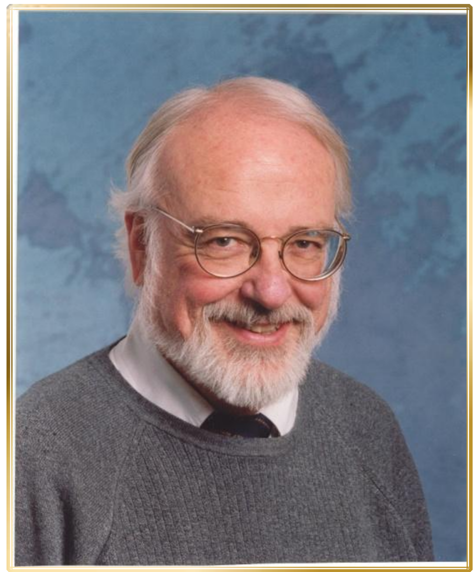
The Nineteenth Spencer Buchanan Lecture
Friday November 11th, 2011
College Station Hilton
College Station, Texas, USA
http://ceprofs.tamu.edu/briaud/buchanan.htm



Reinforced Soil Technology: From Experimental to the Familiar

The 2010 Karl Terzaghi Lecture
By Prof. Robert D. Holtz

Cold War Legacy – Design, Construction, and Performance of a Land-Based Radioactive Waste Disposal Facility

The 2011 Spencer J. Buchanan Lecture
By Dr. Rudolph Bonaparte

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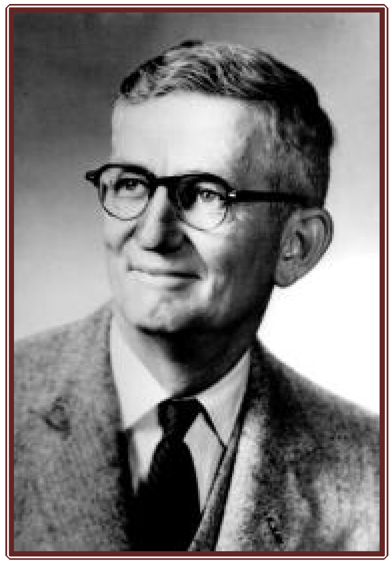
SPENCER J. BUCHANAN



Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star. He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.

Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England. They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy's Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.

The Spencer J. Buchanan '26 Chair in Civil Engineering

The College of Engineering and the Department of Civil Engineering gratefully recognize the generosity of the following individuals, corporations, foundations, and organizations for their part in helping to establish the Spencer J. Buchanan '26 Professorship in Civil Engineering. Created in 1992 to honor a world leader in soil mechanics and foundation engineering, as well as a distinguished Texas A&M University professor, the Buchanan Professorship supports a wide range of enriched educational activities in civil and geotechnical engineering. In 2002, this professorship became the Spencer J. Buchanan '26 Chair in Civil Engineering.

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Spencer J. Buchanan Lecture Series

1993	Ralph B. Peck	"The Coming of Age of Soil Mechanics: 1920 - 1970"
1994	G. Geoffrey Meyerhof	"Evolution of Safety Factors and Geotechnical Limit State Design"
1995	James K. Mitchell	"The Role of Soil Mechanics in Environmental Geotechnics"
1996	Delwyn G. Fredlund	"The Emergence of Unsaturated Soil Mechanics"
1997	T. William Lambe	"The Selection of Soil Strength for a Stability Analysis"
1998	John B. Burland	"The Enigma of the Leaning Tower of Pisa"
1999	J. Michael Duncan	"Factors of Safety and Reliability in Geotechnical Engineering"
2000	Harry G. Poulos	"Foundation Settlement Analysis – Practice Versus Research"
2001	Robert D. Holtz	"Geosynthetics for Soil Reinforcement"
2002	Arnold Aronowitz	"World Trade Center: Construction, Destruction, and Reconstruction"
2003	Eduardo Alonso	"Exploring the Limits of Unsaturated Soil Mechanics: the Behavior of Coarse Granular Soils and Rockfill"
2004	Raymond J. Krizek	"Slurries in Geotechnical Engineering"
2005	Tom D. O'Rourke	"Soil-Structure Interaction Under Extreme Loading Conditions"
2006	Cylde N. Baker	"In Situ Testing, Soil-Structure Interaction, and Cost Effective Foundation Design"
2007	Ricardo Dobry	"Pile response to Liquefaction and Lateral Spreading: Field Observations and Current Research"
2008	Kenneth Stokoe	"The Increasing Role of Seismic Measurements in Geotechnical Engineering"
2009	Jose M. Roesset	"Some Applications of Soil Dynamics"
2010	Kenji Ishihara	"Forensic Diagnosis for Site-Specific Ground Conditions in Deep Excavations of Subway Constructions"
2011	Rudolph Bonaparte	"Cold War Legacy – Design, Construction, and Performance of a Land-Based Radioactive Waste Disposal Facility"

The texts of the lectures and a DVD's of the presentations are available by contacting:

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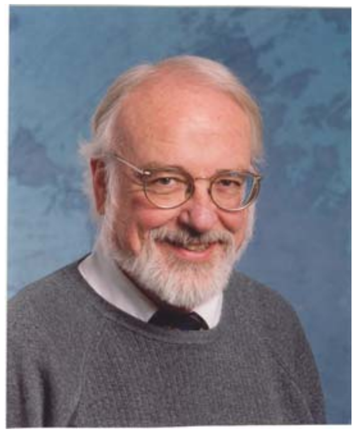
http://ceprofs.tamu.edu/briaud/buchanan.htm

AGENDA

The Nineteenth Spencer J. Buchanan Lecture
Friday, November 11th, 2011
College Station Hilton

2:00 p.m.	Introduction by Jean-Louis Briaud
2:15 p.m.	Introduction of Robert D. Holtz by Stacey Tucker
2:20 p.m.	"Reinforced Soil Technology: From Experimental to the Familiar" The 2010 Terzaghi Lecture by Robert D. Holtz
3:20 p.m.	Introduction of Rudolph Bonaparte by Jean-Louis Briaud
3:25 p.m.	"Cold War Legacy – Design, Construction, and Performance of a Land-Based Radioactive Waste Disposal Facility" The 2010 Buchanan Lecture by Rudolph Bonaparte
4:25 p.m.	Discussion
4:40 p.m.	Closure with Philip Buchanan
5:00 p.m.	Photos followed by a reception at the home of Jean-Louis and Janet Briaud.

Robert D. Holtz, PhD, PE, D.GE



Robert D. Holtz, PhD, PE, D.GE, is Professor Emeritus of Civil Engineering at the University of Washington in Seattle. He has degrees in civil engineering from Minnesota and Northwestern, and he participated in the Special Program in Soil Mechanics at Harvard under Professor A. Casagrande. Before coming to the UW in 1988, he was on the faculty at Purdue and Cal State-Sacramento. In addition to experience with the California Dept. of Water Resources, Swedish Geotechnical Institute, and NRC-Canada, he has worked as a consulting engineer in Chicago, Paris, France, and Milano, Italy. His research and publications are mostly on geosynthetics, foundations, soil improvement, soil properties, and instrumentation, and his research has been sponsored by NSF, FHWA, US Air Force, Indiana Dept of Highways, Washington State Dept of Transportation, Washington Technology Center, ADSC, US Dept. of Energy, and several private companies. Professor Holtz is author, co-author, or editor of 23 books and book chapters, including *Introduction to Geotechnical Engineering*, 2nd Edition (with W. D. Kovacs and T.C. Sheahan, 2011). He is also author or co-author of more than 270 technical papers, discussions, reviews, and major reports.

Professor Holtz is a Fellow, Life, and Distinguished Member of ASCE, was President of the Geo-Institute of ASCE in 2000-01, and currently serves as International Secretary for the Geo-Institute. He is a Member Emeritus of TRB Committee on Soil and Rock Properties, a Past President of North American Geosynthetics Society; and a member of several other professional and technical organizations. Bob is a registered engineer in California and Indiana, and he is a Diplomate of the Academy of Geo-Professionals.

Professor Holtz has taught numerous short courses and given many presentations at seminars and conferences, both in the U.S. and abroad. He was the Cross-Canada and the 38th Ardaman Lecturers in 1999, the 9th Spencer J. Buchanan Lecturer (2001), and he held the J. S. Braun/Braun Intertec Visiting Professorship at the University of Minnesota in 2002. He gave the 8th R. L. Schiffman '44 Lecture (2003), the 3rd G. A. Leonards Lecturer in 2005, the Stanley D. Wilson Memorial Lecturer in 2007, and in 2008, he presented the H. R. Berg and the Lymon C. Reese Lectures. In 2009, he gave the Osterberg Geomechanics Lecture, and in 2010, he was the 46th Karl Terzaghi Lecturer. He also has given the Terzaghi Lecture in Brazil, China, Canada, and at several US venues. In 2008, he was named the Puget Sound Academic Engineer of the Year.

Throughout his academic career, Professor Holtz has had an active consulting practice. His projects have involved various aspects of geosynthetics, foundations, soil reinforcing, soil improvement, properties and containment of nuclear wastes, slope stability and landslides, investigation of failures, and acting as an expert witness. His clients have included federal, state, and local public agencies, civil and geotechnical engineering consultants and contractors, attorneys, and manufacturers, both in North America and overseas.

GEOSYNTHETIC REINFORCED SOIL: FROM THE EXPERIMENTAL TO THE FAMILIAR

46th Terzaghi Lecture

R. D. Holtz, PhD, PE, D.GE
Professor Emeritus
University of Washington
Seattle, Washington USA

ABSTRACT

The lecture begins with a historical review of reinforced soil technology, from the ancients, the developments by H. Vidal and K. Lee on Terre Armée and Reinforced Earth, the early uses of geosynthetics for soil reinforcement in France (Bidim), Sweden (Wager and Broms), and the USA (USFS, FHWA, J. R. Bell, T. A. Haliburton, B. R. Christopher and others). The advantages and basic behavior of geosynthetic reinforced soil (GRS) are presented along with an overview of current design procedures, and with reference to UW analytical research results. Practical suggestions are given for dealing with creep, pullout, and backfill drainage. Geosynthetic properties and then discussed, again with reference to UW research results. Although GRS is quite a mature development, a few technical and professional issues remain; primarily, too many failures of these structures occur. Reasons for these failures and some suggestions as to what the profession can do about them are presented. The lecture ends with several examples of successful applications of GRS and reinforced soil technology.

46th Terzaghi Lecture

GEOSYNTHETIC REINFORCED SOIL: FROM THE EXPERIMENTAL TO THE FAMILIAR

R. D. Holtz, PhD, PE, D.GE
Professor Emeritus
University of Washington
Seattle, Washington USA

Kjellman paper drain installation, Halmsjön, Sweden, 1946 or 47?

Two previous Terzaghi Lectures on Geosynthetics:

R. M. Koerner
(1996)
Geomembranes: properties and behavior

J.-P. Giroud
(2008)
Geotextile and granular filters

My two geosynthetics heroes…

Geosynthetics in Civil Engineering…

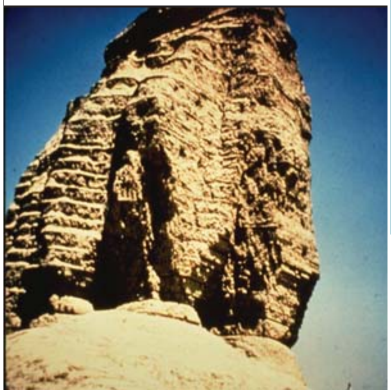
- From the experimental to accepted practice
  - Waste containment
  - Canal and pond liners
  - Drainage and erosion control
  - Construction
  - Transportation
  - Geotechnical
- "Geosynthetics - THE most important development in Civil Engineering practice in the 20th Century."
  (J.-P. Giroud, 2008 Terzaghi Lecture)
- The first new civil engineering material in more than 100 yr…
- Other examples…

My plan:

1. Introduction
2. **Reinforced soil—a historical perspective**
3. Advantages and basic behavior of GRS
4. Design
5. Properties
6. Things we need still need to know and do—technical and professional issues
7. Successful examples
8. Final remarks

Some examples from nature and the ancients:

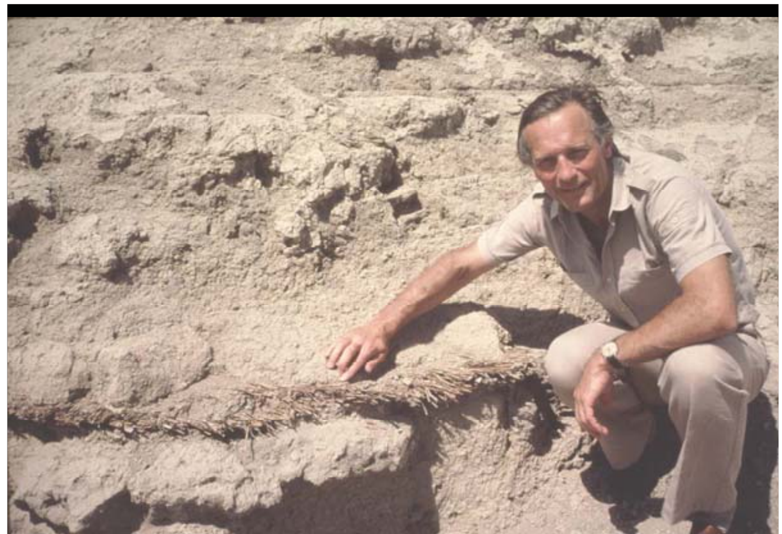
- Birds' nests
- Beaver dams
- Adobe bricks
- ➢Analogy with reinforced concrete?



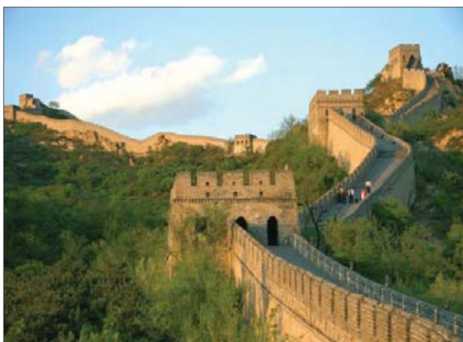
Ziggurat of Aquar-Quf, near Bagdad

~ 1500 BC

Now 45 m high (originally ~ 87 m)



Dr. J.-P. Giroud at Aquar-Quf circa 1980



Great Wall of China

Western wall

How I got into soil reinforcing and geosynthetics: Experience in Sweden, 1970-1975

Oleg Wager

(1915-1992)

The inventor

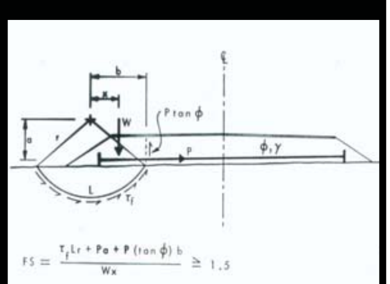
Bengt Broms

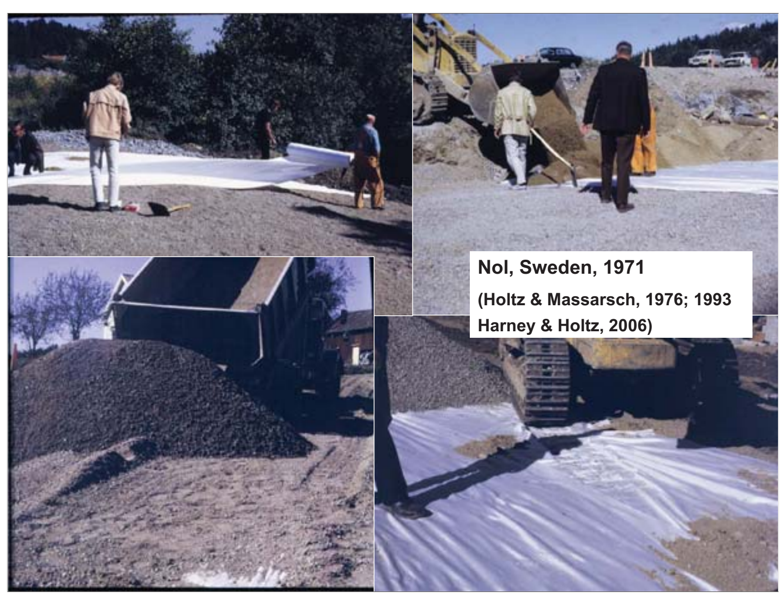
(1925 -   )

Boss & collaborator

Alvängen, Sweden 1966

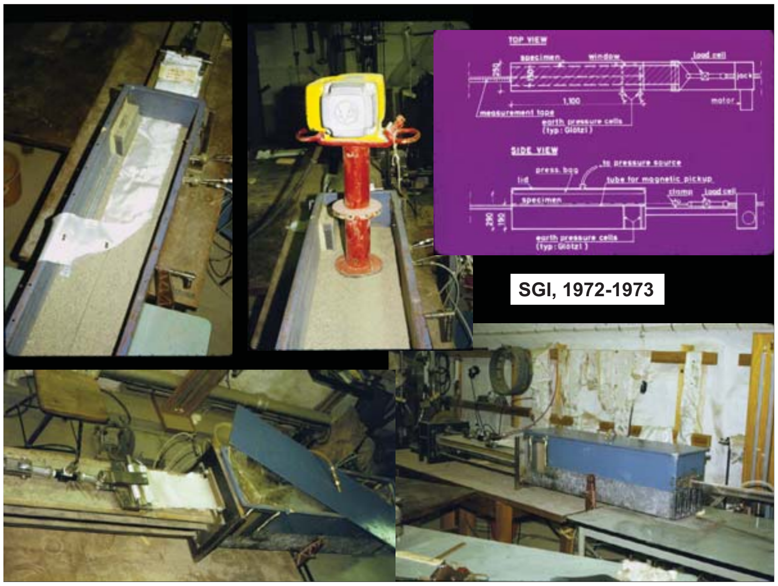
Nol, Sweden 1971



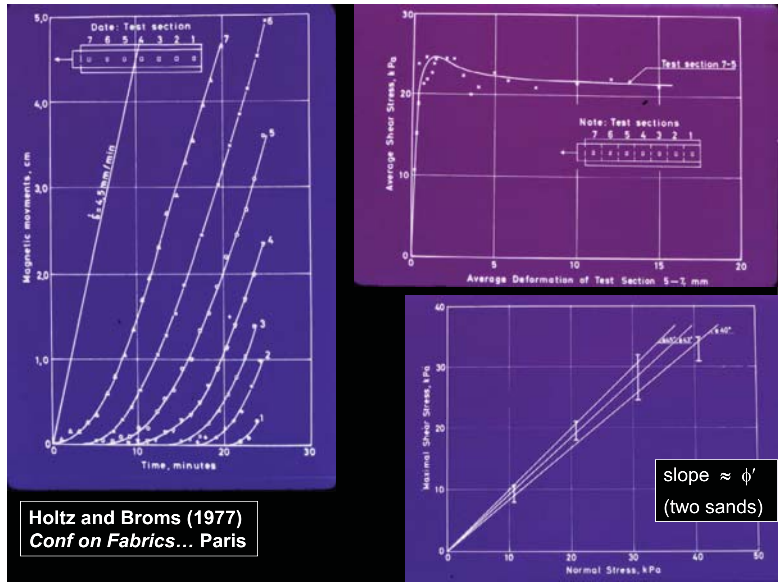


Nol, Sweden, 1971

(Holtz & Massarsch, 1976; 1993 Harney & Holtz, 2006)

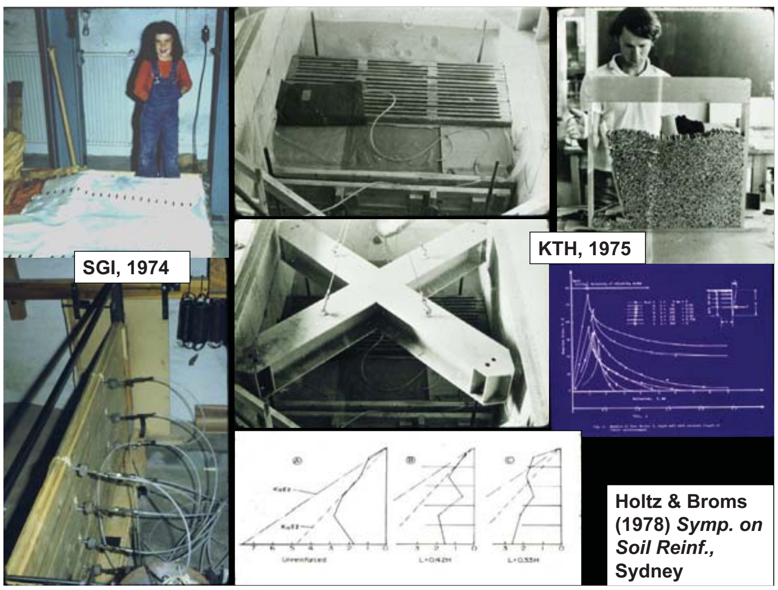


SGI, 1972-1973



Holtz and Broms (1977) *Conf on Fabrics…* Paris

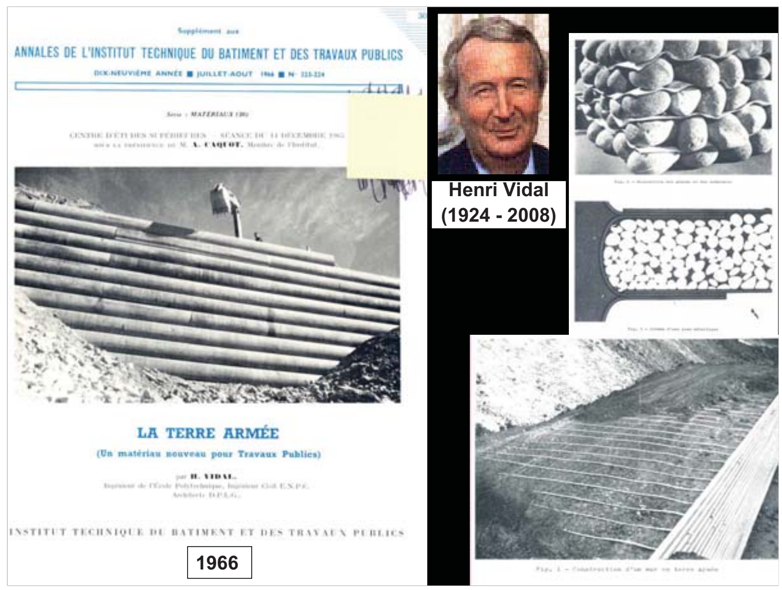
slope ≈ ϕ' (two sands)



SGI, 1974

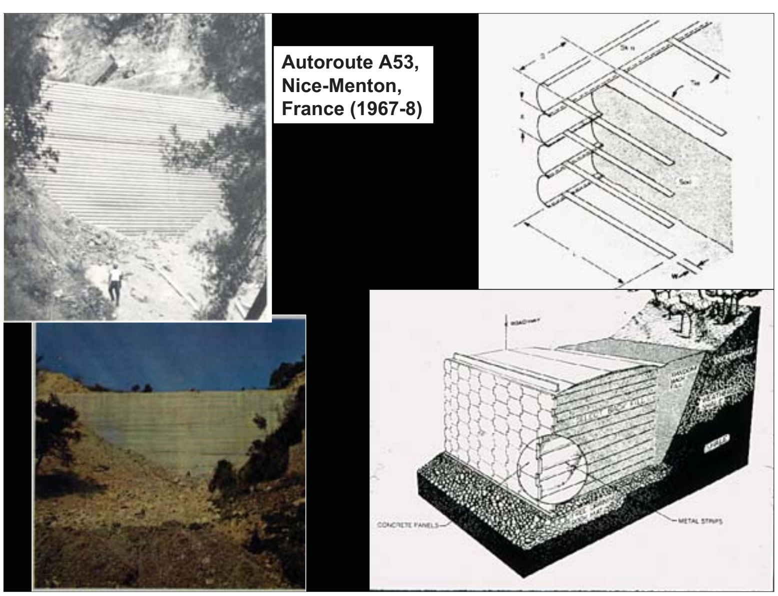
KTH, 1975

Holtz & Broms (1978) *Symp. on Soil Reinf.*, Sydney



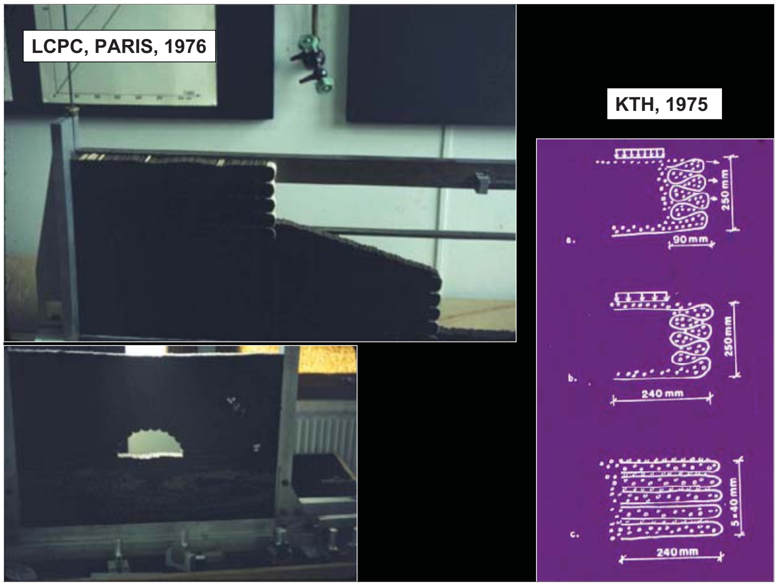
1966

Henri Vidal (1924 - 2008)

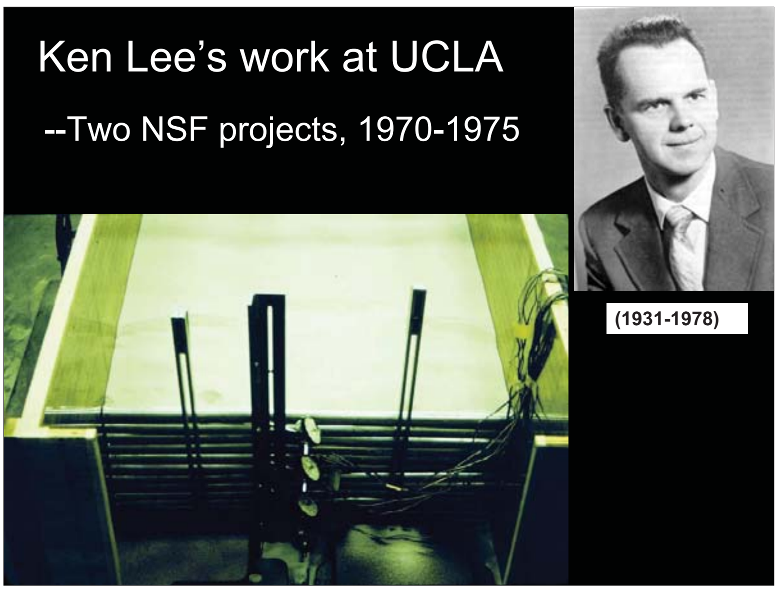


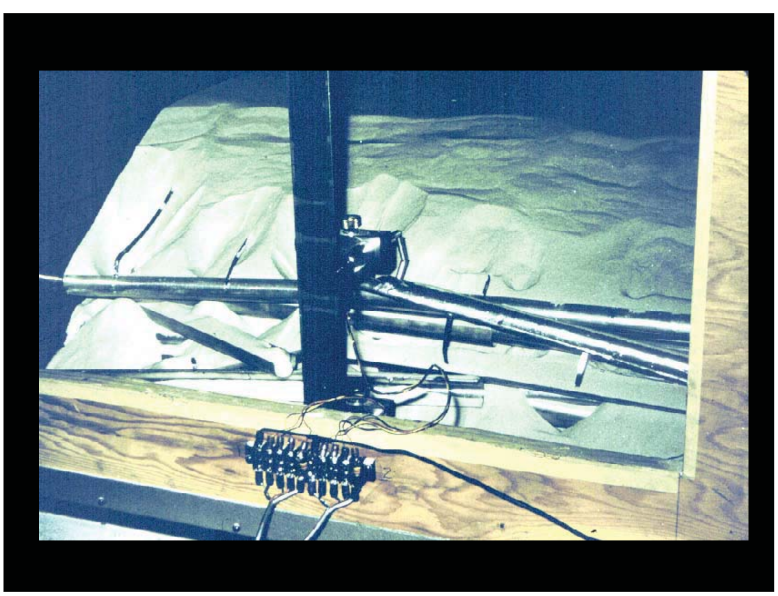
Autoroute A53, Nice-Menton, France (1967-8)

Terre Armée, near Paris, 1976


LCPC, PARIS, 1976
KTH, 1975
250 mm
90 mm
250 mm
240 mm
5 x 40 mm
240 mm


CALIFORNIA ROUTE 39 - AVALANCHE CLOSING THE ROADWAY
FIGURE 2
A view of the "reinforced earth" retaining wall used for this first-of-its-kind construction technique on the North American continent.
First RECo wall in the US, 1972: SR39, Angles Natl Forest, S. Calif.
CALIFORNIA ROUTE 39 - REINFORCED EARTH WALL NEARING COMPLETION
FIGURE 4

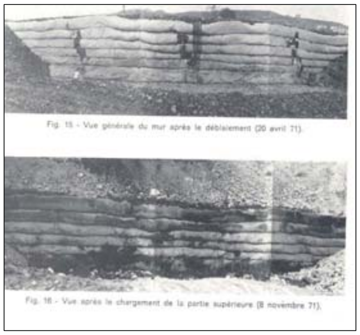

Ken Lee's work at UCLA
--Two NSF projects, 1970-1975
(1931-1978)




I-24, Tennessee

… and walls with geosynthetics in 1971-77

1. Bidim wall in France, 1971-1972, reinforced with a polyester needlepunched nonwoven, 300 g/m^2

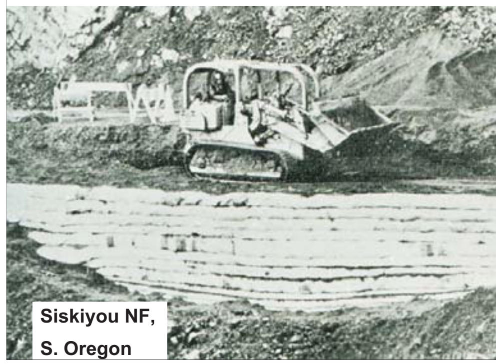


Puig & Blivet (1973)
Bull. liaison Labo. Cent. P. et Ch.

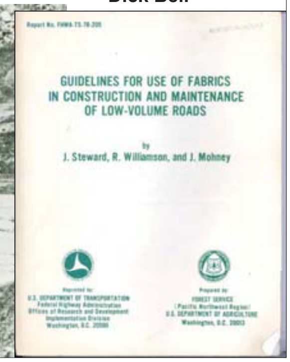
2. USFS walls in Oregon and Washington, 1972-1975

USFS: J. Steward, J. Mohney, B. Vandre
OSU: Prof. J. R. Bell

Dick Bell



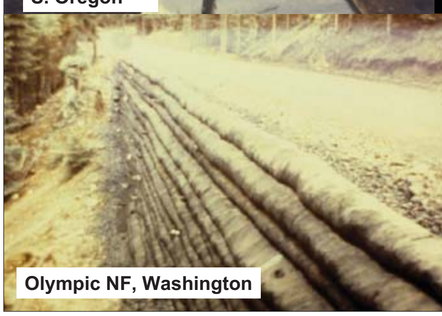
Siskiyou NF, S. Oregon


GUIDELINES FOR USE OF FABRICS IN CONSTRUCTION AND MAINTENANCE OF LOW-VOLUME ROADS

by

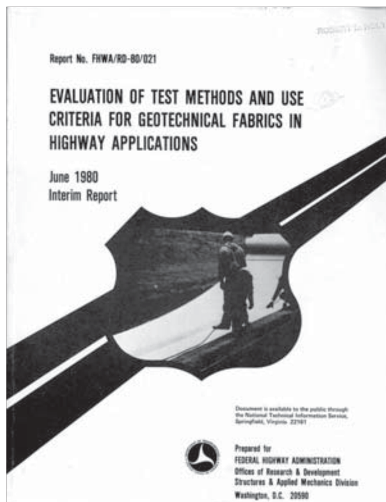
J. Steward, R. Williamson, and J. Mohney

Siskiyou NF, S. Oregon



Olympic NF, Washington

Siskiyou NF, S. Oregon


Report No. FHWA/RD-80/021

EVALUATION OF TEST METHODS AND USE CRITERIA FOR GEOTECHNICAL FABRICS IN HIGHWAY APPLICATIONS

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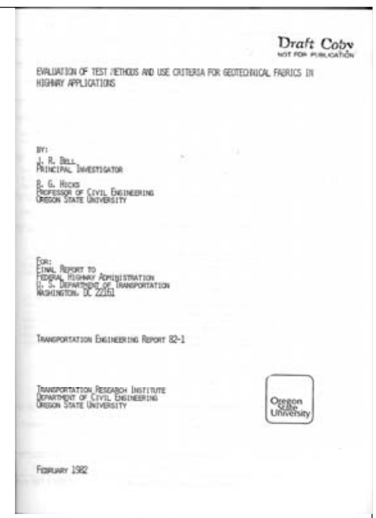
Interim Report: FHWA/RD-80/021 (1980)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EVALUATION OF TEST METHODS AND USE CRITERIA FOR GEOTECHNICAL FABRICS IN HIGHWAY APPLICATIONS

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FOR:
FINAL REPORT TO
FEDERAL HIGHWAY ADMINISTRATION
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WASHINGTON, DC 22151

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DEPARTMENT OF CIVIL ENGINEERING
OREGON STATE UNIVERSITY

FEBRUARY 1982

Draft final report, 1982 (never published by FHWA)

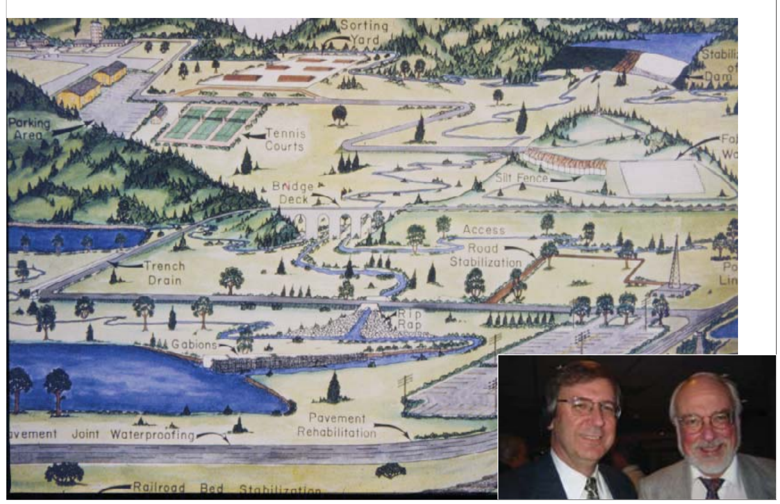
FHWA geosynthetics courses (~1978 -)

- Started by Al Haliburton, Okla St. U.
- Second contract BRC & RDH
- ~150 courses in most states, etc
- Significantly increased use and improved state highway specs and practice

Al Haliburton
(1935-1981)

Barry Christopher (left) and Bob Holtz, Co-editors of Geosynthetics

Cover of Christopher and Holtz (1983) *Geotextile Engineering Manual*, FHWA, FHWA-TS-86/203, 1044 pp.

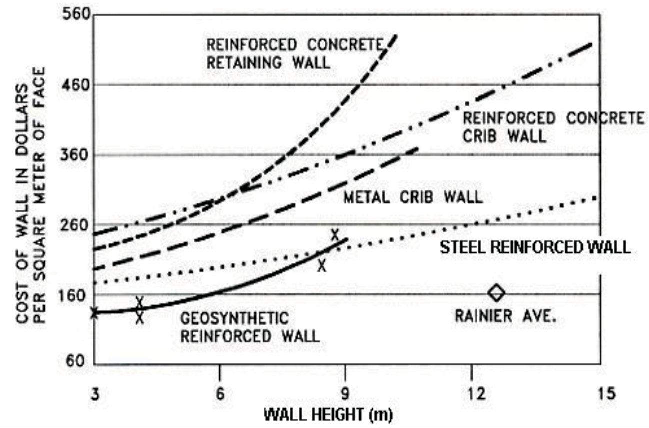


My plan:

1. Introduction
2. Reinforced soil—a historical perspective
3. **Advantages and basic behavior of GRS**
4. Design
5. Properties
6. Things we need still need to know and do—technical and professional issues
7. Successful examples
8. Final remarks

Advantages…

1. Cost:



Other advantages besides cost…

2. Flexibility
   - Settlement tolerance (∴ ¢¢ foundations)
   - Easy to change alignment, grade
   - Seismic stability
3. Simple, rapid construction
4. Attractive facing systems including “green” facings

Advantages (cont.)

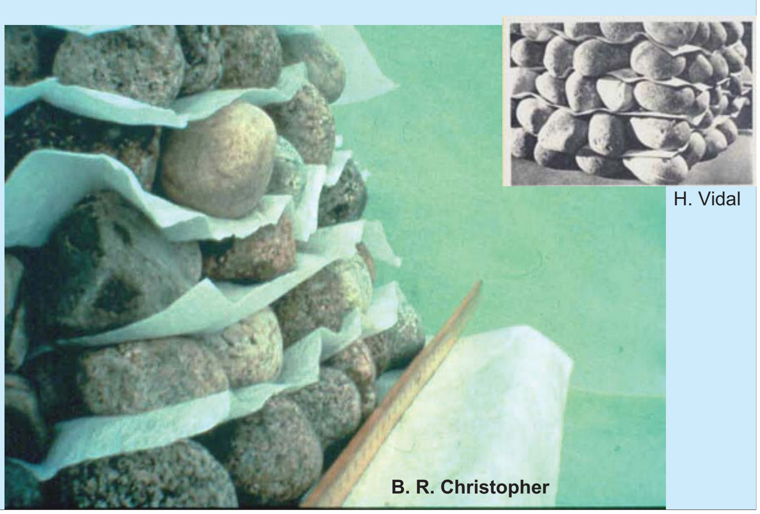
5. Steeper slopes
   – Cohesive >2:1
   – Granular > angle of repose
6. Increased safety

For the same calculated FS, lower probability of failure (reliability greater) for a reinforced steeper slope than an unreinforced flatter slope

(Cheng & Christopher, 1991).

Why do we still design/construct unreinforced soil slopes?

Basic behavior…



H. Vidal

B. R. Christopher

B. R. Christopher

Glenwood Canyon, Colo.

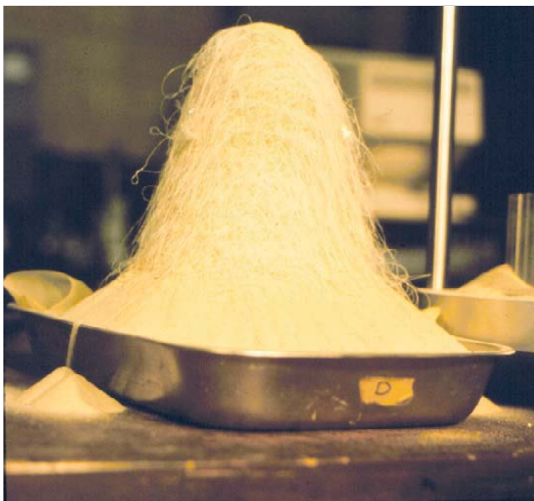
Test walls, 1982-1990

T. M. Allen

Bob Barrett, Colorado

Conclusions…

- Stress at face of wall/steep slope v small
- Therefore, face is only "local"… just necessary to hold soil between layers
  - not necessary to be structural, heavy, clunky (…unless the S_v is large.)
  - Japanese experience with EQs?



Fundamental studies on Texsol (1988-92)
Kim Wargo-Levine and Shaun Stauffer, UW

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Design

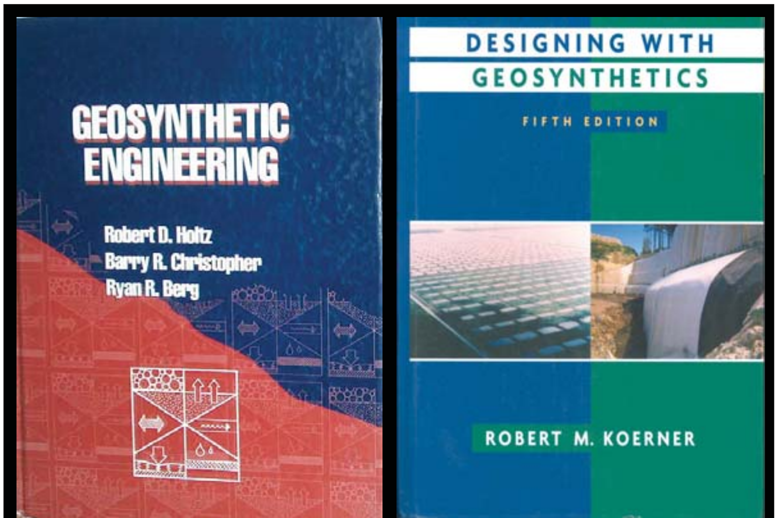
Background (historical-traditional approaches)

- GRS *walls*: Combination of conventional EP theory (Rankine) and Terre Armée
  - Same failure modes (*rupture, pullout, creep of reinforcement*)
  - Design approach of Ken Lee (UCLA) and Dick Bell (OSU-USFS)

  → "Tieback wedge" approach
  - Very conservative
- GRS *slopes*: Used classical slope stability analyses + "tieback" forces
- *Question*: What's the difference between a GRS slope and a very steep GRS slope?
- When does a "very steep slope" become a "wall"??
- Does the soil know the difference?

Design…

- *Koerner*: Our design approaches depend on traditional geotech designs for slopes and retaining walls…and on the way we teach these subjects in our graduate courses…HAS NOTHING TO DO WITH REALITY!
- So, let's see what the "experts" say about this…



BiTech Publishers, Richmond, BC

Prentice-Hall

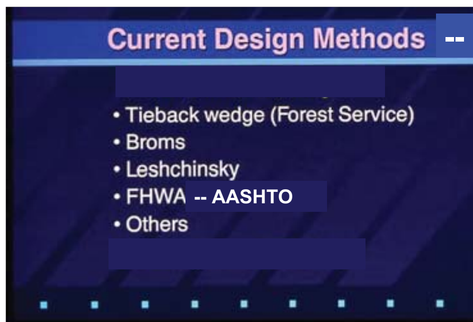
DESIGN: GRS "walls"

- External stability – conventional
  - Bearing capacity, OT, sliding, overall slope
- Internal stability – several approaches
- Drainage
- Seismic design
- Material properties (*next section*)
  - Soil
  - Geosynthetic
  - Facing

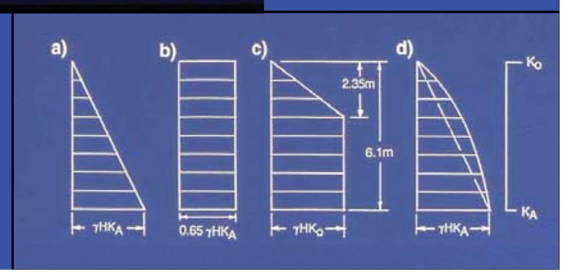
External stability

Roseburg, Ore.

R. R. Berg



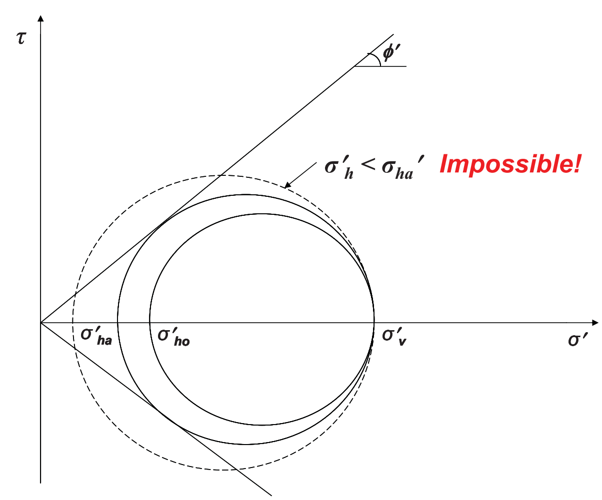
-- Internal stablilty



Empirical development of state of stress:

$$K_h = \frac{\sigma_h}{\sigma_v} = \frac{\text{Measured}}{\gamma h}$$

- Relate to K_a calculated from knowledge of ϕ'
- <u>Problem</u>: Measured K_h often less than K_a!
- Impossible!



Field meas vs. theory? Why is $K_h << K_a$??

- **Properties**
  - MFEs curved, so ϕ' >> higher at low γh or σ_c
  - $\phi'_{\mathbf{TRIAX}} << \phi'_{\mathbf{PS}}$
  - At field densities, high ϕ'
- **Rankine theory** violated by presence of reinforcement (Boyle, 1995, PhD thesis, UW)
- **Apparent cohesion**
  - *"…a little c goes a long way!!"* …but always there??
- **Field meas ??**
  - Interpretation problems
  - Anomalies
  - etc etc..

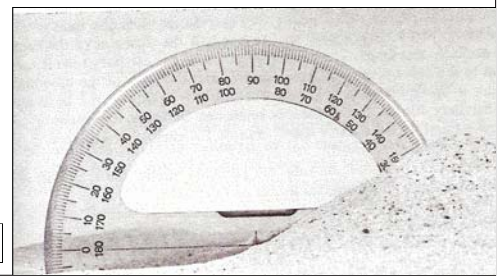
Design: GRS *slopes…*

Combination of classical slope stability analyses
+ "tieback" forces

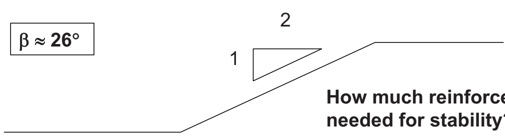
Consider
- how granular slopes actually fail
- how stability analyses are performed.

Start w/ a sand at its angle of repose and then increase the slope angle…

Holtz & Kovacs (1981)

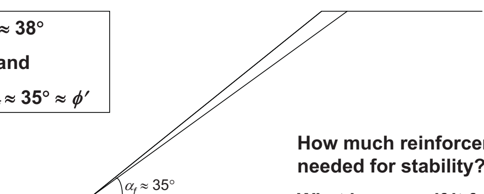


$\beta \approx$ **26°**



How much reinforcement is needed for stability??

$\beta \approx$ **38°**
Sand
$\alpha_f \approx$ **35°** $\approx \phi'$



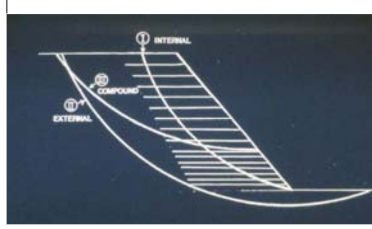
How much reinforcement is needed for stability??

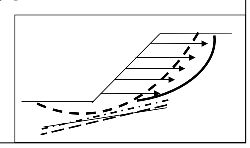
What happens if it fails?

➢**Richard Jewell and the pullout paradox…**

GRS slopes:
Design approaches and procedures

- Sliding wedge
  - One plane
  - Bilinear
- Circular arc
- Log spiral
- Murray
- Schneider & Holtz
- Leshchinsky et al.
- Jewell
- Schmertmann et al.
- Verduin & Holtz
- Others?





➢For stability analyses, several commercial and govt-developed programs have subroutines for GRS

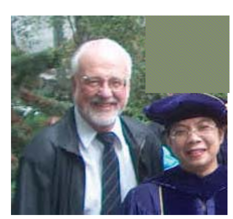
PCSTABL4	New Janbu
STABGM	Tenslo1
XSTABL	Strata Slope
UTEXAS3	RSS
GSLOPE	ReSSA

- ➢ $\approx$ OK
- ➢ See Duncan and Wright (2005) Chap 8

UW Research on GRS Walls:
Analytical (FLAC)

1. Wei-Feng Lee (PhD) -- Analysis of GRS walls; develop working stress analysis
2. Fadzilah Saidin (PhD) -- back analysis of an instrumented full scale GRS wall with poor draining backfill on soft soil

Wei Lee



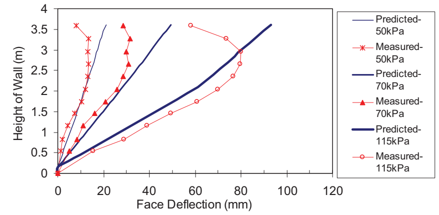
Fazee Saidin

1. Wei Lee (PhD) -- Analysis of GRS walls; develop working stress analysis
   - Model calibrated with field/lab data (Rainier Ave. wall)
   - PS ϕ' & modulus @ low $\sigma_c \rightarrow$ correct dilation angle
   - Class A predictions of three RMC test walls; ~ good agreement

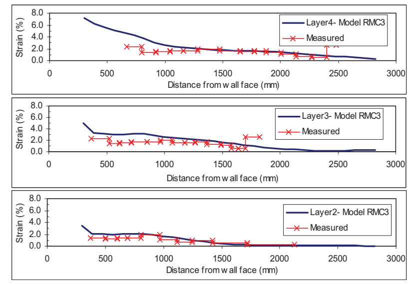
Conclusion: Both external and internal performance can be reproduced, ***IF*** :

- Correct material properties
- Boundary conditions correctly simulated

Wall Deflection – Wall 1



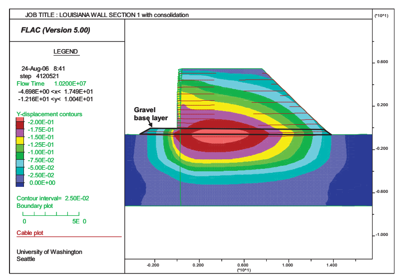
Reinforcement Strain - Wall 3
(50 kPa surcharge)



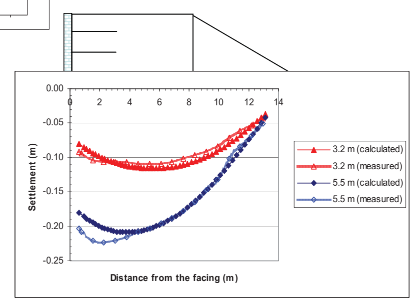
W.-F Lee (2000) PhD Thesis UW

2. Fazee Saidin (PhD) -- back analysis of an instrumented full scale GRS wall with poor draining backfill on soft soil

- Instrumented 6 m LTRC wall
- Numerical simulation (FLAC) of GRS wall on soft foundation
- considered settlement, infiltration, compaction, etc., effects



Some results--settlements

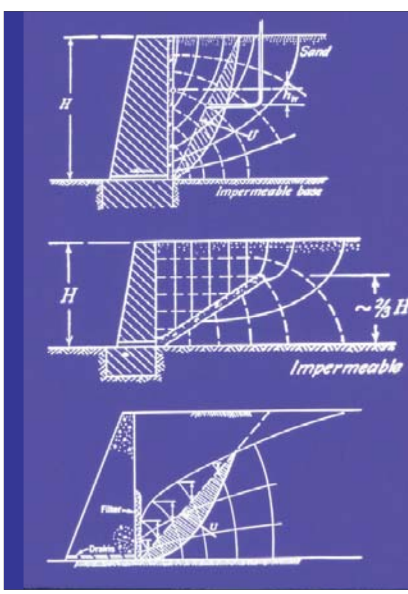


F. Saidin (1997) PhD Thesis, UW

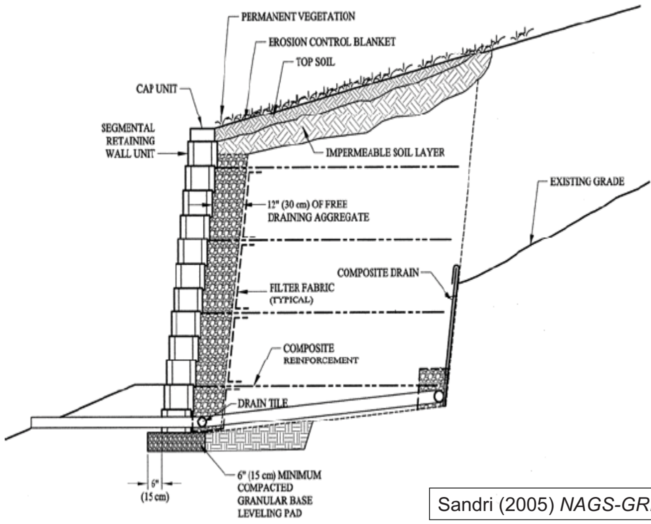
Design recommendations

- Traditional design methods ≈ OK for GRS walls on soft foundations
- Reinforced base layer $\rightarrow$ more uniform settlements
- Traditional settlement analysis ≈ OK
- Rate of construction important
- Adequate provisions for drainage critical

DRAINAGE! DRAINAGE! DRAINAGE!



Terzaghi (1943)
Theoretical Soil Mechanics



Sandri (2005) *NAGS-GRI19*

Other approaches to design:

- Composite material approach
  - UC Davis 1970s
  - Lee et al. (2007) *Proceedings of Geosyn. 2007*
- K-Stiffness method
  - Empirical – many case histories
  - Independent of reinforcing material
  - More accurate estimate of reinforcement loads
  - Step-by-step design procedures developed with a limit states design approach consistent with current design codes (i.e., LRFD)

Allen, Bathurst, Holtz, Lee, and Walters (2003) CGJ and (2004) JGGE

So, what to do for design of GRS ?
If you want to use traditional LE methods…

1. Use correct soil properties: γh + ϕ'_{PS} (not so easy)
   - not many PS devices available
   - hard to conduct triax/PS tests at low confining pressures
   - Use correct dilatancy ∠ (…important if want to do advanced modeling, e.g., with FLAC…and you want the correct answer!)
2. For internal stability of steep GRS slopes, design as…..well, a very steep slope.

   As slope angle increases → more or stronger reinforcing
   - Use SN or tieback programs…w/ adjustments for geometry and properties of reinforcement (??)
   - See Pockoski & Duncan (2000) "Comparison of Computer Programs for Reinforced Slopes," Center for Geotech Practice & Research, Va Tech

Traditional LE methods (cont.)

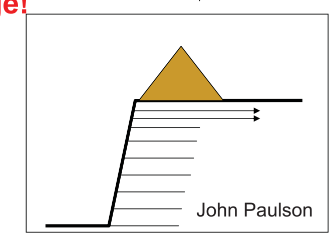
3. Use thin layers of weaker reinforcing -- ¢¢, and better face control
4. Pullout? Not a problem—based on our research at SGI, KTH (described earlier)
   - Geosynthetic will rupture before it pulls out
   - If a problem, easily taken care of in design
5. …and don't forget:

Drainage! Drainage! Drainage!

____

Also, try K-Stiffness Method*

**Let us know how it works*



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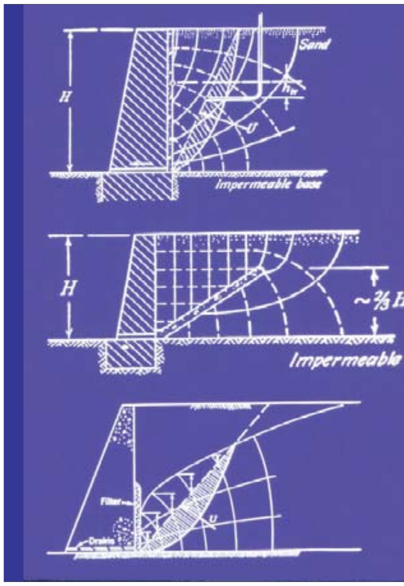
Material Properties

- ***Soils***
- ***Geosynthetics***
- ***Facing***

Soil Properties:

As usual…

- Use clean granular backfill
- ReCo/FHWA specs
- Foundation/slope



Terzaghi (1943)
Theoretical Soil Mechanics

Drainage,
drainage,
drainage!

This is a DESIGN and CONSTRUCTION issue.

Material Properties (cont.)

GEOSYNTHETIC PROPERTIES:

- ***Tensile strength***
- ***Soil-geosynthetic friction***
- ***Creep (?)***
- ***Durability***
- ***Installation damage***

2. Geosynthetic properties:

CRITERIA or PARAMETER	PROPERTY*
1. Design requirements:	
Mechanical	
Tensile strength/modulus	Wide width strength/modulus
Seam strength	Wide width strength
Tension creep	Tension creep
Soil-geosynthetic friction	Soil-geosynthetic friction angle **(?)**
Hydraulic	
Piping resistance	Apparent opening size
Permeability	Permeability/permittivity
2. Constructability Requirements:	
Tensile strength	Grab strength
Puncture resistance	Puncture resistance
Tear resistance	Trapezoidal tear strength
3. Durability:	
UV stability (if exposed)	UV resistance
Chemical and biological (if reqd)	Chemical and biological resistance

*All have ASTM standard tests.

UW Research on GRS Walls (1991 – 2007)

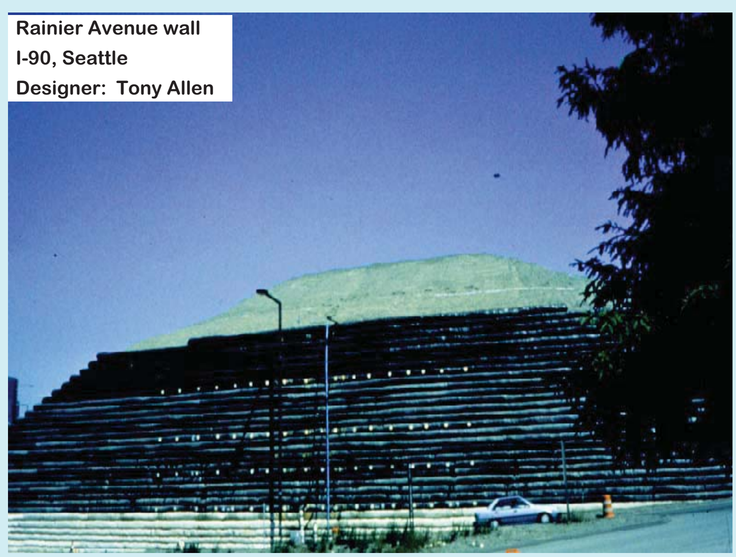
- **Analytical** (FLAC) -- already summarized
- **Experimental**
  - Stanley R. Boyle (PhD) – In-isolation and in-soil load-elongation tests; strain gages on geosynthetics

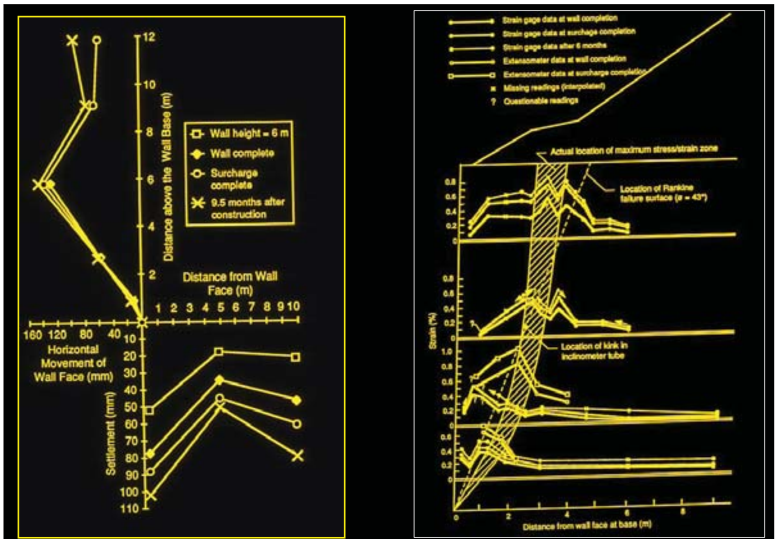
Sponsored by WSDOT
T. M. Allen, contract monitor

Stan Boyle

Tony Allen

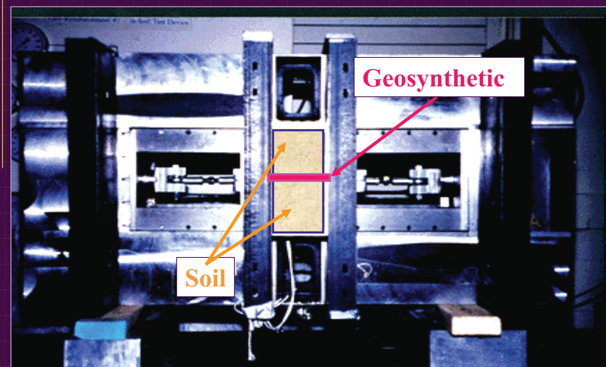
Rainier Avenue wall
I-90, Seattle
Designer: Tony Allen

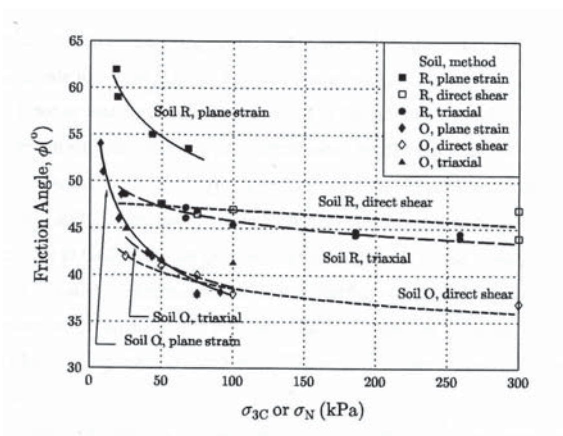




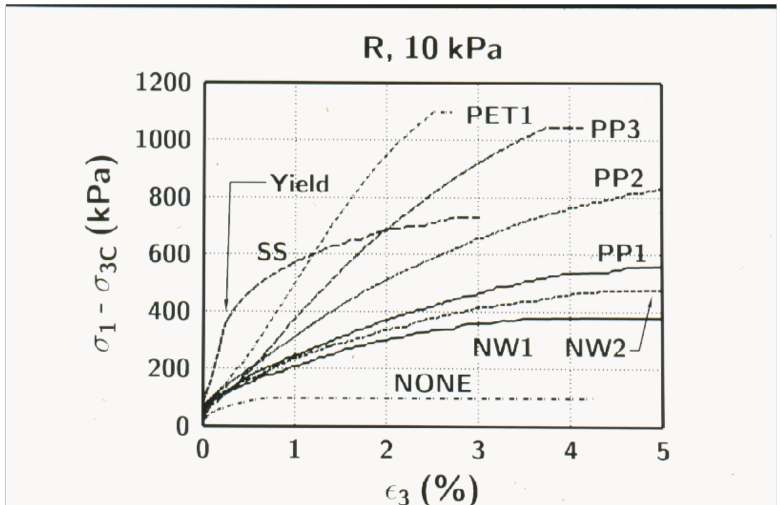
Allen, Christopher & Holtz (1992)

Unit Cell Device – Boyle (1995)



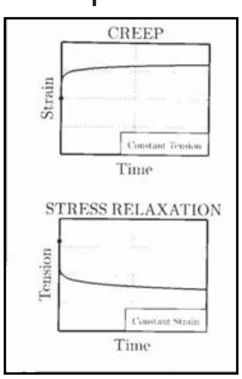


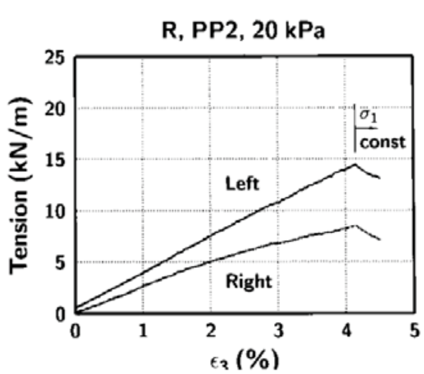
Boyle (1995) Fig. 6.5



Boyle (1995) Fig. 6.9b

Creep vs. Relaxation





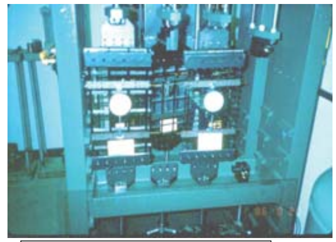
"Bottom line" for GRS wall designers

- For soil-geosynthetic interaction behavior, the induced reinforcement tension must be measured directly…otherwise you are just guessing the interaction parameters.
- The UCD is the only test that does this.
- Geosynthetics much more efficient reinf than steel, because strengths of both sand and geosynthetic are used more or less equally. With steel reinfd soil, steel does most of the work… & sand just goes along for the ride. Not so with geosynthetics.

"Bottom line" for GRS wall designers…

- Creep of GRS "walls" *not* really a problem *at working stresses*. When loading stops, GRS deforms as the geosynthetic relaxes. The GRS system is at equilibrium and no longer moves.
- Also shown by field measurements of real GRS walls [Rainier Ave wall; Norway steep slope (Fannin and Herman, 1990; Fannin, 2001)].

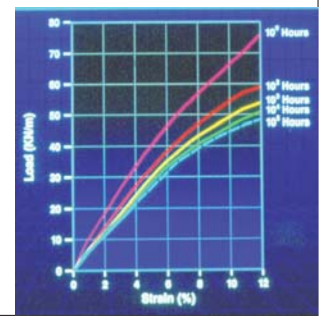
If you still think creep is a problem:

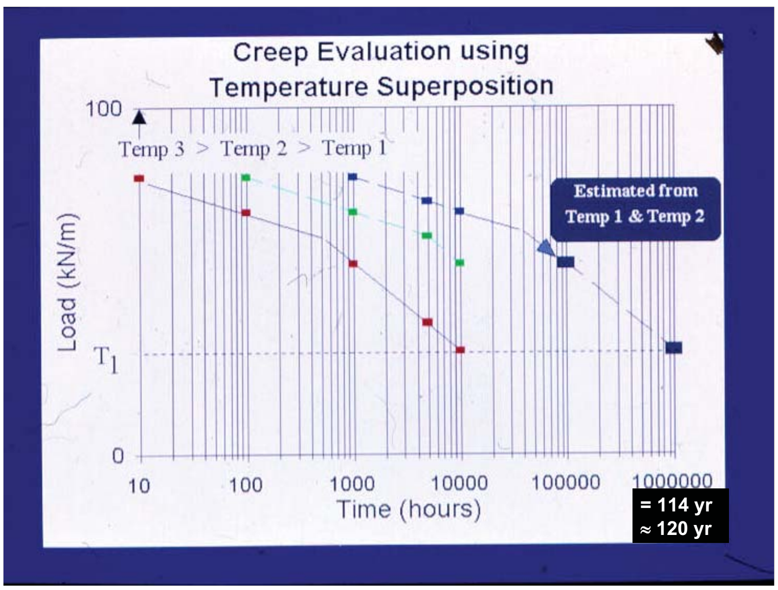


Unconfined creep test

In-soil creep rate??

Isochronous load vs. strain curves -- Geogrid (after McGown)





If you still think creep is a problem:

- See Bob Koerner, Grace Hsuan, and Scott Thornton
- Use Isochronous load vs. strain curves and time-temperature superposition; stepped isothermal method (SIM) Analysis -- ASTM D 6992
- Use BS 8006 (10 000 hr data → ≈ 120 yr)
- Jon Fannin: BS8006 procedure and AASHTO with

  RF_{CR} → ≈ **same T_{al} !**
- Finer grained backfills???? (*Avoid* if possible…)

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Things we need still need to know and do:

1. Technical

GRS is quite mature... but we could use:

- A simpler ("poor-man's") PS device…with *Δvol* measurements
- A seismic design procedure better than M-O pseudo-static….even though we know GRS structures are safer than conventional in EQs
- PBEE? (Most promising…)

Steve Kramer

Things we need still need to know and do:
2. Professional issues

1. **Too many failures!** Most due to
   - Poor quality backfill
   - Poor drainage; saturated backfill
   - Construction problems
   - Inadequate global or external stability
   - Unexpected surcharges
   - …and…
2. **Disconnect** between wall designer, geotech of record, and site civil

   …complicated by wall designs supplied by materials suppliers and distributors

Things we need still need to know and do:
2. Professional (cont.)

3. Other problems
- Lack of proper inspection
- No control of construction by designer
- Economic pressures
- "Value engineered" or "contractor supplied" designs, with no $$ for checking alternates by competent professionals
- Poor training for workers

Question: Is liability avoided by use of vendor-supplied designs?
- – If not, then why give away billable design hours?

➢ Fixing problems always more expensive than proper inspection and control by the designer…

Things we need still need to know and do:
2. Professional (cont.)

4. Jurisdictions that require a GRS "wall" design to be stamped by a registered structural engineer (who usually knows nothing about soil reinforcing and geosynthetics, and only a little about soils and drainage issues…and they are not responsible for construction inspection).

The result? Too many failures! Costly, potentially tragic, and not acceptable!

- How to fix this current state of affairs?
  G-I? ASFE? IGS? ISSMGE?
  Us as individuals?
- Many of these issues are not unique to GRS
- But they threaten a wonderful technology

…and a wonderful profession

Outline

1. Intro
2. Acknowledgements
3. Reinforced soil—a historical perspective
4. Advantages/disadvantages/ characteristics
5. Basic principles/behavior of GRS
6. Design
7. Properties
8. Things we need still need to know and do—technical and professional issues
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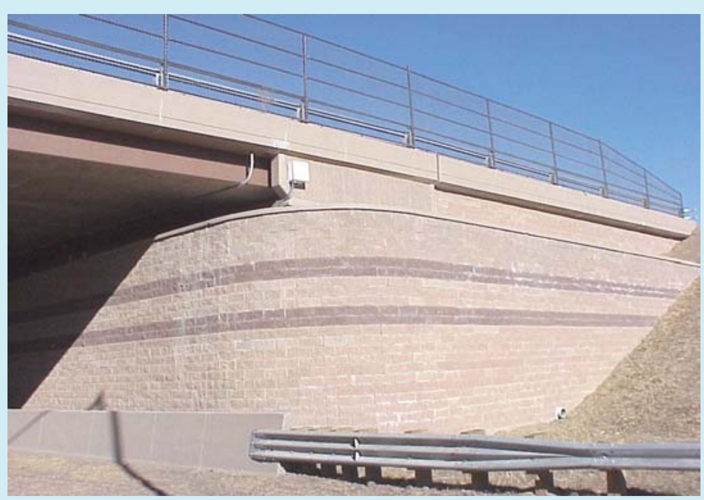
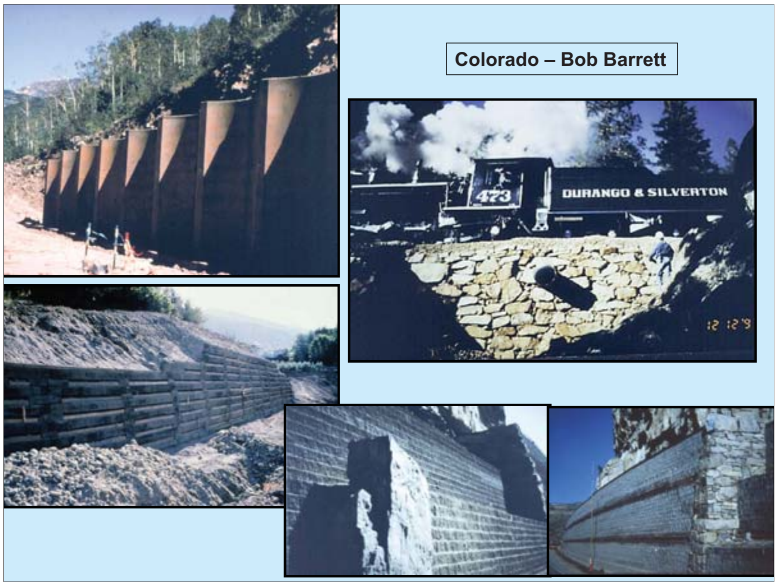


Founders Meadows Structure (I-25, Exit 184), near Denver, Colo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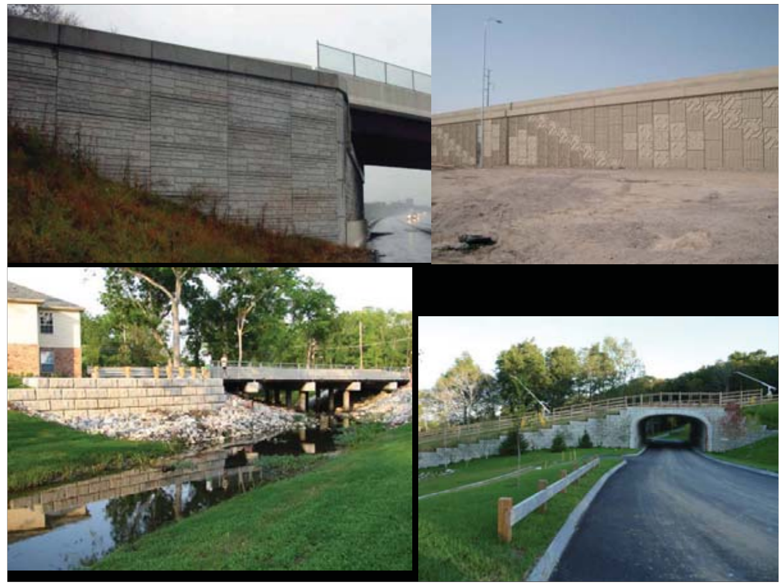

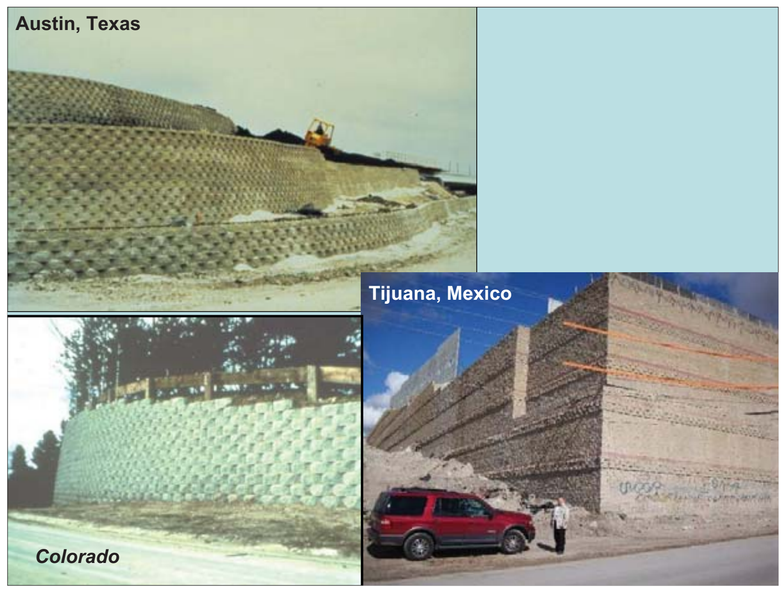
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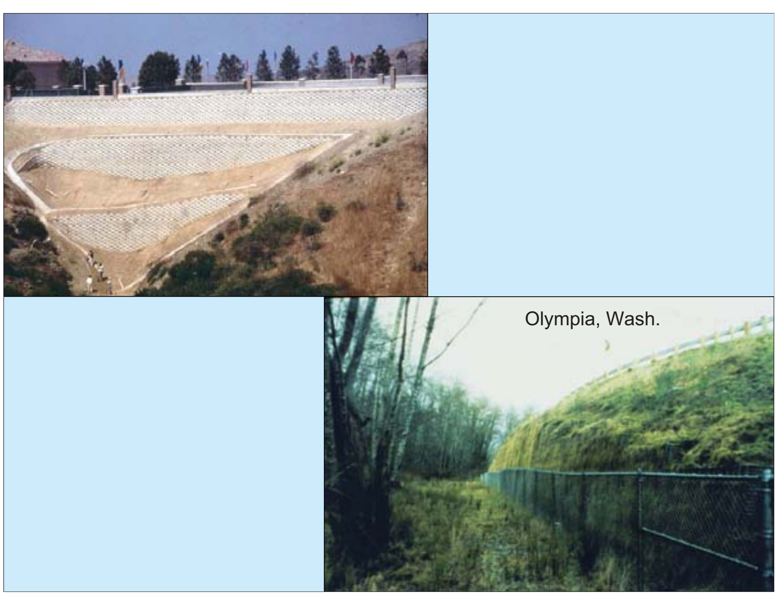
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Taiwan

Rudolph Bonaparte, Ph.D., P.E., NAE

Rudolph (Rudy) Bonaparte received a B.S. in civil engineering from the University of Texas at Austin (1977) and an M.S. (1978), and Ph.D. (1981) in geotechnical engineering from the University of California, Berkeley, where he studied as a National Science Foundation Graduate Research Fellow. For the past 25 years, he has been a Principal at Geosyntec Consultants, Inc., serving as President and CEO for the past 15 years. Geosyntec is an employee-owned 800-person consulting and engineering firm practicing in the environmental, geotechnical, water resources, and structural engineering disciplines.

Dr. Bonaparte has focused his professional engineering practice in the areas of geotechnical and geoenvironmental engineering; geological hazard evaluation and mitigation; and solid, hazardous, and low-level radioactive waste disposal facility design, permitting, and performance evaluation. He is the author or co-author of more than 50 peer-reviewed technical papers, several book chapters, and six major reports published by the U.S. Environmental Protection Agency, Federal Highway Administration, and U.S. Navy on topics related to his practice specialties. He has served on the editorial boards of the ASCE *Journal of Geotechnical and Geoenvironmental Engineering*, the journal *Geosynthetics International*, and the on-line *International Journal of Geoengineering Case Histories*. He is a registered professional engineer in 17 states, a Diplomate of the ASCE Geo-Institute Academy of Geo-Professionals, and a Board Certified Environmental Engineer by the American Academy of Environmental Engineers. In 2002, he served on the Board of Governors of the ASCE Geo-Institute.

Dr. Bonaparte was elected to the U.S. National Academy of Engineering (NAE) in 2007. In 2006 he was elected to the U.T. Austin CAEE Academy of Distinguished Alumni and in 2004 he was selected as Engineer of the Year by the Georgia Alliance of Professional Engineering Societies. He is co-recipient of the 2000 J. James Croes Medal from ASCE, the 1994 IGS Award from the International Geosynthetics Society, and the 1991 Award of Excellence from the North American Geosynthetics Society.

Dr. Bonaparte is active in service to his alma maters. He presently serves as the Chair of the External Advisory Councils to both the Department of Civil Architectural, and Environmental Engineering (CAEE) at U.T. Austin and the Department of Civil and Environmental Engineering at Cal Berkeley.

The Nineteenth Spencer J. Buchanan Lecture
Texas A&M University
November 11, 2011

COLD WAR LEGACY - DESIGN, CONSTRUCTION, AND PERFORMANCE OF A LAND-BASED RADIOACTIVE WASTE DISPOSAL FACILITY

by

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ABSTRACT

A mixed low-level radioactive waste (LLRW) and Resource Conservation and Recovery Act (RCRA) waste on-site disposal facility (OSDF) was constructed as part of the remediation of the U.S. Department of Energy Feed Material Production Center in Fernald, Ohio. The 56-acre OSDF is fully constructed, filled with waste, and closed. Post-closure monitoring is ongoing. This paper presents the design, construction, and performance of the OSDF. Waste acceptance criteria and waste placement requirements are described. Results from three sets of pre-design field and laboratory investigations are summarized. Currently available performance data for the OSDF's leachate collection system and leakage detection system are reported. Post-closure monitoring activities are briefly described. The value of this case study is in providing a detailed framework for the conceptual and detailed design of land-based disposal facilities for mixed LLRW and RCRA waste.

INTRODUCTION

This paper describes the background to, and design, construction, and performance of, a mixed low-level radioactive waste (LLRW) and Resource Conservation and Recovery Act (RCRA) waste on-site disposal facility (OSDF) at the U.S. Department of Energy (DOE) former Feed Material Production Center (FMPC) in Fernald, Ohio. The main focus of the paper relates to the approach developed by the authors to design the OSDF to achieve the DOE design life criterion of "1,000 years, to the extent reasonable and in any case for 200 years." Conventional RCRA land disposal facilities for both municipal and hazardous wastes typically consider a design life in the range of 50 to 100 years. The paper also highlights field and laboratory studies conducted in support of design, construction of the facility, and information generated to date on facility performance.

Significant portions of this Nineteenth Spencer J. Buchanan Lecture were taken from an earlier paper by Bonaparte et al. (2008). However, significant new information has been added to this lecture paper on historical perspective, Fernald site and OSDF background information, and OSDF performance data that have become available since preparation of the 2008 paper.

HISTORICAL PERSPECTIVE

The U.S. government established the Atomic Energy Commission (AEC) in 1946 shortly after the end of World War II. The AEC was given responsibility by Congress for the peaceful development of atomic energy and also for developing the country's nuclear weapons arsenal, taking over these responsibilities from the wartime Manhattan Project. From 1946 through 1991, development of America's nuclear arsenal was conducted against the backdrop of the "Cold War", the political, military, and economic competition and conflict between the "communist world", led by the Soviet Union, and the "western world", led by the U.S. A few of the key events of the Cold War and the years in which they occurred are given in Table 1.

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Table 1. Select dates in the Cold War between western and communist countries, 1946-1991.

Year	Event
1946	Churchill "Iron Curtain"
1947	Marshall Plan
1948	Berlin Blockade and Airlift
1950	Korean War
1961	Bay of Pigs, Berlin Wall
1962	Cuban Missile Crisis
1963	Kennedy "Ich bin ein Berliner"
1968	Soviets Crush Czechoslovakian Revolt
1972	Nixon-Brezhnev Détente
1982	Poland Solidarity
1986-1988	Reagan-Gorbachev Meetings/Treaties
1987	Reagan "Mr. Gorbachev, tear down that wall"
1990	Germany Reunited
1991	Soviet Union Dissolved

Through the AEC, and then DOE starting in 1977, the country developed 21 major facilities in 13 states, plus a number of smaller facilities, to perform research, production, assembly, and testing related to the U.S. nuclear weapons arsenal. This group of facilities became known as America's "nuclear weapons complex". Table 2 summarizes information about the major facilities in the complex. The facility that is the subject of this paper is the Fernald FMPC, discussed below.

Table 2. Major facilities in Cold War U.S. nuclear weapons complex. (www.em.doe.gov/pdfs/pubpdfs/linklegacy.aspx)

Step	Process	Major Sites
1	Uranium Mining, Milling, and Refining	Fernald, Middlesex, Weldon Springs, Oak Ridge, Portsmouth, Paducah, uranium mining and milling sites
2	Isotope Separation	Oak Ridge, Paducah, Portsmouth, Savannah River
3	Fuel and Target Fabrication	Savannah River, Fernald, Ashtabula, Hanford, Oak Ridge
4	Reactor Operations	Hanford, Savannah River
5	Chemical Separations	Hanford, Savannah River, Idaho NEL
6	Weapons Component Fabrication	Rocky Flats, Hanford, Los Alamos, Oak Ridge, Mound, Savannah River
7	Weapons Operations	Pantex, Oak Ridge, Mound, Kansas City, Pinellas, Sandia, Burlington
8	Research, Development, and Testing	National Laboratories: Los Alamos, Lawrence Livermore, Sandia Test Sites: Nevada Test Site, Bikini and Enewetak Atolls, Christmas and Johnston Islands, Amchitka Island, Tonopah Test Range, Salton Sea Test Base

FERNALD SITE AND OSDF BACKGROUND INFORMATION

The DOE FMPC was located on a 1,050-acre site in Fernald, Ohio, approximately 18 miles northwest of downtown Cincinnati. From 1951 to 1989, the facility was used for the processing of uranium ore to produce uranium intermediate products (i.e., uranium trioxide [UO_3] and uranium tetrafluoride [UF_4]) and high-purity uranium metal (99.3% U_{238} and 0.7% U_{235}) for shipment to other sites in the nuclear weapons complex where the intermediate products and high-purity metals were further processed to produce nuclear weapons. Figure 1 shows uranium ore being unloaded at the Fernald site. Figure 2 shows several of the operating plants at the site where uranium ore was processed to produce uranium intermediate products. Some of these intermediate products were sent to the DOE Paducah (Kentucky) gaseous diffusion plant for enrichment to produce weapons-grade uranium (i.e., uranium highly enriched in the fissile U_{235} isotope). One of the principal high-purity metal products produced at Fernald was machined-metal uranium fuel cores (Figure 3). These fuel cores were shipped to nuclear reactors at the DOE Savannah River and Hanford sites for neutron bombardment to produce weapons-grade plutonium (Pu_{239}). During its operating life, the Fernald facility delivered to other facilities within the DOE complex approximately 190,000 tons of high-purity machined uranium metal products, and 38,000 tons of intermediate products, primarily uranium trioxide and uranium tetrafluoride. Employment at the plant peaked in 1956 at nearly 2,900 employees, and at the time of shutdown, the facility contained more than 220 buildings.

Figure 1. Unloading uranium ore from railcars at Fernald site. Source of uranium ore was former Belgian Congo, now Democratic Republic of the Congo. (www.lm.doe.gov/ohio)

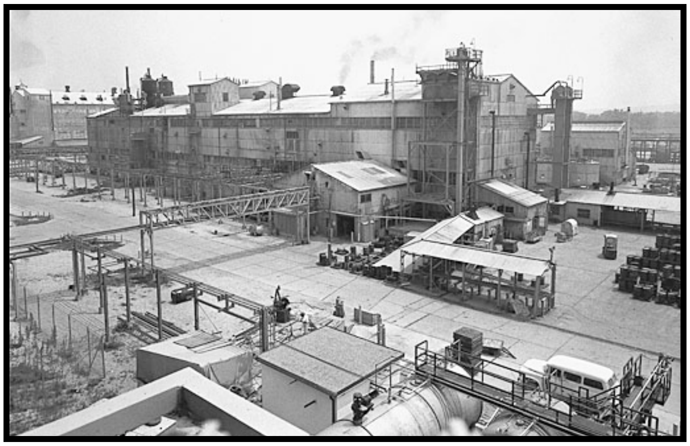

Figure 2. Plants 2 & 3 processed uranium ore to produce concentrated uranium trioxide, which was shipped to either the gaseous diffusion plant at Paducah, Kentucky, or Plant 4 for further processing. (www.lm.doe.gov/ohio)

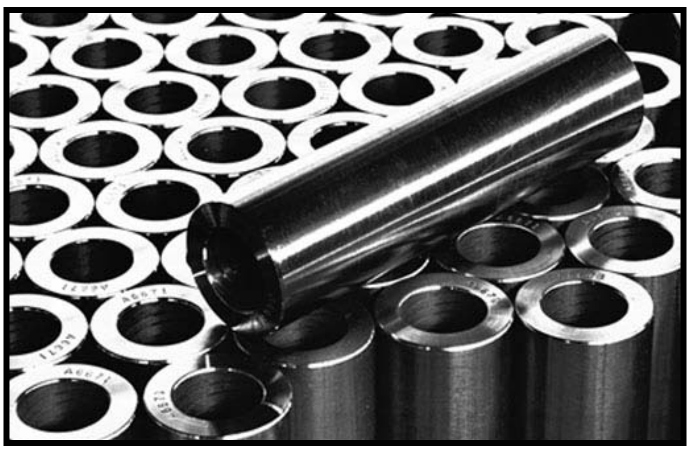

Figure 3. High-purity uranium fuel cores produced at Fernald. (www.lm.doe.gov.ohio)

In 1989, DOE ceased uranium processing operations at Fernald as the Cold War came to a close and the government's need for Fernald's uranium intermediates and products fell sharply. Fernald's environmental legacy at the time operations ceased included 31 million pounds of nuclear metals, nearly 260,000 cubic yards of low-level radioactive solid waste, 1 million tons of waste pit sludges, 2.5 million cubic yards of soils impacted by LLRW and RCRA hazardous constituents, building debris, non-radiological solid waste, and contaminated groundwater.

Clean-up and restoration of the Fernald site was carried out under the remedial process detailed in Title 40 of the U.S. Code of Federal Regulations (CFR), Section 300, which codifies the requirements of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA). Under this process, the site's environmental legacy was divided into a number of large projects to be implemented under five CERCLA Operable Units (OUs). Table 3 summarizes the major environmental remediation and restoration projects at the Fernald site. The total reported cost for remediating the site is $4.4 billion.

Table 3. Major remediation and restoration projects conducted at Fernald site. (www.lm.doe.gov/ohio)

Project Name	Description	Status
Nuclear Material Disposition Project	Characterize, package and ship 31 million pounds of nuclear product to other DOE sites or sell to private sector	Complete – 2002
Silos 1 and 2	Excavate and chemically stabilize 8,900 yd^3 of radium-containing LLRW and transport to DOE Nevada Test Site for disposal	Complete – 2006
Silo 3	Excavate 5,100 yd^3 of LLRW and transport to off-site commercial LLRW facility for disposal	Complete – 2006
Waste Pits	Remediate the contents of six waste pits up to 30 feet deep. Excavate and dry 1 million tons of uranium and thorium containing sludge and ship (9,100 rail cars) to Utah LLRW disposal facility	Complete – 2005
Decontamination & Demolition	Decontaminate and demolish 259 former production facilities, buildings, support facilities, and associated structures	Complete – 2006
Off-site Waste Disposition	Off-site disposal of 213,000 tons of contaminated soil and demolition debris not meeting the OSDF WAC (2,000 rail cars)	Complete – 2006
Soil Remediation and OSDF	Build OSDF and fill with contaminated soil and demolition debris meeting OSDF WAC. Certify site as "clean".	Complete – 2006
Aquifer Restoration	Groundwater cleanup of Great Miami Aquifer, one of the largest sole-source drinking water aquifers in the U.S., to reduce total uranium (U_T) concentrations to less than 30 mg/kg. At year-end 2010, 30 billion gallons of water have been extracted and treated and more than 5 tons of U_T removed.	Ongoing
Restoration	Ecological restoration of 900 acres of the site.	Complete – 2007

The U.S. EPA Record of Decision (ROD) for OU2 (DOE, 1995b) at the Fernald site addressed decommissioning and demolition (D&D) of buildings and excavation of soils impacted by LLRW and RCRA hazardous constituents at concentrations above clean-up criteria. The OU2 ROD allowed wastes from these sources to be placed in an OSDF if the wastes satisfied certain waste acceptance criteria (WAC). DOE estimated the quantity of such materials at 2.5 million bank cubic yards.

Conceptual design of the OSDF was developed through a CERCLA feasibility study (FS) and ROD (DOE, 1995a,b).

DOE next performed a site pre-design investigation (DOE, 1995c) and established functional requirements for the OSDF (FERMCO, 1995). Design criteria were then developed (Geosyntec, 1997a), and the detailed design was completed (Geosyntec, 1997b). Construction of the OSDF commenced in May 1997, first waste was placed in November 1997, and the facility was completely filled and closed in October 2006. Upon completion, the OSDF contained 2.96 million in-place cubic yards of waste.

The DOE-reported actual cost for construction, filling, and closure of the OSDF is $224 million. This cost also includes engineering, design, construction management and quality control/quality assurance (QC/QA), waste placement and compaction, and construction of the leachate collection and transmission (including valve houses and pump station), stormwater management, and environmental monitoring systems. The cost excludes waste pre-processing, waste transport from the source to the OSDF, operation of the LCS and groundwater monitoring system, and site administration and management. The reported cost equates to: $4 million per acre for the eight-cell, 56-acre lined footprint of the OSDF; $3 million per acre for the 74 acre footprint of the final cover system; $76 per in-place cubic yard of waste; and $45 per estimated ton of waste.

Figure 4 presents an aerial photo of the Fernald site in 1996, prior to the start of OSDF construction in the field on the right side of the photograph (east side of facility). Figure 5 shows the OSDF in 2002 with Cells 1 and 2 closed, Cell 3 being closed, Cells 4 and 5 being operated, and Cells 6 and 7 in construction. Figure 6 shows the site in October 2006 with site remediation complete, the OSDF filled, and Cell 8 in the final stages of closure. Post-remediation land use at the Fernald site includes 400 acres of woodlands, 390 acres of prairie, 140 acres of wetlands and surface waters, and 97 acres for the OSDF and various infrastructures.

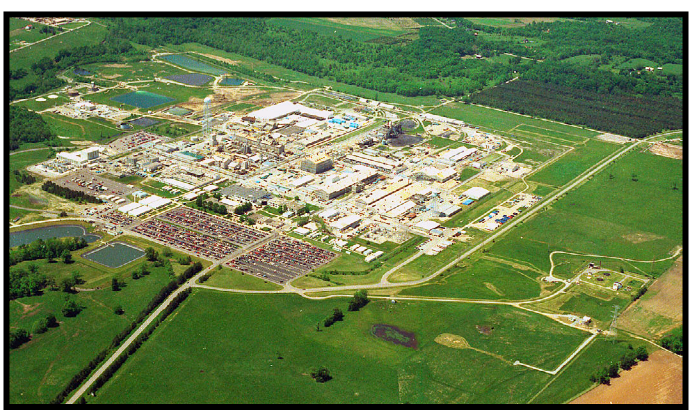

Figure 4. Aerial photo of Fernald site, June 1996. Future OSDF location is open field on right side photo.

Figure 5. Aerial photo of OSDF in 2002 with various cells being constructed, operated, and closed.

Figure 6. Aerial photo of Fernald site in final stages of closure, October 2006.

Subsurface conditions at the OSDF site are illustrated in Figure 7. Preconstruction ground elevations ranged from El. 618 feet (ft.) National Geodetic Vertical Datum (NGVD) on the northeast corner of the site to El. 586 ft. in the southwest corner. Brown and gray glacial till form the surficial stratigraphic unit at the OSDF site. Brown till, covered by a thin topsoil veneer, had typical pre-construction thicknesses of 10 to 15 ft. within the OSDF footprint. As shown on Figure 8, a portion of this material was removed to achieve the OSDF design base grades. The thickness of the underlying gray till ranges from about 45 ft. at the OSDF north end to 15 ft. at the south end. The till is underlain by sand and gravel of the Great Miami aquifer, an important source of drinking water for the region. This sand and gravel unit is approximately 200 ft. thick beneath the OSDF and is, in turn, underlain by shale and fossiliferous limestone with essentially horizontal bedding. Information on the geotechnical and hydrogeological characteristics of the soil units underlying the OSDF is presented in Table 4.

TABLE 4. Geotechnical characteristics of soil units underlying OSDF.

Unit and Description	Liquid Limit/ Plasticity Index (ranges)	Gravel/Sand/ Silt/Clay (%)
Brown Till: Predominantly silty low-plasticity clay (CL), with pockets of high plasticity clay (CH) and silt (MH), pockets of clayey sand (SC), contains scattered gravel ($k=1 \times 10^{-8}$ – 6×10^{-6} cm/s).	21 – 50/ 7 – 32	0 – 20/ 1 – 40/ 30 – 60/ 20 – 60
Gray Till: Predominantly sandy lean clay (CL) with lenses and pockets of sand (SW), and clayey sand (SC), contains scattered gravel ($k=1 \times 10^{-8}$ – 3×10^{-8} cm/s).	19 – 33/ 5 – 7	0 – 31/ 1 – 39/ 28 – 79/ 18 – 58
Great Miami Aquifer: Sand and gravel mixtures, very stiff to hard.	NP	0 – 35/ 57 – 91/ 4 – 10/ 0 – 3

Note: Information summarized from Parsons, 1995. Hydraulic conductivity (k) values for till soils obtained for Shelby tube samples tested in accordance with ASTM D 5084.

Radiological fate and transport modeling performed as part of the FS resulted in a requirement that at least 12 ft. of undisturbed gray till be left in place below the OSDF to function as both a hydraulic barrier and geochemical barrier to potential downward migration of radiological waste constituents. The gray till was not penetrated in construction of the OSDF, thereby meeting this requirement (Figure 8).

WASTE CHARACTERISTICS

Materials disposed in the OSDF consist of about 85 percent soil and soil-like materials (SLMs) excavated as part of the remediation of the Fernald site and about 15 percent building demolition debris, structural members, mass concrete, decommissioned equipment, lime sludge, coal flyash, municipal solid waste, asbestos waste, and small quantities of other materials.

OSDF waste acceptance criteria (WAC) for radiological and hazardous constituents in soil are given in Table 5. These criteria were established during the FS through fate and transport modeling of leaching and leakage scenarios from the OSDF to groundwater. If waste materials at the Fernald site exceeded any of these criteria, the waste could not be placed in the OSDF.

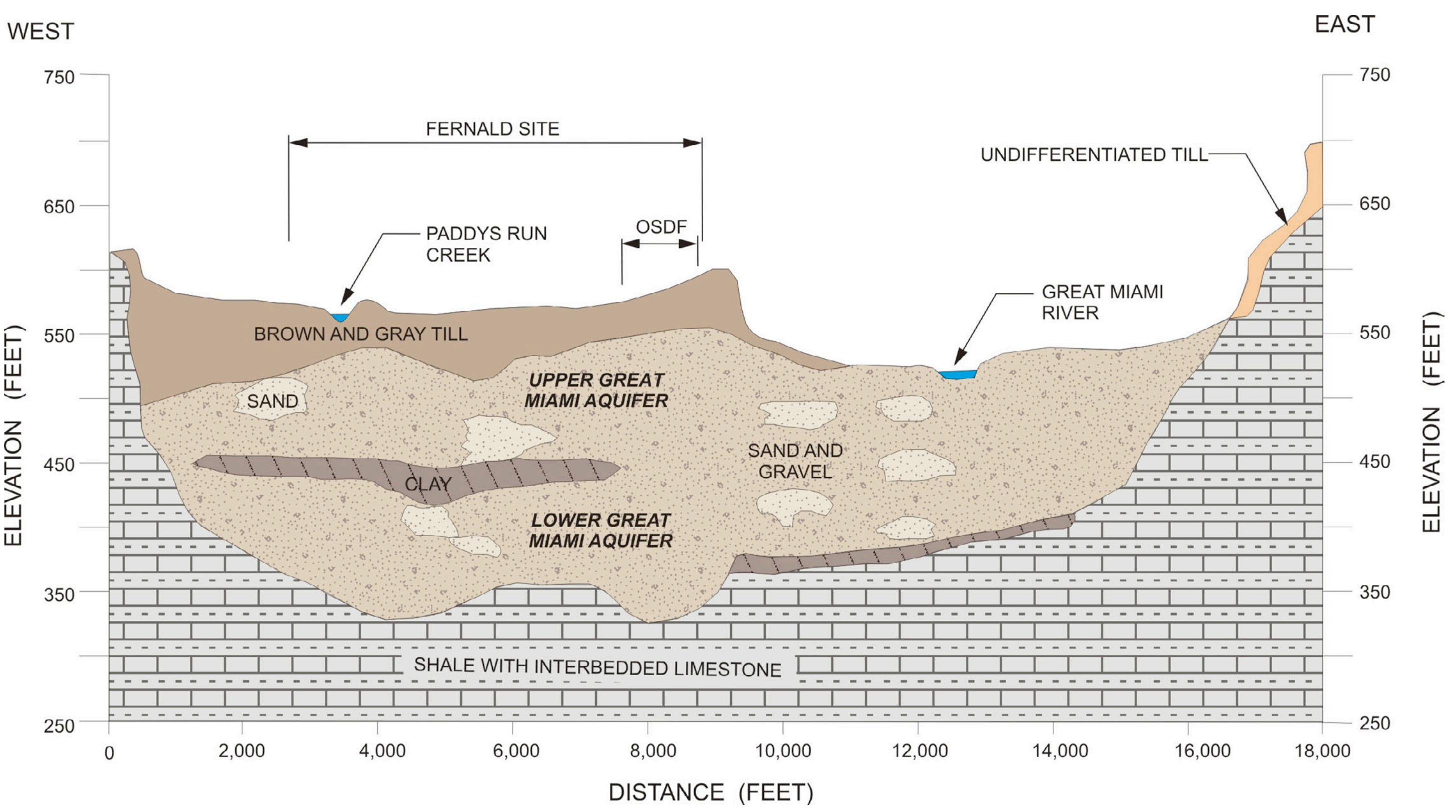

Figure 7. Idealized subsurface profile. Vertical exaggeration = 20x.

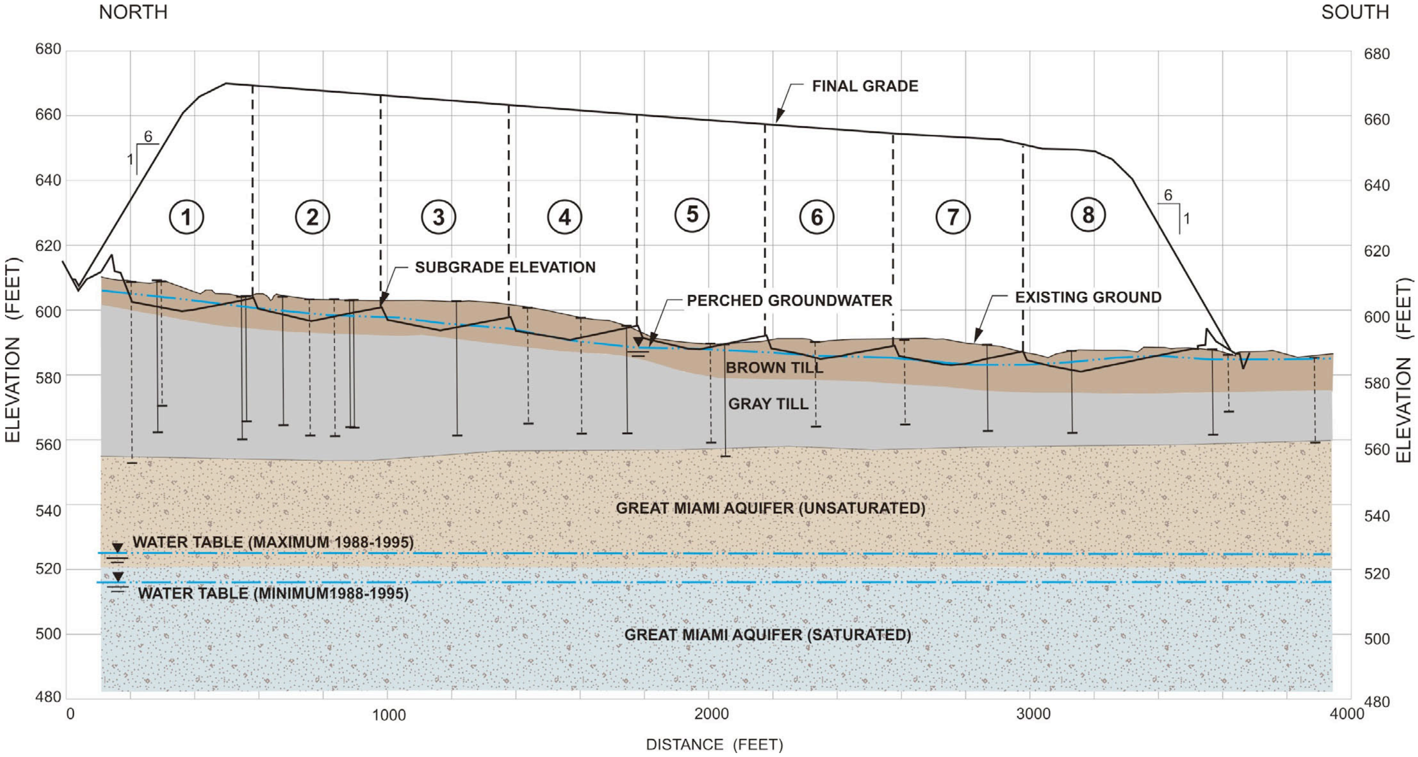

Figure 8. OSDF north-south cross section. Vertical exaggeration = 10x

TABLE 5. OSDF radiological and hazardous constituent waste acceptance criteria for soil.

	Constituents of Concern (COCs)	Maximum Concentrations	
	Radionuclides:		
1	Neptunium-237	3.12×10^{9}	pCi/g
2	Strontium-90	5.67×10^{10}	pCi/g
3	Technetium-99	29.1	pCi/g
4	Uranium-238	346	pCi/g
5	Total Uranium	1,030	mg/kg
	Inorganics:		
6	Boron	1.04×10^{3}	mg/kg
7	Mercury	5.66×10^{4}	mg/kg
	Organics:		
8	Bromodichloromethane	9.03×10^{-1}	mg/kg
9	Carbazole	7.27×10^{4}	mg/kg
10	Alpha-chlordane	2.89	mg/kg
11	Bis(2-chloroisopropyl)ether	2.44×10^{-2}	mg/kg
12	Chloroethane	3.92×10^{5}	mg/kg
13	1,1-Dichloroethene	11.4	mg/kg
14	1,2-Dichloroethene	11.4	mg/kg
15	4-Nitroaniline	4.42×10^{-2}	mg/kg
16	Tetrachloroethene	128	mg/kg
17	Toxaphene	1.06×10^{5}	mg/kg
18	Trichloroethene	128	mg/kg
19	Vinyl chloride	1.51	mg/kg

Note: pCi/g = picoCuries per gram; mg/kg = milligrams per kilogram; pCi = 0.037 disintegrations/second.

The OSDF also had a large number of physical WAC, including for example:

- concrete structural members could not be more than 10 ft. long nor more than 18 in. thick and 4 ft. wide;
- reinforcing bars protruding from concrete debris were cut to within 12 in. of the concrete;
- metal structural members could not be more than 10 ft. long nor more than 18 in. thick and 10 ft. wide;
- building rubble, HVAC components, electrical equipment, and mechanical equipment needed to be size reduced to less than 18 in. thick;
- process piping with a diameter larger than 12 in. was split in half; and
- all equipment was drained of oil and other liquids prior to disposal.

In addition to the physical WAC given above, soil and SLMs brought to the OSDF had to have moisture contents that allowed the materials to be compacted to required levels using standard soil compaction equipment and procedures. As necessary, soil and SLMs were dried by disking and air drying, or by blending with drier soil.

For purposes of waste placement in the OSDF, impacted materials meeting all WAC were segregated into one of the following five categories:

Category 1 impacted materials were soils and SLMs that did not contain hard agglomerations greater than 12 in. in largest dimension. Category 1 materials could also contain a maximum of 20 percent, by volume, of non-soil-like Category 2 and/or Category 4 material not greater than 12 in. in largest dimension if the remainder of the material was soil and/or

SLM finer than 1 in. particle size. These impacted materials were compactable using standard soil compaction equipment. Category 1 material was placed in 12 to 15 in. loose lifts and compacted to a minimum standard Proctor (ASTM D 698) relative compaction (SPRC) of 90 percent using Caterpillar 815 or 825 soil compactors.

Category 2 impacted materials were materials that could be transported, placed, spread, and compacted *en masse.* These materials could be spread in loose lifts of 21 in. ±3 in. thick and were compacted using a Caterpillar 826 landfill compactor or approved similar equipment. Examples of Category 2 materials include broken-up concrete foundations and impacted soil mixed with broken-up concrete. This category also included general building rubble and debris of irregularly shaped metals and other components of the superstructure or substructure with a maximum length of 10 ft. and a maximum thickness of 18 in. Category 2 material was placed at designated grid locations in areas with lateral dimensions not exceeding 100 ft. Each compacted lift of Category 2 material was covered with at least 4 ft. of Category 1 material. Placement of Category 2 material is shown in Figure 9.

Figure 9. Typical grid system for placement of impacted material, with Category 2 material being placed in the center of the photograph. Placards were used to identify the grids to facilitate placement and construction documentation. Cell stormwater catchment area is also visible in upper-center portion of photograph.

Category 3 impacted materials were materials that had to be individually handled and placed in the OSDF, and that were suitable for having Category 1 material placed around and against them. These impacted materials had maximum cross-sectional dimension of no more than 4 ft. Examples of these materials include bundles of transite panels and broken concrete foundation members. These items were placed at least 50 ft. laterally inward from the edge of the OSDF and at least 100 ft. away from Category 4 and 5 materials. Any voids in the Category 3 material larger than one cubic foot, and areas between members where Category 1 material could not be placed and compacted, were filled with flowable sand or quick set grout. Lifts of Category 3 material were separated vertically by at least 4 ft. of compacted Category 1 material.

Category 4 impacted materials were high in organic content and/or prone to decomposition. Examples of these materials are municipal solid wastes from an on-site solid waste landfill, and green waste from clearing, stripping, and grubbing operations around the Fernald facility. Category 4 material was placed at designated grid points in loose thicknesses of not more than 18 in. and lateral dimensions of not more than 100 ft. This material was compacted with the landfill compactor or large dozer. Not more than two lifts of Category 4 material could be placed at a grid location. Subsequent grid locations were not allowed to be placed in the vertical space above previously-placed Category 4 grids.

Category 5 impacted materials were materials that require special handling due to their specific nature. Examples of these materials include double-bagged asbestos, piping with asbestos containing material, and sludges. Each of these materials had customized placement procedures.

A 3-ft. thick “select layer” of compacted Category 1 soil was placed on top of the liner system to protect the liner during placement of other categories of waste. Similarly, a 3-ft. thick soil “select layer” was placed above the OSDF waste mass just prior to cover system installation for the purpose of protecting the cover system. These select layers are shown in Figures 11 and 14. The select layer above the liner system was compacted lightly so as to not damage the liner system (i.e., to about 85 percent SPRC). Select impacted material below the cover system was compacted to 90 percent SPRC.

The overall philosophy for waste placement within the envelope of the select layers was to create a relatively homogenous mass at a large scale by the controlled placement of heterogeneous materials at a smaller scale. This was achieved by using impacted soil and SLMs to form the overall matrix of the waste mass and distributing heterogeneous materials such as structural members, dismantled machinery, and double-bagged asbestos at discrete grid locations both laterally and vertically throughout the soil/SLM matrix.

As part of the design, short-term and long-term settlements were estimated for the OSDF foundation and the OSDF waste mass. To obtain these estimates, the waste mass was modeled as a homogenous soil-like material and classical methods were used for analyzing immediate, primary, and secondary settlements. Calculated maximum settlement of the OSDF foundation is 2.8 ft. and the time to complete 95 percent of primary consolidation is estimated to be in the range of 10 to 40 years. The impacted materials within the OSDF were estimated to undergo up to 3.8 ft. of compression under self weight, with most of this settlement occurring during filling. Settlement of the cover system results from post-filling compression of the impacted materials and settlement of the foundation. Maximum total settlements of the cover system are estimated to be about 3.5 ft. Calculated differential settlements for both the liner system and cover system resulted in acceptable post-settlement grades and geosynthetic tensions, with adequate factors of safety.

OSDF DESIGN

Functional Requirements

Table 6 presents select OSDF functional requirements (i.e., essentially performance and design criteria) developed by DOE that derive from relevant federal and state regulations, from siting criteria, and from engineering design considerations. The design approach used for the OSDF was developed to achieve these functional requirements.

TABLE 6. Select functional requirements for Fernald OSDF.

Functional Requirements
Location (exclusions) • within 200 ft. laterally of stream, lake, or wetland • within 15 ft. vertically of the uppermost aquifer • within a regulatory floodplain • within an area of potential subsidence • within 200 ft. laterally of a Holocene fault
Layout • locate on east side of site, between main facility and power transmission lines • achieve capacity of 2.5 million bank cubic yards (ultimately 2.95 million in place cubic yards) • maximum height should be less than 70 ft. above original ground (visual impact) • final cover system slopes must be between 5 and 25 percent • liner system must overlie at least 12 ft. thickness of undisturbed gray till • LCS drainage slope must be at least 2 percent
Engineering • design life of 1,000 years to the extent reasonably achievable, and, in any case, at least 200 years • long-term static slope stability factors of safety (FS) must exceed 1.5 • pseudo-static FS for 2,300-year recurrence interval earthquake must exceed 1.0 • double-liner system with secondary composite liner must be installed beneath waste • secondary liner must include 3-ft. thick CCL with maximum hydraulic conductivity of 1 × 10-7 cm/s overlain by HDPE geomembrane at least 60 mil thick • final cover system must have a composite cap consisting of a 24-in. thick CCL with maximum • hydraulic conductivity of 1 × 10-7 cm/s overlain by HDPE geomembrane at least 60 mil thick • cover system must include a biointrusion barrier at least 3 ft. thick • final cover system topsoil must have a predicted erosion rate of less than 5 tons/acre/year and must resist gully initiation under the anticipated runoff tractive stresses • final stormwater management system must accommodate 2,000-year, 24-hour storm flow

Conceptual Design Approach

The function of the OSDF is to isolate impacted material from the environment "for up to 1,000 years to the extent reasonably achievable, and, in any case, for 200 years." This performance criterion was adopted by DOE from 40 CFR §192.02(a) which provides minimum federal disposal criteria for uranium and thorium mill tailings. The design was also developed using the radiation protection goal of DOE Order 5400.5, which requires application of "As Low As Reasonably Achievable (ALARA)" principles to activities involving the excavation, transportation, and disposal of LLRW (DOE, 1989). These criteria were achieved in design by addressing five potential mechanisms for OSDF performance failure:

internal hydrologic control – provide leachate containment and collection within the OSDF to prevent OSDF leachate from entering the environment;

external hydrologic control – provide resistance to external hydrologic impacts, including infiltration through the cover system and damage by surface-water runon and runoff;

geotechnical stability – provide adequate OSDF slope and foundation stability during construction, filling, closure, and post-closure, including conditions associated with potential long recurrence-interval earthquake events;

erosional stability – provide resistance to erosion of OSDF soil layers to achieve minimal erosional impacts throughout the performance period; and

biointrusion resistance – provide resistance to OSDF intrusion by plant roots and burrowing animals.

Appendix A summarizes the way in which specific design elements were used to address the potential for OSDF performance failure.

The OSDF design approach incorporated the following additional measures to satisfy the performance period:

Natural (i.e., geological) materials were used in preference to manufactured (i.e., geosynthetic) materials for certain functions (e.g., internal drainage layers).

Relatively thick compacted-clay liners (CCLs) were incorporated into the design of both the liner and cover systems in preference to liner systems constructed completely of geosynthetics, or with thinner CCLs.

High density polyethylene (HDPE) geomembranes were specified in preference to other types of geomembranes based, in part, on their durability characteristics. Studies available in 1995 indicated that the HDPE service life would be on the order of hundreds of years (Koerner et al., 1992; Bonaparte, 1995). More recent studies (e.g., Bonaparte et al., 2002;

Rowe, 2005) indicate that at an ambient ground temperature of about 55°F (12°C), the design life for buried HDPE geomembranes will be on the order of 1,000 years. Regulations required the HDPE geomembrane to be at least 60 mil thick. However, the design specified that the geomembrane be at least 80 mil thick as another measure to increase the service life of this material. The rationale for the thicker material is that the primary degradation processes for HDPE geomembrane involve polymer chain oxidation that starts at the surface and works inward. In the event of surface oxidation, a thicker material will retain its properties longer than a thinner material, all other factors being the same. The specifications also required the HDPE formulation to contain 2 to 3 percent antioxidant-containing carbon black (ASTM D 1603) and to have a minimum environmental stress crack resistance (ESCR) of 500 hours when tested in accordance with the notched constant tensile load (NCTL) method of ASTM D 5397. In 1995, HDPE geomembrane specifications typically required ESCR of 100 to 200 hours; the more stringent specification for the OSDF provides a material with better aging potential and less potential for long-term brittle rupture under stress.

All hydraulic barriers in the liner and cover systems (primary liner, secondary liner, and cover barrier layer) were designed as soil-geosynthetic composite barriers in preference to single component barrier layers. Composite barriers provide superior hydraulic containment compared to single-component barriers (Giroud and Bonaparte 1989a, b). The individual components of composite barriers also help to protect the other component. For example, CCLs provide good bedding layers for geomembranes and geomembranes help to prevent desiccation cracking of CCLs after installation.

For design, the OSDF performance period was divided into three operating timeframes.

Initial Period. The initial period extends from construction until the end of the 30-year post-closure monitoring period described in the OSDF ROD [DOE, 1995b]. During this period, leachate generation rates are predicted to decrease from a design value at the start of cell filling of 1,150 gallons per acre of lined area per day (gpad) to only 0.002 gpad at the end of the post-closure period (Geosyntec, 1997b). These values are based on a mean annual precipitation of 40 in. and they were estimated using the U.S. EPA computer model "Hydrologic Evaluation of Landfill Performance (HELP)". Throughout this initial period, all components of the OSDF are maintained and functional under the requirements of the OSDF Post-Closure Care and Inspection Plan (DOE, 2006).

Intermediate Period. The intermediate period begins 30 years after final closure of the OSDF and lasts for at least 200 years, and up to 1,000 years to the extent reasonably achievable. During this period, the geomembrane components of the liner and final cover systems remain functional. The leachate collection system (LCS) and leakage detection system (LDS) are maintained as necessary, as is the cover system. The cover system is planted with a variety of native prairie grasses that require periodic mowing and baling to simulate periodic grass fires. This periodic mowing will also prevent the growth of trees on the cover system during this period.

Final Period. The final period does not occur for at least 200 years, and possibly up to 1,000 years, after final closure of the OSDF. During this period, natural earth components of the liner and final cover systems continue to be functional. It is assumed that, at some point in time, the HDPE geomembrane and other geosynthetic components of the liner and cover systems begin to degrade and progressively lose functionality.

Responsibility for maintenance and stewardship for the OSDF rests in perpetuity with the U.S. government. The OSDF design allows government decision makers at the time of the final period to select an appropriate continuing management strategy for the facility. Potential strategies include:

Ending Maintenance. Any small amount of leachate generated by the OSDF (due to infiltration through the degraded OSDF cover system) will be allowed to migrate through the degraded liner system into the brown and gray till that underlies the OSDF. In this case, the LCS and LDS drain pipes from each cell will be sealed by grouting or other appropriate measures. Based on the studies performed for the OU2 FS [DOE, 1995a], this final period management approach will be protective of groundwater quality in the underlying Great Miami aquifer.

Continuing Maintenance. The cover system and LCS and LDS drain pipes will be maintained. While no leachate is expected under this scenario, any LCS or LDS drainage will be collected and transported off-site for treatment, or discharged to a natural treatment system, such as a wetland area established at or near the site, the selected treatment approach will depend on the quality and quantity of the draining liquids.

Reconstruction/Rehabilitation. The cover system and LCS and LDS drain pipes will be reconstructed/rehabilitated using the most appropriate technologies available at that time, or other improvements will be made to the facility based on technologies that have emerged since OSDF construction.

<u>Detailed Design Development</u>

Figures 8 and 10 show, respectively, north-south and east-west cross sections of the OSDF as designed to meet the functional requirements and conceptual design described above. The design includes eight cells constructed sequentially from north (Cell 1) to south (Cell 8) over the active life of the facility. The photograph in Figure 5 illustrates this sequential cell development. The OSDF was designed as an essentially above-ground facility. The bottom of each cell is graded in a

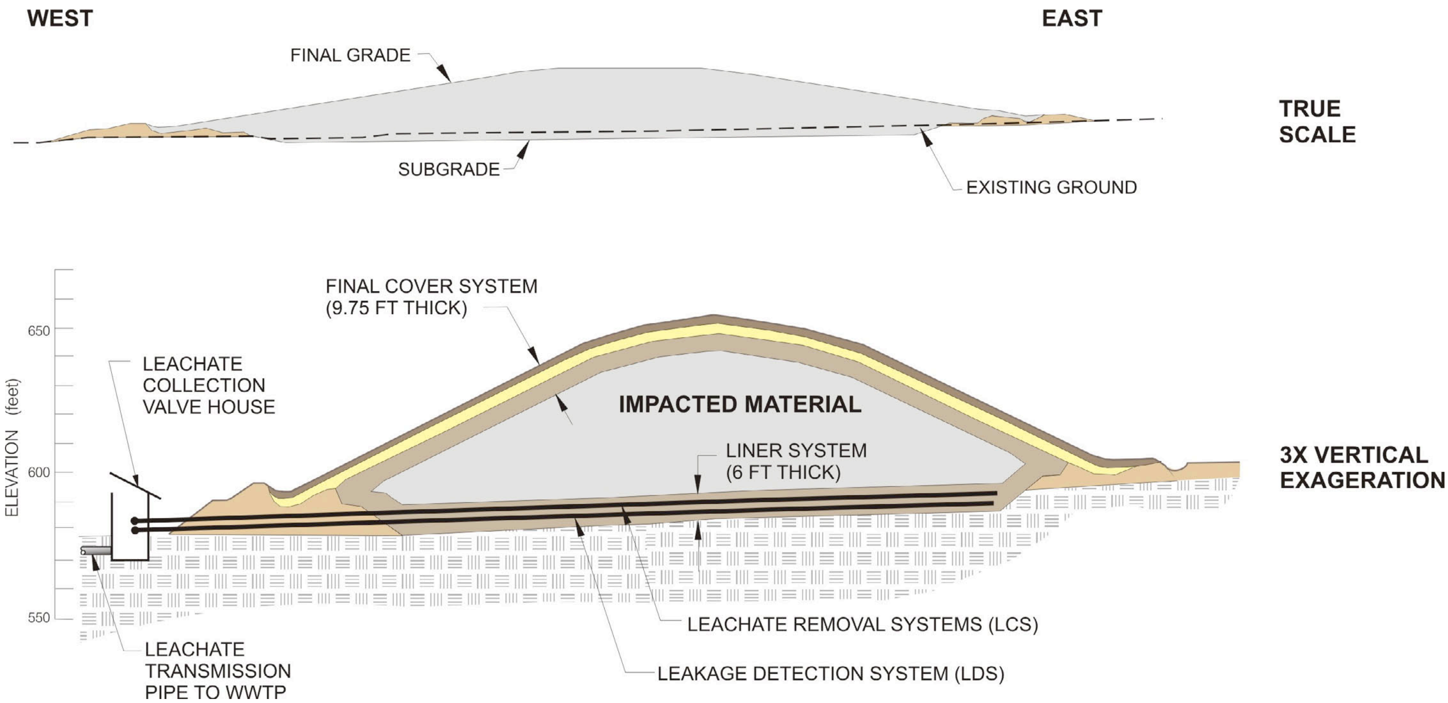

Figure 10. OSDF east-west cross section.

herringbone pattern at a 2 percent slope to drain leachate by gravity to the west side of the cell. This grading configuration was designed to follow the pre-construction natural grades in the project area, thereby limiting foundation excavation requirements. Maximum excavation depth for the OSDF was 15 ft. at the northeast corner of Cell 1, but the average excavation depth was only a few feet. This configuration allows the LCS and LDS drainage pipes to exit the cell at or near the original ground surface elevation. While the base grading plan for each cell is similar, the elevations of the grades step down from Cell 1 to Cell 8, again to follow pre-construction natural grades in the project area, thereby limiting cut and fill volumes.

Figure 11 shows the liner system configuration for the OSDF. The double-composite liner system is constructed of a combination of hydraulic barrier layers, drainage layers, and filter and cushion geotextiles. Gravel was specified for the LCS and LDS drainage layers in preference to geosynthetics due to design life considerations. The gravel had a maximum particle size of 0.75 in. and less than 2 percent fines (Figure 12). Durability considerations also drove selection of the HDPE geomembranes. The durability characteristics of the geosynthetic clay liner (GCL) hydraulic barrier are less well defined than either the HDPE geomembrane or compacted clay. However, the GCL is intended to function principally during the active life when most leachate is produced, so, it was not critical to define a long-term design life for this component.

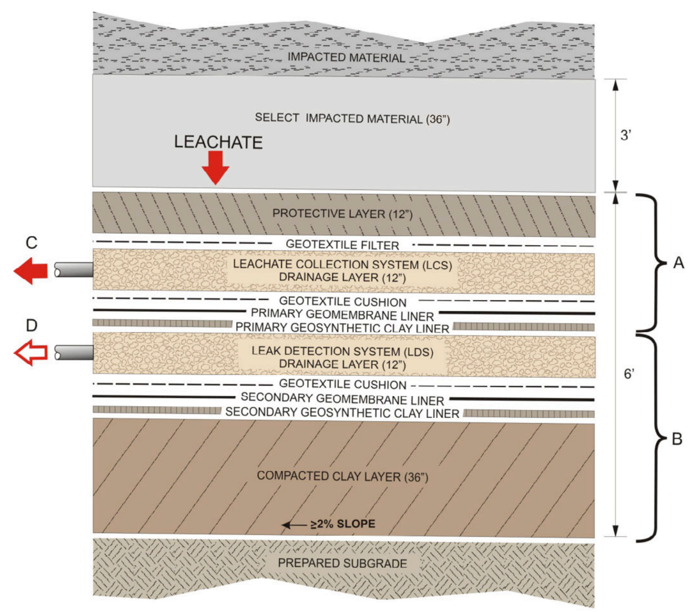

Figure 11. OSDF double-composite liner system: A = primary liner system; B = secondary liner system; C = leachate removal pipe (perforated in cell and solid, double-walled outside of cell) for gravity drainage of leachate to valve house; and D = leakage detection system monitoring and liquid removal pipe (similar pipe design to leachate removal pipe).

Each of the eight OSDF cells was designed with intercell berms so that the LCS and LDS for a cell captured only the liquids produced in that cell. Accordingly, each LCS and LDS has its own liquids removal pipe (Figures 11 and 13). LCS and LDS liquid removal pipes consist of thick-walled HDPE

(6.6-in. outside diameter pipes having a standard dimension ratio (SDR) of 11, meaning the pipe wall thickness is 0.6 in.). In consideration of the OSDF design life, all LCS/LDS pipes gravity drain from the cells to a double-walled collection forcemain located in valve houses outside each cell (Figure 10). Gravity drainage was judged to be a more reliable long-term liquids removal strategy than one using submersible pumps and sideslope riser pipes. This LCS/LDS drainage strategy did necessitate penetration of the LCS and LDS pipes through the liner system at the location of the downgradient perimeter berm. As part of the design, special measures were developed for sealing the liner system to the penetrating pipes. This detail is described subsequently. Each LCS/LDS valve house was designed to contain cleanout connections on the LCS and LDS removal lines.

Figure 12. Placement of the LCS and LDS gravel drainage layers included the use of dump trucks operating on a 3-ft thick access road constructed from gravel and low-ground pressure dozers to spread the gravel.

Redundant features were incorporated in the liner system design, including a second, back-up LCS liquid removal pipe (Figure 13), rather than the one pipe that is customarily used in landfill applications.

One unique aspect of the OSDF design is management of precipitation that falls on an active cell. Standard practice for MSW landfills is to cover the waste with daily or intermediate cover (soil or tarps) and direct the collected precipitation away from the landfill as clean storm water. For the OSDF, all precipitation that fell on an active cell, until at least two layers of the final compacted clay cap were in place, had to be collected and treated. To achieve this, a stormwater catchment area was placed in the southwest corner of each cell. Collection began immediately after the placement of the protective soil layer because impacted soil could be used for this layer. The catchment area was located directly over the leachate collection layer at the low end of each cell. The collected impacted storm water drained out the bottom of the catchment area into the LCS and was managed as leachate. The catchment area was sized to contain run-off from a 25-year, 24-hour storm event. As discussed in the performance section of this paper, this stormwater catchment system may have contributed to unexpectedly high leachate generation rates in the OSDF.

Figure 14 shows the configuration of the final cover system. Final cover slopes are 6 horizontal:1 vertical (6H:1V) on the sides of the OSDF and 6 percent on the top deck. Construction of the CCL in the final cover system is shown in Figure 15. The final cover side slope inclination is flatter than for most MSW landfills and was chosen based on the results of slope stability and erosion gullying analyses to achieve the functional requirements described previously. To maintain the long-term functionality of the final cover system, the biointrusion layer is designed to arrest plant root and/or

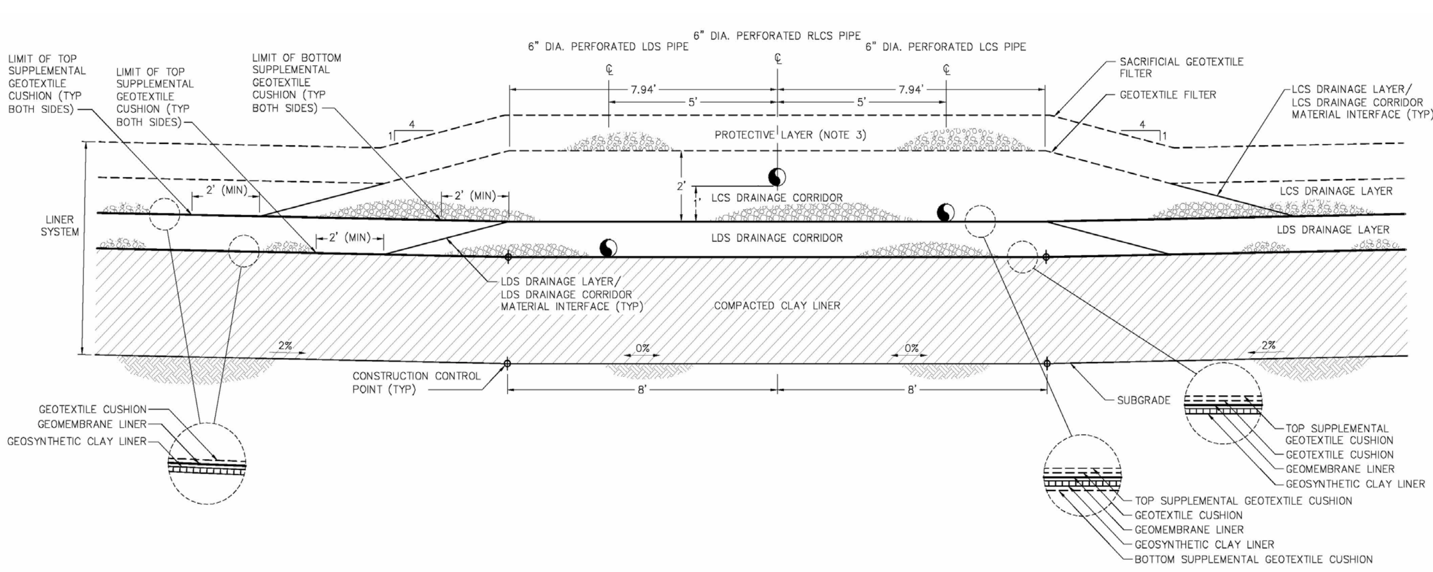

Figure 13. Liner and leachate collection system cross-sectional details at downgradient cell outlet.

burrowing animal intrusion. Placement of the biointrusion layer is shown in Figure 16.

Appendix B lists the design calculation packages prepared by the authors. This list is more extensive than for most landfills due to the design requirements imposed by the long performance period for the OSDF. Two examples of analyses conducted for the OSDF but not typically performed as part of the landfill design process are: (i) evaluation of erosional stability of grass-lined drainage ditches for the 2,000 year storm event using the method of Temple et al. (1987); and (ii) calculation of the average atmospheric release rate of Radon 222 and evaluation against a regulatory standard (40 CFR §192.02(6)) of 20 picocuries per square meter per second (20pCi/m^2/s) using the computer program "Radiation Attenuation Effectiveness and Cover Optimization with Moisture Effects (RAECOM)" (NRC, 1984a,b). Appendix B is included in this paper to help guide engineers in establishing the scope of future design efforts for long-performance-period land disposal facilities.

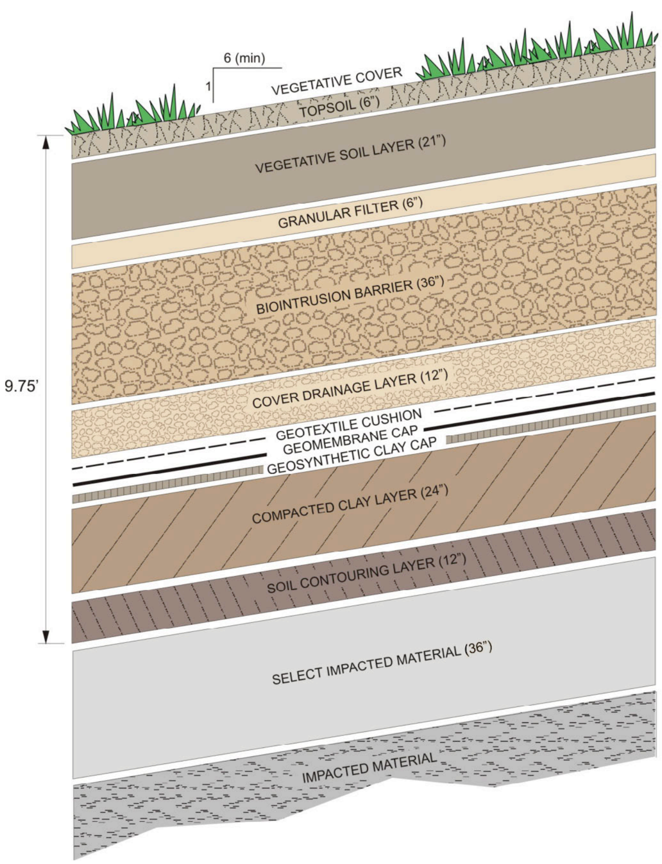

Figure14. OSDF final cover system.

Figure 15. Construction of the final cover system CCL. The soil in the foreground is placed material that has not yet been compacted. The surface has been sealed to promote storm run-off and prevent drying. The small tracked loader holds oversized rock (>2 in.) collected by the workers.

Figure 16. Placement of the 3-ft. thick biointrusion layer over the cover drainage layer.

FIELD AND LABORATORY DESIGN STUDIES

<u>Hydraulic Conductivity of CCLs</u>

The source of clay for the OSDF CCLs was brown till obtained from the OSDF excavation and from an adjacent borrow area located south of the OSDF. The thickness of brown till in these areas generally ranged from 10 to 15 ft., with the material exhibiting distinct visual and geotechnical differences between upper and lower horizons. Materials from both horizons classify as lean clay (CL) according to ASTM D 2487. However, the lower-horizon brown till is less plastic and has fewer fines compared to the upper horizon material. As part of the detailed design, the suitability of both brown till horizons was carefully evaluated using field test pads. Results from the upper horizon brown till evaluation are summarized below.

The evaluation of upper horizon brown till involved pre-construction, construction, and post-construction phase hydraulic conductivity laboratory testing of the till using flexible wall permeameters (ASTM D 5048). Post-construction field-scale permeability testing was also conducted using sealed double-ring infiltrometers (SDRIs)

(ASTM D 5903). Pre-construction laboratory tests were used to design the field test pad program. The SDRI tests were used to assess field construction methods and to establish the acceptable permeability zone (APZ) for OSDF CCL construction. The construction and post-construction laboratory tests were performed to evaluate the quality of thin-walled tube samples for use in QC/QA during OSDF CCL construction. The APZ was defined as those combinations of compaction moisture content and dry unit weight producing a CCL with a hydraulic conductivity not greater than 1×10^{-7} cm/s. The test pad (Figure 17) had six compacted lifts, a nominal compacted thickness of 3 ft., and three lanes, with each lane about 14.3 ft. wide (equal to the full pass width of a Caterpillar 815B padfoot compactor) and 50 ft. long (excluding end ramps).

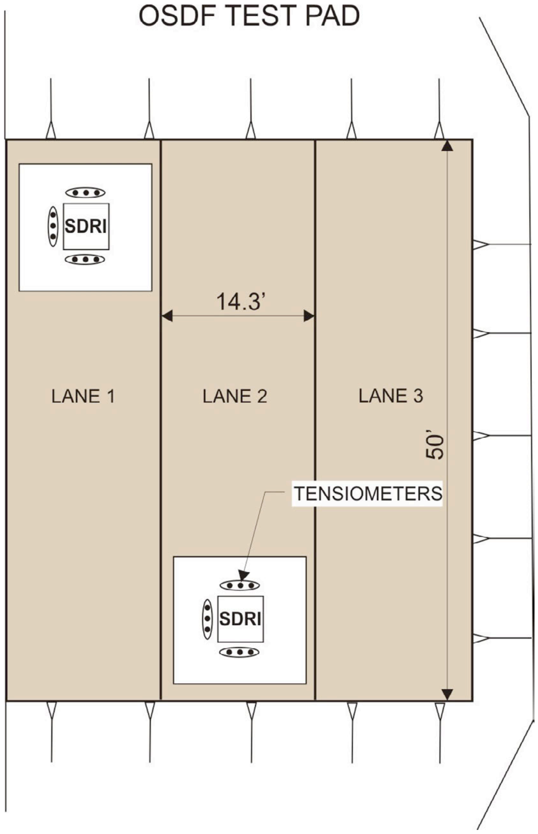

Figure 17. Plan view of test pad for upper horizon brown till (design of test pad for lower horizon brown till was identical).

For the test pad, the target compaction moisture content was the same for all three lanes at 2 percentage points +/- 1 percentage point wet of the standard Proctor optimum moisture content. The compaction effort varied for each lane in an attempt to achieve 95, 97, and 99 percent SPRC. The number of compactor passes for the three lanes was 4, 7, and 10 respectively, with each pass being one back-and-forth full coverage. Soil processing on the test pad prior to compaction consisted of oversize (larger than 2 in.) rock removal, moisture conditioning, and mixing/blending of the soil using a HAMM RACO 250 transverser rotary mixer. Thin wall tube samples of the test pad were obtained at the end of test pad construction and samples from the tubes were extruded in the laboratory and evaluated for hydraulic conductivity in flexible wall permeameters. The field testing program included moisture content, dry unit weight, and field hydraulic conductivity as measured using SDRIs. Two SDRI tests were performed. For each, a test was performed on the lane with 4 compaction passes and a test was performed on a lane with 7 compaction passes. Figure 18 shows an idealization of the SDRI set-up and Figure 19 shows one of the SDRI tests in progress. The duration of the SDRI tests was 26 days. At the conclusion of each test, four thin-walled tube samples were obtained from within each of the SDRI inner ring areas. Two of the tubes were used to obtain samples for post-construction laboratory hydraulic conductivity testing in flexible wall permeameters, and two were used for interval moisture content testing to evaluate the advancement of the wetting front.

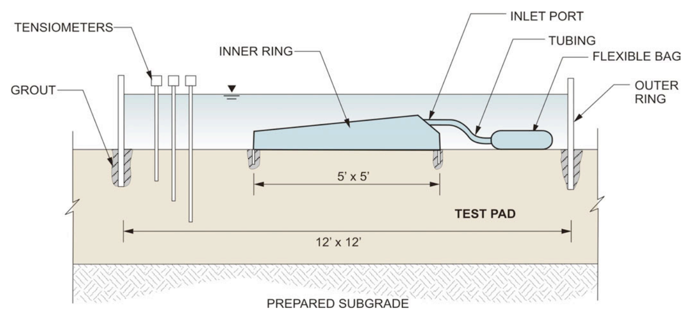

Figure 18. Sealed double-ring infiltrometer test set-up.

Figure 19. Sealed double-ring infiltrometer test in progress.

As can be seen in Table 7, stabilized SDRI hydraulic conductivities averaged 1.5×10^{-8} cm/s. The estimated depth of the wetting front in the SDRI tests ranged from 5 to 7 in., based on the post-construction moisture content testing. This compares well with tensiometers installed at depths of 6, 12, and 18 in. The 6-in. deep tensiometers showed little residual soil suction at the end of the tests, whereas the 12-in. and 18-in. deep tensiometers maintained suction. The final construction APZ for the upper horizon brown till is shown in Figure 20. This APZ is defined by the 90 percent degree of saturation line to the left, a moisture content equal to 3 percentage points wet of the standard Proctor optimum moisture content to the right, and the 95 percent SPRC at the bottom.

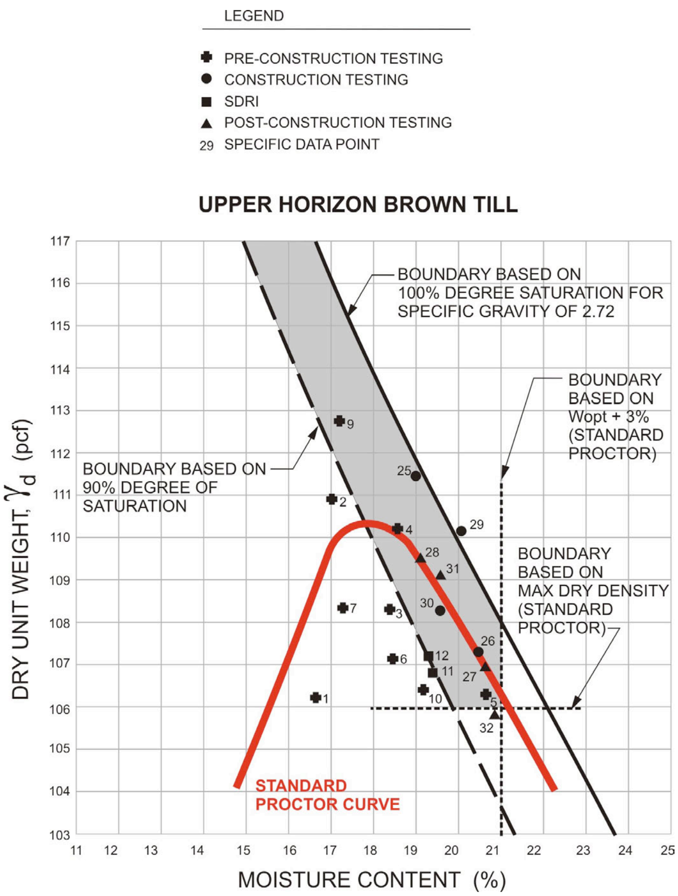

Figure 20. Acceptable permeability zone for upper horizon brown till.

TABLE 7. Summary of laboratory and field hydraulic conductivity test results for upper horizon brown till.

Moisture Content (percent)	Dry Density (pcf)	Degree of Saturation (percent)	Hydraulic Conductivity (confining stress=2 psi) (cm/s)[3]	Figure ID No. [1]
Pre-Construction[2] - Laboratory Results				
16.7	106.3	76.1	5.2×10^{-5}	1
17.0	110.9	87.2	6.4×10^{-7}	2
18.4	108.3	88.2	8.4×10^{-8}	3
18.6	110.2	93.7	4.8×10^{-8}	4
20.7	106.3	94.4	1.1×10^{-8}	5
18.5	107.1	86.1	1.2×10^{-6}	6
17.3	108.3	83.0	6.7×10^{-6}	7
12.4	119.3	79.8	1.4×10^{-7}	8
17.2	112.8	92.7	2.1×10^{-8}	9
19.2	106.4	82.7	5.5×10^{-8}	10
SDRI Results				
19.4	106.8	89.6	1.5×10^{-8}	11
19.3	107.2	90.0	1.4×10^{-8}	12
Construction Phase - Laboratory Results				
19.0	111.5	99.6	9.8×10^{-9}	25
20.5	107.3	95.8	1.5×10^{-8}	26
20.1	110.2	101.2	5.3×10^{-8}	29
19.6	108.3	94.0	1.7×10^{-8}	30
Post-Construction - Laboratory Results				
20.6	106.9	95.3	5.1×10^{-8}	27
19.1	109.5	94.5	1.3×10^{-8}	28
19.6	109.1	95.9	2.3×10^{-8}	31
20.9	105.8	94.1	9.4×10^{-9}	32

Notes:
1. Figure ID No. refers to Figure 20.
2. Upper horizon brown till composite sample: LL=43, PL=20, PI=23; 74% passing #200 sieve; specific gravity=2.72.
3. Pre-construction laboratory results obtained with flexible wall falling-head permeameter tests (ASTM D 5084) using laboratory compacted samples. Construction and post-construction laboratory results obtained using same tests and thin-wall tube samples obtained from field test pad.

<u>Compatibility of Liner System with Leachate</u>

As part of the detailed design, a literature-based durability evaluation was undertaken for the geomembrane and soil components of the liner system. Based on this "desk top" evaluation, it was concluded that additional data were needed on the potential for OSDF leachate to affect the properties of HDPE geomembranes. A leachate-geomembrane compatibility testing program was developed to fill this data gap. Five different commercial 80-mil thick HDPE geomembranes from the four major U.S. manufacturers in 1995 were evaluated using U.S. EPA Method 9090, "Compatibility Test for Waste and Membrane Liners." This method involves submersion of geomembrane samples in test leachate at 23°C (73°F) and 50°C (122°F) for 120 days. Specimens of the various samples are retrieved at 30, 60, 90, and 120 days and evaluated against controls for changes in physical, mechanical, and chemical properties. A detailed presentation of the leachate-geomembrane compatibility testing program is contained in Geosyntec (1997c). A brief summary is presented below.

The leachate composition for the testing program was based on the results of modeling performed during the FS and the chemical composition of perched shallow groundwater beneath the plant site. The test leachate had neutral pH, high specific conductance, low levels of volatile organic compounds (VOCs), and the radionuclide concentrations shown in Table 8.

TABLE 8. Comparison of OSDF test leachate to Fernald perched groundwater wells and OSDF modeling results.

Radionuclide	Clarifier Pit Perched Groundwater Concentrations	Modeled Concentrations	OSDF Test Leachate Concentrations
Technetium-99	15.8–6130	56.6	64262
Uranium-234	0.001–25000	–	220000
Uranium-235/236	0.2–2490	–	29000
Uranium-238	0.3–39000	2240	240000
Uranium, total	0.4–436000	6670	582000
Neptunium-237	0.626[3]	18.6	0.14
Strontium-90	1.01–7.68	37.9	0.33

Notes:
1. Analyses were performed on unfiltered samples.
2. All results are in picoCuries per liter (pCi/L) except total uranium which is in micrograms per liter (µg/L).
3. Neptunium-237 was detected in only one sample.

Radionuclides in the test leachate included alpha, beta, and gamma emitters at significant concentrations. Total uranium (alpha, gamma) concentrations in the test leachate exceeded 500 mg/L. In Table 8, it can be seen that the concentration of total uranium in the test leachate exceeds the range of concentrations in the modeled leachate by almost two orders of magnitude. Concentrations of neptunium-237 and strontium-90 in the test leachate are about two orders of magnitude less than in the modeled leachate. However, based on the decay frequencies and energies of the various radionuclides, the test leachate has a radiological activity for alpha, beta, and gamma emitters about two orders of magnitude higher than that of the modeled leachate.

Test data interpretation methodology is described in Geosyntec (1997c) and included statistical comparisons of means and standard deviations, temporal and temperature trends, and consistency of trends between related properties. Typical results for stress at yield and for burst strength are shown in Figures 21 and 22, respectively. The test results showed negligible to only very minor changes in properties of the exposed samples compared to the control samples for mass, thickness, dimensions, specific gravity, extractables content, stress and strain at yield, hardness, and puncture resistance. The burst strength and stress/strain at break results showed the most change in the exposed mean versus the control mean. These latter observed changes were not, however, consistent between geomembranes, and based on the lack of observed change in the other properties, coupled with the results of surface analysis of the exposed geomembranes by Fourier-transform infrared (FTIR) spectroscopy, were judged not to be significant with respect to geomembrane performance. The FTIR results did not reveal any indication of surface oxidation of the geomembranes due to exposure to the test leachate. All five of the HDPE geomembranes tested were qualified for use based on radiological compatibility. It is interesting to note, however, that only four of the five geomembranes qualified based on the ESCR criterion of 500 hours (ASTM D 5397).

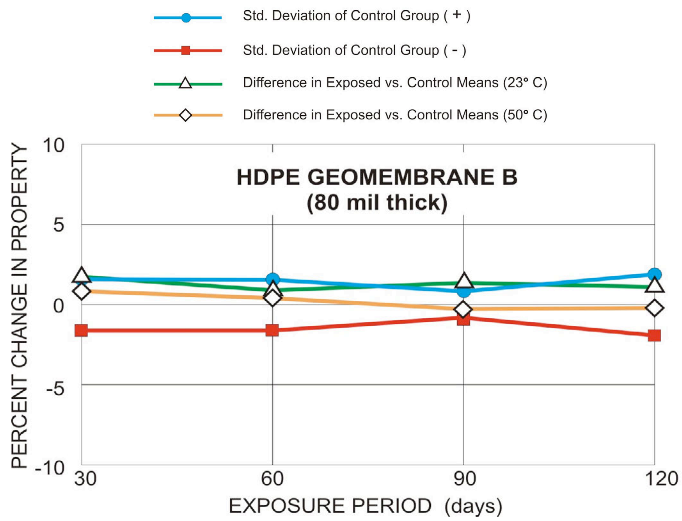

Figure 21. Effect of leachate exposure on HDPE geomembrane stress at yield.

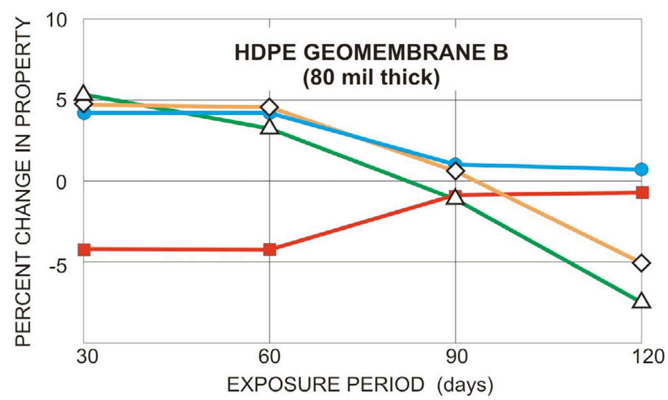

Figure 22. Effect of leachate exposure on HDPE geomembrane burst strength.

<u>Soil-Geosynthetic Interface Strength and GCL Internal Strength Shear Testing Program</u>

An extensive direct shear testing program involving soil-geosynthetic interfaces and GCL specimens alone was conducted to support slope stability analyses performed as part of the detailed design. Twenty-seven direct shear tests were performed in a 12 in. × 12 in. shear box in accordance with ASTM D 5321. The program also included testing for moisture content, compaction characteristics, particle size distribution, soil plasticity, and soil classification. Tested materials included HDPE geomembranes, brown till to be used to construct the CCLs, and several different internally-reinforced GCLs. Interface shear test samples were first soaked for one week under a seating stress of 43 pounds per square foot (psf), then consolidated for 48 hours at normal stresses of 720, 2,900, and 6,500 psf, and then sheared under the consolidation stress at rates of 0.04 in./minute and 0.004 in./minute. The lowest normal stress (720 psf) used in the test program represents the approximate normal stress acting on the geomembrane in the final cover system. The higher two normal stresses used in the program (2,900 and 6,500 psf) represent those acting on the liner system. The faster shearing rate represents the default ASTM rate. The slower shearing rate was selected based on previous testing that had shown close agreement between test results at this rate and even slower rate tests designed to achieve fully-drained porewater conditions. Interestingly, on the basis of seven side by side sets of tests conducted on this project, peak textured geomembrane-GCL interface shear strengths were 2 percent higher at the slower shear rate of 0.004 in./minute compared to the results at 0.04 in./minute. The slower rate large-displacement interface shear strengths were on average 6 percent higher than at the ASTM default rate. Several individual test differences were larger than these averages.

Fresh GCL, geomembrane, and soil specimens were used for each consolidation stress and shear rate (i.e., no multi-stage testing). Peak and large-displacement (2 in.) shear resistance versus displacement parameters were calculated for each test. Results from the interface tests were used to establish the conditions (e.g., CCL moisture content and dry unit weight) under which constructed interfaces would produce shear strengths meeting or exceeding the interface shear strengths used to establish the design. This process was conducted for liner system and cover system interfaces under short-term, interim, and long-term conditions considering both static and seismic loading. Figure 23 presents test results for the textured geomembrane to GCL (woven geotextile) interface for peak conditions and long-term static loading. The design failure envelope shown in this figure was developed prior to the interface testing program by using conservative literature values for interface strength. Using this envelope, a minimum long-term static slope stability factor of safety (FS) of 1.9 was calculated. This FS exceeds the project functional requirement of a minimum peak FS of 1.5. As can been seen in the figure, the measured interface strengths exceed the design failure envelope indicating a true long-term static FS for this interface larger than 1.9.

Figure 24 shows the design envelope for internal shearing of GCLs under large-displacement conditions. This design envelope was also developed using conservative literature values. This large-displacement envelope corresponds to a slope FS of 1.5 which exceeds the functional requirement of a minimum large-displacement FS of 1.25. Also shown on Figure 24 are the measured large-displacement internal shear strengths for the three GCLs tested. These are the lowest shear strengths obtained for all of the materials and interfaces evaluated for the OSDF design. These large-displacement internal shear strengths meet or exceed the design failure envelope.

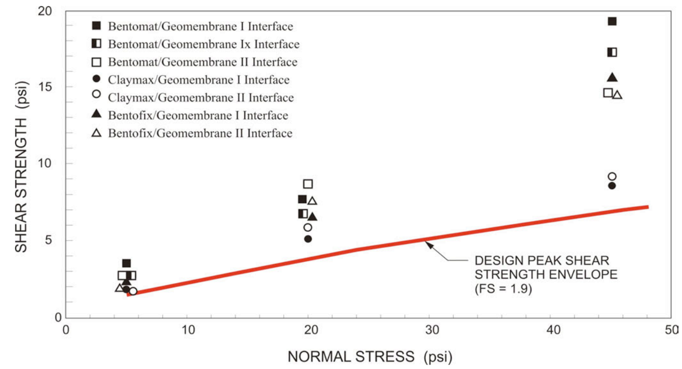

Figure 23. Design and measured peak GCL/geomembrane interface shear strengths, OSDF final configuration. Notes: Geomembrane I = 80 mil HDPE with spray-applied texture; Geomembrane II = 80 mil HDPE with blown-film texture; and GCL woven geotextile against geomembrane in all tests.

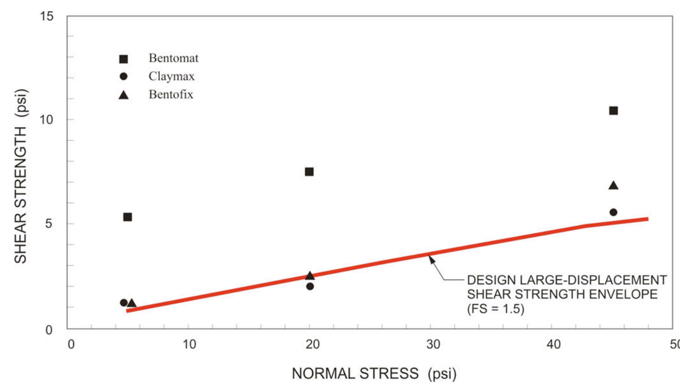

Figure 24. Design and measured large-displacement GCL internal shear strengths, OSDF final configuration.

CONSTRUCTION

As already noted, the Fernald OSDF was constructed sequentially as eight contiguous cells. Construction of the first cell (Cell 1) began in May 1997 and closure of the last cell (Cell 8) was completed in October 2006. The sequence of activity for each cell essentially involved excavation of topsoil and brown till to the design base grades; construction of earthen perimeter and inter-cell berms; installation of the liner system, LCS/LDS piping, and liner penetration boxes; placement of protective layer soil over the liner system; construction of the valve houses and leachate transmission piping; construction of a truck haul road into the cell; installation of interim stormwater management controls; placement of the 3-ft. thick select layer (Category 1, soil and SLM); placement of Category 1 through 5 wastes in accordance with the Impacted Materials Placement Plan to designated final grades; placement of another 3-ft. thick select layer and soil contouring layer on top of the Category 1 through 5 waste; installation of the final cover system; topsoil seeding; and construction of final stormwater management controls. Contract documents for this work included the construction plans and specifications and a variety of support plans, including: Construction Quality Assurance Plan, Impacted Material Placement Plan, Borrow Area Management and Restoration Plan, Surface Water Management and Erosion Control Plan, Cultural Resource Unexpected Discovery Plan, Systems Plan (including leachate management, utilities, site security, haul roads, decontamination facilities, and emergency spill response), Groundwater Monitoring Plan, Air Monitoring Plan, and Post-Closure Care and Inspection Plan.

The OSDF was designed to be constructed from conventional materials used in liner and final cover systems for waste containment facilities. These materials have established installation procedures, which provided a level of confidence in the ability to construct the OSDF to meet the functional requirements identified in Table 6. Even so, efforts were made throughout the project to improve the construction process. Several of the lessons learned and improvements made during construction are outlined below.

A critical aspect of the project was to always provide constructed disposal capacity in time to prevent delays in overall site remediation. During the nine year active life of the OSDF, cell construction and cell closure occurred concurrently with impacted material placement. The preferred construction season in Ohio starts in late April to early May and continues until mid-November. A full season was required to construct the multiple layers of a cell's liner or final cover system. Key to maintaining the construction schedule was the advanced procurement and processing of materials, including the necessary QC/QA testing. The procedure used for procurement of geosynthetics and processing site soils is described below. Early testing was also used for the granular material used for the various drainage layers and the biointrusion layer. These procedures were developed based lessons learned during the first year of construction.

Geosynthetics (i.e., geomembrane, GCL, and geotextile) were procured during the winter months preceding the start of the construction season. This was done to avoid delays that could result from rejection of any nonconforming materials already delivered to the site. To further reduce the possibility of delay, QC/QA personnel made plant visits to the geosynthetics manufacturers and obtained QC/QA samples as the materials were manufactured. All testing was conducted, and the results reviewed by QC/QA personnel, prior to releasing material from the plant to the OSDF site. Several times during the course of construction, the early testing of geosynthetics identified non-conforming materials.

At the start of construction for Cell 1, brown till was excavated, and tested for compliance with specifications. As had been learned in the test pad program, the brown till contained significant amounts of oversized material (> 2 in.). The contractor's initial plan was to remove oversize particles by hand. Hand picking was found to be very time consuming and slowed productivity. This experience led to the decision to pre-screen the material in the borrow area. Ultimately, soil processing for a phase of construction was conducted well in advance of the start of that phase. The processed soil was stockpiled in 5,000 to 10,000 cubic yard lots, and the required QC/QA testing was conducted during the processing operation. This allowed all conformance testing to be completed prior to the start of construction. Once test results demonstrated a stockpile met the CCL specification requirements, it was labeled as suitable for such. Soils not meeting the specification were reserved for other on-site uses. Pre-processing and early QC/QA testing of materials was a key factor in achieving construction schedule milestones.

As noted previously, a critical component of the design involved the creation of a watertight seal for the three LCS and LDS pipe penetrations through the liner system of each cell. Recognizing the importance of the penetrations, a special penetration box, prefabricated from HDPE flat stock, was developed. A detail of the penetration box is shown in Figure 25 and an actual box prior to installation is shown in Figure 26.

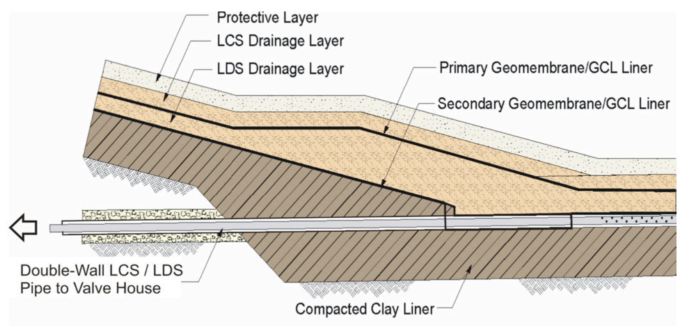

Figure 25. Detail of the penetration box used at LCS and LDS pipe penetrations.

Installation of the penetration box presented several challenges due to the detailed sequencing required for pipe installation outside the cell, earthwork, and CCL construction, geosynthetics installation, and pipe installation inside the cell. Also, placement of the box required detailed handwork to align the top of the box with the finished grade of the CCL. Two improvements were made on the installation process based on lessons learned during Cell 1 construction. The first lesson is that the boxes are heavy and difficult to move by hand. As a result, lifting rings that could be removed after installation were added to the boxes during fabrication. The lifting rings allowed a hoist or crane to be used to unload and position the penetration boxes. The second lesson was that it is difficult to cut an opening in the geomembrane panel and align it with the edge of the box. Thus, the geomembrane installer was allowed the option of welding a geomembrane skirt to the edge of the box and then trimming the skirt to fit the opening in the geomembrane panel. Overall, the boxes were found to be an effective means to seal the liner system at the locations of pipe penetrations (Figure 27). Additional information on the penetration boxes can be found in Vander Linde and Beech (2000).

Figure 26. Penetration box delivered from the fabricator, prior to installation. The penetration box was fabricated with a chamber to allow pressure testing of the welds. Welds were tested at the fabricator and following installation using the pressure gauge shown. Following pressure testing, the gauge was removed, and the chamber filled with bentonite pellets.

Figure 27. Liner penetration boxes installed in the secondary liner. HDPE geomembranes are welded directly to HDPE flat stock used to construct boxes.

A final aspect of construction worth noting is that electrical leak location surveys (ELLS) were performed on the composite primary liner as part of the QC/QA program for Cells 3 through 8 and for the composite cap component of the final cover system for all cells. In Cell 3, the ELLS was conducted only in the drainage corridor, while in Cells 4 through 8, it was conducted over the entire primary liner surface. The ELLS was conducted on the exposed primary geomembrane surface prior to placement of the geotextile cushion and the LCS drainage layer. The test procedure involved wetting the exposed geomembrane and then applying a direct current (DC) voltage to conductive media above (water) and below (GCL) the geomembrane. The electrical potential field in the conductive medium above the geomembrane was then monitored. The presence of a geomembrane defect completed an electrical circuit and created an anomaly in the potential field. Geomembrane defects found using the ELLS were repaired under the QC/QA program.

PERFORMANCE TO DATE

<u>Overview</u>

Filling of Cell 1 of the OSDF began in November 1997 with placement of the protective layer, so this cell has been functioning for nearly 15 years. Cells 2 through 8 have been functioning for successively shorter periods of time, with Cell 8 being the shortest at 7 years. OSDF operational data include LCS and LDS liquid removal rates, LCS and LDS liquid chemical constituent concentrations, and results from inspections of the final cover system. Overall, performance of the OSDF to date is within expectations and the project is considered by DOE to be a critical success. In September 2011, DOE issued a report titled "Third Five-Year Review Report for the Fernald Preserve" (DOE, 2011). The report summarizes DOE's findings with respect to OSDF performance as follows: "*The cap and liner systems of the OSDF are functioning as designed and are successfully containing disposed waste materials. The volume of leachate generated from the OSDF is continuing to decline, and the leachate is being effectively collected and treated to minimize impacts to human health and the environment.*"

<u>LCS and LDS Operational Data</u>

Available Data

Available operational data include cumulative combined monthly liquid volumes removed from the LCS of Cells 1 through 8 from January 2000 through December 2008, monthly liquid volumes removed from the LCS of each of the eight cells from January 2009 through June 2011, and monthly liquid volumes removed from the LDS of each of the eight cells from January 2000 through June 2011. Limited data are also available on the chemical constituents present in LCS and LDS flows.

LCS Flow Rates

Leachate generation rates in a typical landfill cell are expected to be highest at the start of cell operations (initial stage), decrease somewhat as the cell is filled (operational stage), and then decrease still further after cell closure. During the first few months after the start of waste disposal in a cell (initial stage), there is not sufficient covered waste in the cell to create grades that can produce runoff and much of the rain that falls into the cell percolates quickly into the cell LCS. To the extent rainfall occurs during this period, it will rapidly find its way to the LCS sump. LCS flow rates during this stage usually respond quickly to rainfall events. As the cell is progressively filled with waste (operational stage), some of the incident rainfall may fall onto covered waste slopes and become runoff. Some of the rainfall is also absorbed by the significant amount of waste now in the cell and only percolates slowly to the LCS, if at all. As a consequence, LCS flow rates decrease during the operational stage compared to the initial stage. After the cell has been closed with a final cover system, infiltration of rainwater into the waste is greatly reduced, resulting in a substantial reduction in LCS flow rates. In addition to rainwater, other sources of water may contribute to the production of leachate. As discussed below, another important source of water in the OSDF was water used for the suppression of dust during OSDF construction and operations.

LCS flow rates measured for the OSDF from January 2000 through June 2011 are shown in Figure 28 along with calculated LCS flow rates developed in 1995 during design of the OSDF. The primary sources of water contributing to leachate generation in the OSDF have been precipitation and water used to suppress dust generation during cell operations. The volume of dust suppression water was very significant for this facility as DOE had strict air quality criteria related to airborne particulates and radionuclides. The significance of the dust suppression activities is evidenced by the fact that in 2006, 10.6 million gallons of precipitation fell onto the active cells of the OSDF while 25.9 million gallons of dust suppression water were applied in that same year.

Figure 28 was developed by taking the total flow collected from the LCS for the entire facility and dividing it by the lined area of the cells that were constructed and contained waste (active and closed) at the time of the flow measurement. From Figure 28, it can be seen that the OSDF LCS flow rate was highest in early 2000 (monthly average of 3,700 gpad), averaged about 1,200 gpad from 2001 through 2004 and 700 gpad from 2004 through mid-2006, and then decreased rapidly as the OSDF was completely closed in October 2006. The monthly-average OSDF LCS flow rate decreased by approximately one order of magnitude within one month after facility closure and it has been stable at a monthly average value of approximately 10 gpad since that time. From Figure 28, it can be seen that monthly average leachate flows had considerable month to month variation. This variability was associated with the occurrence of rainfall events and the operational status of the various OSDF cells at any point in time. The very low and high "peakiness" of the measured LCS flow rates was also affected by the operating schedule for the leachate transmission system that conveyed leachate from the OSDF, through the valve houses, to the on-site advanced wastewater treatment plant.

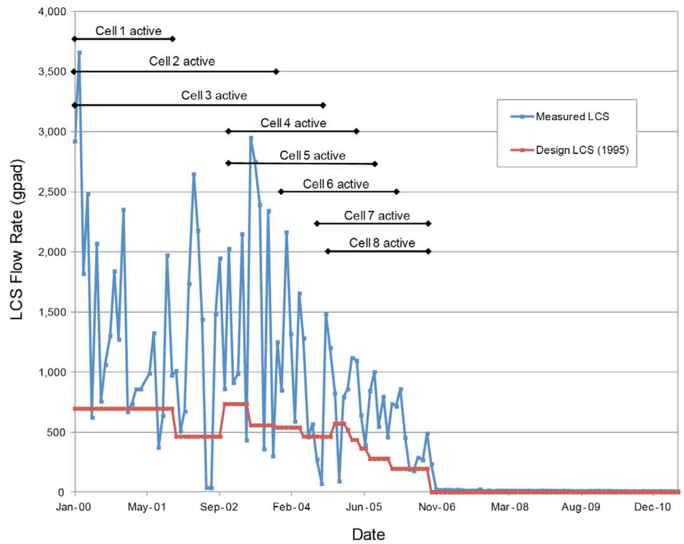

Figure 28. Measured and design monthly-average LCS flow rates for OSDF, January 2000 to June 2011.

The measured OSDF LCS flow rates can be compared to the original design estimates made in 1995 using the U.S. EPA HELP model. The estimated average annual leachate generation rates obtained using the HELP model were 1,150 (gpad) for a cell in the initial stage (10 ft thickness of waste), 700 gpad for a cell at the operational stage (30 ft. thickness of waste plus soil cover), less than 1 gpad shortly after closure, and 0.002 gpad at the end of the post-closure period. For the HELP analysis design estimates reported in Figure 28, for each month, an area-weighted summation was made of the HELP-calculated flow rates based on the status of each cell (i.e., initial stage, intermediate stage, or post closure). From Figure 28, it can be seen that the leachate generation rates calculated using actual LCS flow data are significantly higher than the original design estimates based on the HELP model. Two project-specific factors have been identified that likely account for the differences between the design estimates and actual leachate generation rates: (i) the presence of the impacted stormwater catchment area in each cell (previously described); and (ii) the large volume of dust suppression water applied to active cells. The stormwater catchment areas were introduced to the design after the HELP modeling was completed and the decision was made at the time not to revise the analyses. The application of dust suppression water was also not included in the original analyses.

The measured OSDF LCS flow rates can also be compared to the measured LCS flow rates in a U.S. EPA database reported by Bonaparte et al. (2002). The U.S. EPA database includes data for 73 municipal solid waste (MSW) landfill cells and 32 hazardous waste (HW) landfill cells. The LCS data are evaluated in several ways including as a function of average annual precipitation. The historical average annual precipitation in the Fernald vicinity is approximately 40 in. This rainfall average is closest to the northeast (NE) and southeast (SE) facility categories in the database. The average LCS flow rates in the database for the MSW cells in the NE and SE during the operational stage were 378 and 314 gpad, respectively, and for the HW cells, 576 and 524 gpad respectively. These average LCS flow rate results from the database are somewhat lower than the design values calculated in 1995 for the OSDF project using the HELP model, and significantly lower than the measured OSDF intermediate stage LCS flow rates.

Leachate generation rates after OSDF closure are compared to rates for other closed landfills in the U.S. EPA database, cited above, in Figure 29, taken from Bonaparte et al. (2002). This figure includes data for 11 MSW and 22 HW cells from the database, plus the 8 OSDF cells. The OSDF data are for 2009, 2010, and the first half of 2011. From the figure, it can be seen that the post-closure LCS flow rates for the OSDF are consistent across cells, on the order of 10 gpad, and, so far, not showing a decreasing trend during the post-closure period.

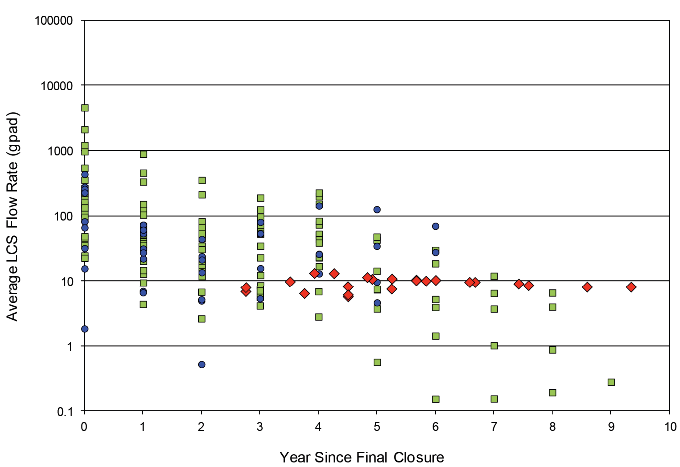

Figure 29. Average LCS flow rates (gpad) after closure for eleven MSW cells (shown as blue circles), 22 HW cells (shown as green squares), and the 8 OSDF cells at Fernald (shown as red diamonds). The OSDF data are for 2009, 2010, and the first half of 2011. The MSW and HW data are from Bonaparte et al. (2002).

LDS Flow Rates

The liquid of concern in monitoring the LDSs of the OSDF cells is leakage through the composite primary liner. However, leakage is not the only potential source of LDS liquids in a typical landfill, as indicated by Figure 30. Potential other sources of LDS liquids relevant to the liner system and siting configuration of the OSDF are drainage of water (mostly rainwater) that infiltrates the LDS during construction and drains after the start of facility operation (construction water), water expelled from the LDS during waste placement as a result of LDS compression under the weight of the impacted material (compression water), and infiltration of groundwater perched in the brown till that finds a pathway through the secondary composite liner (infiltration water). Gross et al. (1990) provides a detailed discussion of each of these potential sources along with methods to estimate liquid quantities from each source given site conditions and facility design details. At the start of cell operation, much of the LDS flow is often attributable to construction water. Continuing flow in the LDS after the construction water has fully drained can be due to one or several of the sources in Figure 30. Site-specific analyses using the methods described in Gross et al., coupled with comparisons of the chemical compositions of the LDS and LCS flows, allow evaluation of the most likely source(s) for the LDS flows.

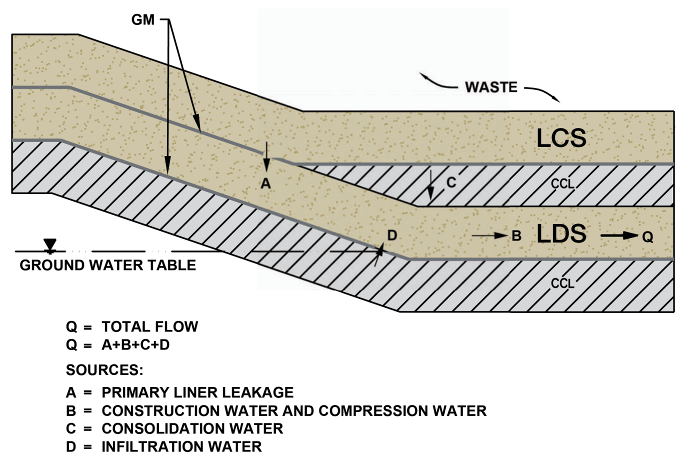

Figure 30. Potential sources of LDS liquids (modified from Bonaparte and Gross, 1990).

As an illustration of OSDF LDS performance, measured LDS flow rates for Cell 4 from December 2002, after the cell was just placed into service, to June 2011, approximately six years after Cell 4 was closed (April 2005), are shown in Figure 31. LDS flow rates ranged up to 5 gpad during cell operation then decreased after the cell was closed. Flow rates are currently stabilized at less than 0.1 gpad. Figure 31 also shows measured LCS flow rates for Cell 4 since February 2009. (LCS flow rates before this time were only available on a cumulative basis for all cells.) Cell 4 LDS flows since February 2009 have ranged from 3,000 to more than 5,000 times less than LCS flows, indicating that the leachate containment performance of the composite primary liner in Cell 4 has been very good. LDS flow rates from the other cells are also very low and demonstrate that the primary liners in the cells are containing the vast majority of leachate, allowing it to drain through the LCS pipes and be conveyed to the on-site advanced wastewater treatment facility. Figure 32 shows the post-closure LDS flow rates for all eight OSDF cells. All eight LDSs are producing very low flows, indicating effective performance of the composite primary liner in all cells.

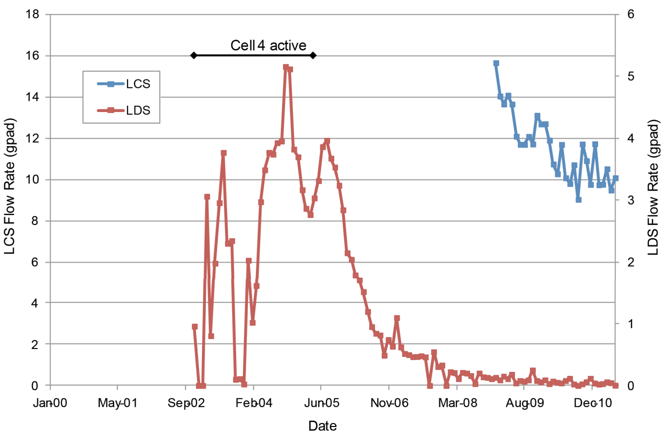

Figure 31. Measured monthly-average LCS and LDS flow rates for OSDF Cell 4, December 2002 to June 2011.

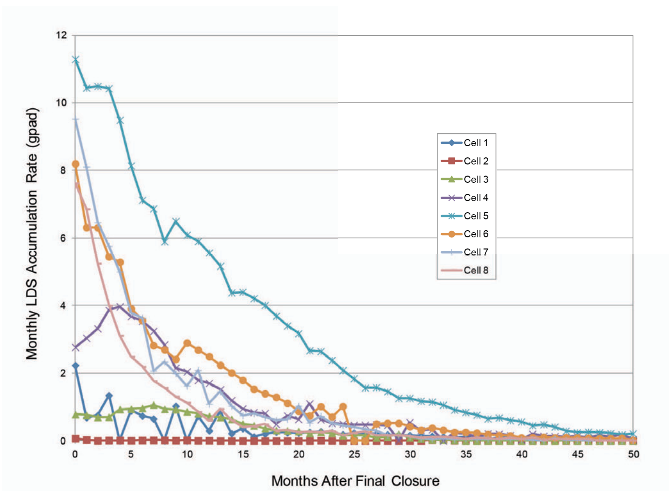

Figure 32. Monthly-average post-closure LDS flow rates for the eight OSDF cells.

The measured OSDF LDS flow rates can also be compared to average LDS flow rates reported in the U.S. EPA database in Bonaparte et al. (2002). The comparison is made for operating landfills with geomembrane/GCL composite primary liners (i.e., the OSDF primary liner). The database indicates the following typical monthly-average values for MSW and HW landfills: up to about 30 gpad during the initial stage, less than 1 gpad during the operational stage, and less than 0.2 gpad after closure. Inspection of Figure 31 shows that the measured LDS flow rates for Cell 4 were lower than the database values during the active life of the cell and are now approximately equal to the post-closure flow rates reported for the database. Based on Figure 32, post-closure LDS flow rates are generally consistent with the database.

Chemical Constituent Data

Average-annual uranium (total) concentrations in the LCS and LDS flows from each cell are available for 2006 to 2010. In addition, more limited data are available for a few other chemical constituents. Average uranium concentrations in the LCS flows have ranged from approximately 50 to 180 µg/L, more than three orders of magnitude lower than the concentration of total uranium in the OSDF test leachate used for the liner system compatibility test program (Table 8). Similarly, technetium-99 concentrations in OSDF leachate (typical concentration on order of 10 pCi/L) are three orders of magnitude lower than the concentration of this parameter in the test leachate. The much lower radiological activity of the actual OSDF leachate to the test leachate brings a degree of conservatism to the interpretation of the results of the leachate-geomembrane chemical compatibility testing program described earlier in this paper.

Average total uranium concentrations in the LDS flows from the eight ODF cells have been consistent, ranging from 11 to 24 µg/L. LDS uranium concentrations for individual cells were 3 to 9 times lower than the LCS uranium concentrations for the same cells. Figure 33, taken from Powell et al. (2011), shows an example uranium-sodium bivariate plot for the LCS, LDS, and horizontal till well (HTW, located in the till beneath the OSDF liner system) liquids for an individual OSDF cell. The data in Figure 33 indicate that the LCS and LDS flows for the OSDF cell have distinctly different chemical fingerprints and the water draining from the cell LDS is not principally leachate. The very low levels of uranium found in the LDS may be indicative of several possible sources, including minor amounts of primary liner leakage, site background contributions, and/or infiltration of perched groundwater. If the source is primary liner leakage, the bivariate plot suggests that the leachate is being diluted by non-leachate sources at a ratio of 3:1 to 5:1, meaning that only a fraction of the very low flows from the LDS for the subject cell could potentially be due to primary liner leakage.

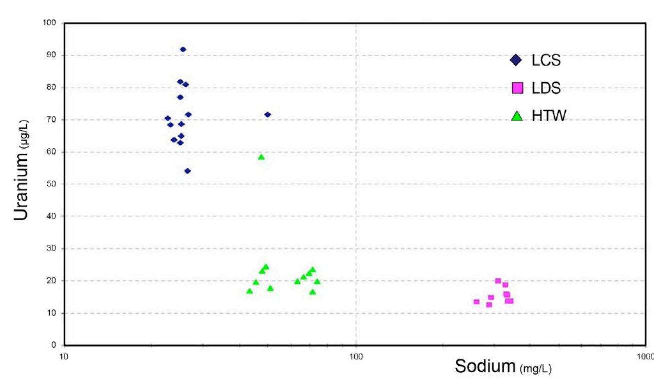

Figure 33. Uranium-sodium bivariate plot for typical OSDF cell. Figure is modified from Powell et al. (2011).

Apparent Leachate Collection Efficiency

To further evaluate the OSDF LCS and LDS flow data, apparent leachate collection efficiencies, ALCE (percent), were calculated for the LCS/composite primary liner (Bonaparte et al., 1996):

ALCE = (1 – LDS Flow Volume/LCS Flow Volume) × 100 (Equation 1)

The cumulative LCS and LDS flow volumes for the entire facility, and the calculated OSDF facility ALCE, are plotted in Figure 34.

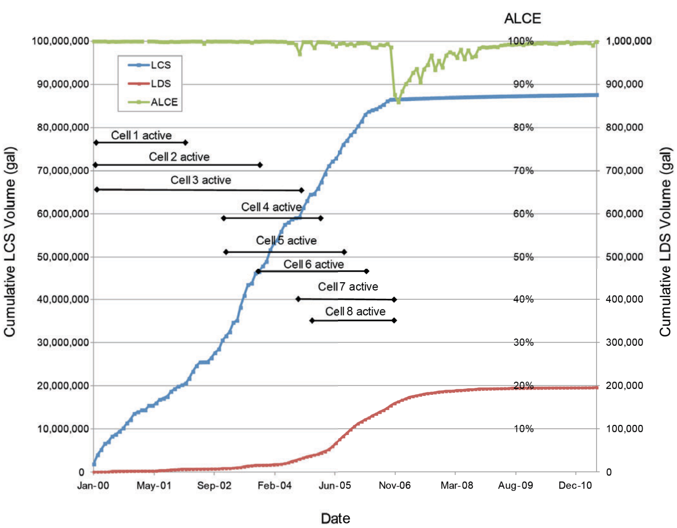

Figure 34. Cumulative volume of liquid removed from the LCS and LDS of cells 1 through 8 and calculated facility ALCE, January 2000 to June 2011.

The calculated ALCE for the OSDF for the period from January 2000 to June 2011 is 99.8 percent, which is consistent with efficiencies for composite liners reported by Bonaparte et al. (2002), and is indicative of a very high level of leachate containment and collection by, respectively, the composite primary liner and LCS. This means that the LDS has produced only 0.2 percent of the flow of the LCS. The true hydraulic efficiency of the composite primary liner in containing leachate is actually higher than this apparent value because, as discussed above, it has been shown through the bivariate plots that the LDS liquids are not principally leachate. Explained differently, the efficiency in Equation 1 is referred to as an apparent efficiency because, as previously described, there are potential sources for LDS flow other than leakage through the composite primary liner. Inclusion of any contributions from these other sources of flow as "apparent primary liner leakage" in the ALCE calculation in Equation 1 results in a conservative (i.e., low) calculated leachate collection efficiency. The lowest monthly ALCE calculated for the OSDF is 86 percent and corresponds to December 2006 shortly after the closure of Cells 7 and 8. While the LCS flows quickly dropped after closure, the LDS flows responded more slowly resulting in a temporary drop in ALCE. However, over time the ALCE values again increased to greater than 99 percent.

<u>Cover System Erosion and Maintenance</u>

The final cover system for the OSDF is inspected quarterly for erosion rills/channels, depressions, cracks, ponded water, seepage, vegetation coverage, presence of woody vegetation, evidence of biointrusion, and evidence of vehicle traffic. The final cover system was designed with a maximum calculated erosion rate of 4.3 tons/acre/year, based on the Modified Universal Soil Loss Equation (MUSLE). If erosion occurs at this maximum calculated rate, the thickness of the topsoil and vegetative soil layer of the final cover system will be reduced from 27 in. to 22 in. after 200 years, and to 3.5 in. after 1,000 years. Design calculations were also performed to evaluate the potential for rills and gullies to form in the final cover system during the 2,000-year storm event. Analyses conducted using the Temple (Temple et al., 1987), Horton/NRC (NRC, 1990), and permissibly velocity (NRC, 1990) methods indicated that the formation of rills and gullies would not occur during the design storm as long as the final cover system was vegetated with a well-established stand of native grasses. Further, in the event that erosion of cover soils did occur, the biointrusion barrier was designed to resist erosion in the 2000-year storm and probable maximum precipitation (PMP). The Stephenson (Abt et al., 1988) and Hartung and Scheuerlein (1970) methods were used to evaluate the stability of the biointrusion barrier under these design storm events. The analyses indicate that the biointrusion barrier layer would stop the advancement of gullies in the 2,000 year storm, although this scenario is not expected given the analysis results for the vegetation and topsoil layer described above.

Based on a review of quarterly inspection checklists, relatively modest levels of final cover system maintenance and repair have been required since OSDF closure. These include repair of erosion rills, repair of small mammal (e.g., mole to groundhog) burrows, management of non-native and noxious plant species, removal of rocks that surface as topsoil settles, and other general site maintenance (e.g., perimeter fence repair and litter removal). Since the start of 2007, 70 rills have been noted in quarterly inspection reports, some requiring repair by backfilling and reseeding. This equates to, on average, 1 rill per 600 feet of OSDF cover crest length per year. Rills are repaired if they reach 6 in. in depth. The frequency of erosion rills observed in the final cover system has decreased in the time since facilitate closure as vegetation coverage has increased and appears to have stabilized. Comparison of the inspection reports shows that the frequency of rillings is decreasing on an annual basis in parallel with the establishment of denser and more stable cover vegetation.

The initiation of rilling today is primarily associated with mammal burrows.

Animal burrows have occurred in the final cover system of every cell, have been documented each year in the final cover system of at least three of the eight OSDF cells, and have been found more frequently during winter months. Burrow holes are repaired by backfilling, re-seeding, mulching, and applying temporary irrigation. Due to the continuing presence of animal burrows, the biointrusion barrier component of the final cover system has proven integral to preventing vertical migration of the burrow holes.

From transects conducted on the final cover system, all cells have had at least 90% vegetative cover with at least 50% native species since 2010. Due to the progressive increase in vegetative coverage and growth since OSDF closure (Figure 35), erosion issues due to exposure of bare or poorly vegetated topsoil have largely been eliminated. Conversely, the frequency of undesired vegetation, such as woody materials, shrubs, and thistle, has increased. Current maintenance procedures require removal of these species at the root level, backfilling the root holes, and re-seeding. Additionally, spot applications of herbicides are routinely applied to control non-native species and noxious weeds.

Cell 1 (East Face, Sept 2007)

Cell 1 (East Face, Sept 2011)

Fig. 35. East face of Cell 1 showing the progressive increase in cover vegetation from September 2007 (A) to September 2011 (B).

ACKNOWLEDGEMENTS

The S. M. Stoller Corporation holds a contract with the DOE Office of Legacy Management which includes environmental monitoring and reporting at Fernald. Data and inspection results collected by Stoller can be found at www.lm.doe.gov/Fernald/. The views expressed in this paper are those of the authors and do not necessarily represent those of either DOE or the United States.

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Appendix A. Design strategy to achieve OSDF performance criteria.

	Potential Pathways for OSDF Performance Degradation				
	Internal Hydrologic Control	**External Hydrologic Control**	**Geotechnical Stability**	**Resistance to Erosion**	**Resistance to Biointrusion**
DESIGN FEATURE TO PREVENT PERFORMANCE DEGRADATION	• Double composite liner system to achieve leachate collection efficiency (LCE) > 99.9% and to provide LCE system performance monitoring • Thick HDPE geomembrane liner (80 mil) used to maximize service life • Thick compacted clay liner (3 ft.) remains functional through final period • Leachate collection and leak detection systems drain by gravity and are maintainable • Geochemical attenuation provided by 3 ft. of compacted clay liner, two geosynthetic clay liners, and at least 12 ft. of in-situ native gray till	• Facility designed to prevent uplift under extreme perched water conditions • Site designed to prevent stormwater runon to the OSDF under 2,000-year , 24-hour storm event • Facility sited or constructed out of 2,000-year floodplain • Multi-component soil and geosynthetic cover used to minimize infiltration into the OSDF • Thick HDPE geomembrane cap (60 mil) used to maximize service life • Thick compacted clay cap (2 ft.) remains functional through final period • Primarily above-ground facility allows visual monitoring and maintenance	• Located on stable glacial till foundation • Final slopes designed for stability (6H:1V) • No permanent seismic deformation under 2,400 year design seismic event • Impacted material placed and compacted in stable configuration • Construction materials selected to enhance stability (e.g., textured geomembrane)	• Facility geometry mimics local stable geomorphic landforms • Cover system has smooth transitions between top slopes and side slopes; corners are rounded • Cover system slopes designed to be gentle to limit runoff velocity (6H:1V) • Cover system slopes designed to resist gully initiation under design storm conditions • Predicted sheet erosion over 1,000 years is less than topsoil thickness • Biointrusion barrier beneath final cover system blocks potential depth of erosion or gullying	• Biointrusion barrier designed to impede plant root and animal intrusion • Primarily above-ground facility allows visual monitoring and maintenance • Access to site limited and institutional controls implemented

Appendix B. OSDF design calculations.

1. INTRODUCTION
   - 1.1 Design Parameter Summary
   - 1.2 Computer Program Validation
   - 1.3 Select Technical References
   - 1.4 Geotechnical Data

2. OSDF LAYOUT
   - 2.1 Required Volume
   - 2.2 Capacity Verification
   - 2.3 Earthwork Required Volume

3. GEOTECHNICAL – STATIC SLOPE STABILITY
   - 3.1 OSDF Foundation
   - 3.2 Liner System
   - 3.3 Impacted Material Configurations
   - 3.3.1 Interim
   - 3.3.2 Final
   - 3.4 Intercell Berm
   - 3.5 Final Cover System
   - 3.6 Access Corridor
   - 3.7 Borrow Area Cut Slopes

4. GEOTECHNICAL – SEISMIC SLOPE STABILITY
   - 4.1 Hazard Assessment
   - 4.2 Site Response Analysis
   - 4.3 Performance Analysis
     - 4.3.1 Pseudo-Static Stability
     - 4.3.2 Deformation Analysis

5. GEOTECHNICAL – SETTLEMENT
   - 5.1 Foundation Settlement
   - 5.2 Localized Impacted Material Settlement
   - 5.3 Overall Impacted Material Settlement

6. LINER SYSTEM
   - 6.1 Hydrostatic Uplift
   - 6.2 Liner Geosynthetics Selection
     - 6.2.1 Geosynthetic Clay Liner
     - 6.2.2 Geomembrane Liner
     - 6.2.3 Geotextile Cushion
     - 6.2.4 Geosynthetic Selection to Preclude Tension
   - 6.3 Liner Frost Protection

7. LEACHATE MANAGEMENT-LEACHATE GENERATION
   - 7.1 Calculated Rates
     - 7.1.1 During Filling
     - 7.1.2 After Closure
   - 7.2 Required Cell Storage

8. LEACHATE MANAGEMENT - LEACHATE COLLECTION SYSTEM
   - 8.1 Maximum Head in LCS
     - 8.1.1 Maximum Head in LCS Drainage Layer
     - 8.1.2 Maximum Head in LCS Drainage Corridor
   - 8.2 Geotextile Filter Design
     - 8.2.1 Geotextile Filtration
     - 8.2.2 Geotextile Biological Clogging Potential
   - 8.3 LCS Pipe Design
     - 8.3.1 LCS Pipe Flow Capacity
     - 8.3.2 LCS Pipe Perforation Sizing
     - 8.3.3 LCS Pipe Structural Stability

9. LEACHATE MANAGEMENT – LEAK DETECTION SYSTEM
   - 9.1 Migration through Primary Liner
   - 9.2 Maximum Head in LDS
     - 9.2.1 Maximum Head in LDS Drainage Layer
     - 9.2.2 Maximum Head in LDS Drainage Corridor
   - 9.3 Time of Travel in LDS
   - 9.4 LDS Pipe Design
     - 9.4.1 LDS Pipe Flow Capacity
     - 9.4.2 LDS Pipe Perforation Sizing
     - 9.4.3 LDS Pipe Structural Stability
   - 9.5 Action Leakage Rate

10. LEACHATE MANAGEMENT - LEACHATE TRANSMISSION SYSTEM
    - 10.1 Permanent LTS Gravity Line Design
      - 10.1.1 LTS Gravity Line Flow Capacity
      - 10.1.2 LTS Gravity Line Structural Stability
      - 10.1.3 LTS Gravity Line Frost Protection
    - 10.2 Temporary Gravity Line Design
      - 10.2.1 Temporary Gravity Line Flow Capacity
    - 10.3 LCS and LDS Manhole Design
      - 10.3.1 LCS and LDS Manhole Uplift
      - 10.3.2 LCS and LDS Manhole Structural Design
    - 10.4 Permanent Lift Station
      - 10.4.1 Permanent Lift Station Storage Volume
      - 10.4.2 Permanent Lift Station Manhole Uplift
      - 10.4.3 Permanent Lift Station Structural Design
    - 10.5 Permanent LTS Pipe Hydrograph

11. FINAL COVER SYSTEM
    - 11.1 Temporary Erosion Mat Design
    - 11.2 Vegetation Design
    - 11.3 Cover System Erosion Resistance
    - 11.4 Cover Frost Penetration Depth
    - 11.5 Granular Filter Layer Design
    - 11.6 Biointrusion Barrier Design
    - 11.7 Drainage Layer Design
      - 11.7.1 Cover System Water Balance
      - 11.7.2 Cover Drainage Layer Maximum Head
    - 11.8 Cover Geosynthetics Selection
      - 11.8.1 Geotextile Cushion
      - 11.8.2 Geomembrane Cap
      - 11.8.3 Geosynthetic Clay Cap

12. SURFACE WATER MANAGEMENT DURING OSDF CONSTUCTION/ FILLING/CLOSURE
    - 12.1 Stormwater Runon/Runoff and Drainage Control Structures
    - 12.2 OSDF Sediment Basins

13. SURFACE WATER MANAGEMENT AFTER OSDF CLOSURE
    - 13.1 Stormwater Runon/Runoff and Drainage Control Structures
      - 13.1.1 Northern Area
      - 13.1.2 Eastern Area
      - 13.1.3 Southern Are
      - 13.1.4 Western Area
    - 13.2 Drainage Control Structure Erosion Resistance

14. SUPPORT FACILITIES
    - 14.1 Electrical Power Demand
    - 14.2 Potable Water Demand
    - 14.3 Sanitary Wastewater Discharge
    - 14.4 Construction Water Demand
    - 14.5 Decontamination Facility Water Demand
    - 14.6 Decontamination Facility
    - 14.7 Construction Admin. Area Surfacing
    - 14.8 Construction Haul Road
    - 14.9 Leachate Transmission System Access Corridor

15. BORROW AREA
    - 15.1 Borrow Area Required Volume
    - 15.2 Borrow Area Capacity Verification
    - 15.3 Borrow Area Water Demand
    - 15.4 Stormwater Runoff Routing
    - 15.5 Borrow Area Sediment Basin

16. IMPACTED MATERIALS MANAGEMENT
    - 16.1 Haul Road Design
    - 16.2 Impacted Runoff from Haul Road
    - 16.3 OSDF Methane Generation
    - 16.4 OSDF Radon 222 Release

17. HORIZONTAL MONITORING WELL
    - 17.1 Differential Settlement and Tensile Strain
    - 17.2 Structural Stability

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