Progress Energy

December 17, 2010

Mr. Michael P. Halpin, P.E.
Administrator
Florida Department of Environmental Protection
Siting Coordination Office
3900 Commonwealth Blvd.
MS No. 48
Tallahassee, Fl 32399-3000

Certified Mail
Return Receipt Requested

Re: Florida Power Corporation
d/b/a Progress Energy Florida, Inc. (PEF)
Crystal River Fossil Plants
Petition to Modify Site Certification PA 77-09
Ash Storage Vertical Expansion and Other Misc. Projects (Mod P)
Response To FDEP's Request for Additional Information
Dated October 11, 2010

Dear Mr. Halpin:

On September 10, 2010, the Florida Department of Environmental Protection's (FDEP or Department) Siting Coordination Office received a petition from Progress Energy Florida, Inc. (PEF) for a modification to the Crystal River Energy Complex (CREC) Site Certification. On October 11, 2010, the FDEP submitted a letter to PEF requesting additional information (RAI). In FDEP's RAI, the Southwest District Office (SWD) Solid Waste Program, Industrial Wastewater Program and Environmental Resource Permitting Program provided questions and/or comments. For ease of reference, PEF is restating below in italics the Department's questions/comments followed by PEF's response. Copies of this submittal are also being provided to Mr. Danny Stubbs, FDEP Southwest District.

A. SOLID WASTE PROGRAM

The CREC manages coal combustion products (CCPs) generated at the facility, in this case fly ash and bottom ash, in a dry storage/disposal area. The ash storage area at the CREC is governed under specific Conditions of Certification issued pursuant to Florida's Electrical Power Plant and Transmission Line Siting Act, Part II, Chapter 403, Florida Statutes. More specifically, the ash storage area is addressed in Condition of Certification V1.H.1., Progress Energy Florida Conditions of Certification Crystal River Complex , Unit 3 Nuclear Power Plant, Unit 4 and Unit 5 Fossil Plant, PA 77-09M (July 9, 2009 Typical solid waste management facilities and related practices at electric utility power plants were summarized as part of the information exchange between the electric utility industry and FDEP pursuant to former Rule 62-701.720, F.A.C.., as referenced in Rule 62-701.220(4), F.A.C., and FDEP's written determinations regarding electric utility power plant solid waste disposal facilities. As you are aware, pursuant to that process, FDEP determined that Class I landfill standards are not automatically applicable to facilities like CREC's ash storage area. Examples of the Class I landfill requirements in Chapter 62-701, F.A.C., that are not automatically applied to these facilities include liner and leachate collection systems and groundwater monitoring (given that groundwater monitoring is typically addressed for these facilities in power plant site certification conditions or other relevant permits like NPDES permits). Generally, design and operational criteria for industrial solid waste disposal facilities (like the CREC ash storage area) are established on a case-by-case basis using scientific data and other information to provide reasonable assurance that applicable FDEP standards will be met.

FDEP Comment

1. ***Rule 62-701 430(1)(a), F.A.C.*** *Please provide documentation that demonstrates that the proposed vertical expansion will not cause or contribute to any leachate leakage from the existing landfill and shall not adversely affect the closure design of the landfill.*

PEF Response:

In 2010, PEF began work on developing a new lined ash storage/disposal area at the CREC. However, given the length of time that is expected to design, permit, and construct the new lined ash storage/disposal area, the amount of remaining airspace; and decreased markets for ash re-use, PEF believes that a vertical expansion (even if only as a contingency until the new ash storage/disposal area is opened) is needed.

PEF does not believe that the proposed vertical expansion will adversely impact groundwater quality or the closure design of the landfill based on: (1) the results of CCP ash leach testing and historical groundwater monitoring data (see Section 4.2.5 in the Hydrogeological and Geotechnical Site Evaluation attached to the document entitled *Supporting Technical Information* (Golder, August 25, 2010 submitted as part of PEF's September 10, 2010 petition); (2) the fact that CCPs are non-degradable and behave as a compacted earthen fill with minimal potential for internal settlement or "squeezing" of fluid from the pore spaces under the increased load that would be imposed by the proposed vertical expansion; and (3) with the increased height of embankment, the infiltration rate will actually decrease (see HELP Model run in Attachment 1).

With regard to closure, as previously discussed with FDEP staff, following approval of the vertical expansion PEF is committed to beginning the phased closure of the ash storage area, starting with an approximate 10-acre closure along the north slope by the end of 2011 (assuming approval of the vertical expansion is received in the 1stfirst quarter of 2011).

FDEP Comment

2. ***Rules 62-701.430(1)(c) , 62-701.400, and 62 - 701.320(7)(e), F.A.C. Landfill design.*** *Please explain why the proposed drawings do not include a liner between the existing waste and the proposed vertical expansion? Why does PEF propose that a liner is not required?*

PEF Response

Initially, as discussed in the introduction to this response to the Solid Waste Program's comments, liner requirements are not automatically applicable to facilities like the CREC ash storage area. In 1988, FDEP approved a vertical expansion of the CREC ash storage area and the agency did not require the installation of an interface liner at that time. As in 1988, PEF does not believe there is any environmental protection or other bases for installing such a liner for this proposed vertical expansion. As indicated in the response to FDEP Comment No. 1 above, PEF believes that the monitoring data collected and technical analyses performed demonstrates that the current CCP ash management program and the proposed increase in height of the ash storage area (without installation of an interface liner) will not have adverse impacts on groundwater quality at the site.

FDEP Comment:

3. ***Rules 62 -701.430(2) and 62-701.410(2), F.A.C. Geotechnical Investigation.***

a. *Please provide revised stability analyses that include an interface liner, equipment loading, and block and circular failures. Please include copies of the relevant portions of all references, assumptions, figures, etc., supporting the analyses.*

PEF Response:

The stability analyses presented in the *Supporting Technical Information* (Golder, August 25, 2010) included block and circular failure modes. Copies of the references quoted in the stability calculations are included in Attachment 2 to this response. A veneer stability analysis for the final cover (including equipment loading) is included in Attachment 3 to this response. As indicated in PEF's response to Comment Nos. 1 and 2 above, PEF does not consider the use of an interface liner necessary for this site from an environmental protection standpoint. Therefore, an analysis of stability incorporating an interface liner was not conducted.

FDEP Comment:

b. Engineering Report §5.4.2.

1) This section indicates that cavities in the limestone "will be addressed with a compaction grouting program scheduled for August 2010." Please provide the procedures for this compaction grouting program.

PEF Response:

A description of the compaction grouting program is included as Attachment 4 to this response.

2) This section indicates that "there were no active sinkholes identified in the immediate vicinity of the CCP disposal area, ... "However, a sinkhole investigation was not provided. Please evaluate the potential for sinkholes at the site as required by Rule 62-701.410(2)(b), F.A.C. It is the Department's understanding that active sinkholes were observed onsite during the construction of the FGD pond system, and several of the boring logs identify soft/loose soils and/or losses of drilling circulation (e.g., PB-4, PB-4B, etc.).

PEF Response:

As the comment points out, there were intervals within some of the borings drilled at the site that encountered materials of a low consistency. However, where these intervals were encountered in borings AB-1 and AB-3 they were on the order of 10 to 15 feet below the base of the fly ash. As described in the punching failure analysis under very conservative conditions, even a moderate cover of limestone materials, on the order of 5 feet would be sufficient to support the weight of the landfill over a large void (see Section 5.4.2 and Appendix I in the Hydrogeological and Geotechnical Site Evaluation attached to the *Supporting Technical Information* (Golder, August 25, 2010)). It is noted that the cavities exposed during grading work at the FGD pond referenced in the comment were few (only 3 across the footprint), small (ranging from about 3 or 5 feet in diameter), and depths of only 4 to 8 feet.

In a report on the vulnerability of the Citrus County Aquifer, the site area is mapped as an area of *"None to Low"* recharge potential and lower karst potential *">3,346 feet from karst features"-"* – see Attachment 5 *(Reference: Baker, A.E., Wood, A.R., and Cichon, J.R., 2008 (in review), The Citrus County Aquifer Vulnerability Assessment: Contract Deliverable Report Prepared for the Florida Geological Survey -- Florida Department of Environmental Protection).*

The CREC is located within Sections 28 through 36, Township 17 South, Range 16 East and parts of Sections 3, 4, 5, 9, and 10 in Township 18 South, Range 16 East. According to the Florida Geological Survey "Subsidence Incident Report' database (*Reference: http://www.dep.state.fl.us/geology/gisdatamaps/sinkhole_database.htm*)there are a total of 343 entries in Citrus County. Of those 343 entries, there are four (4) in T17S, R16E and one (1) in T18S, R16E. None of these entries were located within the Sections (each approximately 1 square mile) in which the CREC is located.

Finally, a review of a 1967 aerial photograph of the site (pre-landfill ash storage area development) does not indicate the presence of any obvious sinkholes or lineaments in the vicinity of the landfill ash storage area (see Attachment 5).

FDEP Comment:

c. Appendix I.

1) Please clarify if the figures provided in this appendix are for circular or block failures, and if the analyses included equipment loading. Please specify the equipment loading.

PEF Response:

See response to FDEP Comment No. 3a.

2) Please provide revised settlement analyses that include an interface liner and equipment loading. Please include copies of the relevant portions of all references, assumptions, figures, etc., supporting the analyses.

PEF Response

As indicated in PEF's response to FDEP Comment Nos. 1 and 2 above, PEF does not consider the use of an interface liner necessary for this site from an environmental protection standpoint. Therefore, analysis of settlement incorporating an interface liner was not conducted.

FDEP Comment:

*4. **Rules 62-701.320(7)(e) and 62-701.500, F.A.C. Landfill operation.** Please provide a landfill operation plan that includes the information required by this rule. An operation plan was not provided.*

PEF Response

The Crystal River ash storage area is permitted under the Site Certification Application and governed under the Conditions of Certification and as such, is not a landfill governed under all of the criteria set forth in Chapter 62-701 F.A.C. Therefore, the ash storage area operations plan requested by the Department is being developed to address the recent horizontal expansion of the ash storage area and the operations plan will appropriately be written to include information that typically reflects operations of the ash storage area. The operations plan will be submitted to the Department within 120 days of the approval of this modification request.

FDEP Comment:

*5. **Rules 62-701.410(1) and 62-701.510, F.A.C. Hydrogeological investigation and groundwater monitoring plan.***

a. Please note that Rule 62-701.320(18), F .A.C., defines the zone of discharge for the landfill to be a maximum of 100 feet from the disposal footprint. Rules 62-701.510(3)(a) and (b), F.A.C., identify wells within the zone of discharge as "detection" wells, and wells located at or immediately adjacent to the compliance line of the zone of discharge as "compliance" wells, respectively. In order to determine the appropriate designation in accordance with Chapter 62-701 F.A.C., please provide the distances from the edge of waste for each of the downgradient monitoring wells: TW-1, TW-4, MWI-2R2, TW-5 and TW-3.

PEF Response:

PEF disagrees with FDEP's assertion that the horizontal extent of the zone of discharge (ZOD) for the CREC's ash storage area as being "100 feet from the disposal footprint." Pursuant to FDEP's groundwater rules, an "existing installation" is afforded a ZOD extending to the facility property boundary. Rule 62-520.465(1), F.A.C. An "existing installation" is defined in Rule 62-520.200(10), F.A.C.., as:

> [A]ny installation which had filed a complete application for a water discharge permit on or before January 1, 1983, or which submitted a groundwater monitoring plan no later than six months after the date required for that type of installation as listed in former Rule 17-4.245, F.A.C., (1983) and a plan was subsequently approved by the Department, or which was in fact an installation reasonably expected to release contaminants into the groundwater on or before July 1, 1982, and operated consistently with statutes and rules relating to groundwater discharge in effect at the time of the operation.

The CREC (including the ash storage area) is an "existing installation" as it is an installation that was reasonably expected to release contaminants into the groundwater on or before July 1, 1982, and operated consistently with statutes and rules relating to groundwater discharge in effect at the time of operation. The existing Industrial Wastewater permit (IWW Permit No. FLA016960) also recognizes the facility's existing installation status as reflected in the compliance parameters and ZOD definition contained in the permit. Also, as stated previously the ash storage area at the CREC is the subject of a comprehensive facility-wide groundwater monitoring plan that has been approved by FDEP under both the Site Certification Order for the CREC as well as the facility's FDEP-issued industrial wastewater permit.

FDEP Comment

b. The transmittal letter indicates that a site-wide groundwater monitoring plan is proposed to be submitted 120 days after the vertical expansion modification of the COC is issued. Since a groundwater monitoring plan that complies with Rule 62-701.510, F.A.C., is required for the landfill prior to approval of the vertical expansion modification, please provide the revised site-wide groundwater monitoring plan.

PEF Response

The existing IWW permit (FLA016960) governs current groundwater monitoring and specific requirements for monitoring groundwater throughout the site. Per the IWW permit, the groundwater chemical analyses are made on samples from all monitored wells identified in Section III.B.2. The frequency and selected chemical analyses are given in Section III.B.3. The groundwater monitoring program has been approved by the FDEP Industrial Wastewater Section and quarterly data are submitted to the Department as required. PEF will submit a revised groundwater monitoring plan to incorporate additional wells requested by the Department.

PEF believes a separate groundwater monitoring plan under FDEP rule Chapter 62-701 F.A.C. is unnecessary and inapplicable to the permitting construct of the CREC.

FDEP Comment

c. Engineering Report, Table 1.

This table includes a summary of monitor well construction data. Please provide a revised table that includes latitude and longitude coordinates (in degrees, minutes, seconds) for each of the listed monitoring wells.

PEF Response:

The monitoring well coordinates are summarized in a revised Table 1 (see Attachment 6) originally included in the Hydrogeological and Geotechnical Site Evaluation attached to the *Supporting Technical Information* (Golder, August 25, 2010)).).

FDEP Comment

d. Engineering Report, Table 3.

This Table includes a summary of groundwater results for the wells TW-1, TW-2, TW-3, TW-4, and TW-5. Since these sampling results are approximately 1 year old, please provide more recent sampling results. It is the Department's understanding that these wells would be sampled at the same frequency as the existing Industrial Wastewater wells (i.e., quarterly). Please provide results for Q1, Q2 and Q3 of 2010.

PEF Response:

PEF has not provided additional sampling results to the Department because there is no requirement to sample and report data from TW-1, TW-2, TW-3, TW-4, and TW-5. PEF installed these temporary groundwater monitoring wells during the time period when agency approval was sought for the ash storage area's horizontal expansion project. Sampling of these wells is anticipated to commence with inclusion of these wells under the industrial wastewater monitoring requirements contained in the modified COC.

FDEP Comment:

6. **Rule 62-701.320(7)(e), 62-701.320(7)(f) and 62-701.330(3)(c), F.A.C. Drawings.**

a. Please provide a more current topographic survey. It appears that the vertical expansion design is based on contours that are approximately 1 year old. Please provide revised design drawings as appropriate.

b. Please provide drawings that include details referenced to the plan view or cross-section sheets.

PEF Response:

The Drawings have been revised to incorporate the latest topographic information dated October 26, 2010 (see Attachment 7).

The Drawings provided in the package entitled the *Supporting Technical Information* (Golder, August 25, 2010) includes details referenced to plan views and cross-section sheets.

FDEP Comment:

7. ***Coal Combustion Product (CCP)/Solid Waste Materials Management Plan, (3-ring binder) revision 4, September 2010 (received September 23, 2010)***

a. As only CD copies of this report were provided, please provide 1 paper copy of this report for the Department's files.

b. Please provide a revised CCP Management Manual that includes the revised groundwater monitoring plan discussed above. It also appears that the groundwater monitoring discussion for "product management units" other than the ash disposal area may not have been updated based on recent construction or operational changes.

PEF Response:

As requested by the FDEP-SW District's Solid Waste Section, a hard copy of the most recent version of the Crystal River Fossil Plant's Coal Combustion Product/Solid Waste Materials Management Plan (September 2010) is included in this response (see Attachment 9).

The next revision to the Coal Combustion Product/Solid Waste Materials Management Plan will include the applicable updates for product management units following the CREC's recent construction and operational changes and will make reference to the site-wide groundwater monitoring plan as the appropriate regulatory vehicle for monitoring groundwater in the vicinity of applicable CCP management units.

Also see response to FDEP response 5.b concerning a groundwater monitoring plan.

The following comment is for information only and does not require a response.
Please be advised that based on the descriptions and historic practices at the facility, some of the materials disposed in the landfill (e.g., non-CCPs and miscellaneous solid wastes) are not considered to be industrial by-products and may require management within lined or contained areas.

PEF Response:

PEF acknowledges the Department's comment.

B. INDUSTRIAL WASTEWATER PROGRAM

FDEP Comment:

Incorporation of the CREC Industrial Wastewater Permit (FLA016960)

1. The facility needs to be consistent with the naming of the IWW percolation pond for Unit 4 and 5 and the IWW percolation pond system for Units 1, 2 and 3.

PEF Response:
PEF acknowledges the Department's comment.

FDEP Comment:

2. Please propose two Groundwater Compliance Monitoring wells for the Northern boundary of the property.

PEF Response:
Attachment 10 contains proposed locations for the suggested wells on the northern boundary of the property. In addition, it is our understanding that the Department believes the existing background well may be influenced by the mariculture center ponds and may not be indicative of true background conditions. This attachment also contains a proposed location for a relocated background monitoring well.

FDEP Comment:

Increase permitted flow to the South Percolation Pond System

3. Page three of the Supplemental Hydrogeologic Investigation and Ground Water Modeling evaluation for the IWW pond area, section 3.2, indicates that Pond 4 has an approximate surface area of 7.2 acres. However, page eleven, section 7.1, indicates that Pond 4 has a 7.5 acre surface area. Please clarify and make corrections accordingly.

PEF Response:
Please refer to Attachment 11 from Geosyntec Consultants for a response.

FDEP Comment:

4. As stated above, comments regarding Ground Water will be provided upon receipt of comments from the Ground Water Section following completion of their review.

PEF Response:
Comments to the Ground Water Section's requests are provided in Attachment 11 from Geosyntec Consultants.

FDEP Comment:

North Plant Area Stormwater Modeling Report

5. According to the stormwater model summary, basins 1B-i, 1B-ii, and 1B-iii were excluded from the model as they discharge to coal pile runoff ponds, constructed at the eastern end of the coal pile. However, PEF requested that the discharge from the new coal pile runoff treatment ponds has an alternative flow path to the existing coal pile runoff treatment (outfall I-CHO). Therefore, the facility needs to revise the Stormwater model to incorporate the basins 1B-i, 1B-ii, and 1B-iii into the calculation.

PEF Response:
The existing coal pile storm water runoff system was designed and constructed to retain the entire coal yard area runoff from a 10-year, 24 hour rainfall event (approximately 8.34 inches) with disposal by means of evaporation and percolation. If a storm in excess of these design criteria is encountered, runoff is designed to overflow via the weir at NPDES Outfall I-CH0 as outlined in NPDES Permit FL0036366 and the TSS limit is waived. If the system discharges at Outfall I-CH0 during runoff from less than a 10-year, 24-hour event, the TSS limit applies. These requirements are based on the Steam Electric Effluent Guidelines found in 40 CFR Part 423. Regardless, flow monitoring and sampling requirements apply in the event of all discharges from I-CH0. Per our responses below under the ERP section, the North Plant Stormwater Modeling Study was meant for informational purposes only, not due to any regulatory requirement regarding these types of collection and treatment systems that are already covered under the NPDES permit. Nevertheless, the North Plant Stormwater Modeling Report has been revised to include the north coal yard basins 1B-i, 1B-ii, and 1B-iii. These basins were modeled as one basin 1b-i-iii as they discharge into the hydraulically connected coal pile runoff ponds. The coal pile runoff ponds are designed to detain a 25-year, 24-hour storm event while runoff is pumped to the coal pile runoff treatment facility. The treatment facility discharges to two settling ponds for sediment retention before being pumped to the stormwater canal north of the coal yard (basin 1B-iv) existing coal pile runoff system) The revised North Plant Stormwater Modeling Report in included in Attachment 8.

FDEP Comment:

Incorporation of the Crystal River North Domestic Wastewater Permit

6. The Domestic Wastewater Section has no objections or comments regarding incorporation of the Domestic Wastewater plant into the Conditions of Certification.

PEF Response:
PEF acknowledges the Department's comment.

FDEP Comment:

Additional comments

7. Under the NPDES permit FL0036366 renewal, the facility requested that discharge from the new coal pile runoff treatment ponds has an alternative flow path to the existing coal *pile runoff treatment system. The facility needs to provide calculation to the CR 4 & 5 IWW Percolation Pond (CRN IWW Percolation Pond) and demonstrate this pond has the capacity to handle the discharge from the new coal pile runoff treatments ponds.*

PEF Response:

Demonstrations that the CR Units 4&5 percolation pond can handle additional flows from the coal pile runoff treatment system have been provided to the Department on two previous occasions. The first during the initial COC modification relative to construction of the new coal pile runoff treatment system and again as part of an RAI response to the Units 4&5 NPDES permit renewal. COC Section C.I.D.1 recognizes the discharge from this source since this has triggered quarterly percolation pond monitoring requirement during the first quarter of 2010.

Nevertheless, another summary is presented here. Results of the WorleyParsons percolation pond assessment study (see Attachment 12) demonstrates that the percolation pond can handle the flows of all existing wastewater sources, including discharge from the new coal pile runoff treatment system..On page 5 of 7 of the report, the following calculation is presented comparing the existing percolation pond infiltration rate to the annual daily average hydraulic loading rate (highlights added for clarification):

$$Q=0.7 \text{ feet/day} \times 197{,}458 \text{ ft}^2 = 138{,}221 \text{ ft}^3\text{/day} = 1{,}033{,}962 \text{ gpd} > 727{,}597 \text{ gpd}$$

What this formula is saying is that the pond infiltration rate of 1,033,962 gpd **is <u>greater</u>** <u>than</u> the annual average expected IWW inflow flow rate of 727,597 gpd, which includes flow from the coal pile runoff treatment system.

Additionally, the report goes on to investigate the maximum hydraulic loading rate as allowed by rule to the average annual inflow flow rate :

$$197{,}458 \text{ ft}^2 \times 5.6 \text{ GPD/ ft}^2 = 1{,}105{,}765 \text{ gpd} > 727{,}597 \text{ gpd.}$$

The conclusion on page 5 of 7 is that both the predicted infiltration rate of 1,033,962 gpd and the maximum allowable annual average hydraulic loading rate of 1,105,765 gpd are both greater than the average annual predicted IWW inflow flow rate of 727,597 gpd.

In other words, the predicted inflow industrial wastewater flow rate, including flows from the new coal pile runoff treatment system, is expected to be well below the actual percolation rate and the allowable percolation rate of the pond.

FDEP Comment:

8. Please provide an updated Industrial Wastewater Operation & Maintenance Plan that incorporates the protocol for the maintenance of the Crystal River Units 4 & 5 Industrial Wastewater percolation pond including dewatering, cleaning and disposal of accumulated solids.

PEF Response:

In previous discussion with the Department, it is our understanding that the development of an Industrial Wastewater Operations & Maintenance Plan (IWW O&M Plan) will be a post certification submittal. PEF will submit a draft IWW O&M plan for Department review and approval no later than 120 days following COC modification. The plan will incorporate protocols for future cleaning of the Units 4&5 the percolation pond.

C. <u>ENVIRONMENTAL RESOURCE PERMITTING</u>

The stormwater management system was constructed in the 1970s before Chapter 373, Florida Statutes (F.S.), was adopted and, applying existing rules, should be considered an exempt treatment or disposal system under Rule 40D-

4.051(11)(a), Florida Administrative Code (F.A.C.).[1] The system is permitted under NPDES Permit FL0036366 consistent with the Department's authority under § 403.0885, F.S.

As noted PEF's request for modification dated September 9, 2010, and as confirmed by the modeling performed by Golder Associates in support of PEF's request, the existing SWMS **is** capable of collecting and treating all stormwater from the area associated with Crystal River North Units 4 and 5 (CRN) and could do so even if the entire area were impervious. Sufficient stormwater capacity currently exists to accommodate runoff generated by any project within the CRN area resulting in additional impervious surface.

That being the case, PEF requested the Department to treat any authorization for the installation of impervious surfaces as post-certification submittals limited to a notice-of-intent (NOI), description of the installation, identification of the appropriate drainage basin, an aerial photograph showing the location of the installation and certification that wetlands will not be impacted. PEF suggested that it be allowed to commence installation of the new impervious surface 30 days after the Department receives the NOI unless the Department objects, determines that a formal modification of the Conditions of Certification is needed or issues an RAI.

The RAI provided by the Department on October 11, 2011, appears to assume that the CRN SWMS is a stormwater system constructed under the authority of Chapter 373, Part IV, F.S., rather than a permitted treatment system under § 403.0885, F.S. The questions in the RAI seem to assume PEF is applying for an Environmental Resource Permit (ERP) or requesting that its treatment system be treated as permitted under the ERP program. PEF complied with requests to provide ERP-type modeling in the past, because similar engineering principals are applicable whether the calculations are related to an exempt stormwater system or a system regulated under Chapter 373, F.S.

Consequently, PEF does not believe that the Department's RAI questions addressing Environmental Resource Permitting are applicable to the CRN SWMS. The system is an exempt system regulated under the NPDES program and not ERP program. However, to the extent any additional technical information is necessary to allow the Department to fully evaluate calculations provided by Golder Associates, PEF will be pleased to provide any such information.

FDEP Comment

Stormwater

It is recommended that PEF contact the DEP SWD ERP section prior to the submittal of an RAI response to schedule a meeting or a teleconference to discuss the response and any possible concerns with the proposed project. If you have any questions please contact Douglas Hyman at (813) 632-7600 ext. 393.

PEF Response:

PEF has attempted to meet with the FDEP SWD ERP section as requested; however, the SWMS is regulated under the NPDES permitting program pursuant to § 403.0885, F.S. and it is not an ERP system regulated under Chapter 373, F.S. While certain of the underlying calculations, assumptions and concepts are equally applicable regardless of whether the SWMS is an ERP system or an exempt NPDES treatment system, those calculations do not determine ERP jurisdiction or convert an exempt system to a jurisdictional system.

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[1] As incorporated by reference by DEP, and as appearing in the version of the rule maintained by the Department, the exemption is referenced under Rule 40D-4.051(11)(a), F.A.C.; the current version of the rule as adopted by the Southwest Florida Water Management District, the applicable exemption appears at Rule 40D-4.051(12)(a)1, F.A.C. It appears that an earlier version of the rule remains posted on the Department's website.

FDEP Comment:

1. Pursuant to Basis of Review (BOR) Chapter 1.2.1,F.A.C., please provide all information requested in Sections A, C and E of the Application for an Environmental Resource Permit for the Crystal River Energy Complex (CREF) north plant area stormwater management system (SWMS), except as otherwise noted below.

The information required in Section E, Parts I, II, III, Items P, T, U, Part IV and Part V, Items A 1, 3, 4, 8, 10, 12, is not required at this time.

PEF Response:

PEF does not believe that the ERP application is applicable and requests clarification from the Department as to the basis for this request.

FDEP Comment:

2. Please determine how the initial stages in the ponds for ICPR routing purposes were determined. For planning purposes, the procedure of BOR Section 5.2 is normally followed.

- *Please provide the littoral zone areas, and as a percentage of area at normal water level.*
- *Please provide all water quality calculations including water quality volumes discharged from the control structures at hours 36 and 60.*
- *Please clarify how the tail water elevation in time-stage boundary node D-OCO was determined.*
- *Please clarify whether all elements of the modeled SWMS are actually in the field and operational.*
- *Please determine whether the entire north plant SWMS is in compliance with Chapter, 373, F.S., complete the attached Inspection Certification Form #62-3434.343.900 (6) FAC and provide a copy with the RAI response.*

PEF Response:

PEF does not believe that the Department's ERP criteria are applicable and requests clarification from the Department as to the basis for this request. To the extent the Department's questions seek additional information as to information provided by Golder Associates, that information will be provided (see Attachment 8), however, PEF did not assume or imply that the CRN SWMS was to be considered an ERP stormwater system for regulatory purposes.

The stormwater calculation included in the North Plant Stormwater Modeling Report incorporated additional impervious surface into the existing FDEP approved Jacobs Engineering stormwater model. The input parameters for the ICPR routing were selected from the Jacob's model except as to account for impervious surfaces, site changes, and recent survey data. The purpose of the stormwater calculations in the North Plant Stormwater Modeling Report was to demonstrate that the existing approved stormwater management system is capable of containing the runoff from a 10-year, 24-hour storm event and can safely manage a25-year, 24-hour storm event.

FDEP Comment:

3. Please submit a list of north plant project areas and indicate the current phase of construction (i.e. constructed, under construction, construction not commenced). If *available, a map of the north plant area indicating the locations of all projects would be preferable.*

PEF Response:

An aerial of the North Plant Site depicting the recent projects either under construction, complete, or proposed is enclosed. See Attachment 13

FDEP Comment:

4. Please provide a comprehensive O & M Plan for the entire north plant SWMS.

PEF Response:

As noted in PEF's correspondence of September 9, 2010, PEF proposes the development and submission of an Industrial Wastewater Operations & Maintenance Plan that would include a description of the IWW system site-wide including IWW treatment systems, monitoring wells and percolation ponds. PEF would appreciate clarification as to the Department's basis for this request in light of PEF's proposal to develop the IWW O & E Plan.

FDEP Comment:

5. Please clarify whether the 95.5-acre basin 1A is the same basin in which the IW ash pile vertical expansion and closure pond will be located. If affirmative, then is it true that the ERP calculations pertain to existing conditions in which the closure pond has not been constructed?

PEF Response:

Basin 1A incorporates the ash storage area and associated pond system. The calculations pertaining to Basin 1A presented in the North Plant Stormwater Modeling Report are consistent with the stormwater configuration presented in the Horizontal Reconfiguration package dated March 18, 2010 and the Vertical Expansion package dated August 25, 2010. The calculations provided were intended to demonstrate that the existing exempt SWMS can accommodate all runoff even if the entire subject area were impervious. The calculations were not intended to imply or concede that the existing system was now an ERP system regulated under Chapter 373, F.S.

FDEP Comment:

6. Pursuant to Chapter 62-343.900(1) Section E, V, B., please provide reasonable assurance that there are no sinkholes or karst anomalies in any element of the SWMS.
Please determine whether any portion of the north plant area SWMS is in a sensitive karst area.

PEF Response:

The SWMS is not a stormwater system regulated by Chapter 373, Part IV, F.S., or rules promulgated thereunder, and has been in place for over 30 years. PEF suggests that a karst analysis 30 years after the fact provides no new information and at considerable cost. PEF believes this RAI request is beyond the scope of the RAI and PEF objects to this request. Please reference Attachment 5 and the response to FDEP Comment No. 2.

FDEP Comment:

7. Pursuant to BOR Section 3.2.3.1, please provide reasonable assurance that no element of the SWMS receives contact water runoff.

PEF Response:

The SWMS system is intended to collect and treat contact stormwater runoff as described in NPDES Industrial Wastewater Permit FL0036366. The SWMS is not a stormwater system as anticipated by Chapter 373, Part IV, F.S. or rules promulgated thereunder.

Stormwater runoff management for the product management areas, including the ash storage area; coal pile runoff and air pollution control equipment areas; and the main Units 4 & 5 plant areas, collect in systems designed and constructed to retain runoff from a 10 year, 24 hour rainfall event (approximately 8.34 inches) with disposal by means of evaporation or percolation. If a storm in excess of these design criteria is encountered, runoff is designed to overflow via the weirs at outfall I-C40 (ash area runoff), I-CH0 (coal pile runoff) or D-0C0 (runoff collection system discharge) pursuant to NPDES Permit FL0036366. Flow monitoring and sampling requirements apply in the event of a discharge from any of these outfalls.

FDEP Comment:

8. Below are staff comments and/or recommendations. Your project will not be held incomplete if you do not respond to the following:

- *The introduction to the SWMS Modeling Report states that the north plant SWMS modeling effort will allow the applicant to bank the maximum modeled acreage of impervious surface with subsequent future "debits" against the bank as new areas are developed within the basins, a principal against which the Department has no objections. For planning purposes, the submittal of modification requests to the COC will normally be required in advance of the development or placement of new infrastructure.*

- *Pursuant to Rule 62-17.293(1)(c)2., a modification fee shall be based on the number of agencies whose review is required in order to modify the Conditions of Certification (COC). The number of agencies whose review is required shall be determined by the Department based on the changes proposed to the COC. The fee for this modification may be adjusted to more appropriately reflect agency involvement in changes to the COC for this project.*

PEF Response:

PEF engaged Golder Associates to prepare the modeling, to demonstrate the ability of the existing exempt NPDES SWMS to accommodate the entire subject area even if impervious, in support of its request to simplify the process for approval of projects resulting in additional impervious surface. As noted above, PEF requested that the projects be reviewed as post-certification submittals limited to a notice-of-intent (NOI), description of the installation, identification of the appropriate drainage basin, an aerial photograph showing the location of the installation and certification that wetlands will not be impacted. PEF suggested that it be allowed to commence installation of the new impervious surface 30 days after the Department receives the NOI *unless* the Department objects, determines that a formal modification of the Conditions of Certification is needed or issues an RAI. PEF reiterates this request and would appreciate the Department's careful consideration of this process.

PEF appreciates the Department's timely attention to this matter and believes this response to the aforementioned RAI addresses the Department's request for information contained in its October 11, 2010 correspondence. PEF would appreciate a modification to the Site Certification Application PA 77-09 to include all of the information that was contained in PEF's petition dated September 10, 2010.

If you have any questions regarding this submittal, please contact Mr. Bob Stafford at (727) 820-5538 or me at (727) 820-5588.

Sincerely,

Michael Shrader
Progress Energy Florida, Inc.
Lead Environmental Specialist
Environmental Services and Strategy

Enclosures:

Attachment 1	Leachate Generation Calculation (HELP Model Run)
Attachment 2	Slope Stability Calculation References
Attachment 3	Cover Veneer Stability Calculation
Attachment 4	Compaction Grouting Program
Attachment 5	Karst References
Attachment 6	Table 1: Summary of Monitoring Well Construction and Groundwater Elevation Data
Attachment 7	Revised Drawings (incorporating October 26, 2010 topographic survey)
Attachment 8	North Plant Area Stormwater Modeling Report (Revised)
Attachment 9	Coal Combustion Product (CCP)/ Solid Waste Materials Management Plan
Attachment 10	Proposed Well Locations
Attachment 11	Supplemental Hydrogeological Investigation and Groundwater Modeling Evaluation for the IWW Pond Area
Attachment 12	Worley Parsons North Percolation Pond Assessment
Attachment 13	Project Areas

c: Ms. Cindy Mulkey, FDEP Siting Office
Mr.. Danny Stubbs, FDEP SW District
Ms. Susan Pelz, FDEP SW District
Mr. Greg Nieboer, FDEP SW District
Ms. Yanisa Angulo, FDEP SW District

b: P.Q. West
K.S. McDaniel
D. Yowell (enc)
R.E. Stafford (enc)
R.A. Odom
N.P. Maltese
C. Wilkinson (enc)
E. Tuchbaum-Biro(enc)
I.J. Chesser (enc)
S.K. Parrish (enc)
J. Lynn
B.A. Coppola (enc)
R. D. Reynolds (enc)
K. Karably, Golder Associates Inc.
EHSS File: CR Common – Ash Landfill Expansion (enc)

Golder Associates

CALCULATIONS

Date:	11/18/2010	**Made by:**	S. Stafford
Project No.:	083-82671.1	**Checked by:**	N. Mohanan
Subject:	Vertical Expansion - Leachate Generation	**Reviewed by:**	K. Kacasky
Project Short Title:	PEF/CREC Ash Disposal Area Vertical Expansion/FL		

Objective:

The objectvie is to estimate the leachate generation rates within the ash disposal area before and after vertical expansion.

Method:

The HELP model is used to calculate the percolation rates through the ash.

Options Considered:

A.) Before Expansion:

Top: 80 ft thick fly ash @ 5% slope

Side-Slopes: 40 ft thick fly ash (average thickness) @ 4H:1V (25%) slope

B.) After Expansion:

Top: 120 ft thick fly ash @ 5% slope

Side-Slopes: 60 ft thick fly ash (average thickness) @ 3H:1V (33%) slope

Areas:

A.) Before Expansion (4H:1V to 80 ft):

Top Area = 15.0 acres

Side-Slope Area = 47.0 acres

B.) After Expansion (3H:1V to 120 ft):

Top Area = 6.3 acres

Side-Slope Area = 55.7 acres

HELP Model Results - Percolation Rates Through Ash Layer (per acre):

A.) Before Expansion (4H:1V to 80 ft):

Top Area = 620.163 ft^3/yr

Side-Slope Area = 525.648 ft^3/yr

B.) After Expansion (3H:1V to 120 ft):

Top Area = 620.129 ft^3/yr

Side-Slope Area = 525.414 ft^3/yr

(HELP model output attached)

Total Leachate Generation Rates:

A.) Before Expansion (4H:1V to 80 ft):

Q = 15 ac * 620.163 ft^3/yr/ac + 47.0 ac * 525.648 ft^3/yr/ac = 34,007.9 ft^3/yr

B.) After Expansion (3H:1V to 120 ft):

Q = 6.3 ac * 620.129 ft^3/yr/ac + 55.7 ac * 525.414 ft^3/yr/ac = 33,172.4 ft^3/yr

Conclusion:

Based on HELP model results; vertically expanding the ash disposal area by 40 feet and steepening the side-slopes from 4H:1V to 3H:1V is expected to resude the leachate generated by approximately 2.5%.

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**                                                                            **
**                                                                            **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE                 **
**              HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                    **
**              DEVELOPED BY ENVIRONMENTAL LABORATORY                         **
**              USAE WATERWAYS EXPERIMENT STATION                             **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY               **
**                                                                            **
**                                                                            **
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PRECIPITATION DATA FILE:      C:\HELP3\model\TAM.D4
TEMPERATURE DATA FILE:        C:\HELP3\model\TAM.D7
SOLAR RADIATION DATA FILE:    C:\HELP3\model\TAM.D13
EVAPOTRANSPIRATION DATA:      C:\HELP3\model\TAM.D11
SOIL AND DESIGN DATA FILE:    C:\HELP3\TOP4-1.D10
OUTPUT DATA FILE:             C:\HELP3\TOP4-1.OUT


TIME:  11: 6      DATE:  11/18/2010


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     TITLE:  top4-1

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
            COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                 LAYER  1
                                 --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER
                        MATERIAL TEXTURE NUMBER  30
          THICKNESS                  =    960.00   INCHES
          POROSITY                   =      0.5410 VOL/VOL
          FIELD CAPACITY             =      0.1870 VOL/VOL
          WILTING POINT              =      0.0470 VOL/VOL
          INITIAL SOIL WATER CONTENT =      0.1871 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.499999987000E-04 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
               FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                    GENERAL DESIGN AND EVAPORATIVE ZONE DATA
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NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 250. FEET.

SCS RUNOFF CURVE NUMBER	=	96.90	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.000	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	179.650	INCHES
TOTAL INITIAL WATER	=	179.650	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM TAMPA FLORIDA

STATION LATITUDE	=	27.58	DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	78.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	76.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
2.17	3.04	3.46	1.82	3.38	5.29
7.35	7.64	6.23	2.34	1.87	2.14

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TAMPA FLORIDA
AND STATION LATITUDE = 27.58 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	36.88	133874.422	100.00
RUNOFF	13.085	47497.008	35.48
EVAPOTRANSPIRATION	23.434	85066.797	63.54
PERC./LEAKAGE THROUGH LAYER 1	0.129786	471.124	0.35
CHANGE IN WATER STORAGE	0.231	839.537	0.63
SOIL WATER AT START OF YEAR	179.650	652130.062	
SOIL WATER AT END OF YEAR	179.881	652969.625	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.051	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	53.83	195402.875	100.00
RUNOFF	24.092	87453.906	44.76
EVAPOTRANSPIRATION	30.524	110802.664	56.70
PERC./LEAKAGE THROUGH LAYER 1	0.060124	218.248	0.11
CHANGE IN WATER STORAGE	-0.846	-3071.952	-1.57
SOIL WATER AT START OF YEAR	179.881	652969.625	
SOIL WATER AT END OF YEAR	179.035	649897.687	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.014	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.26	182443.812	100.00
RUNOFF	21.978	79780.484	43.73
EVAPOTRANSPIRATION	28.086	101951.836	55.88
PERC./LEAKAGE THROUGH LAYER 1	0.344221	1249.521	0.68
CHANGE IN WATER STORAGE	-0.148	-537.997	-0.29
SOIL WATER AT START OF YEAR	179.035	649897.687	
SOIL WATER AT END OF YEAR	178.887	649359.687	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.033	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	49.08	178160.391	100.00
RUNOFF	20.482	74351.266	41.73
EVAPOTRANSPIRATION	28.488	103411.680	58.04
PERC./LEAKAGE THROUGH LAYER 1	0.287498	1043.617	0.59
CHANGE IN WATER STORAGE	-0.178	-646.173	-0.36
SOIL WATER AT START OF YEAR	178.887	649359.687	
SOIL WATER AT END OF YEAR	178.709	648713.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.003	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	41.86	151951.781	100.00
RUNOFF	17.249	62612.918	41.21
EVAPOTRANSPIRATION	24.621	89375.547	58.82
PERC./LEAKAGE THROUGH LAYER 1	0.032590	118.303	0.08
CHANGE IN WATER STORAGE	-0.043	-154.980	-0.10
SOIL WATER AT START OF YEAR	178.709	648713.500	
SOIL WATER AT END OF YEAR	178.666	648558.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.002	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.10	3.15	2.12	1.00	2.70	4.38
	10.70	8.09	5.48	3.49	0.69	2.47
STD. DEVIATIONS	1.69	1.53	1.12	0.86	2.87	3.49
	2.10	2.42	3.64	0.80	0.66	1.37
RUNOFF						
TOTALS	0.774	1.008	0.844	0.228	1.201	1.557
	5.411	3.689	2.464	1.218	0.139	0.844
STD. DEVIATIONS	0.959	0.821	0.645	0.408	1.931	2.289
	1.229	1.668	2.756	0.595	0.207	0.642

EVAPOTRANSPIRATION

TOTALS	1.344	2.021	2.070	0.761	1.365	2.888
	4.932	3.951	3.618	2.274	0.894	0.913
STD. DEVIATIONS	0.348	0.650	0.961	0.447	0.859	1.571
	0.907	0.985	0.627	0.703	0.598	0.236

PERCOLATION/LEAKAGE THROUGH LAYER 1

TOTALS	0.0150	0.0120	0.0186	0.0198	0.0147	0.0161
	0.0187	0.0149	0.0108	0.0110	0.0086	0.0106
STD. DEVIATIONS	0.0157	0.0131	0.0247	0.0263	0.0200	0.0135
	0.0197	0.0172	0.0119	0.0139	0.0131	0.0134

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	46.38 (6.865)	168366.7	100.00
RUNOFF	19.377 (4.3111)	70339.12	41.777
EVAPOTRANSPIRATION	27.031 (2.9232)	98121.70	58.279
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.17084 (0.13850)	620.163	0.36834
CHANGE IN WATER STORAGE	-0.197 (0.3973)	-714.31	-0.424

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	4.11	14919.301
RUNOFF	3.404	12356.1348
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.002392	8.68206
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.2882
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0470

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                FINAL WATER STORAGE AT END OF YEAR     5
      ----------------------------------------------------------------------
                 LAYER         (INCHES)        (VOL/VOL)
                 -----         --------        ---------
                   1           178.6663         0.1861

                SNOW WATER        0.000

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY                     **
**                  USAE WATERWAYS EXPERIMENT STATION                       **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\HELP3\model\TAM.D4
TEMPERATURE DATA FILE:        C:\HELP3\model\TAM.D7
SOLAR RADIATION DATA FILE:    C:\HELP3\model\TAM.D13
EVAPOTRANSPIRATION DATA:      C:\HELP3\model\TAM.D11
SOIL AND DESIGN DATA FILE:    C:\HELP3\SIDE4-1.D10
OUTPUT DATA FILE:             C:\HELP3\SIDE4-1.OUT


TIME:  11: 7      DATE:  11/18/2010


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     TITLE:  side4-1

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
                                  --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER  30
          THICKNESS                    =    480.00   INCHES
          POROSITY                     =      0.5410 VOL/VOL
          FIELD CAPACITY               =      0.1870 VOL/VOL
          WILTING POINT                =      0.0470 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.1872 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.499999987000E-04 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
                 FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                     GENERAL DESIGN AND EVAPORATIVE ZONE DATA
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SIDE4-1.OUT

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 25.% AND A SLOPE LENGTH OF 320. FEET.

SCS RUNOFF CURVE NUMBER	=	97.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.959	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	89.850	INCHES
TOTAL INITIAL WATER	=	89.850	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM TAMPA FLORIDA

STATION LATITUDE	=	27.58	DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	78.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	76.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
2.17	3.04	3.46	1.82	3.38	5.29
7.35	7.64	6.23	2.34	1.87	2.14

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

11/29

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TAMPA FLORIDA
AND STATION LATITUDE = 27.58 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	36.88	133874.422	100.00
RUNOFF	13.378	48563.316	36.28
EVAPOTRANSPIRATION	23.194	84194.937	62.89
PERC./LEAKAGE THROUGH LAYER 1	0.088692	321.953	0.24
CHANGE IN WATER STORAGE	0.219	794.229	0.59
SOIL WATER AT START OF YEAR	89.850	326154.687	
SOIL WATER AT END OF YEAR	90.069	326948.906	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.016	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	53.83	195402.875	100.00
RUNOFF	24.540	89080.086	45.59
EVAPOTRANSPIRATION	30.120	109337.352	55.95
PERC./LEAKAGE THROUGH LAYER 1	0.073306	266.102	0.14
CHANGE IN WATER STORAGE	-0.904	-3280.687	-1.68
SOIL WATER AT START OF YEAR	90.069	326948.906	
SOIL WATER AT END OF YEAR	89.165	323668.219	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.030	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.26	182443.812	100.00
RUNOFF	22.368	81197.484	44.51
EVAPOTRANSPIRATION	27.675	100461.539	55.06
PERC./LEAKAGE THROUGH LAYER 1	0.392333	1424.170	0.78
CHANGE IN WATER STORAGE	-0.176	-639.388	-0.35
SOIL WATER AT START OF YEAR	89.165	323668.219	
SOIL WATER AT END OF YEAR	88.989	323028.844	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.004	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	49.08	178160.391	100.00
RUNOFF	20.860	75721.023	42.50
EVAPOTRANSPIRATION	28.151	102187.234	57.36
PERC./LEAKAGE THROUGH LAYER 1	0.128475	466.362	0.26
CHANGE IN WATER STORAGE	-0.059	-214.219	-0.12
SOIL WATER AT START OF YEAR	88.989	323028.844	
SOIL WATER AT END OF YEAR	88.930	322814.625	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.011	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	41.86	151951.781	100.00
RUNOFF	17.580	63815.738	42.00
EVAPOTRANSPIRATION	24.271	88105.125	57.98
PERC./LEAKAGE THROUGH LAYER 1	0.041227	149.653	0.10
CHANGE IN WATER STORAGE	-0.033	-118.727	-0.08
SOIL WATER AT START OF YEAR	88.930	322814.625	
SOIL WATER AT END OF YEAR	88.897	322695.906	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.005	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.10	3.15	2.12	1.00	2.70	4.38
	10.70	8.09	5.48	3.49	0.69	2.47
STD. DEVIATIONS	1.69	1.53	1.12	0.86	2.87	3.49
	2.10	2.42	3.64	0.80	0.66	1.37
RUNOFF						
TOTALS	0.791	1.035	0.862	0.234	1.224	1.589
	5.496	3.752	2.503	1.249	0.144	0.866
STD. DEVIATIONS	0.977	0.835	0.655	0.415	1.952	2.311
	1.247	1.677	2.774	0.602	0.212	0.652

EVAPOTRANSPIRATION

TOTALS	1.338	2.017	2.027	0.754	1.337	2.865
	4.852	3.906	3.570	2.234	0.881	0.901
STD. DEVIATIONS	0.334	0.655	0.926	0.441	0.835	1.548
	0.899	0.993	0.627	0.684	0.594	0.250

PERCOLATION/LEAKAGE THROUGH LAYER 1

TOTALS	0.0169	0.0151	0.0127	0.0135	0.0107	0.0075
	0.0089	0.0112	0.0091	0.0102	0.0116	0.0174
STD. DEVIATIONS	0.0187	0.0157	0.0147	0.0209	0.0132	0.0082
	0.0117	0.0097	0.0129	0.0122	0.0169	0.0257

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	46.38 (6.865)	168366.7	100.00
RUNOFF	19.745 (4.3686)	71675.53	42.571
EVAPOTRANSPIRATION	26.682 (2.8698)	96857.23	57.528
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.14481 (0.14189)	525.648	0.31220
CHANGE IN WATER STORAGE	-0.191 (0.4239)	-691.76	-0.411

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	4.11	14919.301
RUNOFF	3.422	12421.8945
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.002337	8.48314
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.2861
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0470

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	88.8969	0.1852

SNOW WATER 0.000

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
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PRECIPITATION DATA FILE:     C:\HELP3\model\TAM.D4
TEMPERATURE DATA FILE:       C:\HELP3\model\TAM.D7
SOLAR RADIATION DATA FILE:   C:\HELP3\model\TAM.D13
EVAPOTRANSPIRATION DATA:     C:\HELP3\model\TAM.D11
SOIL AND DESIGN DATA FILE:   C:\HELP3\top3-1.D10
OUTPUT DATA FILE:            C:\HELP3\top3-1.OUT


TIME:  11: 7      DATE:  11/18/2010


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     TITLE:  top3-1

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                  LAYER  1
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                    TYPE 1 - VERTICAL PERCOLATION LAYER
                        MATERIAL TEXTURE NUMBER  30
          THICKNESS                  =  1440.00   INCHES
          POROSITY                   =     0.5410 VOL/VOL
          FIELD CAPACITY             =     0.1870 VOL/VOL
          WILTING POINT              =     0.0470 VOL/VOL
          INITIAL SOIL WATER CONTENT =     0.1871 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.499999987000E-04 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
                FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                   GENERAL DESIGN AND EVAPORATIVE ZONE DATA
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NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 250. FEET.

SCS RUNOFF CURVE NUMBER	=	96.90	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.000	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	269.410	INCHES
TOTAL INITIAL WATER	=	269.410	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM TAMPA FLORIDA

STATION LATITUDE	=	27.58	DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	78.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	76.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
2.17	3.04	3.46	1.82	3.38	5.29
7.35	7.64	6.23	2.34	1.87	2.14

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR TAMPA FLORIDA
AND STATION LATITUDE = 27.58 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	36.88	133874.422	100.00
RUNOFF	13.085	47497.008	35.48
EVAPOTRANSPIRATION	23.434	85066.797	63.54
PERC./LEAKAGE THROUGH LAYER 1	0.129740	470.958	0.35
CHANGE IN WATER STORAGE	0.231	839.482	0.63
SOIL WATER AT START OF YEAR	269.410	977958.750	
SOIL WATER AT END OF YEAR	269.641	978798.250	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.170	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	53.83	195402.875	100.00
RUNOFF	24.092	87453.906	44.76
EVAPOTRANSPIRATION	30.524	110802.664	56.70
PERC./LEAKAGE THROUGH LAYER 1	0.059202	214.903	0.11
CHANGE IN WATER STORAGE	-0.845	-3068.462	-1.57
SOIL WATER AT START OF YEAR	269.641	978798.250	
SOIL WATER AT END OF YEAR	268.796	975729.750	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.130	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.26	182443.812	100.00
RUNOFF	21.978	79780.484	43.73
EVAPOTRANSPIRATION	28.086	101951.836	55.88
PERC./LEAKAGE THROUGH LAYER 1	0.343113	1245.500	0.68
CHANGE IN WATER STORAGE	-0.147	-534.065	-0.29
SOIL WATER AT START OF YEAR	268.796	975729.750	
SOIL WATER AT END OF YEAR	268.649	975195.687	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.056	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	49.08	178160.391	100.00
RUNOFF	20.482	74351.266	41.73
EVAPOTRANSPIRATION	28.488	103411.680	58.04
PERC./LEAKAGE THROUGH LAYER 1	0.289527	1050.984	0.59
CHANGE IN WATER STORAGE	-0.180	-653.484	-0.37
SOIL WATER AT START OF YEAR	268.649	975195.687	
SOIL WATER AT END OF YEAR	268.469	974542.250	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	-0.054	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	41.86	151951.781	100.00
RUNOFF	17.249	62612.918	41.21
EVAPOTRANSPIRATION	24.621	89375.547	58.82
PERC./LEAKAGE THROUGH LAYER 1	0.032589	118.300	0.08
CHANGE IN WATER STORAGE	-0.043	-154.980	-0.10
SOIL WATER AT START OF YEAR	268.469	974542.250	
SOIL WATER AT END OF YEAR	268.426	974387.250	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.005	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.10	3.15	2.12	1.00	2.70	4.38
	10.70	8.09	5.48	3.49	0.69	2.47
STD. DEVIATIONS	1.69	1.53	1.12	0.86	2.87	3.49
	2.10	2.42	3.64	0.80	0.66	1.37
RUNOFF						
TOTALS	0.774	1.008	0.844	0.228	1.201	1.557
	5.411	3.689	2.464	1.218	0.139	0.844
STD. DEVIATIONS	0.959	0.821	0.645	0.408	1.931	2.289
	1.229	1.668	2.756	0.595	0.207	0.642

EVAPOTRANSPIRATION

TOTALS	1.344	2.021	2.070	0.761	1.365	2.888
	4.932	3.951	3.618	2.274	0.894	0.913
STD. DEVIATIONS	0.348	0.650	0.961	0.447	0.859	1.571
	0.907	0.985	0.627	0.703	0.598	0.236

PERCOLATION/LEAKAGE THROUGH LAYER 1

TOTALS	0.0150	0.0120	0.0181	0.0199	0.0148	0.0162
	0.0192	0.0149	0.0104	0.0111	0.0086	0.0107
STD. DEVIATIONS	0.0157	0.0131	0.0238	0.0266	0.0203	0.0137
	0.0195	0.0171	0.0119	0.0138	0.0130	0.0133

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES		CU. FEET	PERCENT
PRECIPITATION	46.38	(6.865)	168366.7	100.00
RUNOFF	19.377	(4.3111)	70339.12	41.777
EVAPOTRANSPIRATION	27.031	(2.9232)	98121.70	58.279
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.17083	(0.13877)	620.129	0.36832
CHANGE IN WATER STORAGE	-0.197	(0.3970)	-714.30	-0.424

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	4.11	14919.301
RUNOFF	3.404	12356.1348
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.002379	8.63545
SNOW WATER	0.00	0.0000
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.2882
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0470

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	268.4262	0.1864

SNOW WATER 0.000

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**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE              **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)               **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                  **
**                    USAE WATERWAYS EXPERIMENT STATION                    **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY            **
**                                                                          **
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PRECIPITATION DATA FILE:      C:\HELP3\model\TAM.D4
TEMPERATURE DATA FILE:        C:\HELP3\model\TAM.D7
SOLAR RADIATION DATA FILE:    C:\HELP3\model\TAM.D13
EVAPOTRANSPIRATION DATA:      C:\HELP3\model\TAM.D11
SOIL AND DESIGN DATA FILE:    C:\HELP3\SIDE3-1.D10
OUTPUT DATA FILE:             C:\HELP3\SIDE3-1.OUT


TIME:  11: 7      DATE:  11/18/2010


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     TITLE:  side3-1

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
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                        TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER  30
          THICKNESS                  =    720.00   INCHES
          POROSITY                   =      0.5410 VOL/VOL
          FIELD CAPACITY             =      0.1870 VOL/VOL
          WILTING POINT              =      0.0470 VOL/VOL
          INITIAL SOIL WATER CONTENT =      0.1871 VOL/VOL
          EFFECTIVE SAT. HYD. COND.  =  0.499999987000E-04 CM/SEC
      NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
               FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                  GENERAL DESIGN AND EVAPORATIVE ZONE DATA
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NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #30 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 33.% AND A SLOPE LENGTH OF 360. FEET.

SCS RUNOFF CURVE NUMBER	=	97.00	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.959	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.410	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.470	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	134.730	INCHES
TOTAL INITIAL WATER	=	134.730	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM TAMPA FLORIDA

STATION LATITUDE	=	27.58	DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00	
START OF GROWING SEASON (JULIAN DATE)	=	0	
END OF GROWING SEASON (JULIAN DATE)	=	367	
EVAPORATIVE ZONE DEPTH	=	10.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.60	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	74.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	72.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	78.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	76.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
2.17	3.04	3.46	1.82	3.38	5.29
7.35	7.64	6.23	2.34	1.87	2.14

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
59.80	60.80	66.20	71.60	77.10	80.90
82.20	82.20	80.90	74.50	66.70	61.30

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR TAMPA FLORIDA AND STATION LATITUDE = 27.58 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	36.88	133874.422	100.00
RUNOFF	13.378	48563.316	36.28
EVAPOTRANSPIRATION	23.194	84194.937	62.89
PERC./LEAKAGE THROUGH LAYER 1	0.088161	320.024	0.24
CHANGE IN WATER STORAGE	0.219	796.167	0.59
SOIL WATER AT START OF YEAR	134.730	489068.375	
SOIL WATER AT END OF YEAR	134.949	489864.562	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.026	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	53.83	195402.875	100.00
RUNOFF	24.540	89080.086	45.59
EVAPOTRANSPIRATION	30.120	109337.352	55.95
PERC./LEAKAGE THROUGH LAYER 1	0.072187	262.040	0.13
CHANGE IN WATER STORAGE	-0.903	-3276.616	-1.68
SOIL WATER AT START OF YEAR	134.949	489864.562	
SOIL WATER AT END OF YEAR	134.046	486587.937	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.021	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	50.26	182443.812	100.00
RUNOFF	22.368	81197.484	44.51
EVAPOTRANSPIRATION	27.675	100461.539	55.06
PERC./LEAKAGE THROUGH LAYER 1	0.387685	1407.297	0.77
CHANGE IN WATER STORAGE	-0.171	-622.466	-0.34
SOIL WATER AT START OF YEAR	134.046	486587.937	
SOIL WATER AT END OF YEAR	133.875	485965.469	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.054	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	49.08	178160.391	100.00
RUNOFF	20.860	75721.023	42.50
EVAPOTRANSPIRATION	28.151	102187.234	57.36
PERC./LEAKAGE THROUGH LAYER 1	0.134422	487.952	0.27
CHANGE IN WATER STORAGE	-0.065	-235.848	-0.13
SOIL WATER AT START OF YEAR	133.875	485965.469	
SOIL WATER AT END OF YEAR	133.810	485729.625	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00

ANNUAL WATER BUDGET BALANCE	0.0000	0.029	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	41.86	151951.781	100.00
RUNOFF	17.580	63815.738	42.00
EVAPOTRANSPIRATION	24.271	88105.125	57.98
PERC./LEAKAGE THROUGH LAYER 1	0.041255	149.754	0.10
CHANGE IN WATER STORAGE	-0.033	-118.810	-0.08
SOIL WATER AT START OF YEAR	133.810	485729.625	
SOIL WATER AT END OF YEAR	133.777	485610.812	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.023	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	2.10	3.15	2.12	1.00	2.70	4.38
	10.70	8.09	5.48	3.49	0.69	2.47
STD. DEVIATIONS	1.69	1.53	1.12	0.86	2.87	3.49
	2.10	2.42	3.64	0.80	0.66	1.37
RUNOFF						
TOTALS	0.791	1.035	0.862	0.234	1.224	1.589
	5.496	3.752	2.503	1.249	0.144	0.866
STD. DEVIATIONS	0.977	0.835	0.655	0.415	1.952	2.311
	1.247	1.677	2.774	0.602	0.212	0.652

EVAPOTRANSPIRATION

TOTALS	1.338	2.017	2.027	0.754	1.337	2.865
	4.852	3.906	3.570	2.234	0.881	0.901
STD. DEVIATIONS	0.334	0.655	0.926	0.441	0.835	1.548
	0.899	0.993	0.627	0.684	0.594	0.250

PERCOLATION/LEAKAGE THROUGH LAYER 1

TOTALS	0.0171	0.0152	0.0135	0.0133	0.0110	0.0075
	0.0088	0.0104	0.0091	0.0103	0.0108	0.0178
STD. DEVIATIONS	0.0192	0.0160	0.0154	0.0209	0.0135	0.0081
	0.0116	0.0097	0.0128	0.0121	0.0150	0.0263

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	46.38 (6.865)	168366.7	100.00
RUNOFF	19.745 (4.3686)	71675.53	42.571
EVAPOTRANSPIRATION	26.682 (2.8698)	96857.23	57.528
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.14474 (0.13991)	525.414	0.31207
CHANGE IN WATER STORAGE	-0.190 (0.4231)	-691.51	-0.411

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	4.11	14919.301
RUNOFF	3.422	12421.8945
PERCOLATION/LEAKAGE THROUGH LAYER 1	0.002342	8.50304
SNOW WATER	0.00	0.0000

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.2861
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0470

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            FINAL WATER STORAGE AT END OF YEAR    5
    ----------------------------------------------------------------------
              LAYER        (INCHES)       (VOL/VOL)
              -----        --------       ---------
                1          133.7771        0.1858

            SNOW WATER       0.000

*******************************************************************************
*******************************************************************************
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over a representative depth for the footing, as we did for the Meyerhof (1965) method. Equation (9.26) was based on the assumption that settlements predicted by the Terzaghi and Peck (1967) correlation would produce excessively conservative results (that is, excessively large settlements).

The groundwater correction factor C_{gw} is simply the ratio of the vertical effective stress at a depth of $B/2$ below the base of the footing when the water table is deep versus when it is within the zone of influence for the footing:

$$C_{gw} = \frac{\left(\sigma'_v\Big|_{z_f=\frac{B}{2}}\right)_{\text{without WT}}}{\left(\sigma'_v\Big|_{z_f=\frac{B}{2}}\right)_{\text{with WT}}} \tag{9.29}$$

The Peck and Bazaraa (1969) method should be corrected for embedment, although exactly what this correction should be was not elaborated on by the authors of the equations. Presumably, the same correction suggested by Meyerhof (1965) would be appropriate.

Another, more recent method for estimating settlements of footings in sand or gravel was proposed by Burland and Burbidge (1985):

$$\frac{w}{L_R} = 0.10 f_s f_L f_t I_c \frac{q_b - \frac{2}{3}\sigma'_{vp}\Big|_{z_f=0}}{p_A}\left(\frac{B}{L_R}\right)^{0.7} \tag{9.30}$$

where f_s = shape factor, f_L = layer thickness factor, f_t = time factor, q_b = unit load at footing base (including both structural loads and the weight of the backfill and foundation element), $\sigma'_{vp}|_{z_f=0}$ = maximum previous vertical effective stress at footing base level, z_f = depth measured from foundation base level, B = footing width; I_c = compressibility index, L_R = reference length = 1 m = 3.281 ft = 39.37 in.; p_A = reference stress = 100 kPa ≈ 1 tsf.

The use of $\sigma'_{vp}|_{z_f=0}$ in Eqs. (9.24)–(9.26) and (9.30) allows for the fact that any measurable settlement will only develop after the vertical effective stress exceeds the maximum vertical effective stress ever experienced by the soil below the foundation base.

Burland and Burbidge (1985) proposed values of the compressibility index I_c as a function of SPT blow count:

$$I_c = \frac{1.71}{\overline{N}^{1.4}} \tag{9.31}$$

where $\overline{N}$ is the average blow count over the depth of influence z_{f0} below the footing base. The authors are not explicit as to whether to normalize blow counts, but we may assume $\overline{N}$ to be an energy-corrected, standardized blow count N_{60}. The depth of influence can be calculated from

$$\frac{z_{f0}}{L_R} = \left(\frac{B}{L_R}\right)^{0.79} \tag{9.32}$$

The blow counts are corrected as follows:

1. For gravel or sandy gravel, increase measured blow counts by 25%;
2. For sands below the water table, any excess of the measured blow count over 15 blows should be halved (for example, 21 would really be $15 + 6/2 = 15 + 3 = 18$).

The shape factor is given by

$$f_s = \left(\frac{1.25 \dfrac{L}{B}}{\dfrac{L}{B} + 0.25} \right)^2 \tag{9.33}$$

The sand layer thickness factor f_L is used to account for cases in which the depth of influence z_{f0} is larger than the actual thickness H of the sand layer. This means that, if a very hard layer (typically bedrock) exists at a depth smaller than the depth of influence, the settlement should be less; this result is obtained in the method by using f_L. This factor is given as

$$f_L = \begin{cases} \dfrac{H}{z_{f0}}\left(2 - \dfrac{H}{z_{f0}}\right) & \text{if } H \le z_{f0} \\ 1 \quad \text{if } H > z_{f0} \end{cases} \tag{9.34}$$

The time factor is based on observations of increasing settlement with time by Burland and Burbidge (1985). Some like to refer to this as a creep factor, but this factor is also intended to capture the effect, particularly on tall structures (such as tall chimneys, silos, high-rise buildings), of wind loads, which is not creep in the strict definition of the term. The repeated application of wind loads can always generate some additional settlement. Over time, these can add up to a nonnegligible amount. Burland and Burbidge (1985) proposed the following expression for it:

$$f_t = \left(1 + R_3 + R_t \log \frac{t}{3}\right) \tag{9.35}$$

where t is the time in years, R_3 = ratio of settlement developing over a period of 3 yr to the immediate settlement, and R_t = ratio of settlement developing over a log cycle of time to the immediate settlement. It is not necessarily easy to estimate the values of R_3 and R_t because precise measurements of immediate versus long-term settlement are not available. A design based on $f_t = 1$ is typically acceptable, although Burland and Burbidge give values of $R_t = 0.2$ and $R_3 = 0.3$.

EXAMPLE 9-4

A 6-ft square footing (see Fig. 9-7), which is backfilled, is embedded 2.5 ft into a sand. SPT N_{60} values for the sand are given in Table 9-3. Estimate the settlement of this footing caused by a column load of 182 tons. The sand is approximately 40 ft thick and has an average unit

Bearing Capacity

The bearing-capacity problem is really one of passive earth pressure. The shearing stresses at the contact face BC of Zone I are expressed by Coulomb's equation (Eq. 3.8) in which shear stress s is replaced by the total passive resistance and the normal stress σ_n is replaced by the normal component of the passive earth pressure per unit of area of the contact face. At the failure instant, the pressure on BC is equal to the resultant of the passive resistance plus cohesion along the failure surface BC. The total passive resistance includes the weight of the mass in Zones II and III, a cohesive resistance along the failure surface AE, and a surcharge q_s from the overburden soil.

Terzaghi's Expression

A closed form for this problem in plasticity has been found only for the case of weightless soil [Prandtl (1921)[37]]. The real nature of soil with weight can be considered approximately (with an error <20% for ϕ between 30 and 40°) by Terzaghi's (1943)[34] expression for drained loading conditions for a long rectangular footing (the Buisman-Terzaghi equation):

$$q_b = cN_c + qN_q + 0.5\gamma BN_\gamma \qquad (6.10)$$

where q_b = ultimate bearing capacity = Q_b/B
B = width of smaller footing side
c = cohesion
γ = effective soil unit weight
q = overburden stress at footing base = γD_f
N_c, N_q, N_γ = the *bearing-capacity factors*, dimensionless functions of ϕ

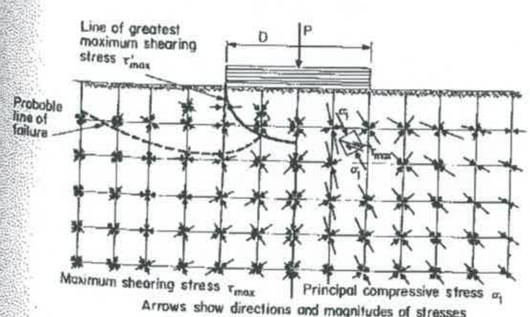

FIG. 6.29 Distribution of compressive and shearing stresses beneath a circular loaded area during general shear. [After Converse (1939)[35]; from Tschebotarioff (1973).[36] Reprinted with permission of McGraw-Hill Book Company.]

The three parts of Eq. 6.10 provide for soil cohesion, foundation depth (overburden pressure), and foundation width, with ϕ a factor in all three parts. Even a small amount of cohesion substantially increases q_b. The first term is a function of parameters ϕ and c, the second term is a function of the vertical confining effective stress ($\bar{\sigma}_v = \gamma D_f$) at the elevation of the footing base, and the third term is a function of the footing width on the average weight of the soil mass under the footing. All three terms also are a function of the assumed shape of the rupture surface and of ϕ. [For *deep foundations (see Art. 7.3.3)*, the third term is very small in comparison to the second and is neglected.]

A number of values are available for the *bearing-capacity factors*, which are exclusively a function of ϕ and the failure surface geometry. The current trend of many practitioners is to use values developed by Prandtl (1921)[37] and Reissner (1924)[38] for N_c and N_q and those developed by Caquot and Kerisel (1953)[39] for N_γ, as given in Table 6.7.

Bearing-Capacity Analysis

Factors to Consider

- Drained versus undrained conditions and, for the drained case, general versus local shear
- Foundation shape and depth and base inclination
- Loading eccentricity and inclination
- Ground-slope condition
- Groundwater-table condition

Drained Loading Conditions

Drained loading conditions occur normally where loads are applied gradually as the structure is erected and parameters c and ϕ are mobilized. *General shear* prevails for relatively strong materials, and bearing capacity is commonly found for continuous footings by Eq. 6.10. For square and circular footings Terzaghi (1943)[34] proposed

$$q_b = \frac{Q}{A} = 1.2\,cN_c + qN_q + s_\gamma BN_\gamma \qquad (6.11)$$

where the shape factor s_γ = 0.4 for square footings and 0.6 for circular footings.

TABLE 6.7
BEARING-CAPACITY FACTORS

ϕ	N_c	N_q	N_γ	N_q/N_c	$\tan\phi$
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	1.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.97	762.89	1.20	1.19

Local shear applies to weaker materials and Terzaghi accounted for soil compressibility by reducing the ultimate strength parameters c_g for ϕ_g (general shear) as follows:

- Soft to firm clays:

$$c_{local} = \tfrac{2}{3} c_g \tag{6.12}$$

- Loose sands:

$$\tan \phi_{local} = \tfrac{2}{3} \tan \phi_g \tag{6.13}$$

This results in an abrupt transition from general to local shear but is considered to be suitable for cases with overburden pressure although conservative for surface to near-surface loadings [Ismael and Vesić (1981)[33]]. See, for example, case 2 in Fig. 6.33.

Sands (c = 0)

In sands, where $c = 0$ (see for example, case 3 in Fig. 6.33), Eq. 6.10 for the gross ultimate bearing capacity becomes

$$q_b = \gamma D_f N_q + 0.5\gamma B N_\gamma \tag{6.14}$$

The *net* ultimate bearing capacity $q_b' = q_b - \gamma D_f$, or the gross capacity minus overburden pressure at footing level. In terms of q_b', Eq. 6.14 can be expressed in the form [Peck et al. (1974)[17]] for the allowable bearing value as

$$q_a = \frac{q_b'}{FS} = \left[\gamma(N_q - 1)\frac{D_f}{B} + 0.5\gamma N_\gamma\right]\frac{B}{FS} \tag{6.15}$$

where FS = safety factor. For a particular value of D_f/B and a given sand deposit, the expression within the brackets is a constant. Thus, the relation between footing width B and the net allowable soil pressure q_a for a given value of FS can be plotted as in Fig. 6.18 as a family of straight lines radiating from the origin. In Fig. 6.18, FS = 2. Each line corresponds to a different value for ϕ as determined roughly from the N value of the SPT as given in Fig. 6.30. Note that the values for N_γ given in Fig. 6.30 are lower than those given in Table 6.7.

Bearing-Capacity Equation: General Form

In recent years Eq. 6.10 has been expanded by various investigators to provide for the various factors that influence q_b as follows:

$$q_b = cN_c s_c b_c i_c g_c + qN_q s_q b_q i_q g_q + 0.5\gamma B N_\gamma s_\gamma b_\gamma i_\gamma g_\gamma \tag{6.16}$$

Z: 300
R = 76.625

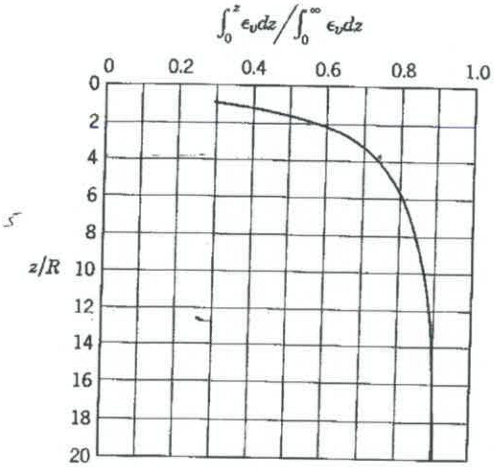

Fig. 14.21 Effect upon I_ρ of considering only a limited depth of strains.

▶ Example 14.13 method 3

Given. The tank loading and subsoil shown in Example 8.9.

$$E = 2000 \text{ kips/ft}^2$$
$$\mu = 0.45$$

Find. The settlement at the center of the tank for the condition of homogeneous, isotropic soil of infinite depth.

Solution.

$$\rho_{℄} = \Delta q_s \frac{R}{E} 2(1 - \mu^2) \qquad \text{Eq. 14.15}$$

$$\left.\begin{aligned} \Delta q_s &= 5.50 \text{ kips/ft}^2 \\ R &= \frac{D}{2} = \frac{153\frac{1}{4}\text{ ft}}{2} \end{aligned}\right\} \quad \text{given in Example 8.9}$$

$$\rho_{℄} = \frac{5.50 \text{ kips/ft}^2 \times \dfrac{153\frac{1}{4}\text{ ft}}{2} \times 2(1 - 0.45^2)}{2000 \text{ kips/ft}^2}$$
$$= \underline{\underline{0.346 \text{ ft}}} = \underline{\underline{4 \text{ in.}}}$$

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method 4

Settlement may be estimated by multiplying an average strain times the depth of the bulb of stresses. The following tabulation shows several ways in which this might be done.

Assumed Depth of Bulb	Average Strain	Settlement (in.)
$3R = 230$ ft	Use strain at depth of $3R/2$: $\epsilon_v = 0.00106$	3.0
$4R = 306$ ft	Use strain at depth of $2R$: $\epsilon_v = 0.00076$	2.8

The first method, using a bulb of depth $3R$, gives a closer estimate to the actual result of 4 in. ◀

$\rho = \frac{106}{100} \times 230 \times 12 \frac{\text{in}}{\text{ft}} =$

loaded area the horizontal strains at the surface must be tensile, and this can happen only if the horizontal stress increments are tensile. Circular tension cracks are often observed around heavy loads resting on the surface of grounds. This general pattern of horizontal strains is somewhat similar to that in a fixed-end beam carrying a concentrated load at midspan.

Equation 14.14 may also be used when the elastic body is of limited depth. However, a different value of I_ρ must be used. Figure 14.20 gives values of I_ρ for two cases of an elastic stratum of limited depth. As would be expected, decreasing the depth of the elastic body decreases the settlement. When the elastic body is thin in comparison to the dimensions of the load, points outside of the loaded area may heave instead of settling. Burmister (1956) has compiled charts and tables that are especially useful when dealing with settlements of strata of limited thickness.

Elastic Theory for Settlement under Other Uniform Loads

The settlement at the corner of a rectangular area carrying a uniform stress Δq_s may be calculated from

$$\rho = \Delta q_s \frac{B(1 - \mu^2)}{E} I_\rho \tag{14.16}$$

where

B = the width (least dimension) of the rectangle
L = the length (greatest dimension) of the rectangle
I_ρ = an influence coefficient given by Fig. 14.22

Settlements for points other than the corner of a rectangular area, and for any shape of loaded area that can be divided into rectangles, can be obtained using the method of superposition, as explained in Chapter 8 in connection with computing stresses (see Example 8.3). In particular, the settlement at the center of a square loaded area is

$$\rho = \Delta q_s \frac{B}{E} 1.12(1 - \mu^2) \tag{14.17}$$

As L/B becomes very great (i.e., for a strip footing), I_ρ gradually increases beyond all bounds. Thus a strip footing resting on an elastic body of infinite depth would experience an infinite settlement. In real problems, of course, soil strata are not infinite in depth and strip footings are not infinite in length. For a rectangular loaded area on an elastic stratum of thickness D overlying a rigid body, the approximate settlement at the corner of the area may be computed using Eq. 14.16 and

$$I_\rho = (1 - \mu^2)F_1 + (1 - \mu - 2\mu^2)F_2 \tag{14.18}$$

where the functions F_1 and F_2 may be read from Fig. 14.23. Burmister (1956) also presents useful charts for such problems.

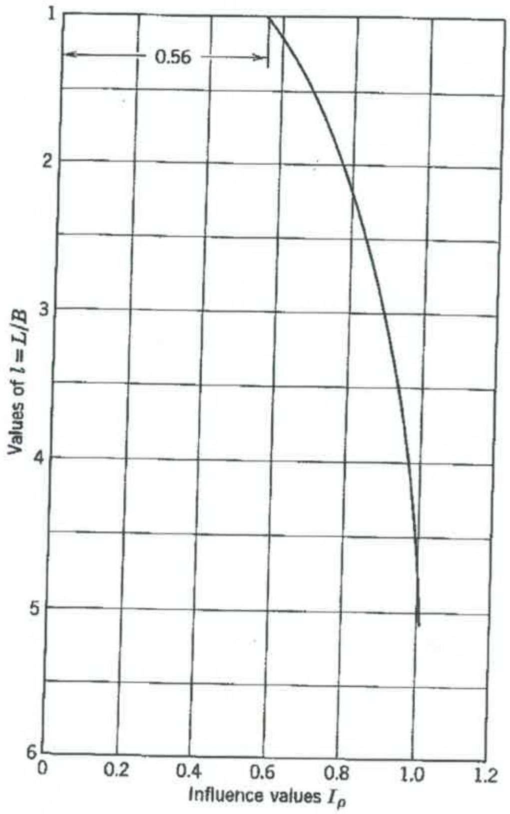

Fig. 14.22 Influence coefficient for settlement of rectangular loaded area (From Terzaghi, 1943).

Solutions have also been obtained for many other types of loading conditions, including loading by shear stresses. Scott (1963) gives a useful summary. With computer techniques, numerical values applicable to specific cases can be obtained.

Elastic Theory for Settlement under Rigid Footings

The condition of a uniformly distributed load occurs in practical problems, such as steel tanks for the storage of fluids. In many practical problems, however, the structural member (such as a footing) in contact with the soil will be quite rigid, and the settlement is more or less uniform over the area of contact between the footing and the soil. Since a uniform stress causes a dish-shaped pattern of settlement, in order to produce a uniform settlement the contact stresses must increase on the outside of the loaded area and decrease near the center line. The curves in Fig. 14.24 marked $K_r = \infty$ show the theoretical distribution of contact stress for the case of a truly rigid foundation. At the edge of the loaded area the contact stress theoretically is infinite.

A change in the distribution of stress over the contact area means a change in the relationship between load and settlement. For a circular rigid loaded area this becomes

$$\rho = \Delta q_s \frac{R}{E} \frac{\pi}{2} (1 - \mu^2) \qquad (14.19)$$

where Δq_s = average stress over the loaded area. Comparing Eq. 14.19 with Eq. 14.15, we see that the settlement of a rigid footing is 21% less than the center line settlement under a uniform load. Whitman and Richart (1967) present load-settlement relationships for rigid rectangular footings with various types of loading.

In some problems the structural member in contact with the soil cannot be considered perfectly flexible or perfectly rigid. Figure 14.24 can be used to estimate the contact stresses for intermediate conditions.

14.9 THEORETICAL PROCEDURES FOR USE WITH SOILS

As was discussed in Chapters 10 and 12, a mass of soil does not behave as an elastic, homogeneous, and isotropic material. Nonelasticity influences (*a*) the distribution of stress increments caused by these loads, and (*b*) the strains resulting from these stress increments. At present there are no theoretical procedures that consider both these difficulties, although such procedures are under development. Fortunately, experience has shown that useful predictions of settlement can be made by using the distribution of stress increments predicted by elastic theory, but employing special procedures to determine the resulting strains.

Stress Path Method

As applied to estimating the settlements, the stress path method consists of the following four steps:

1. Select one or more points within the soil under the proposed structure.
2. Estimate for each point the stress path for the loading to be imposed by the structure.
3. Perform laboratory tests which follow the estimated stress paths.
4. Use the strains measured in these tests to estimate the settlement of the proposed structure.

This same general approach, which is a powerful aid to understanding and solving deformation and stability problems, has already been used in Chapter 13.

Example 14.14 illustrates the application of this approach to the tank foundation of Example 8.9. Stress paths for selected points have already been given in Example 8.9. Figure 10.23 presents stress-strain results from triaxial tests following the stress paths for points A, B, D, and G. The vertical and horizontal strains as measured in these tests have been plotted in Example 14.14. Integration of these strains over a depth of 300

SUBJECT:	FINAL COVER SLOPE STABILITY				
Job No.	083-82671.1	Made by	SFS		
Ref. :	PEF CREC ASH STORAGE/ DISPOSAL AREA VERTICAL EXPANSION	Checked	NPM	Sheet	1 of 5
		Reviewed	KBK	Date	11/4/2010

OBJECTIVE:

1. Analyze the stability of the final cover system and determine the factor of safety during and after landfill construction .
2. Determine the stability of the landfill including seepage loading conditions

GEOMETRY:

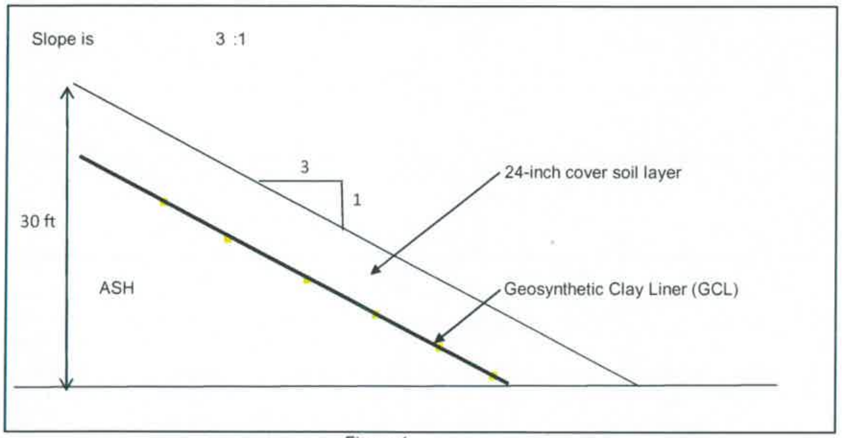

Figure-1

METHOD:

GOLDER RECOMMENDED FACTORS OF SAFETY FOR LANDFILL FINAL COVER ARE AS BELOW

Shear Strength	During Construction (Short term)	Long Term
Design	1.1	1.5, 1.1 [a]

[a] Recommended factor of safety with seepage forces included

If the calculated factors of safety exceeds the recommended factors of safety for landfill final cover, then the stability of the final cover meets the requirement.

Design Parameters:

Top Elevation of Cover: 30.00 ft (side-slope benches every 30 feet max)
Approximate Toe Elevation : 0.00 ft

Material Properties (ref. 1)

Material	c (psf)	c_a (psf)	ϕ (°)	δ (°)	γ (pcf)	Thickness (ft)	
Cover soil	0	-	30	-	100	2.00	
ash/GCL/cover soil	-	28	-	36.9	-	0.03	(interface shear results)

Where:
c = Cohension of the cover soil
c_a = Adhesion between cover soil of the active wedge and the GCL
δ = Interface friction angle between cover soil, GCL, and ash
ϕ = Friction Angle of cover soil
γ = Unit weight of the cover soil

Slope Angle = β (°) = 18.4
Slope Height = 30.00 ft (H)

Golder Associates	SUBJECT: FINAL COVER SLOPE STABILITY				
	Job No.	083-82671.1	Made by	SFS	
	Ref. :	PEF CREC ASH STORAGE/ DISPOSAL AREA VERTICAL EXPANSION	Checked	NPM	Sheet 2 of 5
			Reviewed	VBK	Date 11/4/2010

CALCULATIONS:

LONG TERM VENEER STABILITY based on Koerner/Soong Method (page 487 to 490, ref. 2)

Using the Koerner/Soong Method, the factor of safety is calculated using the following equation (Eq. 13.9, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_a - N_a \times \cos\beta)\cos\beta$

$b = -[(W_a - N_a \times \cos\beta) \times \sin\beta \tan\phi + (N_a \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan\phi) \times \sin\beta]$

$c = (N_a \times \tan\delta + C_a) \times \sin^2\beta \times \tan\phi$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

$N_a = W_a \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

Where:

W_a= Total weight of the active wedge.
N_a= Effective force normal to the failure plane of the active wedge
C_a = Adhesive force between cover soil of the active wedge and the GCL
W_p = Total weight of the passive wedge
C = Cohesive force along the failure plane of the passive wedge
γ = Unit Weight of protective cover soil
h = Thickness of cover soil
β = Slope Angle
L = Length of slope measured along the geosynthetic interface
c = Cohension of the cover soil
c_a = Adhesion between cover soil of the active wedge and the GCL
δ = Interface friction angle between cover soil and GCL
ϕ = Friction Angle of cover soil

Where:
h = Thickness of Cover (ft) = 2.00
β = Cover Slope Angle (°) = 18.4
H_{max} = Maximum height = 30.0 feet
L= 94.9 feet

Since h and L are known for **LONG-TERM Conditions,** solve for the FS:

W_a (lbs/ft) = 17,642
N_a (lbs/ft) = 16,737
C_a (lbs/ft) = 89 x c_a

W_p (lbs/ft) = 667
C (lbs/ft) = 0

$(W_a - N_a \times \cos\beta)$ = 1,764
$(C + W_p \times \tan f)$ = 385
$\cos\beta$ = 0.95
$\sin\beta$ = 0.32
$\sin\beta \times \tan\phi$ = 0.18
$\sin^2\beta \times \tan\phi$ = 0.06
$\sin\beta \times \cos\beta$ = 0.30
$\tan\phi$ = 0.58

a= 1673.7
- b= 443.8 + 0.30 x $(N_a \times \tan\delta + C_a)$
c = $(N_a \times \tan\delta + C_a)$ x 0.06

Solve for FS:

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	$(N_a \times \tan\delta + C_a)$	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
36.90	*28*	*0.8*	*2,435*	*15,001*	*-4,944*	*866*	*4318.2*	*2.8*

SHORT TERM CONDITIONS (*Dozer on the slope without acceleration)*

Veneer Stability based on Koerner/Soong Method (page 490-497, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = (W_{a+e} - N_{a+e} \times \cos\beta)\cos\beta$

$b = -[(W_{a+e} - N_{a+e} \times \cos\beta) \times \sin\beta \times \tan\phi + (N_{a+e} \times \tan\delta + C_a) \times \sin\beta \times \cos\beta + (C + W_p \times \tan\phi) \times \sin\beta]$

$c = (N_{a+e} \times \tan\delta + C_a) \times \sin^2\beta \times \tan\phi)$

$W_a = \gamma \times h^2 \times (L/h - 1/\sin\beta - \tan\beta / 2)$

W_e = Equipment Weight, see below

$W_{a+e} = W_a + W_e$

$N_{a+e} = W_{a+e} \times \cos\beta$

$C_a = c_a \times (L - h/\sin\beta)$

$W_p = (\gamma \times h^2) / \sin 2\beta$

$C = c \times h / \sin\beta$

The definitions of all the parameters are as same as those in long term FS caculation except W_e, W_{a+e}, and N_{a+e}

For SHORT-Term Conditions, look at 6 inches of soil being placed up slope with a Low Ground Pressure Dozer

$L_{short\ term}$=	94.9	ft
$h_{short\ term}$=	2.00	ft
f =	30.00	degrees
c =	0.00	psf
$\gamma_{soil\ cover}$ =	100.00	pcf

Determination of W_e (See attached dozer specifications from manufacturer, ref. 3):

Width of Dozer Track =	3.00 ft
Contact Area =	32.13 sq.ft.
Ground Pressure =	9.5 psi
Influence factor (I) =	0.95 (obtained from Figure 13.7, page 493, ref. 2)
Ground Pressure at Geosynthetics =	1304.8 psf
Length of Dozer Track =	10.7 ft

W_e= 13975 lbs/ft

W_a+W_e (lbs/ft) =	31,619		
N_{a+e} (lbs/ft) =	29,996	W_p (lbs/ft) =	667
C_a (lbs/ft) =	89 x c_a	C (lbs/ft) =	0

$(W_{a+e} - N_{a+e} \times \cos\beta)$ =	3,162
$(C + W_p \times \tan f)$ =	385
$\cos\beta$ =	0.95
$\sin\beta$ =	0.32
$\sin\beta \times \tan\phi$ =	0.18
$\sin^2\beta \times \tan\phi$ =	0.06
$\sin\beta \times \cos\beta$ =	0.30
$\tan\phi$ =	0.58

a= 2999.6

\- b= 699.0 + 0.30 x $(N_{a+e} \times \tan\delta + C_a)$

c = $(N_{a+e} \times \tan\delta + C_a)$ x 0.06

Solve for FS :

δ (°)	c_a (psf)	tan δ	C_a (lbs/ft)	$(N_{a+e} \times \tan\delta + C_a)$	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
36.90	*27.50*	*0.8*	2434.95	*24,957*	*-8,186*	*1,441*	*7,051*	*2.5*

Golder Associates	SUBJECT:	FINAL COVER SLOPE STABILITY				
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			Reviewed	KBK	Date	11/4/2010

SEEPAGE BUILT-UP CONDITIONS

Veneer Stability based on Koerner/Soong Method (page 501-508, ref. 2)

$$FS = \frac{-b \pm (b^2 - 4 \times a \times c)^{0.5}}{2 \times a}$$

Where:

$a = W_a \sin\beta \cos\beta + U_H \times (1 - \cos^2\beta)$

$b = -[Wp \times \tan\phi + W_a \times (\sin^2\beta \times \tan\phi + \cos^2\beta \times \tan\delta) - U_{AN} \times \cos\beta \times \tan\delta - U_{PN} \times \tan\phi + U_H \times \sin\beta \times \cos\beta \times (\tan\phi - \tan\delta)]$

$c = (W_a \times \cos\beta - U_{AN} + U_H \times \sin\beta) \times \sin\beta \times \tan\delta \times \tan\phi)$

$U_{AN} = \gamma_w \times h_w \times (H - 0.5 \times h_w \times \cos\beta) / \tan\beta$

$U_H = 0.5 \times \gamma_w \times h_w^2$

$U_{PN} = 0.5 \times \gamma_w \times h_w^2 / \tan\beta$

$W_a = 0.5 \times [\gamma \times (h - h_w) \times (2 \times H \times \cos\beta - h - h_w) + \gamma_{sat} \times h_w \times (2 \times H \times \cos\beta - h_w)] / (\sin\beta \times \cos\beta)$

$W_p = 0.5 \times [\gamma \times (h^2 - h_w^2) + \gamma_{sat} \times h_w^2] / (\sin\beta \times \cos\beta)$

U_H = Resultant of the pore water pressures acting on lateral side of the active wedge or passive wedge
U_{PN} = Resultant of the pore water pressures acting on bottom of the passive wedge
U_{AN} = Resultant of the pore water pressures acting on bottom of the active wedge
h = Thickness of the soil layer
h_w = Depth of seepage water in the soil layer (perpendicular to the slope)
γ_w = Unit weight of water
γ = Moisture unit weight of the soil layer
γ_{sat} = Saturated unit weight of the soil layer

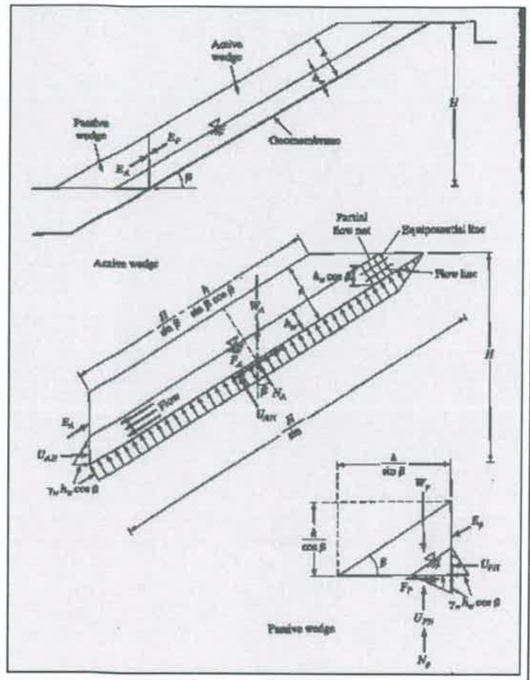


Figure -2

Other paramters are same as in the above calculations

γ_w =	62.4	lb/ft³	
h_w =	1.00	inches	(Conservative because internal drainage layer is designed for no head on synthetic cover - see Design of Internal Cover Drainage Layer Calculation).
H =	30.0	ft	
γ_{sat} =	110	lb/ft³	(Assumed)
U_{AN} =	467	lbs/ft	
U_H =	0.22	lbs/ft	
U_{PN} =	0.65	lbs/ft	
W_a =	18386	lbs/ft	
W_p =	667	lbs/ft	

Wp x tan φ =	385	sin² β x tan φ =	0.06
U_{PN} x tan φ=	0.38	sin β =	0.32
U_H x sin β x cos β =	0.0650	tan φ =	0.58
		cos² β =	0.90
		cos β =	0.95

a = 5516
- b= 1446 + 16103.88266 x tan δ
c = 3099.2 x tan δ

δ (°)	c_a (psf)	tan δ	b	c	$(b^2 - 4ac)^{0.5}$	Factor of Safety
36.9	*27.5*	*0.751*	*-13,537*	*2,327*	*11486*	*2.3*

Golder Associates	SUBJECT:	FINAL COVER SLOPE STABILITY				
	Job No.	083-82671.1	Made by	SFS		
	Ref. :	PEF CREC ASH STORAGE/ DISPOSAL AREA VERTICAL EXPANSION	Checked	NPM	Sheet	5 of 5
			Reviewed	KBK	Date	11/4/2010

CONCLUSION:

SUMMARY OF RESULTS

CASE ANALYZED	REQUIRED FACTOR OF SAFETY	ACTUAL FACTOR OF SAFETY	MEET REQUIREMENT
Long Term using Design Shear Strenght	1.5	2.8	Yes
Short Term using Design Shear Strength - with The Dozer on the Slope	1.1	2.5	Yes
Seepage Analysis	1.1	2.3	Yes

Therefore, the stablity of the final cover meets the recommended factors of safety.

REFERENCES:

1. Material Properties - Unpublished Laboratory Testind Data, Golder Associates, 2000-2009

2. Qian, X., Koerner, R. M., Gray, D. H., Geotechnical Aspects of Landfill Design and Construction, Prentice Hall, New Jersery, US, 2002.

3. Dozer Specifications from Manufacturer

4. Agru Material Manufacturer Information

REF. 2

13.4 VENEER SLOPE STABILITY ANALYSES

This section treats the standard veneer slope stability problem [as shown in Figure 13.1(a) and (b)] and then superimposes upon it a number of situations, all which tend to destabilize slopes. Included are gravitational, construction equipment, seepage and seismic forces, respectively. Each will be illustrated by a design graph and a numeric example.

13.4.1 Cover Soil (Gravitational) Forces

Figure 13.3 illustrates the common situation of a finite-length, uniformly-thick cover soil placed over a liner material at a slope angle β. It includes a passive wedge at the toe and has a tension crack on the crest. The analysis that follows is from Koerner and Soong (1998), but it is similar to Koerner and Hwu (1991). Comparable analyses are also available from Giroud and Beech (1989), McKelvey and Deutsch (1991), and others.

The symbols used in Figure 13.3 are defined a follows:

W_A = total weight of the active wedge
W_P = total weight of the passive wedge
N_A = effective force normal to the failure plane of the active wedge
N_P = effective force normal to the failure plane of the passive wedge
γ = unit weight of the cover soil
h = thickness of the cover soil
L = length of slope measured along the geomembrane
β = soil slope angle beneath the geomembrane

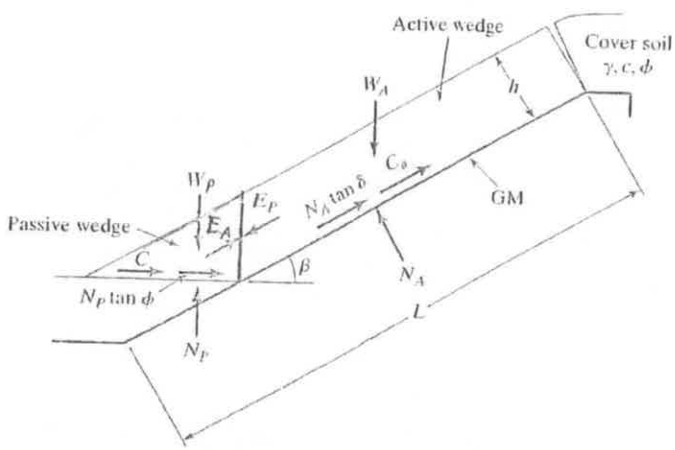


FIGURE 13.3 Limit Equilibrium Forces Involved in a Finite Length Slope Analysis for a Uniformly Thick Cover Soil

ϕ = friction angle of the cover soil
δ = interface friction angle between cover soil and geomembrane
C_a = adhesive force between cover soil of the active wedge and the geomembrane
c_a = adhesion between cover soil of the active wedge and the geomembrane
C = cohesive force along the failure plane of the passive wedge
c = cohesion of the cover soil
E_A = interwedge force acting on the active wedge from the passive wedge
E_p = interwedge force acting on the passive wedge from the active wedge
FS = factor of safety against cover soil sliding on the geomembrane.

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge, the forces acting on it are

$$W_A = \gamma \cdot h^2 \cdot (L/h - 1/\sin\beta - \tan\beta/2) \tag{13.4}$$

$$N_A = W_A \cdot \cos\beta \tag{13.5}$$

$$C_a = c_a \cdot (L - h/\sin\beta) \tag{13.6}$$

By balancing the forces in the vertical direction, the following formulation results:

$$E_A \cdot \sin\beta = W_A - N_A \cdot \cos\beta - \frac{(N_A \cdot \tan\delta + C_a)}{FS}$$

Hence, the interwedge force acting on the active wedge is

$$E_A = \frac{(FS)(W_A - N_A \cdot \cos\beta) - (N_A \cdot \tan\delta + C_a) \cdot \sin\beta}{\sin\beta \cdot (FS)} \tag{13.7}$$

The passive wedge can be considered in a similar manner:

$$W_P = \frac{\gamma \cdot h^2}{\sin 2\beta}$$

$$N_P = W_P + E_P \cdot \sin\beta$$

$$C = \frac{c \cdot h}{\sin\beta} \tag{13.8}$$

By balancing the forces in the horizontal direction, the following formulation results:

$$E_P \cdot \cos\beta = \frac{C + N_P \cdot \tan\phi}{FS}$$

Hence, the interwedge force acting on the passive wedge is

$$E_P = \frac{C + W_P \cdot \tan\phi}{\cos\beta \cdot (FS) - \sin\beta \cdot \tan\phi}$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$, which in this case, using FS-values, results in

$$a \cdot FS^2 + b \cdot FS + c = 0$$

The resulting FS-value is then obtained from the conventional solution of the quadratic equation, which gives

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.9}$$

where $a = (W_A - N_A \cdot \cos\beta) \cdot \cos\beta$

$b = -[(W_A - N_A \cdot \cos\beta) \cdot \sin\beta \cdot \tan\phi + (N_A \cdot \tan\delta + C_a) \cdot \sin\beta \cdot \cos\beta + (C + W_P \cdot \tan\phi) \cdot \sin\beta]$

$c = (N_A \cdot \tan\delta + C_a) \cdot \sin^2\beta \cdot \tan\phi$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated. Thus, a value of greater than 1.0 must be targeted as being the minimum factor of safety. How much greater than 1.0 the FS-value should be, is a design and/or regulatory issue. Recommendations for minimum allowable FS-values under different conditions are available in Koerner and Soong (1998). In order to better illustrate the implications of Equations 13.9, typical design curves for various FS-values as a function of slope angle and interface friction angle are given in Figure 13.4. Note that the curves are developed specifically for the variables stated in the legend of the figure. Example 13.1 illustrates the use of the analytic development and the

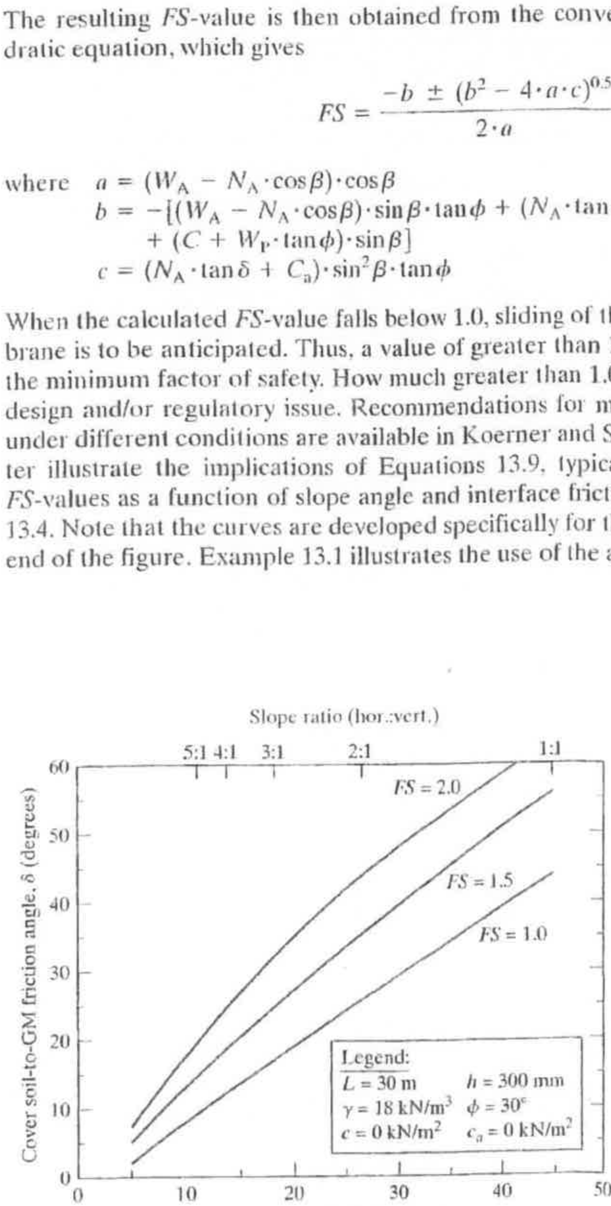


FIGURE 13.4 Design Curves for Stability of Uniform-Thickness Cohesionless Cover Soils on Linear Failure Planes for Various Global Factors of Safety

will be considered as compared.

EXAMPLE 13.1

The following are given: a 30-m slope with a uniformly thick 300-mm-deep cover soil at a unit weight of 18 kN/m^3. The soil has a friction angle of 30° and zero cohesion (i.e., it is a sand). The cover soil is placed directly on a geomembrane as shown in Figure 13.3. Direct shear testing has resulted in an interface friction angle between the cover soil and geomembrane of 22° with zero adhesion. What is the *FS*-value at a slope angle of 3(H)-to-1(V) (i.e., 18.4°)?

Solution Using Equation 13.9 to solve for the *FS*-value results in a value of 1.25, which is seen to be in agreement with the curves of Figure 13.4:

$$a = 14.7 \text{ kN/m}$$

$$b = -21.3 \text{ kN/m}$$

$$c = 3.5 \text{ kN/m}$$

Thus, $FS = 1.25$

This value can be confirmed using Figure 13.4.

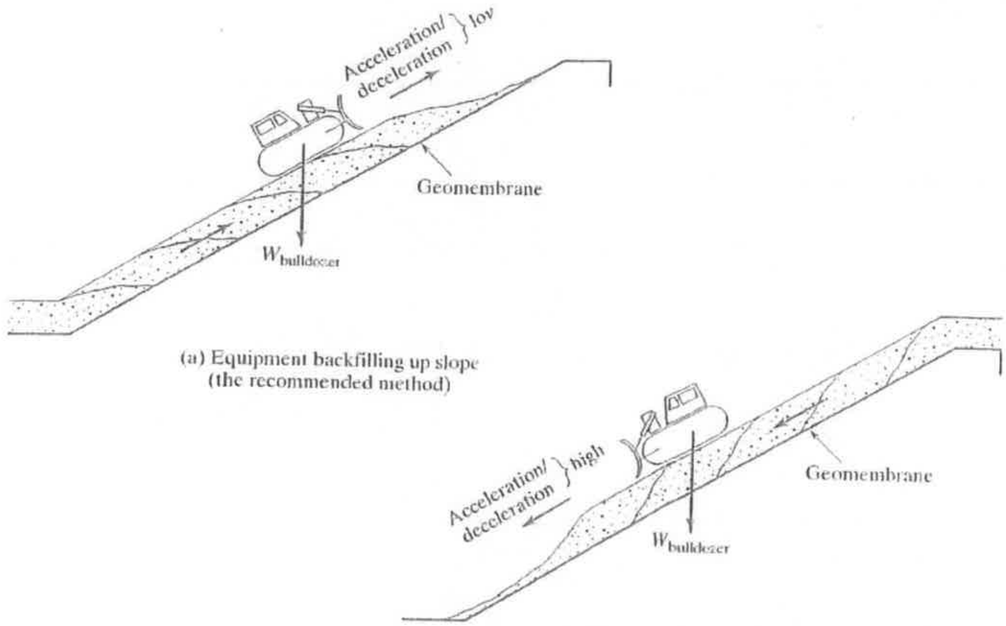
Comment In general, this is too low of a value for a final cover soil factor-of-safety and a redesign is necessary. There are many possible options to increase the value (e.g., changing the geometry of the situation, the use of toe berms, tapered cover soil thickness, and veneer reinforcement, see Koerner and Soong, 1998). Nevertheless, this general problem will be used throughout this section for comparison with other cover soil slope stability situations.

13.4.2 Tracked Construction Equipment Forces

The placement of cover soil on a slope with a relatively low shear strength interface (like a geomembrane) should always start at the toe and move upward to the crest. Figure 13.5(a) shows the recommended method. In doing so, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever-present passive wedge and a stable lower portion beneath the active wedge. While it is necessary to specify low ground pressure equipment to place the soil, the reduction in the *FS*-value for this situation of equipment working up the slope will be seen to be relatively small.

For soil placement down the slope, however, a stability analysis cannot rely on toe buttressing and also a dynamic stress should be included in the calculation. These conditions decrease the *FS*-value—in some cases, to a great extent. Figure 13.5(b) shows this procedure. Unless absolutely necessary, it is not recommended that cover soil be placed on a slope in this manner. If it is necessary, the design must consider the unsupported soil mass and the possible dynamic force of the specific type of construction equipment and its manner of operation.

For the *first case* of a bulldozer pushing cover soil up from the toe of the slope to the crest, the analysis uses the free body diagram of Figure 13.6(a). The analysis uses a



(a) Equipment backfilling up slope (the recommended method)

(b) Equipment backfilling down slope (method is not recommended)

FIGURE 13.5 Construction Equipment Placing Cover Soil on Slopes Containing Geosynthetics

known type of construction equipment (such as a bulldozer characterized by its ground contact pressure) and dissipates this force or stress through the cover soil thickness to the surface of the geomembrane. A Boussinesq analysis is used (see Poulos and Davis, 1974). This results in an equipment force per unit width of

$$W_e = q \cdot w \cdot I \tag{13.10}$$

where W_e = equivalent equipment force per unit width at the geomembrane interface;
$q = W_b/(2 \cdot w \cdot b)$;
W_b = actual weight of equipment (e.g., a bulldozer);
w = length of equipment track;
b = width of equipment track;
I = influence factor at the geomembrane interface (see Figure 13.7).

Upon determining the additional equipment force at the cover soil-to-geomembrane interface, the analysis proceeds as described in Section 13.3.1 for gravitational forces only. In essence, the equipment moving up the slope adds an additional term (W_e) to the W_A-force in Equation 13.4. Note, however, that this involves the generation of a resisting force as well. Thus, the net effect of

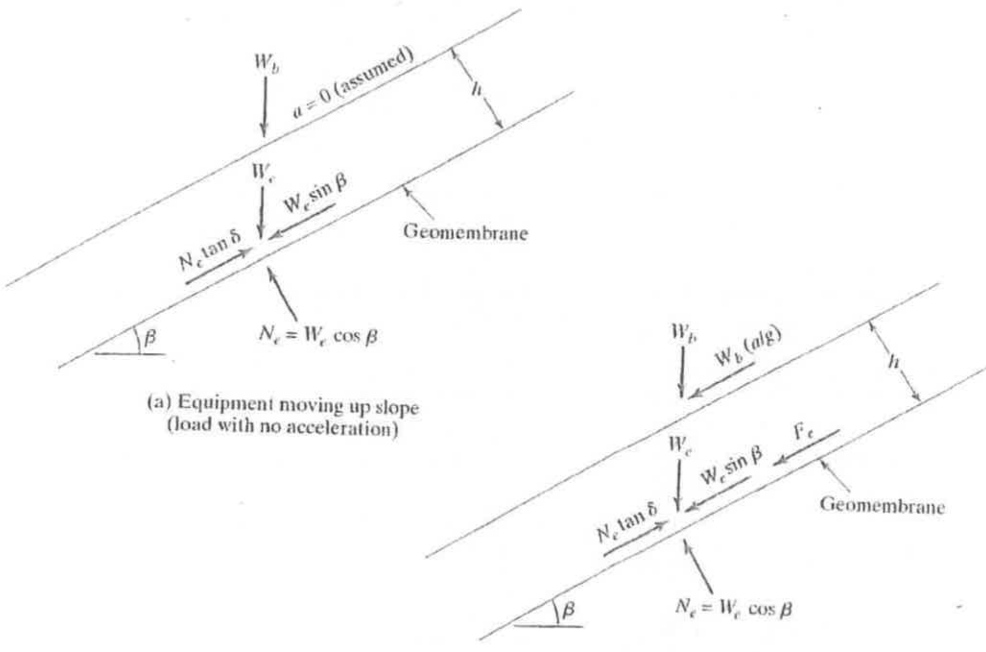


FIGURE 13.6 Additional (to Gravitational Forces) Limit Equilibrium Forces due to Construction Equipment Moving on Cover Soil (see Figure 13.3 for the gravitational soil force to which the above forces are added).

concerned. It should also be noted that no acceleration/deceleration forces are included in this analysis, which is somewhat idealistic. Using these concepts (the same equations used in Section 13.3.1 are used here), typical design curves for various *FS*-values as a function of equivalent ground contact equipment pressures and cover soil thicknesses are given in Figure 13.8. Note that the curves are developed specifically for the variables stated in the legend. Example 13.2 illustrates the use of the formulation.

EXAMPLE 13.2

The following are given: a 30-m-long slope with uniform cover soil of 300 mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30° and zero cohesion (i.e., it is a sand). It is placed on the slope using a bulldozer moving from the toe of the slope up to the crest. The bulldozer has a ground pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The cover soil to geomembrane friction angle is 22° with zero adhesion. What is the *FS*-value at a slope angle 3(H)-to-1(V)(i.e., 18.4°)?

Solution This problem follows Example 13.1 exactly except for the addition of the bulldozer moving up the slope. Using the additional equipment load, Equation 13.10 substituted into Equation 13.9 results in the following:

$$a = 73.1 \text{ kN/m}$$
$$b = -104.3 \text{ kN/m}$$
$$c = 17.0 \text{ kN/m}$$

Thus, $FS = 1.24$

This value can be confirmed using Figure 13.8.

Comment While the resulting *FS*-value is still low, the result is important to assess by comparing it with Example 13.1 (i.e., the same problem except without the bulldozer). It is seen that the *FS*-value has only decreased from 1.25 to 1.24. Thus, in general, a low ground contact pressure bulldozer placing cover soil up the slope with negligible acceleration/deceleration forces does not significantly decrease the factor-of-safety.

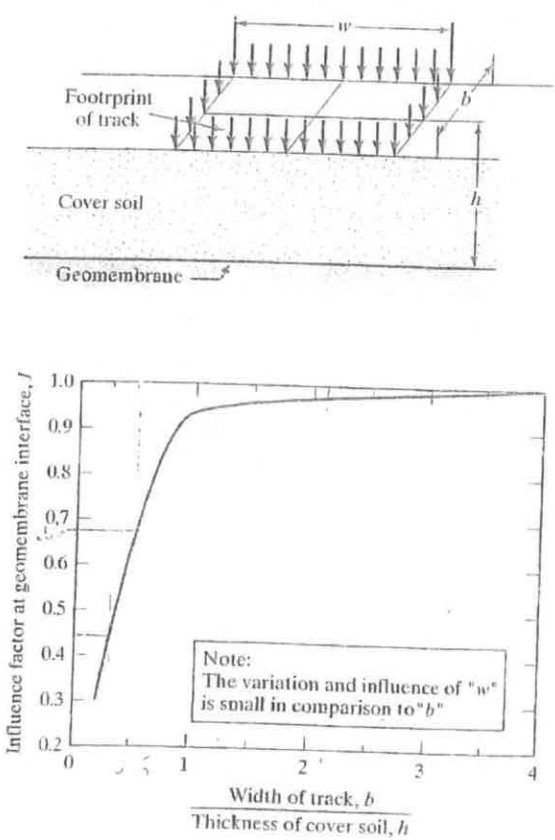


FIGURE 13.7 Values of Influence Factor, "I", for Use in Equation 13.10 to Dissipate Surface Force through the Cover Soil to the Geomembrane Interface (after Soong and Koerner, 1996)

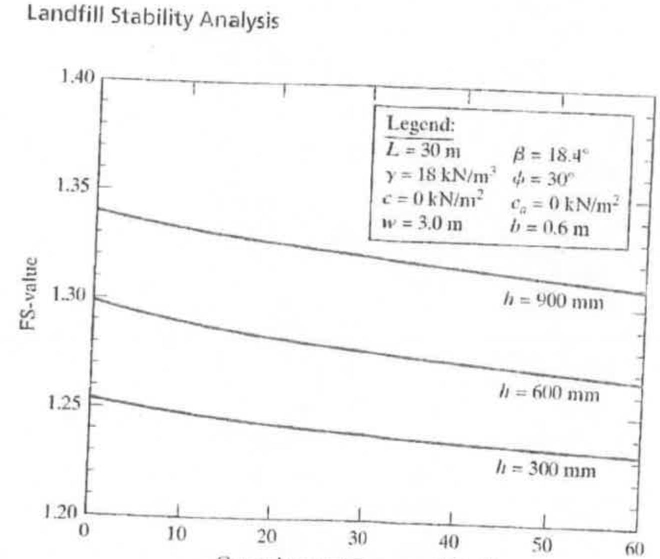


FIGURE 13.8 Design Curves for Stability of Different Thickness of Cover Soil for Various Construction Equipment Ground Contact Pressure

For the *second case* of a bulldozer pushing cover soil down from the crest of the slope to the toe as shown in Figure 13.5b, the analysis uses the force diagram of Figure 13.6(b). While the weight of the equipment is treated as just described, the lack of a passive wedge along with an additional force due to acceleration (or deceleration) of the equipment significantly decreases the resulting *FS*-values. This analysis again uses a specific piece of construction equipment operated in a specific manner. It produces a force parallel to the slope equivalent to $W_b \cdot (a/g)$, where W_b = the weight of the bulldozer, a = acceleration of the bulldozer, and g = acceleration due to gravity. Its magnitude is equipment operator dependent and related to both the equipment speed and time to reach such a speed (see Figure 13.9).

The acceleration of the bulldozer, coupled with an influence factor I from Figure 13.7, results in the dynamic force per unit width at the cover soil to geomembrane interface F_e. The relationship is given by

$$F_e = W_e \cdot (a/g) \tag{13.11}$$

where F_e = dynamic force per unit width parallel to the slope at the geomembrane interface;
W_e = equivalent equipment (e.g., bulldozer) force per unit width at geomembrane interface, recall Equation 13.10;
β = soil slope angle beneath geomembrane;
a = acceleration of the construction equipment;
g = acceleration due to gravity.

Using these concepts, the new force parallel to the cover soil surface is dissipated through the thickness of the cover soil to the interface of the geomembrane. Again, a

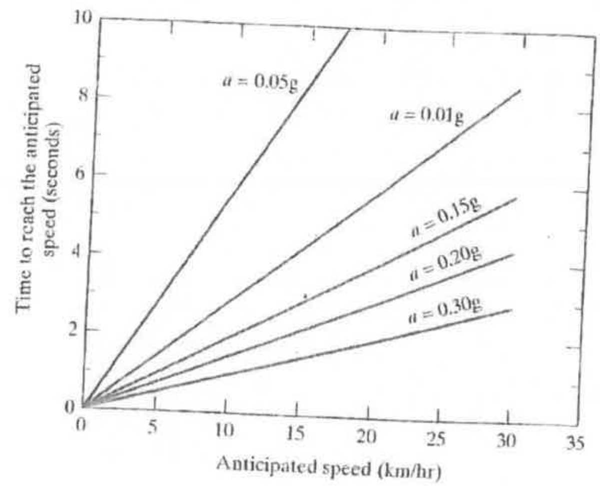


FIGURE 13.9 Graphic Relationship of Construction Equipment Speed and Rise Time to Obtain Equipment Acceleration.

Boussinesq analysis is used (see Poulos and Davis, 1974). The expression for determining the *FS*-value is derived next.

Considering the active wedge and balancing the forces in the direction parallel to the slope, the resulting formulation is

$$E_A + \frac{(N_e + N_A)\cdot\tan\delta + C_a}{FS} = (W_A + W_e)\cdot\sin\beta + F_e$$

where

N_e = effective equipment force normal to the failure plane of the active wedge.

$$N_e = W_E \cdot \cos\beta \tag{13.12}$$

Note that all the other symbols have been previously defined.

The interwedge force acting on the active wedge can now be expressed as

$$E_A = \frac{(FS)[(W_A + W_e)\cdot\sin\beta + F_e]}{FS} - \frac{[(N_A + N_e)\cdot\tan\delta + C_a]}{FS}$$

The passive wedge can be treated in a similar manner. The following formulation of the interwedge force acting on the passive wedge results:

$$E_P = \frac{C + W_P\cdot\tan\phi}{\cos\beta\cdot(FS) - \sin\beta\cdot\tan\phi}$$

By setting $E_A = E_P$, the resulting equation can be arranged in the form of the quadratic equation $ax^2 + bx + c = 0$ which in this case, using *FS*-values, is

$$a\cdot FS^2 + b\cdot FS + c = 0$$

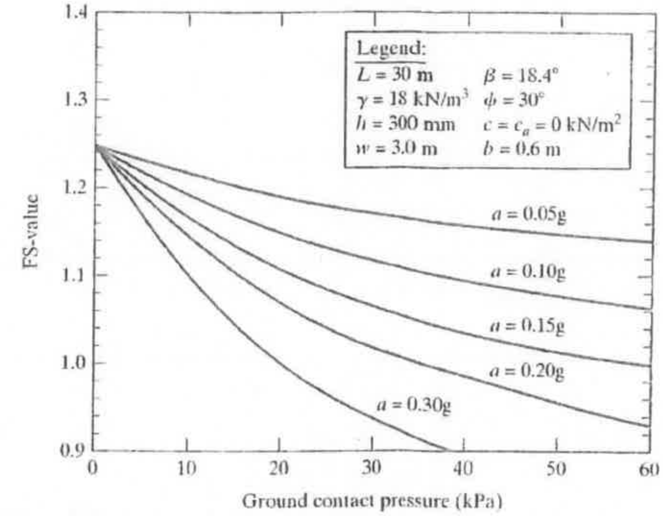


FIGURE 13.10 Design Curves for Stability of Different Construction Equipment Ground Contact Pressure for Various Equipment Accelerations

The resulting FS-value is then obtained from the conventional solution of the quadratic equation

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.13}$$

where
$$a = [(W_A + W_e) \cdot \sin\beta + F_c] \cdot \cos\beta$$
$$b = -\{[(N_A + N_e) \cdot \tan\delta + C_a] \cdot \cos\beta + [(W_A + W_e) \cdot \sin\beta + F_c] \cdot \sin\beta \cdot \tan\phi + (C + W_P \cdot \tan\phi)\}$$
$$c = [(N_A + N_e) \cdot \tan\delta + C_a] \cdot \sin\beta \cdot \tan\phi$$

Using these concepts, typical design curves for various FS-values as a function of equipment ground contact pressure and equipment acceleration can be developed (see Figure 13.10). Note that the curves are developed specifically for the variables stated in the legend. Example 13.3 illustrates the use of the formulation.

EXAMPLE 13.3

The following are given: a 30-m-long slope with uniform cover soil of 300-mm thickness at a unit weight of 18 kN/m^3. The soil has a friction angle of 30° and zero cohesion (i.e., it is a sand). It is placed on the slope using a bulldozer moving from the crest of the slope down to the toe. The bulldozer has a ground contact pressure of 30 kN/m^2 and tracks that are 3.0 m long and 0.6 m wide. The estimated equipment speed is 20 km/hr, and the time to reach this speed is 3.0 seconds. The cover soil to geomembrane friction angle is 22 degrees with zero adhesion. What is the FS-value at a slope angle of 3(H)-to-1(V) (i.e., 18.4°)?

Solution Using the design curves of Figure 13.10 along with Equation 13.13, the solution can be obtained.

- From Figure 13.9, at 20 km/hr and 3.0 seconds, the bulldozer's acceleration is 0.19g.
- From Equation 13.13,

$$a = 88.8 \text{ kN/m}$$
$$b = -107.3 \text{ kN/m}$$
$$c = 17.0 \text{ kN/m}$$

Thus, $FS = 1.03$

This value can be confirmed using Figure 13.10.

Comment This problem solution can now be compared with those of the previous two examples:

Example 13.1.	Cover soil along with no bulldozer loading:	$FS = 1.25$
Example 13.2.	Cover soil plus bulldozer moving up slope:	$FS = 1.24$
Example 13.3.	Cover soil plus bulldozer moving down slope:	$FS = 1.03$

The inherent danger of a bulldozer moving down the slope is readily apparent. Note, that the same result comes about by the bulldozer decelerating instead of accelerating. The sharp breaking action of the bulldozer is arguably the more severe condition, due to the extremely short times involved when stopping forward motion. Clearly, only in unavoidable situations should the cover soil placement equipment be allowed to work down the slope. If it is unavoidable, an analysis should be made of the specific stability situation and the construction specifications should reflect the precise conditions made in the design. The maximum weight and ground contact pressure of the equipment should be stated along with suggested operator movement of the cover soil placement operations. Truck traffic on the slopes can also give stresses as high or even higher than illustrated here and should be avoided in all circumstances.

13.4.3 Inclusion of Seepage Forces

The previous sections presented the general problem of slope stability analysis of cover soils placed on slopes under different conditions. The tacit assumption throughout was that either permeable soil or a drainage layer was placed above the barrier layer with adequate flow capacity to efficiently and safely remove permeating water away from the cross section. The amount of water to be removed is obviously a site-specific situation. Note that, in extremely arid areas, or with very low permeability cover soils, drainage may not be required, although this is generally the exception.

Unfortunately, adequate drainage of final covers has sometimes not been available and seepage-induced slope stability problems have occurred. Figure 13.11 shows a final cover slope failure during a heavy raining. The following situations have resulted in seepage-induced slides:

- Drainage soils with hydraulic conductivity (permeability) too low for site-specific conditions.
- Inadequate drainage capacity at the toe of long slopes, where seepage quantities accumulate and are at their maxim

FIGURE 13.11 Final Cover Slope Failure during a Heavy Raining

- Fine, cohesionless, cover soil particles migrating through the filter (if one is present) either clogging the drainage layer, or accumulating at the toe of the slope, thereby decreasing the as-constructed outlet permeability over time.
- Freezing of the outlet drainage at the toe of the slope, while the top of the slope thaws, thereby mobilizing seepage forces against the ice wedge at the toe.

If seepage forces of the types described occur, a variation in slope stability design methodology is required. Such an analysis is the focus of this subsection. (See Koerner and Soong, 1998; and Qian, 1997; also, Thiel and Stewart, 1993; and Soong and Koerner, 1996.)

Consider a cover soil of uniform thickness placed directly above a geomembrane at a slope angle of β, as shown in Figure 13.12. What is different from previous examples, however, is that within the cover soil there can exist a saturated soil zone for part or all of the thickness. The saturated boundary is shown as two possibly different phreatic surface orientations. This is because seepage can be built up in the cover soil in two different ways: a horizontal buildup from the toe upward, or a parallel-to-slope buildup outward. These two hypotheses are defined and quantified as a horizontal submergence ratio (HSR) and a parallel submergence ratio (PSR). The dimensional definitions of both ratios are given in Figure 13.12.

When analyzing the stability of slopes using the limit equilibrium method, free-body diagrams of the passive and active wedges are taken with the appropriate forces

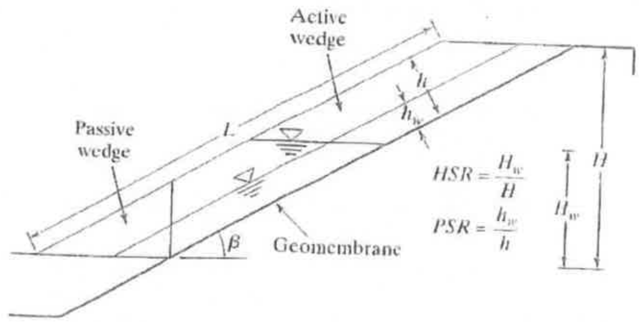


FIGURE 13.12 Cross Section of a Uniform Thickness Cover Soil on a Geomembrane Illustrating Different Submergence Assumptions and Related Definitions (Soong and Koerner, 1996)

being applied (now including pore water pressures). The formulation for the resulting factor of safety for horizontal seepage buildup and also for parallel-to-slope seepage buildup is described next.

13.4.3.1 The Case of the Horizontal Seepage Buildup. Figure 13.13 shows the free-body diagram of both the active and passive wedge assuming horizontal seepage building. Horizontal seepage buildup can occur when toe blockage occurs due to inadequate outlet capacity, contamination or physical blocking of outlets, or freezing conditions at the outlets.

All symbols used in Figure 13.13 were previously defined except the following:

γ_{sat} = saturated unit weight of the cover soil
γ_t = dry unit weight of the cover soil
γ_w = unit weight of water
H = vertical height of the slope measured from the toe
H_w = vertical height of the free water surface measured from the toe
U_h = resultant of the pore pressures acting on the interwedge surfaces
U_n = resultant of the pore pressures acting perpendicular to the slope
U_v = resultant of the vertical pore pressures acting on the passive wedge

The expression for determining the factor of safety can be derived as follows:

Considering the active wedge,

$$W_A = \frac{\gamma_{sat} \cdot h \cdot (2 \cdot H_w \cdot \cos\beta - h)}{\sin 2\beta} + \frac{\gamma_{dry} \cdot h \cdot (H - H_w)}{\sin\beta} \tag{13.14}$$

$$U_n = \frac{\gamma_w \cdot h \cdot \cos\beta \cdot (2 \cdot H_w \cdot \cos\beta - h)}{\sin 2\beta} \tag{13.15}$$

$$U_h = 0.5 \cdot \gamma_w \cdot h^2 \tag{13.16}$$

$$N_A = W_A \cdot \cos\beta + U_h \cdot \sin\beta - U_n \tag{13.17}$$

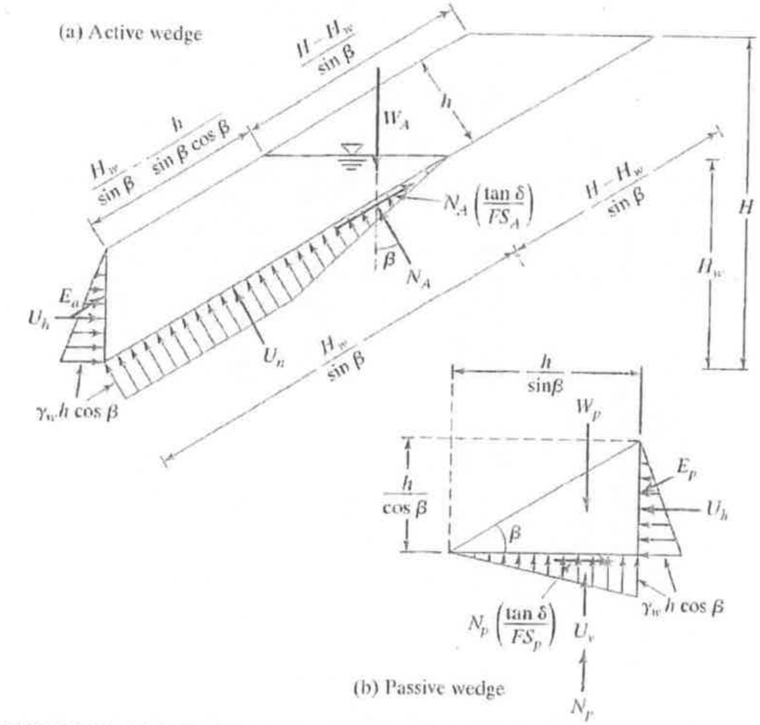


FIGURE 13.13 Limit Equilibrium Forces Involved in a Finite Length Slope of Uniform Cover Soil with Horizontal Seepage Buildup

The interwedge force acting on the active wedge can then be expressed as

$$E_A = W_A \cdot \sin\beta + U_h \cdot \cos\beta - \frac{N_A \cdot \tan\delta}{FS}$$

The passive wedge can be considered in a similar manner and the following expressions result:

$$W_P = \frac{\gamma_{sat} \cdot h^2}{\sin 2\beta} \tag{13.18}$$

$$U_v = U_h \cdot \cot\beta \tag{13.19}$$

The interwedge force acting on the passive wedge can then be expressed as

$$E_P = \frac{U_h \cdot (FS) - (W_P - U_v) \cdot \tan\phi}{\sin\beta \cdot \tan\phi - \cos\beta \cdot (FS)}$$

By setting $E_A = E_P$, the following equation can be arranged in the form of $ax^2 + bx + c = 0$, which in this case is

$$a \cdot FS^2 + b \cdot FS + c = 0$$

The resulting FS-value is then obtained from the conventional solution of the quadratic equation as

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.20}$$

where $a = W_A \cdot \sin\beta \cdot \cos\beta - U_h \cdot \cos^2\beta + U_h$

$b = -W_A \cdot \sin^2\beta \cdot \tan\phi + U_h \cdot \sin\beta \cdot \cos\beta \cdot \tan\phi - N_A \cdot \cos\beta \cdot \tan\delta - (W_P - U_v) \cdot \tan\phi$

$c = N_A \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$

13.4.3.2 The Case of Parallel-to-Slope Seepage Buildup. Figure 13.14 shows the free body diagrams of both the active and passive wedges with seepage buildup in the direction parallel to the slope. Parallel seepage buildup can occur when soils placed above a geomembrane are initially too low in their hydraulic conductivity, or become too low due to long-term clogging from overlying soils that are not filtered. The individual forces, friction angles, and slope angles involved in Figure 13.14 are listed as follows:

- W_A = weight of the active wedge (area times unit weight), lb/ft or kN/m;
- W_P = weight of the passive wedge (area times unit weight), lb/ft or kN/m;
- β = angle of the slope, degree;
- H = height of the cover soil slope from the toe of the cover soil to the top of the slope (see Figure 13.14), ft or m;
- h = thickness of the soil layer (perpendicular to the slope), ft or m;
- h_w = depth of seepage water in the soil layer (perpendicular to the slope), ft or m;
- γ = moisture unit weight of the soil layer, lb/ft^3 or kN/m^3;
- γ_{sat} = saturated unit weight of the soil layer, lb/ft^3 or kN/m^3;
- γ_w = unit weight of water, 62.4 lb/ft^3 or 9.81 kN/m^3;
- ϕ = friction angle of the cover soil, degree;
- δ = interface friction angle between the soil layer and geomembrane, degree;
- N_A = normal force acting on bottom of the active wedge, lb/ft or kN/m;
- F_A = frictional force acting on bottom of the active wedge, lb/ft;
- U_{AN} = resultant of the pore water pressures acting on bottom of the active wedge (perpendicular to the slope), lb/ft or kN/m;
- U_{AH} = resultant of the pore water pressures acting on lower lateral side of the active wedge (perpendicular to the interface between the active and passive wedges), lb/ft or kN/m;
- E_A = force from passive wedge acting on active wedge (unknown in magnitude but assumed direction parallel to the slope), lb/ft or kN/m;
- N_P = normal force acting on the bottom of passive wedge, lb/ft or kN/m;
- F_P = frictional force acting on the bottom of passive wedge, lb/ft or kN/m;

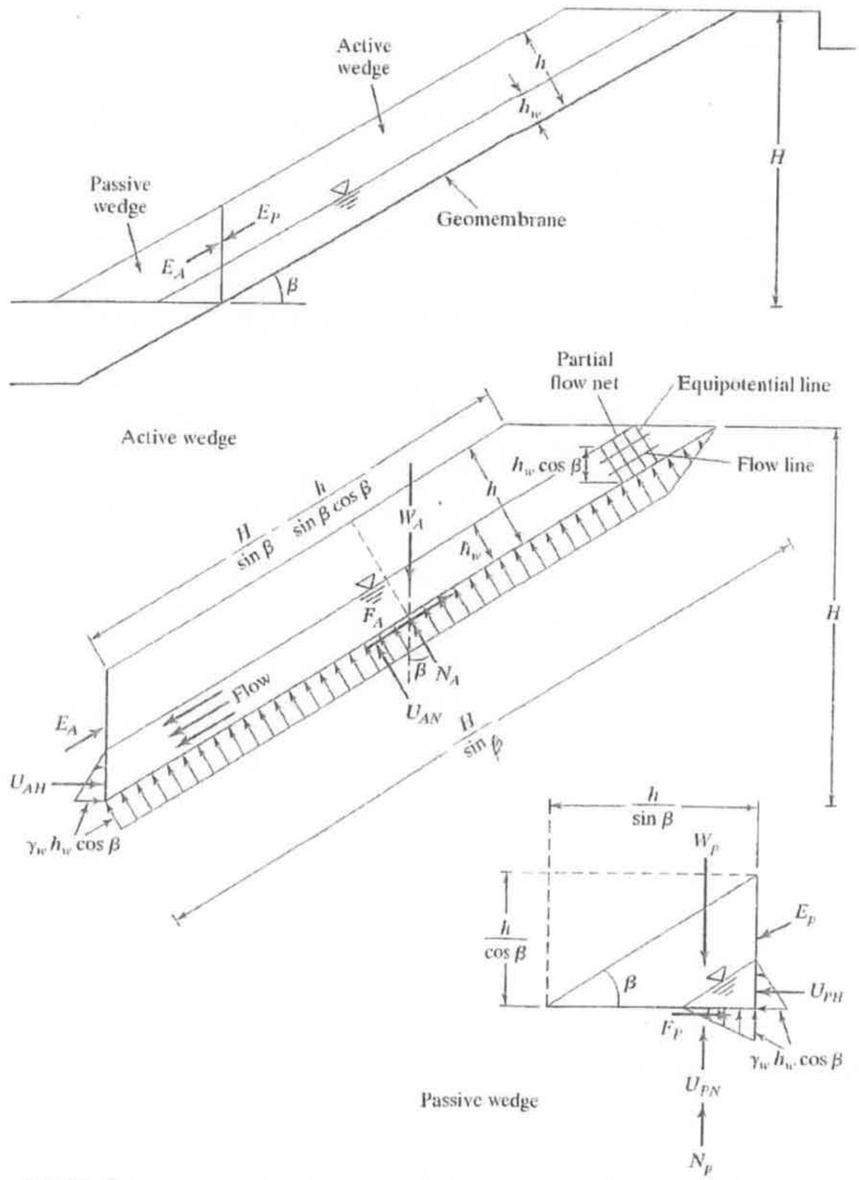
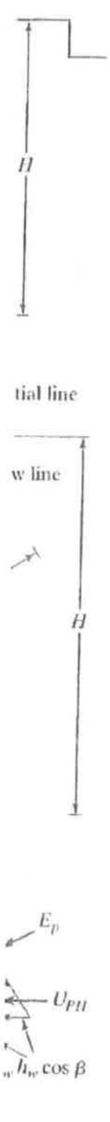


FIGURE 13.14 Cross Section of Sand Layer over Geomembrane on Side Slope with Seepage Parallel to Slope.



U_H = resultant of the pore water pressures acting on lateral side of the active wedge or passive wedge (perpendicular to the lateral side), lb/ft or kN/m, $U_H = U_{AH} = U_{PH}$;

U_{PN} = resultant of the pore water pressures acting on bottom of the passive wedge (perpendicular to bottom of the passive wedge), lb/ft or kN/m;

E_P = force from active wedge acting on passive wedge (unknown in magnitude but assumed direction parallel to the slope), lb/ft or kN/m, $E_A = E_P$;

FS = factor of safety for stability of the cover soil mass.

Considering the force equilibrium of the active wedge (Figure 13.14), we obtain

$$\Sigma F_Y = 0: \quad N_A + U_{AN} = W_A \cdot \cos\beta + U_{AH} \cdot \sin\beta$$

$$N_A = W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta \tag{13.21}$$

$$\Sigma F_X = 0: \quad F_A + E_A + U_{AH} \cdot \cos\beta = W_A \cdot \sin\beta$$

$$E_A = W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - F_A \tag{13.22}$$

$$F_A = N_A \cdot \tan\delta/FS \tag{13.23}$$

Substituting Equation 13.21 into Equation 13.23 gives

$$F_A = (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta/FS \tag{13.24}$$

Substituting Equation 13.24 into Equation 13.22 gives

$$E_A = W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta/FS \tag{13.25}$$

Considering the force equilibrium of the passive wedge (Figure 13.14) yields

$$E_P = E_A \tag{13.26}$$

$$\Sigma F_Y = 0: \quad N_P + U_{PN} = W_P + E_P \cdot \sin\beta \tag{13.27}$$

Substituting Equation 13.26 into Equation 13.27 gives

$$N_P = W_P + E_A \cdot \sin\beta - U_{PN} \tag{13.28}$$

Substituting Equation 13.25 into Equation 13.28 gives

$$N_P = W_P - U_{PN} + [W_A \cdot \sin\beta - U_{AH} \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \tan\delta/FS] \cdot \sin\beta$$

$$N_P = W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta/FS \tag{13.29}$$

$$\Sigma F_X = 0: \quad F_P = U_{PH} + E_P \cdot \cos\beta \tag{13.30}$$

Substituting Equation 13.26 into Equation 13.30 gives

$$F_P = U_{PH} + E_A \cdot \cos\beta \tag{13.31}$$

Substituting Equation 13.25 into Equation 13.31 gives

$$F_P = U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta - (W_A \cdot \cos\beta - U_{AN}$$

$$FS = \frac{N_P \cdot \tan\phi}{F_P} \tag{13.33}$$

Substituting Equations 13.29 and 13.32 into Equation 13.33 gives

$$FS = \frac{\begin{array}{c}(W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \cdot \tan\phi \\ - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi / FS\end{array}}{\begin{array}{c}U_{PH} + W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta \\ - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \cdot \cos\beta \cdot \tan\delta / FS\end{array}}$$

$$\begin{aligned}(U_{PH} &+ W_A \cdot \sin\beta \cdot \cos\beta - U_{AH} \cdot \cos^2\beta) \cdot FS - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \\ &\cdot \cos\beta \cdot \tan\delta = (W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \\ &\cdot \tan\phi - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi / FS\end{aligned}$$

$$\begin{aligned}(W_A &\cdot \sin\beta \cdot \cos\beta + U_{PH} - U_{AH} \cdot \cos^2\beta) \cdot FS^2 - (W_A \cdot \cos\beta - U_{AN} + U_{AH} \cdot \sin\beta) \\ &\cdot \cos\beta \cdot \tan\delta \cdot FS = (W_P - U_{PN} + W_A \cdot \sin^2\beta - U_{AH} \cdot \sin\beta \cdot \cos\beta) \\ &\cdot \tan\phi \cdot FS - (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi\end{aligned}$$

$$\begin{aligned}&(W_A \cdot \sin\beta \cdot \cos\beta + U_{PH} - U_{AH} \cdot \cos^2\beta) \cdot FS^2 - [W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi \\ &+ \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_{AH} \cdot \sin\beta \cdot \cos\beta \\ &\cdot (\tan\phi - \tan\delta)] \cdot FS + (W_A \cdot \cos\beta - U_A + U_{AH} \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi = 0\end{aligned} \tag{13.34}$$

Because $U_H = U_{PH} = U_{AH}$,

$$\begin{aligned}&[W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta)] \cdot FS^2 - [W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi \\ &+ \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \\ &\cdot (\tan\phi - \tan\delta)] \cdot FS + (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi = 0\end{aligned} \tag{13.35}$$

Using $a \cdot x^2 + b \cdot x + c = 0$

The resulting FS can be expressed as

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.36}$$

where

$$\begin{aligned}a &= W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta) \\ b &= -[W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta \\ &\quad - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)] \\ c &= (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi\end{aligned}$$

$$U_{AN} = \gamma_w \cdot h_w \cdot (H - 0.5\, h_w \cdot \cos\beta)/\tan\beta \tag{13.37}$$

$$U_H = 0.5 \cdot \gamma_w \cdot h_w^{\,2} \tag{13.38}$$

$$U_{PN} = 0.5 \cdot \gamma_w \cdot h_w^{\,2}/\tan\beta \tag{13.39}$$

$$\begin{aligned}W_A = 0.5 \cdot [&\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) \\ &+ \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta)\end{aligned} \tag{13.40}$$

$$W_P = 0.5 \cdot [\gamma \cdot (h^2 - h_w^{\,2}) + \gamma_{sat} \cdot h_w^{\,2}]/(\sin\beta \cdot \cos\beta) \tag{13.41}$$

(13.33)

33 gives

$3\cdot\cos\beta)\cdot\tan\phi$
$3\cdot\tan\delta\cdot\tan\phi/FS$
$\cdot\cos^2\beta$
$os\beta\cdot\tan\delta/FS$

$U_{AN} + U_{AH}\cdot\sin\beta)$
$- U_{AH}\cdot\sin\beta\cdot\cos\beta)$
$\cdot\sin\beta\cdot\tan\delta\cdot\tan\phi/FS$

$- U_{AN} + U_{AH}\cdot\sin\beta)$
$U_{AH}\cdot\sin\beta\cdot\cos\beta)$
$\beta\cdot\tan\delta\cdot\tan\phi$

$W_A\cdot(\sin^2\beta\cdot\tan\phi$
$\beta\cdot\cos\beta$
$\beta\cdot\tan\delta\cdot\tan\phi = 0$

(13.34)

$\cdot(\sin^2\beta\cdot\tan\phi$
$\cdot\cos\beta$
$\tan\delta\cdot\tan\phi = 0$

(13.35)

(13.36)

$U_{AN}\cdot\cos\beta\cdot\tan\delta$

(13.37)

(13.38)

(13.39)

$\beta)$ (13.40)

EXAMPLE 13.4

A 44-ft (13.2-m) high and 3(H):1(V) slope has cover sand with a uniform thickness of 2 ft (0.6 m) at a unit weight of 110 lb/ft^3 (17.3 kN/m^3). The cover sand has a friction angle of 32 degrees and zero cohesion. Seepage occurs parallel to the slope and the seepage water head in the sand layer is 6 inches (0.15 m). The saturated unit weight of sand is 115 lb/ft^3 (18 kN/m^3). The interface friction angle between sand drainage layer and geomembrane is 22 degrees and zero adhesion. What is the factor of safety at a slope of 3(H)-to-1(V)?

Solution The side slope angle is at 18.4° for a 3(H):1(V) slope. Hence,

$\sin\beta = \sin(18.4°) = 0.316,\ \cos\beta = \cos(18.4°) = 0.949,\ \tan\beta = \tan(18.4°) = 0.333.$

$H = 44$ ft (13.2 m), $h = 2$ ft (0.6 m), $h_w = 0.5$ ft (0.15 m), $\gamma = 110$ lb/ft^3 (17.3 kN/m^3).

$\gamma_{sat} = 115$ lb/ft^3 (18 kN/m^3), $\gamma_w = 62.4$ lb/ft^3 (9.81 kN/m^3), $\phi = 32°$, $\delta = 22°$.

$\tan\phi = \tan(32°) = 0.625,\ \tan\delta = \tan(22°) = 0.404.$

$$\begin{aligned} U_{AN} &= \gamma_w\cdot h_w\cdot(H - 0.5\,h_w\cdot\cos\beta)/\tan\beta \\ &= (62.4)(0.5)[44 - (0.5)(0.5)(0.949)]/(0.333) = 4{,}100.3 \text{ lb/ft } (58.02 \text{ kN/m}) \end{aligned} \quad (13.37)$$

$$\begin{aligned} U_H &= 0.5\cdot\gamma_w\cdot h_w^2 \\ &= (0.5)(62.4)(0.5)^2 = 7.8 \text{ lb/ft } (0.11 \text{ kN/m}) \end{aligned} \quad (13.38)$$

$$\begin{aligned} U_{PN} &= 0.5\cdot\gamma_w\cdot h_w^2/\tan\beta \\ &= (0.5)(62.4)(0.5)^2/(0.333) = 23.4 \text{ lb/ft } (0.33 \text{ kN/m}) \end{aligned} \quad (13.39)$$

$$\begin{aligned} W_A &= 0.5\cdot[\gamma\cdot(h - h_w)(2\cdot H\cdot\cos\beta - h - h_w) \\ &\quad + \gamma_{sat}\cdot h_w\cdot(2\cdot H\cdot\cos\beta - h_w)]/(\sin\beta\cdot\cos\beta) \\ &= (0.5)\{(110)(2 - 0.5)[(2)(44)(0.949) - 2 - 0.5] \\ &\quad + (115)(0.5)[(2)(44)(0.949) - 0.5]\}/[(0.316)(0.949)] \\ &= (0.5)(13{,}366.98 + 4{,}773.19)/[(0.316)(0.949)] = 30{,}245.3 \text{ lb/ft } (427.6 \text{ kN/m}) \end{aligned} \quad (13.40)$$

$$\begin{aligned} W_P &= 0.5\cdot[\gamma\cdot(h^2 - h_w^2) + \gamma_{sat}\cdot h_w^2]/(\sin\beta\cdot\cos\beta) \\ &= (0.5)\{(110)[(2)^2 - (0.5)^2] + (115)(0.5)^2\}/[(0.316)(0.949)] = 735.7 \text{ lb/ft } (10.4 \text{ kN/m}) \end{aligned} \quad (13.41)$$

Using Equation 13.36,

$$\begin{aligned} a &= W_A\cdot\sin\beta\cdot\cos\beta + U_H\cdot(1 - \cos^2\beta) \\ &= (30{,}245.3)(0.316)(0.949) + (7.8)[1 - (0.949)^2] = 9{,}071 \text{ (128 for SI units)} \end{aligned}$$

$$\begin{aligned} b &= -\{W_P\cdot\tan\phi + W_A\cdot(\sin^2\beta\cdot\tan\phi + \cos^2\beta\cdot\tan\delta) - U_{AN}\cdot\cos\beta\cdot\tan\delta - U_{PN}\cdot\tan\phi \\ &\quad + U_H\cdot\sin\beta\cdot\cos\beta\cdot(\tan\phi - \tan\delta)\} \\ &= -\{(735.7)(0.625) + (30{,}245.3)[(0.316)^2(0.625) + (0.949)^2(0.104)] - (4{,}100.3)(0.949)(0.404) \\ &\quad - (23.4)(0.625) + (7.8)(0.316)(0.949)(0.625 - 0.404)\} \\ &= -(459.8 + 12{,}892.1 - 1{,}572.0 - 14.6 + 0.5) = -11{,}766 \text{ (}-166\text{ for SI units)} \end{aligned}$$

$$\begin{aligned} c &= (W_A\cdot\cos\beta - U_{AN} + U_H\cdot\sin\beta)\cdot\sin\beta\cdot\tan\delta\cdot\tan\phi \\ &= [(30{,}245.3)(0.949) - 4{,}100.3 + (7.8)(0.316)](0.316)(0.625)(0.404) = 1{,}963 \text{ (28 for SI units)} \end{aligned}$$

$$FS = \frac{-b \pm (b^2 - 4\cdot a\cdot c)^{0.5}}{2\cdot a} \quad (13.36)$$

$11{,}766 + [(-11{,}766)^2 - (4)(9{,}071)(1{,}963)]^{0.5}$

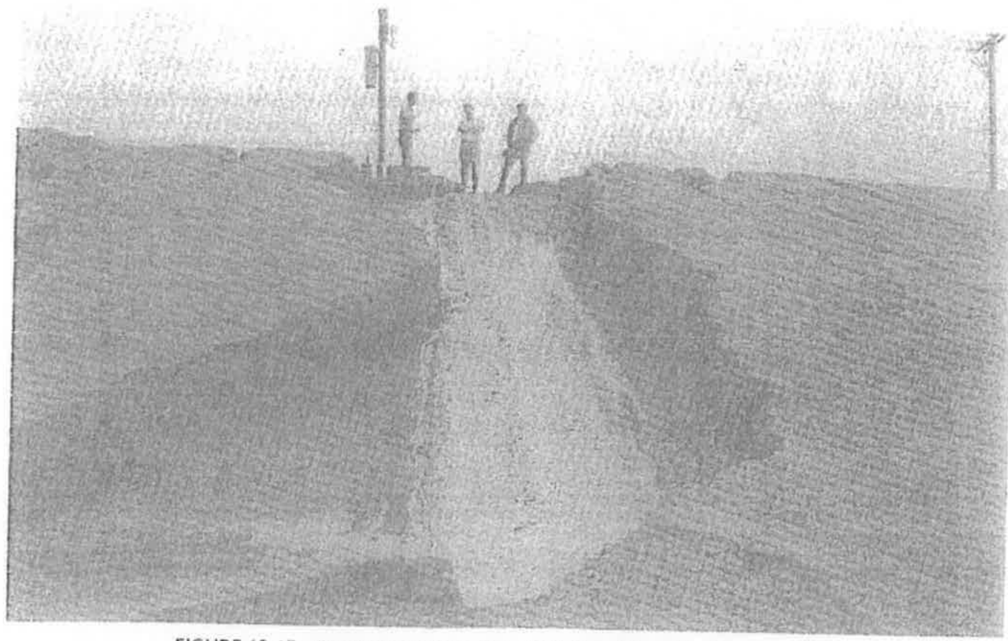


FIGURE 13.15 Sand Layer Failure along Sideslope Caused by Seepage Force

$$= \frac{11{,}766 + 8{,}198}{(2)(9{,}071)}$$
$$= 1.10$$

Comment The seriousness of seepage forces in a slope of this type is immediately obvious. Had the saturation been 100% of the drainage layer thickness, the FS-value would have been still lower. Furthermore, the result using a horizontal assumption of saturated cover soil with the same saturation ratio will give essentially identical low *FS*-values. Clearly, the teaching of this example problem is that adequate long-term drainage above the barrier layer in cover soil slopes must be provided to avoid seepage forces from occurring. Figure 13.15 shows a sand layer sliding failure along sideslope caused by seepage force.

An incremental placement method should be implemented for sideslopes higher than the maximum height that can be built in a single lift with a minimum required factor of safety, such as the previous example. Based on the incremental placement method, the first step is to place the sand drainage layer on the sideslope to the maximum unsupported height. As waste is filled against the sideslope to approximately 2 feet (0.6 m) below the protective layer, the next lift of the layer can proceed. This procedure that is illustrated in Figure 13.16 should be continued until the protective layer reaches the top of the sideslope. The heights of the following lifts of the sand drainage layer should not be higher than the calculated maximum unsupported height minus 2

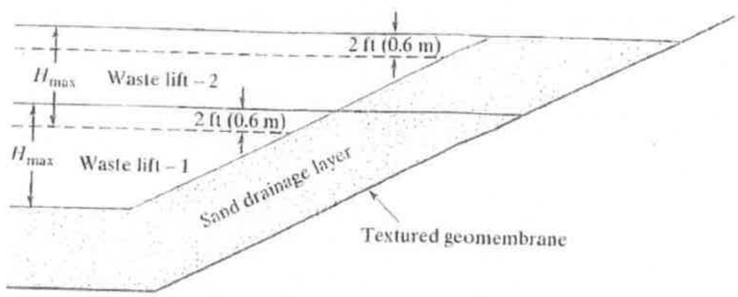


FIGURE 13.16 Incremental Placement of Soil Drainage Layer on Sideslope

feet (0.6 m). The height of the first lift of sand placement can be calculated as shown in the equations that follow (Qian, 1997):

In U.S. units,

$$H = (H_{total} - 2)/n + 2 \tag{13.42}$$

In SI units,

$$H = (H_{total} - 0.6)/n + 0.6 \tag{13.43}$$

where H = height of the first step of sand placement on the sideslope (see Figure 13.14), ft or m;

H_{total} = total height of the cover sand slope from the toe of the cover sand to the top of the slope (see Figure 13.16), ft or m;

n = number of the placement steps.

EXAMPLE 13.5

Continue the calculations of Example 13.4 and use the incremental method to achieve a factor of safety no less than 1.2 for the cover sand resting on the sideslope?

Solution Use the incremental method to place drainage sand on the side slope to achieve a minimum factor of safety of 1.2. Try three steps of sand placement (n = 3) on the sideslope.

$$H = (H_{total} - 2)/n + 2 \tag{13.42}$$

$$= (44 - 2)/3 + 2 = 14 + 2 = 16 \text{ ft } (4.8 \text{ m})$$

So,

H = 16 ft (4.8 m), h = 2 ft (0.6 m), h_w = 0.5 ft (0.15 m), γ = 110 lb/ft^3 (17.3 kN/m^3), γ_{sat} = 115 lb/ft^3 (18 kN/m^3), γ_w = 62.4 lb/ft^3 (9.81 kN/m^3), ϕ = 32°, δ = 22°.

$\tan\phi = \tan(32°) = 0.625$, $\tan\delta = \tan(22°) = 0.404$,

$$U_{AN} = \gamma_w \cdot h_w \cdot (H - 0.5\ h_w \cdot \cos\beta)/\tan\beta \tag{13.37}$$

$$= (62.4)(0.5)[16 - (0.5)(0.5)(0.949)]/(0.333) = 1{,}476.9 \text{ lb/ft } (20.90 \text{ kN/m})$$

$$U_H = 0.5 \cdot \gamma_w \cdot h_w^{\ 2} \tag{13.38}$$

$$= (0.5)(62.4)(0.5)^2 = 7.8 \text{ lb/ft } (0.11 \text{ kN/m})$$

$$U_{PN} = 0.5 \cdot \gamma_w \cdot h_w^{\ 2}/\tan\beta \tag{13.39}$$

$$= (0.5)(62.4)(0.5)^2/(0.333) = 23.4 \text{ lb/ft } (0.33 \text{ kN/m})$$

$$W_A = 0.5 \cdot [\gamma \cdot (h - h_w)(2 \cdot H \cdot \cos\beta - h - h_w) + \gamma_{sat} \cdot h_w \cdot (2 \cdot H \cdot \cos\beta - h_w)]/(\sin\beta \cdot \cos\beta) \tag{13.40}$$

$$= (0.5)\{(110)(2 - 0.5)[(2)(16)(0.949) - 2 - 0.5] + (115)(0.5)[(2)(16)(0.949) - 0.5]\}/[(0.316)(0.949)]$$

$$= (0.5)(4{,}598.22 + 1{,}717.41)/[(0.316)(0.949)] = 10{,}530.1 \text{ lb/ft } (148.9 \text{ kN/m})$$

$$W_P = 0.5 \cdot [\gamma \cdot (h^2 - h_w^{\ 2}) + \gamma_{sat} \cdot h_w^2]/(\sin\beta \cdot \cos\beta) \tag{13.41}$$

$$= (0.5)\{(110)[(2)^2 - (0.5)^2] + (115)(0.5)^2\}/[(0.316)(0.949)] = 735.7 \text{ lb/ft } (10.4 \text{ kN/m})$$

Equation 13.36 yields

$$a = W_A \cdot \sin\beta \cdot \cos\beta + U_H \cdot (1 - \cos^2\beta)$$

$$= (10{,}530.1)(0.316)(0.949) + (7.8)[1 - (0.949)^2] = 3{,}159 \text{ (45 for SI units)}$$

$$b = -[W_P \cdot \tan\phi + W_A \cdot (\sin^2\beta \cdot \tan\phi + \cos^2\beta \cdot \tan\delta) - U_{AN} \cdot \cos\beta \cdot \tan\delta - U_{PN} \cdot \tan\phi + U_H \cdot \sin\beta \cdot \cos\beta \cdot (\tan\phi - \tan\delta)]$$

$$= -\{(735.7)(0.625) + (10{,}530.1)[(0.316)^2(0.625) + (0.949)^2(0.404)] - (1{,}476.9)(0.949)(0.404) - (23.4)(0.625) + (7.8)(0.316)(0.949)(0.625 - 0.404)\}$$

$$= -(459.8 + 4{,}488.5 - 566.2 - 14.6 + 0.5) = -4{,}368 \text{ } (-62 \text{ for SI units})$$

$$c = (W_A \cdot \cos\beta - U_{AN} + U_H \cdot \sin\beta) \cdot \sin\beta \cdot \tan\delta \cdot \tan\phi$$

$$= [(10{,}530.1)(0.949) - 1{,}476.9 + (7.8)(0.316)](0.316)(0.625)(0.404) = 680 \text{ (10 for SI units)}$$

$$FS = \frac{-b \pm (b^2 - 4 \cdot a \cdot c)^{0.5}}{2 \cdot a} \tag{13.36}$$

$$= \frac{4{,}368 + [(-4{,}368)^2 - (4)(3{,}159)(680)]^{0.5}}{(2)(3{,}159)}$$

$$= \frac{4{,}368 + 3{,}238}{(2)(3{,}159)}$$

$$= 1.20$$

Thus, based on the above calculation, the first step is to place the drainage sand on the sideslope to a height of 16 feet (4.8 m). As waste is filled against the sideslope to approximately 2 feet (0.6 m) below the protective layer, the next lift of 14 feet (4.2 m) can be placed. This procedure should be continued until the protective layer reaches the top of the sideslope.

13.4.4 Inclusion of Seismic Forces

In areas of anticipated earthquake activity, the slope stability analysis of a final cover soil over an engineered landfill, abandoned dump, or remediated site must consider seismic forces. In the United States, the Environmental Protection Agency (EPA)

Reference 3

Dozer Specifications from Manufacturer

D5C LGP	
1 kW	90 HP
00 kg	19,800 lb
—	
3204	
2400	
4	
4 mm	4.5"
7 mm	5"
.2 L	318 in³
6	
0 mm	26"
.14 m	7'0.4"
83 m²	4389 in²
.72 m	5'8"
.75 m	5'9.2"
.72 m	8'11"
.07 m	13'4"
.99 m	9'9.8"
—	
.38 m	7'10"
1.4 mm	14.2"
—	
—	
.26 m	10'8"
.95 m	9'8"
167 L	44 U.S. gal

ator. D3C LGP Series II

MODEL	D4H LGP Series III		D5H LGP Series II		D6D LGP		D6H LGP Series II	
Flywheel Power	86 kW	116 HP	97 kW	130 HP	104 kW	140 HP	127 kW	170 HP
Operating Weight*								
(Power Shift)	12 196 kg	26,830 lb	15 337 kg	33,818 lb	17 373 kg	38,300 lb	19 814 kg	43,590 lb
(Direct Drive)	12 356 kg	27,180 lb	15 419 kg	33,999 lb	—		19 969 kg	43,976 lb
(Power Shift Differential Steer)	—		—		—		20 060 kg	44,131 lb
Engine Model	3304		3304		3306		3306	
Rated Engine RPM	2200		2200		1900		1900	
No. of Cylinders	4		4		6		6	
Bore	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"	121 mm	4.75"
Stroke	152 mm	6"	152 mm	6"	152 mm	6"	152 mm	6"
Displacement	7 L	425 in³	7 L	425 in³	10.5 L	638 in³	10.5 L	638 in³
Track Rollers (Each Side)	7		8		7		8	
Width of Standard Track Shoe	760 mm	30"	860 mm	34"	910 mm	36"	915 mm	36"
Length of Track on Ground	2.62 m	8'7"	3.12 m	10'3"	2.87 m	9'5"	3.27 m	10'8.5"
Ground Contact Area (W/Std. Shoe)	3.98 m²	6170 in²	5.37 m²	8320 in²	5.25 m²	8136 in²	5.97 m²	9254 in²
Track Gauge	2.00 m	6'6"	2.16 m	7'1"	2.11 m	6'9"	2.23 m	7'3"
GENERAL DIMENSIONS:								
Height (Stripped Top)**	2.20 m	7'3"	2.30 m	7'6.5"	2.05 m	6'8"	2.32 m	7'7"
Height (To Top of ROPS Canopy)	3.63 m	9'11.4"	3.12 m	10'3"	2.92 m	9'7.5"	3.16 m	10'5"
Height (To Top of Cab ROPS)	—		3.18 m	10'5"	—		3.16 m	10'5"
Overall Length (With P Blade)	4.77 m	15'8"	5.30 m	17'6.3"	—		5.18 m	17'0"
(Without Blade)	—		4.13 m	13'7"	—		4.49 m	14'9"
Overall Length (With S Blade)	—		—		5.16 m	16'11"	—	
(Without Blade)	—		—		3.94 m	12'11"	—	
Width (Over Trunnion)	—		3.26 m	10'8.4"	—		3.43 m	11'3"
Width (W/O Trunnion — Std. Shoe)	2.76 m	9'1"	3.02 m	9'11"	—		3.14 m	10'3.6"
Width (With Standard Shoe)	—		—		3.02 m	9'11"	—	
Ground Clearance	363 mm	14.3"	529 mm	20.8"	310 mm	12.2"	382 mm	15"
Blade Types and Widths:								
Straight	3.26 m	10'8.2"	3.65 m	12'0"	3.71 m	12'2"	3.99 m	13'1"
Angle	—		—		—		—	
Power Angle & Tilt	—		3.98 m	13'0.1"	—		—	
"P" Straight	3.26 m	10'8.2"	—		—		—	
Angled	3.00 m	9'10.1"	3.66 m	11'11.9"	—		—	
Fuel Tank Refill Capacity	200 L	52 U.S. gal	246 L	65 U.S. gal	295 L	78 U.S. gal	337 L	89 U.S. gal

*Operating Weight includes lubricants, coolant, full fuel tank, straight bulldozer, hydraulic controls and fluid, ROPS canopy and operator and rigid drawbar. D5H Series II with P-blade.
**Height (stripped top) — without ROPS canopy, exhaust, seat back or other easily removed encumbrances.
Note: D4H LGP Series III has P-blade.

TECHNICAL MEMORANDUM

Date: December 1, 2010
To: Steven Parrish
Project No.: 083-82671.6
Company: Progress Energy Florida

From: Sam Stafford SFS
Ken Karably KBK

RE: **COMPACTION GROUTING PROGRAM REPORT**
ASH STORAGE/DISPOSAL AREA - EAST SLIVER EXPANSION
PROGRESS ENERGY FLORIDA - CRYSTAL RIVER ENERGY COMPLEX

Dear Mr. Parrish:

Golder Associates Inc. (Golder) is pleased to provide Progress Energy Florida, Inc. (PEF) with this technical memorandum documenting the compaction grouting program for the east sliver expansion of the ash storage/disposal area at Crystal River Energy Complex (CREC). The compaction grouting program for subgrade improvement was performed in accordance with the layout and specifications provided to, and approved by, the Florida Department of Environmental Protection (FDEP)[1]. Golder retained a specialty grouting contractor, Earth Tech, to perform the compaction grouting work and provided oversight and engineering support in accordance with our proposal dated June 28, 2010.

The compaction grouting program began on July 26, 2010 with the installation of injection points (see Figure 1). Rotary wash drilling techniques were used to install the three-inch diameter steel injection points. The design installation depth was 40 feet below ground surface (bgs) or refusal, except for location CP-6 which had a design installation depth of 60 feet bgs or refusal. Installation depths at locations CP-1 through CP-5 ranged from 26 feet to 41 feet bgs and installation depths for location CP-6 ranged from 40 to 60 feet bgs. In total, 30 primary injection points and 4 secondary injection points were installed. Installation dates and depths for injection points are shown in Table 1.

Compaction grouting of injection points began on July 27, 2010 and was completed on August 5, 2010. The grout mix provided by Earth Tech was consistent with project specifications. Grout was delivered to the site in cement trucks and pumped to the injection points using a Schwing WP1000X concrete pump. The injection of grout into the injection points began at the bottom depth and proceeded upwards in 2-foot intervals. Grout injection at each interval continued until maximum grout pressure at the surface reached 300 pounds per square inch (psi), maximum grout quantity of 5 cubic yards per two-foot interval was achieved, maximum of one-inch of surface heave adjacent to the point was noted, or the total grout injected reached the maximum quantity of 50 cubic yards per injection point. Grout takes ranged from 0.2 cubic yards to 50.5 cubic yards. A total of 883.7 cubic yards of grout, delivered by 89 trucks, was injected. A summary of the compaction grout takes is provided in Table 1. Compaction grout logs for each location are provided in Attachment A.

Attachments or Enclosures:

Figure 1 - Compaction Grout Injection Points
Table 1 -Compaction Grout Injection Point Summary
Attachment A - Compaction Grout Logs

SFS/KBK/

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[1] Reference: Response to Request for Additional Information, Golder letter to FDEP dated April 30, 2010

g:\projects\083\083-82\083-82671\083-82671.6 (east sliver bid docs)\crec east sliver compaction grout tech memo.docx

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Jacksonville, FL 32256 USA
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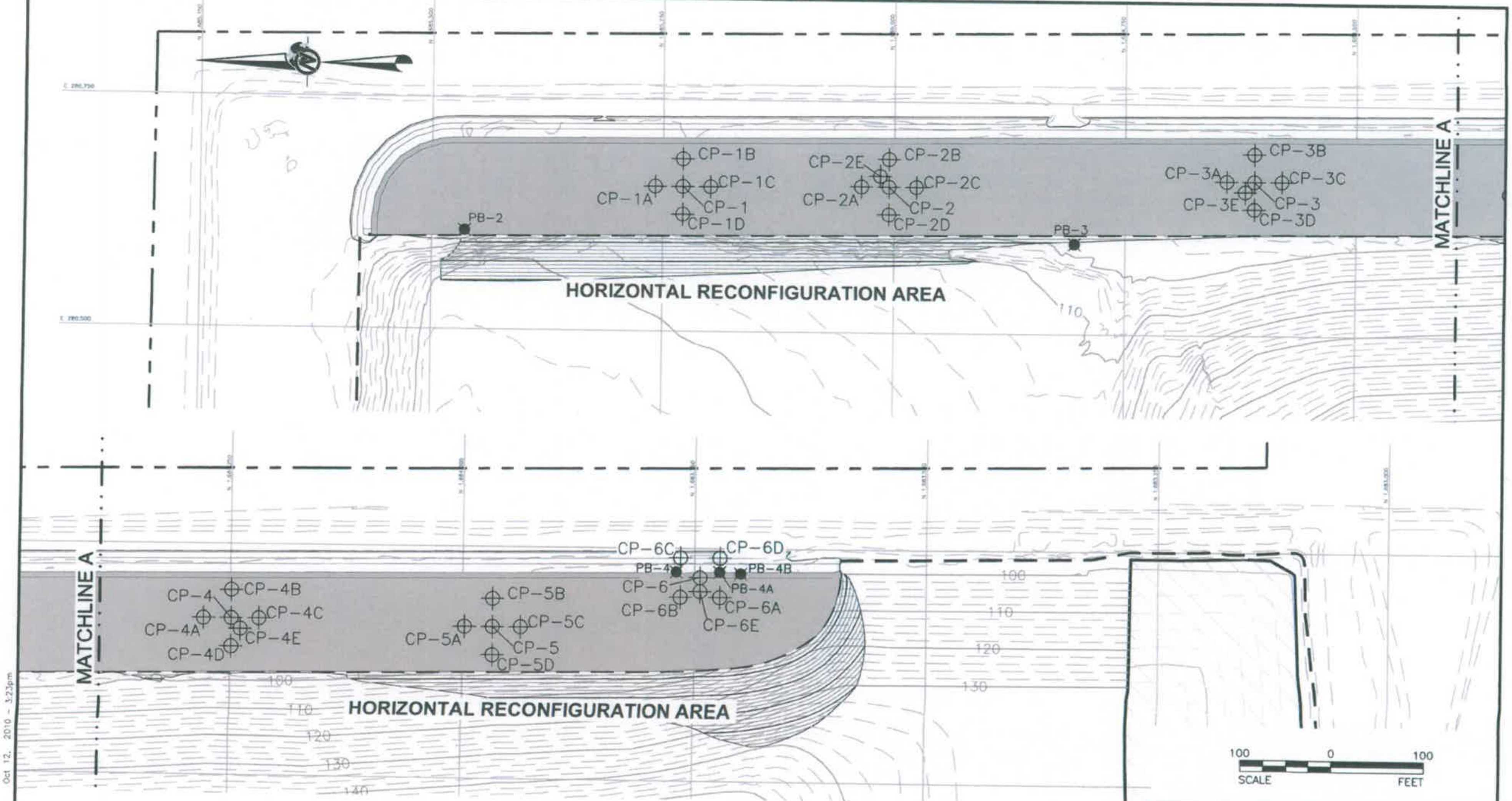
TABLE 1
COMPACTION GROUT INJECTION POINT SUMMARY

Crystal River Energy Complex
Ash Storage/Disposal Area
East Sliver Expansion

Location	Installation Date	Installation Depth (ft-bgs)	Dated Grouted	Pump Stroke Total	Grout Take (CY)
CP-1	7/26/2010	40	7/27/2010	938	29.4
CP-1A	7/26/2010	27	7/27/2010	408	12.8
CP-1B	7/26/2010	27	7/27/2010	6	0.2
CP-1C	7/26/2010	27	7/27/2010	8	0.3
CP-1D	7/26/2010	29	7/27/2010	9	0.3
CP-2	7/26/2010	26	7/27/2010	10	0.3
CP-2A	7/26/2010	41	7/27/2010	1417	44.4
CP-2B	7/26/2010	40	7/27/2010	1558	48.8
CP-2C	7/26/2010	26	7/27/2010	282	8.8
CP-2D	7/26/2010	28	7/27/2010	13	0.4
CP-2E	8/3/2010	26	8/5/2010	16	0.5
CP-3	7/27/2010	40	7/28/2010	1551	48.5
CP-3A	7/27/2010	37	7/28/2010	1268	39.7
CP-3B	7/27/2010	40	7/29/2010	722	22.6
CP-3C	7/27/2010	40	7/29/2010	1410	44.1
CP-3D	7/27/2010	40	7/28/2010	1252	39.2
CP-3E	8/3/2010	38	8/4/2010	1033	32.3
CP-4	7/27/2010	40	7/29/2010	941	29.5
CP-4A	7/27/2010	40	7/29/2010	1349	42.2
CP-4B	7/27/2010	40	7/29/2010	1151	36.0
CP-4C	7/27/2010	40	7/30/2010	261	8.2
CP-4D	7/27/2010	40	7/30/2010	1410	44.1
CP-4E	8/3/2010	36	8/4/2010	276	8.6
CP-5	7/27/2010	40	7/30/2010	1001	31.3
CP-5A	7/27/2010	40	7/30/2010	893	28.0
CP-5B	7/27/2010	40	7/30/2010	1615	50.5
CP-5C	7/27/2010	40	8/2/2010	16	0.5
CP-5D	7/27/2010	40	8/2/2010	1302	40.8
CP-6	7/27/2010	60	8/2/2010	1584	49.6
CP-6A	7/27/2010	55	8/3/2010	1579	49.4
CP-6B	7/27/2010	60	8/3/2010	1614	50.5
CP-6C	7/27/2010	40	8/3/2010	313	9.8
CP-6D	7/27/2010	40	8/2/2010	670	21.0
CP-6E	8/3/2010	60	8/4/2010	354	11.1

Checked by: SPB
Reviewed by: KBK

Golder Associates



REFERENCES

1.) CONTOURS BASED ON TOPOGRAPHIC SURVEY PRODUCED ON OCTOBER 13, 2009 BY SOUTHERN RESOURCE MAPPING, INC

LEGEND

CP-3A AS-BUILT COMPACTION GROUTING LOCATION (APPROXIMATE)

PB-3 SOIL BORING LOCATION

PROJECT
PROGRESS ENERGY FLORIDA
CRYSTAL RIVER ENERGY COMPLEX
CITRUS COUNTY, FLORIDA

TITLE
COMPACTION GROUT INJECTION POINTS
ASH STORAGE/DISPOSAL AREA
EAST SLIVER EXPANSION

Golder Associates
Jacksonville, Florida

PROJECT No.	083-82671.6		FILE No.	08382671-I014
DESIGN	SPS	10/08/10	SCALE AS SHOWN	REV. 0
CADD	BCL	10/12/10		
CHECK	SPS	12/1/10		
REVIEW	KBC	12/1/10		

FIGURE 1

Drawing file: 08382671-I014.dwg Oct 12, 2010 - 3:23pm

ATTACHMENT A
COMPACTION GROUT LOGS

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)	Injection Pt. No. CP-1 Pressure in Line (psi)	CP-1 Pump Strokes	CP-1 Volume (CY)	Injection Pt. No. CP-1A Pressure in Line (psi)	CP-1A Pump Strokes	CP-1A Volume (CY)	Injection Pt. No. CP-1B Pressure in Line (psi)	CP-1B Pump Strokes	CP-1B Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12	300+	1	0.03	300+	2	0.06			
12 to 14	300+	1	0.03	300+	9	0.28			
14 to 16	300+	1	0.03	300+	20	0.63			
16 to 18	300+	1	0.03	300+	68	2.13	300+	1	0.03
18 to 20	300+	1	0.03	300+	2	0.06	300+	1	0.03
20 to 22	300+	1	0.03	300+	2	0.06	300+	1	0.03
22 to 24	300+	2	0.06	300+	3	0.09	300+	1	0.03
24 to 26	300+	2	0.06	300+	151	4.73	300+	1	0.03
26 to 28	300+	2	0.06	300+	151	4.73	300+	1	0.03
28 to 30	300+	1	0.03						
30 to 32	300+	185	5.79						
32 to 34	300+	190	5.95						
34 to 36	300+	180	5.63						
36 to 38	300+	185	5.79						
38 to 40	300+	185	5.79						
Total		938	29.4		408	12.8		6	0.2

Depth Below Ground Surface (ft)	Injection Pt. No. CP-1C Pressure in Line (psi)	CP-1C Pump Strokes	CP-1C Volume (CY)	Injection Pt. No. CP-1D Pressure in Line (psi)	CP-1D Pump Strokes	CP-1D Volume (CY)
0 to 2						
2 to 4						
4 to 6						
6 to 8						
8 to 10						
10 to 12						
12 to 14	300+	1	0.03	300+	1	0.03
14 to 16	300+	1	0.03	300+	1	0.03
16 to 18	300+	1	0.03	300+	1	0.03
18 to 20	300+	1	0.03	300+	1	0.03
20 to 22	300+	1	0.03	300+	1	0.03
22 to 24	300+	1	0.03	300+	1	0.03
24 to 26	300+	1	0.03	300+	1	0.03
26 to 28	300+	1	0.03	300+	1	0.03
28 to 30				300+	1	0.03
30 to 32						
32 to 34						
34 to 36						
36 to 38						
38 to 40						
Total		8	0.3		9	0.3

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)	Injection Pt. No. CP-2			Injection Pt. No. CP-2A			Injection Pt. No. CP-2B		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12	300+	2	0.06						
12 to 14	300+	1	0.03						
14 to 16	300+	1	0.03				200	109	3.41
16 to 18	300+	1	0.03				200	100	3.13
18 to 20	300+	1	0.03				200	100	3.13
20 to 22	300+	2	0.06				200	157	4.91
22 to 24	300+	1	0.03				200	158	4.95
24 to 26	300+	1	0.03	200	157	4.91	200	107	3.35
26 to 28				200	160	5.01	200	106	3.32
28 to 30				200	170	5.32	200	106	3.32
30 to 32				200	150	4.70	200	153	4.79
32 to 34				200	173	5.41	200	154	4.82
34 to 36				200	145	4.54	200	104	3.26
36 to 38				200	148	4.63	200	102	3.19
38 to 40				200	158	4.95	150	102	3.19
40 to 42				200	156	4.88			
Total		10	0.3		1417	44.4		1558	48.8

Depth Below Ground Surface (ft)	Injection Pt. No. CP-2C			Injection Pt. No. CP-2D			Injection Pt. No. CP-2E		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10	300+	5	0.16						
10 to 12	200	260	8.14	300+	1	0.03	300+	2	0.06
12 to 14	300+	5	0.16	300+	2	0.06	300+	2	0.06
14 to 16	300+	2	0.06	300+	1	0.03	300+	2	0.06
16 to 18	300+	1	0.03	300+	1	0.03	300+	2	0.06
18 to 20	300+	4	0.13	300+	1	0.03	300+	2	0.06
20 to 22	300+	2	0.06	300+	1	0.03	300+	2	0.06
22 to 24	300+	1	0.03	300+	3	0.09	300+	2	0.06
24 to 26	300+	2	0.06	300+	1	0.03	300+	2	0.06
26 to 28				300+	2	0.06			
28 to 30									
30 to 32									
32 to 34									
34 to 36									
36 to 38									
38 to 40									
Total		282	8.8		13	0.4		16	0.5

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)	Injection Pt. No. CP-3			Injection Pt. No. CP-3A			Injection Pt. No. CP-3B		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12									
12 to 14									
14 to 16									
16 to 18	200	190	5.947						
18 to 20	175	123	3.85	200	20	0.63			
20 to 22	205	159	4.98	200	143	4.48	200	96	3.00
22 to 24	200	150	4.70	200	145	4.54	300+	3	0.09
24 to 26	200	79	2.47	200	110	3.44	300+	10	0.31
26 to 28	200	100	3.13	200	112	3.51	300+	10	0.31
28 to 30	200	130	4.07	200	110	3.44	300+	11	0.34
30 to 32	150	100	3.13	200	156	4.88	300+	10	0.31
32 to 34	150	100	3.13	200	158	4.95	300+	167	5.23
34 to 36	150	109	3.41	200	157	4.91	160	130	4.07
36 to 38	125	160	5.01	200	157	4.91	150	145	4.54
38 to 40	150	151	4.73				150	140	4.38
Total		1551	48.5		1268	39.7		722	22.6

Depth Below Ground Surface (ft)	Injection Pt. No. CP-3C			Injection Pt. No. CP-3D			Injection Pt. No. CP-3E		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12	300+	1	0.03						
12 to 14	300+	1	0.03						
14 to 16	300+	1	0.03				300+	1	0.03
16 to 18	300+	1	0.03				300+	8	0.25
18 to 20	200	1	0.03	300+	9	0.28	300+	17	0.53
20 to 22	200	174	5.45	150	100	3.13	300+	41	1.28
22 to 24	200	128	4.01	150	100	3.13	300+	3	0.09
24 to 26	200	153	4.79	200	125	3.91	300+	30	0.94
26 to 28	200	153	4.79	200	183	5.73	200	152	4.76
28 to 30	200	77	2.41	200	125	3.91	200	152	4.76
30 to 32	200	110	3.44	175	122	3.82	200	157	4.91
32 to 34	200	141	4.41	175	122	3.82	200	156	4.88
34 to 36	200	193	6.04	150	122	3.82	200	158	4.95
36 to 38	200	180	5.63	150	122	3.82	200	158	4.95
38 to 40	200	96	3.00	175	122	3.82			
Total		1410	44.1		1252	39.2		1033	32.3

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)	Injection Pt. No. CP-4			Injection Pt. No. CP-4A			Injection Pt. No. CP-4B		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12									
12 to 14									
14 to 16									
16 to 18									
18 to 20				200	144	4.51			
20 to 22				200	14	0.44	300+	7	0.22
22 to 24				200	53	1.66	300+	76	2.38
24 to 26				200	107	3.35	300+	160	5.01
26 to 28				200	124	3.88	300+	80	2.50
28 to 30	200	108	3.38	200	174	5.45	300+	149	4.66
30 to 32	200	165	5.16	200	144	4.51	300+	160	5.01
32 to 34	200	164	5.13	200	159	4.98	300+	150	4.70
34 to 36	200	164	5.13	200	130	4.07	300+	163	5.10
36 to 38	200	163	5.10	200	178	5.57	200	106	3.32
38 to 40	200	177	5.54	200	122	3.82	200	100	3.13
Total		941	29.5		1349	42.2		1151	36.0

Depth Below Ground Surface (ft)	Injection Pt. No. CP-4C			Injection Pt. No. CP-4D			Injection Pt. No. CP-4E		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12									
12 to 14									
14 to 16									
16 to 18				200	153	4.79			
18 to 20				200	165	5.16	200	95	2.97
20 to 22	200	97	3.04	200	152	4.76	150	150	4.70
22 to 24	200	50	1.57	200	102	3.19	200-300+	23	0.72
24 to 26	200	50	1.57	200	100	3.13	300+	1	0.03
26 to 28	200	51	1.60	200	100	3.13	300+	1	0.03
28 to 30	300+	3	0.09	200	110	3.44	300+	1	0.03
30 to 32	300+	2	0.06	200	105	3.29	300+	1	0.03
32 to 34	300+	2	0.06	200	110	3.44	300+	2	0.06
34 to 36	300+	2	0.06	200	100	3.13	300+	2	0.06
36 to 38	300+	1	0.03	200	123	3.85			
38 to 40	300+	3	0.09	200	90	2.817			
Total		261	8.2		1410	44.1		276	8.6

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)	Injection Pt. No. CP-5			Injection Pt. No. CP-5A			Injection Pt. No. CP-5B		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2									
2 to 4									
4 to 6									
6 to 8									
8 to 10									
10 to 12							250	35	1.10
12 to 14							250	94	2.94
14 to 16							250	64	2.00
16 to 18							250	64	2.00
18 to 20							250	64	2.00
20 to 22					6	0.19	300+	110	3.44
22 to 24	200	23	0.72		4	0.13	300+	99	3.10
24 to 26	200	113	3.54		112	3.51	200	99	3.10
26 to 28	200	113	3.54		112	3.51	200	169	5.29
28 to 30	200	112	3.51		106	3.32	200	165	5.16
30 to 32	200	111	3.47		118	3.69	200	112	3.51
32 to 34	200	107	3.35		114	3.57	200	112	3.51
34 to 36	200	115	3.60		110	3.44	200	113	3.54
36 to 38	200	154	4.82		105	3.29	200	157	4.91
38 to 40	200	153	4.79		106	3.32	200	158	4.95
Total		1001	31.3		893	28.0		1615	50.5

Depth Below Ground Surface (ft)	Injection Pt. No. CP-5C			Injection Pt. No. CP-5D		
	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0 to 2						
2 to 4						
4 to 6						
6 to 8						
8 to 10						
10 to 12	300+	1	0.03	300+	1	0.03
12 to 14	300+	1	0.03	300+	1	0.03
14 to 16	300+	1	0.03	300+	1	0.03
16 to 18	300+	1	0.03	300+	2	0.06
18 to 20	300+	1	0.03	300+	10	0.31
20 to 22	300+	1	0.03	200	160	5.01
22 to 24	300+	2	0.06	200	161	5.04
24 to 26	300+	1	0.03	200	105	3.29
26 to 28	300+	1	0.03	200	104	3.26
28 to 30	300+	1	0.03	200	104	3.26
30 to 32	300+	1	0.03	200	165	5.16
32 to 34	300+	1	0.03	200	164	5.13
34 to 36	300+	1	0.03	200	108	3.38
36 to 38	300+	1	0.03	200	108	3.38
38 to 40	300+	1	0.03	200	108	3.38
Total		16	0.5		1302	40.8

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)			Injection Pt. No. CP-6			Injection Pt. No. CP-6A			Injection Pt. No. CP-6B		
			Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0	to	2									
2	to	4									
4	to	6									
6	to	8									
8	to	10									
10	to	12	300+	2	0.06						
12	to	14	300+	2	0.06						
14	to	16	300+	2	0.06						
16	to	18	300+	2	0.06						
18	to	20	300+	2	0.06						
20	to	22	300+	4	0.13						
22	to	24	300+	4	0.13				300+	12	0.38
24	to	26	300+	1	0.03				300+	11	0.34
26	to	28	300+	2	0.06				300+	8	0.25
28	to	30	300+	3	0.09				300+	8	0.25
30	to	32	300+	1	0.03	300+	1	0.03	300+	7	0.22
32	to	34	300+	2	0.06	300+	1	0.03	200	167	5.23
34	to	36	300+	1	0.03	300+	1	0.03	200	150	4.70
36	to	38	150	160	5.01	300+	1	0.03	200	100	3.13
38	to	40	150	160	5.01	200	100	3.13	200	100	3.13
40	to	42	150	171	5.35	200	114	3.57	200	100	3.13
42	to	44	150	150	4.70	200	100	3.13	200	150	4.70
44	to	46	200	81	2.54	200	150	4.70	200	161	5.04
46	to	48	200	80	2.50	200	169	5.29	200	128	4.01
48	to	50	200	80	2.50	200	150	4.70	200	128	4.01
50	to	52	200	80	2.50	200	175	5.48	200	128	4.01
52	to	54	200	163	5.10	200	150	4.70	200	125	3.91
54	to	56	200	163	5.10	200	164	5.13	200	131	4.10
56	to	58	200	133	4.16	200	100	3.13			
58	to	60	200	135	4.23	200	203	6.35			
Total				1584	49.6		1579	49.4		1614	50.5

PROGRESS ENERGY FLORIDA

Crystal River Energy Complex - Ash Disposal Area East Sliver Grout Takes

Pumping Rate: 0.03 CY/pump stroke

Depth Below Ground Surface (ft)			Injection Pt. No. CP-6C			Injection Pt. No. CP-6D			Injection Pt. No. CP-6E		
			Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)	Pressure in Line (psi)	Pump Strokes	Volume (CY)
0	to	2									
2	to	4									
4	to	6									
6	to	8									
8	to	10									
10	to	12	300+	1	0.03	300+	11	0.34			
12	to	14	300+	1	0.03	300+	11	0.34			
14	to	16	300+	1	0.03	300+	4	0.13			
16	to	18	300+	1	0.03	300+	4	0.13	300+	2	0.06
18	to	20	300+	1	0.03	300+	4	0.13	300+	2	0.06
20	to	22	300+	2	0.06	300+	3	0.09	300+	2	0.06
22	to	24	300+	1	0.03	300+	3	0.09	300+	2	0.06
24	to	26	300+	2	0.06	300+	2	0.06	300+	2	0.06
26	to	28	300+	2	0.06	300+	1	0.03	300+	2	0.06
28	to	30	300+	7	0.22	300+	2	0.06	300+	2	0.06
30	to	32	200	160	5.01	200	105	3.29	300+	2	0.06
32	to	34	200	130	4.07	200	103	3.22	300+	2	0.06
34	to	36	300+	2	0.06	200	103	3.22	300+	2	0.06
36	to	38	300+	1	0.03	200	157	4.91	300+	4	0.13
38	to	40	300+	1	0.03	200	157	4.91	300+	3	0.09
40	to	42							300+	1	0.03
42	to	44							300+	2	0.06
44	to	46							300+	8	0.25
46	to	48							300+	6	0.19
48	to	50							200	100	3.13
50	to	52							200	141	4.41
52	to	54							300+	3	0.09
54	to	56							300+	11	0.34
56	to	58							300+	33	1.03
58	to	60							300+	22	0.69
Total				313	9.8		670	21.0		354	11.1

The Amy H Remley Foundation

Search

Citrus County

The following analysis is taken from:

Baker, A.E., Wood, A.R., and Cichon, J.R., 2008 (in review), The Citrus County Aquifer Vulnerability Assessment: Contract deliverable report prepared for the Florida Geological Survey - Florida Department of Environmental Protection.

THE CITRUS COUNTY AQUIFER VULNERABILITY ASSESSMENT

Part of Phase II of the Florida Aquifer Vulnerability Assessment (FAVA) Project, Florida Department of Environmental Protection Contract No. RM059, July, 2008.

Prepared for the Florida Department of Environmental Protection by Advanced GeoSpatial Inc.

THE CITRUS COUNTY AQUIFER VULNERABILITY ASSESSMENT

NOTE: In discussion with the principal author of the Analysis, it was confirmed that no consideration was given to the effect of fault lines or fracture sets which are prevalent in the area covered by the analysis. [But see Fracture Sets Example]

Interpretation of Results in Context of FAVA

Results of the CAVA project have allowed delineation of new and unique zones of relative vulnerability for the FAS in Citrus County, based on the county-specific model boundary used, use of numerous well points for aquifer confinement characterization, incorporation of most recent soils data, and application of recently-developed approaches for karst estimation in a GIS. These new results, though refined and highly detailed, do not replace results of previous studies. In other words, the FDEP's regional FAVA results (Arthur et al., 2005) for the FAS indicate that the Citrus County study area occurs in primarily a "more vulnerable" zone relative to other areas in Florida (Figure 17); as a result the new CAVA model output should be interpreted in the context of this major regional project. The new zones delineated in the CAVA project are unique to the CAVA study area, and reveal more detailed information regarding aquifer vulnerability within the regional "more vulnerable", and "vulnerable" zones identified in the FAVA project.

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Figure 17. Results of the Florida Aquifer Vulnerability Assessment project (Arthur et al., 2005) for the FAS in Citrus County. The CAVA model relative vulnerability zones, while based on more refined data than the FAVA project, occur within the context of this regional model.

Figure 1. Citrus County Aquifer Vulnerability Assessment project study area corresponds to the County's political boundary.

Study Area

The political boundary of Citrus County was used as the CAVA model study area extent (shown in Figure 1).

Soil Hydraulic Conductivity and Soil Pedality Themes

The rate that water moves through soil is a critical component of any aquifer vulnerability analysis, as soil is literally an aquifer system's first line of defense against potential contamination (Arthur et al., 2005). Two parameters of soils were evaluated for input into the CAVA model: *soil hydraulic conductivity*, which is the "amount of water that would move vertically through a unit area of saturated soil in unit time under unit hydraulic gradient" (U.S. Department of Agriculture, 2005); and *soil pedality*, which is calculated based on soil type, soil grade, and soil pedon size, and is a unitless parameter. Soil pedality is a relatively new concept used to estimate the hydrologic parameter of soil and is generated for CAVA using the pedality point method developed by Lin et al. (1999).

In 2006, Citrus County soils data were expanded for the study area and made available by the Natural Resources Conservation Service. This expansion included adaptation into ESRI geodatabase compatible format, and specific soils values were updated (U.S. Department of Agriculture, 2006). As a result, more detailed information is available for analysis for the CAVA project than during previous projects (e.g., Arthur et al., 2005). Countywide data sets representing soil hydraulic conductivity and soil pedality were developed for use as input into the CAVA model. This is completed for both hydraulic conductivity and soil pedality.

Weights calculated during sensitivity analysis for soil pedality were much stronger (i.e., had higher absolute value) than weights calculated for soil hydraulic conductivity. As a result, soil pedality was chosen as the better predictor of aquifer vulnerability because it shared the best association with training points.

Soil pedality, a unitless parameter, ranges from 0.0157 to 0.0533 across the study area. The analysis indicated that areas underlain by 0.0429 to 0.0533 were more associated with the training points, and therefore associated with higher aquifer vulnerability. Conversely, areas underlain by 0.0428 to 0.0157 were less associated with the training points, and therefore lower aquifer vulnerability. Based on this analysis, the evidential theme was generalized into two classes as displayed in Figure 10.

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Figure 10. Generalized soil pedality evidential theme; based on calculated weights analysis blue areas share a weaker association with training points and thereby relatively lower aquifer vulnerability, whereas red areas share a stronger association with training points.

Recharge Potential

In Copeland et al. (1991), the area of the Brooksville Ridge in central Florida is defined as having higher recharge potential than adjacent areas. The Brooksville Ridge is chiefly composed of Undifferentiated Hawthorn Group sediments which are poorly to moderately consolidated clayey sands and silty clays (Scott et al., 2001). In Citrus County, these sediments reach a maximum calculated thickness of 199 feet, can be discontinuous, deeply weathered and highly perforated by karst features.

In Citrus County these sediments are generally highly weathered, leaky, thin and intensely breached by karst features. These factors combine to increase the recharge potential to the FAS in the study area.

Recharge potential values were calculated for the study area by subtracting the USGS 2000 potentiometric surface of the FAS (USGS, 2000) from land surface elevation derived from USGS 7.5" quadrangles. Resulting recharge potential values range from -18 ft to greater than 150 ft (relative to mean sea level). Negative values generally correspond to areas where the aquifer is estimated to be discharging while higher positive values are restricted to the more substantial hills located on the Brooksville Ridge.

Categories of recharge potential were derived from records of the potentiometric surface. A preliminary weights of evidence indicated a very strong relationship between training points and recharge potential. Categories of recharge potential were ranked as displayed in Figure 5. Use of recharge potential via this approach is restricted to areas of Florida where the FAS is not well confined (e.g., this layer may not be usable in areas which are also underlain by thicker, contiguous Intermediate confining unit sediments), and where there is not a laterally contiguous Surficial aquifer system present.

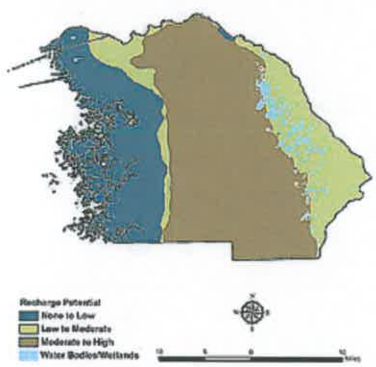

Figure 5. Recharge potential estimated from FAS potentiometric surface data, land surface elevation and estimates developed for Copeland et al., (1991).

Intermediate Confining Unit / Overburden Thickness Themes

Aquifer confinement – either in the form of overburden overlying the FAS, or the Intermediate confining unit (ICU) – is another critical layer in determining aquifer vulnerability. Where aquifer confinement is thick and the FAS is deeply buried, aquifer vulnerability is generally lower, whereas in areas of thin to absent confinement, the vulnerability of the FAS is generally higher.

To allow the calculation of aquifer confinement thickness in various study areas, surface models were developed using a dataset of borehole records supplemented with well gamma logs that contain descriptions of subsurface materials. AGI (the consultants) used these surfaces to calculate thickness of the ICU (Figure 6) and thickness of overburden overlying the FAS (Figure 7) in the study area. These two layers were tested for input in the model as described in *Sensitivity Analysis*.

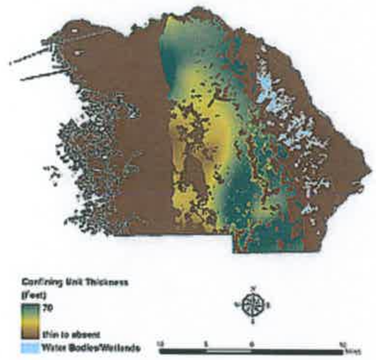

Figure 6. Thickness of the ICU calculated by subtracting predicted surface of ICU (Figure 6) from predicted surface of FAS as generated by FGS/FDEP.

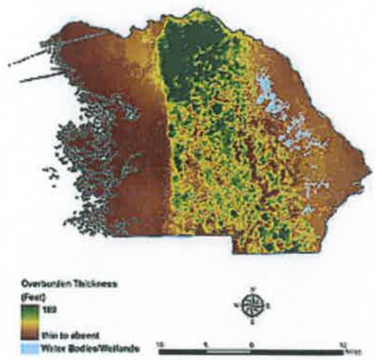

Figure 7. Thickness of sediments overlying the FAS calculated by subtracting digital elevation data from predicted surface of FAS as generated by FGS/FDEP.

Potential Karst Feature Theme

Karst features, or sinkholes and depressions, can provide preferential pathways for movement of surface water into the underlying aquifer system and enhance an area's aquifer vulnerability where present. The closer an area is to a karst feature, the more vulnerable it may be considered. Closed topographic depressions extracted from U.S. Geological Survey 7.5-minute quadrangle maps served as the initial dataset from which to estimate potential karst features in the study area. To supplement these data, the FGS/FDEP sinkhole database was included to identify karst features possibly not represented on USGS maps. These two data sources displayed in Figure 8 were combined and analyzed to develop a potential karst features evidential theme. See Figure 13 below.

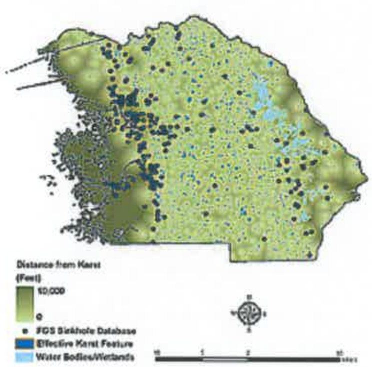

Figure 13. Potential karst features evidential theme buffered into 100-ft zones for proximity analysis in the weights of evidence analysis.

Potential Karst Features

As mentioned above, areas closer to a potential karst feature are normally associated with higher aquifer vulnerability. Based on this, features were buffered into 100-ft zones to allow for a proximity analysis (Figure 13). The analysis indicated that areas within 3,346 feet of a karst feature were more associated with the training points, and therefore with higher aquifer vulnerability. Conversely, areas greater than 3,346 feet from a karst feature were less associated with the training points, and therefore lower aquifer vulnerability. Based on this analysis, the evidential theme was generalized into two classes as displayed in Figure 14.

Figure 14. Generalized potential karst feature evidential theme; based on calculated weights analysis blue areas share a weaker association with training points and thereby relatively lower aquifer vulnerability, whereas red areas share a stronger association with training points.

Response Theme

Using evidential themes representing soil pedality, recharge potential, ICU thickness, and potential karst, weights of evidence was applied to generate a response theme, which is a GIS raster consisting of *posterior probability* values ranging from 0.0002697 to 0.0343457 across the study area (Figure 15). These probability values describe the relative probability that a unit area of the model will contain a training point - i.e., a point of aquifer vulnerability as defined above in *Training Points* - with respect to the prior probability value of 0.01043. Prior probability is the probability that a training

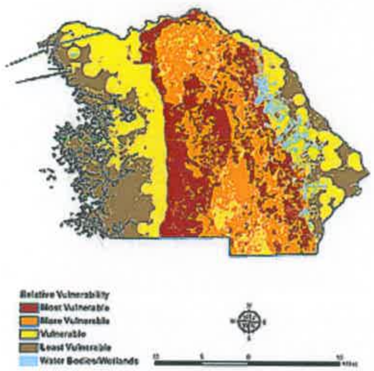

Figure 15. Relative vulnerability map for the Citrus County Aquifer Vulnerability Assessment project. Classes of vulnerability are based on calculated probabilities of a unit area containing a training point, or a monitor well with water quality sample results indicative of vulnerability.

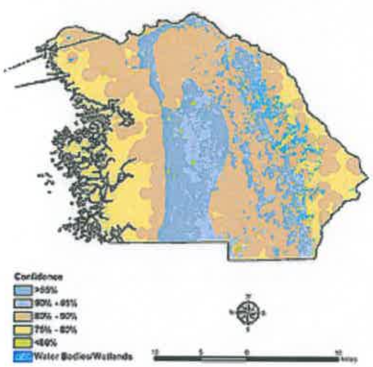

Figure 18. Confidence map for the CAVA model calculated by dividing the posterior probability values by the total uncertainty for each class to give an estimate of how well specific areas of the model are predicted.

Model Limitations and Scale of Use

When implementing the CAVA model results, it is vital to remember that aquifer systems in Florida are vulnerable to contamination; an invulnerable aquifer does not exist. Model results are based on features of the natural system that have significant association with the location of training points and thereby aquifer vulnerability. The CAVA project results provide a probability map that identifies zones of relative vulnerability in the study area based on input data; as a result the CAVA model output is an estimation of natural aquifer vulnerability and the results do not account for activities at land surface, contaminant type, groundwater flow paths or fate/transport of chemical constituents.

Derivative Products: Protection Zones

Relative vulnerability zones defined in this project may be applied to develop derivative maps, such as a protection-zone map (Cichon et al., 2005). Ideally, data layers not included as input in the aquifer vulnerability model would be considered to help in defining such protection zones and may include groundwater flow modeling, stream-sink features, induced drawdown areas from large well fields, and distribution of drainage wells. These layers, while important to aquifer vulnerability, do not form usable input into this aquifer vulnerability assessment project.

Confidence Map

As mentioned above, a confidence map of the model's posterior probability values can be calculated by dividing the posterior probability by its total uncertainty. This essentially applies an informal student T-test (as in Table 2) to the posterior probability values. The higher the confidence values, the greater the certainty is with regard to the posterior probability. This map essentially indicates the degree of confidence to which the posterior probabilities are meaningful and should be referenced when interpreting and implementing the model results. In other words, the confidence map should be used to help guide implementation of the vulnerability map as it reveals the confidence level associated with each vulnerability class (Mihalasky and Moyer, 2004).

Surface Water Areas

In addition to large surface-water bodies omitted from the analysis, there are many other surface-water features which were not removed. Many of these features may represent areas of groundwater discharge; however, these discharging surface waters are not part of the aquifer, although they originate from it. Accordingly, the CAVA model is not intended to be used to assess contamination potential of surface waters, though the discharging surface waters are highly vulnerable to contamination.

Recommendations on Scale of Use

Use of highly detailed evidential theme data as model input results in highly resolute model output as can be seen in the model response theme. These resolute features are reflections of real data used as input; however, the final maps should not be applied to very large scales such as to compare adjacent small parcels.

CAVA model output is, in a sense, as accurate as the most detailed input layer, and as inaccurate as the least detailed layer. For example, wells used to define confinement thickness represent an area up to 25 square miles (mi 2), the potentiometric surface map used in the development of the recharge potential evidential theme was mapped at 1:500,000, and soils polygonal data represent an area as small as 19,375 ft 2 .

Every raster cell of the model output coverage has significance per the model input as discussed above. However, it is important to note that aquifer vulnerability assessments are predictive models and no assumptions are made that all input layers are accurate, precise or complete at a single-raster cell scale. As mentioned above, the confidence map, because it is an indicator of the meaningfulness of the vulnerability classes, should be used to help guide implementation of the vulnerability map. For example, in the CAVA confidence map (Figure 18), local-scale land-use decisions might be more defensible in with the higher vulnerability classes (more vulnerable and most vulnerable) as these areas are associated with highest confidence values.

Ultimately, accuracy of the maps does not allow for evaluation of aquifer vulnerability at a specific parcel or site location. It is the responsibility of the end-users of the CAVA model output to determine specific and appropriate applications of these maps. In no instance should use of aquifer vulnerability assessment results substitute for a detailed, site-specific hydrogeological analysis.

CONCLUSION

As demands for fresh groundwater from the Floridan aquifer system underlying Citrus County increase resulting from continued population growth, identification of zones of relative vulnerability becomes an increasingly important tool for implementation of a successful groundwater protection and management program.

The results of the CAVA project provide a science-based, water-resource management tool allowing for a pro-active approach to protection of the FAS, and, as a result, have the potential to increase the value of protection efforts.

Model results will enable improved decisions to be made about aquifer vulnerability based on the input selected, including focused protection of sensitive areas such as springsheds and ground-water recharge areas.

The results of the CAVA vulnerability model are useful for development and implementation of groundwater protection measures; however, the vulnerability output map included in this report should not be viewed as a static evaluation of the vulnerability of the FAS. Because the assessments are based on snapshots of best-available data, the results are static representations; however, a benefit of this methodology is the flexibility to easily update the response themes as more refined or new data becomes available. In other words, As the scientific body of knowledge grows regarding hydrogeologic systems, this methodology allows the ongoing incorporation and update of datasets to modernize vulnerability assessments thereby enabling end users to better meet their objectives of protecting these sensitive resources. The weights of evidence modeling approach to aquifer vulnerability is a highly adaptable and useful tool for implementing ongoing protection of Florida's vulnerable groundwater resources.

NOTE: In discussion with the principal author of the Analysis, it was confirmed that no consideration was given to the effect of fault lines or fracture sets which are prevalent in the area covered by the analysis. [**See Fracture Sets Example**]

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TABLE 1

Summary of Monitoring Well Construction and Groundwater Elevation Data

Crystal River Energy Complex Ash Disposal Area
Crystal River, Florida

Well ID	Date Installed	Northing	Easting	Latitude	Longitude	TOC Elevation (ft.-NGVD)	Well Depth (ft.-bgs)	Screen Interval Depth (ft.-bgs)	Groundwater Elevations		
									10/23/2009 (ft-NGVD)	10/29/2009 (ft-NGVD)	1/19/2010 (ft-NGVD)
TW-1	10/19/09	1685576.11	279664.10	28°58'08.3"	82°41'20.6"	99.49	20.0	10.0 - 20.0	**92.68**	**92.30**	**92.91**
TW-2	10/20/09	1683788.14	280723.80	28°57'50.6"	82°41'08.5"	99.62	20.0	10.0 - 20.0	**92.60**	**92.27**	**93.07**
TW-3	10/19/09	1683437.14	279714.58	28°57'47.1"	82°41'19.9"	101.45	20.0	10.0 - 20.0	**92.27**	**92.33**	**93.02**
TW-4	10/20/09	1684871.64	279533.80	28°58'01.3"	82°41'22.0"	102.15	20.0	10.0 - 20.0	**92.20**	**92.45**	**92.99**
TW-5	12/21/09	1683984.79	279563.30	28°57'52.5"	82°41'21.6"	100.59	19.5	9.5 - 19.5	**NM**	**NM**	**92.75**
MWI-2R2	5/21/08*	1684481.25	279561.56	28°57'57.4"	82°41'21.7"	100.36	20.0	10.0 - 20.0	**92.43**	**92.55**	**93.10**
MWC-12R	--	1685340.17	280809.33	28°58'06.0"	82°41'07.7"	98.88	20*	10.0 - 20.0*	**92.61**	**92.28**	**92.90**
MWC-21R	--	1683294.81	278230.18	28°57'45.6"	82°41'36.6"	99.70	20*	10.0 - 20.0*	**NM**	**92.50**	**92.94**
MWB-30	7/9/07*	1682363.04	280971.22	28°57'36.5"	82°41'05.7"	97.85	18.67*	8.4 - 18.4*	**NM**	**92.18**	**92.97**

Notes:

Northings and Eastings reported in NAD27.

ft-bgs = feet below ground surface

TOC = top of casing (elevations reported in site vertical datum)

DTW = Depth to Water below top of casing

* = Information from previous Groundwater Monitoring Plans for CREC

NM = Not Measured

Checked by: KBK

Golder Associates

December 13, 2010

103-82558

Mr. Doug Yowell
Progress Energy Florida, Inc.
299 1st Avenue North, PEF-903
St. Petersburg, FL 33701

RE: NORTH PLANT AREA STORMWATER MODELING REPORT (REVISED)
CRYSTAL RIVER ENERGY COMPLEX
CRYSTAL RIVER, FLORIDA

Dear Mr. Yowell:

Golder Associates Inc. (Golder) is pleased to present the revised stormwater modeling results for the north plant area (a.k.a. Units 4 and 5) of the Crystal River Energy Complex (CREC) to Progress Energy Florida, Inc. (PEF). This report summarizes the analysis by Golder of the capacity of the current permitted stormwater system to manage additional impervious surfaces within Units 4 and 5 which comprise the north plant area of the CREC.

BACKGROUND INFORMATION

The existing stormwater management system for the north plant area of CREC was constructed in the late 1970s to treat/convey stormwater runoff for the 25-year, 24-hour design storm. The stormwater management system discharges to the Gulf of Mexico via NPDES Permit number FL0036366 - Runoff Collection Overflow outfall (Outfall D-0C0). A model for the stormwater management system was developed by Jacobs Engineering Inc. (Jacobs) in November 2006 and approved by the Florida Department of Environmental Protection (FDEP) as part of the modification of the Conditions of Certification (COC) to support the Crystal River Clean Air (CRCA) Project. Over the last four years, site construction activities related to the CRCA Project have triggered additional modeling exercises to demonstrate that the stormwater management system has sufficient capacity to handle a specific change in land use/ground cover due to proposed works within the stormwater management system basin. Stormwater permitting issues were discussed between PEF and FDEP Southwest District during a meeting held in January 2010. In order to reduce the number of future construction Environmental Resource Permit (ERP) reviews issued via COC amendments for individual projects involving land use/ground cover changes within the north plant area basin it was suggested by FDEP that PEF demonstrate via modeling that the entire north plant area stormwater system has sufficient capacity to handle additional impervious surface within the area. If the system was incapable of handling all impervious area, FDEP suggested that a "ledger" system could be created by FDEP that would allow the site to "bank" the maximum modeled acreage of impervious surface with subsequent future "debits" against the bank as new areas are developed within the basin in lieu of submitting an ERP modification. As a result of the meeting with FDEP, PEF requested Golder modify the stormwater model to include the entire north plant area as impervious surface.

STORMWATER MODEL SUMMARY

The stormwater model consists of eight basins that drain to a series of interconnected ditches, canals, and wet detention ponds. The stormwater model for the site was developed using the Interconnected Pond Routing (ICPR) program. A layout of the site and drainage basins is shown on Figure 1. Golder generally followed the same basin identification nomenclature as Jacobs' 2006 model for ease of review. The north coal yard (formerly basins 1B-i, 1B-ii, and 1B-iii) was included as basin 1B-i-iii. Runoff from basin 1B-i-iii accumulates in the hydraulically connected coal pile runoff ponds which are designed to detain a 25-year, 24-hour storm event and are equipped with emergency overflow structures for larger storm events. The north coal yard ponds and overflow structures have been included in the ICPR model.

Accumulated runoff from the coal pile runoff ponds is pumped to the coal pile runoff treatment facility (located in Area 1B-iv). The treatment facility discharges to two settling ponds for sediment retention before being pumped to the stormwater canal north of the coal yard. A settling pond discharge of 2.2 cubic feet per second (1,000 gallons per minute) has been included as base flow into basin 1B-iv. The basins, 1A, 1B-i-iii, 1B-iv, 1B-v, 1B-vi, and 1B-vii, were modeled as impervious surfaces with a runoff curve number (CN) of 98. A summary of the drainage basins modeled is shown below:

Basin Designation	Area (acres)	Runoff Curve Number	Time of Concentration
1A	94.8	98	27.0
1B-i-ii	59.1	98	7.5
1B-iv	16.7	98	7.7
1B-v	44.1	98	16.9
1B-vi	74.0	98	11.4
1B-vii	22.9	98	3.8

Time of concentration calculations were modified from Jacobs' 2006 model to reflect flow conditions over impervious surfaces using the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Technical Release Number 55 (TR-55) methodology. The time of concentration value for basin 1A was used from a previous stormwater design that was performed by Golder on March 17, 2010 as part of the horizontal reconfiguration of the ash disposal area. The time of concentration value for basin 1B-i-iii was calculated based on topographical features consistent with a coal pile for overland flow and a rip-rap lined swale for channel flow. This stormwater design and model verified that proposed modifications to the basins which make up basin 1A (f.k.a. basins 1A-iii, 1A-ii, and 1A-i) meet NPDES storage requirements and design high water elevations for 10-year, 24-hour and 25-year, 24-hour storm frequencies. Overall, the time of concentration values decreased in comparison to the 2006 Jacobs model. A minimum time of concentration value of 10 minutes was used for the IPCR Model for Basins 1B-iv and 1B-vii. Time of concentration calculations are shown in Table 1.

Limited stormwater surveys were performed in November 2009 (ash disposal area) and May 2010 (north plant area) by Harry W. Marlow, Inc. to verify conveyance structure details and pond cross-sections. The survey data was used to validate the model inputs parameters from the Jacob's model and revise them as necessary. Coal pile runoff pond stage area data and overflow structure dimensions were taken from design drawings by EPCR dated February 6, 2008. Model input data is provided in Appendix A.

STORMWATER MODEL RESULTS

Stormwater modeling was performed for the 10-year, 24-hour storm event and the 25-year, 24-hour storm event using ICPR and the results for peak flow rates and peak stages are shown in Table 2. These results and comparisons to the results reported for the 2006 Jacobs model indicated the following:

- The peak stages at each node were higher in comparison to the 2006 Jacobs model. However, none of the warning stages at any of the nodes in the ICPR model were exceeded and adequate freeboard is still available throughout the stormwater management system.
- The peak flow rates were higher in comparison to the 2006 Jacobs model with the exception of the outfall node into the Gulf of Mexico, Node TW-N1 (f.k.a. TW-N1POST).
  - The higher stages and peak flow rates can be attributed to incorporating a higher curve number (CN) for each basin.
  - The lower peak flow rate at node TW-N1 can be attributed to incorporating more detention storage into the model based on the cross-section data that was provided

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by Harry W. Marlow, Inc. and Southwest Florida Water Management District historical topographic data (dated February 1991).

CONCLUSION

In conclusion, the revised stormwater model results indicate that the stormwater management system is capable of managing the entire north plant area, defined by the drainage basins shown on Figure 1, as impervious surface.

CLOSING

Golder appreciates being given the opportunity to provide this report to PEF. Please feel free to contact the undersigned with any questions or comments.

Sincerely,

GOLDER ASSOCIATES INC.

Samuel F. Stafford
Project Engineer

Kenneth B. Karably, PE, ~~PG~~
Principal
12/13/2010

KENNETH B. KARABLY
LICENSE
No. 42886
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

SFS/BTH/kbk/ams

cc: George Hixon

Attachments:	Table 1	Time of Concentration Calculations
	Table 2	ICPR Modeling Results Summary
	Figure 1	Site Layout
	Appendix A	ICPR Modeling Inputs and Results

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TABLES

TABLE 1
TIME OF CONCENTRATION CALCULATIONS
CRYSTAL RIVER ENERGY COMPLEX - UNITS 4 AND 5

		Basin					
Sheet Flow	Units	1B	1B-IV	1B-V	1B-VI	1B-VII	1A
Surface Description	--	Gravel (Coal)	Paved	Paved	Paved	Paved	Refer to Appendix
Manning's Roughness Coefficient (n)	--	0.011	0.011	0.011	0.011	0.011	
Flow Length (L)	feet	50	300	300	300	100	
2-yr 24-hr Rainfall (P_2)	inches	5	5	5	5	5	
Land Slope (S)	feet/feet	0.1	0.002	0.002	0.002	0.013	
Travel Time (T_t)[1]	minutes	0.29	5.86	5.86	5.86	1.15	
		Basin					
Shallow Concentrated Flow	Units	1B	1B-IV	1B-V	1B-VI	1B-VII	1A
Surface Description	--	Gravel (Coal)	Paved	Paved	Paved	Paved	Refer to Appendix
Flow Length (L)	feet		150	930	300	226	
Land Slope (S)	feet/feet		0.002	0.002	0.002	0.0066	
Average Velocity (v)	feet/second		1.4	1.4	1.4	1.4	
Travel Time (T_t)[2]	minutes		1.79	11.07	3.57	2.69	
		Basin					
Channel Flow	Units	1B	1B-IV	1B-V	1B-VI	1B-VII	1A
Surface Description	--	Rip-Rap	Paved	Paved	Paved	Paved	
Cross Sectional Area	square feet	24			30		Refer to Appendix
Wetted Perimeter (P)	feet	19	0	0	24	0	
Hydraulic Radius (H_r)	feet	1.29	0	0	1.25	0	
Channel Slope (S)	feet/feet	0.003	0	0	0.002	0	
Manning's Roughness Coefficient (n)	--	0.035	0	0	0.011	0	
Velocity (v)[3]	feet/second	2.76	0	0	7.03	0	
Flow Length (L)	feet	1195	0	0	840.00	0	
Travel Time (T_t)	minutes	7.22	0	0	1.99	0	
Time of Concentration[4]	minutes	**7.51**	**7.65**	**16.94**	**11.43**	**3.84**	**27.00**

[1]Natural Resource Conservation Service (NRCS) (TR-55) Equation was used to determine sheet flow travel time
[2]NRCS Average Velocity Chart (Paved Area) was used to determine shallow concentrated flow travel time
[3]Chezy-Manning equation was used to determine channel flow velocity
[4]A minimum time of concentration value of 10 minutes was used for Interconnected Pond Routing (ICPR) Model

Created by: BTH
Checked by: SFS
Reviewed by: KBK

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TABLE 2
ICPR STORMWATER MODELING RESULTS SUMMARY
CRYSTAL RIVER ENERGY COMPLEX - UNITS 4 AND 5

Outfall Name	Outfall NPDES Designation	Outfall ICPR Model Designation	Contributing Area (acres)	10-yr 24-hr Storm		25-yr 24-hr Storm	
				Peak Discharge (cfs)	Peak Stage (ft)	Peak Discharge (cfs)	Peak Stage (ft)
Ash Disposal Area Weir	I-C40	N1A - N1B2	94.8	0.0	6.7	12.0	7.0
Coal Pile Ditch Weir	I-CHO	N1B-DITCH - N1B2	119.9	54.3	7.3	83.1	7.4
Overall North Plant Weir	D-OCO	N1B2 - N1	311.6	0.0	6.7	32.3	8.1

Notes:

Contributing Area for does not include coal pile basins which are managed separately

NPDES Designations from FL0036366

cfs = cubic feet per second

ft = elevation in feet above mean sea level (NGVD29)

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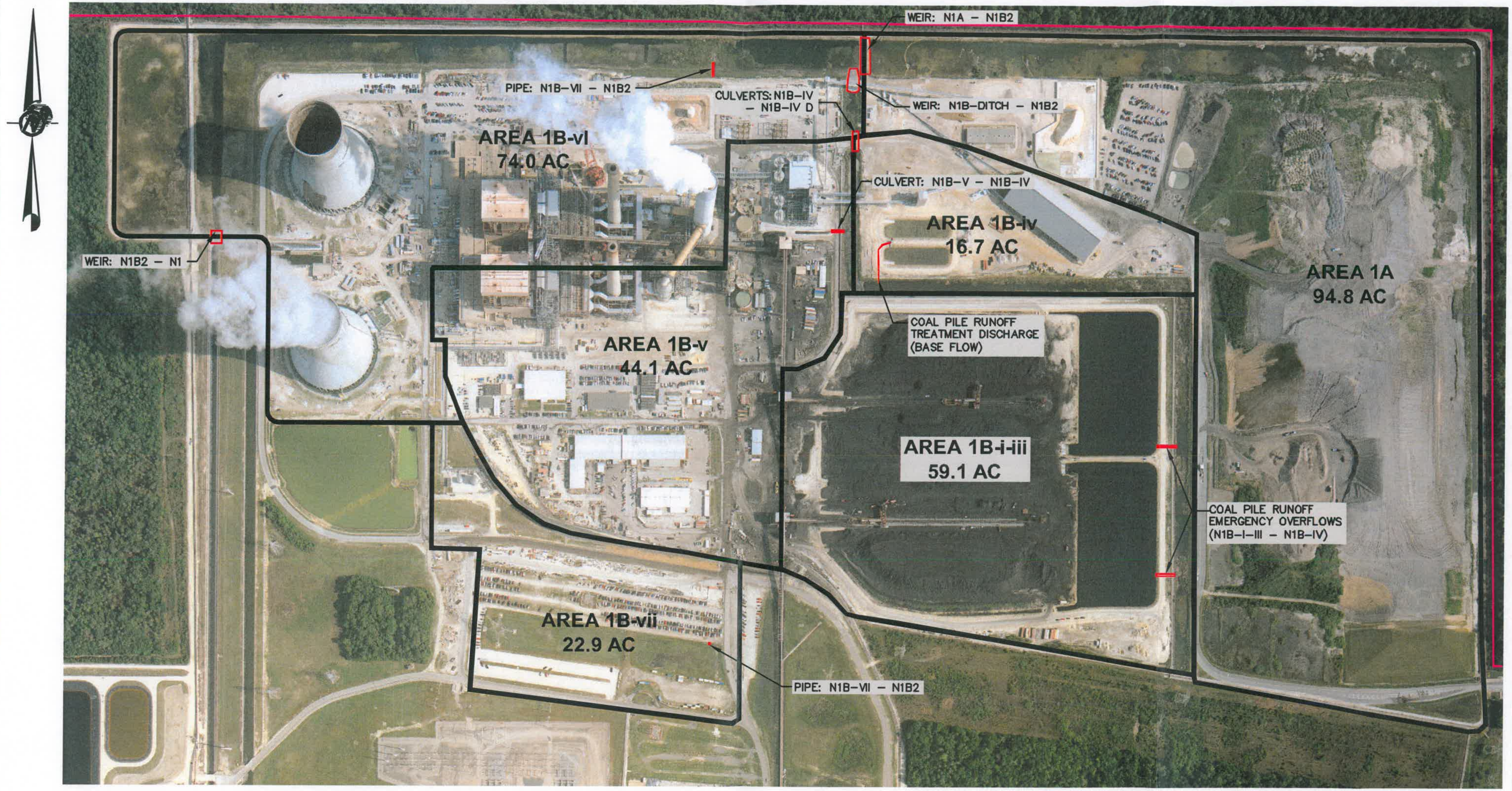
Checked By: BTH
Reviewed By: KBK

FIGURES



REFERENCES

1.) AERIAL PHOTOGRAPH DATED APRIL 20, 2010 FROM SOUTHERN RESOURCE MAPPING, INC.

2.) DRAINAGE BASINS BASED ON MODEL BY JACOBS, DRAWING NUMBER 2830-C-106, REVISION 4, DATED 11/30/2007.

400 0 400
SCALE FEET

PROJECT	PROGRESS ENERGY CRYSTAL RIVER ENERGY COMPLEX CITRUS COUNTY, FLORIDA		
TITLE	SITE LAYOUT		
PROJECT No. 103-82558	FILE No. 10382558A001R1		
DESIGN SFS 06/04/10	SCALE AS SHOWN	REV. 0	
CADD BCL 12/10/10			
CHECK SFS 12/13/10	FIGURE 1		
REVIEW KBK 12/13/10			

Golder Associates
Jacksonville, Florida

Drawing file: 10382558A001r1.dwg Dec 13, 2010 - 11:26am

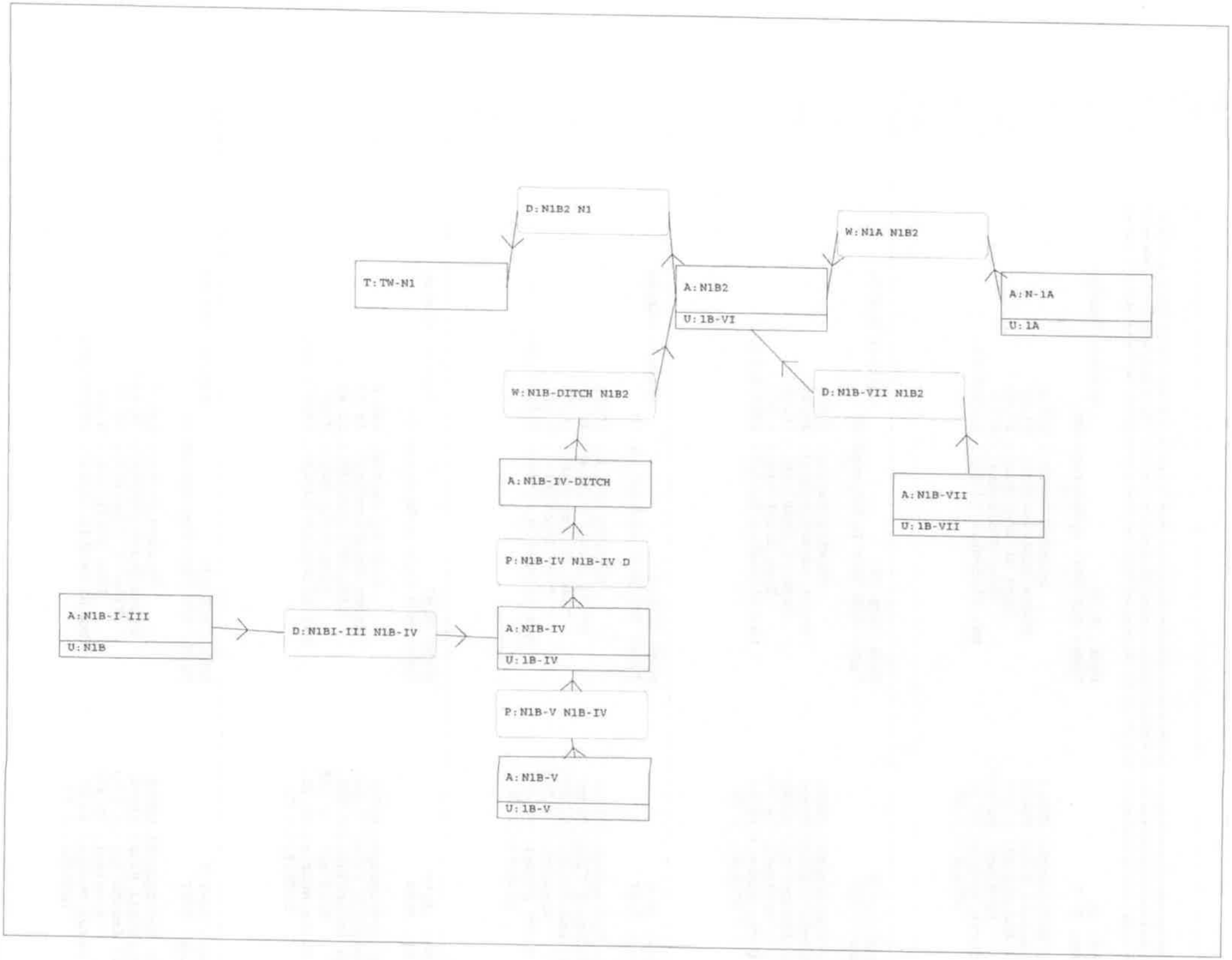
APPENDIX A

ICPR MODELING INPUTS AND RESULTS

Nodes
A Stage/Area
V Stage/Volume
T Time/Stage
M Manhole

Basins
O Overland Flow
U SCS Unit CN
S SBUH CN
Y SCS Unit GA
Z SBUH GA

Links
P Pipe
W Weir
C Channel
D Drop Structure
B Bridge
R Rating Curve
H Breach
E Percolation
F Filter
X Exfil Trench



```
==========================================================================================
==== Basins ==============================================================================
==========================================================================================

       Name: 1A                         Node: N-1A                   Status: Onsite
      Group: POST                       Type: SCS Unit Hydrograph CN

     Unit Hydrograph: Uh256                    Peaking Factor: 256.0
       Rainfall File: Flmod               Storm Duration(hrs): 24.00
 Rainfall Amount(in): 0.000                 Time of Conc(min): 27.00
            Area(ac): 94.800                  Time Shift(hrs): 0.00
        Curve Number: 98.00             Max Allowable Q(cfs): 999999.000
             DCIA(%): 0.00

------------------------------------------------------------------------------------------
       Name: 1B-IV                      Node: NIB-IV                 Status: Onsite
      Group: POST                       Type: SCS Unit Hydrograph CN

     Unit Hydrograph: Uh256                    Peaking Factor: 256.0
       Rainfall File: Flmod               Storm Duration(hrs): 24.00
 Rainfall Amount(in): 0.000                 Time of Conc(min): 10.00
            Area(ac): 16.700                  Time Shift(hrs): 0.00
        Curve Number: 98.00             Max Allowable Q(cfs): 999999.000
             DCIA(%): 0.00

------------------------------------------------------------------------------------------
       Name: 1B-V                       Node: N1B-V                  Status: Onsite
      Group: POST                       Type: SCS Unit Hydrograph CN

     Unit Hydrograph: Uh256                    Peaking Factor: 256.0
       Rainfall File: Flmod               Storm Duration(hrs): 24.00
 Rainfall Amount(in): 0.000                 Time of Conc(min): 16.90
            Area(ac): 44.140                  Time Shift(hrs): 0.00
        Curve Number: 98.00             Max Allowable Q(cfs): 999999.000
             DCIA(%): 0.00

------------------------------------------------------------------------------------------
       Name: 1B-VI                      Node: N1B2                   Status: Onsite
      Group: POST                       Type: SCS Unit Hydrograph CN

     Unit Hydrograph: Uh256                    Peaking Factor: 256.0
       Rainfall File: Flmod               Storm Duration(hrs): 24.00
 Rainfall Amount(in): 0.000                 Time of Conc(min): 11.40
            Area(ac): 74.000                  Time Shift(hrs): 0.00
        Curve Number: 98.00             Max Allowable Q(cfs): 999999.000
             DCIA(%): 0.00

------------------------------------------------------------------------------------------
       Name: 1B-VII                     Node: N1B-VII                Status: Onsite
      Group: POST                       Type: SCS Unit Hydrograph CN

     Unit Hydrograph: Uh256                    Peaking Factor: 256.0
       Rainfall File: Flmod               Storm Duration(hrs): 24.00
 Rainfall Amount(in): 0.000                 Time of Conc(min): 10.00
            Area(ac): 22.900                  Time Shift(hrs): 0.00
        Curve Number: 98.00             Max Allowable Q(cfs): 999999.000
             DCIA(%): 0.00

------------------------------------------------------------------------------------------
       Name: N1B                        Node: N1B-I-III              Status: Onsite
      Group: POST                       Type: SCS Unit Hydrograph CN

     Unit Hydrograph: Uh256                    Peaking Factor: 256.0
       Rainfall File: Flmod               Storm Duration(hrs): 24.00
 Rainfall Amount(in): 0.000                 Time of Conc(min): 10.00
```

Area(ac): 59.100
Curve Number: 98.00
DCIA(%): 0.00

Time Shift(hrs): 0.00
Max Allowable Q(cfs): 999999.000

==== Nodes ====

Name: N-1A
Group: POST
Type: Stage/Area

Base Flow(cfs): 0.000

Init Stage(ft): 2.700
Warn Stage(ft): 7.700

Stage(ft)	Area(ac)
2.700	14.4800
3.700	15.5000
4.700	16.4900
5.700	17.4800
6.500	18.2300
6.700	18.4600
7.700	19.4500

Name: N1B-I-III
Group: POST
Type: Stage/Area

Base Flow(cfs): 0.000

Init Stage(ft): 3.700
Warn Stage(ft): 10.700

Stage(ft)	Area(ac)
3.700	8.8000
4.700	9.1000
5.700	9.4000
6.700	9.7000
7.700	10.0000
8.700	10.4000
9.700	10.7000

Name: N1B-IV-DITCH
Group: POST
Type: Stage/Area

Base Flow(cfs): 0.000

Init Stage(ft): 3.530
Warn Stage(ft): 8.000

Stage(ft)	Area(ac)
4.000	0.2000
8.000	0.3000

Name: N1B-V
Group: POST
Type: Stage/Area

Base Flow(cfs): 0.000

Init Stage(ft): 4.000
Warn Stage(ft): 10.000

Stage(ft)	Area(ac)
4.000	0.9000
8.000	1.5400

Name: N1B-VII
Group: POST
Type: Stage/Area

Base Flow(cfs): 0.300

Init Stage(ft): 3.000
Warn Stage(ft): 10.000

Stage(ft) Area(ac)

3.000	0.2300
4.000	1.1500
5.000	2.2900
6.000	2.9800

Name: N1B2 Base Flow(cfs): 0.000 Init Stage(ft): 3.000
Group: POST Warn Stage(ft): 9.000
Type: Stage/Area

Stage(ft)	Area(ac)
3.000	10.0000
4.000	12.0000
5.000	14.0000
6.000	16.0000
7.000	18.0000
8.000	20.0000
9.000	22.0000

Name: NIB-IV Base Flow(cfs): 2.200 Init Stage(ft): 3.000
Group: POST Warn Stage(ft): 10.000
Type: Stage/Area

Stage(ft)	Area(ac)
3.000	3.2060
4.000	3.9400
5.000	4.4030
6.000	4.8690
7.000	5.3400
8.000	5.8150

Name: TW-N1 Base Flow(cfs): 0.000 Init Stage(ft): 5.600
Group: POST Warn Stage(ft): 5.600
Type: Time/Stage

Time(hrs)	Stage(ft)
0.00	5.600
24.00	5.600

==== Pipes ====

Name: N1B-IV N1B-IV D From Node: NIB-IV Length(ft): 81.10
Group: POST To Node: N1B-IV-DITCH Count: 4
Friction Equation: Automatic
Solution Algorithm: Most Restrictive
Flow: Both
Entrance Loss Coef: 0.50
Exit Loss Coef: 1.00
Bend Loss Coef: 0.00
Outlet Ctrl Spec: Use dc or tw
Inlet Ctrl Spec: Use dn
Stabilizer Option: None

	UPSTREAM	DOWNSTREAM
Geometry:	Circular	Circular
Span(in):	36.00	36.00
Rise(in):	36.00	36.00
Invert(ft):	4.110	3.530
Manning's N:	0.013000	0.013000
Top Clip(in):	0.000	0.000
Bot Clip(in):	0.000	0.000

Upstream FHWA Inlet Edge Description:
Circular CMP: Mitered to slope

Downstream FHWA Inlet Edge Description:
Circular CMP: Mitered to slope

```
------------------------------------------------------------------------------------------------------
        Name: N1B-V N1B-IV          From Node: N1B-V              Length(ft): 50.00
       Group: POST                    To Node: NIB-IV                  Count: 1
                                                          Friction Equation: Average Conveyance
              UPSTREAM       DOWNSTREAM                  Solution Algorithm: Automatic
    Geometry: Rectangular    Rectangular                               Flow: Both
    Span(in): 60.00          60.00                      Entrance Loss Coef: 0.50
    Rise(in): 28.00          28.00                          Exit Loss Coef: 1.00
  Invert(ft): 4.000          3.500                          Bend Loss Coef: 0.00
 Manning's N: 0.013000       0.013000                     Outlet Ctrl Spec: Use dc or tw
Top Clip(in): 0.000          0.000                         Inlet Ctrl Spec: Use dn
Bot Clip(in): 0.000          0.000                       Stabilizer Option: None


Upstream FHWA Inlet Edge Description:
Rectangular Box: 30° to 75° wingwall flares

Downstream FHWA Inlet Edge Description:
Rectangular Box: 30° to 75° wingwall flares


=========================================================================================
==== Drop Structures ====================================================================
=========================================================================================

        Name: N1B-VII N1B2          From Node: N1B-VII            Length(ft): 2430.00
       Group: POST                    To Node: N1B2                    Count: 1

              UPSTREAM       DOWNSTREAM                   Friction Equation: Automatic
    Geometry: Circular       Circular                    Solution Algorithm: Most Restrictive
    Span(in): 18.00          18.00                                     Flow: Both
    Rise(in): 18.00          18.00                      Entrance Loss Coef: 0.500
  Invert(ft): 5.590          3.000                          Exit Loss Coef: 1.000
 Manning's N: 0.013000       0.013000                     Outlet Ctrl Spec: Use dc or tw
Top Clip(in): 0.000          0.000                         Inlet Ctrl Spec: Use dc
Bot Clip(in): 0.000          0.000                          Solution Incs: 10

Upstream FHWA Inlet Edge Description:
Circular Concrete: Square edge w/ headwall

Downstream FHWA Inlet Edge Description:
Circular Concrete: Square edge w/ headwall


*** Weir 1 of 1 for Drop Structure N1B-VII N1B2 ***
                                                                          TABLE
            Count: 1                      Bottom Clip(in): 0.000
             Type: Horizontal                Top Clip(in): 0.000
             Flow: Both                    Weir Disc Coef: 3.200
         Geometry: Rectangular          Orifice Disc Coef: 0.600

         Span(in): 50.40                      Invert(ft): 7.710
         Rise(in): 50.40                Control Elev(ft): 7.710

------------------------------------------------------------------------------------------------------
        Name: N1B2 N1               From Node: N1B2               Length(ft): 34.35
       Group: POST                    To Node: TW-N1                   Count: 3

              UPSTREAM       DOWNSTREAM                   Friction Equation: Automatic
    Geometry: Rectangular    Rectangular                 Solution Algorithm: Most Restrictive
    Span(in): 120.00         120.00                                    Flow: Both
    Rise(in): 24.00          24.00                      Entrance Loss Coef: 0.500
  Invert(ft): 5.060          4.720                          Exit Loss Coef: 1.000
 Manning's N: 0.013000       0.013000                     Outlet Ctrl Spec: Use dc or tw
Top Clip(in): 0.000          0.000                         Inlet Ctrl Spec: Use dc
Bot Clip(in): 0.000          0.000                          Solution Incs: 10

Upstream FHWA Inlet Edge Description:
Rectangular Box: 0° wingwall flares

Downstream FHWA Inlet Edge Description:
```

```
Rectangular Box: 0° wingwall flares


*** Weir 1 of 1 for Drop Structure N1B2 N1 ***
                                                                        TABLE
             Count: 1                   Bottom Clip(in): 0.000
              Type: Horizontal             Top Clip(in): 0.000
              Flow: Both                 Weir Disc Coef: 3.200
          Geometry: Rectangular       Orifice Disc Coef: 0.600

          Span(in): 466.20                   Invert(ft): 6.800
          Rise(in): 126.80             Control Elev(ft): 6.800


----------------------------------------------------------------------------------------
        Name: N1BI-III N1B-IV     From Node: N1B-I-III         Length(ft): 75.00
       Group: POST                  To Node: NIB-IV                 Count: 2

              UPSTREAM         DOWNSTREAM                Friction Equation: Automatic
    Geometry: Circular         Circular                 Solution Algorithm: Most Restrictive
    Span(in): 24.00            24.00                                  Flow: Both
    Rise(in): 24.00            24.00                    Entrance Loss Coef: 0.500
  Invert(ft): 5.700            4.700                        Exit Loss Coef: 1.000
 Manning's N: 0.013000         0.013000                  Outlet Ctrl Spec: Use dc or tw
Top Clip(in): 0.000            0.000                      Inlet Ctrl Spec: Use dc
Bot Clip(in): 0.000            0.000                       Solution Incs: 10

Upstream FHWA Inlet Edge Description:
Circular Concrete: Square edge w/ headwall

Downstream FHWA Inlet Edge Description:
Circular Concrete: Groove end projecting


*** Weir 1 of 1 for Drop Structure N1BI-III N1B-IV ***
                                                                        TABLE
             Count: 2                   Bottom Clip(in): 0.000
              Type: Horizontal             Top Clip(in): 0.000
              Flow: Both                 Weir Disc Coef: 3.200
          Geometry: Rectangular       Orifice Disc Coef: 0.600

          Span(in): 60.00                    Invert(ft): 9.700
          Rise(in): 60.00              Control Elev(ft): 9.700


=========================================================================================
==== Weirs ==============================================================================
=========================================================================================

        Name: N1A N1B2             From Node: N-1A
       Group: POST                   To Node: N1B2
        Flow: Both                     Count: 1
        Type: Vertical: Fread       Geometry: Trapezoidal

         Bottom Width(ft): 108.00
     Left Side Slope(h/v): 10.00
    Right Side Slope(h/v): 10.00
               Invert(ft): 6.460
     Control Elevation(ft): 6.460
    Struct Opening Dim(ft): 9999.00
                                                 TABLE
           Bottom Clip(ft): 0.000
              Top Clip(ft): 0.000
       Weir Discharge Coef: 2.600
    Orifice Discharge Coef: 0.600


----------------------------------------------------------------------------------------
        Name: N1B-DITCH N1B2       From Node: N1B-IV-DITCH
       Group: POST                   To Node: N1B2
        Flow: Both                     Count: 1
        Type: Vertical: Fread       Geometry: Trapezoidal
```

```
        Bottom Width(ft): 50.00
    Left Side Slope(h/v): 10.00
   Right Side Slope(h/v): 10.00
             Invert(ft): 6.810
   Control Elevation(ft): 6.810
  Struct Opening Dim(ft): 9999.00
                                                TABLE
         Bottom Clip(ft): 0.000
            Top Clip(ft): 0.000
     Weir Discharge Coef: 3.200
  Orifice Discharge Coef: 0.600


==========================================================================================
==== Hydrology Simulations ===============================================================
==========================================================================================

      Name: 100YR-24-HR
  Filename: C:\Documents and Settings\bholcomb\Desktop\PROGRESS ENERGY - UNITS 4-5\100YR-24-HR.R32

    Override Defaults: Yes
  Storm Duration(hrs): 24.00
       Rainfall File: Flmod
  Rainfall Amount(in): 11.50

Time(hrs)       Print Inc(min)
--------------- --------------
30.000          5.00

-------------------------------------------------------------------------------------------
      Name: 10YR-24-HR
  Filename: C:\Documents and Settings\bholcomb\Desktop\PROGRESS ENERGY - UNITS 4-5\10YR-24-HR.R32

    Override Defaults: Yes
  Storm Duration(hrs): 24.00
       Rainfall File: Flmod
  Rainfall Amount(in): 7.00

Time(hrs)       Print Inc(min)
--------------- --------------
30.000          5.00

-------------------------------------------------------------------------------------------
      Name: 25YR-24-HR
  Filename: C:\Documents and Settings\bholcomb\Desktop\PROGRESS ENERGY - UNITS 4-5\25YR-24-HR.R32

    Override Defaults: Yes
  Storm Duration(hrs): 24.00
       Rainfall File: Flmod
  Rainfall Amount(in): 8.60

Time(hrs)       Print Inc(min)
--------------- --------------
30.000          5.00

==========================================================================================
==== Routing Simulations =================================================================
==========================================================================================

      Name: 100YR-24-HR        Hydrology Sim: 100YR-24-HR
  Filename: C:\Documents and Settings\bholcomb\Desktop\PROGRESS ENERGY - UNITS 4-5\100YR-24-HR.I32

     Execute: Yes         Restart: No           Patch: No
 Alternative: No

     Max Delta Z(ft): 1.00                      Delta Z Factor: 0.00500
  Time Step Optimizer: 10.000
      Start Time(hrs): 0.000                     End Time(hrs): 30.00
   Min Calc Time(sec): 0.5000               Max Calc Time(sec): 5.0000
      Boundary Stages:                           Boundary Flows:


Time(hrs)       Print Inc(min)
--------------- --------------
```

```
30.000          5.000

Group           Run
--------------- -----
POST            Yes

----------------------------------------------------------------------------------------------
        Name: 10YR-24-HR          Hydrology Sim: 10YR-24-HR
    Filename: C:\Documents and Settings\bholcomb\Desktop\PROGRESS ENERGY - UNITS 4-5\10YR-24-HR.I32

     Execute: Yes        Restart: No            Patch: No
 Alternative: No

       Max Delta Z(ft): 1.00                    Delta Z Factor: 0.00500
   Time Step Optimizer: 10.000
       Start Time(hrs): 0.000                    End Time(hrs): 30.00
     Min Calc Time(sec): 0.5000               Max Calc Time(sec): 5.0000
       Boundary Stages:                         Boundary Flows:

Time(hrs)       Print Inc(min)
--------------- ---------------
30.000          5.000

Group           Run
--------------- -----
POST            Yes

----------------------------------------------------------------------------------------------
        Name: 25YR-24-HR          Hydrology Sim: 25YR-24-HR
    Filename: C:\Documents and Settings\bholcomb\Desktop\PROGRESS ENERGY - UNITS 4-5\25YR-24-HR.I32

     Execute: Yes        Restart: No            Patch: No
 Alternative: No

       Max Delta Z(ft): 1.00                    Delta Z Factor: 0.00500
   Time Step Optimizer: 10.000
       Start Time(hrs): 0.000                    End Time(hrs): 30.00
     Min Calc Time(sec): 0.5000               Max Calc Time(sec): 5.0000
       Boundary Stages:                         Boundary Flows:

Time(hrs)       Print Inc(min)
--------------- ---------------
30.000          5.000

Group           Run
--------------- -----
POST            Yes
```

Name	Group	Simulation	Max Time Stage hrs	Max Stage ft	Warning Stage ft	Max Delta Stage ft	Max Surf Area ft2	Max Time Inflow hrs	Max Inflow cfs	Max Time Outflow hrs	Max Outflow cfs
N-1A	POST	10YR-24-HR	30.00	6.69	7.70	0.0017	803648	12.25	254.37	0.00	0.00
N1B-I-III	POST	10YR-24-HR	25.00	7.27	10.70	0.0022	429929	12.00	241.85	0.00	0.00
N1B-IV-DITCH	POST	10YR-24-HR	13.36	7.27	8.00	0.0042	12302	13.34	54.28	13.36	54.27
N1B-V	POST	10YR-24-HR	12.69	8.27	10.00	0.0050	68941	12.08	148.93	12.37	80.89
N1B-VII	POST	10YR-24-HR	22.80	7.90	10.00	0.0027	187039	12.00	94.01	23.78	2.35
N1B2	POST	10YR-24-HR	30.00	6.69	9.00	0.0019	757230	12.08	290.16	0.00	0.00
NIB-IV	POST	10YR-24-HR	13.35	7.39	10.00	0.0027	240779	12.08	139.55	13.34	54.28
TW-N1	POST	10YR-24-HR	0.00	5.60	5.60	0.0000	0	0.00	0.00	0.00	0.00
N-1A	POST	25YR-24-HR	22.61	7.02	7.70	0.0020	818091	12.25	312.92	24.32	12.01
N1B-I-III	POST	25YR-24-HR	25.00	8.05	10.70	0.0023	441762	12.00	297.49	0.00	0.00
N1B-IV-DITCH	POST	25YR-24-HR	13.24	7.42	8.00	0.0045	12458	13.23	83.09	13.24	83.08
N1B-V	POST	25YR-24-HR	12.68	9.28	10.00	0.0050	76031	12.08	183.19	12.49	93.53
N1B-VII	POST	25YR-24-HR	23.73	8.41	10.00	0.0027	202107	12.00	115.57	12.96	2.76
N1B2	POST	25YR-24-HR	22.57	7.02	9.00	0.0019	785677	12.08	356.91	22.57	32.26
NIB-IV	POST	25YR-24-HR	13.23	7.70	10.00	0.0022	247076	12.08	161.38	13.23	83.09
TW-N1	POST	25YR-24-HR	0.00	5.60	5.60	0.0000	0	22.57	32.26	0.00	0.00

Name	Group	Simulation	Max Time Flow hrs	Max Flow cfs	Max Delta Q cfs	Max Time US Stage hrs	Max US Stage ft	Max Time DS Stage hrs	Max DS Stage ft
N1A N1B2	POST	10YR-24-HR	0.00	0.00	0.047	30.00	6.69	30.00	6.69
N1B-DITCH N1B2	POST	10YR-24-HR	13.36	54.27	0.179	13.36	7.27	30.00	6.69
N1B-IV N1B-IV D	POST	10YR-24-HR	13.34	54.28	-0.193	13.35	7.39	13.36	7.27
N1B-V N1B-IV	POST	10YR-24-HR	12.37	80.89	10.220	12.69	8.27	13.35	7.39
N1B-VII N1B2	POST	10YR-24-HR	23.78	2.35	0.002	22.80	7.90	30.00	6.69
N1B2 N1	POST	10YR-24-HR	0.00	0.00	0.000	30.00	6.69	0.00	5.60
N1BI-III N1B-IV	POST	10YR-24-HR	0.00	0.00	0.000	25.00	7.27	13.35	7.39
N1A N1B2	POST	25YR-24-HR	24.32	12.01	0.218	22.61	7.02	22.57	7.02
N1B-DITCH N1B2	POST	25YR-24-HR	13.24	83.08	0.234	13.24	7.42	22.57	7.02
N1B-IV N1B-IV D	POST	25YR-24-HR	13.23	83.09	0.211	13.23	7.70	13.24	7.42
N1B-V N1B-IV	POST	25YR-24-HR	12.49	93.53	-0.935	12.68	9.28	13.23	7.70
N1B-VII N1B2	POST	25YR-24-HR	12.96	2.76	0.017	23.73	8.41	22.57	7.02
N1B2 N1	POST	25YR-24-HR	22.57	32.26	0.019	22.57	7.02	0.00	5.60
N1BI-III N1B-IV	POST	25YR-24-HR	0.00	0.00	0.000	25.00	8.05	13.23	7.70

Time of Concentration - TR-55 Worksheet

Project ID: **PEF CREC ASH DISPOSAL AREA HORIZONTAL RECONFIGURATION**

Project #: **083-82671.1**

TRAVEL TIME PARAMETERS

Sheet Flow		Rim Ditch		Northwest Ditch		West Ditch	
P =	5	n =	0.03	n =	0.03	n =	0.03
s =	0.02	A =	1.7	A =	8.52	A =	7.38
n =	0.24	WP =	5.1	WP =	12.07	WP =	11.44
		Rh =	0.33	Rh =	0.71	Rh =	0.65
		s =	0.005	s =	0.005	s =	0.005
		v =	1.69	v =	2.78	v =	2.62

	TRAVEL DISTANCES (FT)				TRAVEL TIMES (HOURS)					
ID	Top of BSA	Rim Ditch	Northwest Ditch	West Ditch	Sheet Flow (Top of BSA)	Rim Ditch Flow	Northwest Ditch	West Ditch	**TOC (hr)**	**TOC (min)**
T1	310	225	920	0	0.470	0.037	0.092	0.000	**0.599**	**36**
T2	230	20	0	0	0.370	0.003	0.000	0.000	**0.374**	**22**
T3	230	240	0	0	0.370	0.039	0.000	0.000	**0.410**	**25**
T4	230	210	0	0	0.370	0.035	0.000	0.000	**0.405**	**24**
T5	210	210	0	0	0.344	0.035	0.000	0.000	**0.379**	**23**
T6	310	160	0	0	0.470	0.026	0.000	0.000	**0.497**	**30**
T7	200	170	0	0	0.331	0.028	0.000	0.000	**0.359**	**22**
T8	220	230	0	270	0.358	0.038	0.027	0.029	**0.451**	**27**
T9	230	190	0	710	0.370	0.031	0.071	0.075	**0.548**	**33**

AVERAGE TIME OF CONCENTRATION 27

Notes

P = rainfall for 2-yr, 24-hour storm (inches)

s = watercourse slope (ft/ft)

n = Manning's roughness coefficient

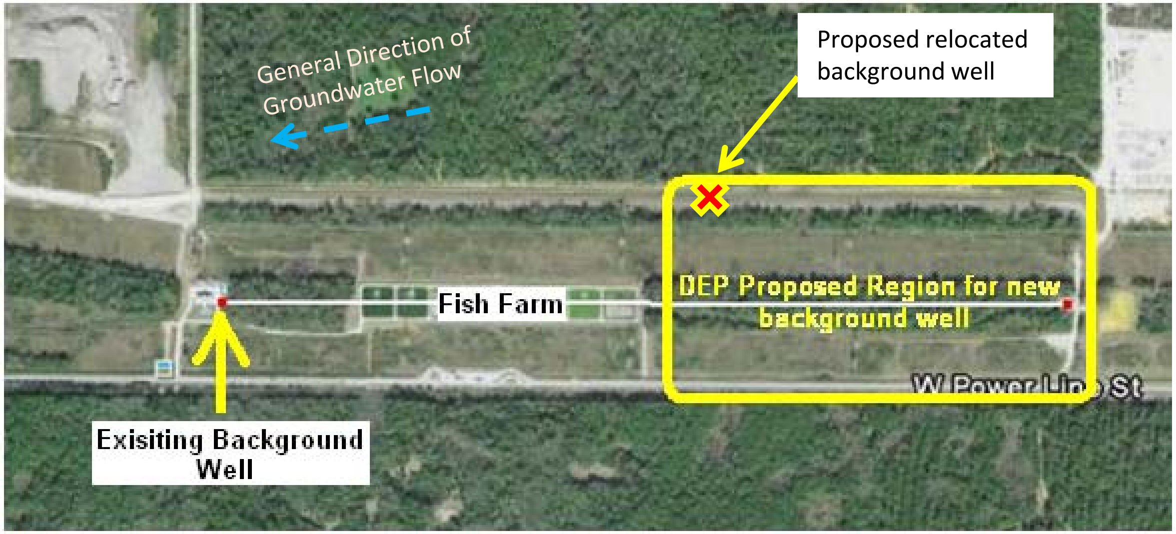
v = velocity (ft/sec) [from chart for shallow concentration flow, calculated in channel travel]

A = Area (ft2) - see Attachment 11

WP = Wet Perimeter (ft) - see Attachment 11

Rh = Hydraulic Radius (ft) [Area/Wet Perimeter]

Attachment 10



Attachment 11

Geosyntec consultants

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10 November 2010

Mr. Ron Johnson
Sr. Environmental Specialist
Progress Energy Crystal River Nuclear Projects & Construction
15760 West Power Line Street
Crystal River, Florida 34428-6708

Re: Responses to FDEP Requests for Additional Information and Addendum to the Report "Supplemental Hydrogeological Investigation and Groundwater Modeling Evaluation for the IWW Pond Area, July 2010"
Petition to Modify Site Certification PA 77-09
Crystal River Energy Complex
Crystal River, Florida 34428

Dear Mr. Johnson:

Geosyntec Consultants (Geosyntec) has prepared the following responses to the Florida Department of Environmental Protection's (FDEP's) requests for additional information regarding the "Supplemental Hydrogeological Investigation and Groundwater Modeling Evaluation for the IWW Pond Area, July 2010" (Geosyntec) prepared for the Crystal River Energy Complex (Site). These requests for additional information were submitted by Mr. Michael Halpin in letters dated 11 October 2010 (comment from Industrial Wastewater Program) and 25 October 2010 (comments from Ground Water Regulatory Section).

RESPONSE TO FDEP COMMENT – 11 October 2010 (Comment B.3. from Industrial Wastewater Program)

For clarity, FDEP's comments are presented below in italics and responses are presented in regular font.

FDEP Comment No. B.3. *Increase permitted flow to the South Percolation Pond System*

Page three of the Supplemental Hydrogeologic Investigation and Ground Water Modeling evaluation for the IWW pond area, section 3.2, indicates that Pond 4 has an approximate surface area of 7.2 acres. However, page eleven, section 7.1, indicates that Pond 4 has a 7.5 acre surface area. Please clarify and make corrections accordingly.

Geosyntec Response to Comment No. B.3. Pond 4 has a surface area that increases with increasing elevation. Following are approximate Pond 4 surface areas based on elevation in feet (ft) above mean sea level (amsl).

Elevation (ft amsl)	Approximate Surface Area (acres)
5	6.5
6	6.7
7	6.9
8	7.1
9	7.2
10	7.4
11	7.5
12	7.7
13	7.8
14	8.0
Top of Bank (approximately 23 ft amsl)	9.0

The noted 7.2 and 7.5 acre surface areas correspond to typical stage elevations of 9 and 11 ft amsl, respectively.

To be conservative, the storage and infiltration capacity of the other three IWW ponds were not included in the model. The cumulative area of these ponds to top of bank is greater than 5 acres.

RESPONSE TO FDEP COMMENTS – 25 October 2010 (Comments from Ground Water Regulatory Section)

FDEP Comment No. 1. Please provide a MODFLOW simulation cell grid map with a greater focus on the pond area. Report Figure 6 scale is simply too small to provide the level of detail needed in evaluating the model grid.

Geosyntec Response to Comment No. 1. Exhibit 1 consists of the requested figure showing grid discretization near the pond area. The horizontal grid spacing is 10 ft by 10 ft in the pond area. The maximum grid spacing within the entire domain is 100 ft by 100 ft.

FDEP Comment No. 2. Please provide an in-depth discussion of how the MODFLOW "Lake Package" attributes provide a suitable simulation of the IWW Ponds discharge.

Geosyntec consultants

Geosyntec Response to Comment No. 2. To account for the change in surface water head elevation in the ponds due to the added discharge and flux exchange with the surrounding aquifer, the "Lake Package" in MODFLOW was employed. The lake package is better suited than the "Reservoir Package" because the simulated heads in the computational cells can rise and fall due to interactions with the surrounding aquifer. The lake boundary cells exchange water with the surrounding aquifer at a rate determined by the relative head difference and by conductances that are based on grid cell dimensions (representing material thickness), hydraulic conductivities of the aquifer material, and user-specified leakance distributions that represent the resistance to flow through the material of the lakebed.

FDEP Comment No. 3. Variables in the MODFLOW "Evapotranspiration Package" (EP) cannot be estimated to a reasonable level of site specific reliability. The EP used in the MODFLOW simulation requires the modeler to choose an "evapotranspiration surface", where loss of water from evapotranspiration is maximized, and an "evapotranspiration extinction depth", below which evapotranspiration ceases. It is GWR's position that these hydrological parameters cannot be defined with a level of accuracy to be applied to a conservative modeling effort. The modeling should be performed without the use of the EP.

Geosyntec Response to Comment No. 3. Predictive simulations were performed without the use of the EP, which removed evapotranspiration (ET) from the model. Removing ET as a boundary condition did not significantly change the predicted amount of water able to be infiltrated in IWW Pond 4 because the originally specified ET rate was low (i.e. conservative). Below is information from Table 7 from the originally submitted report, with the last column added to present the model-predicted highest IWW pond elevation without ET applied to the model. The minimum approximate top of bank for IWW Pond 4 is at 23 ft amsl.

Predictive Simulation Results (From Table 7)				
Simulation #	Simulation Name	Description	Result – Highest Lake Elevation in ft amsl (**with ET**)	Result – Highest Lake Elevation in ft amsl (**without ET**)
1	076_MGD	present day permitted discharge	5.8	5.9
2	076_100	present day permitted discharge + 100 yr storm	6.2	6.3
3	091_MGD	proposed permitted discharge	6.7	6.8
4	091_100	proposed day permitted discharge + 100 yr storm	7.2	7.3
5	Max_MGD	maximum discharge: 1.27 MGD	9.0	9.1
6	Max_100	maximum discharge with 100 yr storm: 1.22 MGD	9.0	9.1

FDEP Comment No. 4. The Report will require a more thorough description of the model calibration process with a figure depicting the calibrated water table contours superimposed over the build out topography of the pond system. How was the model layer 2 horizontal conductivity (Kh) of 400 feet/day derived? There are no on site test data reporting a Kh of 400 feet/day.

Geosyntec Response to Comment No. 4. The model was manually calibrated regionally (steady-state) and then locally (transient).

The regional steady-state calibration used observed heads from nine observation wells. Site-specific data were used when available to specify parameters. Typical or published parameter values were specified if site-specific data were not available. The horizontal hydraulic conductivity for layer 1 of the model was assigned as estimated from slug tests conducted at the Site. The specified hydraulic conductivity for model layer 2 was calibrated by manually adjusting the hydraulic conductivity within the appropriate range of values (250 - 1000 ft/day) obtained from literature applicable to the general vicinity of the Site (Figure 27; Miller, 1986) while matching groundwater elevation data (heads) obtained from the nine observation wells. Exhibit 2, Figures 1 and 2 present calibration results for the steady-state calibration simulation. Exhibit 2, Figures 3 and 4 show predicted steady-state water table contours and the build out topography, respectively.

During the transient simulation that focused on the Site, the hydraulic conductivity and thickness specified for the lake boundary condition cells used to simulate the ponds was calibrated by manually adjusting the parameter values to obtain a satisfactory match between simulated and observed pond elevation data (from Pond 4) obtained at the Site. Exhibit 3 shows the calibration results for the transient simulation. During performance of the transient calibration, the reported hydraulic conductivity and pond bottom thickness were considered and care was taken to keep specified parameters realistic while calibrating the transient simulation.

Reference: Miller, James A., Hydrogeologic Framework of the Floridan Aquifer System in Florida and in Parts of Georgia, Alabama, and South Carolina", United States Geological Survey Professional Paper 1403-B, 1986.

FDEP Comment No. 5. Provide MODFLOW pond stress simulation time in days.

Geosyntec Response to Comment No. 5. For the numerical simulations discussed in the report, the stress periods were set to 365 days for the steady-state stress periods and 1 day for the transient stress periods. The simulation time for steady-state stress periods does not affect model results since temporal effects are not considered in steady-state simulations.

Geosyntec consultants

<u>FDEP Comment No. 6</u>. A post simulation water budget/balance should be performed and submitted accounting for water that enters, leaves, or is retained as storage in the model. The error factor in the water budget should be as near to zero as possible for a properly run simulation.

<u>Geosyntec Response to Comment No. 6</u>. The water budget analysis was conducted for the calibrated model and the errors in water budget were less than 0.1 % for both the steady-state and transient calibrations. The water budget was similarly low for the predictive simulations. A detailed water budget is provided below for both the steady state and transient calibration simulations which are representative of the water budget for the predictive simulations.

Water Budget Analysis for Calibration Simulations (water flux in cubic feet per day)

Boundary Condition	Inflow/Outflow for the Boundary Condition	Calibration Simulation	
		Steady-State	Transient
Constant Head	Inflow	0.0	9725.1
	Outflow	166839.9	272076.4
General Head Boundary	Inflow	100937.2	179927.5
	Outflow	631.1	751.6
Recharge	Inflow	43120.6	43230.6
	Outflow	0.0	0.0
Lake	Inflow	37028.9	48699.3
	Outflow	555.8	495.7
Storage	Inflow	N/A	12494.4
	Outflow	N/A	9305.9
Evapotranspiration	Inflow	0.0	0.0
	Outflow	13138.0	11459.3
Percent Error		-0.04 %	-0.004%

Geosyntec consultants

CLOSURE

If you have any questions regarding this response letter, please do not hesitate to contact either of the undersigned.

Sincerely,

John Spencer

John Spencer
Senior Staff Hydrogeologist

10 November 2010

Chad Drummond, P.E.
Associate
Florida Professional Engineer No. 64378
Geosyntec Consultants, Inc.
Certificate of Authorization No. 4321

Attachments: Exhibits 1 - 3

FW1829

EXHIBITS

EXHIBIT 1

Finite-Difference Grid near Pond 4

Exhibit 1: Finite-difference grid used for the model, superimposed on the base map. The computational grid (in the horizontal direction) is 10 ft by 10 ft in the general vicinity of the IWW ponds.

EXHIBIT 2

Steady-State Calibration Results

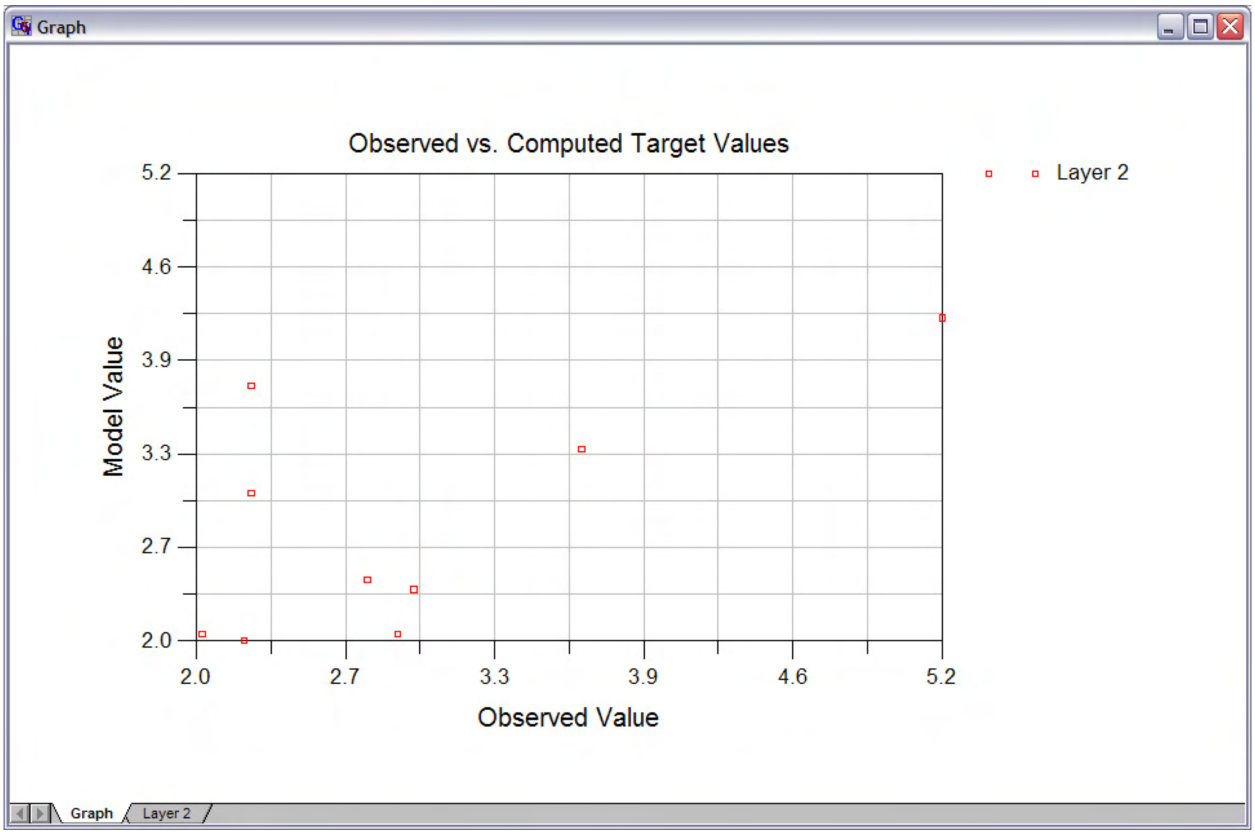


Exhibit 2, Figure 1: Calibration results for steady-state simulation showing observed vs. simulated heads in Layer 2 of the model. Axes scales are enhanced to increase readability.

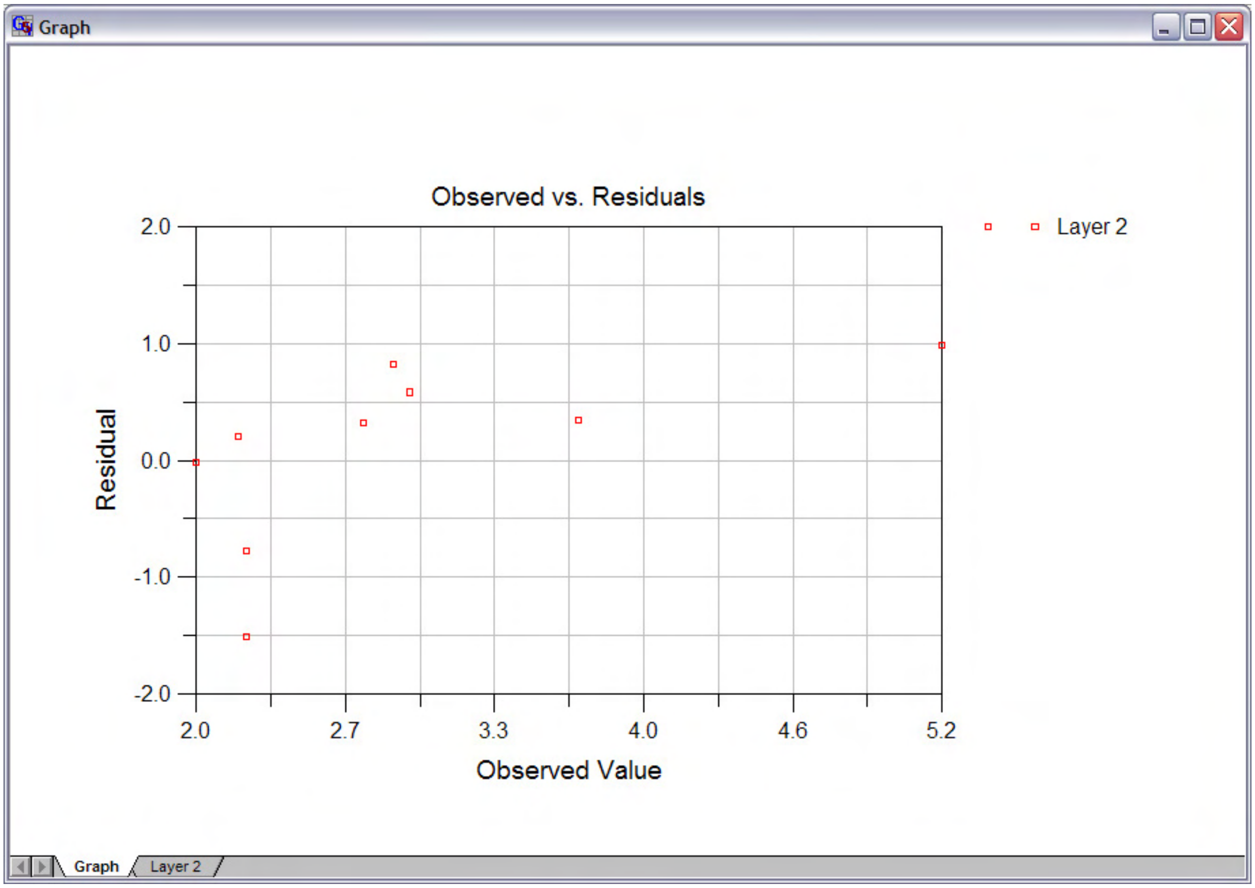


Exhibit 2, Figure 2: Calibration results for steady-state simulation showing observed vs. residual heads in Layer 2 of the model.

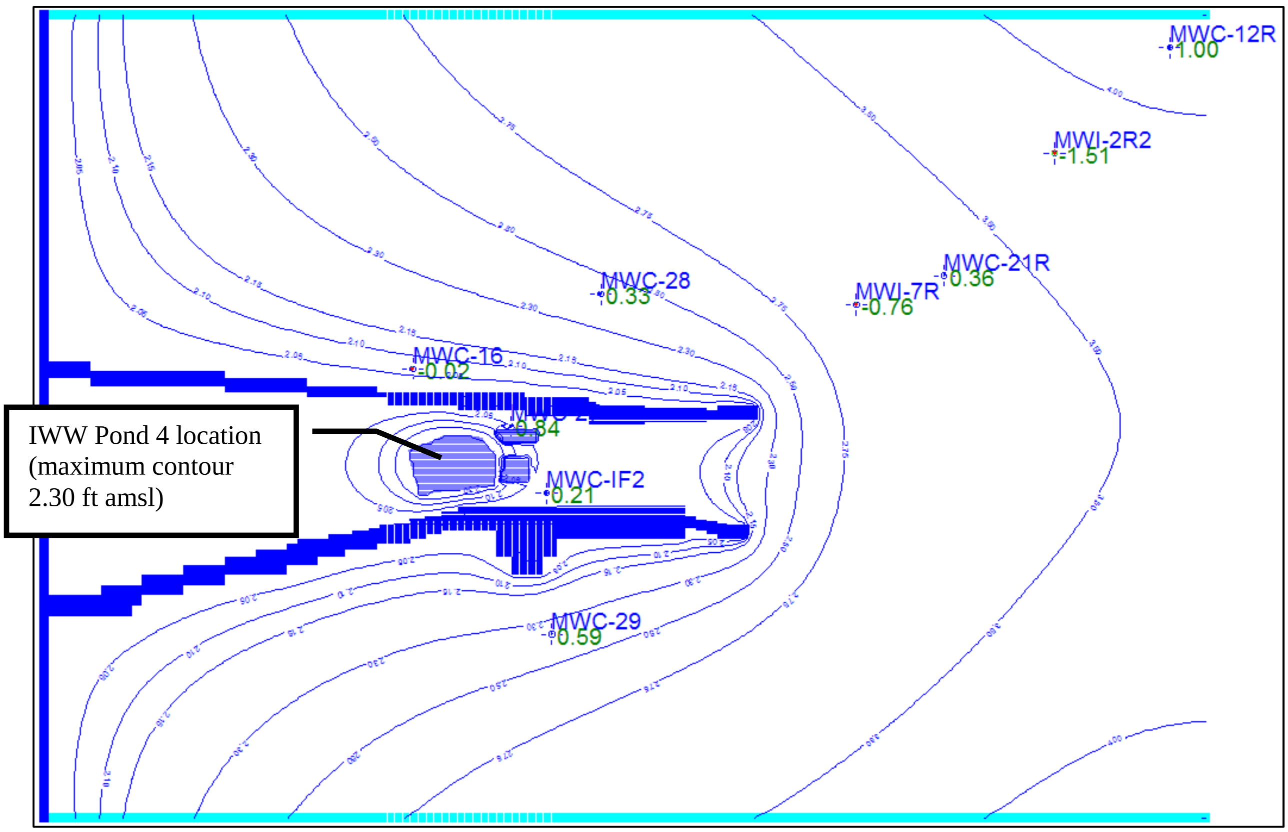


Exhibit 2, Figure 3: Calibration results for steady-state simulation showing predicted heads in Layer 2 of the model. Residuals are shown below each well identifier. Positive values indicate locations where the model predicts greater heads than observed. Negative values indicate locations where the model predicts head values less than observed.

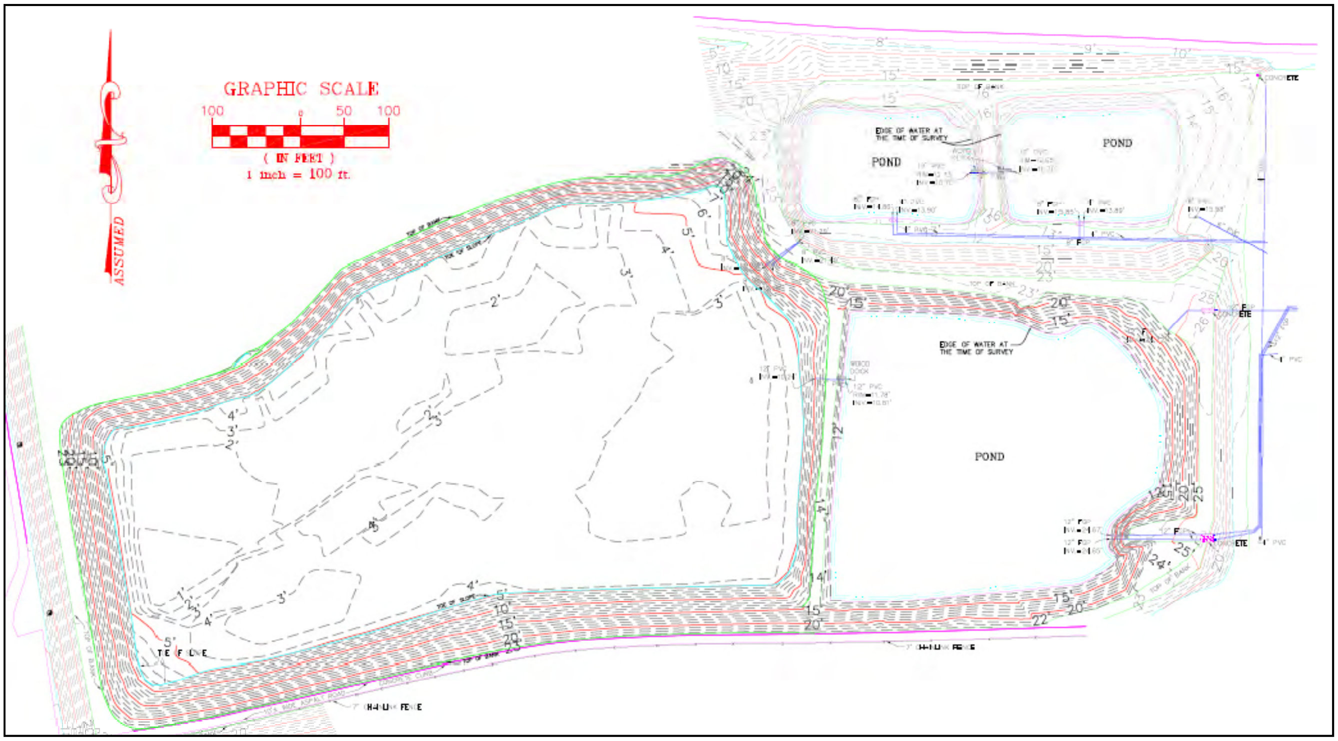


Exhibit 2, Figure 4: Topographic elevation contours for the IWW Pond system. Graphic adapted from GulfWest Surveying, Inc. figure (Progress Energy (09017)). Approximate minimum elevation of IWW Pond 4 (western pond) top of bank is approximately 23 ft amsl.

EXHIBIT 3

Transient Calibration Results

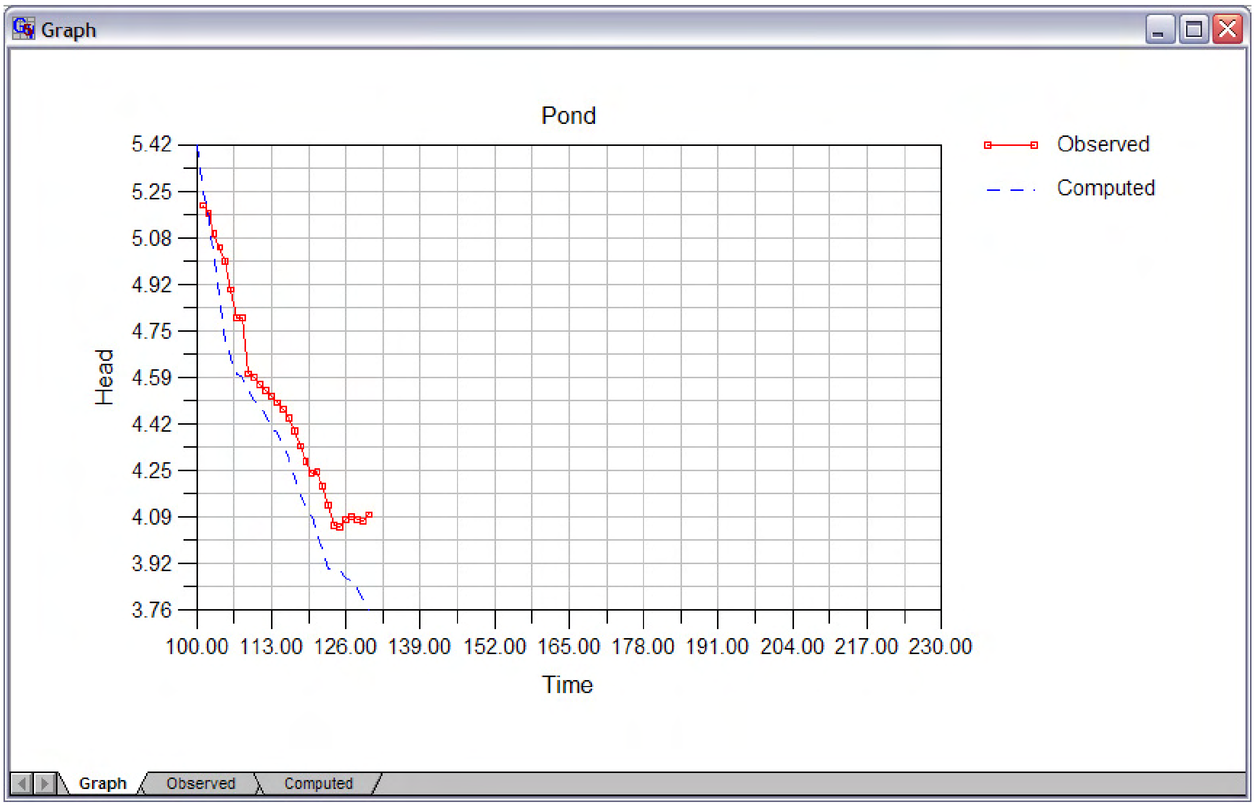


Exhibit 3: Calibration results for transient simulation showing observed vs. computed heads in the IWW Pond 4.

Attachment 12

WorleyParsons

CALCULATION COVER SHEET

CLIENT	Progress Energy Florida
PROJECT	Crystal River FGD
SUBJECT	North Percolation Pond Assessment
JOB NUMBER	570209 WBS NUMBER 024
CALCULATION NO.:	CRCA-0-DC-526-C-003 PAGE 1 OF 7

DESCRIPTION/PURPOSE

This calculation is to demonstrate the anticipated physical recovery performance of the north percolation pond to accommodate both the anticipated process flows and the tributary rainfall.

METHOD OF ANALYSIS

Excel spreadsheets were used to calculate pond volumes by elevation and area. Hand calculations and the MODRET Program, Version 6.1, were used to determine the percolation capacity of the pond.

CODES AND STANDARDS

Florida Administrative Code (FAC) Chapter 62-621 "Reuse of Reclaimed Water and Land Application"

REFERENCES

1. CRCA Encompass Document, CRCA-0-DW-111-002-001, Overall Site Plan, General Arrangement
2. Sitework – Site Grading Plan, Area A, Drawing No. S1003, Black & Veatch, 1978
3. Sitework – Site Grading Sections & Details, Drawing No. S1010, Black & Veatch, 1978
4. "Final Report of Subsurface Soil Exploration, Crystal River Power Plant Expansion," Ardaman & Associates, Inc., October 3, 2006
5. "Permeability Testing and Soil Profiles, Existing WRA Crystal River Energy Complex", Central Testing Laboratory, January 12, 2007
6. "Additional Geotechnical Services, Progress Energy Crystal River Plant", Ardaman and Associates, October 24, 2006
7. "Climatic Atlas of the United States", US Department of Commerce, June 1968
8. "TP 40 Rainfall Frequency Atlas of the United States", US Department of Commerce, May 1961
9. MODRET, Version 6.1, users manual
10. "Stormwater Retention Pond Infiltration Analysis in Unconfined Aquifers", Jammal & Associates, March 1989
11. FAC Chapter 62-621 "Reuse of Reclaimed Water and Land Application"

ASSUMPTIONS

As stated in the calculation.

1-30-07

REV	DATE	DESCRIPTION	PAGES REVISED	PAGES ADDED	PAGES DELETED	BY/DATE	REV/DATE	LDE/DATE
3								
2								
1								
0	1-26-07	ORIGINAL ISSUE	NA	NA	NA	W Anundson 1-26-07	1-30-07	W Winterhalter 1-30-07

WorleyParsons

CALCULATION COVER SHEET

CLIENT	Progress Energy Florida
PROJECT	Crystal River FGD
SUBJECT	North Percolation Pond Assessment
JOB NUMBER	570209 WBS NUMBER 024
CALCULATION NO.:	CRCA-0-DC-526-C-003 PAGE 2 OF 7

CONCLUSION & RECOMMENDATIONS

See page 7.

1. Introduction

The existing percolation pond (see Attachments 1 and 2) is used to treat waste water generated from plant operations from Units 4 and 5. The pond appears to have been built along with the original Unit 4 construction activities in the late 1970's and early 1980's. References 2 and 3 show the original pond configuration.

Based on information provided by mechanical engineering, the current average annual inflow into the pond is 533,000 gpd (370 gpm). It is proposed to add an additional annual average of 105 gpm for flow from the coal pile and 11 gpm for flows from the gypsum pile and limestone building. Therefore total proposed process flows:

370 gpm + 105 gpm + 11 gpm = 486 gpm = 699,840 gpd

In addition to the design process flows for the pond, rainfall from the tributary area needs to be considered. The tributary area for the percolation pond is shown on Attachment 2. This area is 295,520 square feet (6.78 acres). From Reference 7, the normal annual total precipitation for the plant is about 55 inches/year and from Reference 8, the 100-year, 24-hour precipitation is 11 inches in one day.

Therefore, the average annual continuous inflow rate would be:

55 in/yr x 1yr/365 days = 0.151 in/day
(0.151 in/day x 1ft/12 in) x 295,520 ft^2 = 3,711 ft^3/day
3,711 ft^3/day x 7.48 gal/ft^3 = 27,757 gpd

699,840 gpd + 27,757 gpd = <u>727,597 gpd</u>

The maximum inflow rate, based on a 100-year, 24-hour event would be:

(11 in/day x 1ft/12 in) x 295,520 ft^2 = 270,893 ft^3/day
270,893 ft^3/day x 7.48 gal/ft^3 = 2,026,282 gpd

699,840 gpd + 2,026,282 gpd = 2,726,122 gpd

2. Subsurface Explorations and Field Testing

Several borings were performed around the subject pond as part of the work performed by Ardaman and Associates as presented in Reference 4. The borings are UB-9, UB-10, UB-11, UB-12, UB-13, UB-14 and UU-1 as shown on Attachment 2.

To supplement the information presented in those borings, four additional borings were performed by Central Testing Laboratory (CTL) in and adjacent to the subject pond. The results of those borings are presented in Reference 5 and the approximate locations are shown on Attachment 2. In addition to borings performed, field permeability testing of pond bottom soils and laboratory sieve analysis were performed as indicated in Reference 5. Vertical field permeability rates (Kv) were found to be 0.6 feet/day for AB-1 at a depth of 12 inches below the pond bottom and 0.8 feet/day for AB-2 at a depth of 12 inches below the pond bottom.

It should also be noted that an investigation was performed by Ardaman and Associates in October, 2006 in another percolation pond on site. That report is presented as Reference 6. As part of that work, two field permeability tests were performed in the limestone below the pond bottom for the FGD Wastewater Percolation Ponds (see Item 36 on Attachment 1). A recommended average horizontal permeability rate (Kh) of 42.5 feet/day was recommended for the limestone.

The following is a summary of the Ardaman and Associates and CTL boring data around the subject pond (See Attachment 2).

Boring	Ground Surface Elevation (ft)	Groundwater Elevation (ft)	Top of Limestone (ft)
UB-9	98.1	91.7	92.1
UB-10	97.6	90.7	90.6
UB-11	96.8	90.5	85.8
UB-12	94.8	91.3	85.8
UB-13	94.6	90.8	90.1
UB-14	96.9	92.2	88.4
UU-1	97.6	91.4	93.1
B-1	~98	~92.5	~87
B-2	~99.5	~91.5	~91.5
AB-1	~93	~91.7	N/A
AB-2	~92	~91.5	N/A
Average		**91.4**	**89.4**

Note : Elevations for CTL Borings B-1, B-2, AB-1 and AB-2 are estimated from existing topographic maps.

3. Qualitative Infiltration Information for the Percolation Pond

In a conversation with Mr. Ron Johnson of Progress Energy on January 18, 2007, he indicated the following for the North Percolation Pond:

- The pond has never over-flowed that he remembers, nor come close to overflowing.
- The pond bottom is frequently wet for extended periods of time and cattails naturally grow in the pond unless they are cut down.
- The pond is never totally dry.
- After heavy rains, the pond will stage up, but always comes back down eventually.
- The area around the pond was likely filled up with limestone fill.
- After Hurricane Frances in 2004, the pond did not overflow.

4. Infiltration Analysis for Proposed Percolation Pond

Based on the results of the field investigations, it appears that there is a relatively thin layer of sand atop limestone in the pond area. Sieve analysis testing as presented in Reference 5 indicates the sands below the pond bottom range from relatively clean to slightly silty to silty fine sands (SP, SP-SM, SM) with percent passing the No. 200 sieve ranging from 3 to 19 percent (average of 9 percent passing). The groundwater table was found to range from about 90.5' to 92.5' in and around the pond area. The limestone was found to range from about 85.8' to 93.1'. From the topographic information on Attachment 2 and from Reference 2 and 3 drawings, it appears that the bottom of the pond ranges from as low as elevation 91' or 92' to 93' or 94'.

Based on review of the information presented above, it appears that the lower portions of the pond are right at the ambient water table. This would explain why the pond is never fully dry.

It also appears that while there may be a thin layer of sand atop the limestone at the two boring locations, it is possible that that the layer of sand is even thinner at other locations.

Based on the results of the field permeability testing, a vertical permeability rate (Kv) of 0.7 feet per day can be used. Since the bedrock is much more pervious than the sand overlying it, the percolation or seepage of water through the pond bottom will be governed by the percolation rate through the sand. The likely percolation rate through the pond bottom should be approximately as follows:

Q = 0.7 feet/day x 197,458 ft^2 = 138,221 ft^3/day = 1,033,962 gpd > 727,597 gpd (with FS ~ 1.4)

Per FAC 62-610.523, the maximum allowable average annual hydraulic loading rate into the pond is 5.6 GPD/ft^2. Therefore, the maximum allowable average annual rate:

197,458 ft^2 x 5.6 GPD/ft^2 = 1,105,765 gpd > 727,597 gpd

Both the infiltration rates above are greater than the average continuous flow rate of 727,655 gpd, therefore the pond should be able to handle the anticipated additional flow rate described above in Section 1.

The question is now about the 100 year storm event and what will happen during this event. It should be noted that the hydraulic conductivity rate (Kh) is typically about twice the vertical rate. Taking into account mounding and horizontal permeability and the close proximity of the limestone to the bottom of the pond, the actual permeability rate is likely twice that shown above (i.e. 2 x 0.7 feet/day = 1.4). Another consideration is that there may be areas where there is nominal cover of soil atop the limestone. If 5 percent of the pond had very little or no cover of sand over the limestone the effective pond permeability rate would be:

(9,873 ft^2 x 42.5 ft/day + 187,585 ft^2 x 0.7 ft/day)/197,458 ft^2 = 2.8 ft/day

If the permeability value is increased to 1.4 ft/day, then the expected infiltration rate for the pond would be:

Q = 1.4 feet/day x 197,458 ft^2 = 276,441 ft^3/day = 2,067,924 gpd

If the permeability value is increase to 2.8 ft/day, then the expected infiltration rate for the pond would be:

Q = 2.8 feet/day x 197,458 ft^2 = 552,882 ft^3/day = 4,135,848 gpd

It is difficult to say exactly what the effective permeability rate is for the pond, but the answer most likely lies somewhere between 1.4 and 2.8 feet/day. These values are consistent with the values given in Table 3-2 in Reference 10. Even if the infiltration rate does not match the 100 year storm inflow rate, there will be enough pond volume to store the event as shown on the stage storage information on Attachment 3 (cumulative volume = 893,780

ft^3 = 2.45 times the 100 year event). In the unlikely event the pond cannot accommodate the 100 year storm event, the water will overflow into the broad plain area around the cooling tower to the north. The spill over elevation would be about 97.5 feet and this would leave a three foot freeboard for the pond. Shortly after the end of the storm, the water would recede quickly. If the pond staged up to elevation 98 feet, the incremental volume between 97 and 98 feet is about 1,932,700 gallons, which is greater than 70% of the entire 24 hour event.

Evaluation Using MODRET:

MODRET, Version 6.1 program was also utilized to calculate the percolation rate of the subject pond. This program determines the capability of the underlying soils/rock to accommodate groundwater infiltration from a pond. The program requires the input of several variables including the vertical and horizontal hydraulic conductivity of the material below the pond. The program evaluates the mounding and movement of groundwater under and around the pond.

The MODRET program uses a single layer to represent the soil and/or rock beneath the pond. Therefore permeability of the materials beneath the pond is evaluated as an effective aquifer with a single horizontal permeability rate. The rate is determined from the following equation presented in Reference 9.

$Kh_{ave} = (K_{h1} \times m_1 + K_{h2} \times m_2 + \ldots K_{hi} \times m_i)/E_m$

Where Kh = coefficient of horizontal permeability; m = thickness of the individual layer; E_m = Thickness of aquifer

For this analysis the total aquifer thickness is taken as 50', assuming that the bedrock permeability is 42.5 ft/day for the assumed thickness (noting the solution-proneness of the Ocala formation). The actual aquifer thickness will be greater.

$Kh_{ave} \sim$ (1.4 ft/day x 2 ft + 42.5 ft/day x 48 ft)/50 = 40.8 ft/day (say 40 feet/day)

A summary of some of the design input used in the MODRET program are presented below.

- Start with pond bottom at elevation 93' with area of 161,484 ft^2.
- Pond length to width ratio is 1.2.
- Effective aquifer based elevation of 43 ft is conservatively based on an assumed total aquifer thickness of 50 feet. The actual aquifer thickness is greater.
- Elevation of seasonal high groundwater table of 91.4' as indicated above.
- Permeability rates as indicated above.
- Unsaturated vertical infiltration was not included in the analysis.

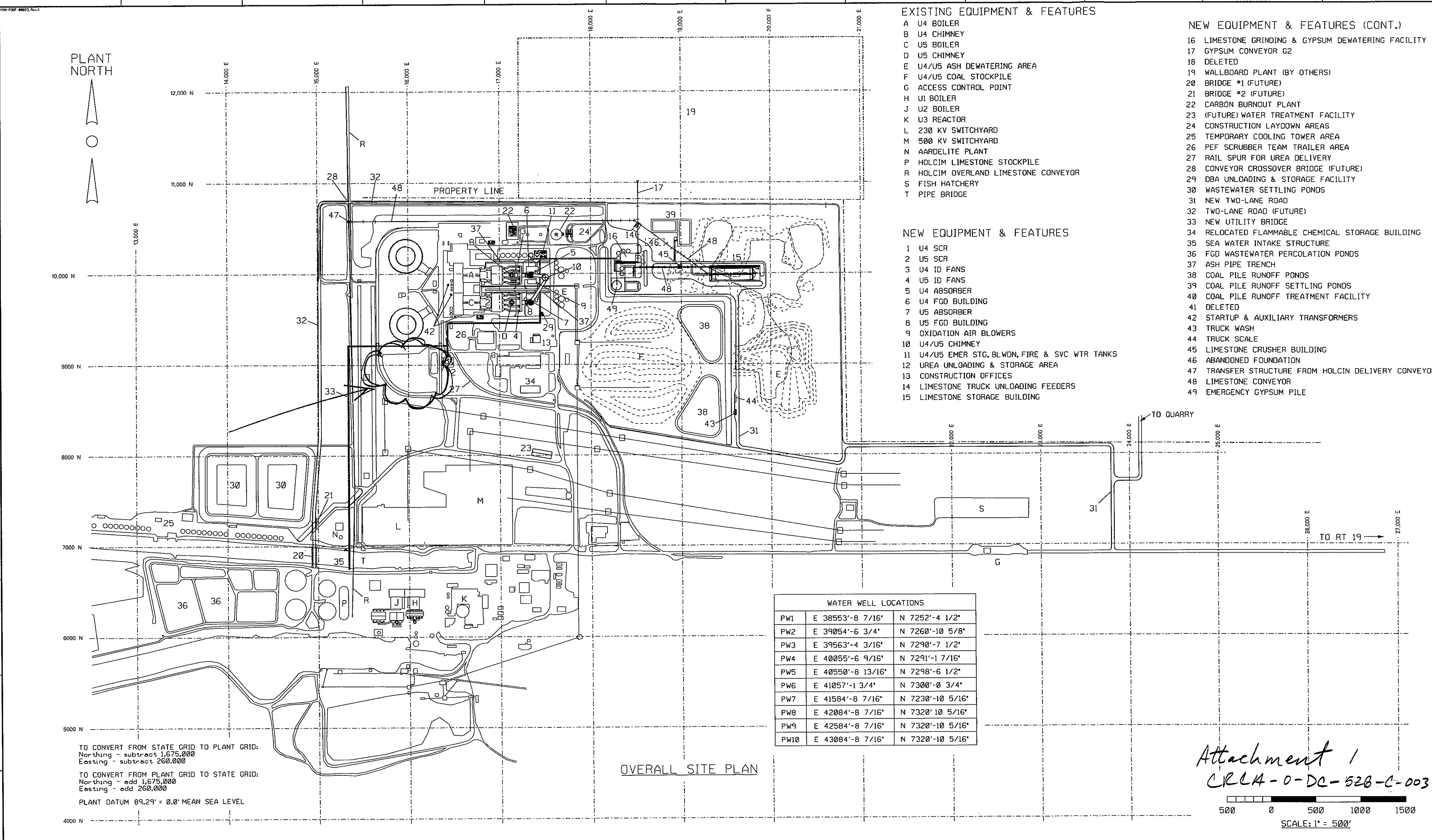
The first MODRET analysis was run using the annual average inflow rate of 97,265 ft^3/day (727,597 gpd) with a starting water level of 93 feet. The volume was conservatively added as a slug volume to the pond. The analysis shows that the pond does not recover back down to elevation 93 within 24 hours (see Attachment 4). The next analysis was run using the annual average inflow rate and a starting water elevation of 94 feet. With the additional head, the pond is able to recover back down to elevation 94 within about 16 hours (see Attachment 5). A third analysis was run using the annual average inflow rate and a starting water elevation of 95 feet. With the additional head, the pond is able to recover back down to elevation 95 within about 10 hours (see Attachment 6). The results of these analyses indicate that the pond will likely never be fully dry, but that the pond water level will stabilize to some nominal elevation above the pond bottom, likely somewhere between elevation 93 and 94 feet. The pond should however, be able to accommodate the average annual flow and provide volume to accept storm surge. To demonstrate this, a fourth analysis was run using the 100 year, 24 hour storm event. With a starting water level elevation of 94 feet, the water level in the pond stages up to 95.4 feet, then within 24 hours returns to 94.7 feet (see Attachment 7). At 24 hours the water level is still above elevation 94 feet; however, the infiltration rate at 24 hours is still greater than the inflow rate into the pond demonstrating that it can still accept the annual average inflow rate and continue to recover after the 100 year, 24 hour event.

5. Conclusions and Recommendations

The existing pond should be able to accommodate the additional annual average flows and provide infiltration into the soil and limestone below the pond. The pond will likely never be fully dry and will likely have perpetual standing water; however, the available pond volume above the stabilized water level should be sufficient to accommodate storm surge.

Over time, the performance of the pond may degrade due to siltation of the pond bottom. Decreased pond performance would be gradual. If pond performance becomes an issue over time, the pond could be bifurcated with a dike to provide two pond cells to allow cleaning of the pond similar to the FGD Wastewater Percolation Ponds.

In the unlikely event of overflow, the pond will overflow in the northwest corner at about elevation 97.5 and will overflow and be contained in the area north of the pond and surrounding the existing cooling tower.



OVERALL SITE PLAN

EXISTING EQUIPMENT & FEATURES

- A U4 BOILER
- B U4 CHIMNEY
- C U5 BOILER
- D U5 CHIMNEY
- E U4/U5 ASH DEWATERING AREA
- F U4/U5 COAL STOCKPILE
- G ACCESS CONTROL POINT
- H U1 BOILER
- J U2 BOILER
- K U3 REACTOR
- L 230 KV SWITCHYARD
- M 500 KV SWITCHYARD
- N AARDELITE PLANT
- P HOLCIM LIMESTONE STOCKPILE
- R HOLCIM OVERLAND LIMESTONE CONVEYOR
- S FISH HATCHERY
- T PIPE BRIDGE

NEW EQUIPMENT & FEATURES

1 U4 SCR
2 U5 SCR
3 U4 ID FANS
4 U5 ID FANS
5 U4 ABSORBER
6 U4 FGD BUILDING
7 U5 ABSORBER
8 U5 FGD BUILDING
9 OXIDATION AIR BLOWERS
10 U4/U5 CHIMNEY
11 U4/U5 EMER STG, BLWDN, FIRE & SVC WTR TANKS
12 UREA UNLOADING & STORAGE AREA
13 CONSTRUCTION OFFICES
14 LIMESTONE TRUCK UNLOADING FEEDERS
15 LIMESTONE STORAGE BUILDING

NEW EQUIPMENT & FEATURES (CONT.)

16 LIMESTONE GRINDING & GYPSUM DEWATERING FACILITY
17 GYPSUM CONVEYOR G2
18 DELETED
19 WALLBOARD PLANT (BY OTHERS)
20 BRIDGE #1 (FUTURE)
21 BRIDGE #2 (FUTURE)
22 CARBON BURNOUT PLANT
23 (FUTURE) WATER TREATMENT FACILITY
24 CONSTRUCTION LAYDOWN AREAS
25 TEMPORARY COOLING TOWER AREA
26 PEF SCRUBBER TEAM TRAILER AREA
27 RAIL SPUR FOR UREA DELIVERY
28 CONVEYOR CROSSOVER BRIDGE (FUTURE)
29 DBA UNLOADING & STORAGE FACILITY
30 WASTEWATER SETTLING PONDS
31 NEW TWO-LANE ROAD
32 TWO-LANE ROAD (FUTURE)
33 NEW UTILITY BRIDGE
34 RELOCATED FLAMMABLE CHEMICAL STORAGE BUILDING
35 SEA WATER INTAKE STRUCTURE
36 FGD WASTEWATER PERCOLATION PONDS
37 ASH PIPE TRENCH
38 COAL PILE RUNOFF PONDS
39 COAL PILE RUNOFF SETTLING PONDS
40 COAL PILE RUNOFF TREATMENT FACILITY
41 DELETED
42 STARTUP & AUXILIARY TRANSFORMERS
43 TRUCK WASH
44 TRUCK SCALE
45 LIMESTONE CRUSHER BUILDING
46 ABANDONED FOUNDATION
47 TRANSFER STRUCTURE FROM HOLCIN DELIVERY CONVEYOR
48 LIMESTONE CONVEYOR
49 EMERGENCY GYPSUM PILE

WATER WELL LOCATIONS		
PW1	E 38553'-8 7/16"	N 7252'-4 1/2"
PW2	E 39054'-6 3/4"	N 7260'-10 5/8"
PW3	E 39563'-4 3/16"	N 7290'-7 1/2"
PW4	E 40055'-6 9/16"	N 7291'-1 7/16"
PW5	E 40550'-8 13/16"	N 7298'-6 1/2"
PW6	E 41057'-1 3/4"	N 7300'-0 3/4"
PW7	E 41584'-8 7/16"	N 7230'-10 5/16"
PW8	E 42084'-8 7/16"	N 7320'10 5/16"
PW9	E 42584'-8 7/16"	N 7320'-10 5/16"
PW10	E 43084'-8 7/16"	N 7320'-10 5/16"

TO CONVERT FROM STATE GRID TO PLANT GRID:
Northing - subtract 1,675,000
Easting - subtract 260,000

TO CONVERT FROM PLANT GRID TO STATE GRID:
Northing - add 1,675,000
Easting - add 260,000

PLANT DATUM 89.29' = 0.0' MEAN SEA LEVEL

Attachment 1
CRCA-0-DC-528-C-003

NOTES:

DRAWING REVISIONS

REV.	DESCRIPTION	DRAWN	CHKD	APPRVD	DATE
G	GENERAL REVISION	CAR	JMN		01/08/07
F	GENERAL REVISION	GRP	JMN		10/20/06
E	GENERAL REVISION	GRP	JMN		10/03/06
D	REVISED ITEM 30	MG	CLS		05/17/06
C	ISSUED FOR EPC BID PACKAGE	GRP	CLS		05/11/06
B	ISSUED FOR REVIEW	CLS	FCC		03/30/06
A	ISSUED FOR INFORMATION ONLY	CLS			11/17/05

DRAWING REFERENCES

DWG NO.	REV.	TITLE

CONTRACTOR/VENDOR INFORMATION
WorleyParsons
resources & energy
2675 MORGANTOWN ROAD - READING, PA 19607 - (610) 855-2000
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PROGRESS ENERGY FLORIDA, INC.
POWER OPERATIONS GROUP

FACILITY: CRY | UNIT: 000 | SYSTEM:
PROJECT NO: | EC NO: | W.O. NO: N/A
SIZE: ANSI-D | SCALE: 1"=500' | PLOT: 1:6000

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Progress Energy

TITLE: FGD & SCR PROJECTS
OVERALL SITE PLAN
GENERAL ARRANGEMENT

SHT 001 OF 001 | DWG NO: CRCA-0-DW-111-002-001 | REV: G

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SERVER: reapds07

BID NO: CRY-

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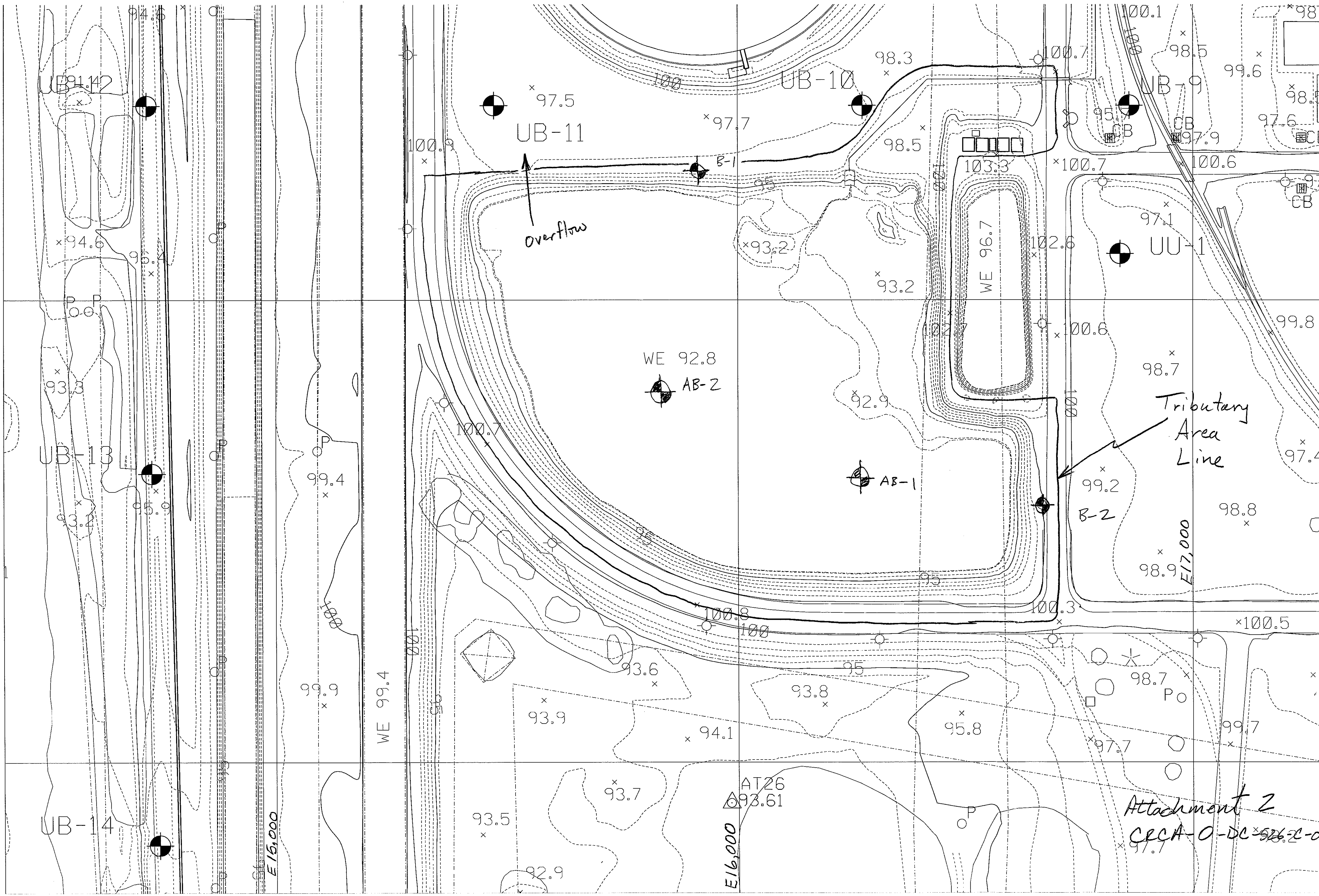

UB-12
UB-11
UB-10
UB-9
UU-1
UB-13
UB-14
B-1
B-2
AB-1
AB-2
Overflow
Tributary Area Line
WE 92.8
WE 96.7
WE 99.4
103.3
102.6
100.6
100.7
100.8
100.3
100.5
98.3
97.5
97.7
98.5
93.2
92.9
99.2
98.8
98.9
98.7
97.1
99.8
97.4
99.4
99.9
93.6
93.8
93.9
94.1
95.8
93.7
93.5
92.9
AT26
93.61
E16,000
E17,000
CB
Attachment 2
CRCA-O-DC-926-C-003

CRCA
West Blowdown Pond

POND VOLUMES BY ELEVATION AND AREA

Volumes are calculated using the conic method.

Enter data sets from the top down. Leave extra rows blank.

	ELEVATION	AREA	
Pond Bottom	92 FT	75000 FT^2	(Estimated)
	93 FT	161485 FT^2	
	94 FT	188558 FT^2	
	95 FT	197458 FT^2	
	96 FT	205419 FT^2	
	97 FT	212611 FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	
	FT	FT^2	

ELEVATION	INCREMENTAL VOLUME	CUMMULATIVE VOLUME	CUMMULATIVE VOLUME
92 FT			0.00 AC-FT
93 FT	115512 CF	115512 CF	2.65 AC-FT
94 FT	174847 CF	290359 CF	6.67 AC-FT
95 FT	192991 CF	483350 CF	11.10 AC-FT
96 FT	201425 CF	684775 CF	15.72 AC-FT
97 FT	209005 CF	893780 CF	20.52 AC-FT

Attachment 3
CRCA-0-DC-526-C-003

MODRET

SUMMARY OF UNSATURATED & SATURATED INPUT PARAMETERS

PROJECT NAME : Crystal River North Pond
POLLUTION VOLUME RUNOFF DATA USED
UNSATURATED ANALYSIS EXCLUDED

Pond Bottom Area	161,485.00 ft^2
Pond Volume between Bottom & DHWL	778,268.00 ft^3
Pond Length to Width Ratio (L/W)	1.20
Elevation of Effective Aquifer Base	43.00 ft
Elevation of Seasonal High Groundwater Table	91.40 ft
Elevation of Starting Water Level	93.00 ft
Elevation of Pond Bottom	93.00 ft
Design High Water Level Elevation	97.00 ft
Avg. Effective Storage Coefficient of Soil for Unsaturated Analysis	0.11
Unsaturated Vertical Hydraulic Conductivity	1.40 ft/d
Factor of Safety	2.00
Saturated Horizontal Hydraulic Conductivity	40.00 ft/d
Avg. Effective Storage Coefficient of Soil for Saturated Analysis	0.30
Avg. Effective Storage Coefficient of Pond/Exfiltration Trench	1.00

Not included

Hydraulic Control Features:

	Top	Bottom	Left	Right
Groundwater Control Features - Y/N	N	N	N	N
Distance to Edge of Pond	0.00	0.00	0.00	0.00
Elevation of Water Level	0.00	0.00	0.00	0.00
Impervious Barrier - Y/N	N	N	N	N
Elevation of Barrier Bottom	0.00	0.00	0.00	0.00

Attachment 4
CRCA-0-DC-526-C-003
1 of 6

Analysis Date: 1/25/2007

MODRET

TIME - RUNOFF INPUT DATA

PROJECT NAME: CRYSTAL RIVER NORTH POND

STRESS PERIOD NUMBER	INCREMENT OF TIME (hrs)	VOLUME OF RUNOFF (ft^3)
Unsat	0.00	0.00
1	1.00	97,265.00
2	1.82	0.00
3	1.82	0.00
4	1.82	0.00
5	1.82	0.00
6	1.82	0.00
7	1.82	0.00
8	1.82	0.00
9	1.82	0.00
10	1.82	0.00
11	1.82	0.00
12	1.82	0.00
13	1.82	0.00
14	1.82	0.00
15	1.82	0.00
16	1.82	0.00
17	1.82	0.00
18	1.82	0.00
19	1.82	0.00
20	1.82	0.00
21	1.82	0.00
22	1.82	0.00
23	1.82	0.00
24	1.82	0.00
25	1.82	0.00
26	1.82	0.00
27	1.82	0.00
28	1.82	0.00
29	1.82	0.00

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Attachment 4
2 of 6

Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
00.00 - 0.00	91.400	0.000 *		
			0.00000	
0.00	91.400	1.61254		
			1.57635	
1.00	93.471	1.54016		0.00
			1.47427	
2.82	93.421	1.37688		0.00
			1.27948	
4.64	93.378	1.21323		0.00
			1.14698	
6.46	93.339	1.09172		0.00
			1.03645	
8.28	93.304	0.99059		0.00
			0.94472	
10.10	93.273	0.90497		0.00
			0.86522	
11.92	93.243	0.83124		0.00
			0.79727	
13.74	93.217	0.76873		0.00
			0.74019	
15.56	93.192	0.71449		0.00
			0.68878	
17.38	93.168	0.66771		0.00
			0.64665	
19.21	93.147	0.62672		0.00
			0.60679	
21.03	93.126	0.58969		0.00
			0.57259	
22.85	93.107	0.55741		0.00

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Attachment 4
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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
			0.54223	
24.67	93.089	0.52910		0.00
			0.51596	
26.49	93.071	0.50305		0.00
			0.49014	
28.31	93.055	0.47949		0.00
			0.46885	
30.13	93.039	0.45900		0.00
			0.44914	
31.95	93.024	0.44031		0.00
			0.43148	
33.77	93.009	0.42264		0.00
			0.41381	
34.99	93.000	0.40668		0.00
			0.39954	
37.41	92.982	0.39229		0.00
			0.38505	
39.23	92.969	0.37904		0.00
			0.37304	
41.05	92.956	0.36681		0.00
			0.36058	
42.87	92.944	0.35492		0.00
			0.34926	
44.69	92.933	0.34518		0.00
			0.34110	
46.51	92.921	0.33465		0.00
			0.32819	
48.33	92.910	0.32457		0.00

CRCA-0-DC-526-C-003
Attachment 4
4 of 6

Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

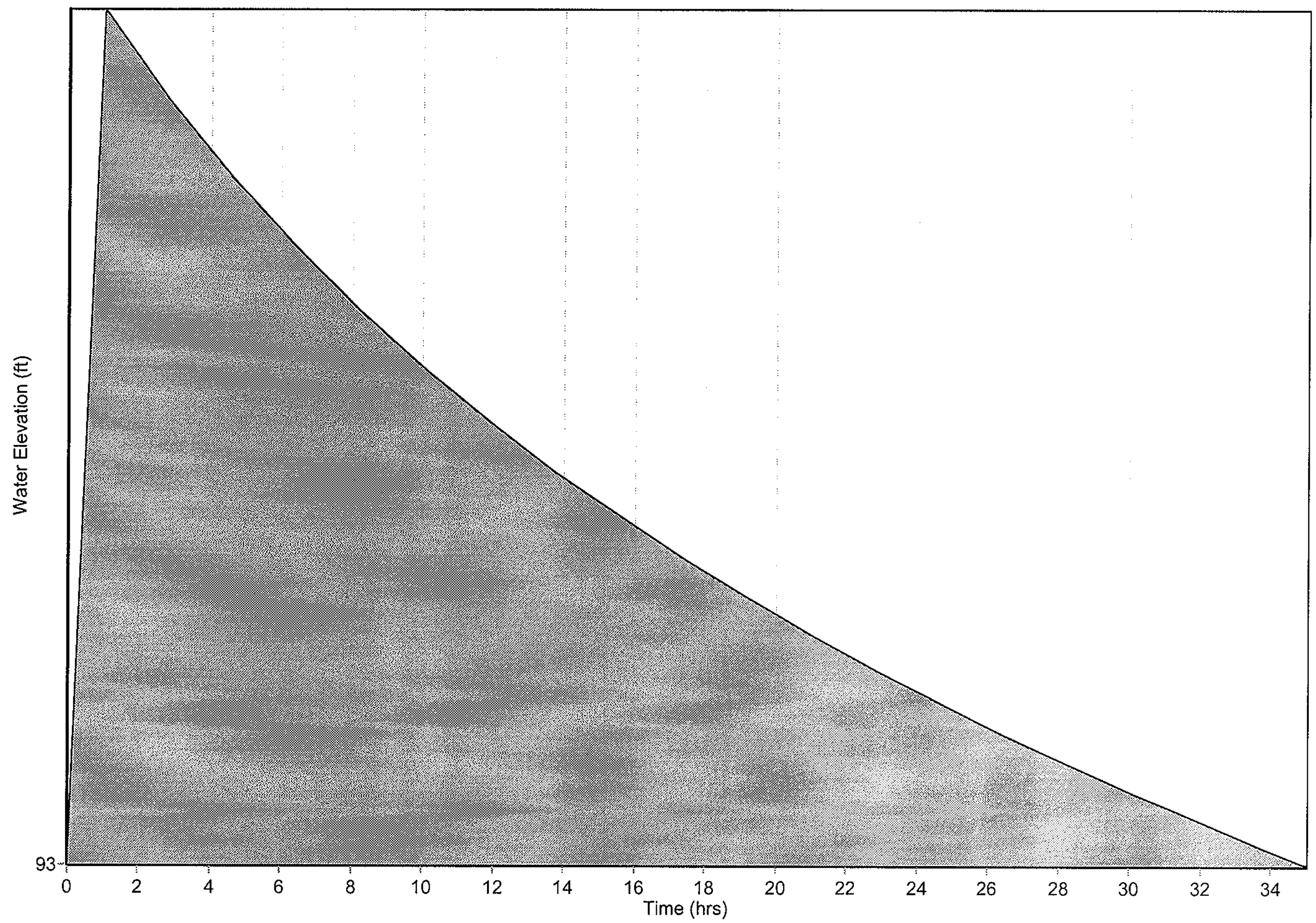
CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
			0.32095	
50.15	92.899	0.31676		0.00
			0.31257	
51.97	92.889	0.30826		0.00
			0.30396	
53.79	92.878	0.30056		0.00
			0.29716	
55.62	92.868	0.29320		0.00
			0.28924	
57.44	92.859	0.28584		0.00
			0.28244	
59.26	92.849	0.27904		0.00
			0.27565	
61.08	92.840	0.27338		0.00
			0.27112	
62.90	92.831	0.26761		0.00
			0.26410	
64.72	92.822	0.26183		0.00
			0.25957	
66.54	92.813	0.25617		0.00
			0.25277	
68.36	92.805	0.25119		0.00
			0.24960	
70.18	92.796	0.24666		0.00
			0.24371	
72.00	92.788			0.00

Maximum Water Elevation: 93.471 feet @ 1.00 hours Recovery @ 34.992 hours
* Time increment when there is no runoff
Maximum Infiltration Rate: 0.700 ft/day

Analysis Date: 1/25/2007

CRCA-0-DC-526-C-003
Attachment 4

INFILTRATION : CRYSTAL RIVER NORTH POND



Max Water Elevation = 93.47 ft

CRCA-0-DC-526-C-003
Attachment 4
6 of 6

MODRET

SUMMARY OF UNSATURATED & SATURATED INPUT PARAMETERS

PROJECT NAME : Crystal River North Pond
POLLUTION VOLUME RUNOFF DATA USED
UNSATURATED ANALYSIS EXCLUDED

Pond Bottom Area	161,485.00 ft^2
Pond Volume between Bottom & DHWL	778,268.00 ft^3
Pond Length to Width Ratio (L/W)	1.20
Elevation of Effective Aquifer Base	43.00 ft
Elevation of Seasonal High Groundwater Table	91.40 ft
Elevation of Starting Water Level	94.00 ft
Elevation of Pond Bottom	93.00 ft
Design High Water Level Elevation	97.00 ft
Avg. Effective Storage Coefficient of Soil for Unsaturated Analysis	0.11
Unsaturated Vertical Hydraulic Conductivity	1.40 ft/d
Factor of Safety	2.00
Saturated Horizontal Hydraulic Conductivity	40.00 ft/d
Avg. Effective Storage Coefficient of Soil for Saturated Analysis	0.30
Avg. Effective Storage Coefficient of Pond/Exfiltration Trench	1.00

not included

Hydraulic Control Features:

	Top	Bottom	Left	Right
Groundwater Control Features - Y/N	N	N	N	N
Distance to Edge of Pond	0.00	0.00	0.00	0.00
Elevation of Water Level	0.00	0.00	0.00	0.00
Impervious Barrier - Y/N	N	N	N	N
Elevation of Barrier Bottom	0.00	0.00	0.00	0.00

Attachment 5
CRCA-0-DC-526-C-003
1 of 6

Analysis Date: 1/25/2007

MODRET

TIME - RUNOFF INPUT DATA

PROJECT NAME: CRYSTAL RIVER NORTH POND

STRESS PERIOD NUMBER	INCREMENT OF TIME (hrs)	VOLUME OF RUNOFF (ft^3)
Unsat	0.00	0.00
1	1.00	97,265.00
2	1.82	0.00
3	1.82	0.00
4	1.82	0.00
5	1.82	0.00
6	1.82	0.00
7	1.82	0.00
8	1.82	0.00
9	1.82	0.00
10	1.82	0.00
11	1.82	0.00
12	1.82	0.00
13	1.82	0.00
14	1.82	0.00
15	1.82	0.00
16	1.82	0.00
17	1.82	0.00
18	1.82	0.00
19	1.82	0.00
20	1.82	0.00
21	1.82	0.00
22	1.82	0.00
23	1.82	0.00
24	1.82	0.00
25	1.82	0.00
26	1.82	0.00
27	1.82	0.00
28	1.82	0.00
29	1.82	0.00

CRCA-0-DC-526-C-003
Attachment 5
2 of 6

Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft³)
00.00 - 0.00	91.400	0.000 *		
			0.00000	
0.00	91.400	2.10180		
			2.10180	
1.00	94.346	2.10180		0.00
			2.10180	
2.82	94.293	2.10180		0.00
			2.10180	
4.64	94.240	1.95976		0.00
			1.81772	
6.46	94.194	1.73089		0.00
			1.64407	
8.28	94.152	1.58216		0.00
			1.52025	
10.10	94.114	1.46619		0.00
			1.41213	
11.92	94.078	1.36442		0.00
			1.31670	
13.74	94.045	1.27548		0.00
			1.23426	
15.56	94.014	1.19696		0.00
			1.15967	
17.38	93.984	1.12705		0.00
			1.09443	
19.21	93.957	1.06408		0.00
			1.03373	
21.03	93.931	1.00610		0.00
			0.97847	
22.85	93.906	0.95582		0.00

CRCA-0-DC-526-C-003
Attachment 5
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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft³)
			0.93317	
24.67	93.882	0.91052		0.00
			0.88787	
26.49	93.860	0.86899		0.00
			0.85012	
28.31	93.838	0.83034		0.00
			0.81056	
30.13	93.818	0.79591		0.00
			0.78126	
31.95	93.798	0.76511		0.00
			0.74895	
33.77	93.779	0.73521		0.00
			0.72147	
35.59	93.761	0.70879		0.00
			0.69610	
37.41	93.743	0.68417		0.00
			0.67224	
39.23	93.726	0.66107		0.00
			0.64990	
41.05	93.710	0.63993		0.00
			0.62996	
42.87	93.694	0.62030		0.00
			0.61064	
44.69	93.679	0.60203		0.00
			0.59342	
46.51	93.664	0.58542		0.00
			0.57742	
48.33	93.649	0.56851		0.00

CRCA-0-DC-526-C-003
Attachment 5
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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
			0.55960	
50.15	93.635	0.55341		0.00
			0.54722	
51.97	93.621	0.53937		0.00
			0.53151	
53.79	93.608	0.52608		0.00
			0.52064	
55.62	93.595	0.51385		0.00
			0.50705	
57.44	93.582	0.50131		0.00
			0.49558	
59.26	93.569	0.48984		0.00
			0.48410	
61.08	93.557	0.47866		0.00
			0.47323	
62.90	93.545	0.46825		0.00
			0.46326	
64.72	93.533	0.45919		0.00
			0.45511	
66.54	93.522	0.45058		0.00
			0.44605	
68.36	93.511	0.44137		0.00
			0.43669	
70.18	93.499	0.43231		0.00
			0.42793	
72.00	93.489			0.00

Maximum Water Elevation: 94.346 feet @ 1.00 hours   Recovery @ 72.000 hours
* Time increment when there is no runoff
Maximum Infiltration Rate: 0.700 ft/day

Analysis Date: 1/25/2007

CRCA-O-DC-526-C-003
Attachment 5
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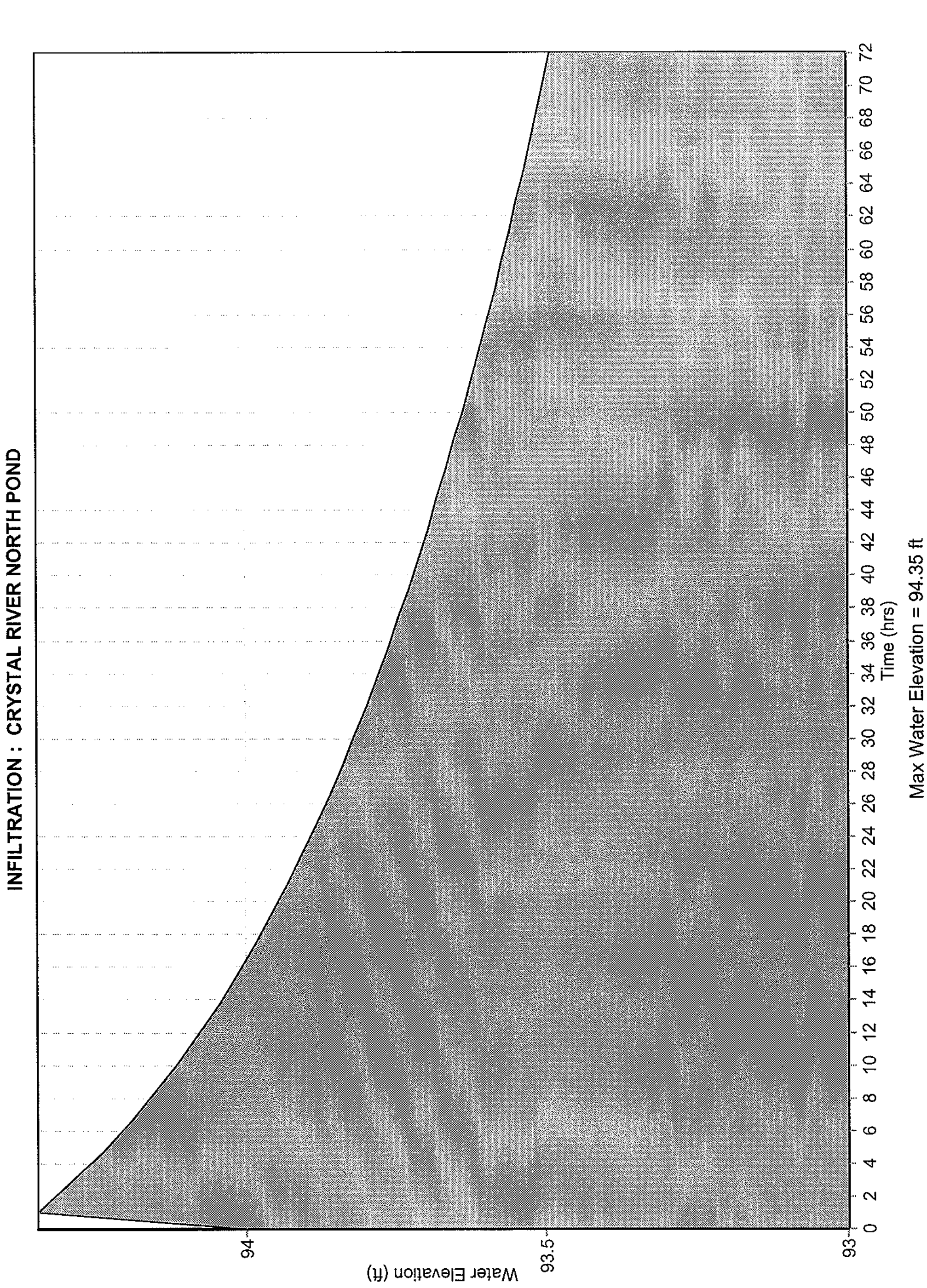

CRCA-0-DC-526-C-003
Attachment 5
6 of 6

MODRET

SUMMARY OF UNSATURATED & SATURATED INPUT PARAMETERS

PROJECT NAME : Crystal River North Pond
POLLUTION VOLUME RUNOFF DATA USED
UNSATURATED ANALYSIS EXCLUDED

Pond Bottom Area	161,485.00 ft^2
Pond Volume between Bottom & DHWL	778,268.00 ft^3
Pond Length to Width Ratio (L/W)	1.20
Elevation of Effective Aquifer Base	43.00 ft
Elevation of Seasonal High Groundwater Table	91.40 ft
Elevation of Starting Water Level	95.00 ft
Elevation of Pond Bottom	93.00 ft
Design High Water Level Elevation	97.00 ft
Avg. Effective Storage Coefficient of Soil for Unsaturated Analysis	0.11
Unsaturated Vertical Hydraulic Conductivity	1.40 ft/d
Factor of Safety	2.00
Saturated Horizontal Hydraulic Conductivity	40.00 ft/d
Avg. Effective Storage Coefficient of Soil for Saturated Analysis	0.30
Avg. Effective Storage Coefficient of Pond/Exfiltration Trench	1.00

Not used

Hydraulic Control Features:

	Top	Bottom	Left	Right
Groundwater Control Features - Y/N	N	N	N	N
Distance to Edge of Pond	0.00	0.00	0.00	0.00
Elevation of Water Level	0.00	0.00	0.00	0.00
Impervious Barrier - Y/N	N	N	N	N
Elevation of Barrier Bottom	0.00	0.00	0.00	0.00

Attachment 6
CRCA-0-DC-526-C-003
1 of 6

Analysis Date: 1/25/2007

MODRET

TIME - RUNOFF INPUT DATA

PROJECT NAME: CRYSTAL RIVER NORTH POND

STRESS PERIOD NUMBER	INCREMENT OF TIME (hrs)	VOLUME OF RUNOFF (ft^3)
Unsat	0.00	0.00
1	1.00	97,265.00
2	1.82	0.00
3	1.82	0.00
4	1.82	0.00
5	1.82	0.00
6	1.82	0.00
7	1.82	0.00
8	1.82	0.00
9	1.82	0.00
10	1.82	0.00
11	1.82	0.00
12	1.82	0.00
13	1.82	0.00
14	1.82	0.00
15	1.82	0.00
16	1.82	0.00
17	1.82	0.00
18	1.82	0.00
19	1.82	0.00
20	1.82	0.00
21	1.82	0.00
22	1.82	0.00
23	1.82	0.00
24	1.82	0.00
25	1.82	0.00
26	1.82	0.00
27	1.82	0.00
28	1.82	0.00
29	1.82	0.00

CRCA-0-DC-526-C-003
Attachment 6
2 of 6

Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
00.00 - 0.00	91.400	0.000 *		
			0.00000	
0.00	91.400	3.19969		
			3.12613	
1.00	95.221	3.05257		0.00
			2.91864	
2.82	95.172	2.82351		0.00
			2.72839	
4.64	95.126	2.64345		0.00
			2.55851	
6.46	95.083	2.48467		0.00
			2.41084	
8.28	95.042	2.34221		0.00
			2.27358	
10.10	95.004	2.21220		0.00
			2.15082	
11.92	94.968	2.09442		0.00
			2.03802	
13.74	94.933	1.98842		0.00
			1.93882	
15.56	94.901	1.89035		0.00
			1.84188	
17.38	94.870	1.80020		0.00
			1.75852	
19.21	94.840	1.71821		0.00
			1.67789	
21.03	94.812	1.64211		0.00
			1.60632	
22.85	94.785	1.57393		0.00

CRCA-0-DC-526-C-003
Attachment 6
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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
			1.54154	
24.67	94.759	1.50847		0.00
			1.47540	
26.49	94.734	1.44890		0.00
			1.42240	
28.31	94.710	1.39568		0.00
			1.36895	
30.13	94.687	1.34268		0.00
			1.31640	
31.95	94.665	1.29398		0.00
			1.27156	
33.77	94.643	1.25117		0.00
			1.23079	
35.59	94.623	1.21040		0.00
			1.19002	
37.41	94.603	1.17054		0.00
			1.15106	
39.23	94.583	1.13407		0.00
			1.11708	
41.05	94.564	1.10146		0.00
			1.08583	
42.87	94.546	1.06929		0.00
			1.05276	
44.69	94.528	1.03804		0.00
			1.02331	
46.51	94.511	1.00950		0.00
			0.99568	
48.33	94.494	0.98345		0.00

CRCA-0-DC-526-C-003
Attachment 6
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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

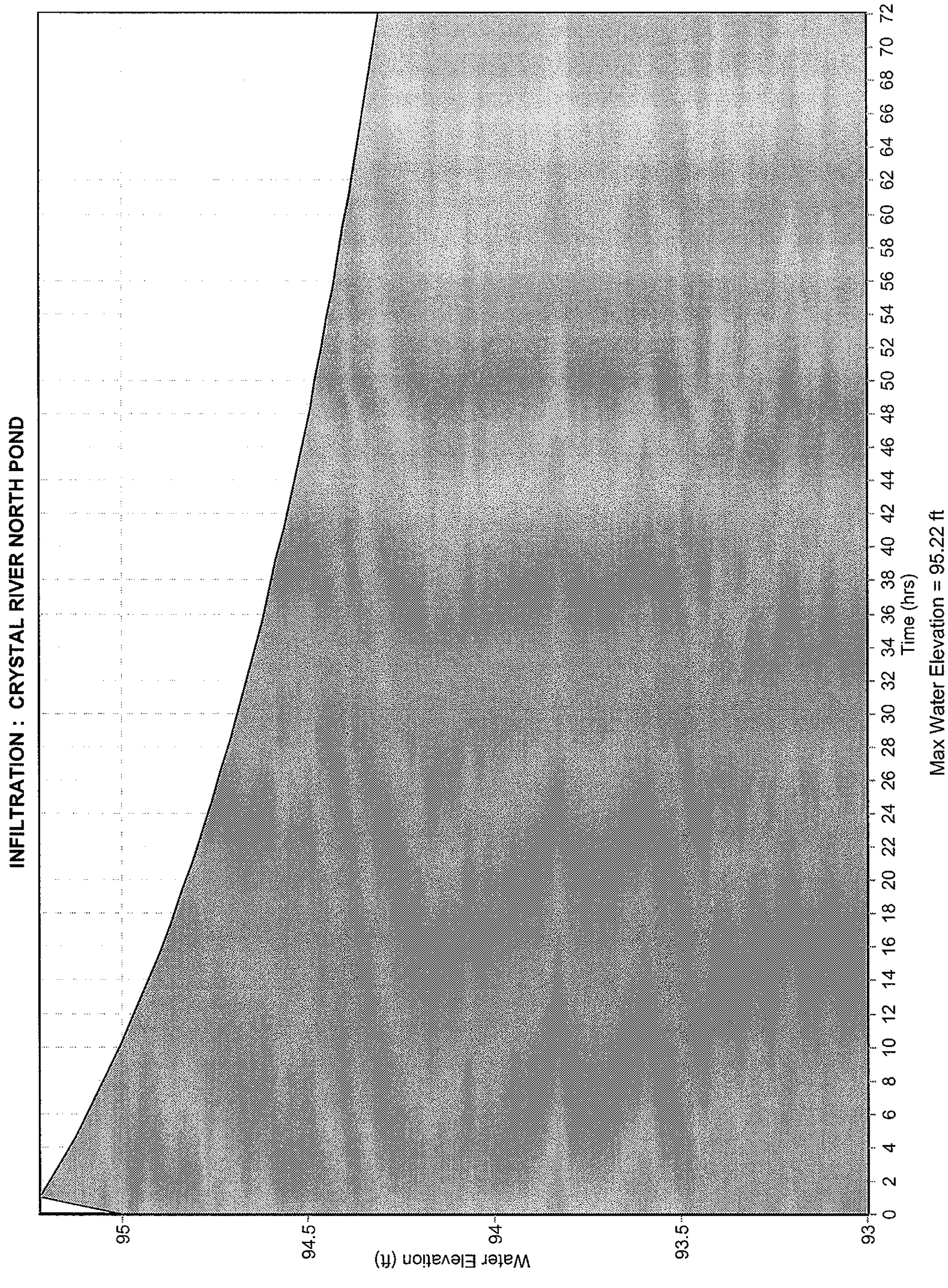
PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
			0.97122	
50.15	94.478	0.95899		0.00
			0.94676	
51.97	94.462	0.93453		0.00
			0.92230	
53.79	94.447	0.91142		0.00
			0.90055	
55.62	94.431	0.89059		0.00
			0.88062	
57.44	94.417	0.87066		0.00
			0.86069	
59.26	94.402	0.85095		0.00
			0.84121	
61.08	94.388	0.83283		0.00
			0.82445	
62.90	94.374	0.81471		0.00
			0.80497	
64.72	94.360	0.79840		0.00
			0.79183	
66.54	94.347	0.78368		0.00
			0.77553	
68.36	94.334	0.76783		0.00
			0.76012	
70.18	94.321	0.75288		0.00
			0.74563	
72.00	94.309			0.00

Maximum Water Elevation: 95.221 feet @ 1.00 hours   Recovery @ 72.000 hours
* Time increment when there is no runoff
Maximum Infiltration Rate: 0.694 ft/day

Analysis Date: 1/25/2007

CRCA-0-DC-526-C-003
Attachment 6



CRCA-0-DC-526-C-003
Attachment 6
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MODRET

SUMMARY OF UNSATURATED & SATURATED INPUT PARAMETERS

PROJECT NAME : Crystal River North Pond
POLLUTION VOLUME RUNOFF DATA USED
UNSATURATED ANALYSIS EXCLUDED

Pond Bottom Area	161,485.00 ft^2
Pond Volume between Bottom & DHWL	778,268.00 ft^3
Pond Length to Width Ratio (L/W)	1.20
Elevation of Effective Aquifer Base	43.00 ft
Elevation of Seasonal High Groundwater Table	91.40 ft
Elevation of Starting Water Level	94.00 ft
Elevation of Pond Bottom	93.00 ft
Design High Water Level Elevation	97.00 ft
Avg. Effective Storage Coefficient of Soil for Unsaturated Analysis	0.11
Unsaturated Vertical Hydraulic Conductivity	1.40 ft/d
Factor of Safety	2.00
Saturated Horizontal Hydraulic Conductivity	40.00 ft/d
Avg. Effective Storage Coefficient of Soil for Saturated Analysis	0.30
Avg. Effective Storage Coefficient of Pond/Exfiltration Trench	1.00

Not Included

Hydraulic Control Features:

	Top	Bottom	Left	Right
Groundwater Control Features - Y/N	N	N	N	N
Distance to Edge of Pond	0.00	0.00	0.00	0.00
Elevation of Water Level	0.00	0.00	0.00	0.00
Impervious Barrier - Y/N	N	N	N	N
Elevation of Barrier Bottom	0.00	0.00	0.00	0.00

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Analysis Date: 1/25/2007

MODRET

TIME - RUNOFF INPUT DATA

PROJECT NAME: CRYSTAL RIVER NORTH POND

STRESS PERIOD NUMBER	INCREMENT OF TIME (hrs)	VOLUME OF RUNOFF (ft^3)
Unsat	0.00	0.00
1	1.00	364,429.00
2	1.82	0.00
3	1.82	0.00
4	1.82	0.00
5	1.82	0.00
6	1.82	0.00
7	1.82	0.00
8	1.82	0.00
9	1.82	0.00
10	1.82	0.00
11	1.82	0.00
12	1.82	0.00
13	1.82	0.00
14	1.82	0.00
15	1.82	0.00
16	1.82	0.00
17	1.82	0.00
18	1.82	0.00
19	1.82	0.00
20	1.82	0.00
21	1.82	0.00
22	1.82	0.00
23	1.82	0.00
24	1.82	0.00
25	1.82	0.00
26	1.82	0.00
27	1.82	0.00
28	1.82	0.00
29	1.82	0.00

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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft³)
00.00 - 0.00	91.400	0.000 *		
			0.00000	
0.00	91.400	2.10180		
			2.10180	
1.00	95.376	2.10180		0.00
			2.10180	
2.82	95.323	2.10180		0.00
			2.10180	
4.64	95.269	2.10180		0.00
			2.10180	
6.46	95.216	2.10180		0.00
			2.10180	
8.28	95.163	2.10180		0.00
			2.10180	
10.10	95.110	2.10180		0.00
			2.10180	
11.92	95.057	2.10180		0.00
			2.10180	
13.74	95.004	2.10180		0.00
			2.10180	
15.56	94.951	2.10180		0.00
			2.10180	
17.38	94.898	2.10180		0.00
			2.10180	
19.21	94.845	1.85285		0.00
			1.60390	
21.03	94.804	1.46498		0.00
			1.32607	
22.85	94.771	1.29270		0.00

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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft³)
			1.25932	
24.67	94.739	1.23064		0.00
			1.20195	
26.49	94.708	1.17446		0.00
			1.14698	
28.31	94.679	1.12312		0.00
			1.09927	
30.13	94.652	1.07647		0.00
			1.05367	
31.95	94.625	1.03343		0.00
			1.01320	
33.77	94.599	0.99402		0.00
			0.97484	
35.59	94.575	0.95733		0.00
			0.93981	
37.41	94.551	0.92411		0.00
			0.90841	
39.23	94.528	0.89331		0.00
			0.87821	
41.05	94.506	0.86507		0.00
			0.85193	
42.87	94.484	0.83789		0.00
			0.82385	
44.69	94.464	0.81282		0.00
			0.80180	
46.51	94.443	0.78987		0.00
			0.77794	
48.33	94.424	0.76828		0.00

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Analysis Date: 1/25/2007

MODRET

SUMMARY OF RESULTS

PROJECT NAME : Crystal River North Pond

CUMULATIVE TIME (hrs)	WATER ELEVATION (feet)	INSTANTANEOUS INFILTRATION RATE (cfs)	AVERAGE INFILTRATION RATE (cfs)	CUMULATIVE OVERFLOW (ft^3)
			0.75861	
50.15	94.405	0.74774		0.00
			0.73687	
51.97	94.386	0.72811		0.00
			0.71936	
53.79	94.368	0.70939		0.00
			0.69942	
55.62	94.350	0.69202		0.00
			0.68463	
57.44	94.333	0.67692		0.00
			0.66922	
59.26	94.316	0.66122		0.00
			0.65322	
61.08	94.299	0.64673		0.00
			0.64023	
62.90	94.283	0.63314		0.00
			0.62604	
64.72	94.267	0.62030		0.00
			0.61456	
66.54	94.252	0.60747		0.00
			0.60037	
68.36	94.237	0.59508		0.00
			0.58980	
70.18	94.222	0.58346		0.00
			0.57711	
72.00	94.207			0.00

Maximum Water Elevation: 95.376 feet @ 1.00 hours   Recovery @ > 72.000 hours
* Time increment when there is no runoff
Maximum Infiltration Rate: 0.700 ft/day

Analysis Date: 1/25/2007

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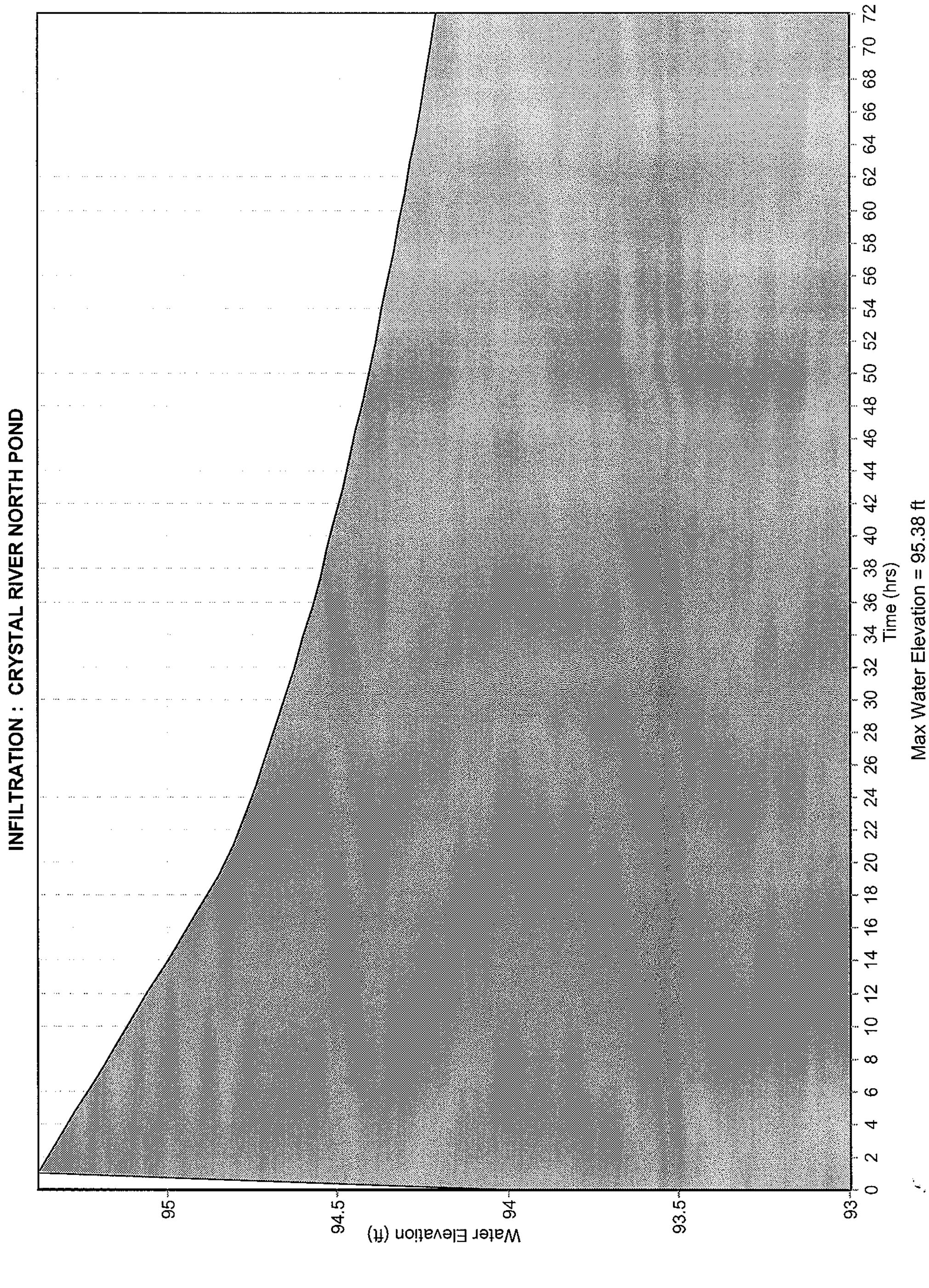

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Attachment 13