was observed (typically overnight). The final dried sample was weighed and the moisture

content was computed by dividing the weight of evaporated water by the weight of the dry sample. The moisture content of some samples was determined in the laboratory along with other tests such as specific gravity, permeability and unconfined compression.

Based on the laboratory tests, the moisture content range and average value for each unit are summarized below:

- Surficial Soil 10.8 to 34.2 percent with average of 24.8 percent.
- Claystone 9.2 to 28.9 percent with average of 18.3 percent.
- Limestone 9.1 to 21 percent with average of 10.1 percent.

Refer to Tables III-4.7 through III-4.9 for detailed moisture content test results.

7.2.7 Density (ASTM D 698)

Core samples were trimmed and measured to determine the volume of the sample. After obtaining the weight of the trimmed sample, the wet density was calculated by dividing the wet density by the moisture content plus one. The density of some samples was determined in the laboratory along with other tests such as specific gravity, permeability, and unconfined compression.

Based on the laboratory tests, the wet density range and average value for each unit are summarized below:

- Surficial Soil 109 to 130 pounds per cubic foot (pcf) with average of 121 pcf
- Claystone 117 to 142 pcf with average of 132 pcf
- Limestone 128 to 146 pcf with average of 140 pcf

Refer to Tables III-4.7 to III-4.9 for detailed wet density test results.

7.2.8 Recompacted Permeability (ASTM D 5084)

Recompacted permeability tests were performed using the composite samples from the soils to be excavated during landfill development in order to verify if the excavated soils can be used for construction of the compacted soil liner. The compacted soil liner is required to meet the permeability of 1.0×10^{-7} cm/sec in accordance with requirements of 30 TAC §330.200(b). The soil samples were compacted to the maximum dry density of 95 percent using the standard Proctor and then tested for the hydraulic conductivity when the specimen is at the optimum moisture content.

The test results indicate that the recompacted permeability of the composite soil samples range from 7.2 x 10^{-8} cm/sec to 5.4 x 10^{-10} cm/sec, indicating that the cohesive soils to be excavated during cell construction are suitable for the compacted soil liner. Refer to Tables III-4.7 and III-4.8 for detailed standard Proctor and recompacted permeability test results.

7.2.9 Permeability (ASTM D 5084 and ASTM D 4525)

This test method ASTM D 5084 is used to obtain a laboratory measurement of the hydraulic conductivity of water-saturated porous materials with a flexible wall permeameter. The method may be utilized with undisturbed or recompacted specimens that have a hydraulic conductivity less than or equal to 1 x 10^{-3} cm/sec. The soil or claystone samples for hydraulic conductivity were trimmed from relatively undisturbed core samples. Each specimen was placed in a waterproof membrane after its dimensions were carefully measured. The specimen was then placed within a permeameter cell, the influent and effluent lines were attached to the specimen, and the cell was filled with water. Permeant was flushed through the flow system to remove air. A back pressure was applied to saturate the specimen. Each specimen was consolidated by applying a specified confining pressure. The specimen was then permeated under a specified hydraulic gradient by increasing the influent pressure. The quantity of inflow and outflow from the specimen was measured and recorded periodically during the test and the associated hydraulic conductivity was calculated. Penetration continued until the calculated hydraulic conductivity values stabilized.

The rock core samples along with three soil/claystone specimens which were too hard to be saturated were tested for hydraulic conductivity using the ASTM D 4525, the Standard Test Method for Permeability of Rocks by Flowing Air. This test method is to measure the permeability of a rock sample by flowing dry air through the specimen and measuring the pressure, the flow rate, and pressure differential of the air. By extrapolation, the test method also determines an equivalent of the liquid permeability.

Based on the permeability test results, the ranges of the permeability for each unit are summarized below:

- Surficial Soil vertical permeability - 1.8 x 10^{-8} to 6.1 x 10^{-10} cm/sec
  horizontal permeability – 5.7 x 10^{-8} to 1.4 x 10^{-8} cm/sec
- Claystone vertical permeability - 1.3 x 10^{-8} to 7.1 x 10^{-11} cm/sec
  horizontal permeability – 3.4 x 10^{-8} to 1.1 x 10^{-10} cm/sec
- Limestone vertical permeability - 1.1 x 10^{-9} to 4.5 x 10^{-10} cm/sec
  horizontal permeability – measured in the field, see Section 8

Refer to Tables III-4.7 to III-4.9 for detailed permeability test results.

7.2.10 Unconfined Compressive Test (ASTM D 2166) & Uniaxial Compression Test (ASTM D 2938)

The unconfined and uniaxial compressive strength tests provide a quick means of obtaining the approximate shear strength for soil (ASTM D 2166) and for rock (ASTM D 2938). The tests are essentially a form of the unconsolidated, undrained test with the confining stress equal to zero. Since a confining stress is not applied, the test ASTM D 2166 is limited to cohesive soils that can maintain a suitable shape for testing. Soils that are dry and crumbly, fissured or varved materials, silts, peats, or sands are typically not tested with this method. Appropriate test specimens were typically trimmed from relatively undisturbed core samples. The specimen was placed within the loading device, and a constant rate of strain was applied, usually one-half to two percent per minute. Load, deformation, and time values were recorded at sufficient intervals to define the stress-strain curve. Loading was continued until the load values decreased with increasing strain or until 15 percent strain was reached. The compression strength of the soil sample was reported as either of the above conditions, depending on which occurred first.

Based on the laboratory test results, the compression strength range and average value for each unit are summarized below:

- Surficial Soil 2,802 to 20,734 pounds per square feet (psf) with average of 7,468 psf
- Claystone 2,671 to 74,622 psf with average of 27,773 psf
- Limestone 32,024 to 127,941with average of 81,396 psf

Refer to Tables III-4.7 to III-4.9 for detailed unconfined compressive test results.

7.2.11 Specific Gravity (ASTM D 854)

Specific gravity test is normally performed in the laboratory on the rock specimen to provide basic volume-weight relationships. Specific gravity of most rock-forming minerals ranges from 2.65 to 2.8. For the limestone samples collected during the field investigation at the Site, specific gravity ranges from 2.68 to 2.69. Refer to Table III-4.9 for detailed specific gravity test results.

7.2.12 Porosity and Void Ratio

Porosity and void ratio of the rock depend largely on rock origin, size, grading, and packing of particles. For massive low-porosity rocks, typical values range from 0.1 to 0.5 percent. Permeability of the intact rock is often related to porosity, although the rock mass permeability is governed by the characteristics of the in-situ rock. For the limestone samples collected during the field investigation at the Site, porosity ranges from 0.19 to 0.25 while the void ratio ranges from 0.23 to 0.34. Refer to Table III-4.9 for detailed porosity and void ratio test results.

7.2.13 Rock Quality Designation (RQD)

RQD, a method for classifying core recovery to reflect the fracturing and alteration of rock mass, is obtained as the percentage of the total length of sound core recovered, which is four inches in length or longer, to the total length cored. If the core is broken by handling or drilling, as evidenced by fresh breaks in the core, the pieces are fitted together and count as one piece. The recorded RQD values for limestone cores generally range from 80 to 100 percent. Some borings during this investigation, recorded the RQD lower than 80 percent. The RQD values are shown on the boring logs found in Appendix III-4C.

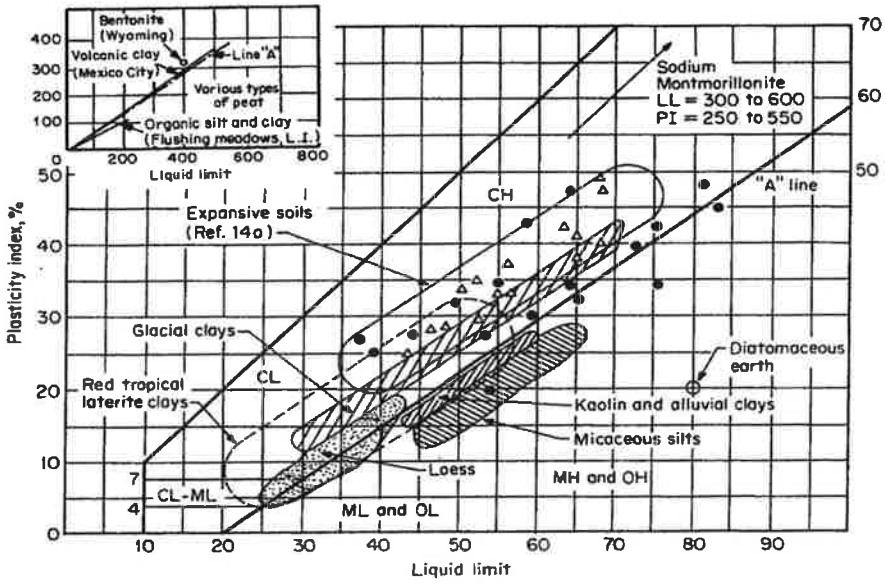
7.3 Geotechnical Analyses

Based on the information provided in the field investigation and laboratory testing, a discussion with conclusions regarding the suitability of the subsoils for the proposed landfill expansion is provided below.

7.3.1 Classification

7.3.1.1 Soil Classification

Review of the test results for percentage fines, sieve and hydrometer, and Atterberg limits indicates that the surficial and claystone units consist of predominantly cohesive soils. In general, the higher the value for the PI and LL, the greater is the tendency of a soil to shrink upon drying and swell upon wetting. It appears that many soil samples tested for Atterberg Limits are in the category of "expansive clay" as shown in the reference chart below.



(Figure 3.12: Plasticity Chart for Uniformed Soil Classification System
Hunt, *Geotechnical Engineering Investigation Manual,* 1984)

The important characteristics of the expansive soils are their high activity, tendency to swell, and undergoing large volume changes by shrinking and swelling. The swelling hazard may cause excessive heave thereby damage to the foundation. Any surface clay with PI greater than about 25 (CH clays) and a relative low natural moisture content approaching the PL should be considered to have swell potential. Soil swell potential can be determined by activity, which is defined as the ratio of the PI to the percent finer by weight than 0.002 millimeters (mm). Based on the PI and the clay contents shown in Table III-4.7, the activities of the surficial CH soils are calculated and the results indicate that the activity ranges from 0.55 to 0.80 with an average value of 0.71. These clays can be classified as normal soil in accordance with the reference table shown below.

Therefore, excessive heave due to swelling is not expected.

CLAY ACTIVITY*

Activity†	Classification
<0.75	Inactive clays (kaolinite)
0.75–1.25	Normal clays (illite)
>1.25	Active clays (montmorillonite)

*After Skempton (1953).[20]
†Ratio of PI to percent by weight finer than 2 μ.

From Reference: Hunt, *Geotechnical Engineering Investigation Manual,* 1984

7.3.1.2 Rock Classification

Engineering terminology refers to rock as either intact or in situ. Intact rock refers to a block or fragment of rock free of defects and is classified by uni-axial compressive strength and hardness. In situ rock refers to the rock mass, which normally contains defects such as fractures or cavities, and is classified by rock quality that is based on percent core recovery, referred to as RQD. Review of the following reference tables (Hunt, 1984) and the limestone test results shown in Table III-4.9 indicates that the rock quality is generally good to excellent and the rock hardness ranges from soft to very soft.

ENGINEERING CLASSIFICATION FOR IN SITU ROCK QUALITY*

RQD, %	Velocity index	Rock-mass quality
90–100	0.80–1.00	Excellent
75–90	0.60–0.80	Good
50–75	0.40–0.60	Fair
50–25	0.20–0.40	Poor
25–0	0–0.20	Very poor

*After Coon and Merritt (1970).[15] Reprinted with permission of the American Society for Testing and Materials.

TABLE 5.20
SUGGESTED HARDNESS CLASSIFICATION FOR INTACT ROCK*

Class	Hardness	Field test	Strength,† kg/cm²
I	Extremely hard	Many blows with geologic hammer required to break intact specimen	>2000
II	Very hard to hard	Hand-held specimen breaks with hammer end of pick under more than one blow	2000–700 700–250
III	Moderate	Cannot be scraped or peeled with knife, hand-held specimen can be broken with single moderate blow with pick.	250–100
IV	Soft	Can just be scraped or peeled with knife. Indentations 1 mm to 3 mm deep show in specimen with moderate blow of pick.	100–30
V	Very soft	Material crumbles under moderate blow with sharp end of pick and can be peeled with a knife, but is too hard to hand-trim for triaxial test specimen.	30–10

*After ISRM Working Party (1975).[10]

†Uniaxial compressive strength [Core Logging Comm. (1978)[11]].

7.3.2 Stability

The relationship between the natural moisture content and LL and PI is an indication of consistency of the soil, which is related to strength and compressibility. Accordingly, the geotechnical test results show that the natural moisture content of the subsoil units is deceasing with depths while the compression strength is increasing with depths. In general, the predominant cohesive materials at the Site are generally stiff to very stiff, and have relatively high shear strengths. Based on the unconfined and uniaxial compression test results, the average shear strength is 3,734 psf for the surficial soil, 13,887 psf for claystone, and 60,204 psf for the limestone.

Slope stability analyses will be provided to verify that the subsoils have sufficient shear strengths against instability in any forms during landfill development, including excavation slopes, foundation, soil liners, and temporary/permanent berms.

7.3.3 Liner Soils

The surficial soil has an average value of 7.5 for pH and 3.05 percent for organic content and is considered inorganic. In general, the surficial soil and some of the claystone will be excavated during landfill construction. These materials consist of predominantly cohesive soils classified as CL and CH based on the USCS. The soils are medium to high plastic. The recompacted permeability of the soils is on the order of 10^{-8} to 10^{-11} cm/sec. Therefore, the excavated cohesive soils can be used for the compacted liner construction.

7.3.4 Excavation Considerations

Information provided by the site investigation and laboratory testing indicates that the soil shear strengths will have sufficient strengths to support the stability of the excavation slopes. Excavation slope stability analysis will be performed for verification.

It is noted that granular soils were encountered in some boring locations located in the eastern portion of the proposed expansion property. The in-situ hydraulic conductivity test results shown in Table III-4.12 indicate that the permeability of the granular soils is on the order of 10^{-3} cm/sec. The thickness varies from one to 14.1 feet where encountered. Depending on the piezometric water level at a particular location, the potential of seepage of groundwater into the excavation area and the stability of the side slope should be considered. The measurements to control the seepage force may be required. The rate and duration of seepage of groundwater into the excavation areas may be variable and dependent on the

orientation, interconnection, continuity, and permeability of exposed soils in the vicinity of the excavation. It is anticipated that groundwater will need to be controlled through a dewatering system, which should effectively reduce hydrostatic pressure on the side slope and under the liner during and after construction until sufficient ballast is placed.

7.4 Geotechnical Conclusions

Based on the information presented and discussed above, it can be concluded that the subsoils at the Site are suitable for landfill construction and the excavated cohesive soils are suitable for construction of a compacted soil liner. In a brief summary:

- The in-situ soils possess sufficient shear strengths to preclude the possibility of development of bearing capacity type foundation failure under the anticipated overburden pressure of the landfill.
- The in-situ soils possess sufficient shear strengths to support the 3H:1V excavation slopes and provide the slope stability.
- The in-situ soils are not susceptible to excessive differential settlement that could detrimentally affect the performance of the landfill liner.
- With proper conditioning and compaction, the on-site cohesive soils will be suitable for clay liner construction and capable of attaining a hydraulic conductivity of 1.0×10^{-7} cm/sec or less.
- The field investigation program did not uncover deposits of soils capable of liquefaction or other features indicative of potentially unstable foundation conditions.

Engineering evaluations in term of slope stability, settlement, etc. will be performed to verify the above conclusions.

8.0 GROUNDWATER INVESTIGATION

In accordance with 30 TAC §330.56d(5)(C), the following section provides a description of the objectives, methodologies, results and discussion associated with the groundwater investigation study.

8.1 Purpose

The purpose of the groundwater investigation is to provide site-specific data related to the understanding of Site groundwater characteristics and behavior in accordance with 30 TAC §330.56d(5)(C). The collection and evaluation of these data are intended to provide a basis for the landfill expansion design. Therefore, the purpose of the groundwater investigation was to generate reliable aquifer characteristics and performance data that would support the effective and appropriate design of the proposed Type I MSW landfill expansion.

8.2 Summary of Activities

The groundwater investigation report is a compilation of hydrogeologic data obtained from previous field studies designed to assess and evaluate the subsurface geologic and hydrogeologic conditions of the Site. This process included reviewing the following documents:

- Williamson County Landfill, Hutto, Williamson County, Texas, Permit Application No. 1405-A, Appendix U, Ground Water Sampling and Analysis Plan (GWSAP), August 8, 1994, Revised January 24, 1995 and April 24, 1996. Prepared by RUST Environment & Infrastructure. Prepared for Waste Management of Texas, Inc.
- Williamson County Landfill and Expansion Area, Williamson County, Texas, MSW Permit 1405, Groundwater Monitoring Design Report, August 18 1994, Revised April 14, 1995. Prepared by McCulley, Frick & Gilman, Inc. Prepared for Waste Management of Texas, Inc.
- Williamson County Landfill, Hutto, Williamson County, Texas, Permit Application No. 1405-A, Attachment 11, Soils Investigation Report, September 1994, Revised February 17, 1995. Prepared by Cook-Joyce, Inc. and McCulley, Frick & Gilman. Prepared for Williamson County.
- Williamson County Landfill, Supplemental Hydrogeologic Data and Recommendations for Waste Monitoring, January 10, 1991, Revised August 1991. Prepared by Hydro-Search, Inc. Prepared for Waste Management of North America.
- Williamson County Phase II Report, April 17, 1990. Prepared by Hydro-Search, Inc. and McCulley, Frick & Gilman, Inc. Prepared for Waste Management of North America.
- Solid Waste - Williamson County near Hutto, Texas, Williamson County, Texas, Williamson County – Permit No. 1405, Geotechnical Investigation, September 17, 1985. Prepared by Frank G. Bryant & Associates, Inc. Prepared for The Honorable Timothy G. Maresh and R.E. Wolfe, Enterprises.

In addition, the investigative process included primary activities associated with a:

- Groundwater Characteristics Investigation (presented below).

To satisfy the groundwater investigation objectives, the following activities were completed:

- Advancement of 44 soil borings, continuous sampling and characterization of the subsurface geology;
- Collection of soil samples and analyses of geotechnical parameters to characterize the subsurface geology;
- Installation of 22 piezometers within the Site expansion area, to support characterization of groundwater flow and assessment of aquifer characteristics at the Site;
- Performance of a Site survey to establish relative elevation and location controls associated with the installed piezometers;
- Performance of aquifer characteristic assessments including single piezometer hydraulic conductivity tests (slug tests);

- Reduction and evaluation of groundwater characterization study data to provide refined estimates of site-specific aquifer characteristic; and,
- The collection of groundwater elevation data from the monitoring network to aid in the characterization groundwater flow at the Site.

8.3 Methodology

Methodologies used during the groundwater investigation activities are presented below. Refer to the respective sections of the referenced reports for a detailed description of subsurface investigation and geotechnical investigation methodologies described within this section.

8.3.1 Subsurface Investigation

The most recent subsurface investigation was conducted to obtain geologic and hydrogeologic information for expansion of the Site to the east of the permitted landfill. Portions of the various subsurface investigations completed to date were designed to determine the suitability of the Site for construction of an MSW; provide a monitoring well network capable of providing groundwater contaminant monitoring; and to assess subsurface conditions and suitability of the Site for the proposed landfill expansion, if deemed suitable. The steps associated with this process included a review of data from all subsurface projects conducted at the Site.

Several subsurface investigations were conducted at the Site permitted acreage, west of the expansion property. The locations of borings and piezometers are shown on Figure III-4.12. Site geologic data for each boring are summarized in Table III-4.3 and piezometer construction data are summarized in Table III-4.4. Lithologic boring logs are presented in Appendix III-4C and historical piezometer construction data are presented in Appendix III-4D. Appendices III-4E and III-4F contain Texas Plugging Reports and current piezometer and monitoring well construction diagrams.

Twenty-eight borings were drilled at the Site from 1980 through 1985. These earlier boreholes were usually advanced by the HSA method. Thin walled samplers were used to obtain undisturbed samples either at discrete intervals or continuously in the surficial clays, to depths of 10 to 20 feet bgs. Continuous sampling in more consolidated strata below the clay was performed by double tube coring (rotary hydraulic). Hole depths were usually between 30 and 50 feet bgs. Only one hole was drilled deeper than 50 feet, to 60 feet bgs.

The initial subsurface and geotechnical investigation was completed by American Soils Limited in March 1980. Six borings B-1 through B-6) were advanced to depths ranging from 27 and 45 feet bgs. In

October 1980, Frank G. Bryant & Associates, Inc. advanced ten borings (D-12, D-18, D-27, F-1, J-7, J-23, O-5, O-18, O-27, and Q-12) with depths ranging from 34 to 60 feet bgs. Both investigations were conducted as part of the preliminary assessment of the land prior to landfill development and as the soils report for the original permit application. In October 1981, Frank G. Bryant & Associates, Inc. advanced four borings (B-1 through B-4) to 50 feet bgs to conduct a fault investigation. The TDH ran resistivity profiles and the four additional boreholes drilled by Frank G. Bryant and Associates, Inc. were assessed. The evaluation of the resistivity data was reported to be inconclusive and the evaluation of the borehole data concluded that there was no evidence of a fault.

In 1983, three monitoring wells (MW-01 through MW-03) were installed on the permitted landfill. No construction details were available.

In August 1985, Frank G. Bryant & Associates, Inc. advanced eight borings (B-20 through B-27) to 30 feet bgs to collect additional geotechnical and field permeability testing data for the permitting process,

From December 5 through December 8, 1989, Hydro-Search, Inc. and MFG performed a Phase II study to describe and correlate the strata underlying Site. The scope of work included the collection of soil samples for geotechnical testing, determining in-situ hydraulic conductivity values within potential water-bearing strata, obtaining water-level data from the installed piezometers, and determining hydraulic gradients. Four soil borings (P-1 through P-4) were advanced within the footprint of the existing permitted area to depths ranging from 33 to 50 feet bgs to establish a geologic profile for the Site. In addition, nine piezometers (P-1D/S, P-2D/S, P-3D/S/I, and P-4D/S) were installed to depths ranging from six to 37 feet bgs. During the investigation to support the groundwater monitoring system design for the existing landfill, continuous samples were collected for classification of lithology, investigation of groundwater conditions, and collection of geotechnical samples. Selected subsurface samples were analyzed in the laboratory for Atterberg limits and particle size analysis.

An additional subsurface investigation for the northern acreage was conducted by CJI, as part of the Permit Amendment Application No. 1405-A. The investigation was conducted in two phases. Phase I was conducted in February 1991 to obtain a general description of shallow lithology and the occurrence of shallow water. The Phase I site investigation included drilling and sampling of three lithologic borings (OSB-1 through OSB-3}, advanced to depths ranging from 29 to 44 feet bgs, and the installation of four piezometers (OSB-1D/S, OSB-2D, and OSB-3D). Phase II was performed in May 1991 and was conducted to generate data to support development of a hydrogeologic model of the Site. The Phase II boring program was determined in consultation with the TDH in light of previously-developed data on

both the existing and proposed expansion areas and approved in a July 17, 1991 letter to the Site operator, WMTX. Continuous core samples were retrieved to the total depth of each boring. The Phase II field investigation for the northern expansion area consisted of the drilling of 20 borings (A-1 through A-18, OSB-2S, and OSB-3S) and the installation of 10 piezometers (A-3D/S, A-14D/S, A-16D/S, A-18D/S, OSB-2S, and OSB-3S). The borings were continuously sampled to total depth, ranging from 36 to 56.5 feet bgs.

In October 1991, four additional piezometers (A-8D, A-11D, A-12D, A-17D) were installed adjacent to pre-existing borehole locations where groundwater was encountered during drilling activities. Depths of the piezometers ranged from 32 to 36.5 feet bgs.

Over the course of the operational history of the groundwater monitoring system, wells have been abandoned and new wells have been added to improve the monitoring network as the Site expands operations. In November 1991, Southwestern Laboratories installed four monitoring wells (MW-04 through MW-07). Three wells (MW-01 through MW-03) were plugged and abandoned by Rust Lichliter/Jameson in January 1996. In addition, 15 piezometers were decommissioned between November 1995 and January 1996 and piezometer P-J was converted to MW-09. During this same field effort, Rust Lichliter/Jameson installed five monitoring wells (MW-08, and MW-10 through MW-13) in general accordance with the TCEQ-approved Permit Amendment No. 1405-A. Monitoring wells, MW-04 and MW-05, remain at the Site, but were removed from the detection monitoring network at this time. On October 7, 2003, MW-09 that was damaged, was plugged and abandoned, and MW-09A was installed as a replacement well.

A comprehensive subsurface investigation was conducted by Earth Tech, Inc. from April 13 through July 27, 2004 and August 31, 2004. The investigation was performed in accordance with the Boring Plan (Earth Tech, December 2003, Revised January 2004 and February 3, 2004) that was approved by the TCEQ on February 27, 2004. In accordance with the Boring Plan, 44 borings were advanced along the perimeter and within the undeveloped portions of the Site expansion area. The lowest anticipated elevation of excavation for the eastern Site expansion is 652 feet msl. As such, pursuant to 30 TAC §330.56(d)(5)(A)(i) and (ii), eighteen borings (E-1D through E-9D and E-10 through E-18) were drilled to a depth of at least 30 feet below the deepest planned excavation to depths of 600.9 to 592 feet MSL. Additionally, 17 soil borings (E-20 through E-22, E-23S, E-24S, E-25 through E-33, E-34S, E-35, and E-36S) were advanced on the expansion area to a depth of at least five feet below the deepest planned excavation to depths of 625.6 to 623.9 feet msl. As part of the groundwater investigation, Earth Tech

installed nine shallow borings/ piezometers (E-1S through E-9S) to provide better subsurface control of the surficial clay/upper claystone unit, to determine groundwater gradient and groundwater flow direction and rate.

During the 2004 subsurface investigation, each boring was installed using HSA and rotary wash drilling techniques. A Mobil B-57 truck-mounted drill rig was initially utilized at the Site; replaced with a Mobil B-59 truck-mounted drill rig that was used to drill the borings until designated target depths were reached. The shallowest boring, E-24S was drilled to a depth of approximately 23 feet bgs and the deepest boring, E-13 was drilled to 118 feet bgs. All of the borings were continuously sampled using a five-foot CME sampler and a five-foot NQ rock core sampler until the desired depth was achieved. In the unconsolidated soil, sampling was conducted using a 3.25-inch O.D. CME sampler. If a piezometer was installed in the surficial clay/upper claystone unit, the boring was advanced using a six-inch diameter HSA, except for E-23S that was advanced using an eight-inch diameter HSA. Once the boring was advanced to the desired depth, the piezometer was installed. The piezometers installed in the base of claystone/top of limestone unit were advanced through the unconsolidated material using a six-inch or eight-inch diameter HSA. When consolidated material was encountered, sampling was conducted using a 3.25-inch O.D. NQ rock core barrel. Water was required during the rock coring procedures and obtained from the on-site water tank supplied by city water from JWSC. When the boring was advanced to the desired depth, the NQ rock coring equipment was removed and the piezometer was installed. Only consolidated portions of five borings (E-1D, E-2D, E-3D, E-4D, and E-7D) were reamed to a five-inch diameter. The locations of the borings/ piezometers are detailed in Figure III-4.12. Construction details are summarized in Table III-4.5 and boring logs/piezometer construction details are presented in Appendices III-4C and III-4F, respectively. Table III-4.6 includes Site geologic data for each boring.

8.3.2 Geotechnical Investigation

In conjunction with the subsurface investigation and in accordance with 30 TAC§330.56(d)(5)(B) subsurface samples were collected and submitted to a geotechnical laboratory for characterization of geotechnical properties of the subsurface materials. Samples were collected from representative subsurface stratum and analyzed for permeability, grain size, pH, percent fines, organic content, Atterberg limits, strength, and moisture content. These data have been used to augment the characterization of the subsurface soils performed during previous investigations. A summary of the geotechnical results is presented in Tables III-4.7 through III-4.9 and the laboratory test results are located in Appendix III-4G. Details of the geotechnical investigation are presented in Section 7.0 of this report.

8.3.3 Groundwater Investigation

Groundwater investigation activities undertaken during the groundwater characterization investigation included construction of piezometers, monitoring of groundwater elevations, and in-situ permeability testing. The investigation was conducted at the Site from April through August 2004.

8.3.3.1 Piezometer Installation

The piezometer network was installed to further characterize the groundwater beneath the Site. Twenty-two piezometers were installed pursuant to 30 TAC §330.242, specifically for the intended delineation, monitoring, and permeability testing of the groundwater at the Site. Shallow/deep piezometers were installed at nine locations (E-1D/1S, E-2D/2S, E-3D/3S, E-4D/4S, E-5D/5S, E-6D/6S, E-7D/7S, E-8D/8S, and E-9D/9S) in the surficial clay, surficial clay/claystone interface, and claystone/limestone interface. Additionally, piezometers E-23S, E-34S, and E-36S were installed in surficial clay and E-24S was installed in the surficial clay/claystone interface.

Drilling services were performed by Core Tech Drilling, Inc. of San Marcos, Texas, with one boring/piezometer completion performed by TWF Drilling of Tyler, Texas. Core Tech Drilling, Inc. provided equipment and personnel for drilling, soil sampling, and piezometer installation. An Earth Tech staff geologist was on Site during the field effort to direct piezometer installation activities and to log retrieved soil and rock core samples. A total of 22 PVC piezometers were installed at the Site. Soil borings advanced to accommodate piezometers ranged in depth from approximately 23 to 106 feet bgs and were completed using HSA or a combination of HSA/rock coring drilling methods. The boreholes advanced to accommodate the piezometers were drilled with six-inch or eight-inch diameter HSA.

Per the TAC requirement, two-inch Schedule 40 PVC screen and casing was used to construct the piezometers. The screen intervals were generally 10 and 15 feet in length, incorporating machine-slotted 0.010-inch PVC, and installed with a 12/20 silica sand filter pack media filling the borehole annulus around the screened interval. The remaining annular space was sealed using 3/8-inch bentonite chips. The bentonite chips were placed on top of the filter pack to approximately two feet bgs and hydrated with water. Piezometers were completed above-grade and housed in a protective casing.

Drill cuttings resulting from boring advancement were left at each location in the Site expansion area. Piezometer completion logs are presented in Appendix III-4.F. A summary of piezometer construction data is presented in Table III-4.5.

Piezometers that contained water after completion were developed by bailing and purging to remove sediment. Development was accomplished by surging and over-pumping the water column until the sediments were removed, the purge water was clear to translucent, and indicator parameters of pH, conductivity, and temperature were stabilized.

Piezometers were surveyed with respect to a known datum point providing measuring point elevations (relative to msl) and easting and northing coordinates (consistent with the Texas State Plane Coordinate system) by SAM, Inc. A surveyed location map is presented as Figure III-4.12.

8.3.3.2 Groundwater Elevation Monitoring

Piezometers locations were selected to allow for representative groundwater elevation monitoring. The groundwater elevation monitoring network was determined based on their vertical and horizontal spatial arrangement. Monitoring locations were utilized for general groundwater elevation data collection. A summary of the groundwater elevation data collected from past and recently installed piezometers and wells is in presented in Tables III-4.10 and III-4.11 for the surficial clay/upper claystone and lower claystone/upper limestone units, respectively.

Depth-to-water measurements were obtained from the surveyed top-of-casing (TOC) elevation mark located on the PVC well casing. Measurements were performed with a hand held Solinst® Water-Level Indicator that is accurate to 0.01 foot. Depth-to-water data were immediately recorded on field data sheets accompanied by the appropriate measurement reference point, time, date and collection location. Groundwater elevations were calculated by subtracting the depth-to-water measurement from the surveyed reference elevation. A list of monitoring points and associated survey data for locations utilized during the groundwater characterization study are presented in Table III-4.5.

8.3.3.3 In-Situ Permeability Tests

Aquifer characteristic assessments were performed to determine in-situ permeability of the saturated media underlying the Site. In-situ hydraulic conductivity tests ("slug tests") were performed on 18 of the 22 installed piezometers located in the expansion area (four piezometers were dry at the time slug testing was conducted). Eighteen piezometers were tested using the variable head test methods. The variable head test methods used either the traditional manual slug method or baildown test method.

<u>Variable Head ("Manual Slug") Test Method</u>

The variable head ("manual slug") tests were conducted using a four-foot long by 1.25-inch diameter stainless steel slug with rope, Instrumentation Northwest, Inc. (INW) PS9800 15 pounds per square inch (psi) pressure transducers, INW AquiStar® DL-2 data logger and a Solinst® Water-Level Indicator. A computer, using Aqua 4 for Windows (Version 1.62), was used to download the data from the data logger and AQTESOLV™ for Windows (Version 3.01), an aquifer test analysis software package (HydroSOLVE, Inc., 2000), was used to analyze the data. Data and AQTESOLV™ for Windows output are included in Appendix III-4H.

Prior to conducting the field tests, a static water level and depth to bottom of the piezometer were recorded with a Solinst® Water-Level Indicator. Next, a transducer was lowered into the piezometer and the static transducer level was recorded on the field form. Both rising and falling head tests were performed on each piezometer. To initiate a falling head test, the slug was lowered into the piezometer to displace the water level upward. At specific time intervals, the data logger recorded the falling water levels measured by the transducer. Readings were taken until the water level had recovered to 100 percent of the initial water-level change. To initiate a rising head test, the slug was raised out of the water, displacing the water level downward. At specific time intervals, the data logger recorded the rising water levels measured by the transducer. Readings were taken until the water level had recovered to 100 percent of the initial head change. Six piezometers (E-1S, E-3S, E-5S, E-7S, E-23S, and E-34S) were tested by this method; however, data for E-3S was considered invalid.

Variable Head ("Baildown") Test Method

The variable head ("baildown") tests were different from the manual slug test method in that no transducer was used to record the water level data and a bailer was used to displace the water level in the well. Similar to the manual slug test method, the water level was displaced (rising test only) and a Solinst® Water-Level Indicator was used to perform the tests. A computer was used to analyze the data with the use of AQTESOLV™ for Windows (Version 3.01), an aquifer test analysis software package (HydroSOLVE, Inc., 2000). Data and AQTESOLV™ for Windows output are included in Appendix III-4H.

Prior to conducting the field tests, a static water level and depth to bottom of the piezometer were recorded with a Solinst® Water-Level Indicator. Rising head tests were performed on each piezometer. To initiate a rising head test, a bailer was used to remove water from the well. The last bailer full of water was removed rapidly and rising water levels were collected with a Solinst® Water-Level Indicator at the start of removal of the bailer. Readings were taken until the water level had recovered to approximately

85 percent for most of the piezometers or until a minimum of 48 hours of data were collected. Piezometers (E-4D, E-5D, E-6S, E-6D, E-7D, E-8D, and E-9D) had water-level recoveries between 82 and 100 percent of the initial head change; however, five piezometers (E-1D, E-2S, E-2D, E-3D, and E-4S) only had between 17 to 70.5 percent recovery of the initial head change.

Selection of the type curve and data plot of the natural logarithm of time versus the ratio of the static water level over the initial displacement was used for the calculation of hydraulic conductivity (Cooper et. al., 1967). Data output sheets from AQTESOLV™ are provided in Appendix III-4H. Results of the analyses are discussed in Section 8.4.6.1.

8.4 Investigation Results

The following sections discuss the results obtained from the 2004 Earth Tech field investigation. Specific attention is given to the geologic setting; groundwater occurrence; groundwater quality; recharge/discharge; groundwater level fluctuations; hydraulic conductivity; hydraulic head relationships; linear flow velocity; and potential migration pathways.

8.4.1 Geologic Setting

30 TAC §330.56(e)(2) requires the identification of the uppermost aquifer and any lower aquifers that are hydraulically connected to it beneath the Site, including groundwater flow direction and rate and the basis for identification. Field observations and available geologic and groundwater level data were evaluated to gain and understanding of the hydrogeological conditions underlying the Site.

The ground surface, where not altered by Site operation, is typically encountered at approximately 700 feet msl. Based upon a review of the Site boring logs, in general, the investigated Site stratigraphy consists of alluvium (primarily clay) and some fluvial unconsolidated materials, and underlying consolidated materials (claystone and limestone). The unconsolidated deposits consist predominantly of clays, with sands and gravels primarily located on the eastern edge of the expansion property. The consolidated deposits consist of claystone, which is the thickest at the topographic high of the expansion area, underlain by limestone.

Three major stratigraphic units can be characterized within the investigated sediments underlying the Site down to approximately 592 feet msl. In general, these units consists of: (1) surficial clay, which generally occurs between elevations 717 and 630 feet msl, thicknesses range typically from 8.6 to 60 feet, and mainly classifies as lean clays (CL), with silty, clayey sands and gravels included to a lesser extent;

(2) claystone, which typically occurs between elevations 705 to 630 feet msl and thicknesses range typically from 5 to 49 feet; and (3) limestone, which is encountered below 668 feet msl to 607 feet msl. The claystone and limestone units dip significantly to the east with minor undulations ("lows' and "highs") associated with their respective surfaces. The geologic cross sections (Figures III-4.16 through III-4.18) and isopach and structure maps (Figures III-4.19 through III-4.23) depict the Site stratigraphy. For individual boring lithologic descriptions, saturated intervals, and unit thicknesses, the site-specific boring logs are included in Appendix III-4C.

8.4.2 Groundwater Occurrence

Groundwater occurrence within the investigated, unconsolidated sediments of the Site are found in the surficial clay deposits that are primarily comprised of clay and to a lesser extent sand and gravel deposits that occur in the lower portion of the unit and located primarily on the eastern edge of the expansion property. Three units underlie the Site to include surficial clay, claystone, and limestone. These units are comprised mainly of fine-grained materials and are considered part of an unconfined aquifer, with water levels occurring within the unconsolidated surficial clay. Some of the coarse-grained material within the fine-grained material may be confined, as evidenced from slug test data.

The fine-grained deposits of the surficial clay are primarily classified as lean clay (CL), as previously described in Section 6.3.1.1. Laboratory hydraulic conductivity tests (horizontal and vertical permeability) were performed on the fine-grained deposits of the surficial clay. The laboratory hydraulic conductivity for these fine-grained deposits ranged from 6.1×10^{-10} cm/sec to 5.7×10^{-8} cm/sec for the most recent 2004 investigation. Laboratory test results are summarized in Table III-4.7 and in Appendix III-4G.

Fine and coarse-grained materials were also field tested to estimate flow. In-situ hydraulic conductivity tests for fine and coarse-grained deposits within the surficial clay ranged from 7.0×10^{-8} cm/sec to 6.9×10^{-2} cm/sec from the most recent 2004 investigation results at the Site. Fine-grained deposits ranged from 7.0×10^{-8} cm/sec to 7.3×10^{-6} cm/sec, consistent with previous in-situ hydraulic conductivity tests that ranged from 4.9×10^{-7} cm/sec to 9.6×10^{-6} cm/sec. Coarse-grained deposits ranged from 5.4×10^{-4} cm/sec to 6.9×10^{-2} cm/sec. The recent 2004 investigation in-situ test results are summarized in Table III-4.12 and included in Appendix III-4H. The coarse-grained deposits within the surficial clay unit beneath the expansion area were also evaluated for continuity and potential influence on groundwater flow. The coarse-grained deposits are shown on the geologic cross sections (Figures III-4.16 through III-4.18) and coarse-grained isopach map (Figure III-4.20).

The claystone underlies the surficial unconsolidated unit. Flow within the claystone is estimated to be primarily through secondary porosity features. Laboratory permeability tests (horizontal and vertical) performed on the claystone deposits ranged from 7.1×10^{-11} cm/sec to 3.4×10^{-8} cm/sec. Laboratory test results are summarized in Table III-4.8 and in Appendix III-4G.

The limestone underlies the claystone unit. Flow within the limestone is estimated to be primarily through secondary porosity features. Laboratory rock permeability tests performed on the limestone deposits ranged from 4.5×10^{-10} cm/sec to 1.1×10^{-9} cm/sec. Laboratory test results are summarized in Table III-4.9 and in Appendix III-4G.

The hydraulic conductivity of the lower claystone and upper limestone was evaluated from in-situ hydraulic conductivity tests performed in nine piezometers installed during the 2004 groundwater investigation. In-situ hydraulic conductivities for the lower claystone and upper limestone ranged from 6.8×10^{-8} cm/sec to 5.7×10^{-7} cm/sec, with a calculated geometric mean of 2.2×10^{-7} cm/sec, which is consistent with five previous in-situ hydraulic conductivity tests that ranged from 5.1×10^{-8} cm/sec to 9.4×10^{-5} cm/sec, with an overall geometric mean of 3.7×10^{-7} cm/sec. Investigation in-situ test results are summarized in Table III-4.12 and included in Appendix III-4H.

Based upon depth-to-water data collected from the monitoring wells and piezometers, groundwater elevations in the surficial clay/upper claystone range from approximately 695 feet msl to 658 feet msl and in the lower claystone/upper limestone range from approximately 702 feet msl to 640 feet msl. Groundwater elevation data indicate that groundwater movement in all the units are radially outward towards the north, east and southerly directions. Figures III-4.24 and III-4.25 present the inferred groundwater potentiometric surfaces for the surficial clay/upper claystone ("shallow') and lower claystone/upper limestone ("deep") piezometers/ monitoring wells, respectively, for August 31, 2004. Figure III-4.26 and Figure III-4.27 represents the potentiometric surfaces of the historical "high" water levels recorded on March 16, 1992 for both groundwater-bearing units. A summary table of available groundwater elevation data collected to date is presented in Tables III-4.10 and III-4.11, for the surficial clay/upper claystone and lower claystone/upper limestone units, respectively.

In general, the units beneath the Site consist of one hydrostatic unit, with three stratigraphic units (surficial clay, claystone, and limestone) down to approximately 592 feet msl, as seen in Table III-4.6. All three units are confined and act as an one aquitard unit to the Edwards aquifer, approximately 600 to 700 feet bgs (100 to 0 feet msl). Technically, the three units are unconfined; however, when analyzed for field permeability tests may show semi-confined or confined conditions. These units are not

hydraulically connected with the underlying Edwards aquifer, which is a major aquifer in central Williamson County and Central Texas based on site stratigraphy. In the vicinity of the Site, the upper Edwards aquifer units are at a depth of approximately 615 feet, separated from the coarse grained surficial clay and lower claystone units by approximately 390 feet of Austin Group, which has a low to moderate infiltration capacity that generally does not transport groundwater through the unit unless fractures are aligned. The Austin Group is underlain by the Eagle Ford Group, which is approximately 70 feet thick at the Site and is considered relatively impermeable. Below the Eagle Ford is the Buda Limestone and the Del Rio Clay, both of which do not yield water in the vicinity of the Site. The average potentiometric surface of the Edwards aquifer, in close proximity to the Site, is approximately 587 feet msl (Senger et al, 1990), which is 102 feet below the average potentiometric surface of the surficial clay/upper claystone and 97 feet below the average potentiometric surface of the lower claystone/upper limestone.

8.4.3 Groundwater Quality

As part of the groundwater investigation, historical analytical data provided by WMTX were reviewed. In 30 TAC §330.56(e)(4), the TCEQ requests information concerning the existence of any groundwater contamination plume resulting from a permitted landfill. As evidenced by historical water-quality analyses results of samples collected from the groundwater monitoring well network from December 23, 1991 to present, there is no groundwater contamination plume present within the uppermost aquifer at the Site. Minor fluctuations in water-quality data have occurred over the years; however, no significant changes or trends of increased concentrations have been observed. Groundwater monitoring is presently conducted on a semi-annual basis.

8.4.4 Recharge/Discharge

The conceptual groundwater system in the vicinity of the Site consists of a low hydraulic conductivity surficial aquitard (surficial clay) with confined, discontinuous coarse-grained material beneath the expansion area, which overlies the uppermost bedrock unit (claystone). The claystone is underlain by a thick bedrock unit (limestone). The surficial clay unit consists primarily of fine-grained clays. The claystone and limestone bedrock deposits are generally low hydraulic conductivity units, and lack continuous coarser deposits.

As water from precipitation falls on the ground surface, infiltration enters the groundwater system and percolates slowly downward through the surficial clay or open bodies of water that might be deep enough to be connected to the surficial clay and/or coarse-grained units within the surficial clay deposits. The

slow percolation of water through the fine-grained deposits generally results in the groundwater surface mimicking the surface topography.

The surface recharge characteristics of the Site and adjacent areas can be evaluated in part through soil survey information (USDA SCS, 1983). Approximately one-half of the Site property is overlain by Branyon Clay soils. These soils are thick, expansive, high clay soils that develop cracks two to five feet deep when dry. Infiltration is rapid when these cracks are present; however, the cracks close as the soil becomes wet and infiltration rates become very slow. The remainder of the Site is mantled by Krum Silty Clay, Houston Black Clay and Altoga Silty Clay Loam. The general area of the Site is dominated by the Branyon and Houston Black Clay Soils. The Houston Black Clay Soils have similar infiltration characteristics to the Branyon Soils. Krum Soils occur in generally narrow bands on surface drainage divides and Altoga Soils occur on parallel steeper slopes. Krum and Altoga soils are less expansive than the Branyon and Houston Black soils, and are not subject to excessive cracking when dry. (CJI and MFG, 1995)

Recharge to the calcareous claystone unit likely occurs as infiltration during periods of high precipitation/low evapotranspiration, and possibly from seepage of intermittent streams (such as Mustang Creek) or ponds. Water movement within the claystone unit is highly variable and controlled primarily by the density and orientation of fractures, and is dependent to some degree on recharge conditions (MFG and CJI, 1995). The relatively small fluctuations associated with recharge/discharge of the claystone unit and the low effective porosity make this unit susceptible to large seasonal changes in water levels. During dry periods with little or no recharge, water levels can be expected to decline slowly as water leakage occurs at very slow rates, downward and laterally (see Section 8.4.7.2). In times of high infiltration, water levels may rise to levels where fracture density is potentially greater and the lateral flow component relatively larger, although still quite small. At such times, discharge could occur as base flow to Mustang Creek, in the south part of the permitted Site, and to the unnamed tributary, within the northern permitted Site, and by evapotranspiration.

In summary, water recharge occurs locally by vertical infiltration of precipitation and possibly seepage from surface drainage features during high flow events. Water conditions are quite variable across the Site and water levels are subject to fluctuations in response to both short-term and long-term climatic conditions. Horizontal water movement is slow and the predominant discharge mechanism is evapotranspiration. Surface drainage features can be seen on the 2002 aerial photograph included as Figure III-4-3.

8.4.5 Groundwater Level Fluctuation

Groundwater data have been collected from 1988 to the present at the Site. Over this time period, water levels from the surficial clay/upper claystone ranged from 716 to 655 feet msl, and the lower claystone/upper limestone ranged from 716 to 611 feet msl. The most noted fluctuation in water levels at the Site for any one well over this time period was approximately 39.6 feet for top of limestone in A-3D. However, the first data point is the lowest elevation by about 15 feet and has typically been approximately 30 feet higher since 1992. Fluctuations could be due to a number of reasons: 1) periodic removal of water from the wells at the Site (i.e., dewatering excavations or lowering, baildown tests); 2) excavation at the Site; or 3) normal seasonal variations within the unit. In general, water levels are at the lowest points in late summer and early fall (excluding "wet" years) about one to three months after the lowest amount of precipitation. Historical data are presented in Tables III-4.10 and III-4.11, for the surficial clay/upper claystone and lower claystone/upper limestone units, respectively.

8.4.6 Hydraulic Conductivity

Field derived hydraulic conductivity (permeability) results were obtained by in-situ hydraulic conductivity tests that were performed on July 21 to 26, 2004. The tests were attempted on all 22 piezometers located in the expansion area; however, only 18 tests were completed since four piezometers were "f\dry" at the time of testing. Piezometers were tested using the variable head test methods by either the traditional "manual slug" method or "baildown" method. Test methods are described in Section 8.3.3.3. Results are presented in Table III-4.12 and Appendix III-4H.

8.4.6.1 Slug Tests

In-situ hydraulic conductivity tests were performed to determine the horizontal hydraulic conductivity of the formation materials in the vicinity of each piezometer screened interval. The primary stratigraphy in the tested piezometers consisted mainly of sands and gravels within the surficial clay unit, clays in the surficial clay unit, and claystone and limestone in the bedrock units. The hydraulic conductivities ranged from 5.4×10^{-4} to 6.9×10^{-2} cm/sec for coarse-grained deposits in the surficial clay, 7.0×10^{-8} to 7.3×10^{-6} cm/sec for the fine-grained deposits in the surficial clay/upper claystone units, and from 6.8×10^{-8} to 5.7×10^{-7} cm/sec for the lower claystone/upper limestone unit.

Previous in-situ hydraulic conductivity tests ranged from 4.9×10^{-7} to 9.6×10^{-6} cm/sec for surficial clay and/or upper claystone unit and from 5.1×10^{-8} to 9.4×10^{-5} cm/sec for the lower claystone and/or upper limestone unit. Representative values from literature (Freeze and Cherry, 1979) of hydraulic

conductivities for sands and silty sands are 1 to 10^{-5} cm/sec, and silts and clays are 10^{-3} to 10^{-10} cm/sec. Representative values from literature (Freeze and Cherry, 1979) of hydraulic conductivities for limestones are 10^{-4} to 10^{-8} cm/sec, and shales (closest representative unit to claystone) are 10^{-7} to 10^{-11} cm/sec.

The calculated hydraulic conductivity values for the corresponding lithologies are within the literature range of values for piezometers. The calculated hydraulic conductivity values for the corresponding lithologies are representative of the hydrostratigraphic units beneath the Site. Test results are presented in Table III-4.12 and Appendix III-4H.

8.4.6.2 Laboratory Permeability

Samples were collected to determine the vertical and horizontal permeability of the in-situ materials below the Site. A total of 18 laboratory permeability tests (seven vertical, seven horizontal, and four rock) were conducted. Six samples from the fine-grained surficial clay, nine samples from the claystone, and three samples from the limestone were tested. Vertical, horizontal, and rock laboratory permeabilities ranged from 7.1×10^{-11} cm/sec to 5.7×10^{-8} cm/sec and presented in Tables III-4.7 through III-4.9.

The horizontal and vertical laboratory permeability tests were generally the same order of magnitude. Horizontal laboratory samples ranged from 1.1×10^{-10} to 5.7×10^{-8} cm/sec with a geometric mean of 7.2×10^{-9} cm/sec. Vertical laboratory samples ranged from 7.1×10^{-11} to 1.8×10^{-8} cm/sec with a geometric mean of 2.2×10^{-9} cm/sec.

8.4.7 Hydraulic Head Relationships

As part of the site characterization activities, both horizontal and vertical hydraulic gradients were calculated, so as to better comprehend groundwater flow characteristics at the Site. Horizontal hydraulic gradients are presented in Table III-4.13 and vertical hydraulic gradients are presented in Table III-4.14.

8.4.7.1 Horizontal Hydraulic Gradient

Horizontal hydraulic gradients were calculated for the surficial clay/upper claystone and lower claystone/upper limestone groundwater flow system utilizing groundwater elevation data collected on March 16, 1992 and August 31, 2004. A summary of the calculated horizontal gradients is presented in Table III-4.13. The horizontal hydraulic gradient for the surficial clay/upper claystone, calculated from Figures III-4.24 and III-4.26, is approximately 0.017 ft/ft.

Horizontal hydraulic gradients were calculated for the lower claystone/upper limestone groundwater flow system utilizing groundwater elevation data collected on March 16, 1992 and August 31, 2004. The horizontal hydraulic gradient for the lower claystone/upper limestone, calculated from Figures III-4.25 and III-4.27, is approximately 0.014 ft/ft.

8.4.7.2 Vertical Hydraulic Gradient

Vertical hydraulic gradients were calculated using water-level data collected on August 31, 2004. A summary of the calculated vertical gradients is presented in Table III-4.14. Vertical hydraulic gradients were compared between the surficial clay and or surficial clay/upper claystone and the lower claystone/upper limestone. The vertical gradients between these units range from approximately 0.76 upward to 0.72 ft/ft downward, with an average vertical hydraulic gradient of 0.05 ft/ft downward.

8.4.8 Average Linear Flow Velocity

The average linear flow velocity of horizontal groundwater flow in surficial clay/upper claystone (shallow) was calculated to be approximately 3.6×10^{-3} feet/day (1.3 feet/year). The claystone/limestone (deep) was calculated to be approximately 9.3×10^{-4} feet/day (0.3 feet/year). Calculations and assumptions are presented in Table III-4.15. The flow is calculated based on a modification of Darcy's Law:

$$V = \frac{KI_h}{n_e}$$

Where: V = average linear velocity
K = horizontal hydraulic conductivity
I_h = horizontal hydraulic gradient
n_e = effective porosity

Surficial Clay/Upper Claystone (Shallow)

A horizontal hydraulic gradient (I_h) of approximately 0.017 ft/ft was used. The effective porosity (n_e) was estimated based on a literature value of 0.4 for clay material. Based on the in-situ hydraulic conductivity test data presented in Table III-4.12 and Appendix III-4H and previous reports, a geometric mean horizontal hydraulic conductivity (K) of 3.0×10^{-5} cm/sec (8.5×10^{-2} feet/day) was estimated from the recent and previously completed in-situ hydraulic conductivity tests. The groundwater velocity calculated for the Site is typical of groundwater velocities encountered for these types of materials.

Claystone/Limestone (Deep)

A horizontal hydraulic gradient (I_h) of approximately 0.014 ft/ft was used. The effective porosity (n_e) was estimated based on a literature value of 0.015 for the limestone material. Based on the in-situ hydraulic conductivity test data presented in Table III-4.12 and Appendix III-4H, a geometric mean horizontal hydraulic conductivity (K) of 3.7 x 10^{-7} cm/sec (1.0 x 10^{-3} feet/day) was estimated from the recent and previously completed in-situ hydraulic conductivity tests. The groundwater velocity calculated for the Site is typical of groundwater velocities encountered for these types of materials.

8.4.9 Potential Migration Pathways

In accordance with 30 TAC §330.56(d)(5)(C)(iv), the most likely pathways for pollutant migration in the event that the primary barrier liner system is penetrated requires an analysis of the hydrogeological framework of the Site.

The surficial clay and claystone are the only units that are in direct contact with the landfill footprint. The majority of the Site is fine-grained, low yield, unconsolidated materials and low yield, consolidated materials as shown and presented by the geotechnical permeability and field permeability data. Permeabilities typically ranged from 10^{-6} to 10^{-8} cm/sec for the field horizontal permeabilities and 10^{-8} to 10^{-11} cm/sec for geotechnical laboratory permeabilities (horizontal and vertical). Also, with the majority of the proposed base grades terminating in the claystone, approximately 10 to 15 feet above the top of limestone, if pollutants were to penetrate the primary barrier liner system, the fine-grained, low yield, unconsolidated materials and low yield, consolidated materials would inhibit distribution of pollutants. In addition, if groundwater flow were affected by the construction of the landfill, the groundwater flow in the vicinity of the expansion would be detected by the monitoring wells, which encompass the entire perimeter of the expansion acreage.

A minority of the Site contains coarser grained materials that potentially will conduct flow if the primary barrier liner system is penetrated. The proposed base grades are to be terminated below this coarser material; however, some of the side slopes of the waste management units (WMUs) will come in contact with this coarser grained materials. The coarse-grained units are located on the eastern portion of the expansion area; located in the northeast corner in the vicinity of E23S and in the southeastern/eastern portion to include the area surrounding borings E-32 and E-35. The full extent of the coarser grained materials to the east and northeast of the expansion area is unknown. These deposits may be

discontinuous; however, the field horizontal conductivities calculated a geometric mean of 5.4 x 10^{-3} cm/sec.

Based on the site-specific geotechnical and hydrogeological data, the fine-grained material is a barrier to potential vertical contaminant migration pathways in the majority of the Site. Only a minority of the Site, located on the eastern edge of the expansion area, contains coarser grained materials that may be laterally discontinuous, but have the potential for migration.

9.0 GROUNDWATER MONITORING SYSTEM

9.1 Proposed Groundwater Monitoring Program

In accordance with 30 TAC §330.401 - .421, this section presents the detailed plans and engineering report describing the groundwater monitoring program to be implemented to meet the requirements of 30 TAC §330.403 relating to Groundwater Monitoring Systems.

An enhanced groundwater monitoring well network was proposed in accordance with 30 TAC §330.403. The enhancements proposed for the Site's groundwater monitoring system included the installation of monitoring wells screened in the lower claystone/upper limestone unit and monitoring wells screened in the surficial clay (coarse-grained zones). The monitoring well network was designed to be protective of State waters and human health and the environment in the unlikely event that a release occurs under the TCEQ's currently approved groundwater monitoring program. The groundwater monitoring network has been developed based on the present knowledge of groundwater flow at the Site. It is anticipated that as cell construction progresses from west to east, the groundwater flow may be impeded in an easterly direction; however, monitoring wells were included to encompass the entire perimeter of the expansion acreage and would detect any releases from WMUs that may occur.

The final approved groundwater monitoring network consists of 36 monitoring wells located along all perimeters of the combined present and proposed landfill footprints. Twenty-six of the present and to-be-installed monitoring wells are/will be installed in the lower claystone/upper limestone unit and 10 monitoring wells will be installed in the shallow, coarse-grained material of the surficial clay unit that is found along the eastern portion of the proposed eastern expansion property. Three wells (MW-09A, MW-11, and MW-12) in the TCEQ-approved monitoring well network are installed in the lower claystone/upper limestone unit and are the only upgradient monitoring wells in the final approved monitoring well network due to groundwater flow. However, when the permit modification is approved, MW-09A will be decommissioned and only two wells (MW-11 and MW-12) will remain as upgradient

monitoring locations. The remaining lower claystone/upper limestone unit wells and the shallow, surficial clay unit wells are all considered downgradient to groundwater flow and landfill operations, as noted on Figure III-4.28. The monitoring wells will be installed on a phased schedule as described in Attachment 5, Section 5.0 of this permit application.

Eight monitoring wells (MW-06 through MW-08, MW-09A, and MW10 through MW-13) are currently utilized to satisfy the monitoring requirements for the Site, a Type I MSW facility. These wells are all screened in the lower claystone/upper limestone unit.

See Attachment 5, Section 5.0 for a description of the groundwater monitoring system.

Figure III-4.28 depicts the permitted landfill limits of waste, the proposed waste limits of the expansion area, the POC, and the to-be-installed groundwater monitoring system. Table III-4.16 summarizes the monitoring well network including well designation, estimated or actual well depth, well status (new or existing groundwater monitoring well), estimated or actual location, estimated or actual screen interval, and type of monitoring point (upgradient, downgradient, and/or POC).

As defined under 30 TAC §330.3, the POC line has been established at locations that parallel the surface projection of a vertical plane that intersects the uppermost water-bearing zone at the most immediate downgradient location from the WMUs. The locations and number of POC wells have been selected based on several critical hydrogeologic and design parameters. These parameters are discussed in detail below.

The positions of the POC and the monitoring wells were selected based on:

- Stratigraphic position;
- Groundwater potentiometric maps;
- Horizontal position relative to existing monitoring wells;
- Local hydraulic gradient and hydraulic conductivity; and,
- Future design considerations.

Stratigraphic Position: Vertical locations of the monitoring wells are based on site stratigraphy and hydrogeology. The depths of the wells and the screened intervals have been selected based on isopach and structure maps generated from data collected during the 2004 site investigation (Figures III-4.19 through III-4.23). The monitoring wells are located vertically in the surficial clay unit (coarse-grained materials) and the lower claystone/upper limestone, based upon the geometry of the expansion area and

recent and historical potentiometric surfaces in the surficial clay/upper claystone and lower claystone/upper limestone units, shown in Figures III-4.24 through III-4.27. The POC wells have been designed to be strategically positioned both spatially and vertically so as to monitor groundwater passing beneath the POC in the uppermost water-bearing zone. The surficial clay/upper claystone monitoring wells (MW-21S through MW-30S) will be installed to a depth that encounters the entire thickness of the coarser grained materials of the surficial clay unit, which ranges in thickness from one to 20 feet on the eastern expansion property. Based on Figures III-4.20 and III-4.21, completion elevations of the shallow wells will range between 675 to 642 feet msl. The lower claystone/upper limestone monitoring wells (MW-14 through MW-33) will be installed to a depth that encounters the contact between the Lower Taylor Marl and the underlying Austin Chalk. Based on Figure III-4.23, completion elevations of the wells will range between 678 to 619 feet msl. The to-be-installed monitoring wells will be screened to monitor groundwater emanating from the base of the landfill from upgradient to passing the downgradient POC. All existing and new wells are/will be constructed in accordance with TCEQ 30 TAC §330.421. Groundwater monitoring well installation will be performed under the guidance of a qualified groundwater scientist.

Groundwater Potentiometric Maps: The use of groundwater potentiometric maps was fundamental in the strategic placement of the POC and detection monitoring wells. The monitoring well locations were based on the spatial distribution of the resultant equipotential lines on the groundwater contour maps. The monitoring wells are located horizontally in the lower claystone/upper limestone and in the surficial clay unit (coarse-grained materials) based upon the geometry of the expansion area and recent and historical potentiometric surfaces in the surficial clay/upper claystone and lower claystone/ upper limestone, shown in Figures III-4.24 through III-4.27.

Horizontal Position: Spatial distribution of the upgradient (background) and downgradient/POC wells is straightforward based on the hydrogeometry of the Site and particle flow-paths observed. See Attachment 5, Section 5.0 for a discussion of well spacing. The to-be-installed monitoring wells, in conjunction with the existing monitoring network wells, are believed to be designed to adequately detect a breakthrough of constituents of concern (COCs).

Local Hydraulic Gradient and Hydraulic Conductivity: Based on the geotechnical results (Section 7.0), the clay liner will maintain a minimum permeability (hydraulic conductivity) of 1.0×10^{-7} cm/sec. The claystone and fine-grained clay, and to a lesser extent, the coarse-grained material are adjacent to a portion of the sidewalls and base of the proposed landfill expansion. The permeability of fine-grained

soils and the claystone typically ranged from 10^{-6} to 10^{-8} cm/sec for the field horizontal permeabilities and 10^{-8} to 10^{-11} cm/sec for geotechnical laboratory permeabilities (horizontal and vertical). The permeability of coarse-grained soils is on the order of 10^{-3} cm/sec (based on the geometric mean of the "slug" tests). Average horizontal hydraulic gradients ranged from approximately 0.01 to 0.02 ft/ft towards the northeast and southeast portions of the expansion area (calculated from potentiometric maps). There is also a component of vertical gradient between the surficial clay/upper claystone and lower claystone/upper limestone that has an average vertical hydraulic gradient of 0.05 ft/ft downward (assuming static conditions have been obtained).

Monitoring of the shallow surficial clay/upper claystone unit was not recommended by CJI and MFG in the Soils Investigation Report for Williamson County Landfill Permit Amendment Application No. 1405-A, completed in February 1995. The decision not to monitor the surficial clay/upper claystone unit was based on the stratigraphic data collected during their investigations and previous investigations that showed the coarse-grained deposits to be isolated occurrences on the permitted landfill. In addition, the report stated that any ephemeral water-bearing zones that developed in the clay unit after high rainfall periods would probably dissipate by evapotranspiration and to a lesser extent by slow seepage into the unfractured claystone adjacent to fractures. However, based on the widespread extent of the coarse-grained deposits along the eastern portion of the expansion area (Figure III-4.20), field horizontal permeabilities of 10^{-3}, and potential contact with the proposed side walls of the landfill, it is recommended that the surficial clay/upper claystone unit be monitored where coarse-grained deposits are encountered, as stated above.

Future Design Considerations: See Attachment 5, Section 5.0.

9.2 Monitoring Well Construction

The proposed enhancement to the present groundwater monitoring network will meet all of the requirements of 30 TAC §330.421, related to monitoring well drilling and completion specifications. Each monitoring well location will be drilled by a Texas-licensed driller and supervised by a qualified geologist or engineer. During drilling of the monitoring well, a log of the boring shall be made by a qualified geologist or engineer who is familiar with the geology of the area.

Each well will be installed using HSA or a combination of HSA/rock coring drilling methods and sampled using either a five-foot CME sampler or a five-foot NQ rock core sampler, or equivalent sampling equipment, until the desired depth is achieved. The monitoring wells that will be installed in the

base of claystone/top of limestone unit should be advanced through the unconsolidated material using a six-inch or eight-inch diameter HSA. When consolidated material is encountered, sampling should be conducted using a rock core barrel. Water will be required during the rock coring procedures and can be obtained from the on-site water tank supplied by city water from JWSC.

When groundwater is encountered, drilling will be stopped and water-level measurements will be recorded on drilling logs. Drilling will then resume and the borings will be augured to the desired depth. All monitoring wells in the surficial clay/upper claystone unit will be installed directly into six- or eight-inch diameter boreholes once the augers are removed. All monitoring wells in the base of claystone/top of limestone unit will be installed into six-inch diameter boreholes.

Each monitoring well will be constructed with two-inch I.D., flush-threaded, Schedule 40 PVC screen and casing. The 0.010-inch machined slotted screen will generally be 10 feet in length for the coarse-grained deposits in the surficial clay unit and 15 feet in length for the lower claystone/upper limestone unit. The filter pack will be constructed of 12/20 silica sand and installed from the bottom of the borehole to approximately two feet above the PVC screen. The remaining annular space will be sealed using bentonite chips. The bentonite chips will be placed on top of the filter pack to approximately two feet bgs and hydrated with water.

The surface completion for each monitoring well will consist of a four-foot by four-foot square by six-inch thick concrete pad, reinforced with rebar; and protected by a six-inch diameter steel protective cover with lock (approximately 3.5 feet above the concrete pad). The surface completion will also include four steel guard posts (one at each corner of the concrete pad, filled with concrete, and painted yellow); and pea gravel will be placed in between the PVC well and protective casing. A 3/8-inch diameter weep hole will be drilled through the protective casing (approximately two inches above the top of the concrete pad) to prevent moisture buildup inside the protective casing. A marker will also be placed on the concrete pad and will be used for the surveyed vertical elevation of the concrete pad at each well.

9.3 Background Monitoring

See the site-specific GWSAP (Part III, Attachment 11, Section 4.0), for details regarding background monitoring

9.4 Detection Monitoring

As this Site is currently operating under a TCEQ-approved GWSAP, groundwater monitoring will continue to be operated in accordance with 30 TAC §330.401-.419 and the proposed GWSAP located in Attachment 11 of this permit application. The procedures for acquiring samples from this system, the analysis of those samples, and the management of the water-quality data resulting from these analyses are presented in the modified GWSAP (Attachment 11 of this application).

10.0 REFERENCES

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11.0 STANDARD OF CARE

The conclusions and recommendations contained in this report are based upon professional opinions with regard to the subject matter. These opinions have been arrived at in accordance with currently accepted hydrogeologic and engineering standards and practices applicable to this location.