Form WR-23 FIELD ENGR. LOG STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Maljamar Repressuring Agreement #6
Street and Number
City State
Well was drilled under Permit No. L-4020 and is located in the SW ¼ SW ¼ SE ¼ of Section 2 Twp. 17 Rge. 32
(B) Drilling Contractor George Pennington License No.
Street and Number
City Loco Hills, State New Mexico
Drilling was commenced 19
Drilling was completed June 2, 19 50

Elevation at top of casing in feet above sea level Total depth of well 200 ft.
State whether well is shallow or artesian shallow Depth to water upon completion

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	139	195	60	Sand and little gravel
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7			0	196	196		153	196
10 3/4			0	145	145	Pulled as	well was gravel	packed.

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

File No. L-4020 Use S. R. O. O. Location No. 17.32.2.433 43

#5 well 2-132-1

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	20		brown	Top soil
20	45			Caliche
45	100		red	Sandrock
100	135			Sand and little gravel (water section)
195	200		red	Shale
				Driller estimated that well was good for
				100 gallons of water per minute.
				This well is located in State Section 2,
				T. 17 S., R. 32 E., N.M.P.M., Lea County,
				New Mexico.
				L S Elev 4195
				Depth to K Trc 135
				~~Elev of K~~ Trc 4060
				17.32.2.43343
				Loc. No.
				Hydro. Survey Field Check X
				SOURCE OF ALTITUDE GIVEN
				Interpolated from Topo. Sheet X
				Determined by Inst. Leveling
				Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

George Pennington
Well Driller

L-4020

17.32.2.433

Form WR-23 FIELD ENGR. LOG STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Maljamar Repressuring Agreement #5

Street and Number

City State

Well was drilled under Permit No. L-4019 and is located in the SE ¼ SW ¼ SE ¼ of Section 2 Twp. 17 Rge. 32

(B) Drilling Contractor Ed. Burke License No.

Street and Number

City Hobbs, State New Mexico

Drilling was commenced 19

Drilling was completed May 6, 1948

Elevation at top of casing in feet above sea level Total depth of well 182 ft.

State whether well is shallow or artesian Depth to water upon completion

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	To	Thickness in Feet	Description of Water-Bearing Formation
1	126	180		Red water sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Bottom	Feet	Type Shoe	Perforations From	To
7			0	182	182		113	182

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
0	182	10			

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

File No. L-4019 Use S.R.O.C. Location No. 17.32.2.434 3/4

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	20		brown	Top Soil
20	38		brown	Loose sand
38	70		grey	Firm sand
70	82		brown	Loose sand
82	98		red.	Sandrock
98	126		brown	Sand and gravel
126	180		red	Water sand
180	182		red	Shale

This well is located in State Section 2, T-17 S., R. 32 E., N.M.P.M., Lea County, New Mexico.

L S Elev 4195
~~Depth to K~~ Trc 180
Elev of K Trc 4015

~~Loc. No.~~ 17.32.2.43434
~~Hydro. Survey~~ ~~Field Check~~ X

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
Determined by Inst. Leveling
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Ed. Burke
Well Driller

L-4019

17.32.2.434

Form WR-23 FIELD ENGR. LOG STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Maljamar Co-op Repressuring Agreement #7

Street and Number

City State

Well was drilled under Permit No. L-4021 and is located in the SW 1/4 SE 1/4 SE 1/4 of Section 2 Twp. 17 S. Rge. 32 E.

(B) Drilling Contractor George Pennington License No.

Street and Number

City State

Drilling was commenced 19

Drilling was completed June 14, 19 50

Elevation at top of casing in feet above sea level Total depth of well 190 ft.

State whether well is shallow or artesian shallow Depth to water upon completion

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	160	185	25	Sand and little gravel.
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7			0	197	197		153	197
10 3/4			0	155	155	Pulled as well was gravel packed.		

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

File No. L-4021 Use S. R. O. Q. Location No. 17.32.2.44333

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	20		brown	Top soil
20	50			Caliche
50	120		Brown	Loose sand
120	160		red	Sand rock
160	185			Sand and little gravel (water section)
185	190		red	Shale
				Eight yards of pea gravel was placed between
				10-3/4" pipe and 7" pipe; 10-3/4" pipe runs
				to 155' and pulled as well was graveled.
				Driller estimated that well was good for
				100 gallons of water per minute.
				This well is located in State Section #2,
				T-17S, R-32E, NMPM, Lea County, New Mexico.
				10" hole was drilled by George Pennington
				of Loco Hills, New Mexico. Completed
				June 14, 1950.
				L S Elev 4203
				Depth to K Trc 185
				~~Elev of~~ K ~~Trc~~ 4018
				F 17.32.2.44333
				Loc. No.
				Hydro. Survey Field Check X
				SOURCE OF ALTITUDE GIVEN
				Interpolated from Topo. Sheet X
				Determined by Inst. Leveling
				Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

George Pennington
Well Driller

L-4021

17.32.2.443

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Mescalero Ridge WAter Coop. Owner's Well No. ______

Street or Post Office Address P.O. Box 49

City and State Maljamar, NM 88264-0002

Well was drilled under Permit No. L-4021-S and is located in the:

a. ____ ¼ NE ¼ SE ¼ SE ¼ of Section 3 Township 17S Range 32E N.M.P.M. in Lea County.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______ Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Alan Eades License No. WD1044

Address 1200 E. Bender Blvd., Hobbs, NM 88240

Drilling Began 1-21-02 Completed 1-21-02 Type tools rotary Size of hole 9 7/8 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 260 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
185	257	72	Sand & SAndy Brown Clay	
			Stringers	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6	160psi				260		180	260

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______

Address ______

Plugging Method ______

Date Well Plugged ______

Plugging approved by: ______

State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY #215199

Date Received 02/05/02

Quad ______ FWL ______ FSL ______

File No. L-4021-S Use Suppl Location No. 17.32.3~~443~~.23422

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material [illegible]red
From	To		
0	1	1	Top Soil
1	26	25	Caliche
26	90	64	Sand
90	132	42	Sandy Brown Clay & Sandstone Stringers
132	185	53	Sand & Sandstone Stringers
185	257	72	Sand & SAndy Brown Clay Stringers
257	260	3	Red Clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Alan Eades by Andrea Root

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ____ ¼ ____ ¼ ____ ¼ ____ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.32.3.4323334

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	40		Caliche
40	116		Anhydrite and sand
116	150		Sand
150	363		Red bed
363	695		Red bed and shells
695	990		Red shale with shells

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.3.4323334 Elevation: 4284' GL
Owner: Chevron U.S.A. Inc.
Maljamar (Grayburg) Unit #12
Record of Casing: 8 5/8" - 1344'

Rotary

660' FSL - 1905' FEL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This fo[illegible]hould be executed in triplicate, preferably typewritten, and submitted t[illegible] appropriate district office of the State Engineer. [illegible]ons, except Section 5, shall be answered as completely and accura[illegible] possible when any well is drilled, repaired or deepe[illegible]hen this form is used as a plugging record, only Section 1(a) and Section [illegible]eed be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use Oil Location No. 17.32.3.44300

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	115		Caliche ?
115	255		Red rock
255	290		Sand ?
290	1055		Red rock
			L S Elev 4285 4285 Depth to K 290? Trc 115 ? Elev of K 3995? Trc 4170 ?

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.3.44300 Elevation: 4285' DF
Owner: Chevron Oil Co.
Maljamar (Grayburg) Unit #14
Record of Casing: 8 5/8" - 1275'

Rotary

330' FSL - 990' FEL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Harry [illegible]

Street and Number

City Maljamar State New Mexico

Well was drilled under Permit No. Misc. 2-1-59 and is located in the NE ¼ NE ¼ NW ¼ of Section 10 Twp. 17S Rge. 32E

(B) Drilling Contractor C. O. Aldredge License No. WD-79

Street and Number Box 379

City Lovington State New Mexico

Drilling was commenced December 23 1961

Drilling was completed January 1, 1962

Elevation at top of casing in feet above sea level ______ Total depth of well 156

State whether well is shallow or artesian shallow Depth to water upon completion 132

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	132	156	24	Red water sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
6 5/8	welded		0	156	156	none	136	156

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		7			5 sacks of drilling mud poured
					in top of hole while drilling
					well to keep hole from caving

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19____

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

ROSWELL N. MEX
DISTRICT II
STATE ENGINEER OFFICE

Date Received 1962 JAN 18 AM 8:14

File No. Misc. 2-6-59 Use Dom Location No. 17.32.10.122

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	5	5	Brown	Top Soil
5	12	7	White	Caliche Rock
12	132	120	Brown	Sandy Clay
132	156	24	Red	Water Sand

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

C. O. Aldredge

Well Driller

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ConocoPhillips Owner's Well No. EW-1
Street or Post Office Address P.O. BOX 2197
City and State Houston, TX 77252

Well was drilled under Permit No. ______ and is located in the:

a. ___ ¼ ___ ¼ ___ ¼ ___ ¼ of Section 21 Township 17S Range 32E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the S.W. Qtr Maljamar E+P1

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in LEA County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor SCARBOROUGH DRILLING, INC. License No. WD1188

Address P.O. BOX 305, LAMESA, TX 79331 806-872-3285

Drilling Began 5-14-2007 Completed 5-15-2007 Type tools Air Rotary Size of hole ______ in.

Elevation of land surface or UNKNOWN at well is ______ ft. Total depth of well 125 ft.

Completed well is ☑ shallow ☐ artesian. Depth to water upon completion of well N/A ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)

STATE ENGINEER OFFICE 2007 JUL 16

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet Top	Depth in Feet Bottom	Length (feet)	Type of Shoe	Perforations From	Perforations To
6	sch 40	PVC	+2	95		.020	95	125

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
0	80	8 3/4	cement		poured
80	120	8 3/4	bentonite		poured
120	125	SAND	SAND		poured

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet Top	Depth in Feet Bottom	Cubic Feet of Cement
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received

Quad ______ FWL ______ FSL ______

File No. No File number w OSE Use monitor well Location No. 17.32.21 SW

Section 6. LOG OF HOLE

Depth in Feet From	To	Thickness in Feet	Color and Type of Material Encountered
0	5	5	SAND: yellowish red, vfg, dry, damp, loose.
5	20	15	Silty SAND: reddish brown - reddish yellow, loose.
20	25	5	SANDSTONE: Loosely consolidated, very pale brown
25	60	35	SAND Minorly consolidated, lt. reddish brown,
			soft, dry, silty, vf-fine, non plastic
60	80	20	SAND Loosely consolidated, very pale brown
			vf-fine, silty, non plastic, dry
80	90	10	SANDSTONE w/shale interbeds; lt yellowish
			brown loosely consolidated, vf-f, dry.
90	110		SAND interbedded w/shale; SAND -
			yellowish brown, loose, slightly damp,
			silty. Shale; brown, very fine
110	120	10	SAND interbedded w/shale: SAND -
			yellowish brown, loose, slightly
			damp, non plastic, silty. Shale:
			greenish gray, silty, dense
120	125	5	SAND, lt. yellowish brown, damp,
			slightly plastic, loose, vf-fine, silty.

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Lee Sedlaczek
Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office ... All sections except Section 5 shall be answered as completely and accurately as possible when any well is

Form WR-23 FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(A) Owner of well Texaco, Inc.
Street and Number Box 3109
City Midland, State Texas
Well was drilled under Permit No. L-5288 and is located in the [illegible] ft. from S. line, [illegible] ft. from East line ¼ SE ¼ ¼ of Section 36 Twp. 17S Rge. 34E

(B) Drilling Contractor [illegible] License No. [illegible]
Street and Number Box 56
City Hobbs, State New Mexico
Drilling was commenced Jan. 11, 1965
Drilling was completed Jan. 17, 1965

#5 on Cap-2-147-6

(Plat of 640 acres)

Elevation at top of casing in feet above sea level Unkown 3788 Total depth of well 231
State whether well is shallow or artesian Shallow Depth to water upon completion 90

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	105	155	50	Sand, consolidated, course
2	175	195	20	Sand
3	220	228	8	Sand & gravel
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
10 3/4	32.75	8	0	231	231	None	105-231	

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor ________ License No. ________
Street and Number ________ City ________ State ________
Tons of Clay used ________ Tons of Roughage used ________ Type of roughage ________
Plugging method used ________ Date Plugged ________ 19____
Plugging approved by:

________ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received 1965 JAN 21 AM 8:24 STATE ENGINEER OFFICE

File No. L-5288 Use Comm Location No. 17.34.36.443 134

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	2	2	Black	Soil & rock
2	26	26	White	Caliche & rock
26	80	52	Grey	Sandy shale
80	85	5	"	Sand rock
85	140	55	"	Sand
140	155	15	"	Sand rock
155	165	10	"	Sandy shale
165	195	30	"	Sand & sand rock
195	220	25	"	Sand rock
220	228	8	"	Sand & gravel
228	231	3	Red	Red bed, shale

L S Elev 3988
~~Depth to K~~ Trc 228
Elev of K Trc 3760

~~Loc. No.~~ 17.34.36.443134
~~Hydro. Survey~~ ~~Field Check~~ X

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling

Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

O. R. Musslewhite
Well Driller

L-5288

17.34.36.443

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well George Kenemore Owner's Well No. RA 8855
Street or Post Office Address PO Box 154
City and State Maljamar NM

Well was drilled under Permit No. RA 8855 and is located in the:

a. SE ¼ NW ¼ NW ¼ ____ ¼ of Section 10 Township 17 S Range R 32 E N.M.P.M.

b. Tract No. ____ of Map No. ____ of the ____

c. Lot No. ____ of Block No. ____ of the ____
Subdivision, recorded in Lea County.

d. X= ____ feet, Y= ____ feet, N.M. Coordinate System ____ Zone in the ____ Grant.

(B) Drilling Contractor J & K Drilling License No. WD 1235

Address Box 1493 Lovington NM 88260

Drilling Began 7/28/94 Completed 8/4/94 Type tools Cable Size of hole 8½ in.

Elevation of land surface or ____ at well is ____ ft. Total depth of well 158 ft.

Completed well is [X] shallow [ ] artesian. Depth to water upon completion of well 0 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
			No water was found drilling	
			this well.	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
	No csg	was ran	in well					

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

94 AUG 10 PM 3 42
STATE ENGINEER OFFICE

Section 5. PLUGGING RECORD

Plugging Contractor ____
Address ____
Plugging Method ____
Date Well Plugged ____
Plugging approved by: ____

State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received August 10, 1994

Quad ____ FWL ____ FSL ____

150

File No. RA-8855 Use Domestic Location No. 17.32.10.11421

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	18	18	Sand top soil light brown in calor
18	20	2	Caliche
20	38	18	red sand
38	40	2	med hard sandstone, red in calor
40	50	10	white sand with red gravel
50	60	10	red sand with red and black flintstone gravel
60	80	20	Brown colored sand with red and white colored sandstone gravel.
80	135	55	Red shale with mixture of multi-colored grave.
135	157	22	Red colored shale with red, blue, and gray gravel
157	158	1	Red bed

Section 7. REMARKS AND ADDITIONAL INFORMATION

Drilled well to 158 feet, 1 foot into Red Bed lormation. No water was encountered.while drilling this well. Owner wants to go on to 200 feet. Rigged down and moved off hole. Hole was left openwith 12 foot 9 5/8 csg in top of well.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Earl Ellison

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. A[illegible]ions, except Section 5, shall be answered as completely and accurate[illegible] possible when any well is drilled, repaired or de[illegible] [illegible]hen this form is used as a plugging record, only Section 1(a) and Sectio[illegible] [illegible]d be completed.

F. Pr.
108

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Maljamar Cooperative Repressuring Agreement

Street and Number Room 200, Booker Bldg.,

City Artesia, State New Mexico

Well was drilled under Permit No. L-2-L51 and is located in the NW ¼ SW ¼ NE ¼ of Section 11 Twp. 17 Rge. 32

(B) Drilling Contractor Burke License No.

Street and Number Hobbs,

City State New Mexico

Drilling was commenced 19

Drilling was completed September 10, 19 47.

Elevation at top of casing in feet above sea level Total depth of well 140 ft.

State whether well is shallow or artesian Depth to water upon completion

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7			0	139	139			

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

File No. L-2-L-51 Use S.R.O. Location No. 17.32.11.~~231.44~~

Well #7 Malj. 2-/32-1 23/432

Section 6

LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	5			Top soil
5	22		white	Packed sand
22	48		gray	Soft sand
48	93		red	Soft sand
93				Top of water sand
93	131			Coarce water sand
131				Bottom of sand
131	140		red	Clay
140				Total depth.

139' of 7" OD Lapwell pipe run, consisting of the last two joints perforated, which amounted to 43'. Total water sand thickness 38'. Hole was bailed in an effort to create a crevice and remove as much sand as possible. Well was gravel packed with 9½ yards. It is hoped that more gravel can be placed between casing and the outer wall after well has been pumped.

It is estimated that the well is capable of producing 100 gallons per minute. This well was completed on September 10, 1947. It was drilled by Burke, Phone No. 90, Hobbs, N. M.

L S Elev 4142

Depth to K Trc 131

Elev of K Trc 4011

17.32.11.23144

Loc. No.

Hydro. Survey ____ Field Check X

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling ____

Other ____

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Burke
Well Driller

L-2-L-51

17.32.11.231

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.32.26.41000

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	15		Caliche
15	80		Red clay
80	105		Red shale
105	210		Red bed
210	265		Blue shale
265	710		Red bed
710	850		Red sand (water 710-810)
850	983		Red bed
983	995		Red sand
995	1024		Red bed

L S Elev 3936
Depth to K Trc 15
Elev of K Trc 3921

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.26.41000 Elevation: 3936' DF
Owner: Continental Oil Co.
MCA Battery 4 #189
Record of Casing: 8 1/4" - 1062'

Cable

1980' FSL--1980' FEL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well FLO CO2 INC. Owner's Well No. ____
Street or Post Office Address 3700 Kermit Hwy.
City and State Odessa, TX 79764

Well was drilled under Permit No. RA-10175 and is located in the:

a. ____ ¼ ____ ¼ NE ¼ NW ¼ of Section 28 Township 17S Range 32E N.M.P.M.
in Lea County.
b. Tract No. ____ of Map No. ____ of the ____
c. Lot No. ____ of Block No. ____ of the ____
Subdivision, recorded in ____ County.
d. X= ____ feet, Y= ____ feet, N.M. Coordinate System ____ Zone in the ____ Grant.

(B) Drilling Contractor Alan Eades License No. WD 1044

Address 1200 E. Bender Blvd., Hobbs, NM 88240

Drilling Began 2-4-02 Completed 2-4-02 Type tools rotary Size of hole 7 7/8 in.

Elevation of land surface or ____ at well is ____ ft. Total depth of well 158 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well ____ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
87	89	2	Sand & Gravel	
89	116	27	Sandy yellow & blue clay	
116	124	8	Hard gray shale	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet Top	Depth in Feet Bottom	Length (feet)	Type of Shoe	Perforations From	Perforations To
5 3/4	160psi				158		118	158

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement

Section 5. PLUGGING RECORD

Plugging Contractor ____
Address ____
Plugging Method ____
Date Well Plugged ____
Plugging approved by:

____
State Engineer Representative

No.	Depth in Feet Top	Depth in Feet Bottom	Cubic Feet of Cement
1			
2			
3			
4			

Date Received 03/06/2002 FOR USE OF STATE ENGINEER ONLY T# 222219

File No. RA-10175 Quad ____ FWL ____ FSL ____

Use Drnk & Sanitary Loc No. 17S.32E.28.12

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	1	1	Top Soil
1	8	7	Sand w/ clay & Sandstone Stringers
8	44	36	Sand & Sandstone Stringers
44	55	11	Sandy Red Clay
55	87	32	SAndy yellow & blue clay
87	89	2	Sand & Gravel
89	116	27	Sandy yellow & blue clay
116	124	8	Hard gray shale
124	158	34	Yellow, blue & red clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Alan Eades by Andrea Root

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use Oil Location No. 17.32.29.11000

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	70		Surface sand
70	190		Red bed
			L S Elev 3937 Depth to K Trc 70 Elev of K Trc 3867

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.29.11000 Elevation: 3937' GR
Owner: Continental Oil Co.
MCA Unit Battery 2 #109
Record of Casing: 8" - 873'

Cable

660' FNL - 660' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______________________ Owner's Well No. ________
Street or Post Office Address ______________________
City and State ______________________

Well was drilled under Permit No. ______________ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______________________

c. Lot No. ______ of Block No. ______ of the ______________________
Subdivision, recorded in ______________ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______________ Zone in the ______________________ Grant.

(B) Drilling Contractor ______________________ License No. ______

Address ______________________

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______________ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______________________
Address ______________________
Plugging Method ______________________
Date Well Plugged ______________________
Plugging approved by:

____
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.32.29.24000

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	85		Surface sand and caliche
85	105		Sandstone
105	755		Shale

L S Elev 3984
Depth to K Trc 105
Elev of K Trc 3879

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.29.24000 Elevation: 3984' DF
Owner: Continental Oil Co.
MCA Unit Battery 2 #154
Record of Casing: 8" - 860'

Cable

1980' FNL - 660' FEL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______________________ Owner's Well No. __________
Street or Post Office Address ______________________
City and State ______________________

Well was drilled under Permit No. __________ and is located in the:

a. _____ ¼ _____ ¼ _____ ¼ _____ ¼ of Section _____ Township _____ Range _____ N.M.P.M.

b. Tract No. _____ of Map No. _____ of the ______________________

c. Lot No. _____ of Block No. _____ of the ______________________
Subdivision, recorded in __________ County.

d. X= __________ feet, Y= __________ feet, N.M. Coordinate System __________ Zone in
the ______________________ Grant.

(B) Drilling Contractor ______________________ License No. __________

Address ______________________

Drilling Began __________ Completed __________ Type tools __________ Size of hole _____ in.

Elevation of land surface or __________ at well is _____ ft. Total depth of well _____ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well _____ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______________________
Address ______________________
Plugging Method ______________________
Date Well Plugged ______________________
Plugging approved by:

____
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad __________ FWL __________ FSL __________

File No. __________ Use 011 Location No. 17.32.29.32000

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	55		Sand and caliche
55	350		Red mud
350	470		Red shale
	✓		
			L S Elev 3933
			Depth to K Trc 55
			Elev of K Trc 3878

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.29.32000 Elevation: --- 3933

Owner: Continental Oil Co.

MCA Unit Battery 2 #170

Record of Casing: 8" - 990'

Cable

1980' FSL - 1980' FWL

✓

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______________________ Owner's Well No. __________
Street or Post Office Address ______________________
City and State ______________________

Well was drilled under Permit No.__________ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No.______ of Map No. ______ of the ______________________

c. Lot No.______ of Block No. ______ of the ______________________
Subdivision, recorded in ______________ County.

d. X= ______ feet, Y=______ feet, N.M. Coordinate System ______________ Zone in the ______________________ Grant.

(B) Drilling Contractor ______________________ License No.__________

Address ______________________

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______________ at well is______ ft. Total depth of well______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______________________
Address ______________________
Plugging Method ______________________
Date Well Plugged______________________
Plugging approved by:

____
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad 107.1.0 FWL ______ FSL______

File No.______________ Use 011 Location No. 17.32.29.33000

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	45		Sand and caliche
45	85		Red sand
85	125		Caliche
125	400		Red bed
			L S Elev 3919 ?
			Depth to K______Trc 85 ?
			Elev of K______Trc 3834 ?

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.29.33000

Owner: Continental Oil Co. 2/4

MCA Unit Battery 2 ~~#124~~

Record of Casing: 8" - 1050'

Cable

660' FSL - 660' FWL OK

Elevation: 4091' DF OK

Topo. Elev. 3909

DF Elev. 3919

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______________________ Owner's Well No. ________

Street or Post Office Address ______________________

City and State ______________________

Well was drilled under Permit No. ______________ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______________________

c. Lot No. ______ of Block No. ______ of the ______________________
Subdivision, recorded in ______________ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______________ Zone in the ______________________ Grant.

(B) Drilling Contractor ______________________ License No. ______________

Address ______________________

Drilling Began ______________ Completed ______________ Type tools ______________ Size of hole ______ in.

Elevation of land surface or ______________ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______________________

Address ______________________

Plugging Method ______________________

Date Well Plugged ______________________

Plugging approved by:

____

State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______________ FWL ______________ FSL ______________

File No. ______________ Use 011 Location No. 17.32.30.13000

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	50		Surface formation
50	575		Red bed
575	580		Shale (water)
580	675		Red bed
675	810		Anhydrite
810	820		Sand water
			L S Elev 3895
			Depth to K Trc 50
			Elev of K Trc 3845

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.30.13000 Elevation: 3895' DF
Owner: Continental Oil Co.
MCA Unit Battery 1 #163
Record of Casing: 8" - 870'

Cable

1980' FNL - 660' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.32.30.33000

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	50		Surface sand and caliche
50	545		Red bed and red rock
545	590		Red bed, sandy
			L S Elev 3871
			Depth to K___Trc 50
			Elev of K___Trc 3821

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.30.33000 Elevation: 3871' DF
Owner: Continental Oil Co.
MCA Unit Battery 1 #218
Record of Casing: 10 3/4" - 68'
8 5/8" - 1018'
Cable

660' FSL - 660' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.32.34.241111

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	64		Sand and caliche
64	82		Red bed
82	792		Sand, red, and shale
			L S Elev 3963 DF
			Depth to K Trc 64
			Elev of K Trc 3898

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.32.34.241111 Elevation: 3952' Sea Level
Owner: Continental Oil Co.
Pearsall BX #2
Record of Casing: 8 5/8" - 59'
5 1/2" - 3515'
Rotary

1345' FNL - 1295' FEL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

SECTION ______________

TOWNSHIP 17S

RANGE 33E

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

STAT[illegible]# No. 1

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well WARTON DRILLING COMPANY

Street and Number Box 2807

City Odessa State Texas

Well was drilled under Permit No. ______ and is located in the ____ ¼ SE ¼ NW ¼ of Section 1 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Abbott Brothers License No. WD-46

Street and Number Box 637

City Hobbs State New Mexico

Drilling was commenced December 10 1957

Drilling was completed December ~~XX~~ 21 1957

Elevation at top of casing in feet above sea level ______ Total depth of well 180

State whether well is shallow or artesian shallow Depth to water upon completion 150

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	150	180	30	water sand
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	16	10	0	180	180	plain	150	180

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

FILED
DEC 30 1957
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

File No. L-3750 Use O.W.D. Location No. 17.33.1.140

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	1	1		soil
1	20	19		caliche
20	150	130		dry sand
150	180	30		water sand

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrell Abbott
Well Driller

Form WR-23 Orig. & 5.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Denver Drilling Company

Street and Number Box 669

City Odessa State Texas

Well was drilled under Permit No. L-3782 and is located in the S E ¼ S E ¼ S E ¼ of Section 2 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Cayton Drilling Co. License No. WD-183

Street and Number Box 1021

City Lovington State New Mexico

Drilling was commenced Feb. 6 1958

Drilling was completed Feb. 8 1958

Elevation at top of casing in feet above sea level ______ Total depth of well 183 ft.

State whether well is shallow or artesian Shallow Depth to water upon completion 151 ft.

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	151	170	19	Water Sand
2	176	183	7	Water Sand
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
6 5/8	17	10	0	183	184	None	140	183

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
18	183	10	400 lbs.		Dry Mix - Hole Gravel Packed

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received FEB 20 1958 OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3782 Use O.W.D. Location No. 17.33.2.444

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	1	1		Soil
1	4	3		Rock
4	12	8		Caliche
12	18	6		Boulder
18	26	8		Caliche
26	80	54		Sandy Clay
80	151	71		Dry Sand
151	170	21		Water Sand
170	176	6		Sandy Clay
176	183	7		Water Sand

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

CAYTON WATER WELL DRILLING COMPANY
Well Driller
Grady Backus

STATE ENGINEER OFFICE

Revised June 1972

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Yates Petroleum Owner's Well No. ______
Street or Post Office Address 105 South 4th. Street
City and State Artesia, New Mexico 88210

Well was drilled under Permit No. L-10,212 and is located in the:

a. ____ ¼ ____ ¼ SE ¼ SE ¼ of Section 2 Township 17-S. Range 33-E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Glenn's Water Well Service, Inc. License No. WD 421

Address P.O. Box 692 Tatum, New Mexico 88267

Drilling Began 7-7-94 Completed 7-7-94 Type tools rotary Size of hole 14 3/4 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 273 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well 168 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
168	268	100	sand	120

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
8 5/8	.250		1	273	273	none	153	273

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received 07/13/94

Quad ______ FWL ______ FSL ______

File No. L-10,212 Use secondary recovery of oil-water flood Location No. 17S.33.2.44423

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	1	1	soil
1	27	26	caleche
27	168	141	sand and rock with stringers of clay
168	268	100	sand (water)
268	273	5	red clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. Al[illegible]ons, except Section 5, shall be answered as completely and accurat[illegible]possible when any well is drilled, repaired or deepen[illegible]en this form is used as a plugging record, only Section 1(a) and Sectio[illegible]d be completed.

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE

DAYA OP... COMPANY
STATE ".." No. 2

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

	o		

(Plat of 640 acres)

(A) Owner of well Carper Drilling Company

Street and Number 300 Carper Building

City Artesia State New Mexico

Well was drilled under Permit No. ______ and is located in the

NW ¼ SE ¼ NW ¼ of Section 2 Twp 17 S Rge. 33 E

(B) Drilling Contractor Ed. D. Burke License No. WD-111

Street and Number P.O. Box 300

City Hobbs State New Mexico

Drilling was commenced July 11 19__

Drilling was completed July 12 19 62

Elevation at top of casing in feet above sea level ______ Total depth of well 204

State whether well is shallow or artesian shallow Depth to water upon completion 102

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	102	201	99	water sand
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	20	10	0	197	197	none	105	197

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received STATE ENGINEER OFFICE ROSWELL, N. MEX. DISTRICT II 1962 JUL 19 AM 8:29

File No. L-4935 Use OWD Location No. 17.33.2.120

OWD - ok

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		top soil
1	22	21		caliche
22	66	44		[illegible] sand, pack
66	71	5		hard sand
71	78	7		loose sand
79	103	24		pack sand
103	201	98		water sand
201	204	3		tight sand

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Edward B. Burke
Well Driller

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Pete Lomex Drilling Co.
Street and Number Box 424
City Hobbs State New Mexico
Well was drilled under Permit No. L-3012 and is located in the ¼ SE ¼ NW ¼ of Section 3 Twp. 17 S Rge. 33 E
(B) Drilling Contractor Cayton & Porter License No. WD-183
Street and Number Box 1021
City Lovington State New Mex.
Drilling was commenced Nov. 1 19 55
Drilling was completed Nov. 1 19 55

Elevation at top of casing in feet above sea level Total depth of well 210
State whether well is shallow or artesian Shallow Depth to water upon completion 155

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	To	Thickness in Feet	Description of Water-Bearing Formation
1	186	198	12	Water Sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	18	10	0	210	210	none	160	210

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received NOV 10 1955

OFFICE GROUND WATER SUPERVISOR

File No. L-3012 Oil Location No. 17.33.3. 140

Section 6 LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	2	2		Rock & Soil
2	14	12		Rock
14	20	6		Caliche
20	180	160		Sand & Rock Shells
180	186	6		Rock
186	198	12		Water Sand
198	210	12		Sandy Clay

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Jack Cayton
Well Driller

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

	250 o		

(Plat of 640 acres)

(A) Owner of well Continental Oil Company

Street and Number P. O. Box 460

City Hobbs State New Mexico

Well was drilled under Permit No. L-3528-S-3 and is located in the SE ¼ SE ¼ NW ¼ of Section 3 Twp. 17S Rge. 33E

(B) Drilling Contractor Walco Drilling, Inc. License No. WD-349

Street and Number P. O. Box 806

City Hereford State Texas

Drilling was commenced December 20 1968

Drilling was completed December 21 1968

Elevation at top of casing in feet above sea level 4,195 Total depth of well 271

State whether well is shallow or artesian Shallow Depth to water upon completion 155

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	150	212	62	Sandrock and red fine sand
2	212	237	25	Clean red sand
3	237	239	2	Red clay and sand
4	239	265	26	Sand and small gravel
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
12-3/4	49.56	---	0	270	270	---	181	227

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

STATE ENGINEER OFFICE ROSWELL, N. MEX. DISTRICT II 1969 JAN 14 AM 8:02

File No. L-3528-S-3 Use WATERFLOOD Location No. 17.33.3.1443

#2 Caprock 2-174-25

Section 6 LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	6			Top Soil
6	30			Rock, câliche and sand
30	50			Sand, sandrock and caliche
50	88			Sand and sandrock
88	90			Rock
90	150			Sand and sandrock
150	212			Sandrock and fine red sand
212	237			Clean red sand
237	239			Red clay and sand
239	265			Sand and small gravel
265	270			Red Bed
				L S Elev 4183
				Depth to K Trc 265
				Elev of K Trc 3918
				Loc. No. 17.33.3. 14443
				Hydro. Survey Field Check X
				SOURCE OF ALTITUDE GIVEN
				Interpolated from Topo. Sheet X
				Determined by Inst. Leveling
				Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

WALCO DRILLING, INC.
Well Driller

BY: R. Paul Coneway
R. Paul Coneway
President

Form WR-23 Orig. & 5 c.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

660	N of	S. line	
660	W of	E. line	
W ter	Lease	W 99	
			0

(Plat of 640 acres)

(A) Owner of well Maljamar Co-op Repressuring Agreement

Street and Number 200 Booker Building

City Artesia State New Mexico

Well was drilled under Permit No. L-3528 and is located in the ¼ SE ¼ SE ¼ of Section 4 Twp. 17 S Rge 33 E

(B) Drilling Contractor Abbott Bros. License No. WD-46

Street and Number Box 637

City Hobbs State New Mexico

Drilling was commenced December 11 1957

Drilling was completed December 16 1957

Elevation at top of casing in feet above sea level ______ Total depth of well 265

State whether well is shallow or artesian Shallow Depth to water upon completion 158

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	160	225	65	Water Sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
16			0	19	19			
10 3/4	34	Welded	0	265	265	plain	170	232
							6 rows 1/8"x12"	

12 cu. yds. gravel pack before pumping.

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

RECEIVED
DEC 30 1957
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

File No. L-3528 Use Oilfield water flood Location No. 17.33.4.44322

#1 Maj. 2-137-1

Section 6 LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	21	20		Caliche
21	150	129		Pack Sand
150	160	10		Hard Shell
160	225	65		Water Sand
225	240	15		Sandy Clay
240	265	25		Red Bed

L S Elev 4179
Depth to K Trc 240
Elev of K Trc 3939

F 17.33.4.44322

Loc. No.
Hydro. Survey Field Check X

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
Determined by Inst. Leveling
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Olga Abbott
Well Driller

L-3528

17.33.4.440

Form WR-23 FIEL R. LOG STATE ENGINEER OFFICE 3

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Yucca Water Co.
Street and Number Ft. Worth Nat'l. Bank Building
City Ft. Worth 2 State Texas
Well was drilled under Permit No. ______ and is located in the SE ¼ NE ¼ NE ¼ of Section 5 Twp. 17S Rge. 33E
(B) Drilling Contractor Abbott Bros. License No. ______
Street and Number Box 697
City Hobbs State New Mexico
Drilling was commenced June 18 19 59
Drilling was completed June 25 19 59

Elevation at top of casing in feet above sea level ______ Total depth of well 272
State whether well is shallow or artesian shallow Depth to water upon completion 160

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation
	From	To		
1	160	260	100	water sand
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth		Feet	Type Shoe	Perforations	
			Top	Bottom			From	To
10 3/4	24	weld	0	272	272	open	165	260

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet		Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
From	To				

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______
Street and Number ______ City ______ State ______
Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______
Plugging method used ______ Date Plugged ______ 19 ___
Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug		No. of Sacks Used
	From	To	

FOR USE OF STATE ENGINEER ONLY

Date Received

FILED
JUL 7 1959
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEX.

Secondary Recovery

File No. L-3598-X Use ~~D. W. P.~~ Location No. 17.33.5.222.20

#1 Map 2-125-2

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	14	13		Caliche
14	95	79		sand and gravel
95	160	65		tight sand (hard)
160	260	100		water sand
260	272	12		red clay

~~L S Elev~~ 4198

Depth to K ____ Trc 260

Elev of K ____ Trc 3938

[illegible] 17.33.5.22220

Loc. No ____

Hydro. Survey ____ Field Check X

~~SOURCE OF ALTITUDE GIVEN~~

~~Interpolated from Topo. Sheet~~ X

~~Determined by Inst. Leveling~~ ____

Other ____

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

____
Well Driller

L-3598-X

17.33.5.222

Form WR-23

STATE ENGINEER OFFICE

FIELD ENGR. LOG

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Yucca Water Company
Street and Number 300 Park Avenue
City New York 22 State N. Y.
Well was drilled under Permit No. L-3598 and is located in the NW ¼ NW ¼ NW ¼ of Section 6 Twp. 17S Rge. 33E
(B) Drilling Contractor D. E. Greenwood License No. WD-115
Street and Number 215 Birdway Avenue
City El Paso, State Texas
Drilling was commenced June 18, 1962 19
Drilling was completed June 25, 1962 19

Elevation at top of casing in feet above sea level 4223 ~~3800~~ Total depth of well 287 feet
State whether well is shallow or artesian shallow Depth to water upon completion 210 feet

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	~~155x~~	~~178x~~	~~23~~	~~Sandy clay and small gravel~~
2	~~230~~	~~255~~	~~25~~	~~Sand with streaks of clay~~
3	255	260	5	~~Small gravel~~ Brown sand and clay - gray gravel
4	265	270	5	Brown sand
5	270	280	10	Brown small gravel and sandy clay

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
12-3/4	30	welded	~~287x~~		287	welded	~~247~~ 242 242	~~287~~ 282

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

STATE ENGINEER OFFICE
DISTRICT II
ROSWELL, NEW MEXICO
1962 JUL 12 AM 8 23

File No. L-3598 Use SRO Location No. 17.33.6.111

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	20	20	White	Dolomite
20	60	40	Brown	Calechi and sand
60	220	160	Brown	Fine Sand
220	230	10	Brown	Clay
230	255	25	Brown	Sand with Streaks of clay
255	260	5	Gray	Gravel
260	265	5	Brown	Clay
265	270	5	Brown	Sand
270	280	10	Brown	Small gravel and sandy clay
280	285	5	Brown	Clay
285	287	2	Purple	Clay

L S Elev 4243
Depth to K Trc 280
Elev of K Trc 3963

Loc. No. 17.33.6.11110
Hydro. Survey Field Check X

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet
Determined by Inst. Leveling
Other Yucca Dept.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

B. E. Greenwood
Well Driller

B. E. Greenwood

L-3598

17.33.6.111

Form WR-23 FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well B. E. Paschall
Street and Number 605 S. 14 th St.
City Artesia State New Mexico
Well was drilled under Permit No. L-4524 and is located in the
¼ SE ¼ SE ¼ of Section 6 Twp. 17 S Rge. 31 E
(B) Drilling Contractor P & P Drilling Co. License No. WD-281
Street and Number 1121 S. Love
City Lovington State New Mexico
Drilling was commenced Sept 28 19 60
Drilling was completed Sept. 28 19 60

Elevation at top of casing in feet above sea level ______ Total depth of well 100 ft.
State whether well is shallow or artesian shallow Depth to water upon completion 90

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
				None				

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		7			

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______
Street and Number ______ City ______ State ______
Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______
Plugging method used ______ Date Plugged ______ 19 ______
Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received STATE ENGINEER OFFICE DISTRICT II ROSWELL, N. MEX. 1960 OCT 26 AM 8:3

File No. L-4524 Use Dom. Location No. 17.31.6.440

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
				This was a clean out job from
				75 ft. to 100 ft. on a domestic well, for
				Stock watering only.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Otis H. Pruett
Well Driller

Form WR-23 FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Dual Drilling Co. c/o S. O. Lamb

Street and Number

City Colorado City State Texas

Well was drilled under Permit No. L-4122 and is located in the near center ¼ NE ¼ SW ¼ of Section 7 Twp. 17 S Rge. 33E

(B) Drilling Contractor P & P Drilling Co. License No. WD-281

Street and Number 1121 South Love

City Lovington State New Mexico

Drilling was commenced May 1 19 59

Drilling was completed May 3 19 59

Elevation at top of casing in feet above sea level Total depth of well 249 ft.

State whether well is shallow or artesian Shallow Depth to water upon completion 214 ft

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	To	Thickness in Feet	Description of Water-Bearing Formation
1	214	249		
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
		None						

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		7		None	

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

FILED
JUN 24 1959
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

File No. L-4122 Use OWD Location No. 17.33.7.323322

Section 6

LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	1			Soil
1	4			Rock
4	25			Cliche
25	75			Sandy Clay
75	140			Dry Sand
140	194			Sandy Clay
194	214			Water Sand
214	230			Sandy Clay
230	244			Sand
244	247			Sand & Gravel
247	249			Red Bed
				L S Elev 4229
				Depth to K ___ Trc 247
				Elev of K ___ Trc 3982
				Loc. No. 17.33.7. 32322
				Hydro. Survey ___ Field Check X
				SOURCE OF ALTITUDE GIVEN
				Interpolated from Topo. Sheet X
				Determined by Inst. Leveling ___
				Other ___

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Otis H. Pruett
Well Driller

L-4122

17.33.7.320

Ori SF

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well KEWANEE OIL COMPANY

Street and Number BOX 124

City MALJAMAR State NEW MEXICO

Well was drilled under Permit No. STATE WATER WELL #5 MUNICIPAL USE and is located in the C ¼ SE ¼ ¼ of Section 7 Twp. 17S Rge. 33E

(B) Drilling Contractor C. O. ALDREDGE License No. 79

Street and Number BOX 379

City LOVINGTON State NEW MEXICO

Drilling was commenced JUNE 29 19 55

Drilling was completed JULY 13 19 55

Elevation at top of casing in feet above sea level ______ Total depth of well 227

State whether well is shallow or artesian SHALLOW Depth to water upon completion 182

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	164	188	24	LIGHT WATER SAND
2	188	215	27	GOOD WATER SAND AND GRAVEL
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
10	32	8	0	217	217	NONE	183	217

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		12½			8 SACKS OF AQUEGEL POURED IN TOP OF
					HOLE TO HOLD BACK QUICKSAND WHILE
					DRILLING WELL

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

RECEIVED

Date Received JUL 28 1955

OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-2771 Use Munic Location No. 17.33.7 40000

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	4	4	WHITE	TOP ROCK
4	12	8	RED	SAND
12	17	5	WHITE	HARD ROCK
17	51	34	RED	SAND
~~x88x~~ 51	64	13	GRAY	CALICHE
64	104	40	RED	SAND
104	117	13	GRAY	HARD CALICHE
117	134	17	GRAY	LIME AND STREAKS OF SAND
134	149	15	GRAY	BROKEN LIME
149	155	6	~~xxxx~~ RED	SAND
155	164	9	GRAY	BROKEN LIME
164	188	24	RED	SAND - LIGHT WATER SAND
188	189	1	GRAY	LIME SHELL
189	215	26	BROWN	SAND AND GRAVEL - GOOD WATER SAND
215	220	5	RED	SANDY SHALE
220	222	2	RED	PACK SAND
222	227	5	RED	SHALE

SET 10" PIPE AT 217 2 FEET INTO RED SANDY SHALE

TOTAL DEPTH 227

L S Elev 4217

Depth to K____ Trc 222

Elev of K____ Trc 3793

PB0 17.33.7.40000

Loc. No.____

Hydro. Survey____ Field Check X

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

C. C. Aldredge

Well Driller

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling____

Other____

L-2771 17.33.7.400

Form WR-23

FIELD [illegible] LOG

STATE ENGINEER OFFICE

WELL RECORD

Phil[illegible]s State # 1

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Thunderbird Drilling Co.

Street and Number 322 Fidelity Union Bldg.

City Dallas State Texas

Well was drilled under Permit No. ______ and is located in the

¼ SW ¼ SW ¼ of Section 9 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Abbott Bros. License No. WD-46

Street and Number Box 637

City Hobbs State New Mexico

Drilling was commenced Dec. 19 19 57

Drilling was completed Dec. 21 19 57

Elevation at top of casing in feet above sea level ______ Total depth of well 230

State whether well is shallow or artesian Shallow Depth to water upon completion 160

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation
	From	To		
1	160	230	70	Water Sand
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth		Feet	Type Shoe	Perforations	
			Top	Bottom			From	To

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet		Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
From	To				

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19 ______

Plugging approved by:

______ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug		No. of Sacks Used
	From	To	

FOR USE OF STATE ENGINEER ONLY

Date Received DEC 30 1957

OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3749 Use O.W.D. Location No. 17.33.9.330

.342113

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	19	18		Caliche
19	160	141		Dry Sand
160	230	70		Water Sand

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Well Driller

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE
CORRECTED COPY
WELL RECORD

Unit Well 24

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Continental Oil Company
Street and Number P.O. Box 460
City Hobbs State New Mexico
Well was drilled under Permit No. L-3528-S-2 and is located in the NW ¼ SW ¼ SW ¼ of Section 9 Twp. 17S Rge. 33E
(B) Drilling Contractor Abbot Brothers License No. WD-46
Street and Number P.O.Box 637
City Hobbs, State New Mexico
Drilling was commenced 7-8-67 19
Drilling was completed 7-19-67 19

Elevation at top of casing in feet above sea level Total depth of well 262'
State whether well is shallow or artesian Shallow Depth to water upon completion 180'

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	198	262	64'	sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
12 3/4	36	welded	-1	262	263	open	170	250
							4 rows 3/16	X 12

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

1967 OCT 18 AM 8

STATE ENGINEER OFFICE

File No. L-3528-S-2 Use SRO (Corrected) Location No. 17.33.9.331432

Section 6 LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	1	1		surface soil
1	26	25		caliche
26	78	52		sand, tight
78	96	18		sand, loose
96	129	33		sand, tight
129	232	103		sand
232	252	20		sandy clay
252	262	10	red	clay

L S Elev 4200
Depth to K Trc 252
Elev of K Trc 3948

SP 17.33.9.331432

Loc. No.
Hydro. Survey Field Check X

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo Sheet X
Determined by Inst. Leveling
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrell Abbott by BJ
Well Driller

L-3528-S-2

17.33.9.331

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

FIELD ENGR. LOG

Section 1. GENERAL INFORMATION

(A) Owner of well Ideal Basic Industries, Inc.%Potash Company of America Owner's Well No. # 8
Street or Post Office Address P.O. Box 31
City and State Carlsbad, New Mexico 88220

Well was drilled under Permit No. L-1880-S-3 and is located in the:

a. ____ ¼ NW ¼ SE ¼ NW ¼ of Section 12 Township 17S Range 33E N.M.P.M.

b. Tract No. ____ of Map No. ____ of the ____

c. Lot No. ____ of Block No. ____ of the ____
Subdivision, recorded in Lea County.

d. X= ____ feet, Y= ____ feet, N.M. Coordinate System ____ Zone in the ____ Grant.

(B) Drilling Contractor Abbott Bros. Drilling License No. WD-46

Address Hobbs, New Mexico 88240

Drilling Began 4/21/81 Completed 5/4/81 Type tools Cable Size of hole 24 in.

Elevation of land surface or ____ at well is ____ ft. Total depth of well 268 ft.

Completed well is [X] shallow [ ] artesian. Depth to water upon completion of well 155 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
159	230	71	Sand	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet Top	Depth in Feet Bottom	Length (feet)	Type of Shoe	Perforations From	Perforations To
14	36.71	Welded	0	269	269		155	268

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement

Section 5. PLUGGING RECORD

Plugging Contractor ____
Address ____
Plugging Method ____
Date Well Plugged ____
Plugging approved by:

____ State Engineer Representative

No.	Depth in Feet Top	Depth in Feet Bottom	Cubic Feet of Cement
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received May 14, 1981

Quad ____ FWL ____ FSL ____

File No. L-1880-S-3 Use ~~IND.~~ MOO Location No. 17.33.12.~~14142~~ .14112

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	1	1	Soil
1	26	25	Caliche
26	125	99	Sand
125	159	34	Sand and sand rock
159	230	71	Sand-water
230	241	11	Sand
241	258	17	Sand and clay streaks
258	268	10	Red clay

L S Elev 4148
Depth to K ___ Trc 241 ?
Elev of K ___ Trc 3907 ?

Loc. No. 17.33.12.14110
Hydro. Survey ___ Field Check PCA Survey

~~SOURCE OF ALTITUDE GIVEN~~
Interpolated from Topo. Sheet
Determined by Inst. Leveling ___
Other ___

Section 7. REMARKS AND ADDITIONAL INFORMATION

'81 MAY 14 AM 8 28
STATE ENGINEER OFFICE
ROSWELL, N.M.

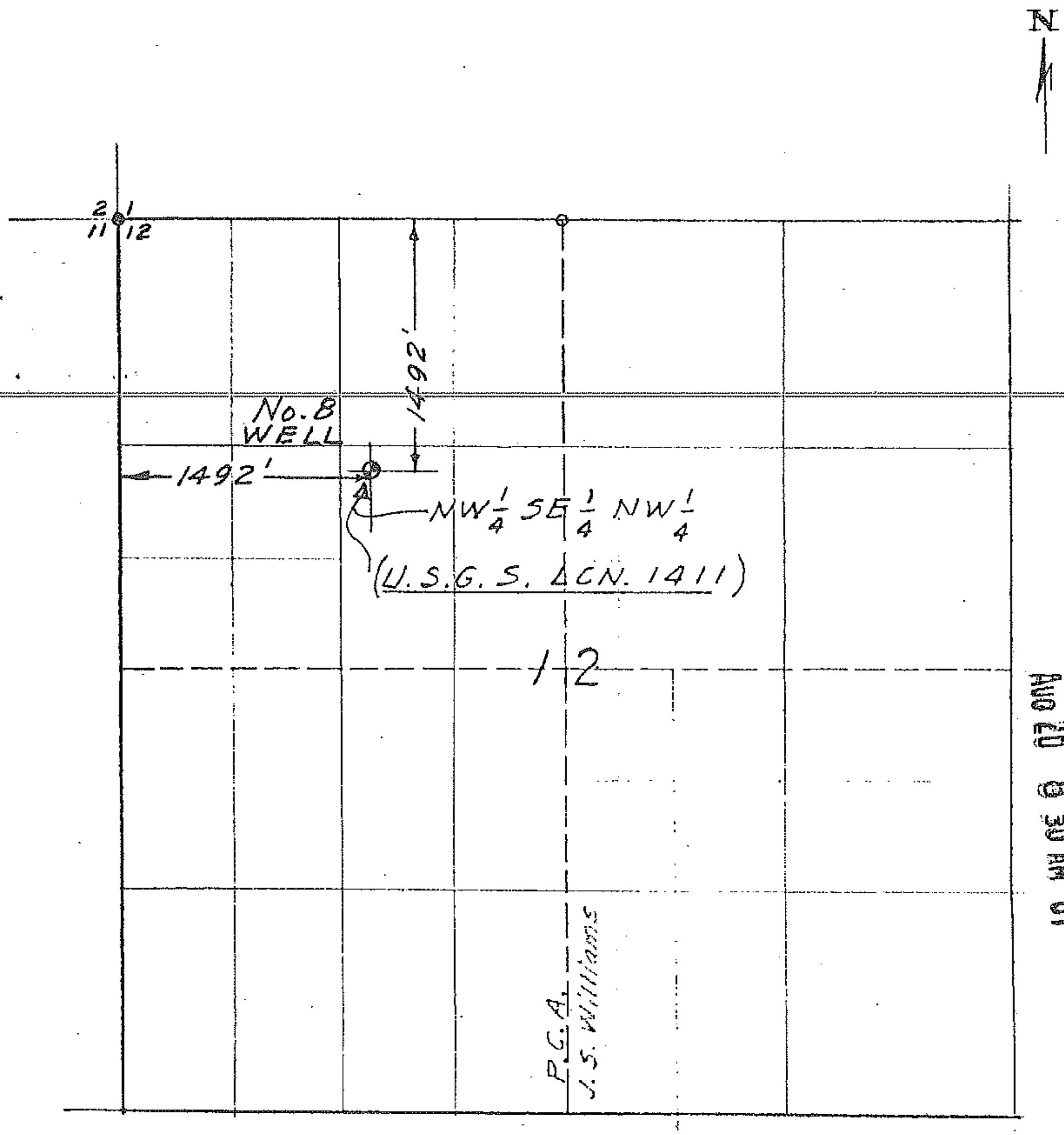
The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Murrell Abbott
Driller
H.B.

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

LOCATION - "CAPROCK" NO. 8 WATER WELL
POTASH CO. OF AMERICA - CARLSBAD, N.M.

SECT. 12, T. 17S., R. 33 E.
($W\frac{1}{2}$ of Sect. - P.C.A. deeded land.)



AUG 20 8 30 AM '01
STATE ENGINEER
ROSWELL, NM

1" = 1000'

L-1880-S-3 .14110

ECJ
8/17/81

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Potash Company of America

Street and Number P. O. Box 31

City Carlsbad State New Mexico

Well was drilled under Permit No. L-1880 thru L-1884 Comb. S and is located in the SE ¼ SW ¼ SW ¼ of Section 12 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Abbott Bros. License No. WD-46

Street and Number P. O. Box 637

City Hobbs State New Mexico

Drilling was commenced May 2 19

Drilling was completed May 5 1966

Elevation at top of casing in feet above sea level ______ Total depth of well 259

State whether well is shallow or artesian shallow Depth to water upon completion 115

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	115	230	115	Water sand
2	235	250	15	Sand and gravel
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
14	85	Weld	0	259	259	open	120	240
Drilled 24" hole								

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19____

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received ✓

ROSWELL, N. MEX.
DISTRICT II
STATE ENGINEER OFFICE
1966 SEP 29 AM 8:31

File No. L-1880 thru L-1884 Comb 5 Use Ind. & Dom Location No. 17.33.12.334 44

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	23	22		Caliche
23	70	47		Sand
70	115	45		Sand, dry
115	230	115		Sand, water
230	235	5		Sandy clay
235	250	15		Sand and gravel
250	259	9	red	Clay

L S Elev 4133

Depth to K Trc 250

Elev of K Trc 3883

Loc. No. 17.33.12.334441

Hydro. Survey ____ Field Check X

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet ✓

Determined by Inst. Leveling ____

Other ____

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrell Abbott Jr.

Well Driller

L-1880 thru 1884 Comb. S

17.33.12.334

Form WR-23 FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well
Street and Number Donnelly Drilling Company
City Box 432 State
Well was Artesia drilled under Permit No. and is New Mexico located in the
¼ ¼ ¼ of Section Twp. Rge.
SW SW 33 17S 33E
(B) Drilling Contractor License No.
Street and Number Ed Burke #211
City Box 306 State
Hobbs New Mexico
Drilling was commenced December 4 19 59
Drilling was completed December 4 19 59

Elevation at top of casing in feet above sea level Total depth of well 237
State whether well is shallow or artesian Shallow Depth to water upon completion 165

Section 2 — PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2	165	202	37	Water Sand
3				
4				
5				

Section 3 — RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	20	10	0	198	198	Open	177	198

Section 4 — RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 — PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received STATE ENGINEER OFFICE DISTRICT II 1959 DEC 10 AM 8:51

File No. L-4333 Use O.W.D. Location No. 17.33.13.110

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	14	14		Caliche
14	68	54		Sand (tight)
68	83	15		Sand (loose)
83	140	57		Sand (tight)
140	165	25		Sand (loose)
165	202	37		Sand (water)
202	217	15		Sand (tight)

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Edward B Burke
Well Driller

Form WR-23

STATE ENGINEER OFFICE

FIELD ENGR. LOG

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Potash Company of America

Street and Number Box 31

City Carlsbad, N.M. State

Well was drilled under Permit No. L 880-S-2 and is located in the NE 1/4 NW 1/4 SW 1/4 of Section 13 Twp. 17S Rge. 33E

(B) Drilling Contractor Abbott Bros. License No WD-46

Street and Number Box 637

City Hobbs, N.M. State

Drilling was commenced March 9, 1972 19

Drilling was completed March 16, 1972 19

Elevation at top of casing in feet above sea level ________ Total depth of well 235

State whether well is shallow or artesian shallow Depth to water upon completion 151

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
14	30	welded	1	238	238	none	118	228

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ________ License No. ________

Street and Number ________ City ________ State ________

Tons of Clay used ________ Tons of Roughage used ________ Type of roughage ________

Plugging method used ________ Date Plugged ________ 19

Plugging approved by:

________ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received ~~72 MAR 16~~

File No L-1880-S-2 Use COM Location No. 17.33.13.31413

Section 6

LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	4	4	brown	surface soil
4	28	24	gray	caliche
28	45	13	brown	snad tight
45	102	57	brown	sand loose
102	153	51	brown	sand tight
153	154	1	red	shale
154	198	44	brown	sand
198	201	3	red	shale
201	218	17	brown	sand
218	225	7	brown	sandy clay
225	230	5	gray	gravel
230	235	5	red	clay

L S Elev 4124

~~Depth to K~~ Trc 230

Elev of K Trc 3894

Loc. No. 17.33.13.31413

Hydro. Survey ________ Field Check HWP

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet 4124

Determined by Inst. Leveling ________

Other ________

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrel Abbott

Well Driller

Form WR-23

STATE ENGINEER OFFICE

Orig in S.S.

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Potash Company of America

Street and Number

City Carlsbad State New Mexico

Well was drilled under Permit No. L-1880 and is located in the SW ¼ SE ¼ SW ¼ of Section 13 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Cayben & Porter License No. WD-183

Street and Number Box 1021

City Lovington State New Mexico

Drilling was commenced August 18 1955

Drilling was completed August 18 1955

Elevation at top of casing in feet above sea level Total depth of well 245

State whether well is shallow or artesian Shallow Depth to water upon completion

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received SEP 30 1955 OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-1880 Use Ind & Dom. Location No. 17.33.13.343

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
This was work done on a repair Permit.				
Well was cleaned out from 232' to 245'.				

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Jack Cayton
Well Driller

Form WR-23 Orig in S.F.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Potash Company of America

Street and Number P. O. Box 31

City Carlsbad, State New Mexico

Well was drilled under Permit No. L-1882 and is located in the SE ¼ SW ¼ SE ¼ of Section 13 Twp. 17 S. Rge. 33 E.

(B) Drilling Contractor Randolph Johnston License No. WD-22

Street and Number West Grand Ave.

City Artesia State New Mexico

Drilling was commenced February 2, 1948

Drilling was completed March 16, 1948

Elevation at top of casing in feet above sea level 4128 Total depth of well 245

State whether well is shallow or artesian shallow Depth to water upon completion 144

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received SEP 12 1950

OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-1882 Use Ind'l Dom. Location No. 17.33.13.434 43444

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
				L S Elev 4128
				Depth to K Trc > 240
				Elev of K Trc < 3888
				Loc. No. 17.33.13.43444
				Hydro. Survey Field Check X
				SOURCE OF ALTITUDE GIVEN
				Interpolated from Topo. Sheet
				Determined by Inst. Leveling X
				Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Randolph Johnston
Well Driller

No. 3 Caproch Water Well

No L-1882 Lea County Basin

Drilled Feb 2, 1948
To Mar 16, 1948

By - Buck Thiering

Location SE¼, SW¼ SE¼
Sec 13 T17S R33E

Collar Elev. 4129.05 ft

Depth — Log

Ground Elev 4128±

Cemented

40

Top of slots
Area of perf: 1130 sq in — 97

Elev: 3988±
Top of Water 4/16/48 — 140±

Top of Water 8/17/55 — 162 / 165

188

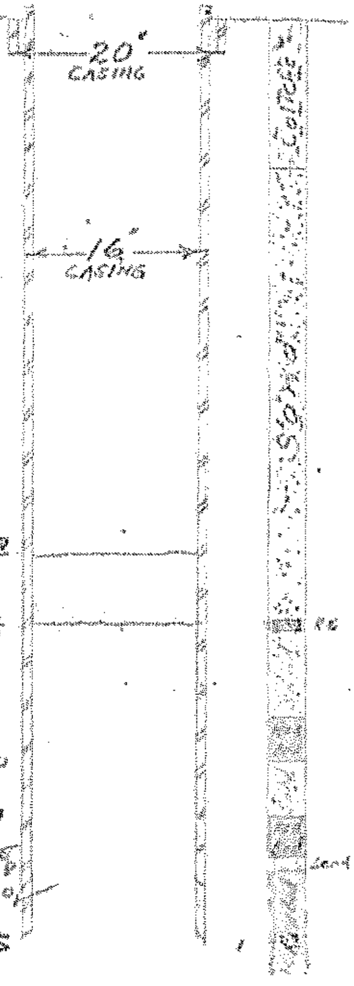
200

214

225 / 240

Total Depth — 240±

8/15/55: Approx 15 FT of Sand Bailed out → 255



APPROVED BY	Log of No 3 Caproch Water Well	POTASH COMPANY OF AMERICA CARLSBAD, NEW MEXICO	
		DRAWN BY Dep Eng	DRAWING NO.
		CHECKED BY	3-1374
	SCALE- 1" = 50' / DATE- 8-24-55	DIRECTED BY R.R.D.	

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Potash Co. of America
Street and Number Box 31
City Carlsbad, State New Mexico
Well was drilled under Permit No. L-1882 and is located in the SE ¼ SE ¼ SE ¼ of Section 13 Twp. 17 S Rge. 33 E
(B) Drilling Contractor P & P Drilling Co. License No. WD-281
Street and Number 1121 S. Love
City Lovington State New Mexico
Drilling was commenced Sept. 22 19 64
Drilling was completed Sept. 24 19 64

Elevation at top of casing in feet above sea level ____ Total depth of well 245
State whether well is shallow or artesian Shallow Depth to water upon completion ____

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
14			226	245	14			

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor ____ License No. ____
Street and Number ____ City ____ State ____
Tons of Clay used ____ Tons of Roughage used ____ Type of roughage ____
Plugging method used ____ Date Plugged ____ 19__
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

1964 OCT 30 AM 8:21

File No. L-1882 Use Ind & Dom (Repair) Location No. 17.33.13.434

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
				This was a repair Job-on Potash Mine well
				Cleaned & Drilled Fr 220 ft to 240 ft.
				Run Pipe Scratcher- Set 14 ft. of 14 in
				casing -in Bottom of Hole & Bailed.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Otis H. Pruett
Well Driller

Form WR-23 Orig S.F.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Potash Company of America

Street and Number

City Carlsbad State New Mexico

Well was drilled under Permit No. L-1883 and is located in the SE ¼ SE ¼ SE ¼ of Section 13 Twp. 17 S. Rge. 33 E.

(B) Drilling Contractor Emmett Barron License No.

Street and Number

City Carlsbad State New Mexico

Drilling was commenced June 11 19 52

Drilling was completed July 24 19 52

Elevation at top of casing in feet above sea level Total depth of well 259

State whether well is shallow or artesian Shallow Depth to water upon completion 147

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	120	135	15	Br. hard chunky sand
2	219	239	20	Br. muddy sands very little gravel
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
16			0	150	150			
13 5/8			12'3"	259				

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received November 1, 1955

File No. L-1883 Use Ind. & Dom. Location No. 17.33.13.444 44

Section 6 LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	20	20		Lime & Caliche
20	50	30		hard fine sand
50	60	10		fine red sand
60	65	5		br. hard sand
65	80	15		fine red sand
80	95	15		br. hard chunky sand
95	120	40		fine sand
120	135	15		br. hard chunks sand
135	145	10		fine sand
145	147	2		hard sand
147	150	3		red bed
150	170	20		fine sand
170	173	3		red bed
173	210			fine & cores sand some gravel
210	219	9		red bed
219	239	20		br. muddy sands
239	241	2		course gravel
241	259			red bed-some gravel

L S Elev 4123
~~Depth to K~~ Trc 241
Elev of K Trc 3882

17.33.13.44444

Loc. No.

Hydro. Survey Field Check

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Levelling

Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

/s/ Emmett Barron
Well Driller

L-1883

17.33.13.444

Form WR-23 [illegible] s. f.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(A) Owner of well Potash Company of America

Street and Number

City Carlsbad State New Mexico

Well was drilled under Permit No. L-1883 and is located in the S E ¼ S E ¼ S E ¼ of Section 13 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Cayton & Porter Drlg. Co. License No. WD-183

Street and Number Box 1021

City Lovington State New Mexico

Drilling was commenced Sept. 20 19 55

Drilling was completed Sept. 26 19 55

(Plat of 640 acres)

Elevation at top of casing in feet above sea level Total depth of well

State whether well is shallow or artesian Depth to water upon completion

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

FILED
JUL 16 1958
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

File No. L-1883 Use Ind. & Dom. Location No. 12.33.13.444

Section 6

LOG OF WELL

Depth in Feet		Thickness In Feet	Color	Type of Material Encountered
From	To			
				Pull 220 ft. 4" pump. Clean out and bail
				hole and reset pump.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

CATTON & PORTER DRILLING COMPANY
Well Driller
M. R. Porter

Form WR-23 FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well POTASH COMPANY OF AMERICA

Street and Number Box 31

City Carlsbad State New Mexico

Well was drilled under Permit No. L-1883 and is located in the ¼ SE ¼ SE ¼ of Section 13 Twp. 17 S Rge. 33 E

(B) Drilling Contractor P & F Drilling Co. License No. WD-281

Street and Number 1121 S. Love

City Lovington State New Mexico

Drilling was commenced Aug 21 1960

Drilling was completed Aug 21 1960

Elevation at top of casing in feet above sea level ______ Total depth of well 100 ft.

State whether well is shallow or artesian Shallow Depth to water upon completion ______

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
				None				

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		7	None		

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received 1960 AUG 24 AM 8:1 STATE ENGINEER OFFICE ROSWELL, N. MEX.

File No. L-1883 Use Ind & Dom. Location No. 17.33.13.440

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
				This was a clean out job, on a Domestic
				well, for a Potash Mine.
				Fished out suction pipe, and cleaned
				well from 70 ft to 100 ft.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Otis W. Pruett
Well Driller

Form WR-23 Ori BSD STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Midland Drilling Company

Street and Number 410 W. Ohio St.

City Midland State Texas

Well was drilled under Permit No. L-3622 and is located in the Center¼ NW¼ N ½ NW¼ of Section 17 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Cayton Drlg. Co. License No. WD-183

Street and Number Box 1021

City Lovington State New Mexico

Drilling was commenced July 22 1957

Drilling was completed July 25 1957

Elevation at top of casing in feet above sea level ______ Total depth of well 226 ft.

State whether well is shallow or artesian Shallow Depth to water upon completion 180 ft.

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	180	200	20	Water Sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	17	10	0	226	226	None	180	226

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
18	226	10	500 lbs.		Dry Mix. Hole Gravel packed

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

FILED

Date Received AUG 1 1957

OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3622 Use Owd Location No. 17.33.17.12444

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	2	2		Soil
2	12	10		Caliche
12	18	6		Boulder
18	180	162		Sand, Shell, & Clay
180	200	20		Water Sand
200	224	24		Sand, Shell, & Gravel
224	226	2		Red Bed

L S Elev 4207
Depth to K Trc 224
Elev of K Trc 3983

Loc. No. 17.33.17.12444
Hydro. Survey ____ Field Check X (NOT FOUND)

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
Determined by Inst. Leveling ____
Other ____

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

CAYTON Drilling Company
Well Driller
Jack Cayton

L-3622

17.33.17

Form WR-23 CA Orig & 5 cf

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well KEWANEE OIL COMPANY

Street and Number MALJAMAR, NEW MEXICO

City ______ State ______

Well was drilled under Permit No. STATE WATER WELL #4 MUNICIPAL USE and is located in the C ¼ NE ¼ ______ ¼ of Section 18 Twp. 17S Rge. 33E

(B) Drilling Contractor C. O. ALDRIDGE License No. 79

Street and Number BOX 379

City LOVINGTON State NEW MEXICO

Drilling was commenced JUNE 6 1955

Drilling was completed JUNE 26 1955

Elevation at top of casing in feet above sea level ______ Total depth of well 214.6

State whether well is shallow or artesian SHALLOW Depth to water upon completion 179

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	169	185	16	LIGHT WATER SAND
2	185	213	28	GOOD WATER SAND
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
10	32	8	0	214.6	214.6	NONE	182	214.6

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		12¼			8 SACKS OF AQUAGEL POURED IN TOP
					OF HOLE TO HOLD BACK QUICKSAND
					WHILE DRILLING WELL

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received FILED JUL 28 1955 OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-2770 Use Munic. Location No. 17.33.18.24111 200

#4, on OCD-2034

Section 6

LOG OF WELL

Depth In Feet From	Depth In Feet To	Thickness in Feet	Color	Type of Material Encountered
0	3	3	Brown	Soil
3	68	65	Red	Sand
68	71	3	Gray	Lime
71	98	17	White	Caliche
98	117	19	Red	Sand
117	129	12	White	Caliche
129	163	34	Red	Sand
163	165	2	Brown	Shale
165	189	24	Red	Sand and Gravel Light Water Sand
189	192	3	Light Gray	Lime Shell
192	198	6	Red	Sand
198	213	15	Brown	Water Sand - Good
213	214	1	Red	Shale

RUN 10" PIPE TO 213-6 CLEANED OUT DROVE PIPE FROM 213-6 TO 214-6 - ONE FOOT IN RED BED

TOTAL DEPTH 214.6

L S Elev 4215

Depth to K Trc 213

Elev of K Trc 4002

PBV 17.33.18.24111

Loc. No.

Hydro. Survey X Field Check

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling

Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

C. O. Aldredge

Well Driller

L-2770

17.33.18.200

Form WR-23

STATE ENGINEER OFFICE

L-2773

L-2773

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

#3 on OC-2-203-1

(Plat of 640 acres)

(A) Owner of well KEWANEE OIL COMPANY

Street and Number

City MALJAMAR State NEW MEXICO

Well was drilled under Permit No. CLEAN OUT OIL CAMP WELL # 2 and is located in the EXTREME CORNER N.E. CORNER SW¼ ¼ ¼ ¼ of Section 18 Twp. 17 Rge. 33

(B) Drilling Contractor C. O. ALDREDGE License No. 79

Street and Number BOX 379

City LOVINGTON State NEW MEXICO

Drilling was commenced JUNE 1 1955

Drilling was completed JUNE 6 1955

Elevation at top of casing in feet above sea level Total depth of well 214

State whether well is shallow or artesian SHALLOW Depth to water upon completion 184

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	196	214	18	QUICK SAND
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
10			± 0	214	214	RED BED		
	WELL ALREADY CASED WHEN CLEANED OUT							

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
		NO MUD USED			

Section 5

PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received JUL 11 1955

OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-2773 Use Misc Location No. 17.33.18.322

#3 on OC-2-203-1

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
196	214	18	RED	QUICK SAND

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

G. O. [illegible]
Well Driller

Form WR-23 STATE ENGINEER OFFICE I 2

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well KEWANEE OIL CO.
Street and Number BOX 124
City MALJMAR State N. MEX.
Well was drilled under Permit No. STATE WATER WELL DRILLED ON 4-1947 MUNICIPAL USE and is located in the
2090' FROM SOUTH LINE 2090' FROM WEST LINE 18 17S 33E
¼ ¼ ½ of Section 18 Twp. 17S Rge. 33E
(B) Drilling Contractor C. O. ALDRIDGE License No.
Street and Number BOX 379
City LOVINGTON State NEW MEXICO
Drilling was commenced JULY 14 1955
Drilling was completed JULY 16 1955

Elevation at top of casing in feet above sea level 4230 Total depth of well 220
State whether well is shallow or artesian SHALLOW Depth to water upon completion 202

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	To	Thickness in Feet	Description of Water-Bearing Formation
1	202	215	13	QUICK SAND
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	To
10 3/4	40.5#	8	0	215.2	215.2			
		WELL WAS DRILLED			4-19-47	WAS ALREADY CASED	WHEN CLEANED OUT	

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
			NO MUD	USED	

FILED
JUL 28 1955
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by: Cement Plugs were placed as follows:

Basin Supervisor

No.	Depth of Plug From	To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

FILED
JUL 28 1955
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

File No. L-2773 Use Munic (Repair) Location No. 17.33.18.322 322331

Section 6 LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
202	215	13	Red	Quick Sand
215	220	5	Red	Red Bed

~~L S Elev~~ 4225
Depth to K Trc 215
Elev of K Trc 4010

FV 17.33.18.32233

Loc. No.
Hydro. Survey Field Check X

SOURCE OF ALTITUDE Given
Interpolated from Topo. Sheet X
Determined by Inst. Leveling
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

C. O. Aldredge
Well Driller

L-2773

17.33.18.322

Form WR-23 Orig S.S.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

Well located 200' east of House, Barn [illegible] Now used for domestic.

(Plat of 640 acres)

(A) Owner of well Henry Black Drilling Company

Street and Number Box 174

City Midland State Texas

Well was drilled under Permit No. L-3726 and is located in the ___ ¼ ___ ¼ N E ¼ of Section 18 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Cayton Drlg. Co. License No. WD-183

Street and Number Box 1021

City Lovington State New Mexico

Drilling was commenced November 25 19 57

Drilling was completed November 30 19 57

Elevation at top of casing in feet above sea level 4216 Total depth of well 208 ft.

State whether well is shallow or artesian Shallow Depth to water upon completion 188 ft.

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	188	194	6	Water Sand
2	203	207	5	Water Sand & Gravel
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	20	10	0	208	208	None	148	208

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
20	208	10	400 lbs.		Dry Mix; hole gravel packed

Section 5 PLUGGING RECORD

Name of Plugging Contractor ___ License No. ___

Street and Number ___ City ___ State ___

Tons of Clay used ___ Tons of Roughage used ___ Type of roughage ___

Plugging method used ___ Date Plugged ___ 19___

Plugging approved by:

___

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received FEB 10 1958 OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3726 Use S.D. Location No. 17.33.18.230

221|31

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	8	7		Caliche
8	12	4		Boulder
12	18	6		Caliche
18	26	8		Boulder
26	188	162		Sandy Clay Shell
188	194	6		Water Sand
194	203	9		Sandy Clay
203	207	5		Water Sand & Gravel
207	208	1		Red Bed

L S Elev 4216'
Depth to K Trc 207'
Elev of K Trc 4009'

Loc. No. 17.33.18.22113
Hydro. Survey Field Check X

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
Determined by Inst. Leveling
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

CAYTON DRILLING COMPANY
Well Driller
Jack Cayton

L-3726

17.33.18.230

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Warren and Bradshaw Expl. & Drg. Co.
Street and Number 2504 West Brunson
City Midland State Texas
Well was drilled under Permit No. L-2875 and is located in the SE ¼ NE ¼ ¼ of Section 20 Twp. 17 S Rge. 33 E
(B) Drilling Contractor Abbott Brothers License No. WD-46
Street and Number P.O. Box 637
City Hobbs, State New Mexico
Drilling was commenced May 18 1955
Drilling was completed May 20 1955

Elevation at top of casing in feet above sea level ______ Total depth of well 250
State whether well is shallow or artesian shallow Depth to water upon completion 190

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	190	235	45	Water sand (Ogallala) (Low yield)
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	17	8	0	250	250	no	190	250

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______
Street and Number ______ City ______ State ______
Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______
Plugging method used ______ Date Plugged ______ 19___
Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FILED
MAY 26 1955
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

FOR USE OF STATE ENGINEER ONLY

Date Received May 26, 1955

File No. L-2875 Use Oil Location No. 17.33.20.220

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	9	8		Caliche
9	60	51		Tight Sand
60	120	60		Sand
120	190	70		Tight Sand
190	235	45		Water Sand
235	250	15		Sandy Clay

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Merrell Abbott

Well Driller

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Phillips Petroleum Corp.

Street and Number Box 1352

City Odessa State Texas

Well was drilled under Permit No. Applied L-3133 and is located in the

Center ¼ NW ¼ SW ¼ of Section 23 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Cayton & Porter Drlg. Co. License No. WD-153

Street and Number Box 1021

City Lovington, State New Mexico

Drilling was commenced February 29 19 56

Drilling was completed March 4 19 56

Elevation at top of casing in feet above sea level Total depth of well 230

State whether well is shallow or artesian Shallow Depth to water upon completion 160

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	158	198	40	Water Sand-Gravel
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	32	8	0	230	230	None	160	230

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

FILED

Date Received MAR 14 1956

OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3133 Use Oil Location No. 17.33.23.310

313207

Section 6

LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
~~0~~	~~8~~	~~8~~		~~Rock~~
~~8~~	~~12~~	~~4~~		~~Caloche~~
~~12~~	~~20~~	~~8~~		~~Rock~~
~~20~~	~~112~~	~~92~~		~~Sand~~
~~112~~	~~158~~	~~46~~		~~Sandy Clay~~
~~158~~	~~198~~	~~40~~		~~Water Sand & Gravel~~
~~198~~	~~220~~	~~22~~		~~Sand~~
~~220~~	~~230~~	~~10~~		~~Red Clay~~
				L S Elev 4143
				Depth to K Trc 220
				Elev of K Trc 3923
				FL 17.33.23.31320
				~~Loc. No.~~
				~~Hydro. Survey~~ Field Check X
				SOURCE OF ALTITUDE GIVEN
				Interpolated from Topo. Sheet X
				Determined by Inst. Leveling
				Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Well Driller

L-3133

17.33.23.310

Form WR-23 Orig. S.F.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Phillip Petroleum Co.
Street and Number Box 758
City Hobbs State New Mexico
Well was drilled under Permit No. L3133 and is located in the ¼ NW ¼ SW ¼ of Section 23 Twp. 17 S Rge. 33 E
(B) Drilling Contractor P & P Drilling Co. License No. WD 281
Street and Number 1121 South Love
City Lovington State New Mexico
Drilling was commenced Sept 2 1958
Drilling was completed Sept 3 1958

Elevation at top of casing in feet above sea level ______ Total depth of well 230 ft
State whether well is shallow or artesian Shallow Depth to water upon completion 70 ft

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7 In.	hole				no casing			

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______
Street and Number ______ City ______ State ______
Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______
Plugging method used ______ Date Plugged ______ 19__
Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received SEP 26 1958 OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3133 Use D.&.D. Location No. 17.33.23.310

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
				This is an old well drilled March 1956 and later plugged, well was 230 ft. of 7" casing. We drilled out plug, clean out and bailed out hole; to be used for oil well Drilling purposes

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Grady Backus
Well Driller

Form WR-23 FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres) 23 o

(A) Owner of well Phillips Petroleum Company
Street and Number Box 2105
City Hobbs State New Mexico
Well was drilled under Permit No. L-3133 and is located in the ¼ NW ¼ SW ¼ of Section 23 Twp. 17S Rge. 33E
(B) Drilling Contractor Clayton Water Well License No. NR
Street and Number P. O. Box 1021
City Lovington State New Mexico
Drilling was commenced 19
Drilling was completed Well reopened 11-21-59 19

Elevation at top of casing in feet above sea level 4144 Total depth of well 230'
State whether well is shallow or artesian Shallow Depth to water upon completion *

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				* See original well record.
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7"	20 & 23	8	0	230	230	-	*	

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
					None

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received 1959 NOV 27 AM 8:20 STATE ENGINEER OFFICE DISTRICT II ROSWELL, N. MEX.

File No. L-3133 Use O.W.D. Location No. 17.33.23.310

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
				See original well record.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Well Driller

PHILLIPS PETROLEUM COMPANY

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Land Commissioners
Prospectors No. M2902

Section 1

#5-	C-2-	153-5	

(Plat of 640 acres)

(A) Owner of well Southwest Potash Co.

Street and Number Box 472

City Carlsbad State N. M.

Well was drilled under Permit No. ________ and is located in the SE ¼ SE ¼ NE ¼ of Section 25 Twp. 17 S Rge. 33E

(B) Drilling Contractor T. M. Theriac License No. ________

Street and Number P.O.Box 1434

City Hobbs State N.

Drilling was commenced April 8 1950

Drilling was completed April 21 1950

Elevation at top of casing in feet above sea level ________ Total depth of well 230

State whether well is shallow or artesian ________ Depth to water upon completion 137 (reported)

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	137	187	50	Tertiary Sands and gravels
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
13 3/8		New seamless			194'8"	Bethleham Texas Pattern	94'2"	193'4"

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ________ License No. ________

Street and Number ________ City ________ State ________

Tons of Clay used ________ Tons of Roughage used ________ Type of roughage ________

Plugging method used ________ Date Plugged ________ 19__

Plugging approved by:

____
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received December 29, 1952

File No. L1695 Use ________ Location No. 17.33.25.244 44r

Section 6

LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	18			Hard crust top soil, caliche various hardness
18	28			Harder caliche fragments
28	38			Larger caliche fragments
38	50			Caliche and fine sil, approx. 20% brown sand
50	60			Fine dry sand, clear red brown particles
60	105			Red, brown and clean sand, few particles
				hard limestone
105	110			Fine sil and brown sand-quicksand
110	115			90% small clear & brown sand, trace of lime
115	130			Sil of various size, small brown & clear sand
130	135			Sil and brown and red sand
135	137			Hit water at 137' brown and clear quicksand
137	160			Larger particles sil-sand more porous
160	174			Few large particles brown and clear sill &
				quarts. Small flakes of red compaction shale
174	180			Clear, brown, red and orange sand
180	185			Sand same - few ½" to 1" and gravel, small
				flakes of red clay
185	190			Red and brownish clay in much larger quantity
190	200			Solid red bed, sand disappearing fast
200	225			Red bed solid, no sand encountered.
				L S Elev 4093
				Depth to K Trc 190
				Elev of K Trc 3903
				Loc. No. 17.33.25.24444
				Hydro. Survey Field Check X

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

T. M. Theriac
Well Driller

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
Determined by Inst. Leveling
Other

17.33.25.24K

Form WR-23 Orig. S.F. STATE ENGINEER OFFICE

Philli[illegible] State Lease
Wa[illegible] well

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well ZAPATA PETROLEUM CORP.
Street and Number Box 2216
City Midland State Texas
Well was drilled under Permit No. ____ and is located in the SW ¼ SE ¼ NW ¼ of Section 28 Twp. 17 S Rge. 33 E
(B) Drilling Contractor Abbott Brothers License No. WD-46
Street and Number Box 637
City Hobbs State New Mexico
Drilling was commenced October 21 1957
Drilling was completed October 23 1957

Elevation at top of casing in feet above sea level ____ Total depth of well 210
State whether well is shallow or artesian shallow Depth to water upon completion none

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	None			
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ____ License No. ____
Street and Number ____ City ____ State ____
Tons of Clay used ____ Tons of Roughage used ____ Type of roughage ____
Plugging method used ____ Date Plugged ____ 19__
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received DEC 30 1957 OFFICE GROUND WATER SUPERVISOR ROSWELL, NEW MEXICO

File No. L-3713 Use O.W.D. Location No. 17.33.28.143

Section 6 LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		soil
1	16	15		caliche
16	210	194		dry sand

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrell Abbott
Well Driller

Form WR-23

East Maljamar Plant W.W. #1

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well El Paso Natural Gas Company

Street and Number P. O. Box 1492

City El Paso State Texas

Well was drilled under Permit No. Misc. 2-L-58 and is located in the NE ¼ NE ¼ NE ¼ of Section 29 Twp. 17S Rge. 33E

(B) Drilling Contractor Abbott Bros. License No.

Street and Number P. O. Box 637

City Hobbs State New Mexico

Drilling was commenced 19

Drilling was completed July 22, 1958 19 58

Elevation at top of casing in feet above sea level Total depth of well 244'

State whether well is shallow or artesian Shallow Depth to water upon completion 204'

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	185	228	43	Water Sand
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
6 5/8			0	244	244		168	244

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received STATE ENGINEER OFFICE DISTRICT II ROSWELL, N. MEX. 1961 MAR 31 PM 2:20

File No. Misc. 2-L-58 Use Ind & Dom. Location No. 17.33.29.222

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1		Soil
1	18	17		Caliche
18	80	62		Sand
80	85	5		Sand rock
85	125	40		Sand
125	185	60		Tight sand and Rock
185	228	43		Water, sand
228	244	16		Sand and Red Clay

L S Elev 4188 re.
Depth to K Trc 244 re.
Elev of K Trc 3944 re.

Loc. No. 17.33.29.222.22
Hydro. Survey Field Check

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
Determined by Inst. Leveling
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Well Driller

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.33.30.11000

Section 6. LOG OF HOLE

Depth in Feet From	To	Thickness in Feet	Color and Type of Material Encountered
0	28		Caliche and gravel
28	223		Shale and shells
223	515		Red rock
515	533		Anhydrite

L S Elev 4039
Depth to K ____ Trc 28
Elev of K ____ Trc 4011

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.33.30.11000 Elevation: 4039' DF
Owner: Continental Oil Co.
MCA Unit Battery 4 #133
Record of Casing: 10" - 21'
7" - 3913'
Rotary

660' FNL - 660' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All questions, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use 011 Location No. 17.33.30.12000

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	45		Caliche and sand
45	375		Red bed
375	1145		Red bed, red rock
			L S Elev 4057
			Depth to K Trc 45
			Elev of K Trc 4012

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.33.30.12000 Elevation: 4057' DF

Owner: Continental Oil Co.

MCA Unit Battery 4 #134

Record of Casing: 8" - 1185'

Rotary

660' FNL - 1980' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ______ ¼ ______ ¼ ______ ¼ ______ ¼ of Section ______ Township ______ Range ______ N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use Oil Location No. 17.33.30.14000

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	30		Caliche
30	85		Caliche and sand
85	810		Red bed and red rock
			L S Elev 4072 DF
			Depth to K Trc 85
			~~Elev of K Trc~~ 3987

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.33.30.14000 Elevation: 4062' GL
Owner: Continental Oil Co.
MCA Unit Battery 4 #135
Record of Casing: 10" - 20'

Rotary

1980' FNL - 1980' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE
WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ______ Owner's Well No. ______
Street or Post Office Address ______
City and State ______

Well was drilled under Permit No. ______ and is located in the:

a. ____ ¼ ____ ¼ ____ ¼ ____ ¼ of Section ____ Township ____ Range ____ N.M.P.M.

b. Tract No. ____ of Map No. ____ of the ______

c. Lot No. ____ of Block No. ____ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor ______ License No. ______

Address ______

Drilling Began ______ Completed ______ Type tools ______ Size of hole ____ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well ______ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ______ FWL ______ FSL ______

File No. ______ Use Oil Location No. 17.33.30.31111

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	66		Sand
66	73		Rock
73	96		Sand
96	160		Red bed
160	270		Red sand and red bed
270	437		Red bed
437	546		Red bed and shells
546	608		Red bed and blue shale
608	628		Red bed
628	650		Sand
650	791		Red bed, sand, shells, shale
791	806		Lime shells
806	1078		Shale, red bed
			L S Elev 4037
			Depth to K Trc 96
			Elev of K Trc 3941

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.33.30.31111 Elevation: 4037' DF
Owner: Continental Oil Co.
MCA Unit #197
Record of Casing: 8 5/8" - 128'
7" - 3963'

Rotary

2615' FSL - 25' FWL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well ____________ Owner's Well No. ____________
Street or Post Office Address ____________
City and State ____________

Well was drilled under Permit No. ____________ and is located in the:

a. ____ ¼ ____ ¼ ____ ¼ ____ ¼ of Section ____ Township ____ Range ____ N.M.P.M.

b. Tract No. ____ of Map No. ____ of the ____________

c. Lot No. ____ of Block No. ____ of the ____________
Subdivision, recorded in ____________ County.

d. X= ____ feet, Y= ____ feet, N.M. Coordinate System ____________ Zone in the ____________ Grant.

(B) Drilling Contractor ____________ License No. ____________

Address ____________

Drilling Began ____________ Completed ____________ Type tools ____________ Size of hole ____ in.

Elevation of land surface or ____________ at well is ____ ft. Total depth of well ____ ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ____ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ____________
Address ____________
Plugging Method ____________
Date Well Plugged ____________
Plugging approved by:

____
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received Typed 5/11/78

Quad ____________ FWL ____________ FSL ____________

File No. ____________ Use 011 Location No. 17.33.30.42000

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	98		Caliche and sand
98	145		Sand and gravel
145	1171		Red rock and red bed
			L S Elev 4060
			Depth to K Trc 145
			Elev of K Trc 3915

Section 7. REMARKS AND ADDITIONAL INFORMATION

This well record is an excerpt from Oil Conservation Commission files at Hobbs, N.M.

Location: 17.33.30.42000 Elevation: 4060' DF
Owner: Cities Service Co.
S. M. G. S. A. Unit Tract 1 #2
Record of Casing: 8 5/8" - 1199'

Rotary

1980' FSL - 660' FEL

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Dillard & Waltermier Drilling CO.

Street and Number PO. Box I206

City Odessa. State Texas.

Well was drilled under Permit No. L - 4363 and is located in the N W ¼ N E ¼ S W ¼ of Section 35 Twp. I7 S Rge. 33 E

(B) Drilling Contractor C. O. Aldredge License No. W D 79

Street and Number PO. Box 379

City Lovington State New Mexico.

Drilling was commenced Dec. 29 1959

Drilling was completed Jan. 5 19 60

Elevation at top of casing in feet above sea level ______ Total depth of well 226

State whether well is shallow or artesian Shallow Depth to water upon completion I60 Ft

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	I70	I80	I0	Brown water sand
2	I83	200	I7	Brown water sand & gravel
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
65/8	Welded		0	222	222	None	I76	222

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used
					6sacks of aquagell pored in hole while well was beeing drilled

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19____

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received 1960 JAN 19 AM 8:25 STATE ENGINEER OFFICE

File No. L-4363 Use O.W.D. Location No. 17.33.35.321 (3314?1)

Section 6 LOG OF WELL

Depth in Feet From	To	Thickness in Feet	Color	Type of Material Encountered
0	2	2	Brown	soil
2	30	28	White	Calchie rock
30	70	40	Brown	sand
70	I40	70	Red	sand
I40	I50	I0	White	Caliche
I50	I52	2	Red	~~Sand~~ Shale
I52	I70	I8	Red	Sand
I70	I80	I0	Brown	water sand
I80	I83	3	Red	shale
I83	200	I7	Brown	water sand &gravel
200	222	22	Red	Shale & sand rock
222	226	4	Red	Red bed

~~L S Elev~~ 4122
Depth to K Trc 222
Elev of K Trc 3900

17.33.35.32142

Loc. No.
Hydro. Survey Field Check X

~~SOURCE OF ALTITUDE GIVEN~~
~~Interpolated from Topo. Sheet~~ X
~~Determined by Inst. Leveling~~
Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

C.O. Aldredge
Well Driller

L-4363 17.33.35.321

Form WR-23

FIELD ENGR. LOG

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

		0	

(Plat of 640 acres)

(A) Owner of well Gulf Oil Corporation

Street and Number P.O. Box 2167

City Hobbs State New Mexico

Well was drilled under Permit No. ______ and is located in the

SE ¼ SE ¼ SE ¼ of Section 35 Twp. 17 S Rge. 33 E

(B) Drilling Contractor Abbott Brothers License No. WD-46

Street and Number P.O. Box 637

City Hobbs State New Mexico

Drilling was commenced April 6 19__

Drilling was completed April 8 19 63

Elevation at top of casing in feet above sea level ______ Total depth of well 233

State whether well is shallow or artesian shallow Depth to water upon completion 180

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	180	230	50	water sand
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	20	10	0	233	233	open	180	233

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

ROSWELL, N. MEX.
DISTRICT II
STATE ENGINEER OFFICE
1963 APR 11 AM 8:05

✓

File No. L-5696 Use OWD Location No. 17.33.35.433

OWD - 8K

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	1	1		soil
1	18	17		caliche
18	150	132		sand
150	230	80		water sand
230	233	3		sandy clay

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrell Abbott Jr.

Well Driller

Form WR-23

STATE ENGINEER OFFICE

~~SANTA FE~~ FIELD ENGR. LOG

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(A) Owner of well [illegible]

Street and Number P.O. Box [illegible]

City [illegible] State [illegible]

Well was drilled under Permit No. L-5096 and is located in the [illegible] ¼ [illegible] ¼ [illegible] ¼ of Section [illegible] Twp. [illegible] S Rge. [illegible] 3 E

(B) Drilling Contractor [illegible] License No. [illegible]

Street and Number P.O. Box [illegible]

City [illegible] State New Mexico

Drilling was commenced March [illegible] 19

Drilling was completed March [illegible] 19 [illegible]

280 FSL
2410 FEL

(Plat of 640 acres)

Elevation at top of casing in feet above sea level Total depth of well 239

State whether well is shallow or artesian Shallow Depth to water upon completion 150

Section 2 — PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	150	230	80	water sand
2				
3				
4				
5				

Section 3 — RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
7	20	10	[illegible]	230	230	open	150	230

Section 4 — RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 — PLUGGING RECORD

Name of Plugging Contractor License No.

Street and Number City State

Tons of Clay used Tons of Roughage used Type of roughage

Plugging method used Date Plugged 19

Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received ✓

ROSWELL, N. MEX.
DISTRICT II
STATE ENGINEER OFFICE
1963 APR 11 AM 8:06

File No. L-5055 Use OWD Location No. 17.33.35.43332

OWD-OK

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
0	1 w	1		soil
1	18	17		caliche
18	150	132		sand
150	230	80		water sand
230	233	3		sandy clay

~~L S Elev~~ 4120
Depth to K ____ Tr 233
Elev of K ____ Tr 3887

Loc. No. 17.33.35.433325

Hydro. Survey ____ Field Check X

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling ____

Other ____ , ____

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Murrell Abbott

Well Driller

SECTION ______

TOWNSHIP 18S

RANGE 32E

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well B.E. Frizzell Owner's Well No. ____
Street or Post Office Address P.O. Box 190
City and State Hobbs, New Mexico 88240

Well was drilled under Permit No. CP-566 and is located in the:

a. ____ ¼ SE ¼ SE ¼ NW ¼ of Section 4 Township 18S Range 32E N.M.P.M.

b. Tract No. ____ of Map No. ____ of the ____

c. Lot No. ____ of Block No. 13 of the Chapparel
Subdivision, recorded in Lea County.

d. X= ____ feet, Y= ____ feet, N.M. Coordinate System ____ Zone in the ____ Grant.

(B) Drilling Contractor Abbott Bros. License No. WD-46

Address P.O. Box 637, Hobbs, New Mexico 88240

Drilling Began 6/1/77 Completed 6/3/77 Type tools Cable Size of hole 8½ in.

Elevation of land surface or ____ at well is ____ ft. Total depth of well 133 ft.

Completed well is [X] shallow [ ] artesian. Depth to water upon completion of well 65 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
65	133	68	Sand	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6 5/8	21	Welded	0	133	133	None	65	133

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				
					Cement at top

Section 5. PLUGGING RECORD

Plugging Contractor ____
Address ____
Plugging Method ____
Date Well Plugged ____
Plugging approved by:

____
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received June 13, 1977

Quad ____ FWL ____ FSL ____

File No. CP-566 Use Dom Location No. 18.32.4.144

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	2	2	Surface soil
2	26	24	Caliche
26	65	59	Sand-tight
65	91	26	Sand-water
91	107	16	Sand-tight
107	129	22	Sand-water
129	133	4	Sandy clay

STATE ENGINEER OFFICE
ROSWELL, N.M.
'77 JUN 13 AM 8 21

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Murrell Abbott H.B.
Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted t[illegible] appropriate district office of the State Engineer. A[illegible]ions, except Section 5, shall be answered as completely and accurat[illegible] possible when any well is drilled, repaired or deepe[illegible]hen this form is used as a plugging record, only Section 1(a) and Section [illegible]eed be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Virgil Linam Estate Owner's Well No. ______
Street or Post Office Address Z Faye L. Klein, P.O. Box 1503
City and State Hobbs, New Mexico 88241

Well was drilled under Permit No. CP-672 and is located in the:

a. Center of ¼ ____ ¼ SE ¼ SE ¼ of Section 7 Township 18S Range 32E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in Lea County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Abbott Bros. Drilling License No. WD-46

Address P.O. Box 637, Hobbs, New Mexico 88240

Drilling Began 7/17/92 Completed 8/7/92 Type tools Cable Size of hole 10 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 524 ft.

Completed well is [X] shallow [ ] artesian. Depth to water upon completion of well 430 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
460	517	57	Sand	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
9 5/8	33	Welded	0	125	125		None	
5½	15	Welded	0	527	527		459	524

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received August 12, 1992

Quad ______ FWL ______ FSL ______

File No. CP-672 Use CLOW STOCK Location No. 18.32.7.44233

18.32.7.44233

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	6	6	Top soil
6	21	15	Caliche
21	94	73	Red and brown clay
94	100	6	Grey sand (Water cased off with 9 5/8" pipe)
100	402	302	Red bed with brown & blue streaks
402	456	54	Red clay
456	460	4	Brown clay
460	489	29	Sand W/clay streaks (WATER)
489	493	4	Red clay
493	517	24	Sand W/clay streaks
517	524	7	Red Bed

Section 7. REMARKS AND ADDITIONAL INFORMATION

STATE ENGINEER OFFICE
ROSWELL NEW MEXICO
'92 AUG 12 AM 10 41

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Murrell Abbott
Driller
J.B.

INSTRUCTIONS: This fo[illegible] should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Enginee[illegible]ctions, except Section 5, shall be answered as completely and accurat[illegible]ossible when any well is drilled, repaired or de[illegible] When this form is used as a plugging record, only Section 1(a) and Sectio[illegible]d be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Virgil Linam Est. by Faye L. Kline Owner's Well No. ______
Street or Post Office Address Carlsbad Hwy.
City and State Hobbs, NM 88240

Well was drilled under Permit No. Cp672 and is located in the:

a. ____ ¼ ____ ¼ SE ¼ SE ¼ of Section 7 Township 18S Range 32E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in Lea County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Larry's Drilling License No. WD882

Address 2601 W.Bender, Hobbs,NM 88240

Drilling Began 1-22 --85 Completed 1-29-85 Type tools tricone Size of hole 83/4 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 540 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well 460 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
498	510	12	clay & gravel, small amt. of sand	12

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
65/8	160PVC		-1	540	541		480	540

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received February 8, 1985

Quad ______ FWL ______ FSL ______

File No. CP-672 Use STOCK Location No. 18.32.7.44144

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	6	6	blowsand
6	12	6	gray & white sand
12	16	4	soft caliche
16	64	48	brown clay
64	150	86	red clay
150	220	70	brown clay
220	498	278	red clay with streaks of brown & gray clay
498	510	12	small gravel, brown clay
510	540	30	brown & red clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

STATE ENGINEER
ROSWELL, NM

FEB 8 8 37 AM '85

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Larry Fopine
Driller

INSTRUCTIONS: This fo uld be executed in triplicate, preferably typewritten, and submit appropriate district office of the State Engineer. A ons, except Section 5, shall be answered as completely and accu possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Secti need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Billy Williams Owner's Well No. TH #1

Street or Post Office Address

City and State Maljamar, N.M.

Well was drilled under Permit No. ______ and is located in the:

a. SE ¼ SW ¼ NE ¼ NE ¼ of Section 16 Township 18S Range 32E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______ Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Larry Felkins License No. ______

Address Hobbs, N.M.

Drilling Began 9/3/91 Completed 9/3/91 Type tools Rotary Size of hole 5¼ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 100 ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well Dry ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______

Address ______

Plugging Method ______

Date Well Plugged ______

Plugging approved by:

______

State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received

Quad ______ FWL ______ FSL ______

File No. None Use EXP Location No. 18.32.16.223433

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	20	20	sand
20	36	16	sand - some gravel
36	42	6	sand, some gravel, red clay
42	70	28	red clay and sand
70	79	9	red clay, some gravel
79	85	6	sand
85	94	9	sand and gravel
94	100	6	red clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All s[illegible]s, except Section 5, shall be answered as completely and accurately [illegible]ssible when any well is drilled, repaired or deepene[illegible] this form is used as a plugging record, only Section 1(a) and Section [illegible] be completed.

Revised June 197[illegible]

STATE ENGINEER OFFICE
WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well T X O Prod. Owner's Well No.
Street or Post Office Address c/o Glenn's Water Well Service, Inc.
City and State Box 692 Tatum, New Mexico 88267

Well was drilled under Permit No. CP-677 and is located in the:

a. ___ ¼ W½ ¼ NW ¼ NW ¼ of Section 26 Township 18-S. Range 32-E. N.M.P.M.

b. Tract No. ___ of Map No. ___ of the ___

c. Lot No. ___ of Block No. ___ of the ___
Subdivision, recorded in ___ County.

d. X= ___ feet, Y= ___ feet, N.M. Coordinate System ___ Zone in the ___ Grant.

(B) Drilling Contractor Glenn's Water Well Service License No. WD 421

Address Box 692 Tatum, New Mexico 88267

Drilling Began 5/9/85 Completed 5/9/85 Type tools Rotary Size of hole 7 7/8 in.

Elevation of land surface or ___ at well is ___ ft. Total depth of well 700 ft.

Completed well is [X] shallow [ ] artesian. Depth to water upon completion of well ___ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
			Dry Hole	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				
	well was pluged with sand and mud				

Section 5. PLUGGING RECORD

Plugging Contractor ___
Address ___
Plugging Method ___
Date Well Plugged ___
Plugging approved by:

___
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received May 15, 1985

Quad ___ FWL ___ FSL ___

File No. CP-677 Use OWD Location No. 18.32.26.11143

18.32.26.11143

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	12	12	sand-loose
12	24	12	clay
24	47	23	caleche
47	58	11	sand
58	84	26	sandy clay
84	102	18	red clay sticky
102	116	14	sand and gravel
116	142	26	red clay sticky
142	315	173	brown clay
315	325	10	purple clay
325	378	53	red clay
378	408	30	pink red clay
408	440	32	brown shale and blue streaks
440	500	60	brown shale-grainey
500	530	30	sand rock -fine
530	545	15	brown shale
545	605	60	sand rock-medium
605	616	11	brown shale
616	675	59	sand rock
675	700	25	red shale

Section 7. REMARKS AND ADDITIONAL INFORMATION

STATE ENGINEER ROSWELL, NM

MAY 15 8 28 AM '85

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

[illegible]

Driller

INSTRUCTIONS: [illegible] should be executed in triplicate, preferably typewritten, and su[illegible] o the appropriate district office of the State Enginee[illegible] sections, except Section 5, shall be answered as completely and [illegible] ely as possible when any well is drilled, repaired or deep[illegible] ned. When this form is used as a plugging record, only Section 1(a) and [illegible] ion 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Duval Corporation Owner's Well No. ______
Street or Post Office Address 5357 East Pima St.
City and State Tucson, AZ 85712

Well was drilled under Permit No. 0-13-0D2 and is located in the:

a. NE ¼ NW ¼ NW ¼ NW ¼ of Section 32 Township 18 S Range 32 E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Boyles Bros. License No. ______

Address 1624 Pioneer Road, Salt Lake City, Utah 84104

Drilling Began May 31, 1977 Completed June 22, 1977 Type tools ______ Size of hole ______ in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 2060 ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
274			TRC	
575			TRS	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
7			0	2D				
4½	9½		0	1195				

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				
1195		5 7/8		10	Displacement

Section 5. PLUGGING RECORD

Plugging Contractor Boyles Bros.
Address 1624 Pioneer Rd, Salt Lake City, U
Plugging Method Displacement
Date Well Plugged June 22, 1977
Plugging approved by: [signature]
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1	0	2040	165
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received July 20, 1981

Quad ______ FWL ______ FSL ______

File No. 0-13-002 Use EXP Location No. 18.32.32.111244

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This f hould be executed in triplicate, preferably typewritten, and submitted appropriate district office of the State Engineer. A tions, except Section 5, shall be answered as completely and accura possible when any well is drilled, repaired or deepe When this form is used as a plugging record, only Section 1(a) and Sectio need be completed.

SECTION ____________

TOWNSHIP 18S

RANGE 33E

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Oxy USA Inc. Owner's Well No. ______

Street or Post Office Address PO Box 56250

City and State Midland, Texas 79710

Well was drilled under Permit No. CP-758 Exploratory and is located in the:

a. ____ ¼ ____ ¼ ____ ¼ SW ¼ of Section 4 Township 18S Range 33E N.M.P.M.

b. Tract No. ____ of Map No. ____ of the ______

c. Lot No. ____ of Block No. ____ of the ______
Subdivision, recorded in ______ County.

d. X= ____ feet, Y= ____ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Dubose Drilling Inc. License No. WD-1107

Address 5407 N. Golder, Odessa, Texas 79764

Drilling Began 5-8-91 Completed 5-10-91 Type tools rerun Size of hole 12 3/4 in.

Elevation of land surface or ______ at well is XXX ft. Total depth of well 250 ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well absent ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
			ABSENT	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet Top	Depth in Feet Bottom	Length (feet)	Type of Shoe	Perforations From	Perforations To

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement

Section 5. PLUGGING RECORD

Plugging Contractor Dubose DrillingInc.

Address ______

Plugging Method Back fill witheuttings

Date Well Plugged 5-10-91

Plugging approved by:

Ken Fraquez

State Engineer Representative

No.	Depth in Feet Top	Depth in Feet Bottom	Cubic Feet of Cement
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received May 16, 1991

Quad ______ FWL ______ FSL ______

File No. CP-758-Exploratory Use EXP Location No. 18.33.4.34233

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	5	5	Surface soil
5	32	27	caliche
32	50	18	purple / grey clay
50	65	15	clay and shale conglomerate purple and g
65	90	25	brown clay wiht rgrey stringers
90	120	30	red bed
120	190	70	Brown clay
190	195	5	color change to light brown clay
195	250	55	brown clay
			no water, back fill hole with cuttings

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All [illegible]ns, except Section 5, shall be answered as completely and accuratel[illegible]ssible when any well is drilled, repaired or deepene[illegible] this form is used as a plugging record, only Section 1(a) and Section [illegible] be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

FIELD ENGR. LOG

Section 1. GENERAL INFORMATION

(A) Owner of well B. J. Woolley dba Caprock Sand & Gravel Owner's Well No. ______

Street or Post Office Address Box 776

City and State Eunice, New Mexico 88231

Well was drilled under Permit No. CP-546 and is located in the:

NW¼ SE¼

a. NE ¼ NE ¼ SE ¼ ____ ¼ of Section 9 Township 18-S Range 33-E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in Lea County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor W. L. Van Noy License No. WD-208

Address Box 74 Oil Center, New Mexico 88266

Drilling Began June 1,1975 Completed June 3, 1975 Type tools Spudder Size of hole 10 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 90 70 ft.

Completed well is ☐ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
70	85	15	fine water sand.	

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6 5/8"	welded		0	90	90	none	70	85

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______

Address ______

Plugging Method ______

Date Well Plugged ______

Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received October 2, 1978

Quad ______ FWL ______ FSL ______

File No. CP-546 Use COM. Location No. 18.~~39~~ 33.9.42241

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	5	5	top soil
5	30	25	caliche
30	65	35	brown sand rock
65	70	5	hard rock
70	85	15	fine water sand
85	90	5	red bed.

L S Elev ______ 3978

Depth to K______Trc 85

Elev of K______Trc 3893

~~Loc. No.~~ 18.33.9. 42241

Hydro. Survey______Field Check FB

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling______

Other______

Section 7. REMARKS AND ADDITIONAL INFORMATION

1970 OCT -2 AM 8:27

STATE ENGINEER OFFICE

DISTRICT II

ROSWELL, N. MEX.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

W. L. Van Noy

Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Heyco's Harvey Yates Owner's Well No.
Street or Post Office Address c/o Glenn's Water Well Service, Inc.
City and State Box 692 Tatum, N.M. 88267

Well was drilled under Permit No. CP- 702 and is located in the:

a. ¼ SE ¼ NW ¼ SW ¼ of Section 11 Township 18-S. Range 33-E. N.M.P.M.

b. Tract No. of Map No. of the

c. Lot No. of Block No. of the
Subdivision, recorded in County.

d. X= feet, Y= feet, N.M. Coordinate System Zone in the Grant.

(B) Drilling Contractor Glenn's Water Well Service, Inc. License No. WD 421

Address Box 692 Tatum, N.M. 88267

Drilling Began 10/21/86 Completed 10/21/86 Type tools Rotary Size of hole 9 7/8 in.

Elevation of land surface or at well is ft. Total depth of well 100 ft.

Completed well is [X] shallow [ ] artesian. Depth to water upon completion of well ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
52	82	30	gravel	40

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6 5/8	.150						50	90

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor
Address
Plugging Method
Date Well Plugged
Plugging approved by:

State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received October 27, 1986

Quad FWL FSL

File No. CP-702 Use OWD Location No. 18.33.11.314112

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	2	2	soil
2	24	22	calecche
24	52	28	sand
52	82	30	gravel
82	100	18	red clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

[illegible signature]
Driller

INSTRUCTIONS: This form sho[illegible]be executed in triplicate, preferably typewritten, and submitted to [illegible]propriate district office of the State Engineer. All [illegible]xcept Section 5, shall be answered as completely and accuratel[illegible]sible when any well is drilled, repaired or deepened. W[illegible]is form is used as a plugging record, only Section 1(a) and Section 5 [illegible]t be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Heyco's Harvey Yates Owner's Well No.
Street or Post Office Address c/o Glenn's Water Well Service, Inc.
City and State Box 692 Tatum, N.M. 88267

Well was drilled under Permit No. CP-701 and is located in the:

a. ___ ¼ E½ ¼ NW ¼ SW ¼ of Section 11 Township 18-S. Range 33-E. N.M.P.M.

b. Tract No. ___ of Map No. ___ of the ___

c. Lot No. ___ of Block No. ___ of the ___
Subdivision, recorded in ___ County.

d. X= ___ feet, Y= ___ feet, N.M. Coordinate System ___ Zone in the ___ Grant.

(B) Drilling Contractor Glenn's Water Well Service, Inc. License No. WD421

Address Box 692 Tatum, New Mexico 88267

Drilling Began 10/20/86 Completed 10/20/86 Type tools Rotary Size of hole 9 7/8 in.

Elevation of land surface or ___ at well is ___ ft. Total depth of well 100 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well ___ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
54	84	30	gravel	40

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet Top	Depth in Feet Bottom	Length (feet)	Type of Shoe	Perforations From	Perforations To
6 5/8	.156						50	90

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement

Section 5. PLUGGING RECORD

Plugging Contractor ___
Address ___
Plugging Method ___
Date Well Plugged ___
Plugging approved by:

___
State Engineer Representative

No.	Depth in Feet Top	Depth in Feet Bottom	Cubic Feet of Cement
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

✓ Date Received October 27, 1986

Quad ___ FWL ___ FSL ___

File No. CP-701 Use OWD Location No. 18.33.11.314121

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	2	2	soil
2	22	20	caleche
22	54	34	sand
54	84	30	gravel
84	100	16	red clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: Th[illegible]ould be executed in triplicate, preferably typewritten, and subm[illegible]e appropriate district office of the State Enginee[illegible]ions, except Section 5, shall be answered as completely and ac[illegible]s possible when any well is drilled, repaired or deepene[illegible] When this form is used as a plugging record, only Section 1(a) and Sec[illegible]on 5 need be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

FIELD ENGR. LOG

Section 1. GENERAL INFORMATION

(A) Owner of well B. J. Wooley Owner's Well No. ______
Street or Post Office Address P.O.Box 207
City and State Hobbs, NM 88240

Well was drilled under Permit No. L-8288 and is located in the:

a. ____ ¼ ____ ¼ SW ¼ SW ¼ of Section 12 Township 18S Range 33E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in Lea County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Larry's Drilling License No. WD882

Address 2601 W. Bender Hobbs, NM 88240

Drilling Began 5-11-82 Completed 5-11-82 Type tools button bit Size of hole 9 7/8 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 79 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well 60 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
60	80	20	sand & gravel	60

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6 5/8	160PVC		+1	79	80		~~XX~~ 60	79

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received September 24, 1982

Quad 107.2.0 FWL ______ FSL ______

File No. L-8288 Use COMMERCIAL Location No. 18.33.12.33334

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	2	2	blow sand
2	15	13	caliche
15	59	44	sand
~~XXX~~ 59	79	20	gravel
79	80	1	gray yellow clay
			L S Elev 3993 Depth to K ____ Trc 79 Elev of K ____ Trc 3914
			Loc. No. 33.12.33334
			Hydro. Survey ____ Field Check LB
			SOURCE OF ALTITUDE GIVEN
			~~Interpolated from Topo. Sheet~~ X XX 90
			Determined by Inst. Leveling ____
			Other ____

Section 7. REMARKS AND ADDITIONAL INFORMATION

SEP 24 9 42 AM '92

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Larry Jelkins
Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1(a) and Section 5 need be completed.

Form WR-23

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well S. P. Yates Drilling Company
Street and Number 311 Carper Building
City Artesia State New Mexico
Well was drilled under Permit No. L-2870 and is located in the ¼ SE ¼ SE ¼ of Section 12 Twp. 18S Rge. 33E
(B) Drilling Contractor Claude Tatum License No. WD33
Street and Number 524 West Washington
City Lovington State New Mexico
Drilling was commenced May 22 1955
Drilling was completed May 30 1955

Elevation at top of casing in feet above sea level ~~unknown~~ 4090 Total depth of well 205
State whether well is shallow or artesian shallow Depth to water upon completion 150

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	150	205	55	Water sands
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
6	20	8	0	205	205	none	150	205

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

FILED
JUN 29 1955
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

Section 5 PLUGGING RECORD

Name of Plugging Contractor License No.
Street and Number City State
Tons of Clay used Tons of Roughage used Type of roughage
Plugging method used Date Plugged 19
Plugging approved by:

Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received June 29, 1955

File No. L-2878 Use Oil Location No. 18.33.12.440

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	10	10	white	Calliche and rock
10	40	30	white	calliche and rock
40	80	40	red	dry sand
80	160	80	white	coarse sand
160	200	40	red	water sands
200	205	5	red	clay

L S Elev 4089

Depth to K Trc 200

~~Elev of~~ K Trc 3889

Loc. No. 18.33.12.441122

Hydro. Survey Field Check X

SOURCE OF ALTITUDE GIVEN

Interpolated from Topo. Sheet X

Determined by Inst. Leveling

Other

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

Claude Tatum

Well Driller

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

FIELD ENGR. LOG

Section 1. GENERAL INFORMATION

(A) Owner of well B.J. Wooley Owner's Well No. ______

Street or Post Office Address P.O.Box 207

City and State Hobbs, NM 88240

Well was drilled under Permit No. CP-623 and is located in the:

a. ___ ¼ ___ ¼ NW ¼ NW ¼ of Section 13 Township 18S Range 33E N.M.P.M.

b. Tract No. ___ of Map No. ___ of the ___

c. Lot No. ___ of Block No. ___ of the ___
Subdivision, recorded in Lea County.

d. X= ___ feet, Y= ___ feet, N.M. Coordinate System ___ Zone in the ___ Grant.

(B) Drilling Contractor Larry's Drilling License No. WD882

Address 2601 W. Bender Hobbs, NM

Drilling Began 5-10-82 Completed 5-10-82 Type tools button bit Size of hole 97/8 in.

Elevation of land surface or ___ at well is ___ ft. Total depth of well 82 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well 60 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
70	80	10	sand & gravel	40

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6 5/8	160PVC		+1	82	83		70	80

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ___
Address ___
Plugging Method ___
Date Well Plugged ___
Plugging approved by:

___
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received September 24, 1982

Quad 107.2.0 FWL ___ FSL ___

File No. CP-623 Use COMMERICAL Location No. 18.33.13.11112

18.33.13.1112

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	6	6	blow sand
6	11	5	caliche
11	70	59	sand
70	80 XX	10XXX	gravel & sand
80	82	2	red bed

L S Elev 3989
Depth to K Trc 80
Elev of K Trc 3909

~~Loc. No.~~ 18.33.13. 11112
Hydro. Survey Field Check FB

SOURCE OF ALTITUDE GIVEN
Interpolated from Topo. Sheet X
~~Determined by Inst. Leveling~~
Other

Section 7. REMARKS AND ADDITIONAL INFORMATION

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Larry Fisher
Driller, pf

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. A[illegible]ons, except Section 5, shall be answered as completely and accurate[illegible]ossible when any well is drilled, repaired or deepene[illegible]hen this form is used as a plugging record, only Section 1(a) and Sectio[illegible]eed be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well Sun Oil Owner's Well No. ______
Street or Post Office Address c/o Glenn's Water Well Service, Inc.
City and State Box 692 Tatum, N.M. 88267

Well was drilled under Permit No. CP-689 and is located in the:

a. ____ ¼ ____ ¼ NE ¼ NW ¼ of Section 13 Township 18-S. Range 33-E. N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor Glenn's Water Well Service License No. WD 421

Address Box 692 Tatum, N.M. 88267

Drilling Began 12/7/85 Completed 12/7/85 Type tools rotary Size of hole 9 7/8 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 100 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well ______ ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
70	95	25	gravel	120

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
102'	.142	steel casing					65	100

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received December 13, 1985

Quad ______ FWL ______ FSL ______

File No. CP-689 Use OWD Location No. 18.33.13.12122

18.33.13.1

Section 6. LOG OF HOLE

Depth in Feet		Thickness in Feet	Color and Type of Material Encountered
From	To		
0	5	5	sand
5	29	24	caleche
29	65	36	sand
65	95	30	gravel
95	100	5	yellow clay

Section 7. REMARKS AND ADDITIONAL INFORMATION

DEC 13 8 31 AM '85
STATE ENGINEER
ROSWELL NM

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Driller

INSTRUCTIONS: This for[illegible]ld be executed in triplicate, preferably typewritten, and submitted t[illegible]ropriate district office of the State Engineer. All [illegible], except Section 5, shall be answered as completely and accuratel[illegible]ssible when any well is drilled, repaired or deepened. [illegible]en this form is used as a plugging rec[illegible]d, only Section 1(a) and Secti[illegible] [illegible]l be completed.

Revised June 1972

STATE ENGINEER OFFICE

WELL RECORD

Section 1. GENERAL INFORMATION

(A) Owner of well KMR, INC. Owner's Well No. ______
Street or Post Office Address P.O.BOX 1832
City and State HOBBS, NM 88240

Well was drilled under Permit No. CP-769-EXPLORATORY and is located in the:

a. ____ ¼ NW ¼ NW ¼ NE ¼ of Section 13 Township 18S Range 33E N.M.P.M.

b. Tract No. ______ of Map No. ______ of the ______

c. Lot No. ______ of Block No. ______ of the ______
Subdivision, recorded in ______ County.

d. X= ______ feet, Y= ______ feet, N.M. Coordinate System ______ Zone in the ______ Grant.

(B) Drilling Contractor LARRY'S DRILLING, INC. License No. WD882

Address 2116 W. BENDER HOBBS, NM 88240

Drilling Began 5-6-92 Completed 5-6-92 Type tools BUTTON BIT Size of hole 97/8 in.

Elevation of land surface or ______ at well is ______ ft. Total depth of well 115 ft.

Completed well is ☒ shallow ☐ artesian. Depth to water upon completion of well 70 ft.

Section 2. PRINCIPAL WATER-BEARING STRATA

Depth in Feet		Thickness in Feet	Description of Water-Bearing Formation	Estimated Yield (gallons per minute)
From	To			
80	115	35	SAND & SANDSTONE	20

Section 3. RECORD OF CASING

Diameter (inches)	Pounds per foot	Threads per in.	Depth in Feet		Length (feet)	Type of Shoe	Perforations	
			Top	Bottom			From	To
6 5/8	160PVC		0	115	115		90	110

Section 4. RECORD OF MUDDING AND CEMENTING

Depth in Feet		Hole Diameter	Sacks of Mud	Cubic Feet of Cement	Method of Placement
From	To				

Section 5. PLUGGING RECORD

Plugging Contractor ______
Address ______
Plugging Method ______
Date Well Plugged ______
Plugging approved by:

______
State Engineer Representative

No.	Depth in Feet		Cubic Feet of Cement
	Top	Bottom	
1			
2			
3			
4			

FOR USE OF STATE ENGINEER ONLY

Date Received May 21, 1992

Quad ______ FWL ______ FSL ______

File No. CP-769-Exploratory Use EXP Location No. 18.33.13.21142
(THIS WELL WILL NOW BE CP-72-A - TO BE USED FOR COM USE) 18.33.13.21142

Section 6. LOG OF HOLE

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color and Type of Material Encountered
0	12	12	SAND
12	21	9	CALICHE
21	36	15	SAND & GRAVEL
36	52	16	XXXXXAN SAND, RED & GRAY CLAY
52	66	14	RIO SAND & SOME CLAY
66	85	19	SAND & SOME GRAVEL
85	110	25	SAND & GRAVEL
110	115	05	RED BED

Section 7. REMARKS AND ADDITIONAL INFORMATION

'92 MAY 21 AM 10 10
STATE ENGINEER OFFICE
ROSWELL, NEW MEXICO

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described hole.

Larry [illegible]
Driller

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the appropriate district office of the State Engineer. A[illegible]ons, except Section 5, shall be answered as completely and accurat[illegible] possible when any well is drilled, repaired or deepe[illegible]n this form is used as a plugging record, only Section 1(a) and Secti[illegible]d be completed.

Form WR-23 STATE ENGINEER OFFICE COPY

FIELD ENGR. LOG

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres) COPY

(A) Owner of well Scharbauer Cattle Company
Street and Number Box 1471
City Midland, State Texas
Well was drilled under Permit No. L-6347 and is located in the ¼ SE ¼ SE ¼ of Section 12 Twp. 18S Rge. 33E
(B) Drilling Contractor O. R. musslewhite License No. WD99
Street and Number Box 56
City Hobbs, State New Mexico
Drilling was commenced July 11, 19 68
Drilling was completed July 12, 19 68

Elevation at top of casing in feet above sea level ______ Total depth of well 170
State whether well is shallow or artesian Shallow Depth to water upon completion 130

Section 2 PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1				Cleaned out old well.
2				
3				
4				
5				

Section 3 RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
6	10							

Section 4 RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5 PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______
Street and Number ______ City ______ State ______
Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______
Plugging method used ______ Date Plugged ______ 19__
Plugging approved by:

______ Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received

FILED
JUL 22 1968
OFFICE
GROUND WATER SUPERVISOR
ROSWELL, NEW MEXICO

41222

File No L-6347 Use Stock Location No. 18.33.12.440

Section 6

LOG OF WELL

Depth in Feet		Thickness in Feet	Color	Type of Material Encountered
From	To			
				Cleaned out old well.

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

A. R. Musslewhite
Well Driller

Form WR-23

Orig. to S.

STATE ENGINEER OFFICE

WELL RECORD

INSTRUCTIONS: This form should be executed in triplicate, preferably typewritten, and submitted to the nearest district office of the State Engineer. All sections, except Section 5, shall be answered as completely and accurately as possible when any well is drilled, repaired or deepened. When this form is used as a plugging record, only Section 1A and Section 5 need be completed.

Section 1

(Plat of 640 acres)

(A) Owner of well Mr. E. H. Ellison

Street and Number Star Route B.

City Hobbs State New Mexico

Well was drilled under Permit No. L-3454 and is located in the N E ¼ N E corner ¼ of Section 30 Twp. 18 S Rge 33 E

(B) Drilling Contractor O.R. Musslewhite License No. W D 99

Street and Number P.O. Box 56

City Hobbs State New Mexico

Drilling was commenced March 29 1957

Drilling was completed March 30 1957

Elevation at top of casing in feet above sea level ______ Total depth of well 100

State whether well is shallow or artesian shallow Depth to water upon completion 35

Section 2

PRINCIPAL WATER-BEARING STRATA

No.	Depth in Feet From	Depth in Feet To	Thickness in Feet	Description of Water-Bearing Formation
1	70	97	27	Red sand and sand rock
2				
3				
4				
5				

Section 3

RECORD OF CASING

Dia in.	Pounds ft.	Threads in	Depth Top	Depth Bottom	Feet	Type Shoe	Perforations From	Perforations To
6 5/8	20	none	0	100	100	none	75	100

Section 4

RECORD OF MUDDING AND CEMENTING

Depth in Feet From	Depth in Feet To	Diameter Hole in in.	Tons Clay	No. Sacks of Cement	Methods Used

Section 5

PLUGGING RECORD

Name of Plugging Contractor ______ License No. ______

Street and Number ______ City ______ State ______

Tons of Clay used ______ Tons of Roughage used ______ Type of roughage ______

Plugging method used ______ Date Plugged ______ 19__

Plugging approved by:

______
Basin Supervisor

Cement Plugs were placed as follows:

No.	Depth of Plug From	Depth of Plug To	No. of Sacks Used

FOR USE OF STATE ENGINEER ONLY

Date Received ______

APR 17 1957

OFFICE

GROUND WATER SUPERVISOR

ROSWELL, N.M.

File No. L-3454 Use Dom Location No. 18.33.30.220

Section 6

LOG OF WELL

Depth in Feet From	Depth in Feet To	Thickness in Feet	Color	Type of Material Encountered
0	1	1	Brown	Soil
1	25	24	White	Cleachie and rock
25	50	25	Grey	Sandy shale
50	97	47	Red	Sand and sand rock
97	100	3	Brown	Quartsite
				L S Elev
				Depth to K Trc
				~~Elev of K Trc~~

The undersigned hereby certifies that, to the best of his knowledge and belief, the foregoing is a true and correct record of the above described well.

C. R. Muselwhite
Well Driller

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME II: FACILITY MANAGEMENT PLANS
SECTION 8: VADOSE ZONE MONITORING PLAN

ATTACHMENT II.8.B

VADOSE ZONE MONITORING FORM

(TYPICAL)

ATTACHMENT II.8.B

Vadose Zone Monitoring Form (Typical)

DNCS Environmental Solutions

Monitoring Personnel ______________________ ______________________

Weather Information

Date and Amount of Last Precipitation: ______________________

Temp: ______________________ °F

Wind Speed: ______________________ mph

Wind Direction: ______________________

Barometric Pressure: ______________________ inches mercury (Hg)

Weather Conditions: ______________________

Equipment Information

Monitoring Equipment Used: ______________________

Date and Time Last Calibrated: ______________________

Monitoring Equipment Used: ______________________

Date and Time Last Calibrated: ______________________

Well I.D.	Monitoring Date (dd/mm/yy)	Total Well Depth (fbtoc)	Depth to Water (fbtoc)	Field Parameter Measurement				Water Volume Removed (gallons)	Sample Collected?		Observations (e.g., color, odor, clarity, etc.)
				Temperature (^oC)	pH (standard units)	Specific Conductance (mS/cm)	Methane (%) or (% LEL)		Y	N	
VM-1											
VM-2											
VM-3											
VM-4											
VM-5											
VM-6											
VM-7											
VM-8											
VM-9											
VM-10											

Notes:

- *fmsl: feet above mean sea level*
- *fbtoc: feet below top of PVC casing*

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME II: LANDFILL MANAGEMENT PLANS
SECTION 9: LEACHATE MANAGEMENT PLAN

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF ATTACHMENTS

VOLUME II: LANDFILL MANAGEMENT PLANS
SECTION 9: LEACHATE MANAGEMENT PLAN

1.0 INTRODUCTION

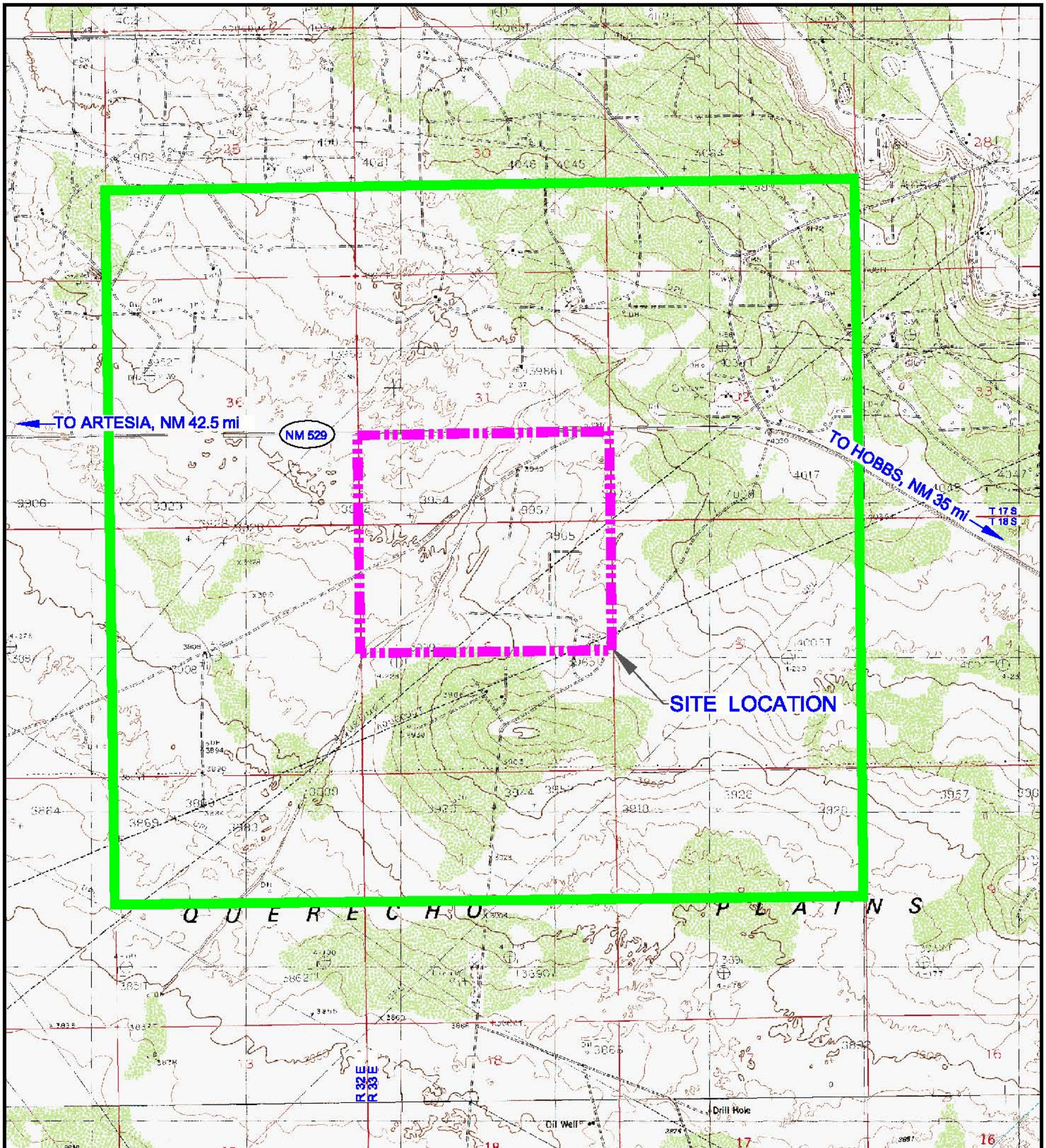
DNCS Environmental Solutions (DNCS Facility) is a proposed Surface Waste Management Facility for oil field waste processing and disposal services. The proposed DNCS Facility is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.36 NMAC, administered by the Oil Conservation Division (OCD). The Facility is designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, DNCS Properties, LLC.

1.1 Site Location

The DNCS site is located approximately 10.5 miles east of the US 82/NM 529 intersection and 6.3 miles south of Maljamar in unincorporated Lea County, New Mexico (NM). The DNCS site is comprised of a 562-acre ± tract of land located south of NM 529 in portions of Section 31, Township 17 South, Range 33 East; and in the northern half of Section 6, Township 18 South, Range 33 East, Lea County, NM (**Figure II.9.1**). Site access will be provided via the south side of NM 529.

1.2 Description

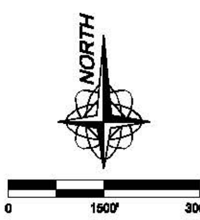
The DNCS Facility is a proposed new Surface Waste Management Facility that will include two main component;, a liquid oil field waste Processing Area (177 acres ±), and an oil field waste Landfill (318 acres ±). Oil field wastes are anticipated to be delivered to the DNCS Facility from oil and gas exploration and production operations in southeastern NM and west Texas. The **Permit Plans, Sheet 3** identifies the locations of the Processing Area and Landfill facilities.



LEGEND

- SITE BOUNDARY
- 1 MILE OFFSET FROM SITE

MAP REFERENCE:
LAGUNA GATUNA NW 1984,
MALJAMAR 1985 PROVISIONAL EDITION,
GREENWOOD LAKE 1985 PROVISIONAL EDITION, AND
DOG LAKE 1985 PROVISIONAL EDITION,
USGS 1:24000, 7.5 MINUTE SERIES, TOPOGRAPHIC MAPS



Drawing:P:\acad 2003\542.01.01\RAI 1\SITE LOC MAP.dwg
Date/Time:Jun. 13, 2014-07:16:05 ; LAYOUT: A (P)

SITE LOCATION MAP

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/03/2014	CAD: SITE LOC MAP.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: DRT	FIGURE II.9.1
APPROVED BY: IKG	gei@gordonenvironmental.com	

1.3 Purpose

A leachate management plan must be developed per 19.15.36.8.C.(12) NMAC that describes the anticipated amount and quality of fluids collected, along with the proposed management, storage and disposal technologies. This Leachate Management Plan (the Plan) details the procedures that will be used to manage contact waters generated at the DNCS Facility Landfill during the permit period and following closure. This Plan has been developed to address the design and performance requirements of 19.15.36.14 NMAC, and addresses the following items:

1. Projected amounts and rates of leachate generation
2. Expected duration of leachate generation
3. Leachate disposal options
4. Proposed treatment and disposal methods

2.0 LEACHATE COLLECTION SYSTEM

The leachate collection system designed for the DNCS Landfill meets or exceeds the minimum design and performance standards specified in 19.15.36.14 NMAC, specifically:

1. The minimum design slope on the landfill liner is 2.8%; and the minimum slope on the leachate piping system is 2.0%.
2. The leachate piping system will consist of perforated and solid pipe with a minimum diameter of 6 inches.
3. Both schedule 80 polyvinyl chloride (PVC) and standard dimension ration (SDR) 11 high density polyethylene (HDPE) piping are demonstrated to meet the site-specific performance standards.
4. The protective soil layer (minimum 24 inches of pervious soil) will provide both protection for the liner and leachate flow to the piping and extraction system.
5. There is a geonet leak detection layer and secondary 60 mil HDPE below the primary liner and leak collections system.

Each new cell will be outfitted with perforated leachate collection piping that is enveloped in aggregate and geotextile to promote flow while minimizing the intrusion of fines. The cell floor and liner system will be sloped at 45º to each pipe, and leachate will flow through the protective soil layer (PSL).

Permanent leachate sumps are designed for each cell at the DNCS Landfill. Temporary sumps and cleanout risers may also be installed as filling progresses in each cell. Therefore, each cell is designed with its own collection piping. Two solid pipe risers will provide access to each permanent leachate sump at the toe of the slope:

- The leachate extraction riser will be used to measure leachate levels in the leachate sump, and to provide access for a submersible pump to remove accumulated fluids.
- A cleanout riser is connected with a pipe elbow to the collection pipe to facilitate cleaning or flushing if necessary.

Compliance with the design standards of 19.15.36.14 NMAC is demonstrated in the **Permit Plans** (**Volume III.1**). The performance standards specified in the same subsections are addressed as follows:

1. The Liner Construction Quality Assurance (CQA) Plan (**Volume II.7**) specifies the materials and installation techniques which will be used for construction of the leachate collection system and protective soil layer.
2. The performance of the design and the specified materials are documented to meet OCD requirements in the following Landfill Engineering Calculations:
    - Pipe Loading Calculations (**Volume III.5**)
    - Geosynthetic Applications and Compatibility Documentation (**Volume III.6**)
    - Settlement Calculations (**Volume III.9**)

3.0 LEACHATE GENERATION

Leachate in the permanent extraction risers will be measured monthly and after significant rainfall events. The storage capacity in each sump is approximately 1,500 gallons. The maximum head accumulation on the liner is not to exceed 12 inches per 19.15.36.14.F NMAC. Fluid levels on the cell floor will be maintained below the regulatory threshold through regular pumping as recorded and reported to OCD. DNCS will maintain a record of actual leachate generation and management volumes, using a form similar to the one provided as **Attachment II.9.A** to track the amount of leachate removed from the sumps throughout a given year at the Facility.

Leachate production is projected to approach zero because of the solid nature of the waste and the paint filter restriction. Therefore, leachate generation is attributable solely to precipitation; and particularly fluids from precipitation in the very early stages of cell development.

The leachate generation rate decreases to nearly zero following the placement of the first lift of waste on the liner. This has been calculated in the HELP Model (**Volume III.4**) and confirmed through experience at other facilities. As demonstrated in the HELP Model, the field capacity of the waste and the local evaporation rate far exceed the volume of rainfall experienced at the site, and therefore liquids do not typically reach the leachate collection system. As discussed in detail in the Operations, Inspection, and Management Plan (**Volume II.1**), routine site operation procedures will dictate that a loose lift of waste (approximately 5 feet thick) be placed over the entire floor of a newly constructed cell as soon as practical. This process will protect the liner and leachate collection system; and reduce the generation of contact water, which is stormwater collected within the cell footprint. During the post-closure care period, the site will have been capped and vegetated (**Permit Plans**); and leachate production is modeled to decline to near zero.

4.0 LEACHATE MONITORING

Routine monitoring of leachate levels and extraction of leachate from the sumps will ensure that the fluid accumulation on the liner will not exceed the regulatory 12-inch threshold. Procedures to ensure leachate does not accumulate on the liner will include the following:

- The level of the leachate in the sumps will be monitored at least monthly, and leachate will typically be extracted on a minimum quarterly basis; or as needed to maintain <12 inches of head on the liner.
- The leachate will be extracted from the sumps with portable submersible pumps, vacuum trucks, or other suitable devices.
- In the future, the leachate sumps may be equipped with remote level sensors and/or dedicated submersible pumps, if routine leachate removal is required.

The Leachate Monitoring Form provided as **Attachment II.9.A** is a template for monitoring levels and extraction data, as well as the disposal technique used.

5.0 LEACHATE DISPOSAL

DNCS is requesting approval to recirculate leachate over lined areas of the landfill during the active life of the DNCS Facility. The following procedures will be adhered to when performing recirculation of leachate at DNCS:

- On an as-needed basis (initially anticipated to be quarterly), leachate will be pumped from the sump(s) with a portable or permanent submersible pump or vacuum to a tank truck, equipped with appropriate fluid transfer hoses, and will be transported to the active cell. Prior to applying daily cover to the cell, the leachate will be sprayed onto the exposed waste. Cover will be placed after the recirculation activities are complete.
- For the most effective recirculation, and to avoid short-circuiting, the leachate will be applied only in areas where the cell surface is at least 10 feet above the liner system. In addition, the leachate will be applied on cells upgradient in the collection system whenever possible. No leachate recirculation will be conducted within 50 feet of the solid waste boundary.
- Monitoring and recirculation activities will be documented on the Leachate Monitoring Form (**Attachment II.9.A**). The information will be maintained in the Facility Operating Record.

Leachate recirculation will be accomplished via similar collection, transport, and application methods in future cells. Alternatively, leachate may be applied directly to waste deposits in lined cells with pumps and hoses attached directly to the collection system. DNCS is seeking OCD's approval of additional leachate management alternatives that include, but are not limited to:

- disposal onsite through the Produced Water processing/evaporation process
- use of dilute leachate for dust control over lined cells
- disposal offsite at a OCD-approved facility

Disposal of leachate onsite through the Produced Water evaporation process will be accomplished by pumping leachate directly from the sump with a submersible pump or extraction hose to a tanker truck, equipped with appropriate fluid transfer hoses. The leachate will be transferred to the Produced Water Load-Out Station and unloaded into the Produced Water Receiving tanks for processing with the routine waste stream.

The use of dilute leachate for dust control over lined cells will be accomplished as follows:

- Leachate will be diluted with collected stormwater to minimize the potential for odors.
- The leachate application method will consist of spraying the dilute leachate with the site's water wagon, or similar type vehicle.
- The application of leachate will be conducted only over lined cell areas.
- Leachate will be sprayed evenly and thinly over lined cell areas to provide for effective dust control and evaporation, and to minimize the potential of recirculation through the waste.
- To enhance safety, leachate will be sprayed only when personnel are not near the spray surface. In addition, leachate will not be sprayed on windy days.
- If there are any issues regarding the potential composition of the leachate (for example, leachate being generated by some means other than heavy rainfall on a new cell), leachate may be analyzed prior to beneficial use in consultation with OCD.

Disposal of leachate offsite at a POTW or OCD-permitted liquids processing facility following closure may be conducted by pumping leachate directly from the sump with a submersible pump or extraction hose to a tanker truck, equipped with appropriate fluid transfer hoses. If the leachate is required to be sampled and analyzed by the disposal facility, the parameters to be analyzed will be determined in consultation with the POTW. Prior to transport, leachate samples will be collected and analyzed to demonstrate compliance with the disposal facility's leachate acceptance criteria for analytical parameters and concentrations. Prior to disposal, the Leachate Management Plan may be updated with OCD approval to reflect the analytical parameters and concentrations, as well as transport methods specified by the selected disposal facility. The updated Plan will be submitted to OCD for approval as an administrative change to the existing Plan prior to implementation of disposal activities. The analytical test results for leachate disposal at the off-site Facility will be maintained in the Facility Operating Record.

Following closure, the most effective treatment and disposal technology for leachate (if produced) will be determined and implemented with the approval of OCD. This disposal technology may include hauling off-site for treatment at an OCD-approved Facility. Leachate monitoring during post-closure will be conducted at least semi-annually. Leachate management information will continue to be documented and maintained in the Facility Operating Record.

6.0 LEAK DETECTION MONITORING

Routine inspection of the leak detection system and sump in each of the Landfill cells and evaporation ponds will be conducted on at least a monthly basis; and documented on the Leachate Monitoring Form (**Attachment II.9.A**), or the Pond Integrity/Leak Detection Inspection Form (**Attachment II.9.B**). At a minimum, the following items will be documented:

- Inspection date, time, and conditions
- Inspector identification
- Depth of liquids in sump
- Sump and piping condition and status
- Volume collected

Prior to placing a newly constructed landfill cell or evaporation pond (or an evaporation pond that has undergone repair or cleaning) into service, liquids will be removed from above the primary liner and from the leak detection system. Once in service, it is anticipated liquid may be present at all times due to condensation and nominal leakage through the primary liner. The sumps are 2 feet deep and have a capacity of approximately 1,500 gallons (gal) using a porosity of 0.40 for the granular material.

Attachment II.9.C is a summary table from an authoritative publication on potential geomembrane liner leakage for 40 mil HDPE lined ponds. As shown on the table, the combined projected permeation/pinhole leakage rate ranges from 9.5 to 138 gal/acre/day. Using a very conservative value of 75 gal/acre/day for the combined leakage/permeation rate (**Attachment II.9.C**), this provides 16 days of storage at a depth of 2 ft in the sump. The rate of 75 gal/acre/day is considered very conservative as it is based on 40 mil HDPE (vs. the actual 60 mil); a fluid depth of 10 ft; and a high number of large pin-holes. Considering that the Landfill leachate collection system is designed to maintain less than 1 ft of liquid on the liner this is and extremely conservative analysis for the Landfill.

The liquid levels in the leak detection sumps will be monitored at least monthly and immediately after the cells or ponds are put into service, and documented. In the event and excessive liquid level [i.e., > corrective action level (ACL)] is observed in a leak detection system, OCD will be notified within 24 hours. If this liquid level is observed in a Landfill cell the Facility will initiate corrective action which may include but is not limited to:

- Additional sump liquid level monitoring and pumping frequencies
- Liquids analytical testing and submittal of results to OCD
- Enhanced vadose zone monitoring (if applicable)

If this liquid level is observed in an evaporation pond, the affected pond area will be drained. Prior to placing the pond back into service, the Facility will initiate corrective action which may include but is not limited to:

- Actions undertaken to locate source of leakage
- Repair procedures
- Additional sump liquid level monitoring and pumping frequencies
- Liquids testing and submittal of results to OCD
- Groundwater monitoring (if required)

Any liquids recovered from the Leak Detection Sump will be disposed of in the same manner as leachate generated from the landfill cells.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME II: LANDFILL MANAGEMENT PLANS
SECTION 9: LEACHATE MANAGEMENT PLAN

ATTACHMENT II.9.A

LEACHATE MONITORING FORM (TYPICAL)

ATTACHMENT II.9.A
Leachate Monitoring Form (Typical)
DNCS Environmental Solutions

Leachate Level Data				Pumping Data			Notes
Date	Sump I.D.	Time	Monitored By	Date	Company	Volume Pumped (gal)	

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME II: LANDFILL MANAGEMENT PLANS
SECTION 9: LEACHATE MANAGEMENT PLAN

ATTACHMENT II.9.B

POND INTEGRITY/LEAK DETECTION INSPECTION FORM (TYPICAL)

ATTACHMENT II.9.B
Pond Integrity/Leak Detection Inspection Form (Typical)
DNCS Environmental Solutions

Date: ______________________ **Inspector(s):**

Time: ______________________ ____

____

Weather:

Temperature ______________________ deg. F Precipitation (last 24 hours) _____________ inches

Skies ______________________

Wind Speed ______________________ mph

Wind Direction ______________________ *(direction blowing from)*

NOTES:
"X" indicates that a Deficiency has been noted. "P" indicates that a Photograph has been taken. "S" indicates that a Sample has been collected. Complete descriptions of Deficiencies, Photographs, and Samples are provided on attached pages. Items are referenced by Location.

Pond Condition

Location	Item			
	Erosion	Vegetation Established	Vectors	Sample

Leak Detection System

Riser #	Deficiency	
	Depth of H_2O	Structural Defect

NOTES: ____

____

____

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME II: LANDFILL MANAGEMENT PLANS
SECTION 9: LEACHATE MANAGEMENT PLAN

ATTACHMENT II.9.C

POTENTIAL GEOMEMBRANE LINER LEAKAGE

Title: Leakage Through Liners Constructed with Geomembranes - Part 1. Geomembrane Liners

Written by: J.P. Giroud and R. Bonaparte
Published in: Geotextiles and Geomembranes **Volume:** 8 **Issue:** 2 **Pages:** 27 to 67
Phone: +31 20-485-3757 ~ **Web Site:** http://www.elsevier.com

How impermeable are 'impermeable liners'? All liners leak, including geomembranes, but how much? What are the mechanisms of leakage through liners constructed with geomembranes? To answer these questions, a detailed review of leakage mechanisms, published and unpublished data, and analytical studies has been carried out with the goal of providing practical design recommendations. In particular, it appears that a composite liner (i.e. geomembrane on low-permeability soil) is more effective in reducing the rate of leakage through the liner than either a geomembrane alone or a soil liner (low-permeability soil layer) alone. However, the paper shows that the effectiveness of composite liners depends on the quality of the contact between the geomembrane and the underlying low-permeability soil layer.

Table 1
Calculated Leakage Rates Due to Pinholes and Holes in a Geomembrane

	Defect Diameter	0.003 m (0.01 ft)	0.03 m (0.1 ft)	0.3 m (1 ft)	3 m (10 ft)	30 m (100 ft)
		Water depth on top of the geomembrane, h_w				
Pinholes	0.1 mm (0.004 in)	0.006 (0.0015)	0.06 (0.015)	0.6 (0.15)	6 (1.5)	60 (15)
	0.3 mm (0.012 in)	0.5 (0.1)	5 (1)	50 (13)	500 (130)	5000 (1 300)
Holes [a]	2 mm (0.08 in)	40 (10)	130 (30)	400 (100)	1300 (300)	4000 (1 000)
	11.3 mm (0.445 in)	1 300 (300)	4 000 (1 000)	13 000 (3 000)	40 000 (10 000)	130 000 (30 000)
Values of leakage rate in liters/day (gallons/day)						

Table 2
Calculated Unitized Leakage Rates Due to Permeation of Water Through an HDPE Geomembrane

	0 m (0 ft)	*0.003 m (0.01 ft)*	*0.03 m (0.1 ft)*	*0.3 m (1 ft)*	*3 m (10 ft)*	*>10 m (>30 ft)*
	Water depth on top of the geomembrane, h_w					
Coefficient of migration, m_g(m^2/s)	0	9×10^{-20}	9×10^{-18}	9×10^{-16}	9×10^{-14}	3×10^{-13}
Unitized leakage rate, q_q (m/s)	0	9×10^{-17}	9×10^{-15}	9×10^{-13}	9×10^{-11}	3×10^{-10}
(lphd)	0	8×10^{-5}	0.008	0.8	80	260
(gpad)	0	8×10^{-6}	0.0008	0.08	8	28

Notes: These values of utilized leakage rates were calculated using eqn (5) and assuming a geomembrane thickness of 1 mm (40 mils). The coefficients of migration used to calculate the unitized leakage rates in this table were obtained from eqns (19) and (20), with $C_1 = 1 \times 10^{-22}$ m^4 $kg^{-2}s^3$, $n = 2$, and $m_{gmax} = 3 \times 10^{-13}$ m^2/s.

The water depths used here correspond to the typical values defined in Section1.3.6. (To use eqn (19), it is necessary to know the pressure difference, Δp. According to eqn (1), water depths, h_w, are approximately equal to hydraulic head differences, Δh, which are related by eqn (12) to pressure differences, Δp.)

- data used in calculations

Home About Advertise Buyer's Guide Calendar Employment Links News Pubs/Tech Docs Resin Specifications Standards Directory

NM1-57

Revised Permit Application

June 2014

Volume 3, Part 1 of 3: Engineering Design and Calculations

STATE OF NEW MEXICO
DIRECTOR OF OIL CONSERVATION DIVISION

IN THE MATTER OF THE APPLICATION OF DNCS PROPERTIES, LLC FOR A SURFACE WASTE MANAGEMENT FACILITY PERMIT

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

NOVEMBER 2013
(UPDATED JUNE 2014)

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS

Prepared For:

DNCS Properties, LLC
2028 E. Hackberry Place
Chandler, AZ 85286
480.437.0044

Submitted To:

New Mexico Energy, Minerals, and Natural Resources Department
Oil Conservation Division
1220 South St. Francis Drive
Santa Fe, NM 87505
505.476.3440

Prepared By:

Gordon Environmental, Inc.
213 S. Camino del Pueblo
Bernalillo, NM 87004
505.867.6990

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

TABLE OF CONTENTS

TABLE OF CONTENTS (cont.)

VOLUME IV: SITING AND HYDROGEOLOGY

LIST OF PERMIT PLANS

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

TABLE OF CONTENTS

LIST OF TABLES

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

LIST OF ATTACHMENTS

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

1.0 INTRODUCTION

DNCS Environmental Solutions (DNCS Facility) is a proposed Surface Waste Management Facility for oil field waste processing and disposal services. The proposed DNCS Facility is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.36 NMAC, administered by the Oil Conservation Division (OCD). The Facility has been designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, DNCS Properties, LLC.

1.1 Description

The DNCS site is comprised of a 562-acre ± tract of land located south of NM 529 in portions of Section 31, Township 17 South, Range 33 East; and in the northern half of Section 6, Township 18 South, Range 33 East, Lea County, NM. A portion of the 562-acre tract is a drainage feature that will be excluded from development. The drainage feature includes a 500-ft setback and totals 67 acres ±. The DNCS Facility will include two main components; a liquid oil field waste Processing Area (177 acres ±), and an oil field waste Landfill (318 acres ±); therefore the DNCS Facility comprises 495 acres ±. Oil field wastes are anticipated to be delivered to the DNCS Facility from oil and gas exploration and production operations in southeastern NM and west Texas. The Site Development Plan provided in the **Permit Plans, Sheet 3**, identifies the locations of the Processing Area and Landfill facilities.

2.0 DESIGN CRITERIA

This Section, “Engineering Design” is provided as a summary of the engineering design elements for the DNCS Landfill and Processing Facility. The Engineering Design has been developed in accordance with the Oil and Gas Rules. More specifically, 19.15.36.17.A

NMAC requires an “Engineering Design Plan” for evaporation, storage, treatment and skimmer ponds. In addition, the construction standards for these facilities are also addressed in compliance with 19.15.36.17.B NMAC. Engineering requirements specific to landfills as referenced in 19.15.36.14.C-F NMAC, including landfill design standards, liner specifications, requirements for the soil component of composite liners, and the leachate collection and removal system are addressed herein. The Engineering Design also addresses the requirements of 19.15.36.13.M NMAC pertaining to the control of run-on and runoff from the 25-year, 24 hour design storm (**Volume III.4** and **Permit Plans, Attachment III.1.A**).

Compliance with the design standards is demonstrated on the **Permit Plans** listed in **Table III.1.1**, which are sealed by Mr. I. Keith Gordon, P.E., of Gordon Environmental, Inc., a New Mexico Professional Engineer with extensive experience in geotechnical engineering and waste containment design employing geosynthetics. The **Permit Plans** are provided for reference in **Attachment III.1.A** as 11 x 17 inch (in.) plots and are also submitted as “D” size sealed plots (i.e., 24 x 36 in.) as part of this Application for Permit.

Table III.1.1
List of Permit Plans
DNCS Environmental Solutions

Sheet No.	Title
1.	Cover Sheet and Drawing Index
2.	Existing Site Conditions
3.	Site Development Plan
4.	Landfill Base Grading Plan
5.	Landfill Final Grading Plan
6.	Landfill Cross Sections
7.	Landfill Completion Drainage Plan
8.	Liner System and Cover Details
9.	Leachate Collection System Details
10.	Stormwater Drainage Details
11.	Processing Area Layout
12.	Evaporation Pond Details
13.	Evaporation Pond and Stabilization/Solidification Area Cross Sections
14.	Processing Area Cross Sections

3.0 LANDFILL DESIGN STANDARDS

The proposed DNCS Landfill will be located within "eastern tract" (318 acres ±) as shown on the **Permit Plans, Sheet 3** (**Attachment III.1.A**). The DNCS Landfill disposal footprint will be approximately 234 acres ± in size with a depth from the top of the 15-foot (ft) perimeter berm to the base grades of approximately 20 ft on the east end and 50 ft on the west end. The base grades of the Landfill are in excess of 100 ft from groundwater. The Landfill consists of nine independent units (Units 1 through 9), each having an independent leachate collection system, cleanout riser, and collection sump located at the west end (**Permit Plans, Sheet 4**).

3.1 Liner System

A double liner and leak detection system design is proposed for the DNCS Landfill. An alternate liner system is being proposed that meets the requirements of 19.15.36.14.C NMAC demonstrated as equivalent in the United States Environmental Protection Agency (USEPA) Hydrologic Evaluation of Landfill Performance (HELP) Model (**Volume III.4)** and has a demonstrated track record for long-term waste containment performance. The liner system consists of, from top to bottom:

- 24-in. protective soil/leachate drainage layer (on-site soils with permeability ≥ 5.2×10^{-4} cm/sec)
- 60-mil HDPE primary liner
- 200-mil HDPE geonet leak detection layer
- 60-mil HDPE secondary liner
- Geosynthetic Clay Liner (GCL)
- 6-in. soil compacted subgrade

The liner system is designed to meet the performance requirement of no more than one foot of leachate on the primary liner as required in 19.15.36.14.F NMAC and demonstrated in the HELP Model (**Volume III.4**).

HDPE material is proposed for the leachate collection layer, leak detection layer and liners as HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to degradation by waste constituents. **Volume III.6** provides documentation regarding HDPE material compatibility in compliance with

19.15.36.14.D.(2)(a) NMAC.

3.2 Leachate Collection and Leak Detection System

The leachate collection system designed for the Landfill consists of an alternate 2-ft protective soil/leachate collection layer consisting of "SM" soil material with a permeability of $\geq 5.2 \times 10^{-4}$ centimeters per second (cm/sec). The leak detection system layer will incorporate a 200-mil geonet specifically prescribed for this application (**Permit Plans)**. With a design transmissivity of 1×10^{-3} square meters per second (m^2/sec), the geonet will provide fluid flow potential superior to the prescriptive soil leak detection layer of 2 ft of pervious soils (19.15.36.14.C.(3) NMAC and 19.15.36.14.C.(5) NMAC). This fact has been demonstrated in the HELP Model (**Volume III.4**).

The leachate collection layer slopes at 2.8% to a 6-in. diameter standard dimension ratio (SDR) 11 high density polyethylene (HDPE or Sch 80 PVC) perforated leachate collection pipe to the center of the units and is directed at a 2% slope to the leachate collection sumps on the west end of the Landfill (**Permit Plans, Sheet 4**). The leak detection geonet slopes at 2.8% to the center of the units and is directed at a 2% slope to each of the nine leak detection sumps located on the west end of the Landfill (**Permit Plans, Sheet 4**). Each of the sumps is approximately 2 ft deep and contains ¾-in. to 2.0-in. diameter pre-qualified select aggregate installed on and wrapped in a geotextile cushion placed over the HDPE liners. Classification criteria for the aggregate are specified in the Liner Construction Quality Assurance (CQA) Plan (**Volume II.7**), which state that it not be angular (i.e., sharp edges which could damage the liners) or calcareous (which could degrade over time).

The fluids collected in the leachate collection and leak detection sumps will be monitored and collected by separate 12-in. diameter sidewall riser pipes, that do not penetrate the liners, in compliance with 19.15.36.14.C.(10) NMAC. The piping is demonstrated to resist degradation by the waste constituents as documented in the Geosynthetic Application and Compatibility Documentation (**Volume III.6**).

The leachate collection system pipe will consist of a minimum 6-in. diameter perforated SDR 11 HDPE. The leachate collection and leak detection sump riser pipes will consist of a 12-in. diameter, SDR 11 HDPE; and will be perforated or slotted for the bottom 2 ft depth within the sump (i.e., 8 ft length at 4:1 slope). HDPE piping has shown superior characteristics for waste containment applications vs. the Schedule (SCH) 80 polyvinylchloride (PVC) specified in the Oil and Gas Rules; and has a greater wall thickness as shown on **Tables III.1.2** and **III.1.3**. The piping is demonstrated to resist degradation by the waste constituents as documented in the Geosynthetic Application and Compatibility Documentation (**Volume III.6**).

TABLE III.1.2
Comparison of 6-in. Diameter PVC and HDPE Leachate Collection Pipe
DNCS Environmental Solutions

Characteristic	**6-in. Diameter Leachate Collection Pipe**	
	Schedule 80	**SDR 11 HDPE**
Dimension Ratio	15.3	11.0
Method of Joining	Gasketed/Glued	Welded
Manning's Number (n)	0.009	0.010
Outside Diameter (in.)	6.625[1]	6.625[2]
Min. Wall Thickness (in.)	0.432[1]	0.602[2]
Tensile Strength (psi)	5,000	5,000
Modulus of Elasticity (psi)	400,000	130,000
Flexural Strength (psi)	14,450	135,000

Notes:
[1]*Handbook of PVC Pipe, pg. 340 (**Attachment III.1.G**)*
[2]*PolyPipe, A-4 (**Attachment III.1.G**)*

TABLE III.1.3
Comparison of 12-in. Diameter PVC and HDPE Sump Riser Pipe
DNCS Environmental Solutions

Characteristic	**12-in. Diameter Leachate and Leak Detection Riser Pipes**	
	Schedule 80	**SDR 11 HDPE**
Dimension Ratio	18.6	11.0
Method of Joining	Gasketed/Glued	Welded
Manning's Number (n)	0.009	0.010
Outside Diameter (in)	12.75[1]	12.75[2]
Min. Wall Thickness (in.)	0.687[1]	1.159[2]
Tensile Strength (psi)	5,000	5,000
Modulus of Elasticity (psi)	400,000	130,000
Flexural Strength (psi)	14,450	135,000

Notes:
[1]*Handbook of PVC Pipe, pg. 340 (**Attachment III.1.G**)*
[2]*PolyPipe, A-4 (**Attachment III.1.G**)*

The details in the **Permit Plans**, **Sheet 10** reflect the deployment of SDR 11 HDPE piping for the leachate collection pipe and leak detection sump riser pipes. HDPE flat stock or four layers of geonet will be placed beneath the beveled edge of the perforated risers in the sumps to prevent potential liner damage (**Permit Plans**). Solid-wall HDPE piping will extend from above the sumps to the permanent wellheads shown on the **Permit Plans**.

The entire leachate collection system will be covered by 2 ft of protective soil with a hydraulic conductivity greater than or equal to $\geq 5.2 \times 10^{-4}$ cm/sec. The HELP Model, provided in **Volume III.4**, confirms that the design meets the requirements of 19.15.36.14.F NMAC.

The leachate collection system and protective soil cover on the top of the liner system in the Landfill will protect the floor and sidewall liner by providing ballast and blocking sunlight (i.e., UV rays), with the upper sections of sidewall liner secured by the anchor trench as depicted on the **Permit Plans**.

3.3 Landfill Final Cover System

The final cover for the top of the Landfill will utilize the prescriptive final cover (defined by 19.15.36.14 (C) (8) NMAC) and consists of the following layers:

- 12-in. soil erosion layer
- 12-in. protection layer
- 12-in. drainage layer (w/saturated hydraulic conductivity $\geq 1 \times 10^{-2}$ cm/sec)
- 60-mil HDPE liner
- 12-in. foundation layer
- Oil Field Waste and soil compacted to 80% Standard Proctor

The sideslopes will utilize an alternative cover system consisting of the following:

- 12-in. erosion layer
- 24-in. infiltration layer
- Oil Field Waste and soil compacted to 80% Standard Proctor

On-site soils will be used to construct the final cover, and the cap will be placed as the Landfill reaches final grades. The Landfill will have 4:1 design sideslopes with drainage benches spaced at a vertical distance of approximately 30-ft; and a top slope of 5%. The final cover (sideslope) was modeled using the HELP Model (**Volume III.4**), and results indicate that percolation through the cover will not exceed that of the bottom liner as required in 19.15.36.14.C.(9) NMAC.

4.0 LANDFILL CONSTRUCTION

Construction of the Landfill will be accomplished by constructing individual cells within the units. Detailed Construction Plans and Technical Specifications will be prepared for the proposed DNCS Landfill cells and submitted to several pre-qualified Liner Installation Contractors for quotes. The cell excavation, construction, floor grading/compaction, and geosynthetics installation will be subject to the rigorous CQA standards specified in the Liner CQA Plan (**Volume II.7**).

OCD will be provided a major milestone schedule in advance of construction; and will be notified via e-mail or phone at least 3 working days prior to the installation of the primary liner. An Engineering Certification Report, sealed by a Professional Engineer with expertise in geotechnical engineering, will be submitted to OCD documenting compliance of completed construction with the Permit, regulatory requirements, industry standards, and the plans and specification.

The Engineering Design, as demonstrated by the Volumetric Calculations (**Volume III.2**) deliberately provides a "sustainable" configuration that does not require the import of off-site soils. The materials equation provides an excess of soils excavated (i.e., cut) and fill for the cover and perimeter berms. The in-situ and on-site fill soil will be pre-qualified in accordance with the CQA Plan (**Volume II.7**). At least one Standard Proctor Density test will be conducted in the laboratory for each 5,000 cubic yards of subgrade soils, fill material or a change in subgrade material. These tests will be the basis for field density measurements during construction (i.e., 90% standard Proctor dry density) conducted at a minimum frequency of 4 tests/acre/lift.

Fill for the berms will be placed in horizontal compacted lifts that do not exceed 12-in. in thickness. The subgrade surface will be inspected to confirm the absence of any deleterious materials, abrupt changes in slope, evidence of erosion, etc. The compliance of the completed subgrade construction will be confirmed prior to secondary liner installation, and documented in the Engineering Certification Report.

The 60-mil HDPE secondary liner will be installed for the proposed Cells in direct contact with the prepared and certified subgrade liner in accordance with the CQA Plan (**Volume II.7**). Installation of the geonet; geotextile, aggregate and riser pipes in the sumps will follow. The installation of all soil and geosynthetic components will meet or exceed the requirements of 19.15.36.14.C NMAC, as detailed in the CQA Plan. Finally, the primary liner will be constructed, and liner/leak detection/leachate collection system elements (i.e., secondary, geonet, primary) will be secured in the common anchor trench at the top of the Landfill sideslope**.** The anchor trench will be carefully backfilled with select on-site soils compacted to 90% of standard Proctor dry density by mechanical and/or hand-tamping devices as required by the CQA Plan. Documentation will be provided in the Engineering Certification Report submitted to OCD upon completion of construction.

5.0 POND DESIGN STANDARDS

The designs for the Ponds are identical, except that Pond elevations are different depending on their site location (**Permit Plans, Sheets 12** and **13; Attachment III.1.A**). Each pond is approximately 420 ft east-west by 200 ft north-south as measured at the top of the surrounding berms, for a footprint of 2.0 ± acres each. The floor of the ponds is designed with a 2% slope to facilitate drainage in the leak detection system to the two sumps in each basin situated on the interior sidewall.

Because the berms have a uniform top elevation, the 2% floor slope creates a pond depth that ranges from a maximum of 12 ft to a minimum of just less than 8 ft. The maximum water depth occurs at the sump locations and does not exceed 8.5 ft. Maintaining a high water elevation of 3,966 ft in the Phase I Ponds; 3,965.5 ft in the Phase III Ponds; and 3,965 ft in the Phase IV Ponds; will provide a freeboard in excess of 3.5 ft in each pond. This is more than adequate to meet the 3 ft minimum freeboard standard; while also accommodating the minimal impact potential of rainfall or wave action (**Volume III.12**). The resultant capacity of each pond is approximately 9.5 acre-ft, not including freeboard, below the maximum 10 acre-ft volume prescribed by 19.15.36.17.B(12) NMAC.

Section 5.0 (Pond Construction) below and the CQA Plan (**Volume II.7**) provide documentation on the installation of berms, soil subgrade, and geosynthetics. Exceeding the standards specified in 19.15.36.17.B(4) NMAC, both the exterior and interior sidewalls of all of the Ponds have design slopes of 3:1. The top platform of the berms surrounding the Ponds has a minimum design width of 10 ft, which is more than adequate for the 2 ft anchor trench shown on the **Permit Plans**; and to accommodate pipe risers.

5.1 Liner System

A double liner and leak detection system design is proposed for each pond. An alternate liner system is being proposed that meets the requirements of 19.15.36.17.B(9) NMAC and has a demonstrated track record for long-term waste containment performance. The pond liner system consists of, from top to bottom:

- 60-mil HDPE primary liner
- 200-mil HDPE geonet leak detection layer
- 60-mil HDPE secondary liner
- GCL under the leak detection sumps
- 6-in. compacted soil subgrade

HDPE material is proposed for the liners and leak detection layer as HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to degradation by waste constituents. **Volume III.6** provides documentation regarding HDPE material compatibility in compliance with 19.15.36.17.B(3) NMAC

5.2 Leak Detection System

The leak detection system layer designed for the ponds consists of a 200-mil geonet specifically prescribed for these applications (**Permit Plans**). With a design transmissivity of 1 x 10^{-3} m^2/sec, the geonet will provide fluid flow potential superior to the prescriptive leak detection layer of 2 ft of pervious soils (19.15.36.17.B(9) NMAC).

The underlying 60-mil HDPE secondary liner, the 200-mil geonet leak detection layer, and the overlaying 60-mil HDPE primary liner, will slope at 2% to the 2 leak detection sumps located in each pond (**Permit Plans**). Fluids collected in the leak detection layer, which encompasses the entire footprint for each pond, are directed with the 2% slope to the leak detection sumps. Each of the sumps will be approximately 2 ft deep, as measured from the secondary liner to the primary liner. The sumps will contain ¾-in. to 2.0-in. diameter pre-qualified select aggregate installed on a geotextile cushion placed over the secondary liner. Classification criteria for the aggregate are specified in the CQA Plan (**Volume II.7**), which state that it not be angular (i.e., sharp edges which could damage the liners) or calcareous (which could degrade over time).

The fluids collected in the leak detection sumps will be monitored and removed through a 6-in. diameter, SDR 11 HDPE sidewall riser pipes that do not penetrate the liners. The leak detection sump riser pipes will be perforated or slotted for the bottom 2 ft depth within the sump (i.e., 6 ft length at 3:1 slope). HDPE piping has shown superior characteristics for waste containment applications vs. the SCH 80 PVC specified in the Oil and Gas Rules; and has a greater wall thickness as shown on **Table III.1.4**. The piping is demonstrated to resist degradation by the waste constituents as documented in **Volume III.6**.

TABLE III.1.4
Comparison of 6-in. Diameter PVC and HDPE Sump Riser Pipe
DNCS Environmental Solutions

Characteristic	6-in. Diameter Leak Detection Riser Pipes	
	Schedule 80	SDR 11 HDPE
Dimension Ratio	15.3	11.0
Method of Joining	Gasketed/Glued	Welded
Manning's Number (n)	0.009	0.010
Outside Diameter (in.)	6.625[1]	6.625[2]
Min. Wall Thickness (in.)	0.432[1]	0.602[2]
Tensile Strength (psi)	5,000	5,000
Modulus of Elasticity (psi)	400,000	130,000
Flexural Strength (psi)	14,450	135,000

Notes:
[1]*Handbook of PVC Pipe, pg. 340 (**Attachment III.1.G**)*
[2]*PolyPipe, A-4 (**Attachment III.1.G**)*

The details in the **Permit Plans** reflect the deployment of SDR 11 HDPE piping for the leak detection sump riser pipes. HDPE flat stock or four layers of geonet will be placed beneath the beveled edge of the perforated risers in the sumps to prevent potential liner damage (**Permit Plans**). Solid-wall HDPE piping will extend from above the sumps to the permanent wellheads shown on **Permit Plans**. The sidewall liners and leak detection geonet will be secured by the anchor trench as depicted on the **Permit Plans.**

6.0 POND CONSTRUCTION

Detailed Construction Plans and Technical Specifications will be prepared for the proposed Ponds, and submitted to several pre-qualified Liner Installation Contractors for quotes. The berm construction, floor grading/compaction, and geosynthetics installation will be subject to the rigorous CQA standards specified in **Volume II.7**.

OCD will be provided a major milestone schedule in advance of construction; and notified via email or phone at least 3 working days prior to the installation of the primary liner in compliance with 19.15.36.17.B(10) NMAC. An Engineering Certification Report, sealed by a Professional Engineer with expertise in geotechnical engineering, will be submitted to OCD documenting compliance of completed construction with the Permit, regulatory requirements, industry standards, and the plans and specification.

The Engineering Design presented on the **Permit Plans** (**Attachment III.1.A**) deliberately provides a "sustainable" configuration that does not require import of off-site soils. The materials equation provides a balance between soils excavation (i.e., pond) and fill for the sidewalls. The in-situ and on-site fill soil will be pre-qualified in accordance with the CQA Plan (**Volume II.7**). At least one standard Proctor dry density test will be conducted in the laboratory for each pond footprint, 5,000 cubic yards (cy) of fill material for berms, or change in subgrade material. These tests will be the basis for field density measurements during construction (i.e., 90% standard Proctor dry density) conducted at a minimum frequency of 4 tests/acre/lift.

Fill for the berms will be placed in horizontal compacted lifts that do not exceed 12 in. in thickness. The subgrade surface will be inspected to confirm the absence of any deleterious materials, abrupt changes in slope, evidence of erosion, etc. The compliance of the completed subgrade construction shall be confirmed prior to secondary liner installation, and documented in the Engineering Certification Report.

The double liner and leak detection system design, planned for the ponds, consists of proven technology with a demonstrated track record of long-term waste containment performance. The secondary liner proposed for the ponds, consists of a smooth 60-mil HDPE geomembrane placed in direct contact with a prepared and compacted soil subgrade, certified in accordance with the CQA Plan (**Volume II.7**). The same HDPE material will be used for the primary liner and the geonet for the leak detection layer. HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to attack by waste constituents.

Volume III.6 provides documentation regarding liner and leak detection material compatibility in compliance with 19.15.36.17.B(3) NMAC. An additional layer of 60-mil HDPE (22.5 ft x 40 ft ±) will be welded above the primary Pond liner where active wastewater discharge will occur (**Permit Plans**). This will protect the Pond liner from excessive hydrostatic force or mechanical damage. External discharge lines and leak detection system discharge lines will not penetrate the liner. The CQA Plan (**Volume II.7**) provides the most current technical specifications for the geosynthetics.

Fluid in the Ponds will protect the floor and lower sidewall liner by providing ballast and deflecting sunlight (i.e., UV rays). The upper sections of pond sidewall liner will be secured by the anchor trench. The anchor trench will be carefully backfilled with select on-site soils compacted to 90% of standard Proctor dry density by mechanical and/or hand-tamping devices (per the CQA Plan). Documentation will be provided in the Engineering Certification Report submitted to OCD upon completion of construction.

Although the freeboard zone of the pond sidewall liner will be exposed to the elements, recent research indicates that exposed HDPE in similar environments has a functional longevity in excess of 25 years (**Attachment III.1.B**). GEI has inspected several similar water storage ponds in New Mexico and has found exposed geomembrane liners to be functionally intact after over 25 years.

7.0 POND OPERATION

Detailed plans for the operation of the Ponds are prescribed in the Operations, Maintenance, and Inspection Plan (**Volume II.1**). Essentially, it is anticipated that some fluids will accumulate in the leak detection sumps as a result of condensation, construction water, etc. As described in **Volume II.1**, the leak detection sumps will be monitored at least monthly for the presence of fluids, which may be extracted and tested when the level in the sump(s) exceeds 24 in. A reduced monitoring frequency may be proposed to OCD dependent upon historical results. The design of the Ponds allows for isolation of potential leaks into isolated drainage basins, facilitating necessary evaluation or repair by allowing each pond to be emptied.

8.0 PROCESS AREA TANK CONTAINMENT

As proposed in this Application, produced water receiving tanks, produced water settling tanks, and the crude oil receiving tanks depicted in **Attachment III.1.C** and oil sales tanks as depicted in **Attachment III.1.D** will be installed in the excavated tank farm as shown on the **Permit Plans**. Detailed operations of the tanks are described in the Operations, Maintenance, and Inspection Plan (**Volume II.1**), and a schematic of the process area is provided in **Attachment III.1.E**. The tanks will be constructed with an underlying, continuous, system which is designed to capture any fluids within the watershed of the tank farm.

The secondary containment liner in the tank area is a 30-mil polyester liner (XR-5 8130 Reinforced Geomembrane). The use of the XR-5 8130 Reinforced Geomembrane in the tank area is primarily based on the chemical compatibility and puncture resistance of the material compared to either PVC or HDPE material. The chemical resistance of the XR-5 material exceeds the chemical compatibility of either PVC or HDPE to hydrocarbon products (see Chemical Resistance Chart, Page 13, "Technical Data and Specifications for XR-5", **Attachment III.1.H**). Since PVC material has marginal chemical resistance in a hydrocarbon environment, physical properties of the XR-5 geomembrane (**Attachment III.1.H**) are compared to 60-mil HDPE geomembrane (**Attachment III.1.I**) as shown in **Table III.1.5**:

TABLE III.1.5
Physical Properties: XR-5 8130 Reinforced Geomembrane
and 60-mil HDPE Geomembrane
DNCS Environmental Solutions

Property	XR-5 8130	60-mil HDPE
Thickness	30-mil	60-mil
Tear Strength	40 lbs	42 lbs
Puncture Resistance	275 lbs	108 lbs
Break Strength	400 lbs/in.	228 lbs/in.
Break Elongation	25%	700%
Hydrostatic Resistance	800 psi	> 450 psi
Hydraulic Conductivity	1×10^{-12} cm/sec	2×10^{-13} cm/sec
Seam Properties		
Shear Strength	500 lbs	120 lbs/in.
Peel Strength	40 lbs/2 in.	91 lbs/in.

The necessary storage capacity for the interconnected tank/containment system will be sufficiently managed by the proposed lined volume of the Ponds. In the unlikely event of a total failure of all affected storage units, the contents of the tanks will flow into the ponds, which have a lined storage capacity of 884,400 barrels (bbl) ± (excluding freeboard). When the freeboard is included, the storage capacity of the ponds is over 1,714,600 bbl, which results in a net surplus of over 830,200 bbl. The entire volume of the proposed receiving tanks will be 70,000 bbl, providing a net excess capacity of over 760,200 bbl. Thus, the Ponds will hold the entire volume of the receiving/settling tanks within the required permanent freeboard of 3 ft.

The maximum proposed number of interconnected tanks is five 1,000 bbl tanks for a total of 5,000 bbl. Allowing for an additional 30% capacity will require a minimum of 6,500 bbl of bermed capacity in the tank farm. The containment area is conservatively sized to surround the entire tank farm, which results in a holding capacity of 13,100 bbl, and is 12,100 bbl greater than the capacity of the largest tank (1,000 bbl) and 6,600 bbl greater than the combined connected tank volume, including a 30% factor of safety within the containment area. Therefore the containment area surrounding the receiving/settling tanks is more than sufficient. Included in this Section is a spreadsheet (**Attachment III.1.F**), that identifies all of the proposed tanks and Evaporation Ponds in this Application.

9.0 STABILIZATION AND SOLIDIFICATION AREA

The design for the stabilization and solidification (S&S) area relies on many of the Pond design characteristics, except that the S&S area is designed to allow dump trucks and tanker trucks delivering materials that require stabilization and/or solidification to discharge directly into the S&S area from a concrete unloading pad. (**Attachment III.1.A**). The S&S area covers approximately 5-acres and measures 660 ft east-west by 330 ft north-south at the top of the surrounding berms. The floor of this area is designed with a 2% slope to facilitate drainage on the liner and in the leak detection system to collect in a sump situated along the east sidewall of the area.

Because the three perimeter berms have a uniform top elevation, the 2% floor slope creates a pond depth that ranges from a minimum of 5 ft at the unloading pad to a maximum of 20 ft at the sump along the eastern perimeter berm. The bottom liner slope allows for a 5-ft-thick protective and operational cover on the liner. This slope also provides operation capacity for the S&S function proposed for this area while providing the capacity to meet the 3 ft minimum freeboard standard and accommodating the minimal impact potential of rainfall. The resultant capacity of the S&S area is approximately 5.6 acre-ft, not including freeboard, well below the maximum 10 acre-ft volume prescribed by 19.15.36.17.B(12) NMAC.

Section 5.0 (Pond Construction) and the CQA Plan (**Volume II.7**) provide documentation on the installation of berms, soil subgrade, and geosynthetics. Exceeding the standards specified in 19.15.36.17.B(4) NMAC, both the exterior and interior sidewalls of S&S area have design slopes of 3:1. The top platform of the berms surrounding the S&S area has a minimum design width of 10 ft, which is more than adequate for the 2 ft anchor trench.

9.1 Liner System

As with the Ponds, the S&S area is designed with a double liner and leak detection system proposing the same alternate liner system that meets the requirements of 19.15.36.17.B(9) NMAC and has a demonstrated track record for long-term waste containment performance. The S&S Area liner system consists of, from top to bottom:

- 5 ft protective soil and operational layer
- 60-mil HDPE primary liner
- 200-mil HDPE geonet leak detection layer
- 60-mil HDPE secondary liner
- GCL under the leak detection sumps
- 6-in. compacted soil subgrade

HDPE material is proposed for the liners and leak detection layer as HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to attack by waste constituents. **Volume III.6** provides documentation regarding HDPE material compatibility in compliance with 19.15.36.17.B(3) NMAC

9.2 Leak Detection System

The leak detection system layer designed for the S&S area consists of a 200-mil geonet specifically prescribed for these applications. With a design transmissivity of 1×10^{-3} m^2/sec, the geonet will provide fluid flow potential superior to the prescriptive leak detection layer of 2 ft of pervious soils (19.15.36.17.B(9) NMAC).

The underlying 60-mil HDPE secondary liner, the 200-mil geonet leak detection layer, and the overlaying 60-mil HDPE primary liner, will slope at 2% to the leak detection sump located on the eastern berm of the S&S area. Fluids collected in the leak detection layer, which encompasses the entire footprint of the S&S area, are directed with the 2% slope to the leak detection sump. This sump will be approximately 2 ft deep, as measured from the secondary liner to the primary liner. The sump will contain ¾-in. to 2.0-in. diameter pre-qualified select aggregate installed on a geotextile cushion placed over the secondary liner. Classification criteria for the aggregate are specified in the CQA Plan (**Volume II.7**), which state that it not be angular (i.e., sharp edges which could damage the liners) or calcareous (which could degrade over time).

The fluids collected in the leak detection sump will be monitored and removed through a 12-in. diameter, SDR 11 HDPE sidewall riser pipe that does not penetrate the liners. The leak detection sump riser pipe will be perforated or slotted for the bottom 2 ft depth within the sump (i.e., 6 ft length at 3:1 slope). HDPE piping has shown superior characteristics for waste containment applications vs. the SCH 80 PVC specified in the OCD standards; and has a greater wall thickness as shown on **Table III.1.4**. The piping is demonstrated to resist degradation by the waste constituents as documented in **Volume III.6**. The details in the **Permit Plans** reflect the deployment of SDR 11 HDPE piping for the leak detection sump riser pipe.

HDPE flat stock or four layers of geonet will be placed beneath the beveled edge of the perforated riser in the sump to prevent potential liner damage. Solid-wall HDPE piping will extend from above the sump to the permanent wellhead shown on the **Permit Plans**. The sidewall liners and leak detection geonet will be secured by the anchor trench as depicted on the **Permit Plans**.

9.3 Stabilization & Solidification Area Construction

Detailed Construction Plans and Technical Specifications will be prepared for the proposed S&S area, and submitted to several pre-qualified Liner Installation Contractors for quotes. The berm construction, floor grading/compaction, and geosynthetics installation will be subject to the rigorous CQA standards specified in **Volume II.7**.

OCD will be provided a major milestone schedule in advance of construction; and notified via email or phone at least 3 working days prior to the installation of the primary liner in compliance with 19.15.36.17.B(10) NMAC. An Engineering Certification Report, sealed by a Professional Engineer with expertise in geotechnical engineering, will be submitted to OCD documenting compliance of completed construction with the Permit, regulatory requirements, industry standards, and the plans and specification.

The Engineering Design presented on the **Permit Plans** (**Attachment III.1.A**) deliberately provides a "sustainable" configuration that does not require import of off-site soils. The materials equation provides a balance between soils excavation (i.e., S&S area) and fill for the sidewalls. The in-situ and on-site fill soil will be pre-qualified in accordance with the CQA Plan (**Volume II.7**). At least one standard Proctor dry density test will be conducted in the laboratory for the S&S area footprint, 5,000 cubic yard (cy) of fill material for berms, or change in subgrade material. These tests will be the basis for field density measurements during construction (i.e., 90% standard Proctor dry density) conducted at a minimum frequency of 4 tests/acre/lift.

Fill for the berms will be placed in horizontal compacted lifts that do not exceed 12 in. in thickness. The subgrade surface will be inspected to confirm the absence of any deleterious materials, abrupt changes in slope, evidence of erosion, etc. The compliance of the completed subgrade construction shall be confirmed prior to secondary liner installation, and documented in the Engineering Certification Report.

The double liner and leak detection system design planned for the S&S area consists of proven technology with a demonstrated track record of long-term waste containment performance. The secondary liner proposed for the area, consists of a smooth 60-mil HDPE geomembrane placed in direct contact with a prepared and compacted soil subgrade, certified in accordance with the CQA Plan (**Volume II.7**). The same HDPE material will be used for the primary liner and the geonet for the leak detection layer. HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to attack by waste constituents. **Volume III.6** provides documentation regarding liner and leak detection material compatibility in compliance with 19.15.36.17.B(3) NMAC. Leak detection system discharge lines will not penetrate the liner. The CQA Plan (**Volume II.7**) provides the most current technical specifications for the geosynthetics.

Protective cover in the S&S area will protect the floor and lower sidewall liner by providing ballast and deflecting sunlight (i.e., UV rays). The upper sections of S&S area sidewall liner will be secured by the anchor trench (**Permit Plans**). The anchor trench will be carefully backfilled with select on-site soils compacted to 90% of standard Proctor dry density by mechanical and/or hand-tamping devices (per the CQA Plan). Documentation will be provided in the Engineering Certification Report submitted to OCD upon completion of construction.

Although the freeboard zone of the S&S area sidewall liner will be exposed to the elements, recent research indicates that exposed HDPE in similar environments has a functional longevity in excess of 25 years (**Attachment III.1.B**). GEI has inspected similar applications in New Mexico and has found exposed geomembrane liners to be functionally intact after over 25 years.

9.4 Stabilization and Solidification Area Operation

Detailed plans for the operation of the S&S area are prescribed in the Operations, Maintenance, and Inspection Plan (**Volume II.1**). To ensure compliance with the capacity limits imposed on the operation of this area, volumes in and out of this area will be tracked to document the volume in processing at any time. Equipment operating within the S&S area may be equipped with Global Positioning System (GPS) equipment (see **Attachment III.1.J** for information on the Computer Aided Earthmoving System provided by Caterpillar) to monitor the location of the equipment relative to the liner system. This system may be implemented to maintain adequate separation of equipment and the liner system during the stabilization and solidification operation. Material that has completed the S&S operation will be relocated to the Landfill for disposal. Solidification material will be excavated from borrow sources within the solid waste management facility.

10. FACILITY DRAINAGE DESIGN

The **Permit Plans**, **Attachment III.1.A,** show the stormwater management systems that will be employed to manage both run-on and runoff for the DNCS Landfill and Processing Facilities. The design event, pursuant to 19.15.36.13.M NMAC (i.e., 25-year, 24 hour storm) will be managed by a series of drainageways that surround the proposed Ponds, Processes, and Landfill and capture stormwater from other on-site areas.

Stormwater detention basins are planned for installation as shown on the **Permit Plans**; and the Stormwater Management Plan is included in **Volume III.3** that demonstrates the efficacy of the proposed system.

The berms surrounding the Landfill and processing area have a maximum exterior slope of 3:1, and an average height of less than 10 ft, minimizing the potential for soil erosion. The drainageways and detention basins will be regularly inspected and cleaned out, as necessary.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.A

PERMIT PLANS

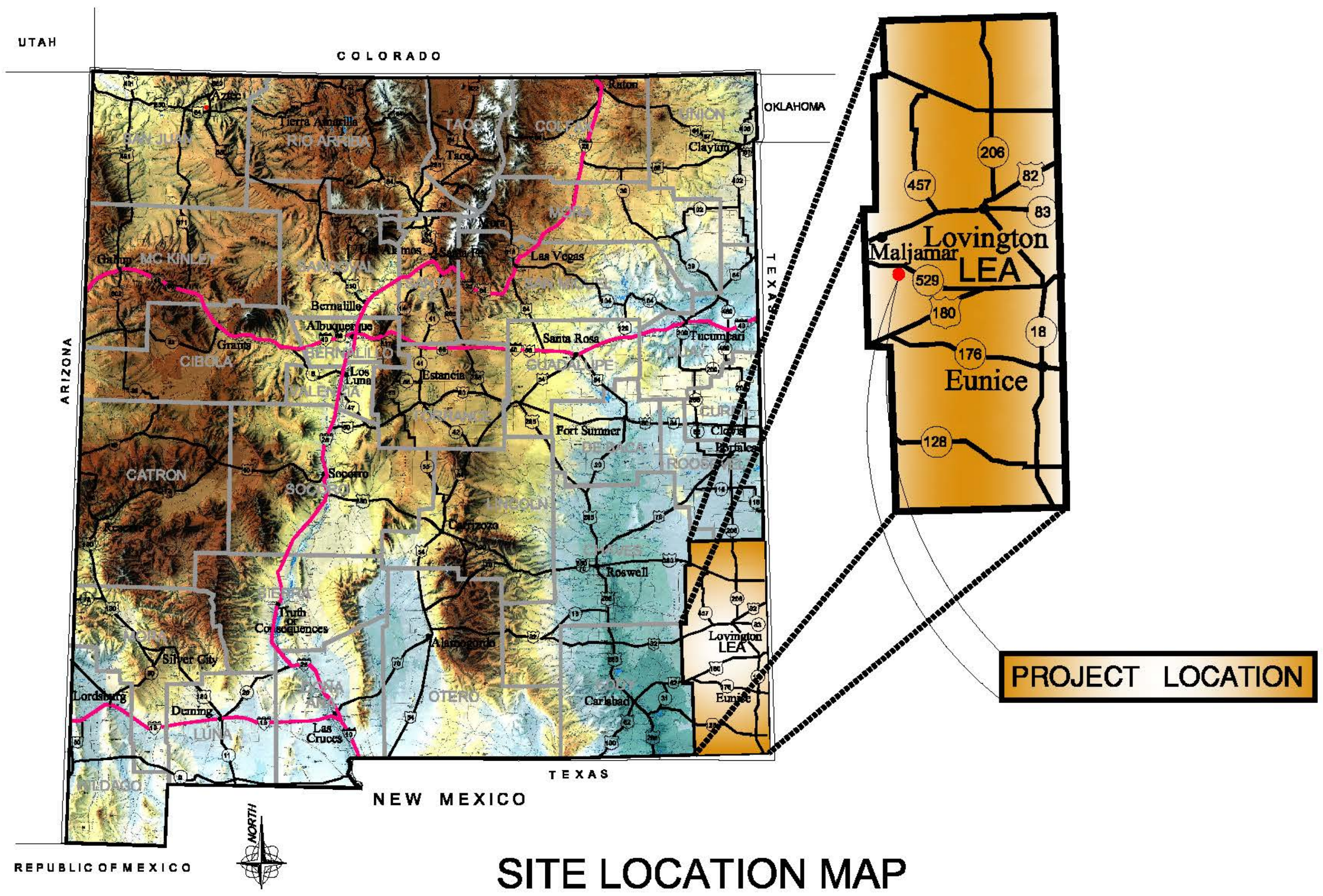
Sheet No.	Title
1.	Cover Sheet and Drawing Index
2.	Existing Site Conditions
3.	Site Development Plan
4.	Landfill Base Grading Plan
5.	Landfill Final Grading Plan
6.	Landfill Cross Sections
7.	Landfill Completion Drainage Plan
8.	Liner System and Cover Details
9.	Leachate Collection System Details
10.	Stormwater Drainage Details
11.	Processing Area Layout
12.	Evaporation Pond Details
13.	Evaporation Pond and Stabilization/Solidification Area Cross Sections
14.	Processing Area Cross Sections

PERMIT PLANS

FOR

DNCS ENVIRONMENTAL SOLUTIONS

LEA COUNTY, NEW MEXICO



SITE LOCATION MAP

SHEET	TITLE
01 COVER.DWG	1 COVER SHEET AND DRAWING INDEX
02 EXIST.DWG	2 EXISTING SITE CONDITIONS
03 SITE DEV.DWG	3 SITE DEVELOPMENT PLAN
04 BASE GRADES.DWG	4 LANDFILL BASE GRADING PLAN
05 FINAL GRADING.DWG	5 LANDFILL FINAL GRADING PLAN
06 X-SECTIONS.DWG	6 LANDFILL CROSS SECTIONS
07 DRAINAGE PLAN.DWG	7 LANDFILL COMPLETION DRAINAGE PLAN
08 LINER DET.DWG	8 LINER SYSTEM AND COVER DETAILS
09 LEACHATE DET.DWG	9 LEACHATE COLLECTION SYSTEM DETAILS
10 STORMWATER DET.DWG	10 STORMWATER DRAINAGE DETAILS
11 PROCESS AREA.DWG	11 PROCESSING AREA LAYOUT
12 EVAP POND DET.DWG	12 EVAPORATION POND DETAILS
13 EVAP X-SECT.DWG	13 EVAPORATION POND AND STABILIZATION/SOLIDIFICATION AREA CROSS SECTIONS
14 PROCESS X-SECT.DWG	14 PROCESSING AREA LAYOUT CROSS SECTIONS

NOT FOR CONSTRUCTION
Drawing:P:\acad 2008\542.01.01\PERMIT PLANS\01 COVER.dwg
Date/Time:Jun. 13, 2014-12:52:52

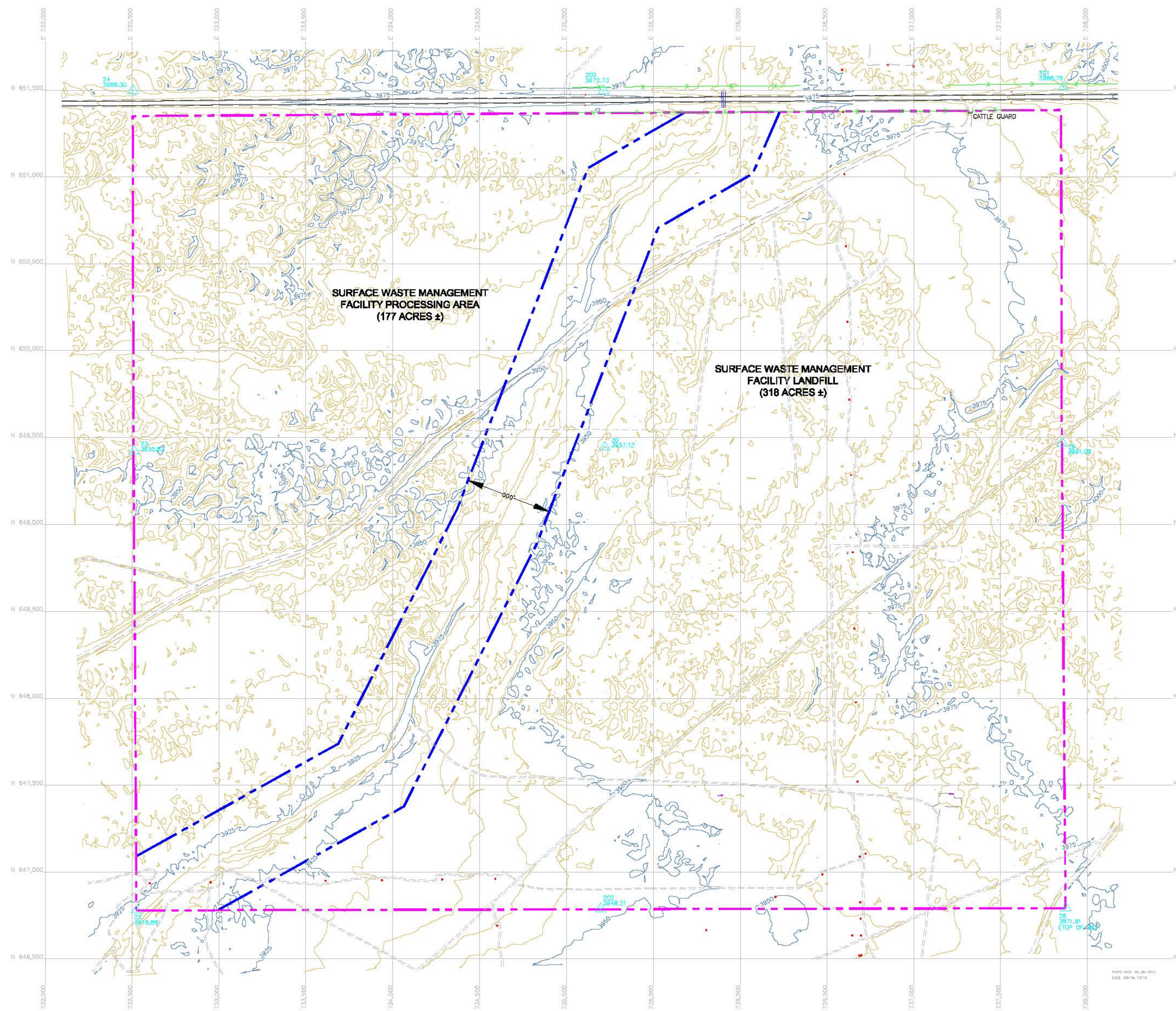
I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

COVER SHEET AND DRAWING INDEX

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/21/2013	CAD: 01 COVER.dwg	PROJECT #: 542.01.01
DRAWN BY: DM	REVIEWED BY: MRH	SHEET 1 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	



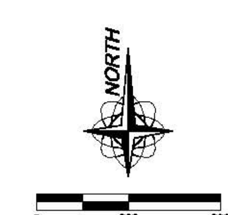
LEGEND

- SITE BOUNDARY (562 ACRES±)
- DRAINAGE FEATURE SETBACK (67 ACRES±)
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- EXISTING FENCE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- POWER POLE
- CULVERT
- CATTLE GUARD
- ROAD SIGN
- ABANDONED WELL
- SURVEY CONTROL POINT
- SITE GRID

SURVEY CONTROL POINT DATA			
POINT	NORTHING	EASTING	ELEVATION
22	646780.31	732525.87	3918.86
23	649422.09	732509.41	3955.82
24	651498.31	732504.10	3968.30
28	646793.35	737874.03	3971.91
29	649469.84	737853.32	3991.09
30	649446.48	735220.56	3957.12
200	651498.13	735212.57	3972.73
201	651518.82	737859.97	3988.76
202	646789.93	735196.38	3948.21

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).



I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

All reports, drawings, specifications, computer files, field data, notes and other documents and instruments prepared by the Engineer as instruments of service shall remain the property of the Engineer. The Engineer shall retain all common law, statutory and other reserved rights, including the copyright thereto.

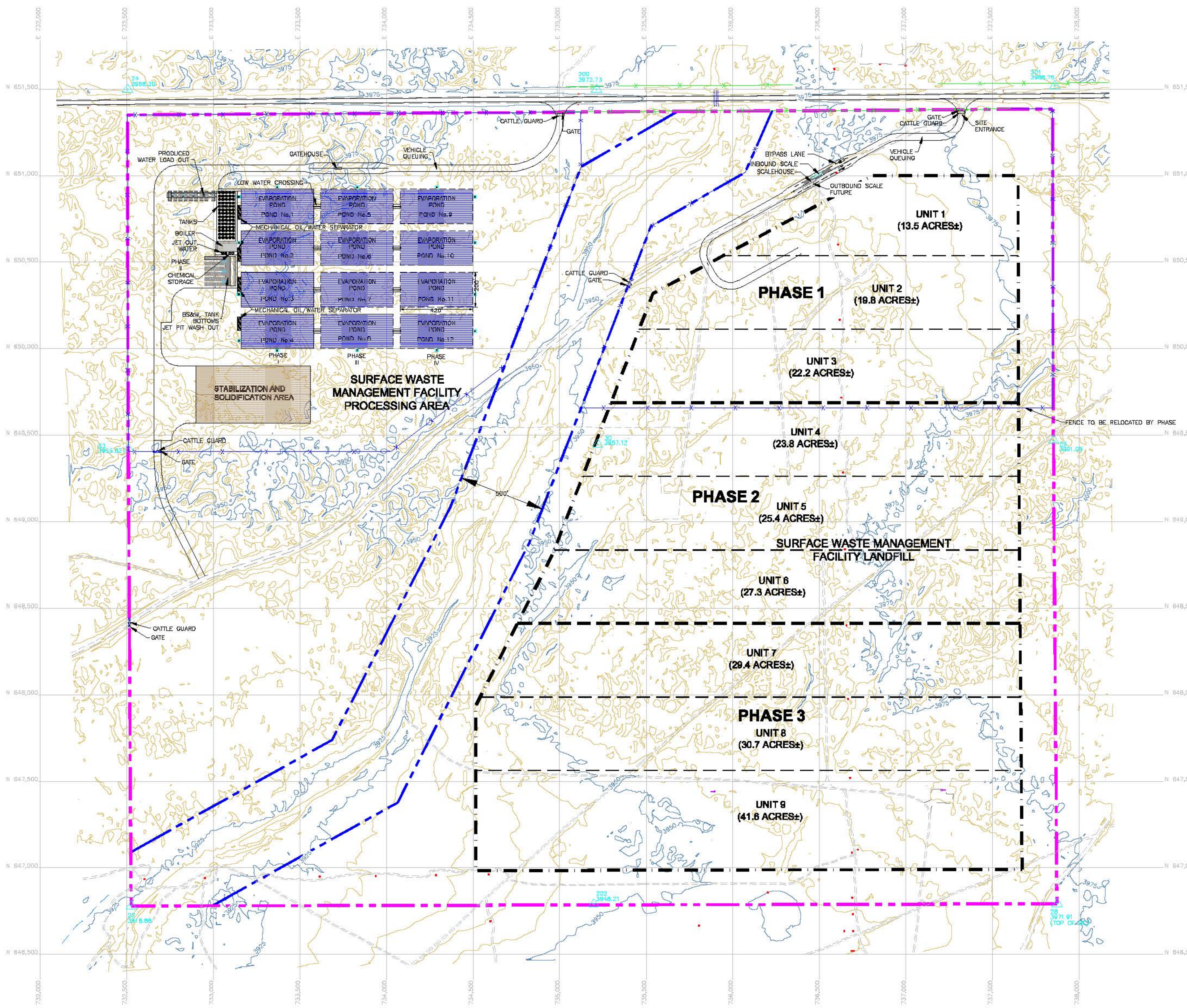
EXISTING SITE CONDITIONS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/21/2013	CAD: 02 EXIST.DWG	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	SHEET 2 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

NOT FOR CONSTRUCTION
Drawing:P:\acad 2008\542.01.01\PERMIT PLANS\02 EXIST.dwg
Date/Time:Jun. 13, 2014-12:58:39 ; LAYOUT: D (L5)



LEGEND

- SITE BOUNDARY (562 ACRES±)
- DRAINAGE FEATURE SETBACK (67 ACRES±)
- LIMIT OF WASTE
- LANDFILL PHASE BOUNDARY
- LANDFILL UNIT BOUNDARY
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- EXISTING FENCE
- PROPOSED FENCE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- POWER POLE (TO BE RELOCATED IN ADVANCE OF CONSTRUCTION)
- EXISTING CULVERT
- CATTLE GUARD
- HYDROGEN SULFIDE MONITORING STATION
- ROAD SIGN
- ABANDONED WELL
- SURVEY CONTROL POINT
- SITE GRID

SURVEY CONTROL POINT DATA

POINT	NORTHING	EASTING	ELEVATION
22	646780.31	732525.87	3918.86
23	649422.09	732509.41	3955.82
24	651498.31	732504.10	3968.30
28	646793.35	737874.03	3971.91
29	649469.84	737853.32	3991.09
30	649446.48	735220.56	3957.12
200	651498.13	735212.57	3972.73
201	651518.82	737859.97	3988.76
202	646789.93	735196.38	3948.21

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

NORTH

0 300 600

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

All reports, drawings, specifications, computer files, field data, notes and other documents and instruments prepared by the Engineer as instruments of service shall remain the property of the Engineer. The Engineer shall retain all common law, statutory and other reserved rights, including the copyright thereto.

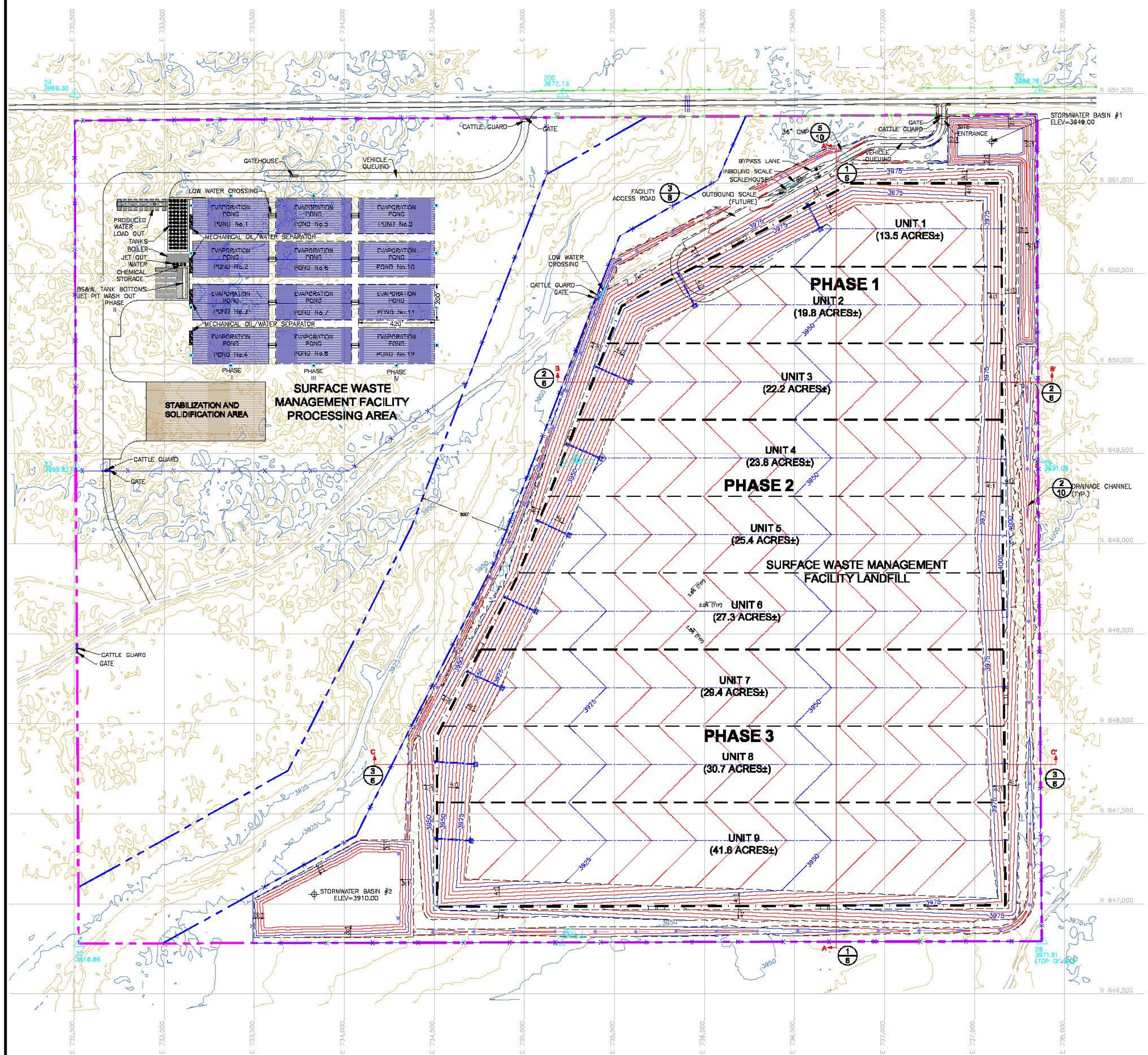
SITE DEVELOPMENT PLAN

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/10/2014	CAD: 03 SITE DEV.DWG	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	SHEET 3 of 14
APPROVED BY: IKG	gel@gordonenvironmental.com	

NOT FOR CONSTRUCTION
Drawing:P:\acad 2009\542.01.01\RAI 1\PERMIT PLAN SHEETS\03 SITE DEV.dwg
Date/Time:Jun. 13, 2014-11:55:33 ; LAYOUT: D (L8)



LEGEND

- SITE BOUNDARY (562 ACRES±)
- WATER FEATURE SETBACK (67 ACRES±)
- LIMIT OF WASTE
- LANDFILL PHASE BOUNDARY
- LANDFILL UNIT BOUNDARY
- EXISTING FENCE
- PROPOSED FENCE
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- 25' DESIGN CONTOUR
- 5' DESIGN CONTOUR
- TOP/TOE OF SLOPE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- DIRECTION OF STORMWATER FLOW
- LEACHATE COLLECTION SUMP & EXTRACTION RISER PIPES
- SURVEY CONTROL POINT
- EXISTING CULVERT
- NEW CULVERT
- HYDROGEN SULFIDE MONITORING STATION
- PPE AND EMERGENCY EQUIPEMENT
- ROAD SIGN
- SITE GRID
- CROSS SECTION LOCATION
- DETAIL NUMBER
- SHEET NUMBER

SURVEY CONTROL POINT DATA

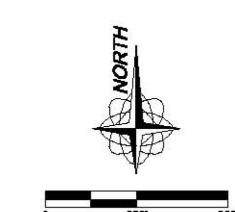
POINT	NORTHING	EASTING	ELEVATION
22	646780.31	732525.87	3918.86
23	649420.79	732507.95	3955.82
24	651497.01	732502.64	3968.19
28	646792.06	737872.55	3971.24
29	649468.54	737851.84	3991.09
30	649445.19	735219.09	3957.12
200	651498.13	735212.57	3972.73
201	651518.82	737859.97	3988.76
202	646789.93	735196.38	3948.21

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

LANDFILL EXCAVATION AND
PERIMETER BERM FILL VOLUMES

CUT VOLUME: 6257969 CUBIC YARDS
FILL VOLUME: 646225 CUBIC YARDS
NET VOLUME: 5611744 CUBIC YARDS <CUT>



I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

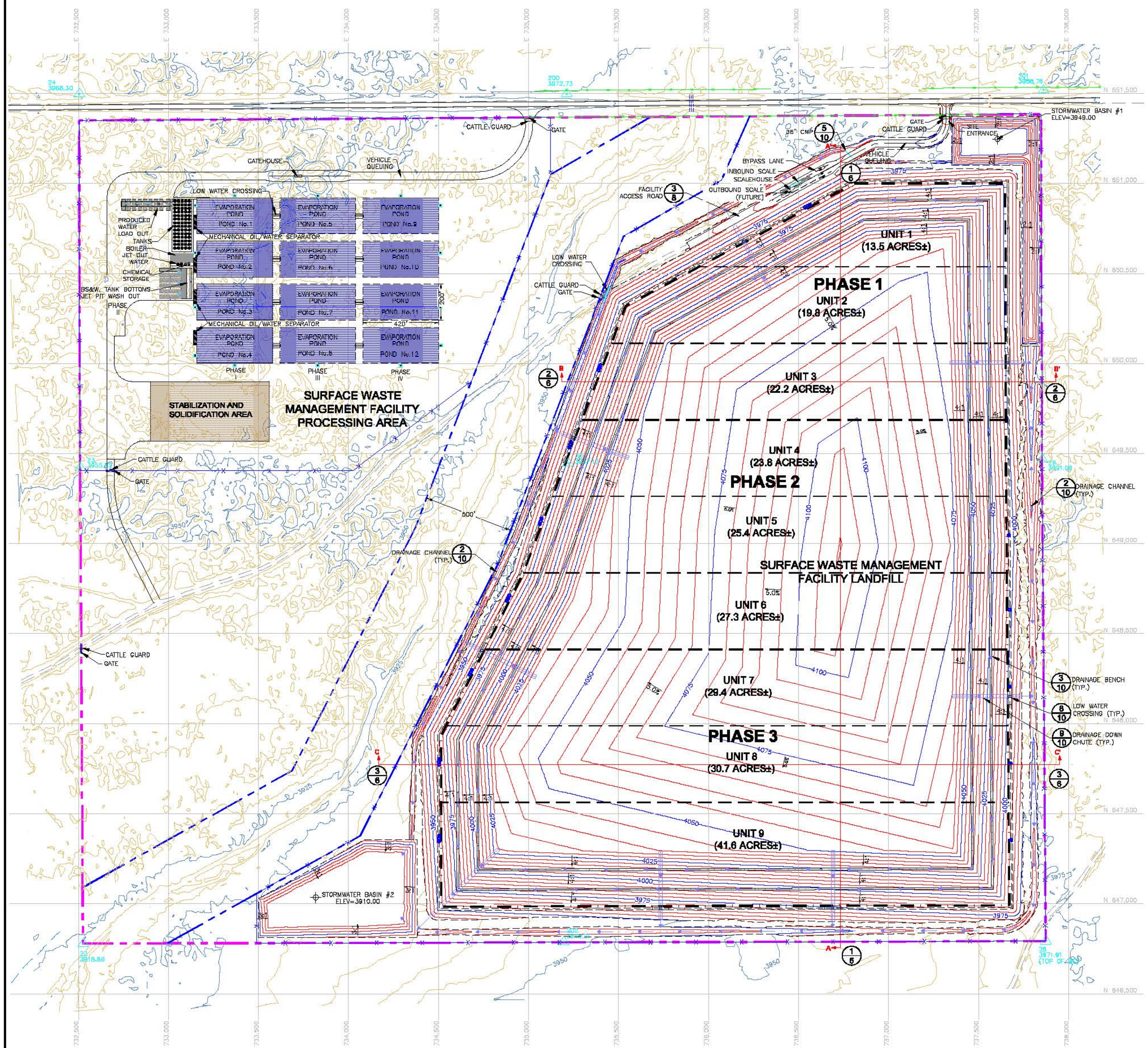
LANDFILL BASE GRADING PLAN

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/10/2014	CAD: 04 BASE GRADING.dwg	PROJECT #: 542.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	SHEET 4 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

NOT FOR CONSTRUCTION
Drawing:P:\acad 2009\542.01.01\RAI 1\PERMIT PLAN SHEETS\04 BASE GRADING.dwg
Date/Time:Jun. 13, 2014-13:04:14 ; LAYOUT: D (LS)



LEGEND

- SITE BOUNDARY (562 ACRES±)
- WATER FEATURE SETBACK (67 ACRES±)
- LIMIT OF WASTE
- LANDFILL PHASE BOUNDARY
- LANDFILL UNIT BOUNDARY
- EXISTING FENCE
- PROPOSED FENCE
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- 25' DESIGN CONTOUR
- 5' DESIGN CONTOUR
- TOP/TOE OF SLOPE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- DIRECTION OF STORMWATER FLOW
- LEACHATE EXTRACTION RISER PIPES
- LEACHATE CLEANOUT RISER PIPES
- SURVEY CONTROL POINT
- POWER POLE
- EXISTING CULVERT
- NEW CULVERT
- HYDROGEN SULFIDE MONITORING STATION
- ROAD SIGN
- SITE GRID
- CROSS SECTION LOCATION
- DETAIL NUMBER / SHEET NUMBER

SURVEY CONTROL POINT DATA

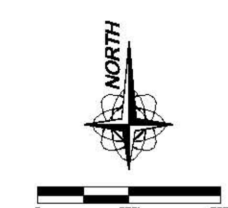
POINT	NORTHING	EASTING	ELEVATION
22	646780.31	732525.87	3918.86
23	649420.79	732507.95	3955.82
24	651497.01	732502.64	3968.19
28	646792.06	737872.55	3971.24
29	649468.54	737851.84	3991.09
30	649445.19	735219.09	3957.12
200	651498.13	735212.57	3972.73
201	651518.82	737859.97	3988.76
202	646789.93	735196.38	3948.21

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

LANDFILL VOLUME

GROSS FILL VOLUME: 39,669,880 CUBIC YARDS



I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

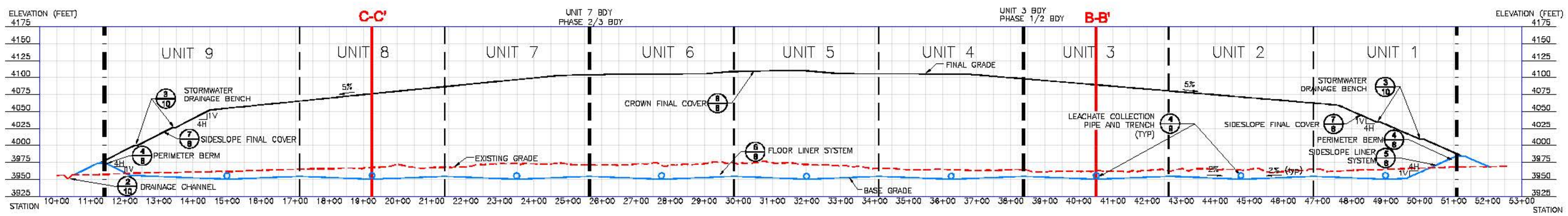
LANDFILL FINAL GRADING PLAN

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

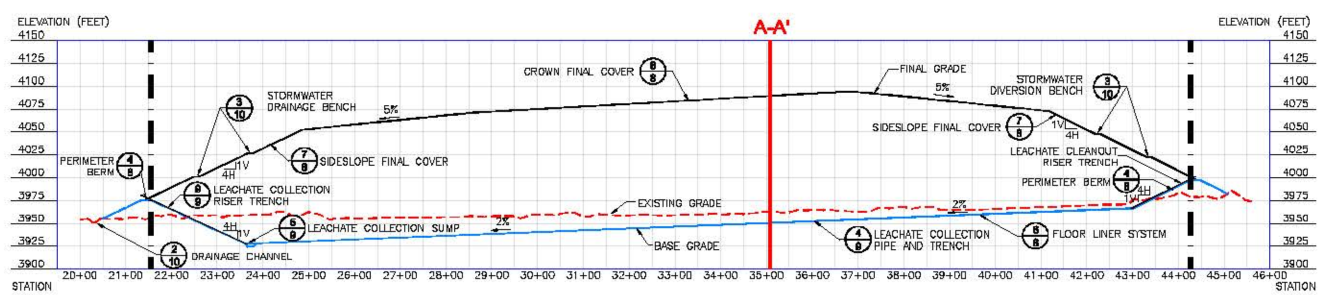
Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/10/2014	CAD: 05 FINAL GRADING .dwg	PROJECT #: 542.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	SHEET 5 of 14
APPROVED BY: IKG	gail@gordonenvironmental.com	

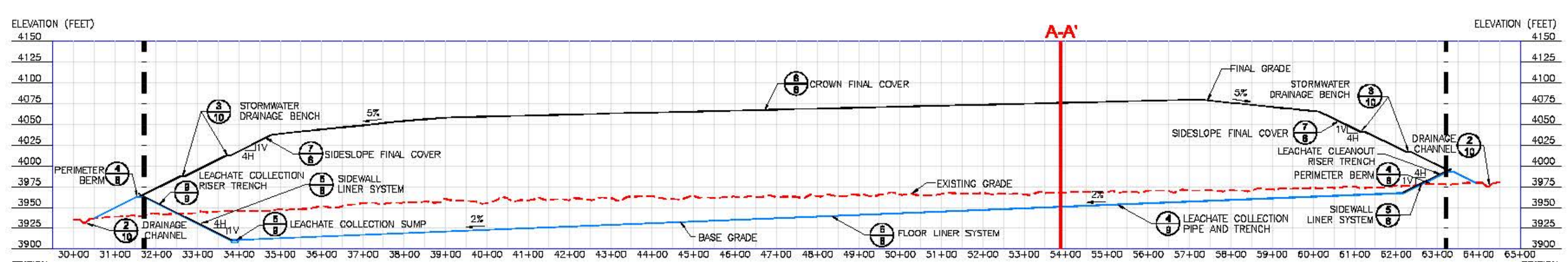
NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\RAI 1\PERMIT PLAN SHEETS\05 FINAL GRADING.dwg
Date/Time:Jun. 13, 2014-12:06:16 ; LAYOUT: D (LS)



1/6 **CROSS SECTION A-A'**



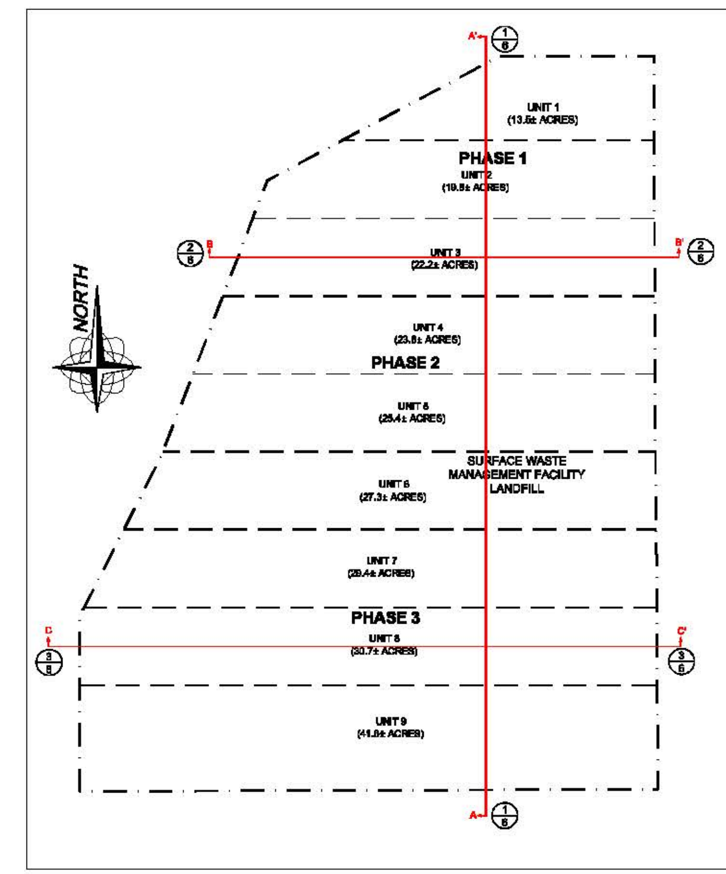
2/6 **CROSS SECTION B-B'**



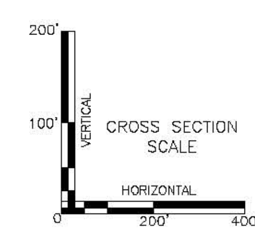
3/6 **CROSS SECTION C-C'**

LEGEND

- LIMIT OF WASTE
- LANDFILL PHASE BOUNDARY
- LANDFILL UNIT BOUNDARY
- EXISTING GRADE
- BASE GRADE
- FINAL GRADE
- A A' CROSS SECTION LOCATION
- 7/8 DETAIL NUMBER / SHEET NUMBER



KEY MAP
N.T.S.



I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

All reports, drawings, specifications, computer files, field data, notes and other documents and instruments prepared by the Engineer as instruments of service shall remain the property of the Engineer. The Engineer shall retain all common law, statutory and other reserved rights, including the copyright thereto.

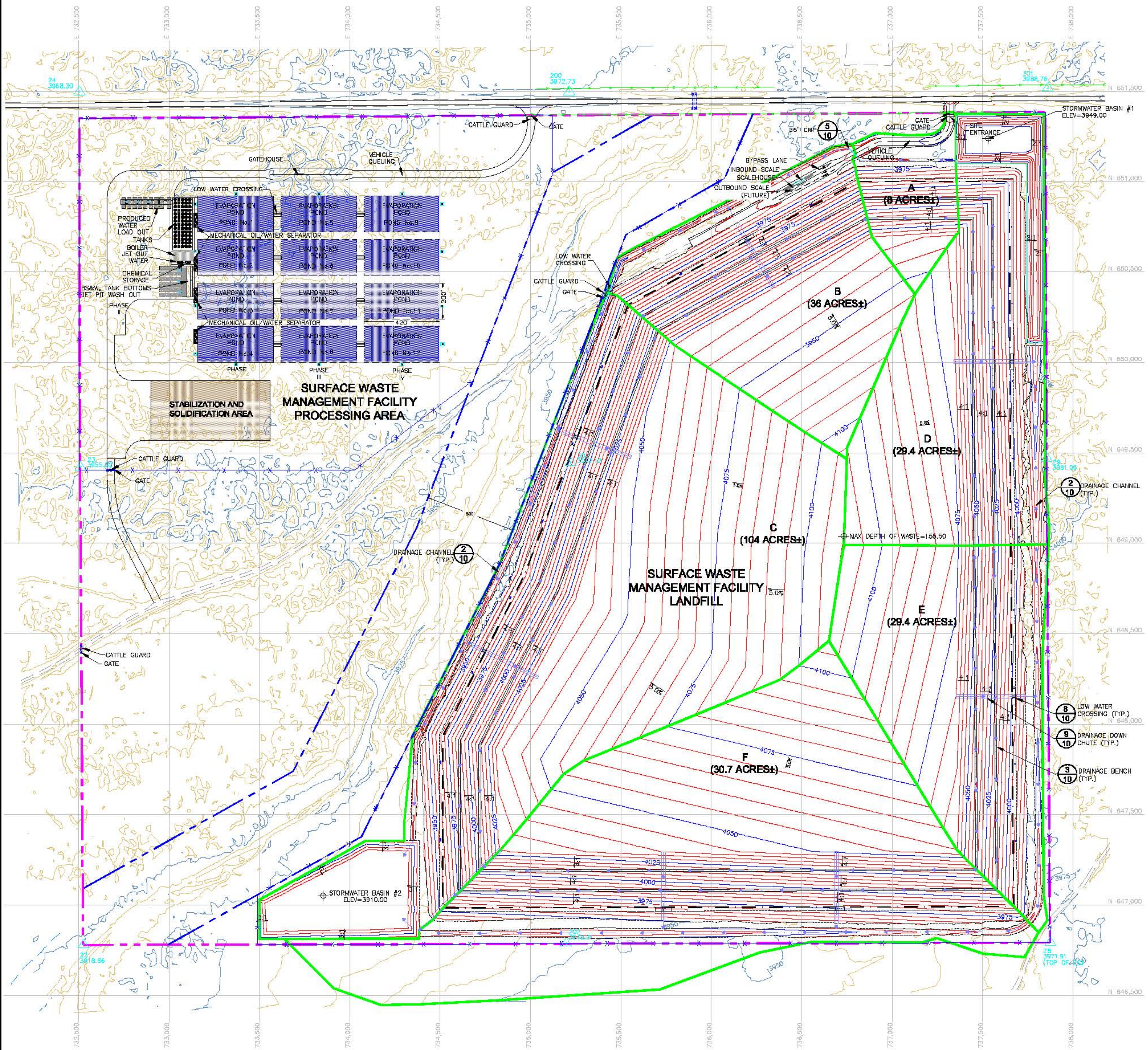
NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\PERMIT PLANS\06 X-SECTIONS.dwg
Date/Time:Jun. 13, 2014-13:06:10 ; LAYOUT: D (LS)

LANDFILL CROSS SECTIONS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/21/2013	CAD: 06 X-SECTIONS.dwg	PROJECT #: 542.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	SHEET 6 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	



LEGEND

- SITE BOUNDARY (562 ACRES±)
- WATER FEATURE SETBACK (67 ACRES±)
- LIMIT OF WASTE
- LANDFILL PHASE BOUNDARY
- LANDFILL UNIT BOUNDARY
- EXISTING FENCE
- PROPOSED FENCE
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- 25' DESIGN CONTOUR
- 5' DESIGN CONTOUR
- TOP/TOE OF SLOPE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- DIRECTION OF STORMWATER FLOW
- LEACHATE EXTRACTION RISER PIPES
- LEACHATE CLEANOUT RISER PIPES
- A (8 ACRES±) DRAINAGE AREA
- SURVEY CONTROL POINT
- EXISTING CULVERT
- NEW CULVERT
- HYDROGEN SULFIDE MONITORING STATION
- ROAD SIGN
- DETAIL NUMBER / SHEET NUMBER
- SITE GRID

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

STORMWATER DISCHARGE

DRAINAGE ID	DRAINAGE AREA (ACRES)	FLOW RATE (CFS)	VOLUME (ACRE-FT)
A	8	42	1.5
B	36	103	6.6
C	104	183	19.1
D	43	142	7.9
E	39	103	7.2
F	89	196	16.3

RETENTION BASIN CAPACITIES

BASIN ID	CONTRIBUTING DRAINAGE AREAS	DISCHARGE VOLUME (ACRE-FT)	BASIN CAPACITY W/ 1 FT. FREEBOARD (ACRE-FT)	BASIN MAX. CAPACITY W/O 1 FT. FREEBOARD (ACRE-FT)	FACTOR OF SAFETY
1	D+NE RUN-ON	55.2	61.0	65.3	1.2
2	A+B+C+E+F+SE RUN-ON	58.1	61.5	68.6	1.2

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

All reports, drawings, specifications, computer files, field data, notes and other documents and instruments prepared by the Engineer as instruments of service shall remain the property of the Engineer. The Engineer shall retain all common law, statutory and other reserved rights, including the copyright thereto.

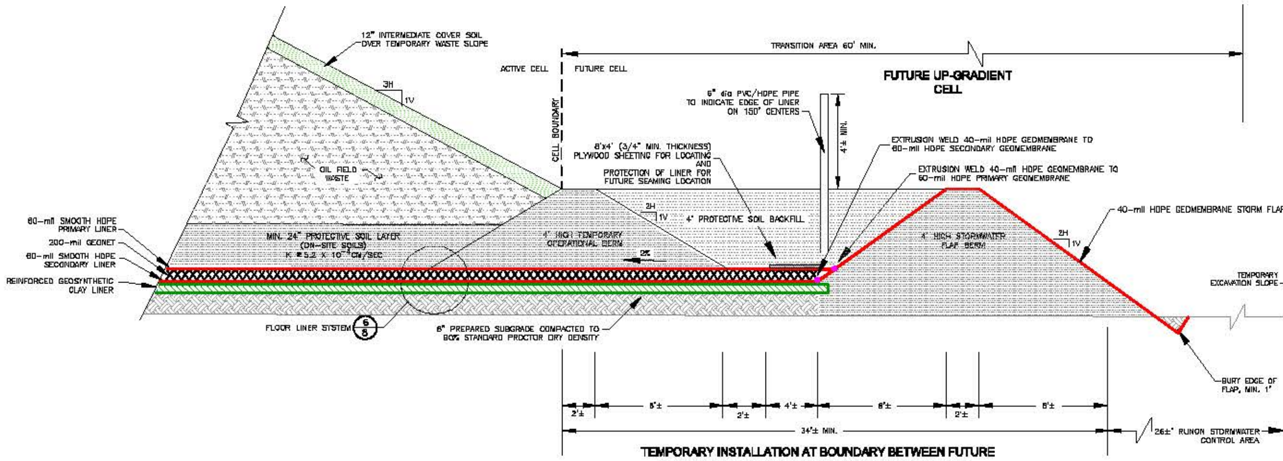
LANDFILL COMPLETION DRAINAGE PLAN

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/10/2014	CAD: 07 COMPLETION PLAN.dwg	PROJECT #: 542.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	SHEET 7 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

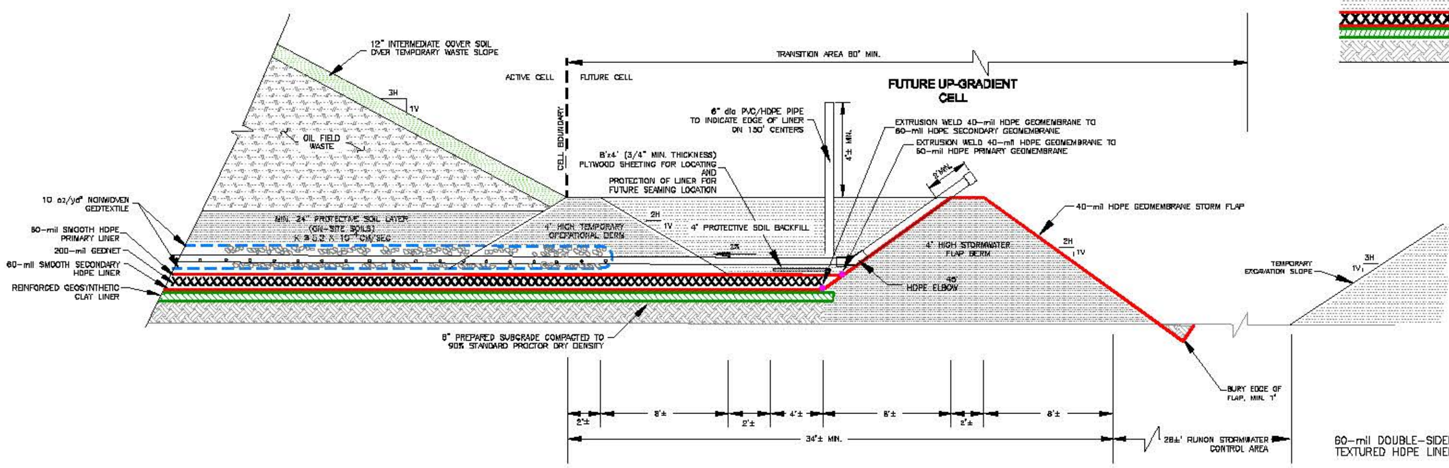
NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\RAI 1\PERMIT PLAN SHEETS\07 DRAINAGE PLAN.dwg
Date/Time:Jun. 13, 2014-12:36:38 ; LAYOUT: D (LS)



TEMPORARY INSTALLATION AT BOUNDARY BETWEEN FUTURE UPGRADIENT CELL AND ACTIVE DOWNGRADIENT CELL

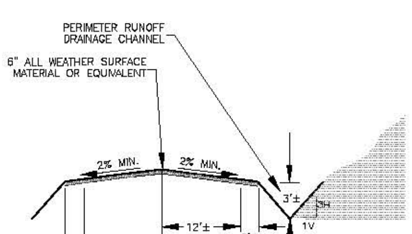
1/8 TYPICAL TRANSITION AREA LINER TERMINATION

NOT TO SCALE



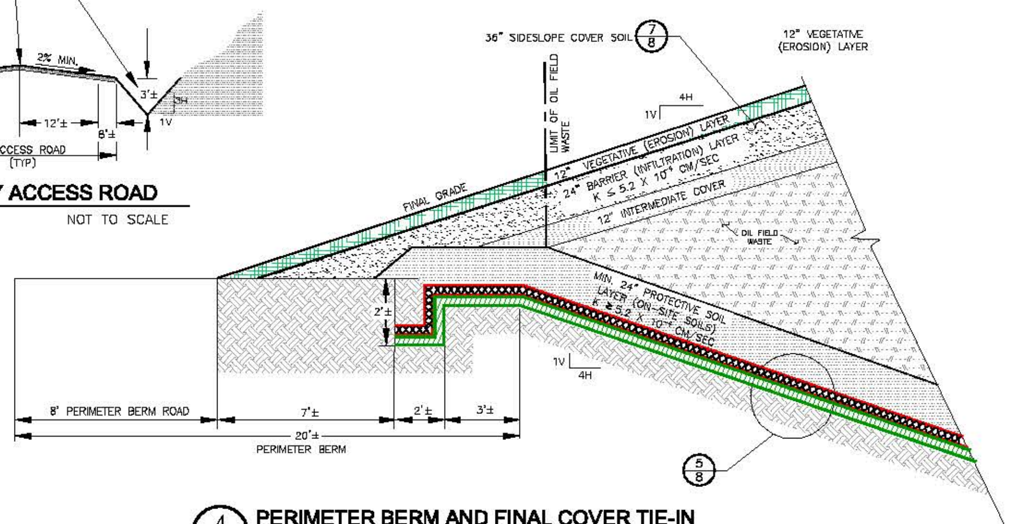
2/8 TEMPORARY LEACHATE COLLECTION RISER PIPE

NOT TO SCALE



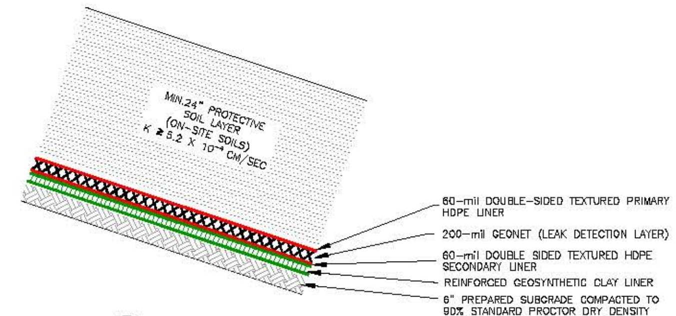
3/8 FACILITY ACCESS ROAD

NOT TO SCALE



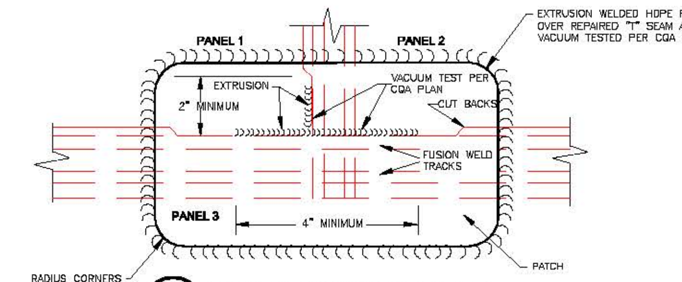
4/8 PERIMETER BERM AND FINAL COVER TIE-IN

NOT TO SCALE



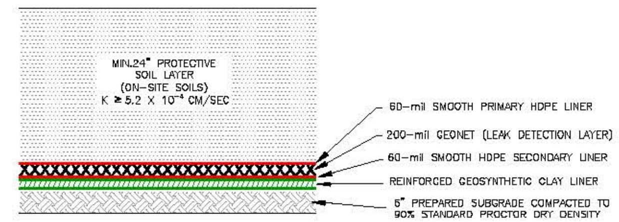
5/8 SIDEWALL LINER SYSTEM

NOT TO SCALE



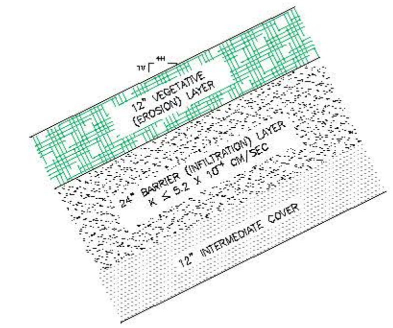
11/8 TYPICAL "T" AT BUTT SEAM

PLAN VIEW NOT TO SCALE



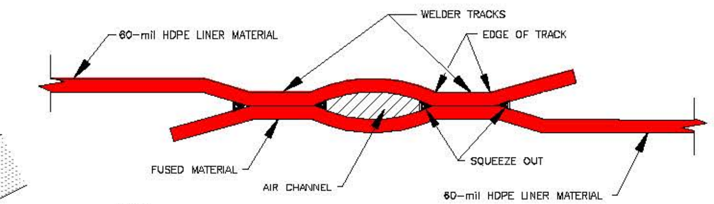
6/8 FLOOR LINER SYSTEM

NOT TO SCALE



7/8 SIDESLOPE SECTION FINAL COVER EVAPOTRANSPORATION DESIGN

NOT TO SCALE

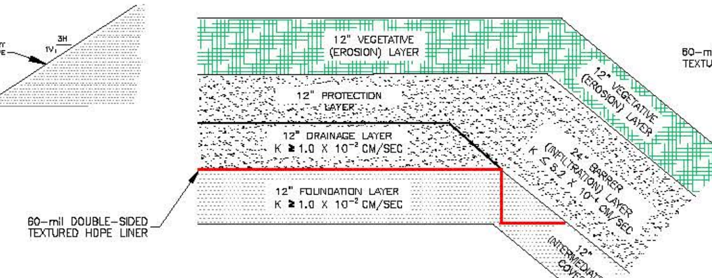


NOTES:

1. BOTH AREAS OF FUSED MATERIAL SHOULD BE VOID OF ANY SEAM LINES.
2. EDGE OF TRACKS NOT TO CUT THE LINER.
3. AIR CHANNEL SHOULD BE CLEAR.
4. BOTH WELDER TRACKS SHALL BE EQUAL WIDTHS.
5. SQUEEZE OUT SHOULD BE JUST BARELY VISIBLE IN ALL 4 LOCATIONS WHEN VIEWED IN THE TEST CROSS-SECTION.

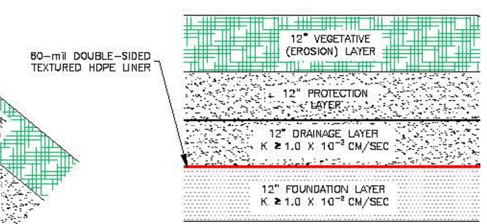
12/8 TYPICAL FUSION WELD

SECTION VIEW NOT TO SCALE



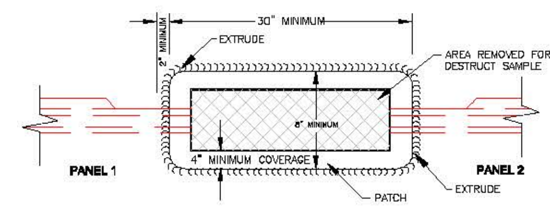
10/8 CROWN 60-mil DOUBLE-SIDED TEXTURED HDPE LINER TERMINATION

NOT TO SCALE



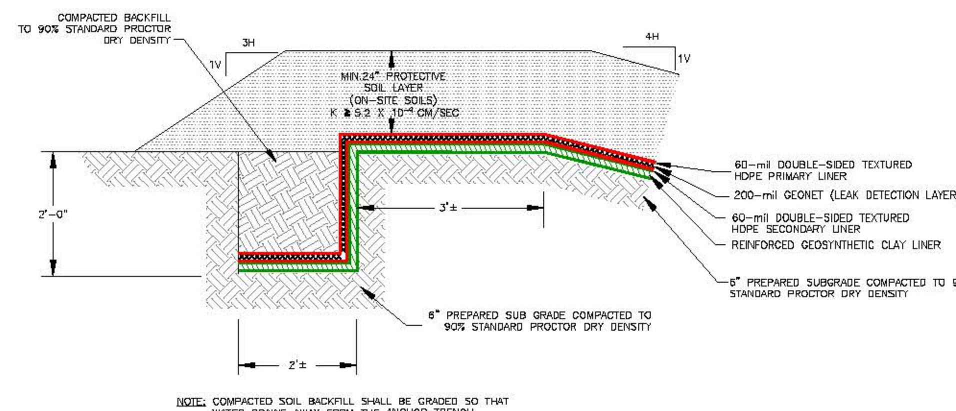
8/8 CROWN OF LANDFILL FINAL COVER SECTION PRESCRIPTIVE DESIGN

NOT TO SCALE



13/8 TYPICAL DESTRUCT SAMPLE PATCH 60-mil HDPE LINER

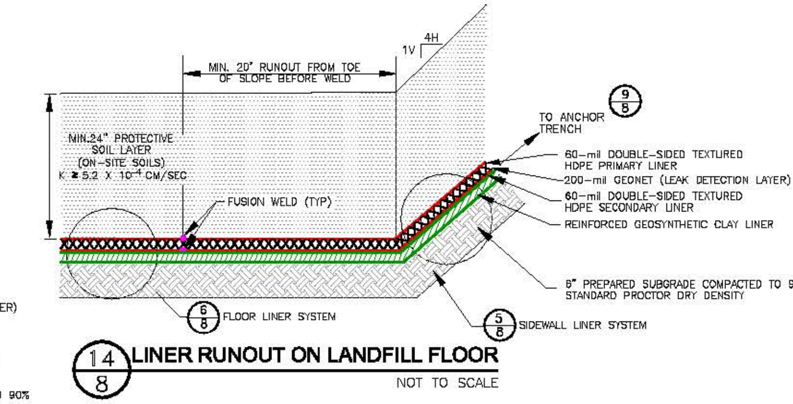
PLAN VIEW NOT TO SCALE



NOTE: COMPACTED SOIL BACKFILL SHALL BE GRADED SO THAT WATER DRAINS AWAY FROM THE ANCHOR TRENCH.

9/8 ANCHOR TRENCH

SECTION VIEW NOT TO SCALE



14/8 LINER RUNOUT ON LANDFILL FLOOR

NOT TO SCALE

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

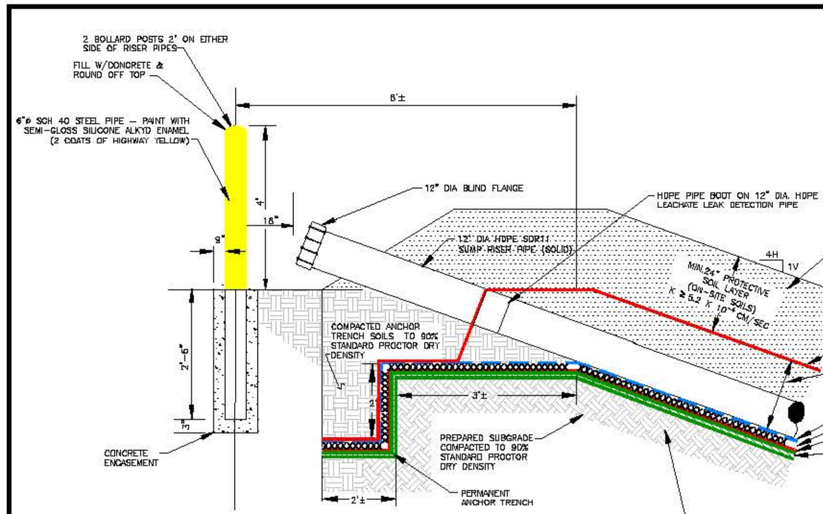
LINER SYSTEM AND COVER DETAILS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

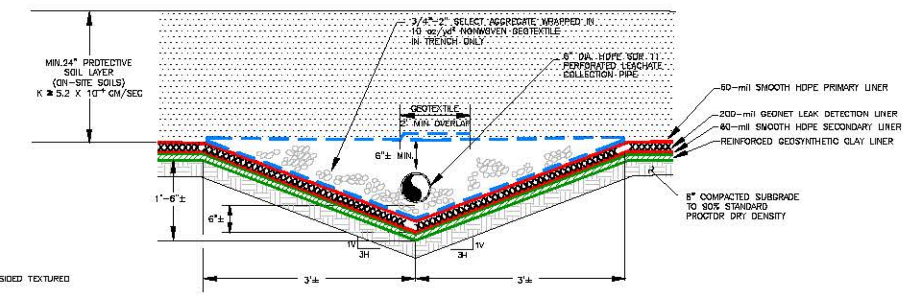
Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/21/2013	CAD: 08 LINER DET.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	SHEET 8 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

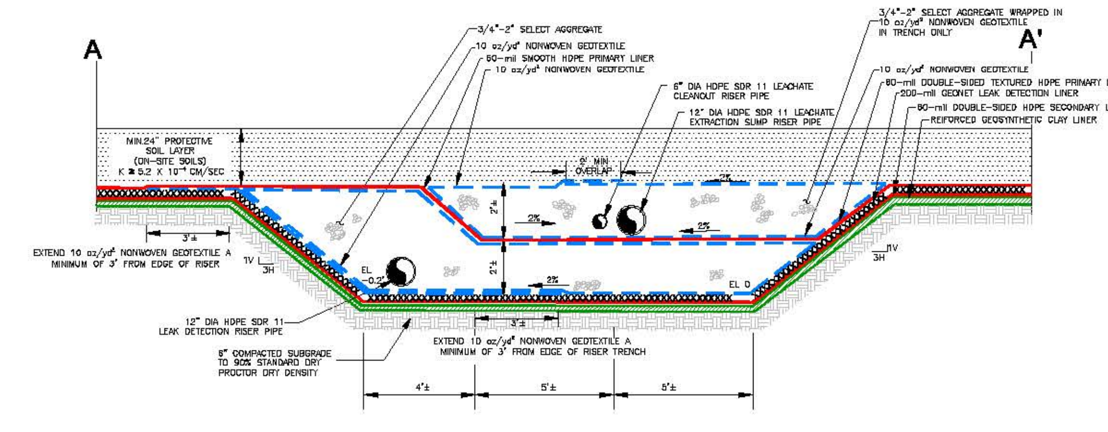
NOT FOR CONSTRUCTION
Drawing: P:\acad 2003\542.01.01\PERMIT PLANS\08 LINER DET.dwg
Date/Time: Jun. 13, 2014-13:10:04



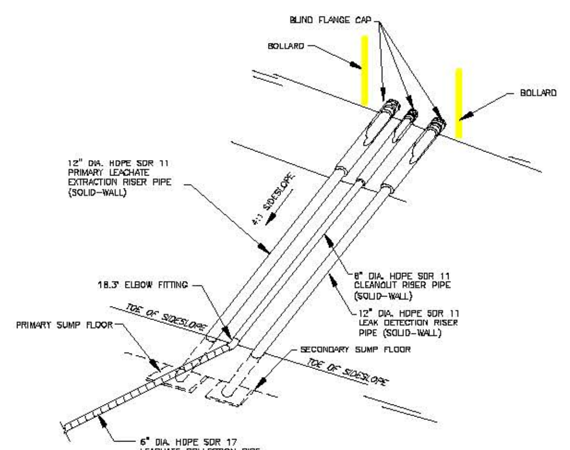
1/9 RISER PIPE TERMINATION
SECTION F-F' NOT TO SCALE



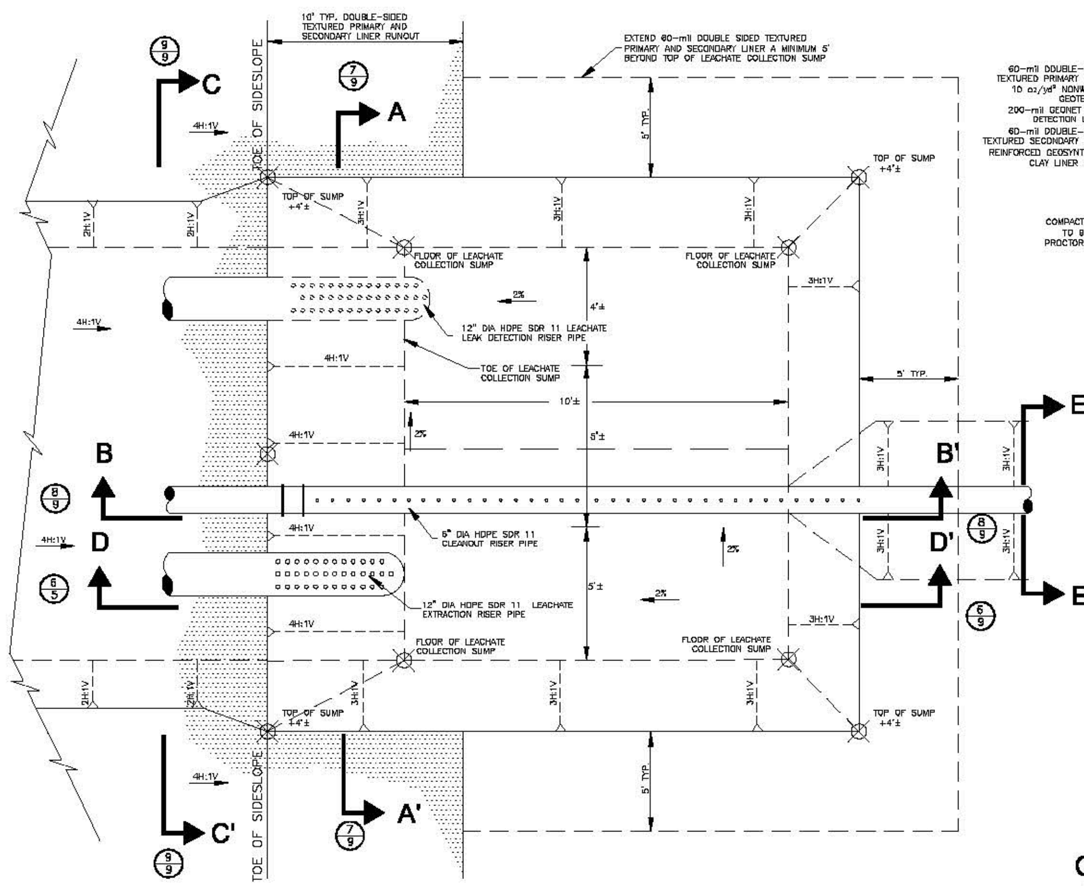
4/9 LEACHATE COLLECTION TRENCH AND PIPE CROSS SECTION E-E'
NOT TO SCALE



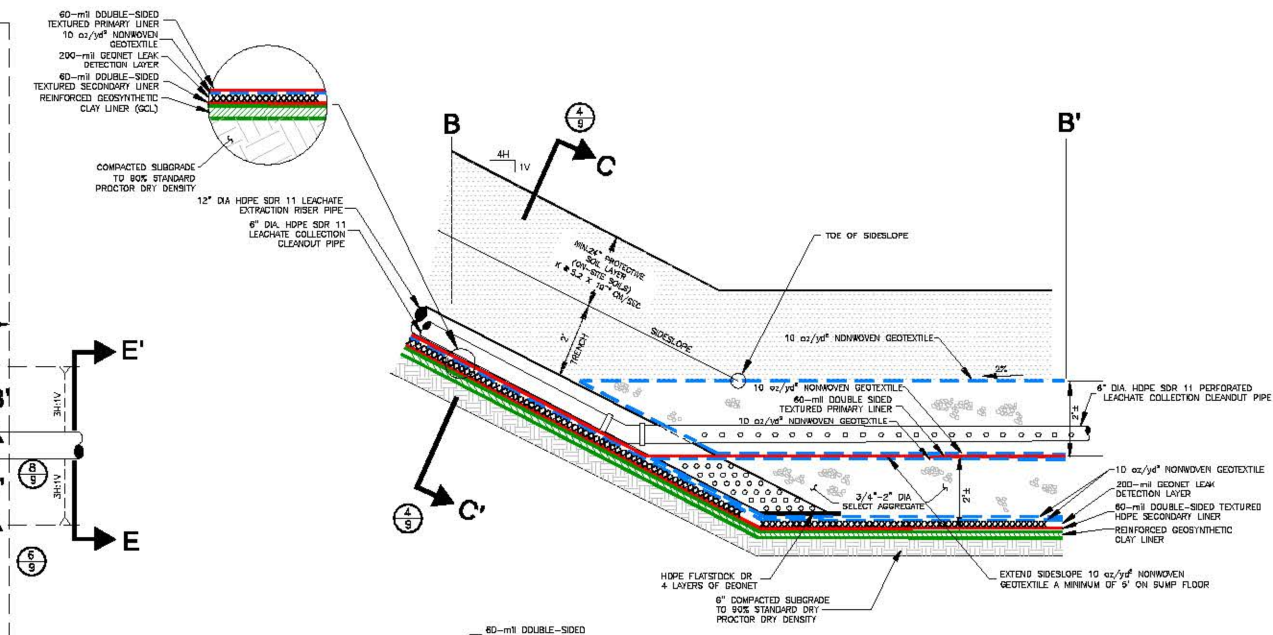
7/9 LEACHATE COLLECTION SUMP
SECTION A-A' NOT TO SCALE



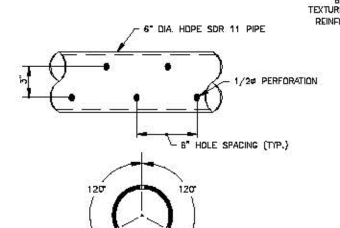
2/9 SUMP CLEANOUT/RISER CONFIGURATION SCHEMATIC
NOT TO SCALE



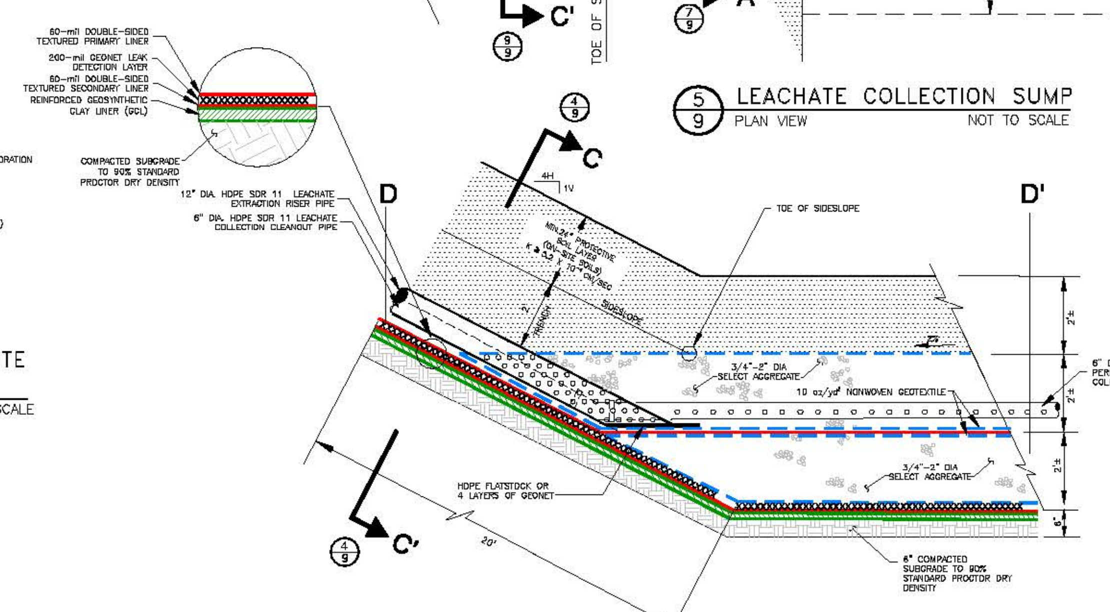
5/9 LEACHATE COLLECTION SUMP
PLAN VIEW NOT TO SCALE



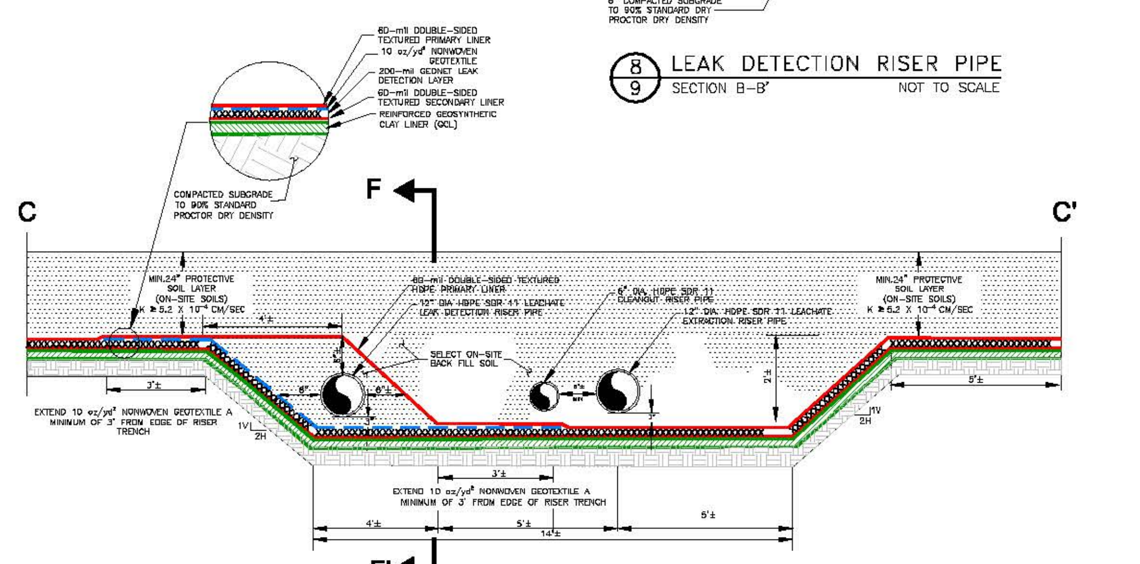
8/9 LEAK DETECTION RISER PIPE
SECTION B-B' NOT TO SCALE



3/9 PERFORATED LEACHATE COLLECTION PIPE
NOT TO SCALE



6/9 LEACHATE EXTRACTION RISER PIPE
SECTION D-D' NOT TO SCALE



9/9 LEACHATE COLLECTION SUMP RISER TRENCH
SECTION C-C' NOT TO SCALE

LEACHATE COLLECTION SYSTEM DETAILS

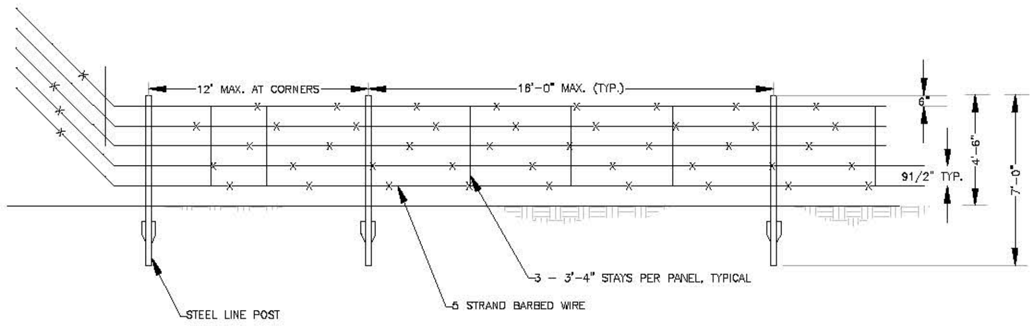
DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

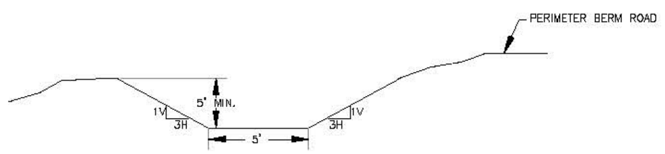
Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/21/2013	CAD: 09 LEACHATE DET.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: CWF/MRH	SHEET 9 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

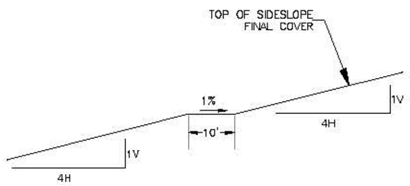
NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\PERMIT PLANS\09 LEACHATE DET.dwg
Date/Time:Jun. 13, 2014-13:12:00



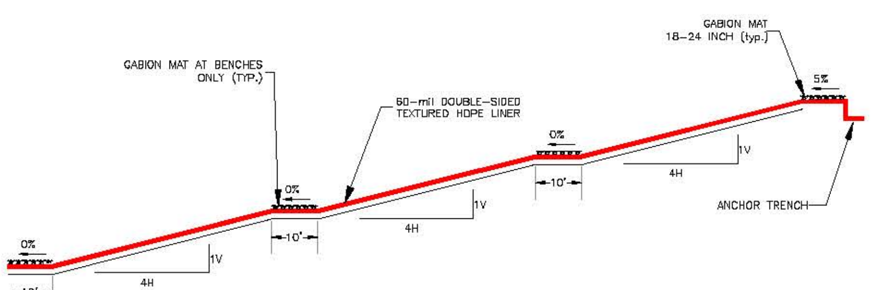
1/10 5-STRAND BARBED WIRE FENCE
NOT TO SCALE



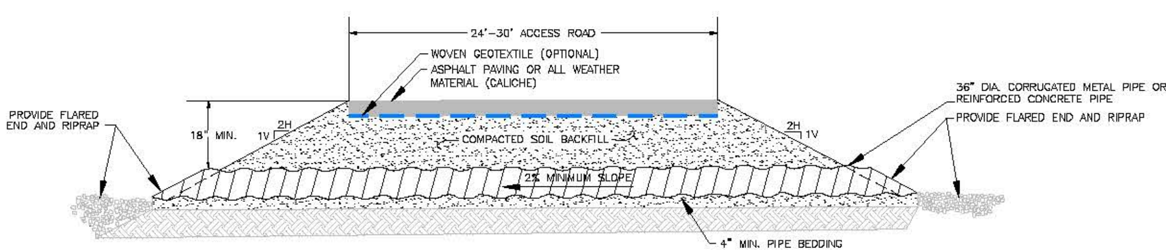
2/10 DRAINAGE CHANNEL TYPICAL SECTION
NOT TO SCALE



3/10 DRAINAGE BENCH
NOT TO SCALE

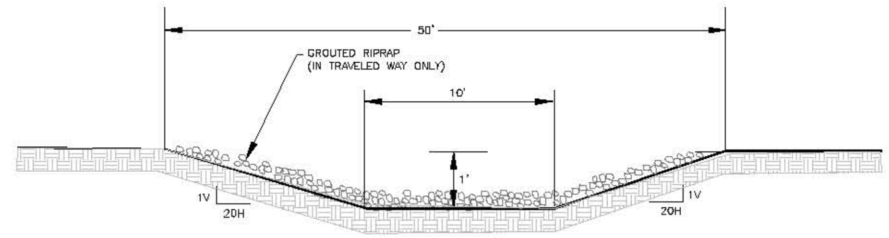


4/10 DOWNCHUTE CROSS SECTION (LONGITUDINAL)
NOT TO SCALE

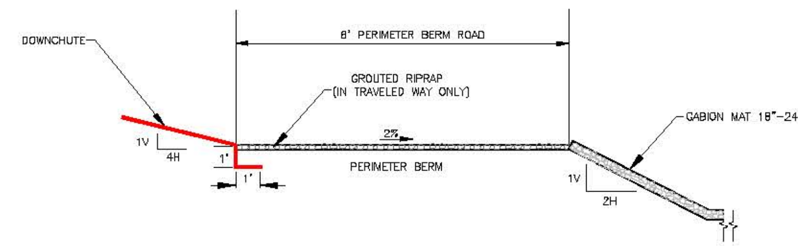


NOTES:
1. INVERT OF CORRUGATED METAL PIPE TO BE DETERMINED DURING INSTALLATION.
2. PROVIDE RIPRAP OR EQUIVALENT EROSION PROTECTION AT INLET AND OUTLET OF CULVERT.
3. PROVIDE FLARED END SECTION AT INLET AND OUTLET OF CULVERT TO PROVIDE A SMOOTH TRANSITION OF FLOW.

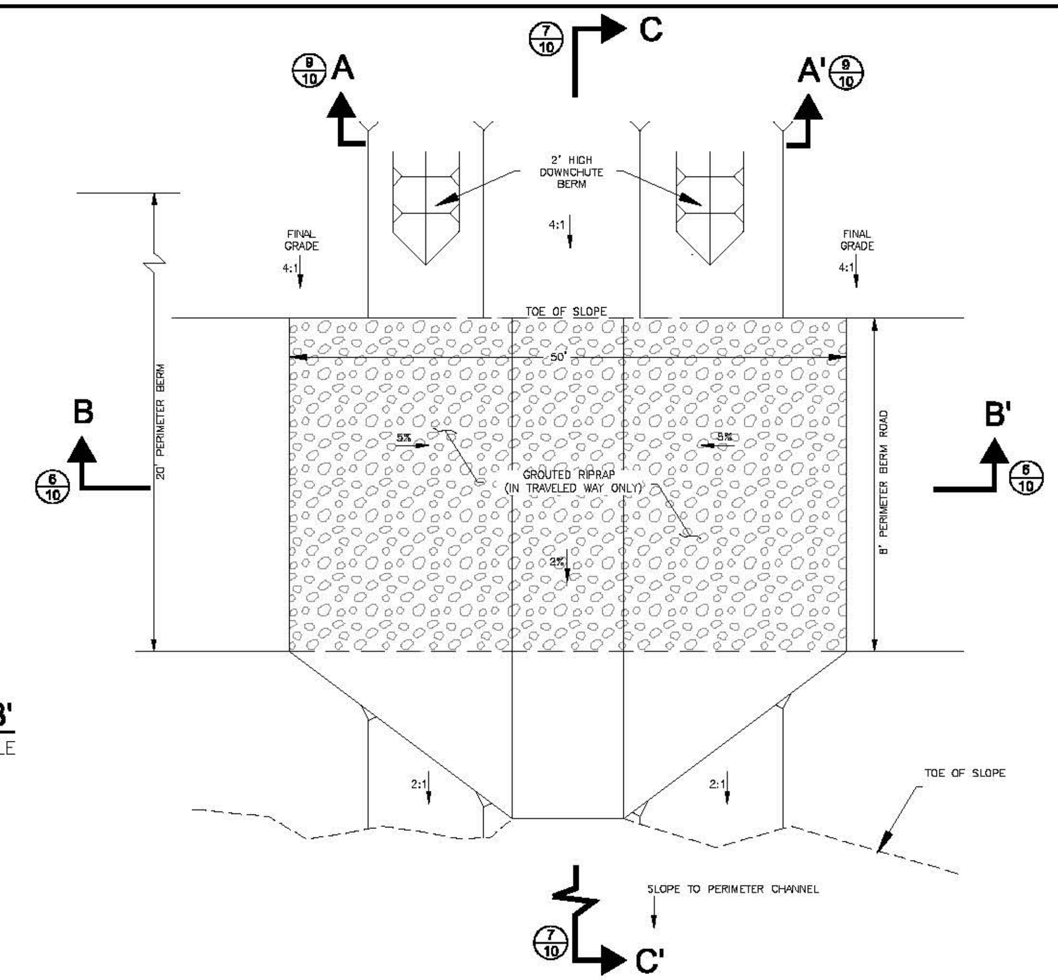
5/10 TYPICAL ROAD CULVERT INSTALLATION
NOT TO SCALE



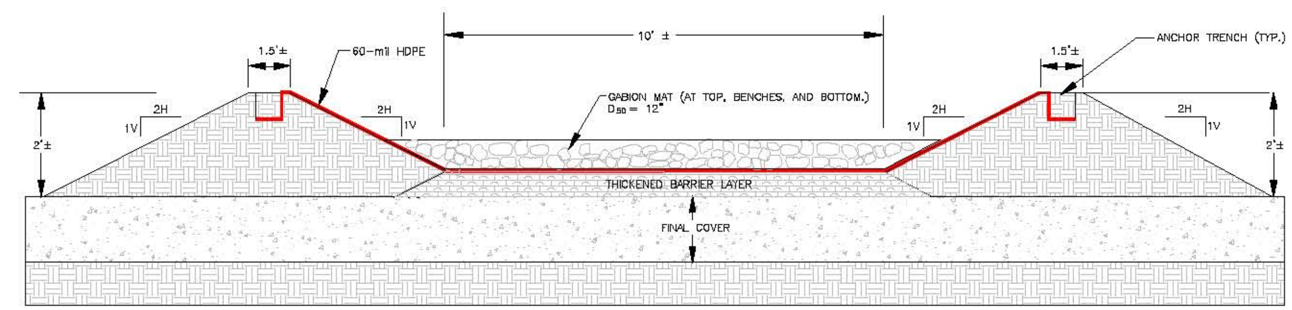
6/10 DOWNCHUTE LOW WATER CROSSING SECTION B-B'
NOT TO SCALE



7/10 DOWNCHUTE LOW WATER CROSSING SECTION C-C'
NOT TO SCALE



8/10 CHANNEL INTERSECTION/DOWNCHUTE LOW WATER CROSSING
PLAN VIEW
NOT TO SCALE



9/10 TYPICAL RIP-RAP LINED PORTION OF DOWNCHUTE SECTION A-A'
NOT TO SCALE

NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\RAI 1\PERMIT PLAN SHEETS\10 STORMWATER DET.dwg
Date/Time:Jun. 13, 2014-12:39:50 ; LAYOUT: D (L9)

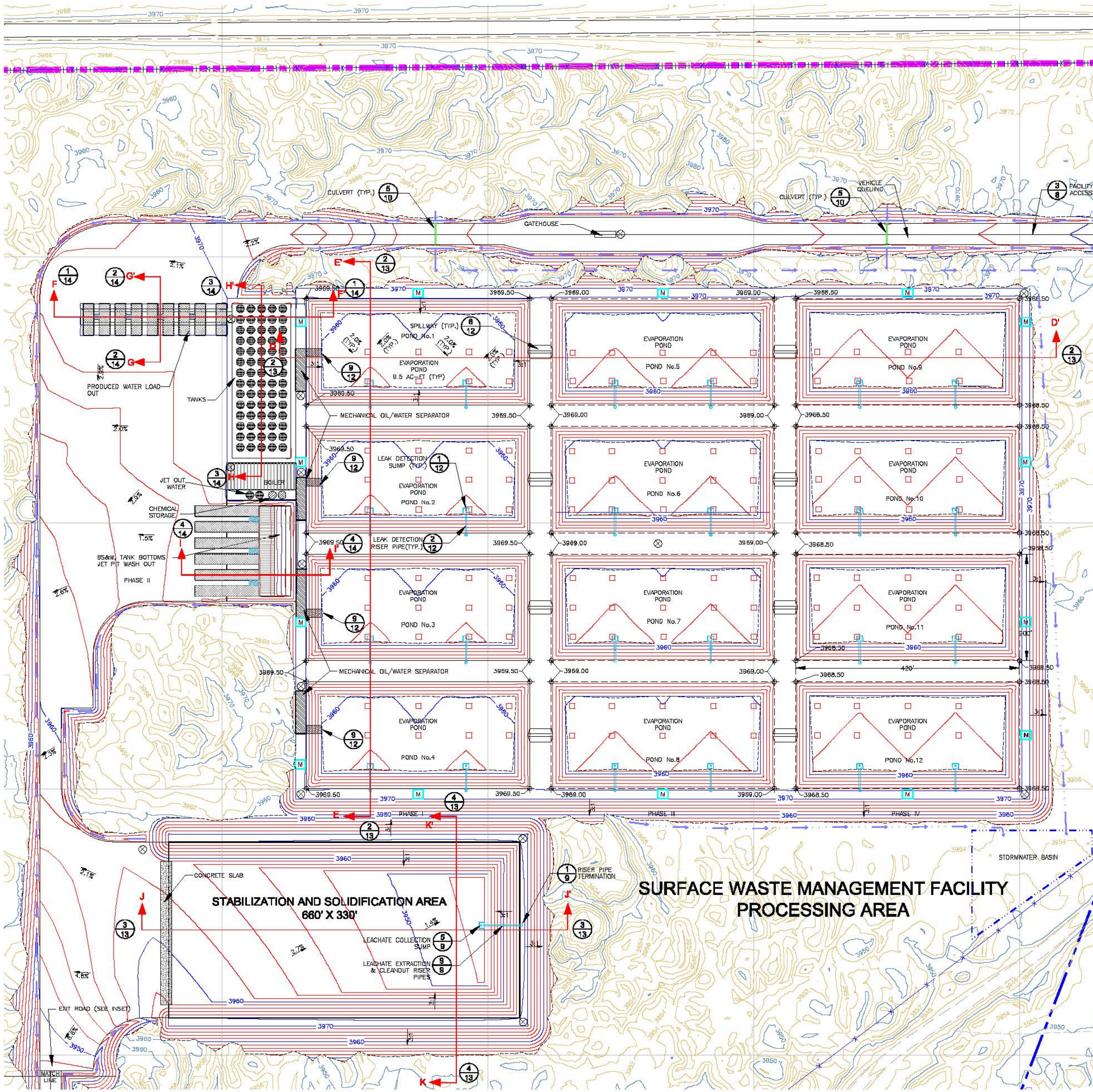
I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

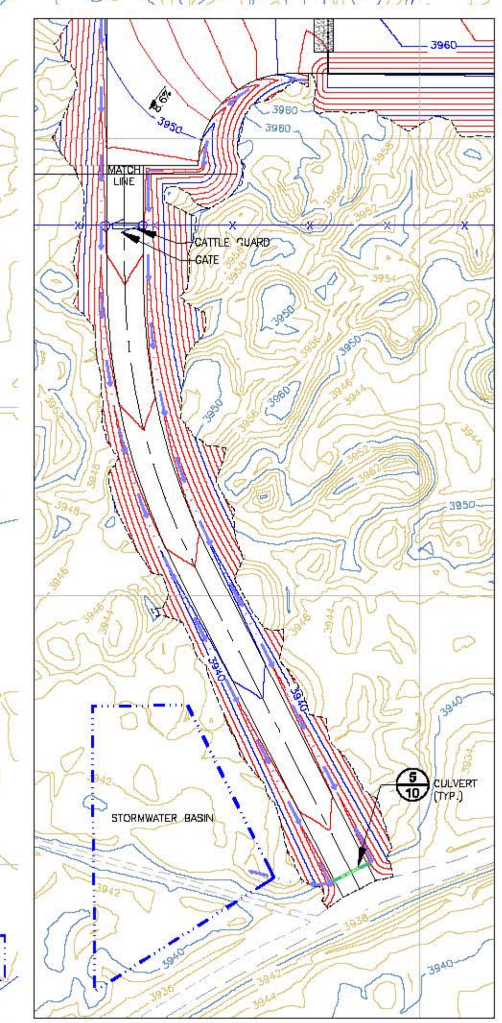
STORMWATER DRAINAGE DETAILS

DNCE ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/13/2014	CAD: 10 STORMWATER DET.dwg	PROJECT #: 542.01.01
DRAWN BY: DM	REVIEWED BY: MRH	SHEET 10 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	





PROCESS AREA EXIT ROAD INSET

LEGEND

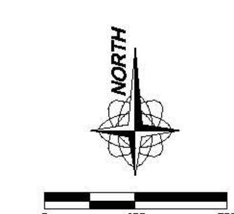
- SITE BOUNDARY (562 ACRES±)
- DRAINAGE FEATURE SETBACK (67 ACRES±)
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- 25' DESIGN CONTOUR
- 5' DESIGN CONTOUR
- TOP/TOE OF SLOPE
- EXISTING FENCE
- PROPOSED FENCE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- DIRECTION OF STORMWATER FLOW
- CULVERT
- CATTLE GUARD
- ROAD SIGN
- HYDROGEN SULFIDE MONITORING STATION
- EVAPORATORS
- PPE AND EMERGENCY EQUIPEMENT
- LEAK DETECTION SUMP & RISER PIPE
- CROSS SECTION LOCATION
- DETAIL NUMBER / SHEET NUMBER
- N 650,500 / E 738,000 SITE GRID

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

VOLUME

ENTRANCE ROAD		
CUT VOLUME	11583	CU. YD.
FILL VOLUME	6290	CU. YD.
NET VOLUME	5293	CU. YD.<CUT>
EVAP PONDS		
CUT VOLUME	182856	CU. YD.
FILL VOLUME	106752	CU. YD.
NET VOLUME	76104	CU. YD.<CUT>
PROCESSING AREA		
CUT VOLUME	51153	CU. YD.
FILL VOLUME	24228	CU. YD.
NET VOLUME	26925	CU. YD.<CUT>
STABILIZATION AND SOLIDIFICATION AREA		
CUT VOLUME	11996	CU. YD.
FILL VOLUME	51002	CU. YD.
NET VOLUME	39006	CU. YD.<FILL>
EXIT ROAD		
CUT VOLUME	18072	CU. YD.
FILL VOLUME	0	CU. YD.
NET VOLUME	18072	CU. YD.<CUT>



I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

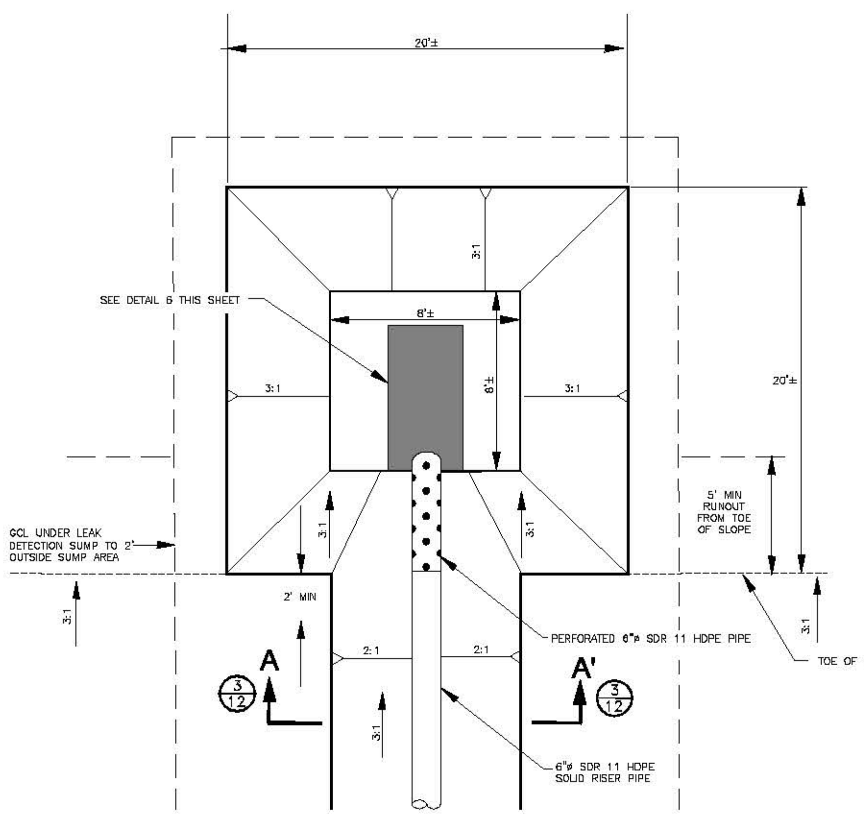
PROCESSING AREA LAYOUT

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

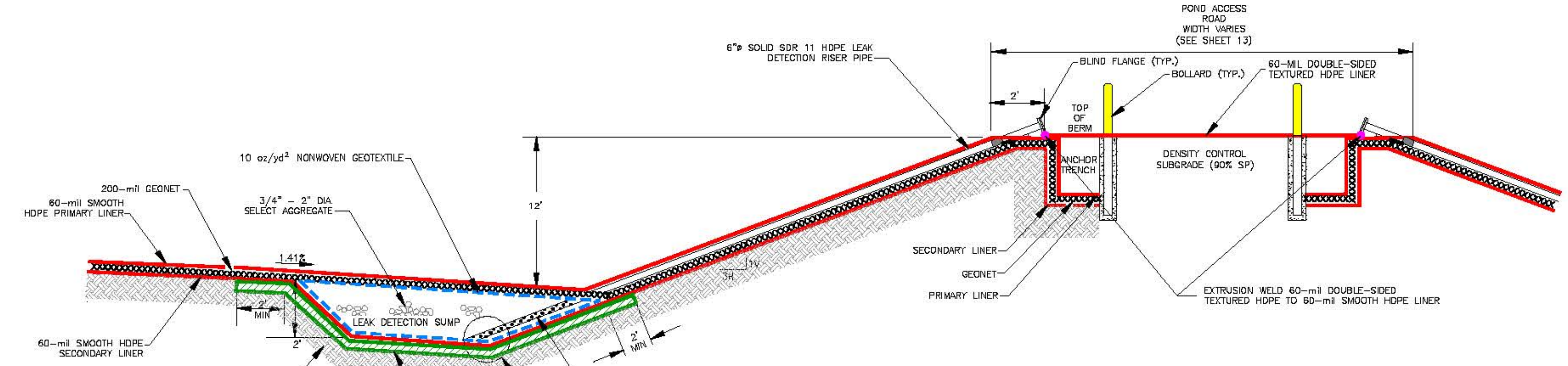
Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/13/2014	CAD: 11 PROCESS AREA .DWG	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	SHEET 11 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

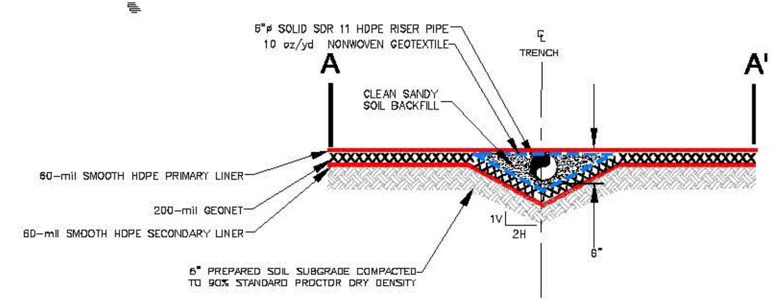
NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\RAI 1\PERMIT PLAN SHEETS\11 PROCESS AREA.dwg
Date/Time:Jun. 13, 2014-11:52:27 ; LAYOUT: D (LS)



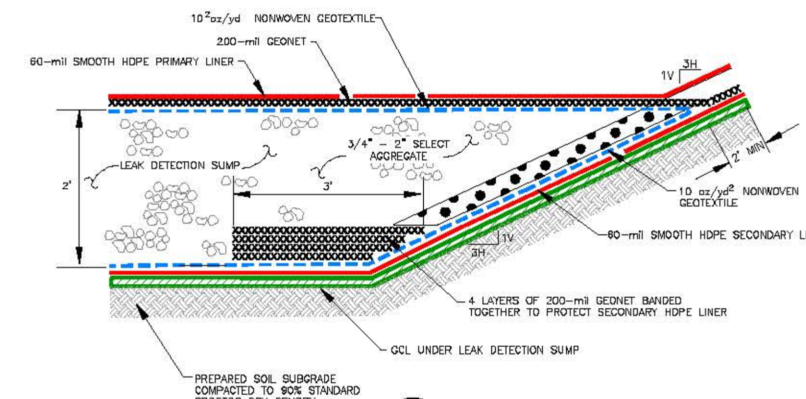
1/12 LEAK DETECTION SUMP
NOT TO SCALE



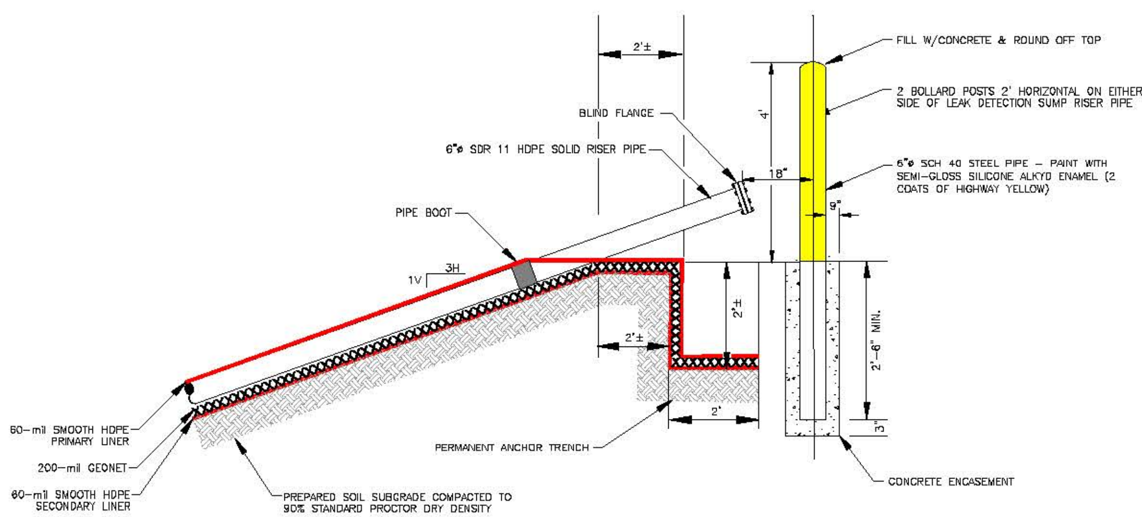
5/12 CROSS-SECTION AT LEAK DETECTION SUMP AND POND ACCESS ROAD DETAIL
NOT TO SCALE



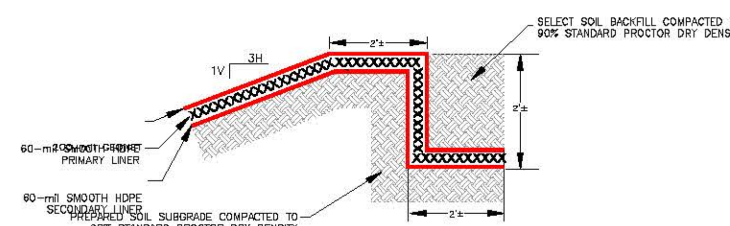
3/12 LEAK DETECTION SUMP RISER PIPE CROSS SECTION A-A'
NOT TO SCALE



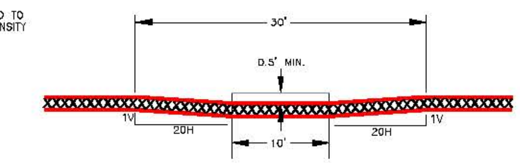
6/12 SUMP FLOOR DETAIL
NOT TO SCALE



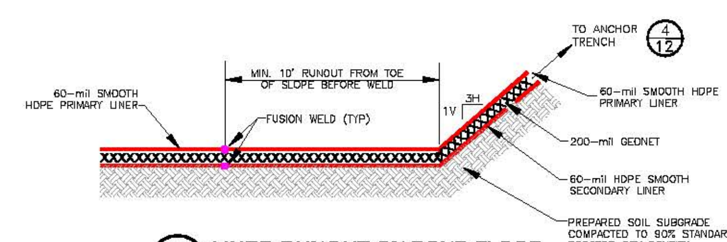
2/12 LEAK DETECTION SUMP RISER PIPE
SECTION VIEW
NOT TO SCALE



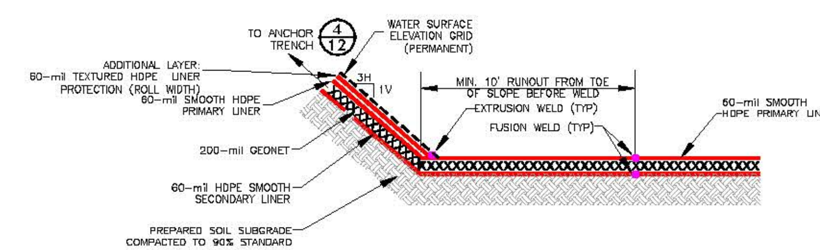
4/12 ANCHOR TRENCH DETAIL
NOT TO SCALE



8/12 SPILLWAY DETAIL
NOT TO SCALE



7/12 LINER RUNOUT ON POND FLOOR
NOT TO SCALE



9/12 POINT DISCHARGE LINER PROTECTION
NOT TO SCALE

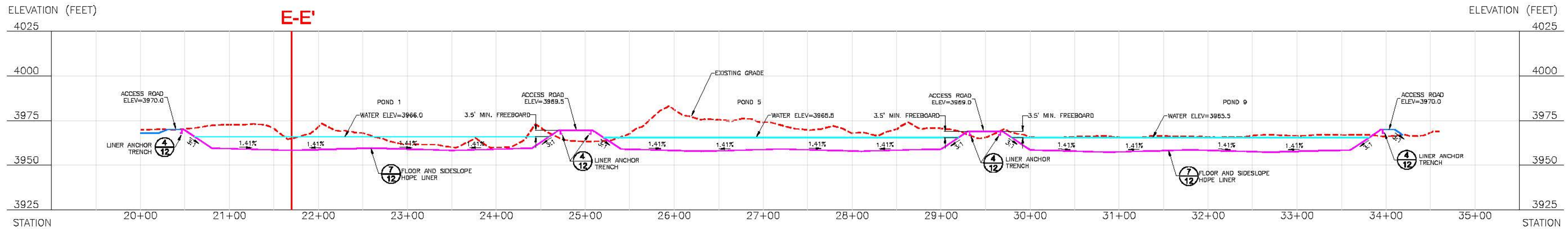
I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

EVAPORATION POND DETAILS
DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

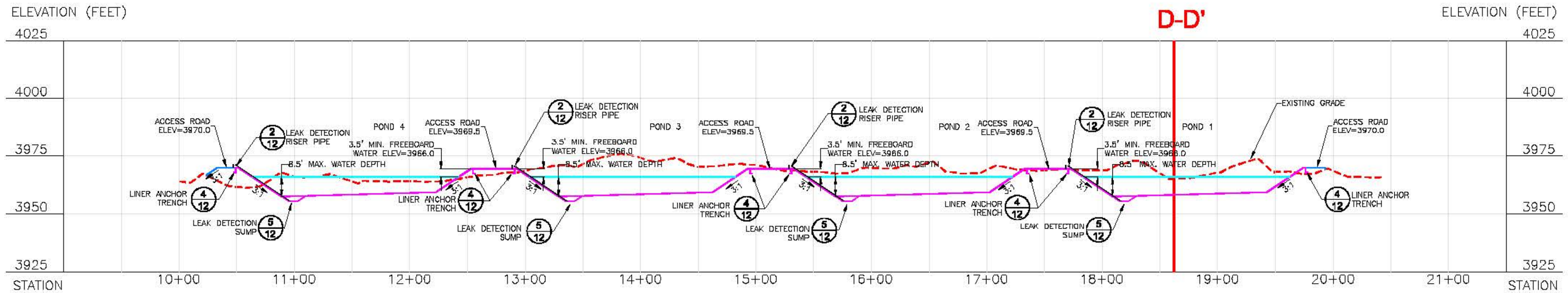
Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991
gei@gordonenvironmental.com

DATE: 10/21/2013	CAD: 12 EVAP POND DET.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	SHEET 12 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

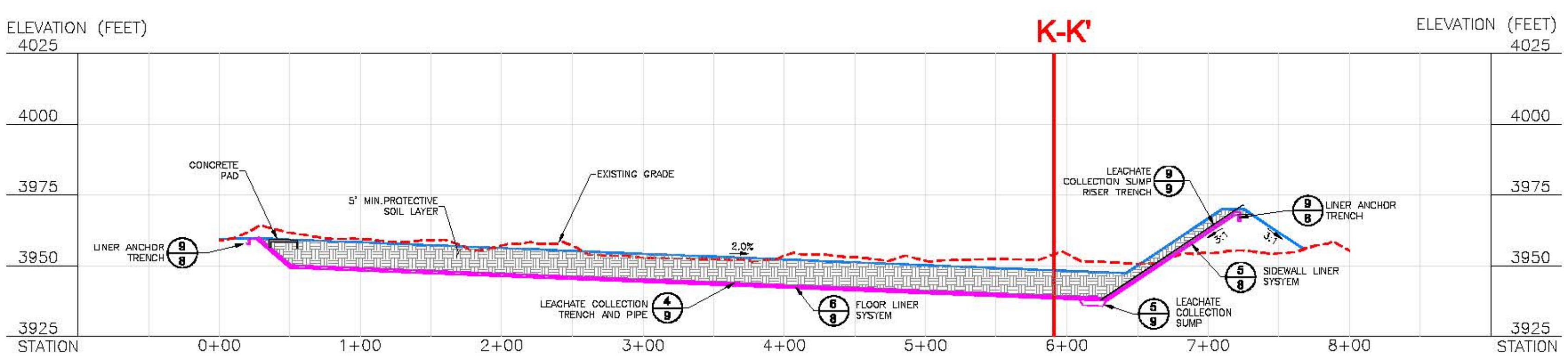
NOT FOR CONSTRUCTION
Drawing:P:\acad 2003\542.01.01\RAI 1\PERMIT PLAN SHEETS\12 EVAP POND DET.dwg
Date/Time:Jun. 13, 2014-12:44:01



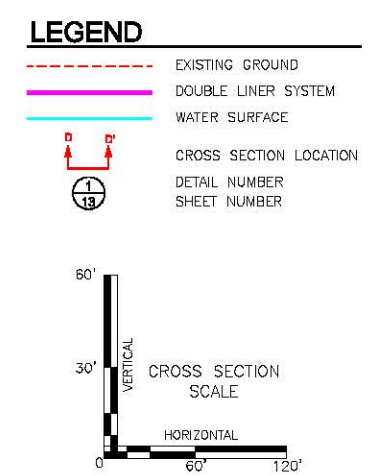
EVAPORATION PONDS
SECTION D-D'

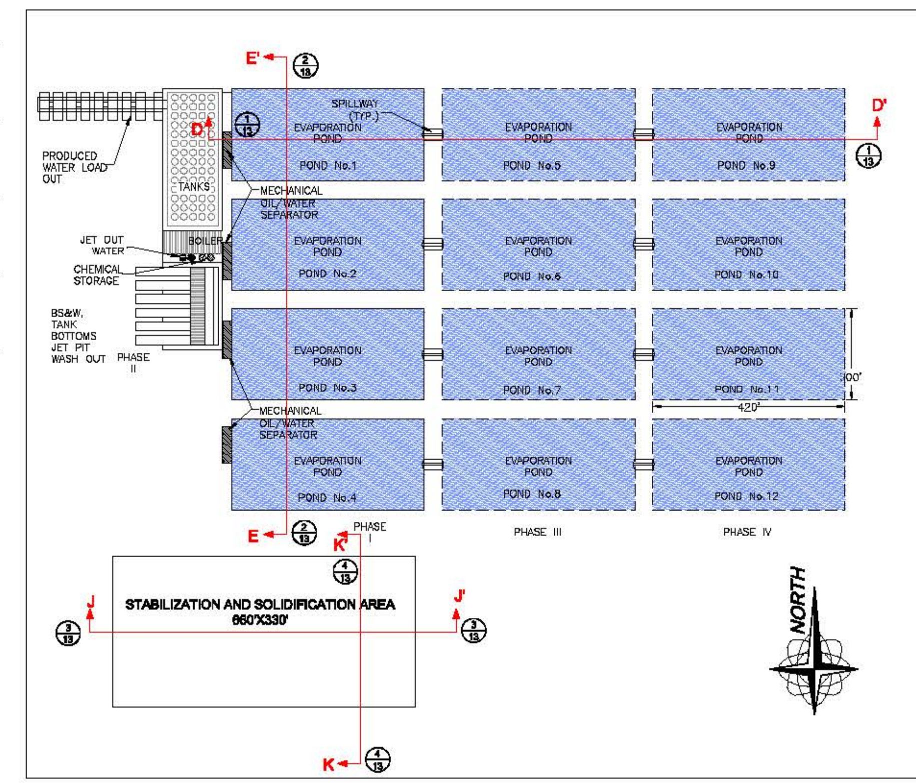


EVAPORATION PONDS
SECTION E-E'

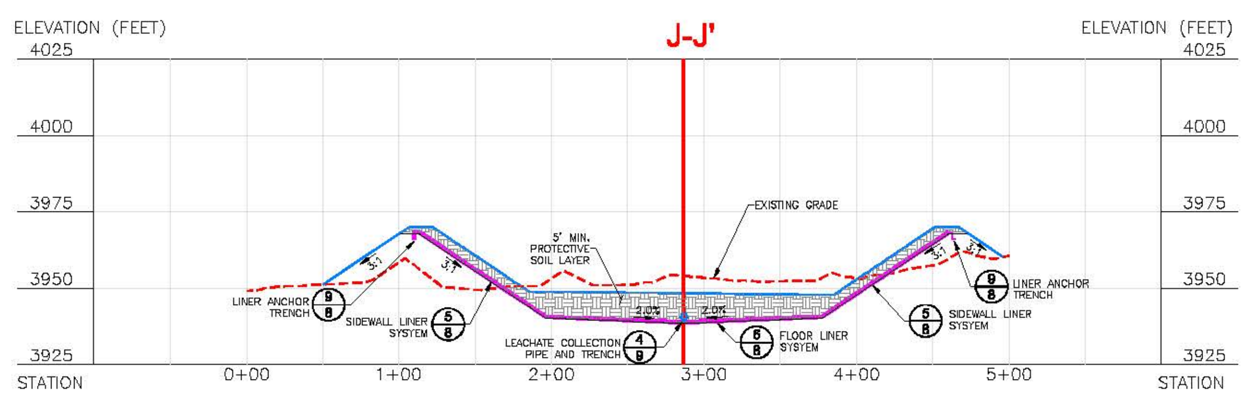


STABILIZATION AND SOLIDIFICATION AREA
SECTION J-J'





KEY MAP
N.T.S.



STABILIZATION AND SOLIDIFICATION AREA
SECTION K-K'

NOT FOR CONSTRUCTION
Drawing:P:\acad 2008\542.01.01\RAI 1\PERMIT PLAN SHEETS\13 EVAP X-SECT.dwg
Date/Time:Jun. 12, 2014-12:49:11 ; LAYOUT: D (LS)

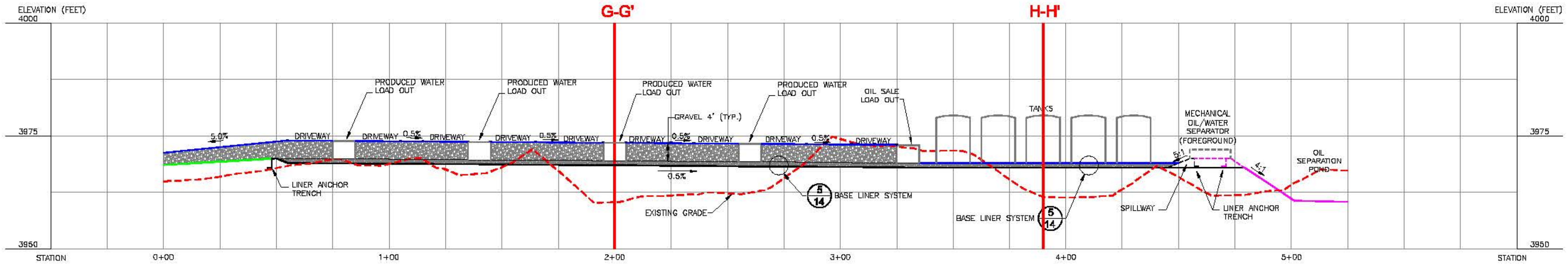
I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

EVAPORATION POND AND STABILIZATION/SOLIDIFICATION AREA CROSS SECTIONS

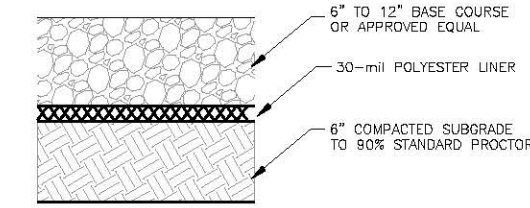
DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

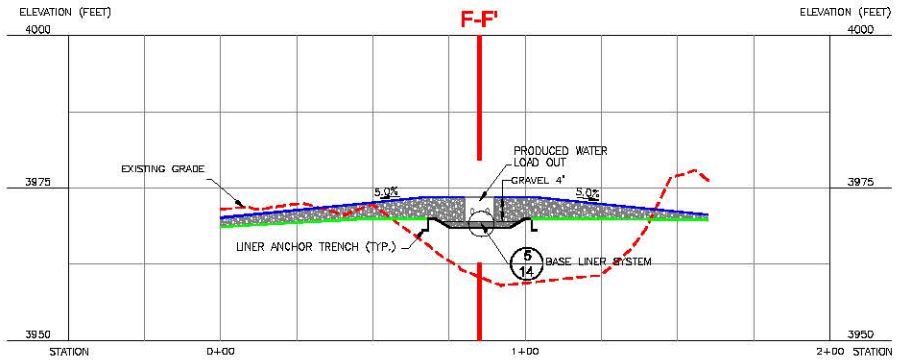
DATE: 10/21/2013	CAD: 13 EVAP X-SECT.dwg	PROJECT #: 542.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	SHEET 13 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	



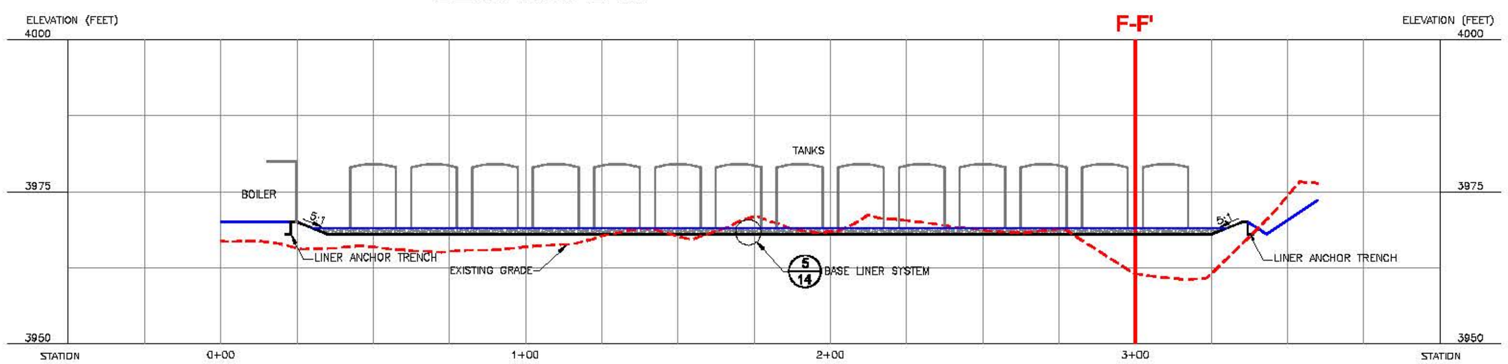
1/14 PROCESSING AREA SECTION F-F'



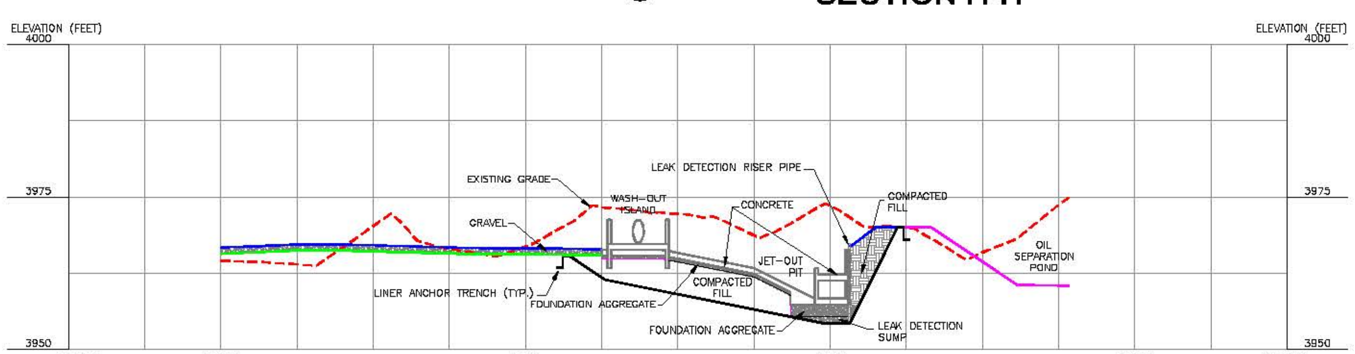
5/14 BASE LINER SYSTEM
NOT TO SCALE



2/14 PROCESSING AREA SECTION G-G'



3/14 PROCESSING AREA SECTION H-H'

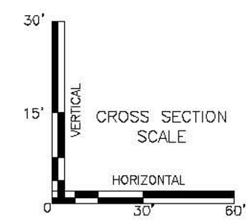


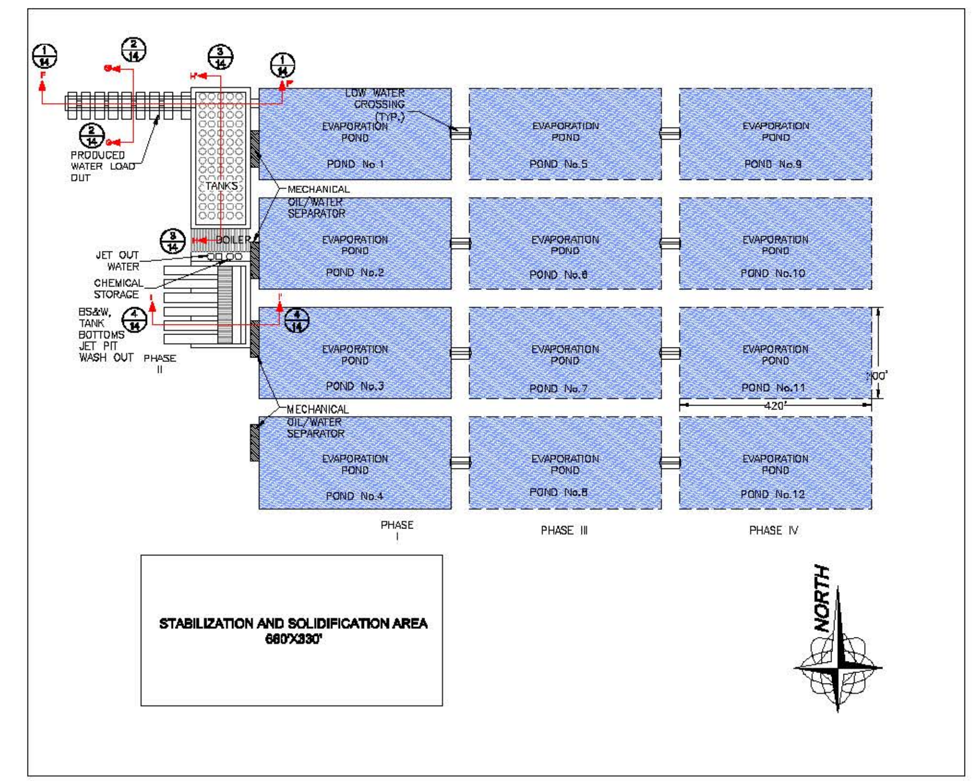
4/14 PROCESSING AREA SECTION I-I'

LEGEND

- EXISTING GROUND
- FINISHED GRADE
- SUBGRADE
- 30-mil POLYESTER LINER
- 60-mil HDPE LINER

CROSS SECTION LOCATION
DETAIL NUMBER
SHEET NUMBER





KEY MAP
N.T.S.

NOT FOR CONSTRUCTION
Drawing:P:\acad 2005\642.01.01\RAI 1\PERMIT PLAN SHEETS\14 PROCESS X-SECT.dwg
Date/Time:Jun. 13, 2014-13:15:14 ; LAYOUT: D (LS)

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

PROCESSING AREA LAYOUT CROSS SECTIONS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991
gei@gordonenvironmental.com

DATE: 06/10/2014	CAD: 14 PROCESS X-SECT.dwg	PROJECT #: 642.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	SHEET 14 of 14
APPROVED BY: IKG	gei@gordonenvironmental.com	

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.B

LINER LONGEVITY ARTICLE:

GEOSYNTHETICS MAGAZINE, OCT/NOV 2008

How long will my liner last?

| What is the remaining service life of my HDPE geomembrane?

By Ian D. Peggs, P.E., P.Eng., Ph.D.

Introduction

In his keynote lecture at the GeoAmericas-2008 conference last March, Dr. Robert Koerner (et al., 2008) of the Geosynthetic Institute (GSI) reported the ongoing Geosynthetic Research Institute (GRI) work to make the first real stab at assessing the service lives of high-density polyethylene (HDPE), linear low-density polyethylene (LLDPE), reinforced PE, ethylene propylene diene terpolymer (EPDM), and flexible polypropylene (fPP) exposed geomembranes.

The selected environment simulated that of Texas, USA, in sunny ambient temperatures between ~7°C (45°F) and 35°C (95°F). Of course, an exposed black HDPE geomembrane in the sun will achieve much higher temperatures, probably in excess of 80°C (176°F).

I do not know what the temperature would be at 150-300mm above the liner (for those still specifying this parameter), but it is quite immaterial. The only temperature of concern is the actual geomembrane temperature.

The lifetimes are shown in **Table 1**, but it must be recognized that these data are for specific manufactured products with specific formulations. The "greater than" notation indicates that laboratory exposures (incubations) are still on-going, not that some samples have failed after the indicated time period. The PE-R-1 material is a thin LLDPE, so it might be expected to be the first to reach the defined end of life; the half-life—the time to loss of 50% of uniaxial tensile properties.

It is interesting to note that HDPE-1 and LLDPE-1 are proceeding apace, but it would be expected that the LLDPE-1 would reach its half-life earlier than HDPE-1. However, this does not automatically follow. With adequate additive formulations, perhaps LLDPE could be left exposed and demonstrate more weathering resistance than some HDPEs. This demonstrates the fact that all PEs, whether HD or LLD, are not identical—they can have different long-term performances dependent on the PE resin used and the formulation of the stabilizer package. However, such differences are not evident in the conventional mechanical properties such as tensile strength/elongation, puncture and tear resistances, and so on.

The two fPPs are performing well. However, there had also been an fPP-1, one of the first PP geomembranes that did not perform well. This was due to a totally inappropriate stabilizer formulation. That particular product lasted 1.5 years in service. In

Final Inspection continued on page 44

Type	Specification	Predicted Lifetime in Texas, USA
HDPE-1	GRI-GM13	>28 years (Incubation ongoing)
LLDPEE-1	GRI-GM17	>28 years (Incubation ongoing)
EPDM-1	GRI-GM21	>20 years (Incubation ongoing)
PE-R-1	GRI-GM22	≈17 years (reached halflife)
fPP-2	GRI-GM18 (temp. susp.)	>27 years (Incubation ongoing)
fPP-3	GRI-GM18 (temp. susp.)	>17 years (Incubation ongoing)

Table 1 | Estimated exposed geomembrane lifetimes

| Ian Peggs is president of I-CORP International Inc. and is a member of *Geosynthetics* magazine's Editorial Advisory Committee.

Final Inspection continued from page 56

the QUV weatherometer, it lasted 1,800 light hours at 70°C (158°F). Therefore, the lab/field correlation is that 1,000 QUV light hours is equivalent to a 0.83yr service life under those specific environmental conditions.

At another location in Texas, Koerner/GRI found 1,000hr of QUV exposure was equivalent to 1.1 year actual field exposure. Consequently, for Texas exposures GRI is using a correlation of 1000hr QUV exposure as equivalent to Iyr of in-service exposure. Clearly, the correlation would be different in less sunny and colder environments.

The failed fPP-1 liner was replaced with a correctly stabilized fPP that, subsequently, performed well.

So how can we evaluate the condition of our exposed liners in a simple and practical manner to ensure they will continue to provide adequate service lifetimes and to get sufficient warning of impending expiration?

For each installation, a baseline needs to be established, and changes from that baseline need to be monitored.

A liner lifetime evaluation program

Rather than be taken by surprise when a liner fails or simply expires, it should be possible to monitor the condition of the liner to obtain a few years of notice for impending expiration. One can then plan for a timely replacement without the potential for accidental environmental damage and undesirable publicity. A program of periodic liner-condition assessment is proposed.

... it should be possible to monitor the condition of the liner to obtain a few years of notice for impending expiration.

While estimated correlations might be made for other locations using historical weather station sunshine and temperature data, there is no question that the best remaining lifetime assessments will be obtained using samples removed from the field installation of interest.

A lifetime in excess of 28yr, demonstrated for a recently-made HDPE geomembrane, is comparable to the present actual service periods of as long as 30-35yr. However, actual lifetimes of as low as ~15yr have also been experienced.

Do service lifetimes now exceeding 30yr mean that we might expect to see another round of stress cracking failures as exposed liners finally oxidize sufficiently on the surface to initiate stress cracking?

This would be frustrating after resolving the early 1980s problems with stress cracking failures at welds and stone protrusions when the liners contracted at low temperatures, but it is the way end-of-life will become apparent. And will that be soon or in another 5-20 years? It would be useful to know.

For baseline data, it would be useful to have some archive material to test, but that is not usually available. Manufacturers often discard retained samples after about 5 years. Perhaps facility owners should be encouraged to keep retained samples at room temperature and out of sunlight. The next best thing is to use material from the anchor trench or elsewhere that has not experienced extremes in temperature and that has not been exposed to UV radiation or to expansion/contraction stresses.

Less satisfactory options are to use the original NSF 54 specifications, the manufacturer's specifications, or the GRI-GM13 specifications at the appropriate time of liner manufacturing. The concern with using these specifications is that while aged material may meet them, there is no indication of whether the measured values have significantly decreased from the actual as-manufactured values that generally significantly exceed the specification.

A final option for the baseline would be to use the values at the time of the first liner assessment.

The first liner condition assessment would consist of a site visit during which a general visual examination would be done together with a mechanical probing of the edges of welds. A visual examination would include the black/gray shades of different panels that might indicate low carbon contents.

A closer examination should be done using a loupe (small magnifier) on suspect areas such as wrinkle peaks, the tops and edges of multiple extrusion weld beads, and the apex-down creases of round die-manufactured sheet.

The last detail is significant because the combination of oxidizing surface and exposed surface tension when the liner contracts at low temperatures and the crease is pulled flat can be one of the first locations to crack. The apex-up creases do not fail at the same time because the oxidized exposed surface is under compression (or less tension) when the crease is flattened out.

Appropriate samples for detailed laboratory testing will be removed.

It may be appropriate to do a water lance electrical integrity survey on the exposed sideslopes, but this would only be effective on single liners, and on double liners with a composite primary liner, a conductive geomembrane, or a geocomposite with a conductive geotextile on top.

A sampling and testing regime

A liner lifetime evaluation program should be simple, meaningful, and cost-effective.

While it will initially require expert polymer materials science/engineering input to analyze the test data and to define the critical parameters, it should ultimately be possible to use an expert system to automatically make predictions using the input test data.

Small samples will be taken from deep in the anchor trench and from appropriate

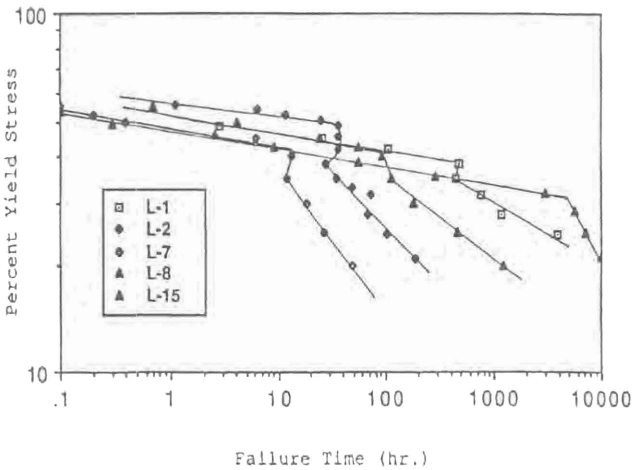


Figure 1 | Standard stress rupture curves for five HDPE geomembranes (Hsuan, et al. 1992)

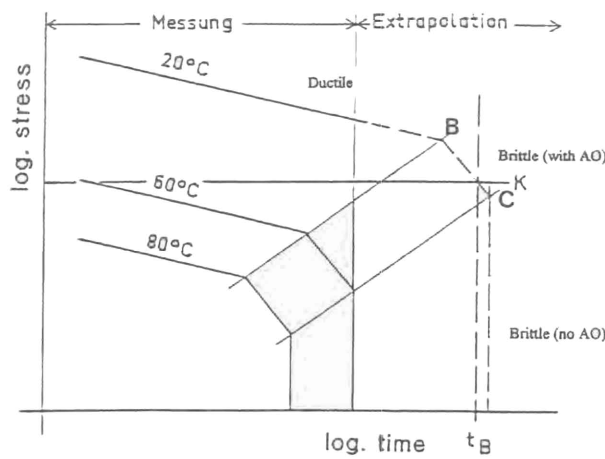


Figure 2 | Stress rupture curves showing third stage (Brittle no AO) oxidized limit. (Gaube, et al. 1985)

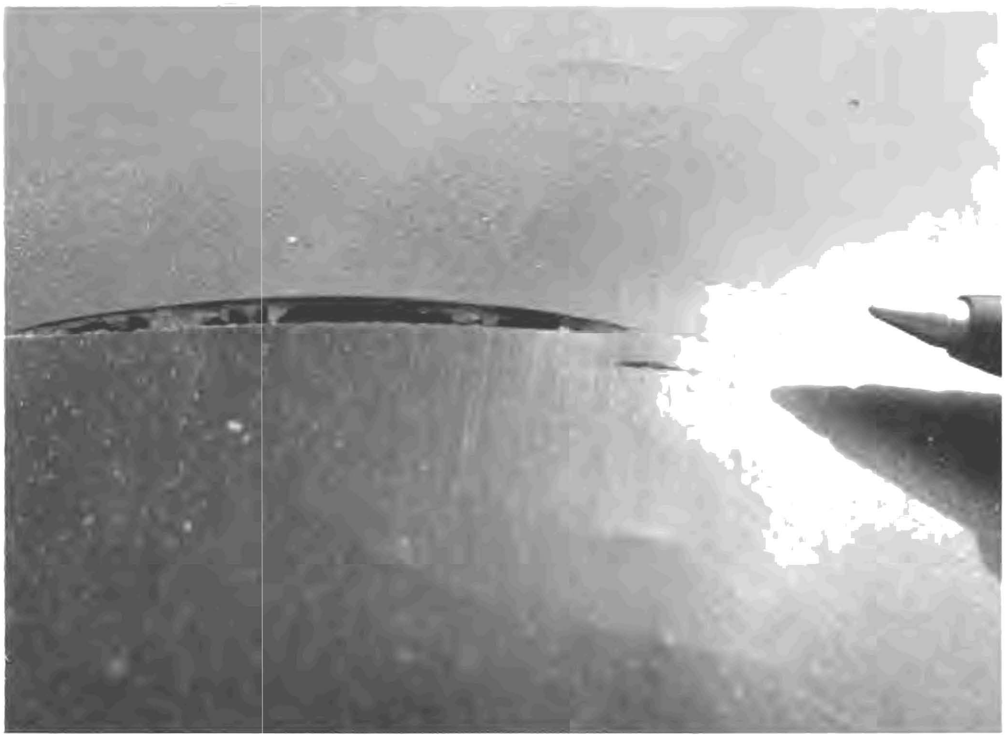


Figure 3 | Stress crack initiated by extruder die line at stone protrusion

exposed locations. Potential sites for future sample removal by the facility owner for future testing will be identified and marked by the expert during the first site visit.

The baseline sample(s) will be tested as follows:

- Single-point stress cracking resistance (SCR) on a molded plaque by ASTM D5397
- High-pressure oxidative induction time (HP-OIT) by ASTM D5885
- Fourier transform infrared spectroscopy (FTIR-ATR) on upper surface to determine carbonyl index (CI) on nonarchive samples only
- Oven aging/HP-OIT (GRI-GM13)
- UV resistance/HP-OIT (GRI-GM13)

The exposed samples will be tested as follows:

- Carbon content (ASTM D1603)
- Carbon dispersion (ASTM D5596)
- Single-point SCR on molded plaque (ASTM D5397)
- Light microscopy of exposed surface, through-thickness cross sections, and thin microsections (~15 µm thick) as necessary
- HP-OIT on 0.5-mm-thick exposed surface layers from basic sheet and from sheet at edge of extruded weld bead (ASTM D5885), preferably at a double-weld bead
- FTIR-ATR on exposed surface to determine CI
- Oven aging/HP-OIT on 0.5mm surface layer (GRI-GM13)
- UV resistance/HP-OIT on 0.5 mm surface layer (GRI-GM13)

Carbon content is done to ensure adequate basic UV protection. Carbon dispersion is done to ensure uniform surface UV protection and to evaluate agglomerates that might act as initiation sites for stress cracking.

HP-OIT is used to assess the remaining amount of stabilizer additives, both in the liner panels and in the sheet adjacent to an extrusion weld. Most stress cracking is observed at the edges of extrusion weld beads in the lower sheet, so it is important to monitor this location.

While standard OIT (ASTM D3895 at 200°C) better assesses the relevant stabilizers effective at processing (melting) and welding temperatures, the relevant changes in effective stabilizer content during continued service, including in the weld zone, will be provided by measurement of HP-OIT. There will be no future high temperature transient where knowledge of S-OIT will be useful. It is expected that the liner adjacent to the weld bead will be more deficient in stabilizer than the panel itself. Therefore, S-OIT is not considered in this program.

Note that HP-OIT is measured on a thin surface layer because the surface layer may be oxidized while the body of the geomembrane may not. If material from the full thickness of the geomembrane is used it could show a significant value of OIT, implying that there is still stabilizer present and that oxidation is far from occurring. However, the surface layer could be fully oxidized with stress cracks already initiated and propagating. A crack will then propagate more easily through unoxidized material than would initiation and propagation occur in unoxidized material.

The fact that the HP-OIT meets a certain specification value in the as-manufactured condition provides no guarantee that thermo- and photo-oxidation protection will be provided for a long time. Stabilizers might be consumed quickly or slowly while providing protection. They may also be consumed quickly to begin with, then more slowly, or vice versa.

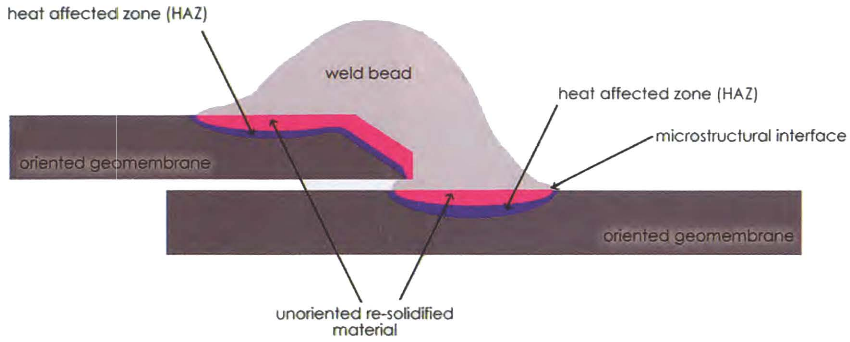


Figure 4 | Schematic of microstructure at extrusion weld

Hence, the need for continuing oven (thermal) aging and UV resistance tests. These two parameters, assessed by measuring retained HP- OIT, are critical to the assessment of remaining service life.

Oven (thermal) aging and UV resistance tests performed in this program will provide an extremely valuable data base that relates laboratory testing to in-service performance and that will further aid in more accurately projecting in-service performance from laboratory testing results.

Special considerations

Because we do not know, by OIT measurements alone, whether the surface layer is or is not oxidized (unless OIT is zero), and since we do not yet know at what level of OIT loss there might be an oxidized surface layer (the database has not yet been generated), FTIR directly on the surface of the geomembrane is performed using the attenuated total reflectance (ATR) technique to deny or confirm the presence of oxidation products (carbonyl groups).

Following the practice of Broutman, et al. (1989) and Duvall (2002) on HDPE pipes, if the ratio of the carbonyl peak at wave number 1760 cm-1 and the C-H stretching (PE) peak at wave number 1410 cm -1 is more than 0.10, there is a sufficiently oxidized surface layer that stress cracking might be initiated. For those familiar with the two slope stress rupture curve (**Figure 1**) where the brittle stress cracking region is the steeper segment below the knee, there is a third vertical part of the curve (**Figure 2**) where the material is fully oxidized and fracture occurs at the slightest stress. This is what will happen at the end of service life. But first note the times to initiation of stress cracking (the knees in the curves) in **Figure 1**—they range from ~10/hr to ~5,000/hr—clearly confirming that all HDPEs are not the same. Some are far more durable than others.

At the end of service life, at some level of OIT, there will be a critically oxidized surface layer that when stressed, such as at low temperatures by an upwards protruding stone, or by flexing due to wind uplift, will initiate a stress crack on the surface that will propagate downward through the geomembrane, as shown by the crack in **Figure 3**.

This crack, initiated at a stress concentrating surface die mark, occurred when the liner contracted at low temperatures, and tightened over an upwardly protruding stone. The straight morphology of the crack, and the ductile break at the bottom surface as the stress in the remaining ligament rose above the knee in the stress rupture curve, are typical of a stress crack. Note the shorter stress cracks initiated along other nearby die marks.

Stress cracks are preferentially initiated along the edges of welds because the adjacent geomembrane has been more depleted of stabilizers during the high temperature welding process. Thus, under further oxidizing service conditions, it will become the first location to

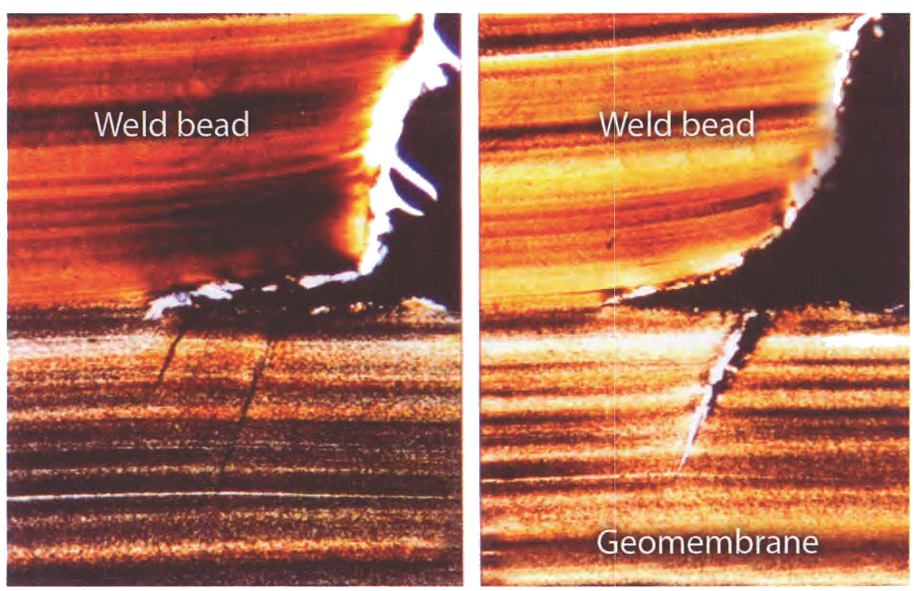


Figure 5 | Typical off-normal angle of precursor crazes (left) and stress crack (right) at edge of extrusion weld.

Type	Specification	Predicted Lifetime in Texas, USA
Side wall exposed	54	5
Side wall concrete side	81	71
Lower launder exposed	16	3
Lower launder concrete side	145	1

Table 2 | S-OIT values on solution and concrete liner surfaces (Peggs, 2008).

be oxidized to the critical level at which stress cracks will be initiated under any applied stress. In addition, the geometrical notches at grinding gouges and at the edges of the bead increase local stresses to critical levels for SC to occur.

I also believe that an internal microstructural flaw exists between the originally oriented geomembrane structure and the pool of more isotropic melted and resolidified material at the edge of the weld zone, as shown schematically in **Figure 4**. Most stress cracks occur at an off-normal angle at the edge of the weld bead that may be related to the angle of this molten-pool to oriented-structure interface (**Figure 5**). It is also known that stress increases the extraction of stabilizers from polyolefin materials.

With all of these agencies acting synergistically, it is not surprising that stress cracking often first occurs adjacent to extrusion welds.

Looking ahead

With the first field assessment test results available to us, and the extent of changes from the baseline sample known, removal of a second set of samples by the facility owner (at locations previously identified and marked by the initial surveyor), will be planned for a future time, probably in 2 or 3 years.

Why 2 or 3 years? In an extreme chemical environment, extensive reductions in S-OIT of studded HDPE concrete protection liners in mine solvent extraction facilities using kerosene/aromatic hydrocarbon/sulfuric acid process solutions at 55°C (131°F) have been observed on the solution and concrete sides of the liner (**Table 2**) within 1 year (Peggs 2008). But it is unlikely that such rapid decreases will be observed in air-exposed material.

With this second set of field samples, and with three sets of data points, practically reliable extrapolations of remaining lifetime can start to be made.

It is expected that a few years of notice for impending failures will be possible.

The key point to note in making these condition assessments is that, while all HDPE geomembranes have very similar conventional index properties, they can have widely variable photo-oxidation, thermal-oxidation, and stress-cracking resistances. Therefore, some HDPEs are more durable than others.

Thus, while one HDPE geomembrane manufactured in 1990 failed after 15 years in 2005, another HDPE geomembrane made in 1990 from a different HDPE resin (or more correctly a medium-density polyethylene [MDPE] resin), and with a better stabilizer additive package, could still have a remaining lifetime of 5, 20, or 30 years.

So, keep a close eye on those exposed liners and we'll learn a great deal more about liner performance and get notice of the end of service lifetime. And if owners can retain some archive material from new installations, so much the better.

References

Broutman L.J., Duvall, D.E., So, P.K. (1989). "Fractographic Study of a Polyethylene Sewer Pipe Failure." SPE Antec, pp 1599-1602.

Duvall, D.E. (2002). "Analyses of Large Diameter Polyethylene Piping Failures." Proceedings of the Society of Plastics Engineers, 60th Annual Technical Conference.

Gaube, E., Gebler, H., Müller, W., and Gondro, C. (1985). "Creep Rupture Strength and Aging of HDPE Pipes 30 Years Experience in Testing of Pipes." Kunststoffe 74 7, pp 412-415.

Koerner, R.M., Hsuan, Y.G., Koerner, G. (2008). "Freshwater and Geosynthetics: A Perfect Marriage." Keynote Lecture at GeoAmericas 2008, IFAI, Roseville, Minn., USA.

Hsuan, Y.G., Koerner, R.M., Lord, A.E., Jr., (1992). "The Notched Constant Tensile Load (NCTL) Test to Evaluate Stress Cracking Resistance." 6th GRI Seminar, MQC/MQA and CQC/CQA of Geosynthetics, Folsom, Pa., USA, pp 244-256.

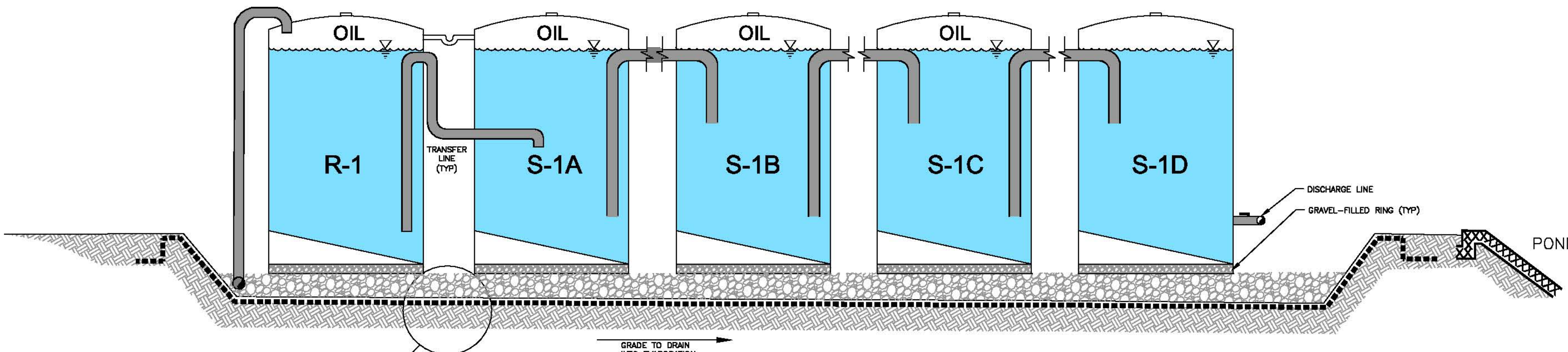
Peggs, I.D., (2008). "The Performance of Concrete Protection liners in Mine SX/EW Mixers and Settlers: The Need for Chemical Resistance Testing." Proceedings of GeoAmericas 2008, IFAI, Roseville, Minn., USA.

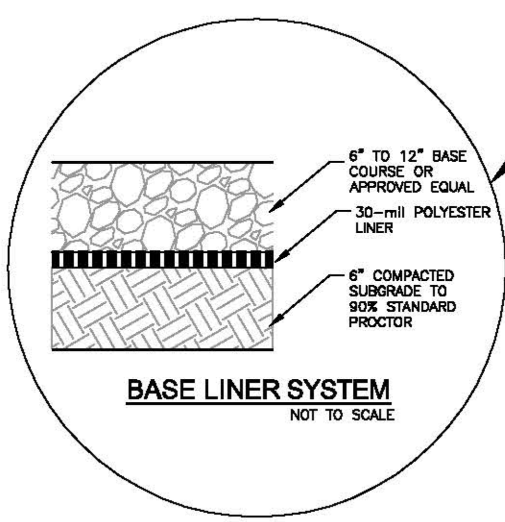
APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.C

TYPICAL RECEIVING TANK INSTALLATION DETAILS





ELEVATION - SECTION A-A'

NOT TO SCALE

TYPICAL RECEIVING TANK INSTALLATION DETAILS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers

213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/02/13	CAD: TANK ELEV.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: DRT	ATTACHMENT III.I.C
APPROVED BY: IKG	gei@gordonenvironmental.com	

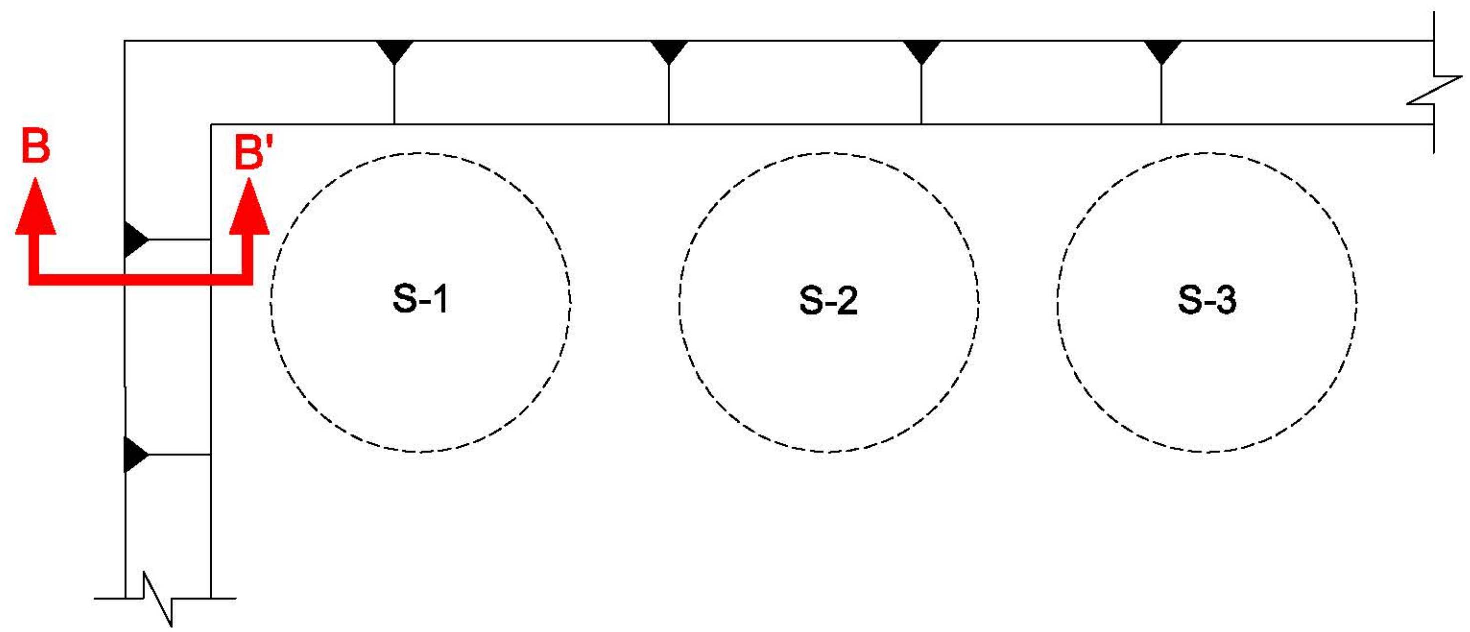
Drawing:P:\acad 2003\542.01.01\PERMIT FIGURES\TANK ELEV.dwg
Date/Time:Oct. 22, 2013-09:47:39

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.D

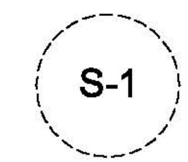
TYPICAL SALES TANK INSTALLATION DETAILS



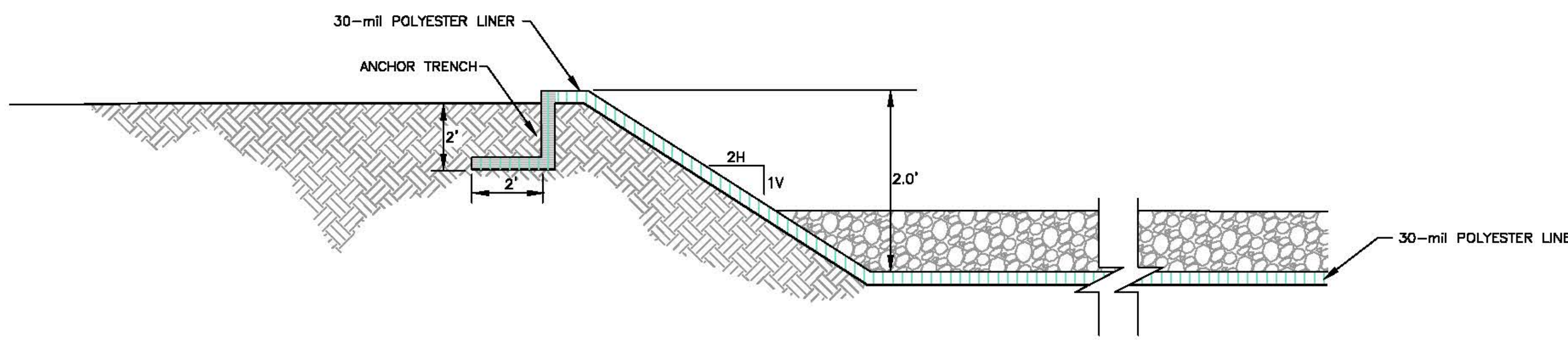
PLAN VIEW

NOT TO SCALE

LEGEND



PROPOSED TANK



CROSS SECTION B-B'

NOT TO SCALE

Drawing:P:\acad 2003\542.01.01\PERMIT FIGURES\TYP TANK.dwg
Date/Time:Oct. 22, 2013-09:50:58

TYPICAL SALES TANK INSTALLATION DETAILS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

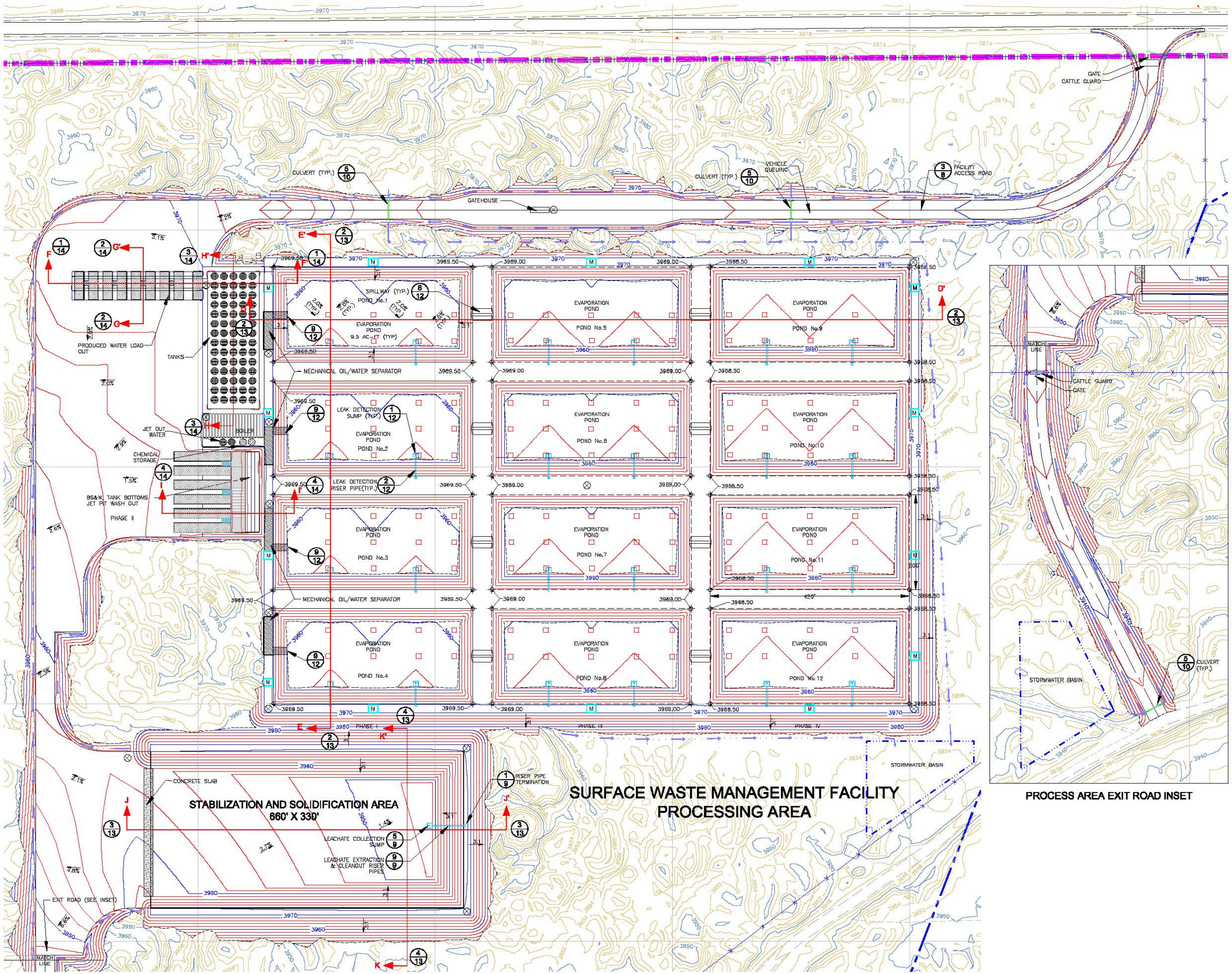
Gordon Environmental, Inc.
Consulting Engineers

213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/02/13	CAD: TYP TANK.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	ATTACHMENT III.I.D
APPROVED BY: IKG	gei@gordonenvironmental.com	

ATTACHMENT III.1.E

SITE SCHEMATIC



PROCESS AREA EXIT ROAD INSET

LEGEND

- SITE BOUNDARY (562 ACRES±)
- DRAINAGE FEATURE SETBACK (67 ACRES±)
- 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- 25' DESIGN CONTOUR
- 5' DESIGN CONTOUR
- TOP/TOE OF SLOPE
- EXISTING FENCE
- PROPOSED FENCE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- DIRECTION OF STORMWATER FLOW
- CULVERT
- CATTLE GUARD
- ROAD SIGN
- HYDROGEN SULFIDE MONITORING STATION
- EVAPORATORS
- PPE AND EMERGENCY EQUIPEMENT
- LEAK DETECTION SUMP & RISER PIPE
- CROSS SECTION LOCATION
- DETAIL NUMBER / SHEET NUMBER
- SITE GRID (N 650,500; E 738,000)

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

VOLUME

ENTRANCE ROAD		
CUT VOLUME	11583	CU. YD.
FILL VOLUME	6290	CU. YD.
NET VOLUME	5293	CU. YD.<CUT>
EVAP PONDS		
CUT VOLUME	182856	CU. YD.
FILL VOLUME	106752	CU. YD.
NET VOLUME	76104	CU. YD.<CUT>
PROCESSING AREA		
CUT VOLUME	51153	CU. YD.
FILL VOLUME	24228	CU. YD.
NET VOLUME	26925	CU. YD.<CUT>
STABILIZATION AND SOLIDIFICATION AREA		
CUT VOLUME	11996	CU. YD.
FILL VOLUME	51002	CU. YD.
NET VOLUME	39006	CU. YD.<FILL>
EXIT ROAD		
CUT VOLUME	18072	CU. YD.
FILL VOLUME	0	CU. YD.
NET VOLUME	18072	CU. YD.<CUT>

NORTH

0 100 200

SITE SCHEMATIC

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/13/2014	CAD: SITE SCHEMATIC .DWG	PROJECT #: 542.01.01
DRAWN BY: DM	REVIEWED BY: MRH	ATTACHMENT III.1.E
APPROVED BY: IKG	gei@gordonenvironmental.com	

NOT FOR CONSTRUCTION
Drawing:P:\acad 2008\542.01.01\RAI 1\SITE SCHEMATIC.dwg
Date/Time:Jun. 13, 2014-11:31:06 ; LAYOUT: D (LS)

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.F

TANK CAPACITY CALCULATIONS

ATTACHMENT III.1.F
Tank Capacity Calculations
DNCS Environmental Solutions

DNCS is a surface waste management facility.

A. Produced Water is delivered by trucking companies into one of twelve proposed heated Produced Water Receiving Tanks located within a bermed, lined containment area:

Proposed Tank No.	Volume	Permitted
R-1	1000 bbls	Permitted under this Application
R-2	1000 bbls	Permitted under this Application
R-3	1000 bbls	Permitted under this Application
R-4	1000 bbls	Permitted under this Application
R-5	1000 bbls	Permitted under this Application
R-6	1000 bbls	Permitted under this Application
R-7	1000 bbls	Permitted under this Application
R-8	1000 bbls	Permitted under this Application
R-9	1000 bbls	Permitted under this Application
R-10	1000 bbls	Permitted under this Application
R-11	1000 bbls	Permitted under this Application
R-12	1000 bbls	Permitted under this Application

i. The Receiving tanks serve to gravity separate solids and oil from the water. Solids collect in the bottoms and oil floats to the tops of the receiving tanks.
ii. The Receiving Tanks bottoms are solidified and taken to the OCD permitted Landfill.
iii. The Receiving Tanks are set on gravel or sand pads on top of a lined bermed impermeable pad.

B. Water from each Receiving Tanks flows in series through four additional Settling Tanks to remove oil prior to discharge in the mechanical oil water separator:

Proposed Tank No.	Volume	Permitted
S-1A	1000 bbls	Permitted under this Application
S-1B	1000 bbls	Permitted under this Application
S-1C	1000 bbls	Permitted under this Application
S-1D	1000 bbls	Permitted under this Application
S-2A	1000 bbls	Permitted under this Application
S-2B	1000 bbls	Permitted under this Application
S-2C	1000 bbls	Permitted under this Application
S-2D	1000 bbls	Permitted under this Application
S-3A	1000 bbls	Permitted under this Application
S-3B	1000 bbls	Permitted under this Application
S-3C	1000 bbls	Permitted under this Application
S-3D	1000 bbls	Permitted under this Application
S-4A	1000 bbls	Permitted under this Application
S-4B	1000 bbls	Permitted under this Application
S-4C	1000 bbls	Permitted under this Application
S-4D	1000 bbls	Permitted under this Application
S-5A	1000 bbls	Permitted under this Application
S-5B	1000 bbls	Permitted under this Application
S-5C	1000 bbls	Permitted under this Application
S-5D	1000 bbls	Permitted under this Application
S-6A	1000 bbls	Permitted under this Application
S-6B	1000 bbls	Permitted under this Application
S-6C	1000 bbls	Permitted under this Application

S-6D	1000 bbls	Permitted under this Application
S-7A	1000 bbls	Permitted under this Application
S-7B	1000 bbls	Permitted under this Application
S-7C	1000 bbls	Permitted under this Application
S-7D	1000 bbls	Permitted under this Application
S-8A	1000 bbls	Permitted under this Application
S-8B	1000 bbls	Permitted under this Application
S-8C	1000 bbls	Permitted under this Application
S-8D	1000 bbls	Permitted under this Application
S-9A	1000 bbls	Permitted under this Application
S-9B	1000 bbls	Permitted under this Application
S-9C	1000 bbls	Permitted under this Application
S-9D	1000 bbls	Permitted under this Application
S-10A	1000 bbls	Permitted under this Application
S-10B	1000 bbls	Permitted under this Application
S-10C	1000 bbls	Permitted under this Application
S-10D	1000 bbls	Permitted under this Application
S-11A	1000 bbls	Permitted under this Application
S-11B	1000 bbls	Permitted under this Application
S-11C	1000 bbls	Permitted under this Application
S-11D	1000 bbls	Permitted under this Application
S-12A	1000 bbls	Permitted under this Application
S-12B	1000 bbls	Permitted under this Application
S-12C	1000 bbls	Permitted under this Application
S-12D	1000 bbls	Permitted under this Application

i. The Settling Tanks increase the detention time available to provide additional gravity separation of oil from the water,
ii. The Settling Tank bottoms are taken to the Stabilization/Solidification Area.
iii. The Settling Tanks are set on gravel or sand pads on top of a lined bermed impermeable pad.

C. The separated oil flows into one of five heated Crude Oil Receiving Tanks:

Proposed Tank No.	Volume	Permitted
C-1	1000 bbls	Permitted under this Application
C-2	1000 bbls	Permitted under this Application
C-3	1000 bbls	Permitted under this Application
C-4	1000 bbls	Permitted under this Application
C-5	1000 bbls	Permitted under this Application

i. The Crude Oil Receiving Tanks are set inside the proposed lined containment berm.
ii. The Crude Oil Receiving Tanks are interconnected at the top of the tanks for oil removal.
iii. Water recovered from the Crude Oil Receiving Tanks is redirected to the Produced Water Receiving Tanks.
iv. Sludges recovered from the Crude Oil Receiving Tanks are stabilized, solidified and sent for landfill disposal.

D. The water from the Settling Tanks is discharged through one of up to four Dissolved Air Floatation (DAF) Units.

Proposed Tank No.	Volume	Permitted
D-1	10 bbls	Permitted under this Application
D-2	10 bbls	Permitted under this Application
D-3	10 bbls	Permitted under this Application
D-4	10 bbls	Permitted under this Application

i. The DAF Units are situated on the lined Evaporation Pond berm in a location where any leackage would drain
ii. The DAF use air bubles to lift any remaining oil from the water prior to dischage into one of four Ponds.
iii. The oil containing foam generated by the DAF is collected and discharged into the Crude Oil Receiving Tanks for further processing.

E.

Proposed Pond No.	Storage Volume	Permitted
P-1	73,700 bbls	Permitted under this Application

P-2	73,700 bbls	Permitted under this Application
P-3	73,700 bbls	Permitted under this Application
P-4	73,700 bbls	Permitted under this Application
P-5	73,700 bbls	Permitted under this Application
P-6	73,700 bbls	Permitted under this Application
P-7	73,700 bbls	Permitted under this Application
P-8	73,700 bbls	Permitted under this Application
P-9	73,700 bbls	Permitted under this Application
P-10	73,700 bbls	Permitted under this Application
P-11	73,700 bbls	Permitted under this Application
P-12	73,700 bbls	Permitted under this Application

i. Surface aeration and bleach are used to maintain water chemistry parameters:
   :O_2 at or above 0.5 ppm one foot off the bottom of the pond.

   :pH above 8
ii. H2S monitors are placed around the pond covering the four major points on the compass.
iii. The H2S monitors continually monitor the ambient air.
iv. Two chlorine monitors are placed around the ponds covering the North and West borders.
v. Treatment capacity of each Pond is 73,994 bbls (~9.5 acre feet)
vi. 3.5 Feet of Freeboard is proposed, storage volume does include freeboard
vii. Volume including freeboard is 122,640 bbls (15.76 acre-feet)per pond
viii. Inside grade shall be no steeper than 3H:1V
ix. Levees shall have an outside grade no steeper than 3H:1V
x. Levees' tops shall be wide enough to install an anchor trench and provide adequate room for inspection/maintenance.
xi. Liner seams shall be minimized and oriented up and down, not across a slope
   Each pond shall have a:
   :primary liner (60-mil HDPE liner, UV resistant)
   :secondary liner (60-mil HDPE liner, UV resistant)
xii. Slope shall be 2% (2 ft V for 100 ft H)
xiii. A mechanical evaporation system shall be installed in each pond to enhance evaporation.
xiv. Approximate size of each pond is 200 x 420 feet x 7.6 feet deep

F. Bleach for H2S management is stored in two proposed chemical tanks:

Proposed Tank No.	Volume	Permitted
B-1	60 bbls	Permitted under this Application
B-2	60 bbls	Permitted under this Application

i. The Chemical Tanks are set on a bermed concrete pad that drains into the pond.
ii. The Bleach is pumped through lines to discharge points in each of the ponds.

G. Water from Pond 1 (P-1) is:
i. Pumped through lines to floating evaporators in Ponds 2, 3, and 4 (P-2, P-3, P-4).
ii. Three floating evaporators are situated in each Pond.
iii. Water that does not evaporate from Ponds 2, 3, or 4 is pumped to floating evaporators in Ponds 5 and 6.
iv. Water that does not evaporate from Ponds 5 and 6 is pumped to floating evaporators in Ponds 7 and 8.
v. Water that does not evaporate from Ponds 7 and 8 is pumped to floating evaporators in Ponds 9 and 10.

H. The Jet-Out Pit receives discharges from tankers bringing oil contaminated drilling mud, BS&W, tank bottoms and washout from tank cleanings.

Proposed Pit No.	Volume	Permitted
J-1	1000 bbls	Permitted under this Application

Proposed Tank No.	Volume	Permitted
WW-1	1000 bbls	Permitted under this Application
FW-1	1000 bbls	Permitted under this Application

i. Wash-Water for the Jet-Out Pit is recycled through a line from Pond-10 to WW-1. A pump connected to WW-1 pumps the water through a line to one of six wash-out stations for use cleaning the tankers.
ii. Fresh-Water for the Jet-Out Pit is discharged from the water supply through an air gap into FW-1. A pump connected to FW-1 pumps the water through a line to one of six wash-out stations for use cleaning the tanks.
ii. Oil from the Jet-Out Pit is transferred through a line to the Crude Oil Receiving Tanks for further Processing..
iii. Water from the Jet-Out Pit is transferred through a line to the Produced Water Receiving Tanks for processing.
iv. Sludges and sediments from the Jet Out Pit is removed with a bucket loader and transferred to the waste stabilization area for stabilization, solidification and disposal.

I. Oil from the Crude Oil Receiving Tanks C1-C5 completed the dewatering process with the finished product transferred to the Oil Sales Tanks.

Proposed Tank No.	Volume	Permitted
S-1	1000 bbls	Permitted under this Application
S-2	1000 bbls	Permitted under this Application
S-3	1000 bbls	Permitted under this Application
S-4	1000 bbls	Permitted under this Application
S-5	1000 bbls	Permitted under this Application

i. The proposed Oil Sales Tanks are set inside the lined berm next to the Crude Oil Receiving Tanks.
ii. Oil is removed from the Oil Sales tank to a tanker at the Oil Sales Load-Out

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.G

PIPE WALL THICKNESS INFORMATION

PVC PIPE DIMENSIONS

Nominal Pipe Size	Wall Thickness		Outside Diameters	Tolerance	
	Minimum	Tolerance	Average OD	Average	Out-of-Roundness
ASTM D 1785, PVC PIPE, SCHEDULE 40					
1	0.133	+0.020	1.315	±0.005	±0.010
1¼	0.140	+0.020	1.660	±0.005	±0.012
1½	0.145	+0.020	1.900	±0.006	±0.012
2	0.154	+0.020	2.375	±0.006	±0.012
2½	0.203	+0.024	2.875	±0.007	±0.015
3	0.216	+0.026	3.500	±0.008	±0.015
3½	0.226	+0.027	4.000	±0.008	±0.050
4	0.237	+0.028	4.500	±0.009	±0.050
5	0.258	+0.031	5.563	±0.010	±0.050
6	0.280	+0.034	6.625	±0.011	±0.050
8	0.322	+0.039	8.625	±0.015	±0.075
10	0.365	+0.044	10.750	±0.015	±0.075
12	0.406	+0.049	12.750	±0.015	±0.075
ASTM D 1785, PVC PIPE, SCHEDULE 80					
1	0.179	+0.021	1.315	±0.005	±0.010
1¼	0.191	+0.023	1.660	±0.005	±0.012
1½	0.200	+0.024	1.900	±0.006	±0.012
2	0.218	+0.026	2.375	±0.006	±0.012
2½	0.276	+0.033	2.875	±0.007	±0.015
3	0.300	+0.036	3.500	±0.008	±0.015
3½	0.318	+0.038	4.000	±0.008	±0.015
4	0.337	+0.040	4.500	±0.009	±0.015
5	0.375	+0.045	5.563	±0.010	±0.030
6	0.432	+0.052	6.625	±0.011	±0.035
8	0.500	+0.060	8.625	±0.015	±0.075
10	0.593	+0.071	10.750	±0.015	±0.075
12	0.687	+0.082	12.750	±0.015	±0.075
ASTM D 2241, PVC PIPE (SDR-PR), SDR 21 (200)					
1	0.063	+0.020	1.315	±0.005	±0.015
1¼	0.079	+0.020	1.660	±0.005	±0.015
1½	0.090	+0.020	1.900	±0.006	±0.030
2	0.113	+0.020	2.375	±0.006	±0.030
2½	0.137	+0.020	2.875	±0.007	±0.030
3	0.167	+0.020	3.500	±0.008	±0.030
3½	0.190	+0.023	4.000	±0.008	±0.050
4	0.214	+0.026	4.500	±0.009	±0.050
5	0.265	+0.032	5.563	±0.010	±0.050

Table A-2 (cont'd)
PIPE WEIGHTS AND DIMENSIONS *(IPS)*
PE3608 *(BLACK)*

OD Nominal in.	OD Actual in.	OD Actual mm.	SDR	Nominal ID in.	Nominal ID mm.	Minimum Wall in.	Minimum Wall mm.	Weight lb. per foot	Weight kg. per meter
			7	2.44	61.98	0.500	12.70	2.047	3.047
			7.3	2.48	63.08	0.479	12.18	1.978	2.943
			9	2.68	67.96	0.389	9.88	1.656	2.464
			9.3	2.70	68.63	0.376	9.56	1.609	2.395
			11	2.83	71.77	0.318	8.08	1.387	2.065
3	3.500	88.90	11.5	2.85	72.51	0.304	7.73	1.333	1.984
			13.5	2.95	74.94	0.259	6.59	1.153	1.716
			15.5	3.02	76.74	0.226	5.74	1.015	1.511
			17	3.06	77.81	0.206	5.23	0.932	1.386
			21	3.15	79.93	0.167	4.23	0.764	1.136
			26	3.21	81.65	0.135	3.42	0.623	0.927
			7	3.14	79.68	0.643	16.33	3.384	5.037
			7.3	3.19	81.11	0.616	15.66	3.269	4.865
			9	3.44	87.38	0.500	12.70	2.737	4.073
			9.3	3.47	88.24	0.484	12.29	2.660	3.958
			11	3.63	92.27	0.409	10.39	2.294	3.413
4	4.500	114.30	11.5	3.67	93.23	0.391	9.94	2.204	3.280
			13.5	3.79	96.35	0.333	8.47	1.906	2.836
			15.5	3.88	98.67	0.290	7.37	1.678	2.497
			17	3.94	100.05	0.265	6.72	1.540	2.292
			21	4.05	102.76	0.214	5.44	1.262	1.879
			26	4.13	104.98	0.173	4.40	1.030	1.533
			32.5	4.21	106.84	0.138	3.52	0.831	1.237
			7	3.88	98.51	0.795	20.19	5.172	7.697
			7.3	3.95	100.27	0.762	19.36	4.996	7.435
			9	4.25	108.02	0.618	15.70	4.182	6.224
			9.3	4.29	109.09	0.598	15.19	4.065	6.049
			11	4.49	114.07	0.506	12.85	3.505	5.216
5	5.563	141.30	11.5	4.54	115.25	0.484	12.29	3.368	5.012
			13.5	4.69	119.11	0.412	10.47	2.912	4.334
			15.5	4.80	121.97	0.359	9.12	2.564	3.816
			17	4.87	123.68	0.327	8.31	2.353	3.502
			21	5.00	127.04	0.265	6.73	1.929	2.871
			26	5.11	129.78	0.214	5.43	1.574	2.343
			32.5	5.20	132.08	0.171	4.35	1.270	1.890
			7	4.62	117.31	0.946	24.04	7.336	10.917
			7.3	4.70	119.41	0.908	23.05	7.086	10.545
			9	5.06	128.64	0.736	18.70	5.932	8.827
			9.3	5.11	129.92	0.712	18.09	5.765	8.579
			11	5.35	135.84	0.602	15.30	4.971	7.398
6	6.625	168.28	11.5	5.40	137.25	0.576	14.63	4.777	7.109
			13.5	5.58	141.85	0.491	12.46	4.130	6.147
			15.5	5.72	145.26	0.427	10.86	3.637	5.413
			17	5.80	147.29	0.390	9.90	3.338	4.967
			21	5.96	151.29	0.315	8.01	2.736	4.072
			26	6.08	154.55	0.255	6.47	2.233	3.322
			32.5	6.19	157.30	0.204	5.18	1.801	2.680

See ASTM D3035, F714 and AWWA C-901/906 for OD and wall thickness tolerances.
Weights are calculated in accordance with PPI TR-7.

Table A-2 (cont'd)
PIPE WEIGHTS AND DIMENSIONS *(IPS)*
PE3608 *(BLACK)*

OD Nominal in.	OD Actual in.	OD Actual mm.	SDR	Nominal ID in.	Nominal ID mm.	Minimum Wall in.	Minimum Wall mm.	Weight lb. per foot	Weight kg. per meter
			7	6.01	152.73	1.232	31.30	12.433	18.503
			7.3	6.12	155.45	1.182	30.01	12.010	17.872
			9	6.59	167.47	0.958	24.34	10.054	14.962
			9.3	6.66	169.14	0.927	23.56	9.771	14.541
			11	6.96	176.85	0.784	19.92	8.425	12.538
8	8.625	219.08	11.5	7.04	178.69	0.750	19.05	8.096	12.049
			13.5	7.27	184.67	0.639	16.23	7.001	10.418
			15.5	7.45	189.11	0.556	14.13	6.164	9.174
			17	7.55	191.76	0.507	12.89	5.657	8.418
			21	7.75	196.96	0.411	10.43	4.637	6.901
			26	7.92	201.21	0.332	8.43	3.784	5.631
			7	7.49	190.35	1.536	39.01	19.314	28.743
			7.3	7.63	193.75	1.473	37.40	18.656	27.764
			9	8.22	208.73	1.194	30.34	15.618	23.242
			9.3	8.30	210.81	1.156	29.36	15.179	22.589
			11	8.68	220.43	0.977	24.82	13.089	19.478
10	10.750	273.05	11.5	8.77	222.71	0.935	23.74	12.578	18.717
			13.5	9.06	230.17	0.796	20.23	10.875	16.184
			15.5	9.28	235.70	0.694	17.62	9.576	14.251
			17	9.41	239.00	0.632	16.06	8.788	13.078
			21	9.66	245.48	0.512	13.00	7.204	10.721
			26	9.87	250.79	0.413	10.50	5.878	8.748
			32.5	10.05	255.24	0.331	8.40	4.742	7.058
			7	8.89	225.77	1.821	46.26	27.170	40.433
			7.3	9.05	229.80	1.747	44.36	26.244	39.056
			9	9.75	247.57	1.417	35.98	21.970	32.695
			9.3	9.84	250.03	1.371	34.82	21.353	31.777
			11	10.29	261.44	1.159	29.44	18.412	27.400
12	12.750	323.85	11.5	10.40	264.15	1.109	28.16	17.693	26.330
			13.5	10.75	272.99	0.944	23.99	15.298	22.767
			15.5	11.01	279.56	0.823	20.89	13.471	20.047
			17	11.16	283.46	0.750	19.05	12.362	18.397
			21	11.46	291.16	0.607	15.42	10.134	15.081
			26	11.71	297.44	0.490	12.46	8.269	12.305
			32.5	11.92	302.73	0.392	9.96	6.671	9.928
			7	9.76	247.90	2.000	50.80	32.758	48.750
			7.3	9.93	252.33	1.918	48.71	31.642	47.089
			9	10.70	271.84	1.556	39.51	26.489	39.420
			9.3	10.81	274.54	1.505	38.24	25.745	38.313
			11	11.30	287.07	1.273	32.33	22.199	33.036
14	14.000	355.60	11.5	11.42	290.05	1.217	30.92	21.332	31.746
			13.5	11.80	299.76	1.037	26.34	18.445	27.449
			15.5	12.09	306.96	0.903	22.94	16.242	24.170
			17	12.25	311.25	0.824	20.92	14.905	22.181
			21	12.59	319.70	0.667	16.93	12.218	18.183
			26	12.86	326.60	0.538	13.68	9.970	14.836
			32.5	13.09	332.40	0.431	10.94	8.044	11.970

See ASTM D3035, F714 and AWWA C-901/906 for OD and wall thickness tolerances.
Weights are calculated in accordance with PPI TR-7.

ATTACHMENT III.1.H

TECHNICAL DATA AND SPECIFICATIONS FOR XR GEOMEMBRANES

Technical Data and Specifications for

XR® Geomembranes

XR-3®
XR-5®
XR-3® PW

Industrial, Municipal and Potable Water Grade Geomembranes

Seaman Corporation
1000 Venture Blvd.
Wooster, Ohio 44691
(330) 262-1111
www.xr-5.com

Seaman Corp. XR Geomembranes

Section 1 - Product Overview/Applications

- All XR Geomembrane products are classified as an Ethylene Interpolymer Alloy (EIA)
- XR-5 grade is high strength and chemically resistant for maximum resistance to high temperature, and broad chemical resistance, including acids, oils and methane
- XR-3 grade for moderate chemical resistant requirement applications such as stormwater and domestic wastewater
- NSF 61 approved XR-3 PW grade for potable water contact
- Heat weldable-thermal weldable for seams as strong as the membrane. Factory panels over 15,000 square feet (1400 sq meters) for less field seaming
- Stability is excellent, with low thermal expansion-contraction properties
- 30+ year application history

Product Application Chart

	XR-5			XR-3	XR-3 PW
	8130	**8138**	**6730**	**8228**	**8130**
High Puncture Resistance	X	X	X		X
UV Resistance	X	X	X	X	X
High Strength Applications	X	X	X		X
Floating Covers (Nonpotable)	X	X	X	X	
Diesel/Jet Fuel Containment	X	X	X		
Industrial Wastewater	X	X	X		
Stormwater	X	X	X	X	
Municipal/Domestic Wastewater	X	X	X	X	
Floating Diversion Baffles/Curtains	X		X		X
Potable Water					X
<-65 Deg F Applications	Contact Seaman Corp.				
Chemically Resistant Applications	X	X	X		

Section 2 - Physical Properties

Part 1- Material Specifications

Property	Test Method	8130 XR-5	8138 XR-5	6730 XR-5
Base Fabric Type Base Fabric Weight	ASTM D 751	Polyester 6.5 oz/yd^2 nominal (220 g/m^2 nominal)	Polyester 6.5 oz/yd^2 nominal (220 g/m^2 nominal)	Polyester 7 oz/yd^2 nominal (235 g/m^2 nominal)
Thickness	ASTM D 751	30 mils min. (0.76 mm min.)	40 mils nom. (1.0 mm nom.)	30 mils min. (0.76 mm min.)
Weight	ASTM D 751	30.0 +- 2 oz/sq yd (1017 +- 2 g/m^2)	38.0 +- 2 oz/sq yd (1288 +- 70 g/m^2)	30.0 +- 2 oz/sq yd (1017 +- 70 g/m^2)
Tear Strength	ASTM D 751 Trap Tear	40/55 lbs. min. (175/245 N min.)	40/55 lbs. min. (175/245 N min.)	
Breaking Yield Strength	ASTM D 751 Grab Tensile	550/550 lbs. min. (2,447/2,447 N min.)	550/550 lbs. min. (2,447/2,447 N min.)	600/550 lbs. min. (2,670/2,447 N min.)
Low Temperature Resistance	ASTM D 2136 4 hrs-1/8" Mandrel	Pass @ -30° F Pass @ -35° C	Pass @ -30° F Pass @ -35° C	Pass @ -30° F Pass @ -35° C
Dimensional Stability	ASTM D 1204 100° C-1 Hr.	0.5% max. each direction	0.5% max. each direction	0.5% max. each direction
Hydrostatic Resistance	ASTM D 751 Procedure A	800 psi min. (5.51 MPa min.)	800 psi min. (5.51 MPa min.)	800 psi min. (5.51 MPa min.)
Blocking Resistance	ASTM D 751 180° F	#2 Rating max.	#2 Rating max.	#2 Rating max.
Adhesion-Ply	ASTM D 413 Type A	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)
Adhesion (minimum) Heat Welded Seam	ASTM D 751 Dielectric Weld	40 lbs./2in. RF weld min. (17.5 daN/5 cm min.)	40 lbs./2in. RF weld min. (17.5 daN/5 cm min.)	15 lbs./in. RF weld min. (15 daN/5 cm min.)
Dead Load Seam Strength	ASTM D 751, 4-Hour Test	Pass 220 lbs/in @ 70° F (Pass 980 N/2.54 cm @ 21° C) Pass 120 lbs/in @ 160° F (Pass 534 N/2.54 cm @ 70° C)	Pass 220 lbs/in @ 70° F (Pass 980 N/2.54 cm @ 21° C) Pass 120 lbs/in @ 160° F (Pass 534 N/2.54 cm @ 70° C)	
Bonded Seam Strength	ASTM D 751 Procedure A, Grab Test Method	550 lbs. min. (2,450 N min.)	550 lbs. min. (2,450 N min.)	550 lbs. min. (2,560 N min.)

Abrasion Resistance	ASTM D 3389 H-18 Wheel 1 kg Load	2,000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss	2,000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss	2,000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss
Weathering Resistance	Carbon-Arc ASTM G 153	8,000 hours min. with no appreciable changes or stiffening or cracking of coating	8000 hours min. with no appreciable change or stiffening or cracking of coating	8000 hours min. with no appreciable change or stiffening or cracking of coating
Water Absorption	ASTM D 471, Section 12 7 Days	0.025 kg/m^2 max. @70° F/21° C 0.14 kg/m^2 max at 212° F/100° C	0.025 kg/m^2 max. @70° F/21° C 0.14 kg/m^2 max at 212° F/100° C	0.025 kg/m^2 max. @70° F/21° C 0.14 kg/m^2 max at 212° F/100° C
Wicking	ASTM D 751	1/8" max (0.3 cm max)	1/8" max. (0.3 cm max.)	1/8" max. (0.3 cm max.)
Bursting Strength	ASTM D 751 Ball Tip	750 lbs. min. (3,330 N min.)	750 lbs. min. (3,330 N min.)	750 lbs. min. (3,330 N min.)
Puncture Resistance	ASTM D 4833	275 lbs. min. 1,200 N min.	275 lbs. min. 1,200 N min.	275 lbs. min. 1,200 N min.
Coefficient of Thermal Expansion/ Contraction	ASTM D 696	8 x 10^{-6} in/in/° F max. (1.4 x 10^{-5} cm/cm/° C max.)	8 x 10^{-6} in/in/° F max. (1.4 x 10^{-5} cm/cm/° C max.)	8 x 10^{-6} in/in/° F max. (1.4 x 10^{-5} cm/cm/° C max.)
Environmental/Chemical Resistant Properties		See Chemical Resistance Table, Page 8	See Chemical Resistance Table, Page 8	See Chemical Resistance Table, Page 8
Puncture Resistance	FED-STD-101C Method 2031	350 lbs. (approx.)	350 lbs. (approx.)	
Cold Crack	ASTM D 2136 4 Hrs, 1/8" Mandrel	Pass at -30° F/-34° C	Pass @ -30° F/-34° C	Pass @ -30° F/-34° C

Section 2 - Physical Properties

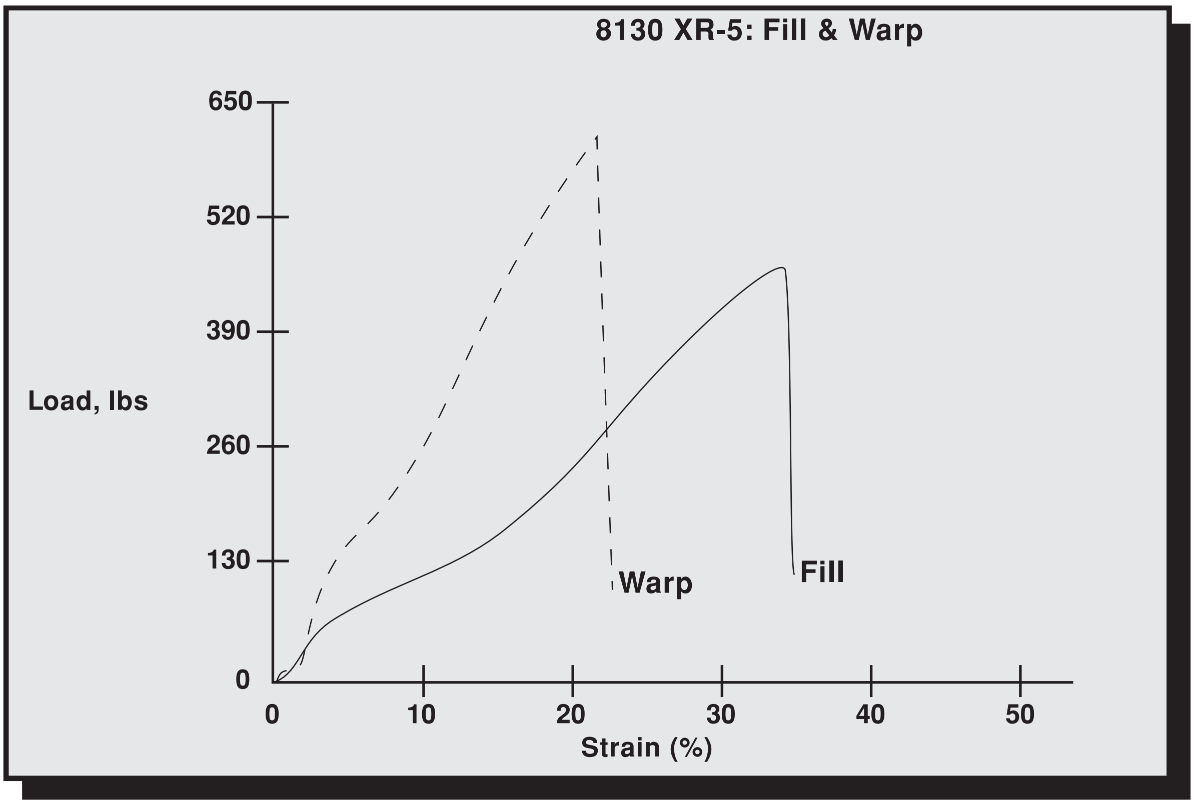
Part 1- Material Specifications (cont.)

Property	Test Method	8130 XR-3 PW	8228 XR-3
Base Fabric Type Base Fabric Weight	ASTM D 751	Polyester 6.5 oz/yd² nominal (220 g/m² nominal)	Polyester 3.0 oz/yd² nominal (100 g/m² nominal)
Thickness	ASTM D 751	30 mils min. (0.76 mm min.)	30 mils min. (0.76 mm min.)
Weight	ASTM D 751	30.0 +- 2 oz./sq. yd. (1017 +- 70 g/sq. m)	28.0 +- 2 oz./sq. yd. (950 +- 70 g/sq. m)
Tear Strength	ASTM D 751 Trap Tear	40/55 lbs. min. (175/245 N min.)	30/30 lbs. nom. (133/133 N nom.)
Breaking Yield Strength	ASTM D 751 Grab Tensile	550/550 lbs. min. (2,447/2447 N min.)	250/200 lbs. min. (1,110/890 N min.)
Low Temperature Resistance	ASTM D 2136 4hrs-1/8" Mandrel	Pass @ -30° F (Pass @ -35° C)	Pass @ -25° F (Pass @ -32° C)
Dimensional Stability	ASTM D 1204 100° C-1 hr.	0.5% max. each direction	5% max. each direction
Hydrostatic Resistance	ASTM D 751 Method A	800 psi min. (5.51 MPa min.)	300 psi min. (2.07 MPa min.)
Blocking Resistance	ASTM D 751 180° F	#2 Rating max.	#2 Rating max.
Adhesion-Ply	ASTM D 413 Type A	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)	12 lbs./in. (approx.) (10 daN/5 cm approx.)
Adhesion- Heat Welded Seam	ASTM D 751 Dielectrc Weld	40 lbs./2in. min. (17.5 daN/5 cm min.)	10 lbs./in min. (9 daN/5 cm min.)
Dead Load Seam Strength	ASTM D 751, 4-Hour Test	Pass 220 lbs/in. @ 70° F (Pass 980 N/2.54 cm @ 21° C) Pass 120 lbs/in. @ 160° F (Pass 534 N/2.54 cm @ 70° C)	Pass 100 lbs/in @ 70° F (Pass 445 N @ 21° C) Pass 50 lb @ 160° F (Pass 220 N @ 70° C)
Bonded Seam Strength	ASTM D 751 Procedure A, Grab Test Method	550 lbs. min. (2,450 N min.)	250 lbs. (approx.) (1,112 N min.)

Abrasion Resistance	ASTM D 3389 H-18 Wheel 1 kg Load	2000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss	2000 cycles min.
Weathering Resistance	ASTM G 153	8000 hours min. with no appreciable change or stiffening or cracking of coating	8000 hours min.
Water Absorption	ASTM D 471, Section 12 7 Days	0.025 kg/m^2 max. @ 70° F/21° C 0.14 kg/m^2 max @ 212° F/100° C	0.05 kg/m^2 max. @ 70° F/21° C (approx.) 0.28 kg/m^2 max. @ 212° F/100° C (approx.)
Wicking	ASTM D 751	1/8" max. (0.3 cm max.)	1/8" max (0.3 cm max.)
Bursting Strength	ASTM D 751 Ball Tip	750 lbs. min. (3330 N min.)	350 lbs. (approx.) (1557 N min.)
Puncture Resistance	ASTM D 4833	275 lbs. min. 1200 N min.	50 lb typ. (225 N typ.)
Coefficient of Thermal Expansion/ Contraction	ASTM D 696	8×10^{-6} in/in/° F max. (1.4×10^{-5} cm/cm/° C max.)	8×10^{-6} in/in/° F max. (approx.) (1.4×10^{-5} cm/cm/° C max. approx.)
Environmental/Chemical Resistant Properties	ASTM D 741 7-Day Total Immersion With Exposed Edges	NSF 61 approved for potable water	Crude oil 5% max. weight gain Diesel fuel 5% max. weight gain
Puncture Resistance	FTMS 101C Method 2031	350 lbs. (approx.)	205 lbs. (approx.)
Tongue Tear	ASTM D 751		50 lbs. (approx.)

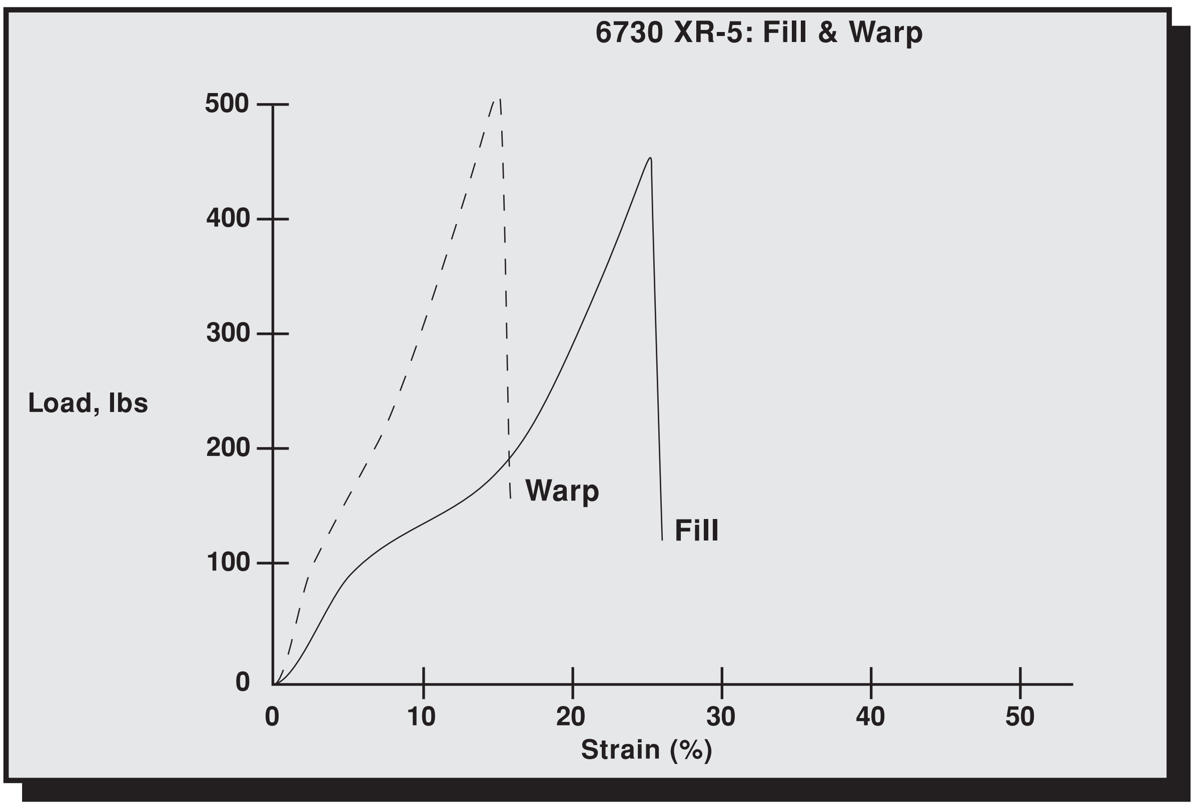
Part 2 - Elongation Properties Test

8130 XR-5



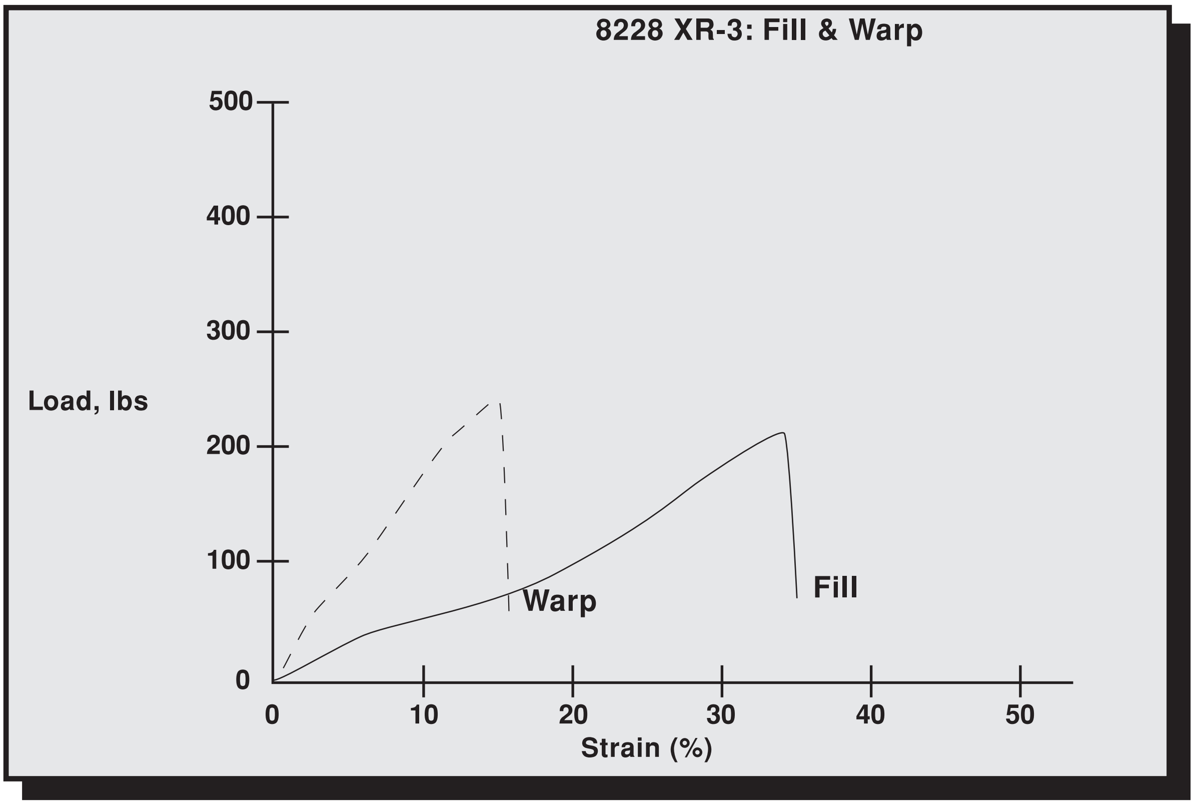
Part 2 - Elongation Properties Test

6730 XR-5



Part 2 - Elongation Properties Test

8228 XR-3



Section 3 - Chemical/Environmental Resistance

Part 1 - XR-5® Fluid Resistance Guidelines

The data below is the result of laboratory tests and is intended to serve only as a guide. No performance warranty is intended or implied. The degree of chemical attack on any material is governed by the conditions under which it is exposed. Exposure time, temperature, and size of the area of exposure usually varies considerably in application, therefore, this table is given and accepted at the user's risk. Confirmation of the validity and suitability in specific cases should be obtained. Contact a Seaman Corporation Representative for recommendation on specific applications.

When considering XR-5 for specific applications, it is suggested that a sample be tested in actual service before specification. Where impractical, tests should be devised which simulate actual service conditions as closely as possible.

EXPOSURE	RATING
AFFF	A
Acetic Acid (5%)	B
Acetic Acid (50%)	C
Ammonium Phosphate	T
Ammonium Sulfate	T
Antifreeze (Ethylene Glycol)	A
Animal Oil	A
Aqua Regia	X
ASTM Fuel A (100% Iso-Octane)	A
ASTM Oil #2 (Flash Pt. 240° C)	A
ASTM Oil #3	A
Benzene	X
Calcium Chloride Solutions	T
Calcium Hydroxide	T
20% Chlorine Solution	A
Clorox	A
Conc. Ammonium Hydroxide	A
Corn Oil	A
Crude Oil	A
Diesel Fuel	A
Ethanol	A
Ethyl Acetate	C
Ethyl Alcohol	A
Fertilizer Solution	A
#2 Fuel Oil	A
#6 Fuel Oil	A
Furfural	X
Gasoline	B
Glycerin	A
Hydraulic Fluid- Petroleum Based	A
Hydraulic Fluid- Phosphate Ester Based	C
Hydrocarbon Type II (40% Aromatic)	C
Hydrochloric Acid (50%)	A
Hydrofluoric Acid (5%)	A
Hydrofluoric Acid (50%)	A
Hydrofluosilicic Acid (30%)	A
Isopropyl Alcohol	T
Ivory Soap	A
Jet A	A
JP-4 Jet Fuel	A
JP-5 Jet Fuel	A
JP-8 Jet Fuel	A
Kerosene	A
Magnesium Chloride	T
Magnesium Hydroxide	T
Methanol	A
Methyl Alcohol	A
Methyl Ethyl Ketone	X
Mineral Spirits	A
Naphtha	A
Nitric Acid (5%)	B
Nitric Acid (50%)	C
Perchloroethylene	C
Phenol	X
Phenol Formaldehyde	B
Phosphoric Acid (50%)	A
Phosphoric Acid (100%)	C
Phthalate Plasticizer	C
Potassium Chloride	T
Potassium Sulphate	T
Raw Linseed Oil	A
SAE-30 Oil	A
Salt Water (25%)	B
Sea Water	A
Sodium Acetate Solution	T
Sodium Bisulfite Solution	T
Sodium Hydroxide (60%)	A
Sodium Phosphate	T
Sulphuric Acid (50%)	A
Tanic Acid (50%)	A
Toluene	C
Transformer Oil	A
Turpentine	A
Urea Formaldehyde	A
UAN	A
Vegetable Oil	A
Water (200°F)	A
Xylene	X
Zinc Chloride	T

Ratings are based on visual and physical examination of samples after removal from the test chemical after the samples of Black XR-5 were immersed for 28 days at room temperature. Results represent ability of material to retain its performance properties when in contact with the indicated chemical.

Rating Key:
A – Fluid has little or no effect
B – Fluid has minor to moderate effect
C – Fluid has severe effect
T – No data - likely to be acceptable
X – No data - not likely to be acceptable

Vapor Transmission Data

Tested according to ASTM D814-55 Inverted Cup Method

Perhaps a more meaningful test is determination of the diffusion rate of the liquid through the membrane. The vapor transmission rate of Style 8130 XR-5® to various chemicals was determined by the ASTM D814-55 inverted cup method. All tests were run at room temperature and results are shown in the table.

Chemical	8130 XR-5 Black g/hr/m2
Water	0.11
#2 Diesel Fuel	0.03
Jet A	0.11
Kerosene	0.15
Hi-Test Gas	1.78
Ohio Crude Oil	0.03
Low-Test Gas	5.25
Raw Linseed Oil	0.01
Ethyl Alcohol	0.23
Naphtha	0.33
Perchlorethylene	38.58
Hydraulic Fluid	0.006
100% Phosphoric Acid	7.78
50% Phosphoric Acid	0.43
Ethanol (E-96)	0.65
Transformer Oil	0.005
Isopropyl Alcohol	0.44
JP4 (E-96)	0.81
JP8 (E-96)	0.42
Fuel B (E-96)	6.28
Fuel C (E-96)	7.87

Note: The tabulated values are measured Vapor Transmission Rates (VTR). Normal soil testing methods to determine permeability are impractical for synthetic membranes. An "equivalent hydraulic" permeability coefficient can be calculated but is not a direct units conversion. Contact Seaman Corporation for additional technical information.

Seam Strength

Style 8130 XR-5 Black Seam Strength After Immersion

Two pieces of Style 8130 were heat sealed together (seam width 1 inch overlap) and formed into a bag. Various oils and chemicals were placed in the bags so that the seam area was entirely covered. After 28 days at room temperature, the chemicals were removed and one inch strips were cut across the seam and the breaking strength immediately determined. Results are listed below.

Chemical	Seam Strength
None	340 Lbs. Fabric Break- No Seam Failure
Kerosene	355 Lbs. Fabric Break- No Seam Failure
Ohio Crude Oil	320 Lbs. Fabric Break- No Seam Failure
Hydraulic Fluid- Petroleum Based	385 Lbs. Fabric Break- No Seam Failure
Toluene	0 Lbs. Adhesion Failure
Naphtha	380 Lbs. Fabric Break- No Seam Failure
Perchloroethylene	390 Lbs. Fabric Break- No Seam Failure

Even though 1-inch overlap seams are used in the tests to study the accelerated effects, it is recommended that XR-5 be used with a 2-inch nominal overlap seam in actual application. In some cases where temperatures exceed 160ºF and the application demands extremely high seam load, it may be necessary to use a wider width seam.

Long Term Seam Adhesion

11 Years Immersion
ASTM D 751
Lbs./In.

Seam samples of 8130 XR-5® were dielectrically welded together and totally immersed in the liquids for 11 years. The samples were taken out, dried for 24 hours and visually observed for any signs of swelling, cracking, stiffening or degradation of the coating. The coating showed no appreciable degradation and no stiffening, swelling, cracking or peeling.

The adhesion, or resistance to separation of the coating from the base cloth, was then measured by ASTM D 751. Results show 8130 XR-5 maintains seam strength over this long period (11 years).

	Control	Crude Oil	JP-4 Jet Fuel	Diesel Fuel	Kerosene	Naphtha
8130 XR-5	20+	18	33	25	40	33*

Values in lbs./in.

*The naphtha sample was sticky.

We believe this information is the best currently available on the subject. We offer it as a suggestion in any appropriate experimentation you may care to undertake. It is subject to revision as additional knowledge and experience are gained. We make no guarantee of results and assume no obligation or liability whatsoever in connection with this information.

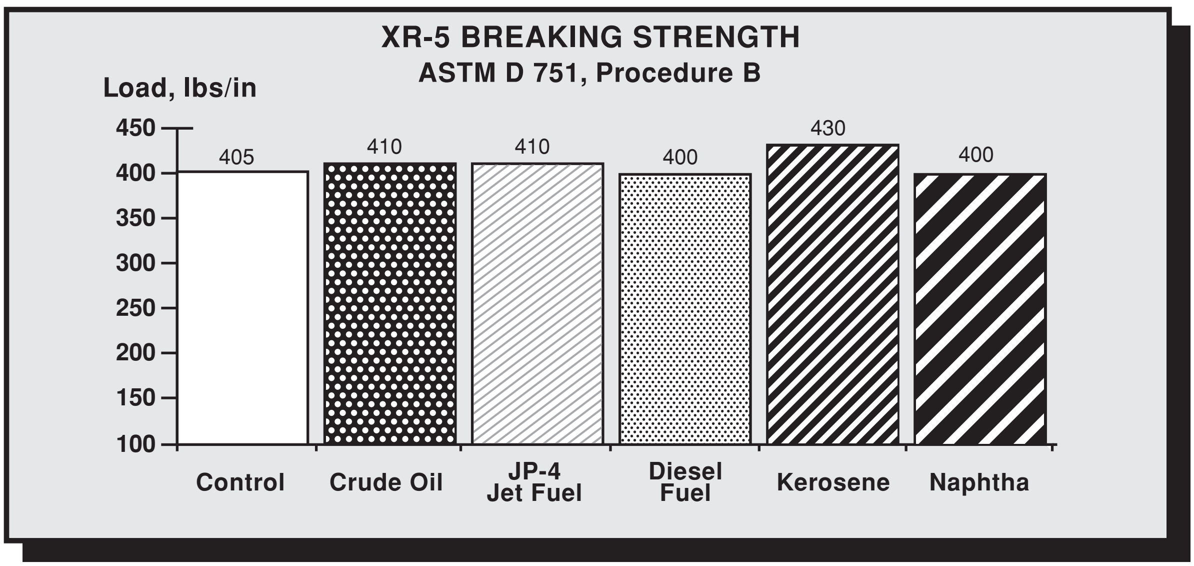
Fuel Compatibility - Long Term Immersion

Test: Samples of 8130 XR-5® Black were immersed in Diesel Fuel, JP-4 Jet Fuel, Crude Oil, Kerosene, and Naphtha for 6 1/2 years.

The samples were then taken out of the test chemicals, blotted and dried for 24 hours. The samples were observed for blistering, swelling, stiffening, cracking or delamination of the coating from the fiber.

Results: It was found in all cases that the 8130 XR-5, after immersion for six years, maintained its strength and there was no evidence of blistering, swelling, stiffening, cracking or delamination.

The strip tensile strength, or breaking strength, of the samples was measured after six years of immersion and the following are the results.



XR-3 Chemical Resistance Statement (Summary)

XR-3® is recommended for moderate chemical resistant applications such as stormwater and municipal wastewater and is not recommended for prolonged contact with pure solutions. XR-3 PW® membranes are recommended only for contact with drinking water and are resistant to low levels of chlorine found in drinking water. XR-5 has a broad range of chemical resistance which is detailed in this section.

Part 2: XR-5® Comparative Chemical Resistance

Chemical Resistance Chart

Comparative Chemical Resistance

	XR-5	HDPE	PVC	Hypalon	Polypropylene
Kerosene	A	B	C	C	C
Diesel Fuel	A	A	C	C	C
Acids (General)	A	A	A	B	A
Naphtha	A	A	C	B	C
Jet Fuels	A	A	C	B	C
Saltwater, 160° F	A	A	C	B	A
Crude Oil	A	B	C	B	C
Gasoline	B	B	C	C	C

A= Excellent **B= Moderate** **C= Poor**

Source: Manufacturer's Literature

XR-5 data based on conditions detailed in Section 3, Part 1.

Part 3: Weathering Resistance

Accelerated Weathering Test

XR-5 has been tested in the carbon arc weatherometer for over 10,000 hours of exposure and in the Xenon weatherometer for over 12,000 hours of exposure. The sample showed no loss in flexibility and no significant color change. Based on field experience of Seaman Corporation products and similar weatherometer exposure tests, XR-5 should have an outdoor weathering life significantly longer than competitive geomembranes, particularly in tropical or subtropical applications.

EMMAQUA Testing: ASTM E-838-81 was performed on a modified form of XR-5, FiberTite, used in the single-ply roofing industry. After 3 million Langleys in Arizona, no signs of degradation were noted with no evidence of cracking, blistering, swelling or adhesion delamination failure of the coating.

Natural Exposure

After over 17 years as a holding basin at a large oil company in the Texas desert, XR-5 showed no signs of environmental stress cracking, thermal expansion/contraction, or low yield strength problems. Temperature ranges from near zero to over 100° F.

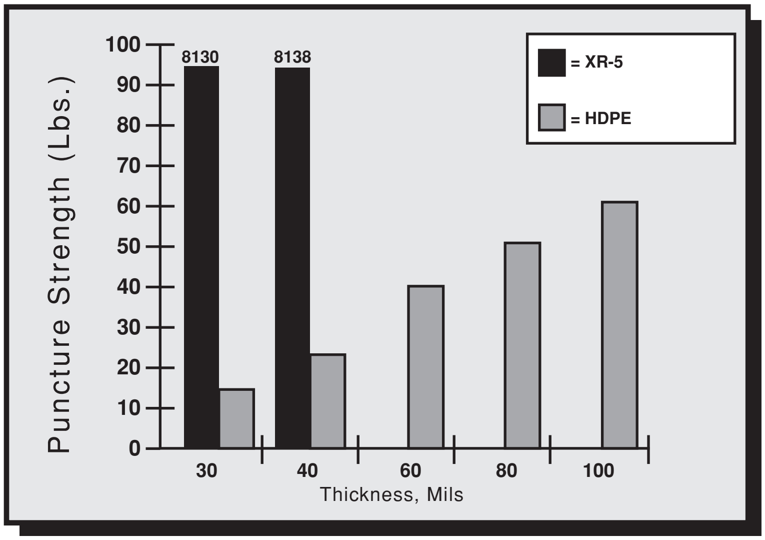
In service approximately 17 years in a solar pond application at a research facility in Ohio, UV exposed samples, as well as immersed samples, retained over 90% of the tensile strength. Examination of the material determined there was little effect on the coating compound. The solar pond was exposed to temperatures from below zero to over 100° F.

XR5 was exposed for 12½ years in Sarasota, Florida, on a weathering rack, facing the southern direction at 45°. No significant color loss, cracking, crazing, blistering, or adhesion delamination failure of the coating was noted.

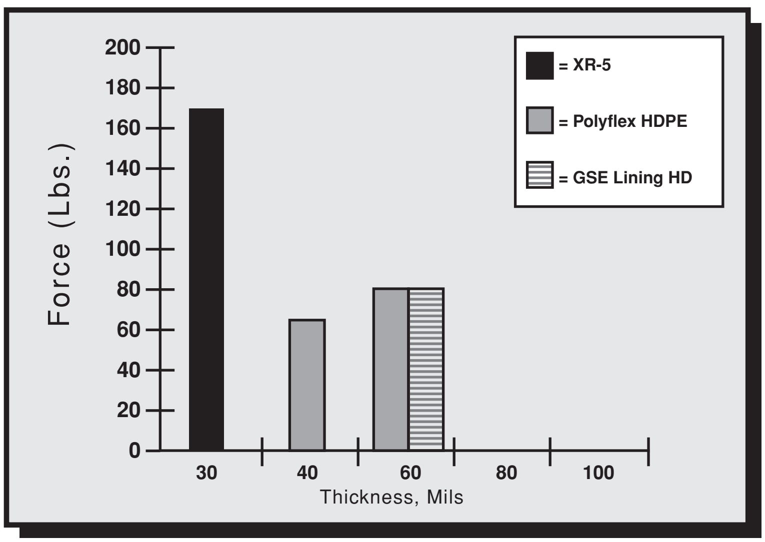
Section 4 - Comparative Physical Properties

XR-5/HDPE Comparative Properties

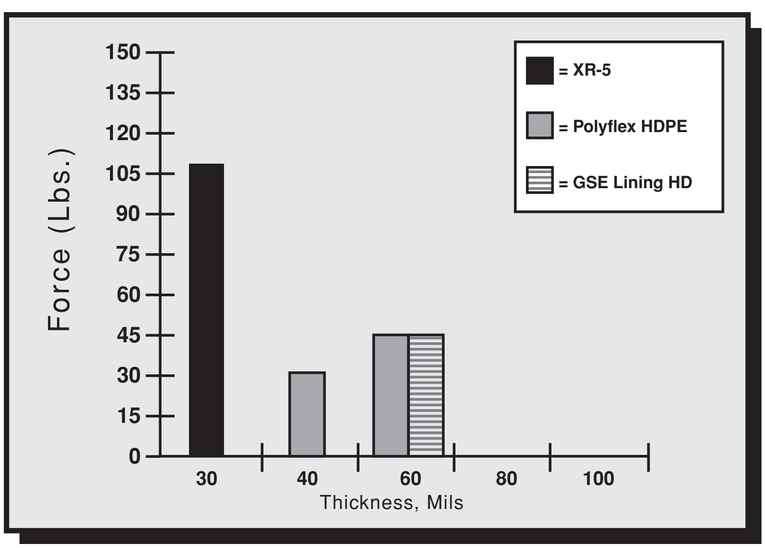
Puncture Resistance



1. ASTM D 751, Screwdriver Tip, 45° Angle (Room Temperature) Puncture Resistance, XR5 vs. HDPE



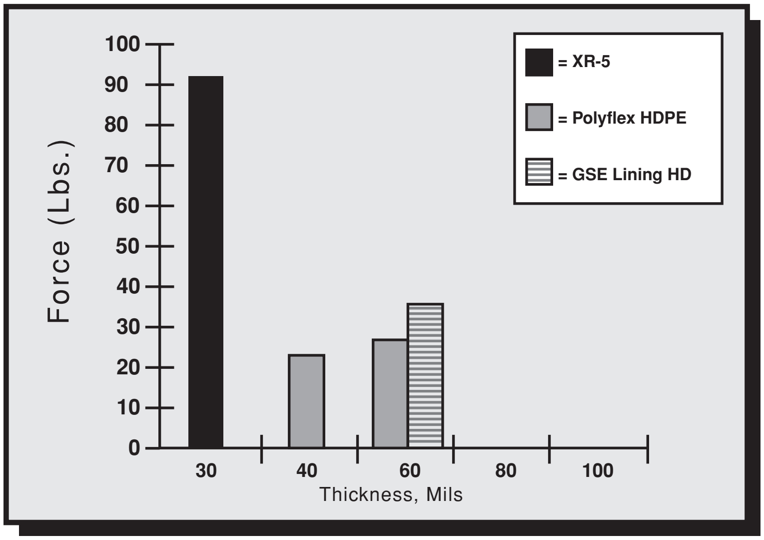
2. FED-STD-101C Method 2065 (Room Temperature)*



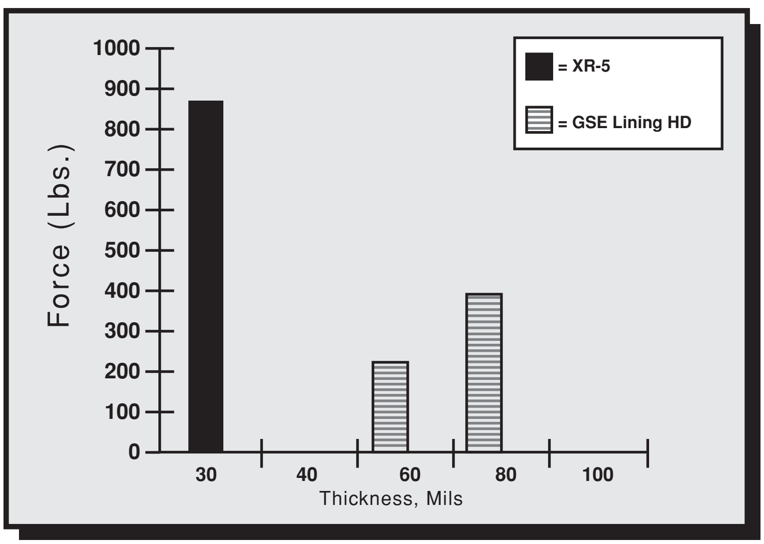
3. FED-STD-101C Method 2065 (70°C)*

* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

GSE is a registered trademark of GSE Lining Technology, Inc.

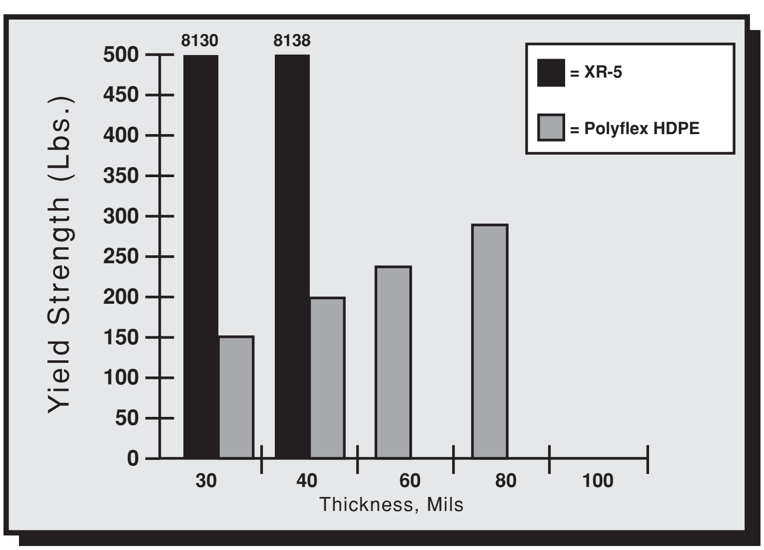


4. FED-STD-101C Method 2065 (100°C)*



5. ASTM D 751 Ball Burst Puncture

Yield Strength

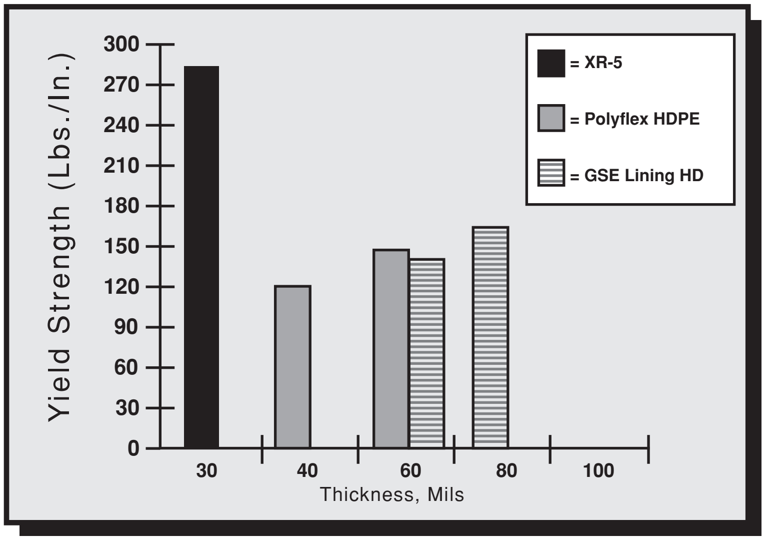


1. Yield Strength, XR-5 vs. HDPE

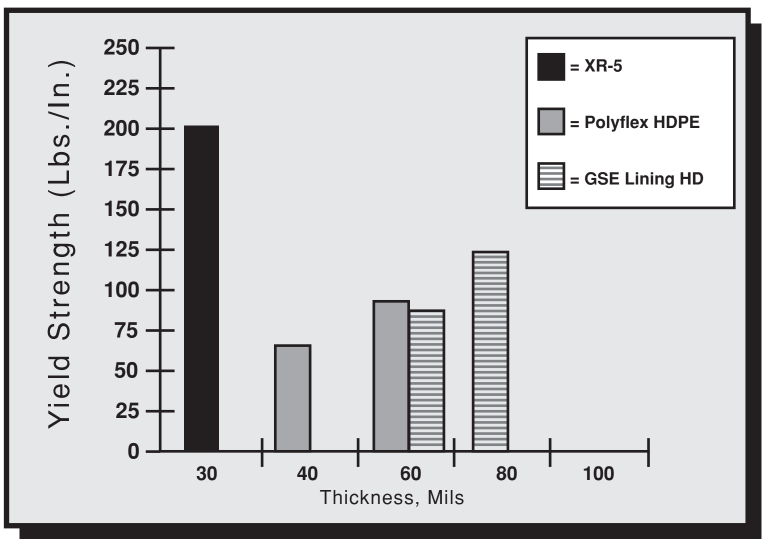
Test Method: Grab Tensile, ASTM D 751, 70° C

* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

GSE is a registered trademark of GSE Lining Technology, Inc.

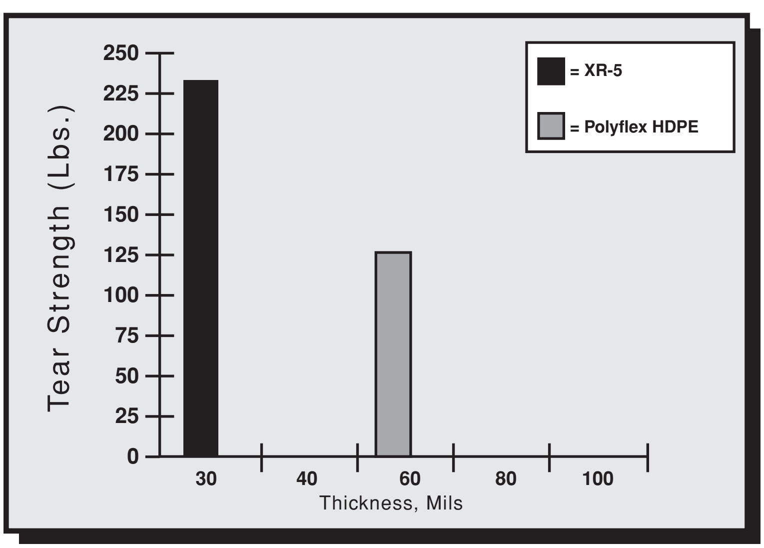


2. Strip Tensile, ASTM D 751, Room Temperature*



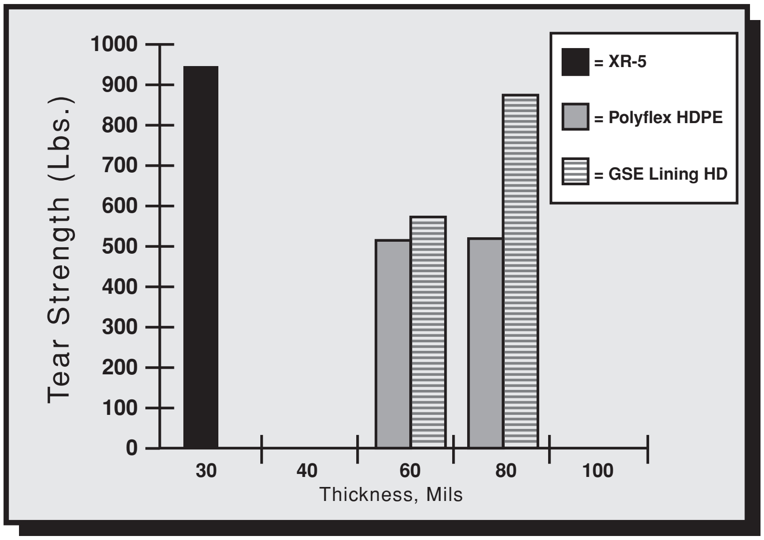
3. Strip tensile, ASTM D 751, 70°C*

Tear Strength

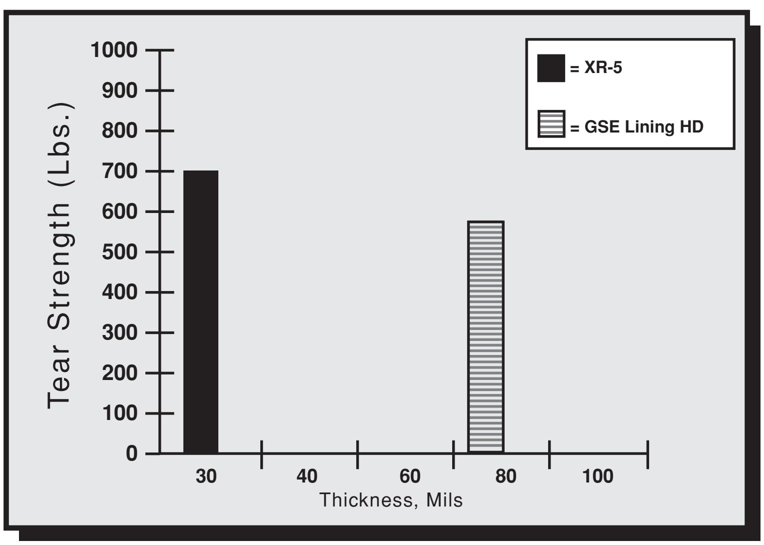


1. Tongue Tear (8" x 10" Specimens), ASTM D 751, Room Temperature*

* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware



1. Graves Tear, ASTM D 624, Die C, Room Temperature*

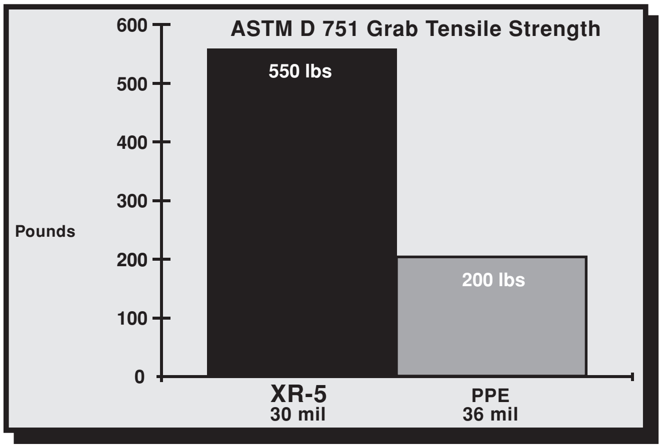


2. Graves Tear, ASTM D 624, Die C, 70°C*

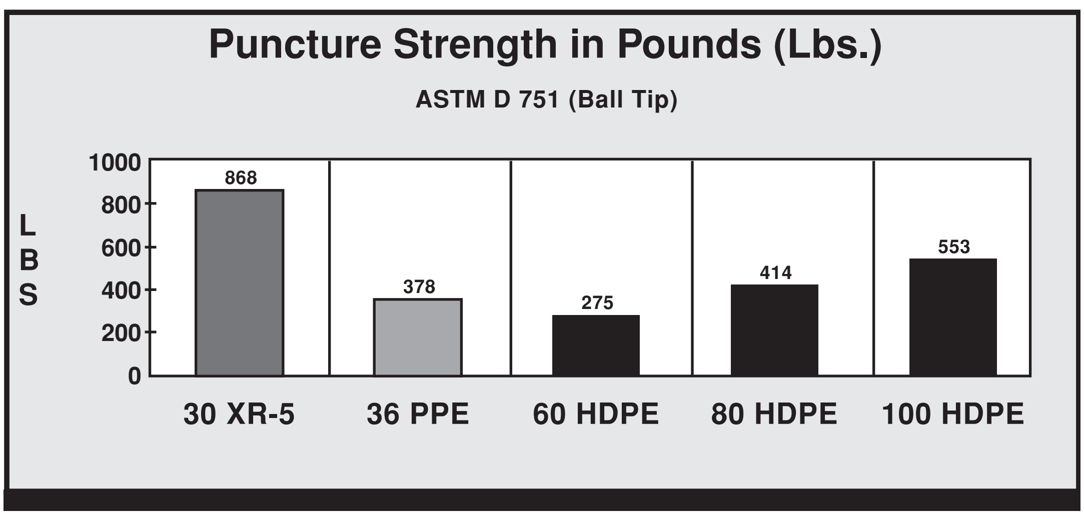
* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

GSE is a registered trademark of GSE Lining Technology, Inc.

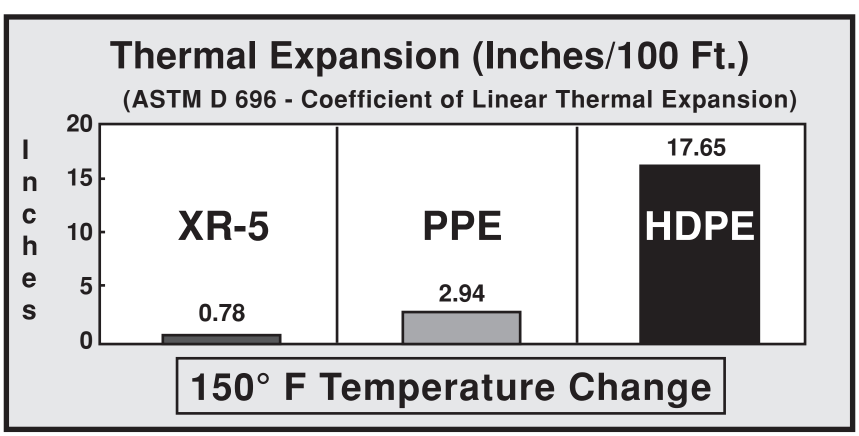
Grab Strength – XR-5® vs. Polypropylene Tensile



Puncture Strength Comparison



Coated Fabric Thermal Stability



Specification For Geomembrane Liner

(Sample specification: 8130 XR-5®. For other product specifications, go to www.xr-5.com)

General

1.01 Scope Of Work
Furnish and install flexible membrane lining in the areas shown on the drawings. All work shall be done in strict accordance with the project drawings, these specifications and membrane lining fabricator's approved shop drawings.

Geomembrane panels will be supplied sufficient to cover all areas, including appurtenances, as required in the project, and shown on the drawings. The fabricator/installer of the liner shall allow for shrinkage and wrinkling of the field panels.

1.02 Products
The lining material shall be 8130 XR-5 as manufactured by Seaman Corporation (1000 Venture Boulevard, Wooster, OH 44691; 330-262-1111), with the following physical specifications:

Base- (Type)	Polyester
Fabric Weight (ASTM D 751)	6.5 oz./sq. yd.
Finished Coated Weight (ASTM D 751)	30 ± 2 oz./sq. yd.
Trapezoid Tear (ASTM D 751)	40/55 lbs. min.
Grab Yield Tensile (ASTM D 751, Grab Method Procedure A)	550/550 lbs. min.
Elongation @ Yield (%)	20% min.
Adhesion- Heat Seam (ASTM D 751, Dielectric Weld)	40 lbs./2in. weld min.
Adhesion- Ply (ASTM D 413, Type A)	15 lbs./in. or film tearing bond
Hydrostatic Resistance (ASTM D 751, Method A)	800 psi min.
Puncture Resistance (ASTM D 4833)	275 lbs. min.
Bursting Strength (ASTM D 751 Ball Tip)	750 lbs. min.
Dead Load (ASTM D 751) Room Temperature	220 lbs. min.
(2" overlap seam, 4 hours) 160°F	120 lbs. min.
Bonded Seam Strength (ASTM D 751 Grab Test Method, Procedure A)	575 lbs. min.
Low Temperature (ASTM D 2136, 4 hours- 1/8" Mandrel)	Pass @ -30°F
Weathering Resistance ASTM G 153 Carbon Arc	8,000 hours min. With no appreciable changes or stiffening or cracking of coating
Dimensional Stability (ASTM D 1204, 212°F 1 Hour, Each Direction)	0.5% max.
Water Absorption (ASTM D 471, 7 Days)	0.025 kg/m^2 max. @ 70°F 0.14 kg/m^2 max. @ 212°F
Abrasion Resistance ASTM D 3389,	2000 cycles before fabric exposure;
H-18 Wheel, 1000 g load	50 mg/100 cycles max. wgt. Loss
Coefficient of Thermal Expansion/Contraction (ASTM D 696)	8 x 10^{-6} in/in/° F max.

1.03 Submittals
The fabricator of panels used in this work shall prepare shop drawings with a proposed panel layout to cover the liner area shown in the project plans. Shop drawings shall indicate the direction of factory seams and shall show panel sizes consistent with the material quantity requirements of 1.01.

Details shall be included to show the termination of the panels at the perimeter of lined areas, the methods of sealing around penetrations, and methods of anchoring.

Placement of the lining shall not commence until the shop drawings and details have been approved by the owner, or his representative.

1.04 Factory Fabrication
The individual XR-5® liner widths shall be factory fabricated into large sheets custom designed for this project so as to minimize field seaming. The number of factory seams must exceed the number of field seams by a factor of at least 10.

A two-inch overlap seam done by heat or RF welding is recommended. The surface of the welded areas must be dry and clean. Pressure must be applied to the full width of the seam on the top and bottom surface while the welded area is still in a melt-type condition. The bottom welding surface must be flat to insure that the entire seam is welded properly. Enough heat shall be applied in the welding process that a visible bead is extruded from both edges being welded. The bead insures that the material is in a melt condition and a successful chemical bond between the two surfaces is accomplished.

Two-inch overlapped seams must withstand a minimum of 240 pounds per inch width dead load at 70° F. and 120 pounds per inch width at 160° F. as outlined in ASTM D 751. All seams must exceed 550 lbs. bonded seam strength per ASTM D 751 Bonded Seam Strength Grab Test Method, Procedure A.

1.05 Inspection And Testing Of Factory Seams
The fabricator shall monitor each linear foot of seam as it is produced. Upon discovery of any defective seam, the fabricator shall stop production of panels used in this work and shall repair the seam, and determine and rectify the cause of the defect prior to continuation of the seaming process.

The fabricator must provide a Quality Control procedure to the owner or his representative which details his method of visual inspection and periodic system checks to ensure leak-proof factory fabrication.

1.06 Certification and Test Reports
Prior to installation of the panels, the fabricator shall provide the owner, or his representative, with written certification that the factory seams were inspected in accordance with Section 1.05.

1.07 Panel Packaging and Storage
Factory fabricated panels shall be accordian-folded, or rolled, onto a sturdy wooden pallet designed to be moved by a forklift or similar equipment. Each factory fabricated panel shall be prominently and indelibly marked with the panel size. Panels shall be protected as necessary to prevent damage to the panel during shipment.

Panels which have been delivered to the project site shall be stored in a dry area.

1.08 Qualifications of Suppliers
The fabricator of the lining shall be experienced in the installation of flexible membrane lining, and shall provide the owner or his representative with a list of not less than five (5) projects and not less than 500,000 square feet of successfully installed XR-5 synthetic lining. The project list shall show the name, address, and telephone number of an appropriate party to contact in each case. The manufacturer of the sheet goods shall provide similar documentation with a 10 million square foot minimum, with at least 5 projects demonstrating 10+ years service life.

The installer shall provide similar documentation to that required by the fabricator.

1.09 Subgrade Preparation By Others
Lining installation shall not begin until a proper base has been prepared to accept the membrane lining. Base material shall be free from angular rocks, roots, grass and vegetation. Foreign materials and protrusions shall be removed, and all cracks and voids shall be filled and the surface made level, or uniformly sloping as indicated

on the drawings. The prepared surface shall be free from loose earth, rocks, rubble and other foreign matter. Generally, no rock or other object larger than USCS sand (SP) should remain on the subgrade in order to provide an adequate safety factor against puncture. Geotextiles may be used to compensate for irregular subgrades. The subgrade shall be uniformly compacted to ensure against settlement. The surface on which the lining is to be placed shall be maintained in a firm, clean, dry and smooth condition during lining installation.

1.10 Lining Installation
Prior to placement of the liner, the installer will indicate in writing to the owner or his representative that he believes the subgrade to be adequately prepared for the liner placement.

The lining shall be placed over the prepared surface in such a manner as to assure minimum handling. The sheets shall be of such lengths and widths and shall be placed in such a manner as to minimize field seaming.

In areas where wind is prevalent, lining installation should be started at the upwind side of the project and proceed downwind. The leading edge of the liner shall be secured at all times with sandbags or other means sufficient to hold it down during high winds.

Sandbags or rubber tires may be used as required to hold down the lining in position during installation. Materials, equipment or other items shall not be dragged across the surface of the liner, or be allowed to slide down slopes on the lining. All parties walking or working upon the lining material shall wear soft-sole shoes.

Lining sheets shall be closely fit and sealed around inlets, outlets and other projections through the lining. Lining to concrete seals shall be made with a mechanical anchor, or as shown on the drawings. All piping, structures and other projections through the lining shall be sealed with approved sealing methods.

1.11 XR-5 Field Seaming
All requirements of Section 1.04 and 1.05 apply. A visible bead should be extruded from the hot air welding process.

Field fabrication of lining material will not be allowed.

1.12 Inspection
All field seams will be tested using the Air Lance Method. A compressed air source will deliver 55 psi minimum to a 3/16 inch nozzle. The nozzle will be directed to the lip of the field seam in a near perpendicular direction to the length of the field seam. The nozzle will be held 4 inches maximum from the seam and travel at a rate not to exceed 40 feet per minute. Any loose flaps of 1/8" or greater will require a repair.

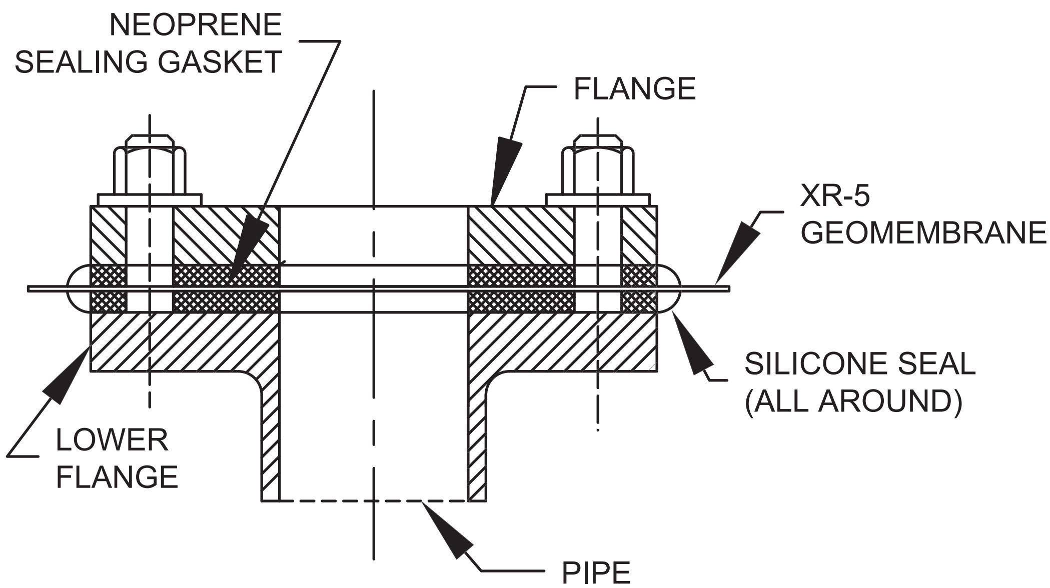
Alternatively all field seams should also be inspected utilizing the Vacuum Box Technique as described in Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber (ASTM D 5641-94 (2006)), using a 3 to 5 psi vacuum pressure. All leaks shall be repaired and tested.

All joints, on completion of work, shall be tightly bonded. Any lining surface showing injury due to scuffing, penetration by foreign objects, or distress from rough subgrade, shall as directed by the owner or his representative be replaced or covered, and sealed with an additional layer of lining of the proper size, in accordance with the patching procedure.

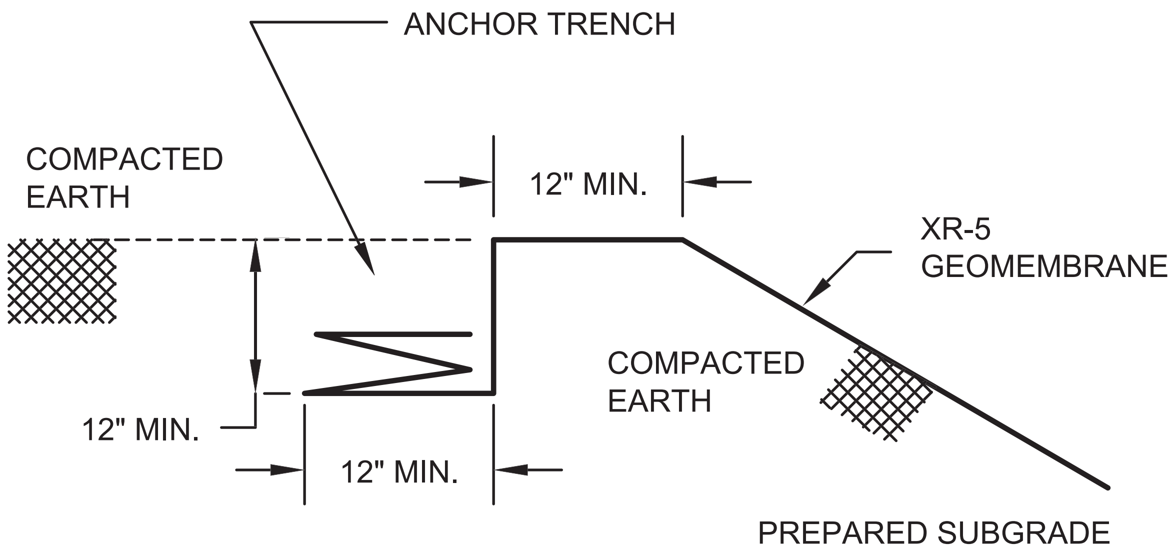
1.13 Patching
Any repairs to the lining shall be patched with the lining material. The patch material shall have rounded corners and shall extend a minimum of four inches (4") in each direction from the damaged area.

Seam repairs or seams which are questionable should be cap stripped with a 1" wide (min.) strip of the liner material. The requirements of Section 1.11 apply to this cap stripping.

1.14 Warranty
The lining material shall be warranted on a pro-rated basis for 10 years against both weathering and chemical compatibility in accordance with Seaman Corporation warranty for XR-5® Style 8130. A test immersion will be performed by the owner and the samples evaluated by the manufacturer. Workmanship of installation shall be warranted for one year on a 100% basis.



Seaman Corporation ENGINEERED PRODUCTS GROUP 1000 Venture Blvd.,Wooster, Ohio 44691	
FLANGE CONNECTION TO PIPE SECTION	
SCALE: NONE	
SHEET 1 of 1	DRAW NO. XRD-019



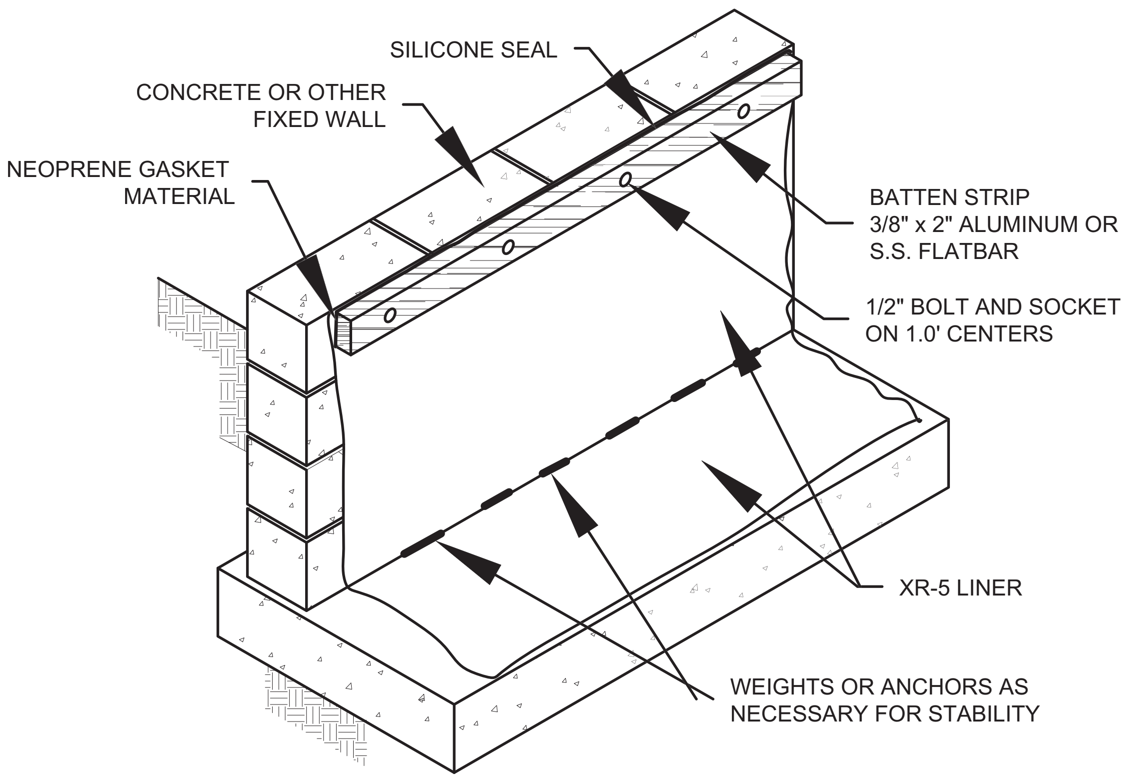
Seaman Corporation

ENGINEERED PRODUCTS GROUP

1000 Venture Blvd.,Wooster, Ohio 44691

ELEVATION VIEW
TYPICAL ANCHOR DETAILS
XR-5 LINER

SCALE: NONE	
SHEET 1 of 1	DRAW NO. XRD-001



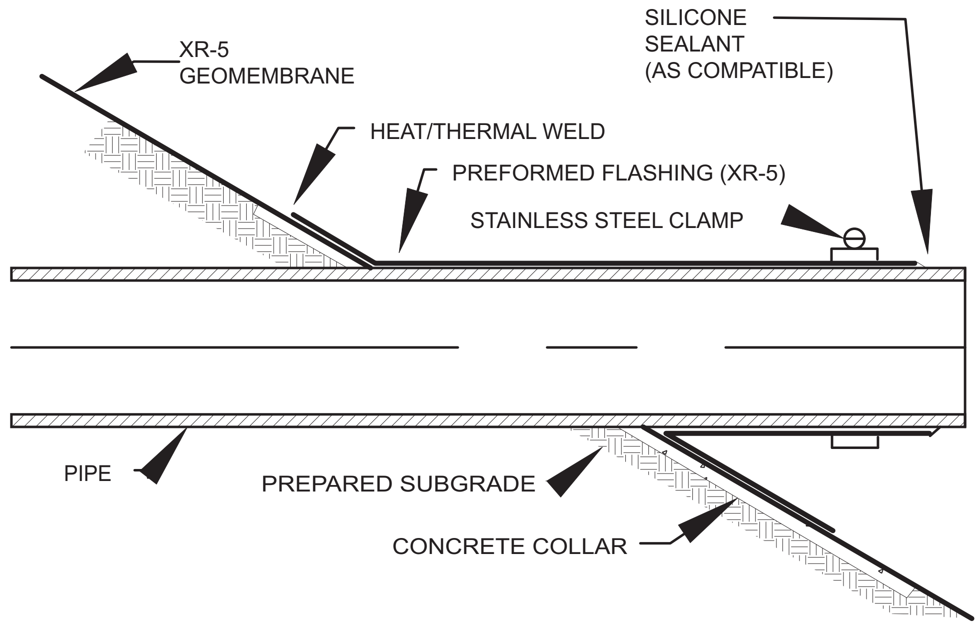
Seaman Corporation

ENGINEERED PRODUCTS GROUP

1000 Venture Blvd.,Wooster, Ohio 44691

ANCHORING DETAIL
XR-5 LINER TO FIXED WALL

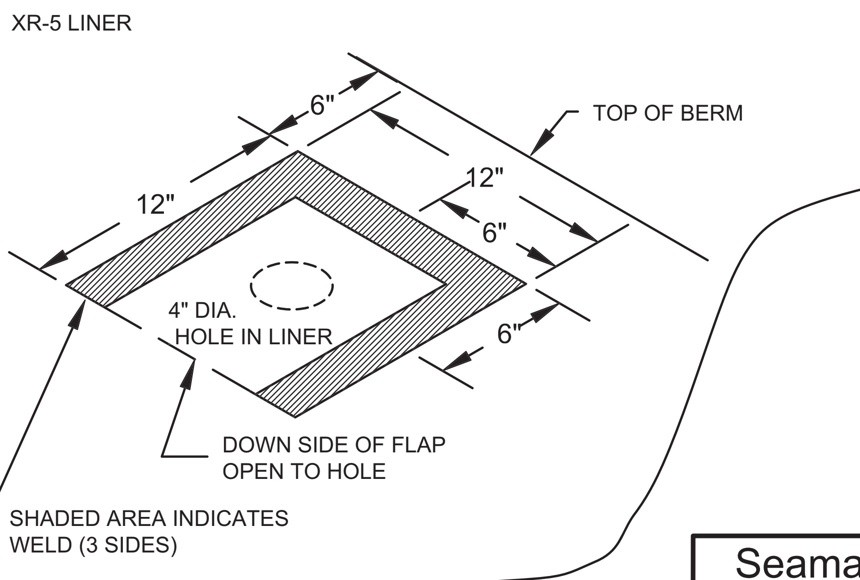
SCALE: NONE	
SHEET 1 of 1	DRAW NO. XRD-023



Seaman Corporation
ENGINEERED PRODUCTS GROUP
1000 Venture Blvd.,Wooster, Ohio 44691

ELEVATION VIEW
TYPICAL PIPE PUNCTURE
SEAL DETAIL XR-5 LINER

SCALE: NONE
SHEET 1 of 1
DRAW NO. XRD-018



Seaman Corporation
ENGINEERED PRODUCTS GROUP
1000 Venture Blvd.,Wooster, Ohio 44691

PLAN VIEW
FLAP VENT DETAIL
XR-5 LINER

SCALE: NONE
SHEET 1 of 1
DRAW NO. XRD-012

Section 6 - Warranty Information

Warranty

XR-5® is offered with Seaman Corporation standard warranty which addresses weathering and chemical compatibility for a 10-year period. A test immersion is required with subsequent testing and approval by Seaman Corporation.

Instructions for XR-5 Test Immersions and Warranty Requests

1. Completely immerse six Style 8130 XR-5 samples (8-1/2" x 11" size) in the liquid to be contained.
2. At the end of approximately thirty days, retrieve three of the samples. The samples should be rinsed with fresh water and dried.
3. Send the three samples to:

   Attn: Geomembrane Department
   Seaman Corporation
   1000 Venture Blvd.
   Wooster, OH 44691
4. Keep the other three samples immersed until further notice in case longer immersion data is required.
5. Complete and return the information form on the liner application.

8228 XR-3® and all PW Geomembranes are offered with a standard 10-year warranty for weathering. The attached information form should be completed.

XR® Membrane Application and Utilization Form

Installation Owner and Address:

Physical Location of Installation:

Expected Date of Installation:

Expected Beginning Date of Service:

Description of Application:

(Example: impoundment used to contain brine on an emergency basis.)

Physical Features of Application:

(Example: 1.3 million gallon earthen impoundment with overall top dimensions of 160’ x 160’ with 3:1 slopes and 10’ deep.)

Description of Liquid:

(Describe content of liquid including pollutants and expected temperature extremes in basin and at application point. Attach analysis of liquid chemistry, composition taken on a representative basis.)

Operational Characteristics:

(Describe the operation of the facility such as filling schedules, fluctuating liquid levels, operating temperatures, etc.)

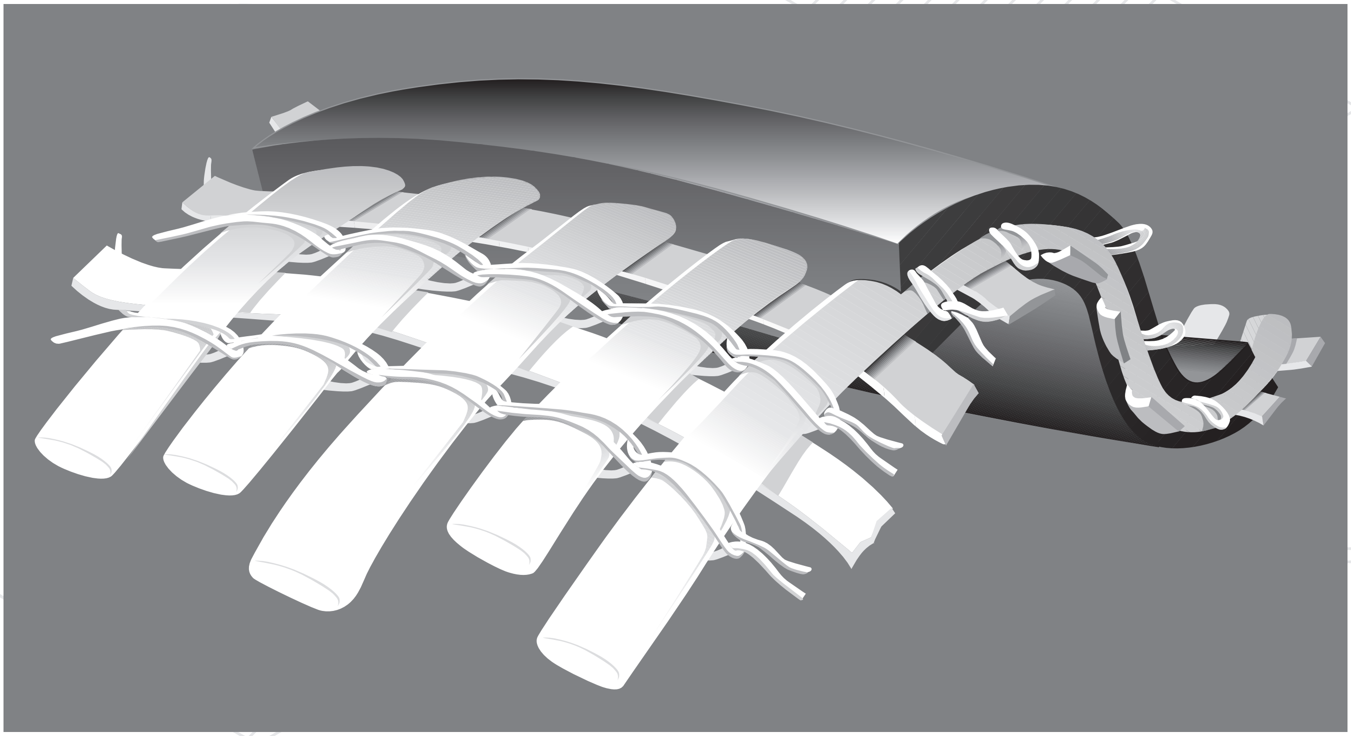
Performance Requirements, Etc:

(State any other requirements, such as rate of permeability required.)

Owner represents the information herein is complete and accurate, and understands and agrees that issuance of Seaman Corporation Warranty for XR products are conditioned upon such completeness and accuracy.

OWNER'S SIGNATURE

Reference Materials:



XR-5®: High Performance Composite Geomembrane

Seaman Corporation
1000 Venture Blvd.
Wooster, Ohio 44691
(330) 262-1111
www.xr-5.com

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.I
SMOOTH HDPE GEOMEMBRANE

SMOOTH HDPE GEOMEMBRANE
ENGLISH UNITS

Property	Test Method	Minimum Average Values 30 mil	40 mil	60 mil	80 mil	100 mil
Thickness, mils	ASTM D 5199					
minimum average		30	40	60	80	100
lowest individual reading		27	36	54	72	90
Sheet Density, g/cc	ASTM D 1505/D 792	0.940	0.940	0.940	0.940	0.940
Tensile Properties[1]	ASTM D 6693					
1. Yield Strength, lb/in		63	84	126	168	210
2. Break Strength, lb/in		114	152	228	304	380
3. Yield Elongation, %		12	12	12	12	12
4. Break Elongation, %		700	700	700	700	700
Tear Resistance, lb	ASTM D 1004	21	28	42	56	70
Puncture Resistance, lb	ASTM D 4833	54	72	108	144	180
Stress Crack Resistance[2], hrs	ASTM D 5397 (App.)	300	300	300	300	300
Carbon Black Content[3], %	ASTM D 1603	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596			--Note 4--		
Oxidative Induction Time (OIT)						
Standard OIT, minutes	ASTM D 3895	100	100	100	100	100
Oven Aging at 85°C	ASTM D 5721					
High Pressure OIT - % retained after 90 days	ASTM D 5885	60	60	60	60	60
UV Resistance[5]	GRI GM11					
High Pressure OIT[6] - % retained after 1600 hrs	ASTM D 5885	50	50	50	50	50
Seam Properties	ASTM D 6392 (@ 2 in/min)					
1. Shear Strength, lb/in		57	80	120	160	200
2. Peel Strength, lb/in - Hot Wedge		45	60	91	121	151
- Extrusion Fillet		39	52	78	104	130
Roll Dimensions						
1. Width (feet):		23	23	23	23	23
2. Length (feet)		1000	750	500	375	300
3. Area (square feet):		23,000	17,250	11,500	8,625	6,900
4. Gross weight (pounds, approx.)		3,470	3,470	3,470	3,470	3,470

1 Machine direction (MD) and cross machine direction (XMD) average values should be on the basis of 5 test specimens each direction. Yield elongation is calculated using a gauge length of 1.3 inches; Break elongation is calculated using a gauge length of 2.0 inches.
2 The yield stress used to calculate the applied load for the SP-NCTL test should be the mean value via MQC testing.
3 Other methods such as ASTM D 4218 or microwave methods are acceptable if an appropriate correlation can be established.
4 Carbon black dispersion for 10 different views: Nine in Categories 1 and 2 with one allowed in Category 3.
5 The condition of the test should be 20 hr. UV cycle at 75°C followed by 4 hr. condensation at 60°C.
6 UV resistance is based on percent retained value regardless of the original HP-OIT value.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 1: ENGINEERING DESIGN

ATTACHMENT III.1.J

COMPUTER AIDED EARTHMOVING SYSTEM

Computer Aided Earthmoving System

CAES for Landfills

Landfill Compactors
Track-Type Tractors
Wheel Tractor Scrapers
Motor Graders

System Components	
Communications Radio	TC900B
GPS Antenna	L1/L2
GPS Receiver	MS840
In-Cab Display	CAES Touch Screen Display
CAESoffice™/METSmanager	

Computer Aided Earthmoving System for Landfills

Advanced GPS technologies for earthmoving equipment improve machine efficiency, maximize air space utilization, and extend landfill life.

Caterpillar is helping customers revolutionize the way they compact trash, grade slopes and manage their operation with new technology solutions for landfills. Solutions that provide greater accuracy, higher productivity, lower operating costs, more profitability and longer landfill life.

The Computer Aided Earthmoving System (CAES) is a high technology earthmoving tool that allows machine operators to achieve maximum landfill compaction, desired grade/slope, and conserve and ensure even distribution of valuable cover soil with increased accuracy without the use of traditional survey stakes and crews. Using global positioning system (GPS) technology, machine-mounted components, a radio network, and office management software, this state-of-the-art machine control system delivers real-time elevation, compaction and grade control information to machine operators on an in-cab display. By monitoring grade and compaction progress, operators have the information they need to maximize the efficiency of the machine, resulting in proper drainage and optimum airspace utilization.

This advanced technology tool also aids in the identification of site-specific storage areas for hazardous, medical, industrial, and organic waste requiring special handling and placement records.

Applications

CAES is an ideal tool for landfill planning, engineering, surveying, grade control, and production monitoring applications in dump areas. CAES is specifically designed for use on landfill compactors, track-type tractors, wheel tractor scrapers, and motor graders.

On-Board Components

- CAES Touch Screen Display
- GPS Receiver
- GPS Antenna (L1/L2)
- Communications Radio

Off-Board Components

- GPS Reference Station
- Radio Network
- CAESoffice/METSmanager

Operation

CAES uses GPS technology, a wireless radio communications network, and office software to map landfills, create site plans, locate a machine's position, and track compaction and earthmoving progress with complete accuracy.

The receiver uses signals from GPS satellites to determine precise machine positioning. Two receivers are used to capture and collect satellite data – one located at a stationary spot on the landfill site, and another located on the machine. Signals from the ground-based reference station and on-board computer are used to remove errors in satellite measurements for centimeter accuracy.

The CAES-enabled machine is driven over the site to create a digital terrain design file. Using the radio network and office software, landfill terrain data is transmitted from the machine to the landfill office. Landfill managers can then send the work plan from the office to the in-cab display to show operators the work to be done.

The in-cab display provides the operator with an overhead and cross-sectional three-dimensional surface view of the color-coded work plan and precise machine location. The software continuously updates terrain and machine position information as the machine traverses the site.

CAES gives the operator the ability to control grade by monitoring progress on the in-cab display, which shows a graphical representation of lift thickness and compaction density. Cut/fill numbers are displayed in real-time as the machine moves across the site, which allows the operator to know precise elevation, material spread, compaction passes, and required cut or fill at any point on the job.

The *compactor* display shows colored grids representing the number of compaction passes the machine has made across each area. As the compactor wheel travels over an area, the screen changes color to acknowledge the pass. Green areas indicate when optimum compaction has been reached. The system also monitors thick lift information and visually displays when a lift exceeds maximum site parameters.

In *tractor, scraper and motor grader* applications, the color display graphically shows the operator cut, fill, and grade work to be done according to plan. As the machine works, the screen changes color. Green indicates when the operator has achieved plan grade.

By providing immediate feedback on the accuracy of each pass, CAES operators have the information and confidence they need to work more efficiently, productively and profitably.

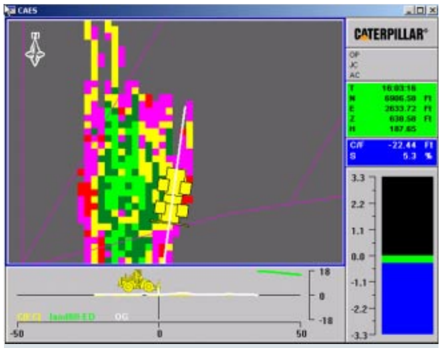
On-Board Components

Communications Radio. The rugged radio, mounted on the roof of the machine, is used for transmitting, repeating and receiving real-time data from GPS receivers. The radio broadcasts real-time, high-precision data for GPS applications. Under normal conditions, the 900 MHz radio broadcasts data up to 10 km (6.2 miles) line-of-sight. Coverage can be enhanced with a network of repeaters, which allows coverage over a broader area. Optimized for GPS with increased sensitivity and jamming immunity, the radio features error correction and high-speed data transfer, ensuring optimum performance. A 450 MHz radio solution is also available.

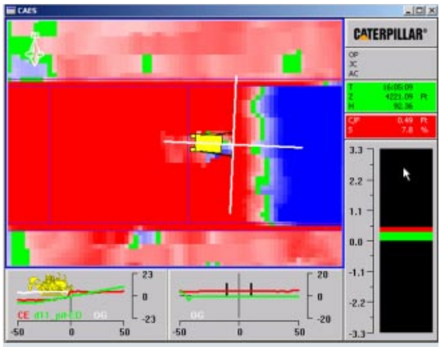
GPS Antenna (L1/L2). The dual frequency external antenna, mounted on the roof of the machine and reference station, is used to pick up the signals from the GPS satellites to determine the machine's position for high precision, real-time machine guidance and control. A low-noise amplifier provides sensitive performance in demanding applications. The compact, low profile design and sealed housing ensure reliable performance in harsh weather conditions.

GPS Receiver. The dual frequency real-time kinematic (RTK) GPS receiver is used to send and receive data simultaneously across the radio network. The system computes differential corrections for real-time positioning with centimeter accuracies, to ensure precise machine guidance and control.

CAES Touch Screen Display. The in-cab graphical display provides real-time operating information to the operator. Designed for simple operation, the 264 mm (10.4 in) custom configurable, integrated touch screen display allows operators to easily interface with the CAES system. The display utilizes the latest infrared touch and transflective backlight technology for superior viewing in bright light conditions and a broad-range dimmable backlight for viewing in low light conditions. Designed for reliable performance in extreme operating conditions, the unit is guarded against shock and sealed to keep out dust and moisture.



Compactor Screen



Dozer Screen

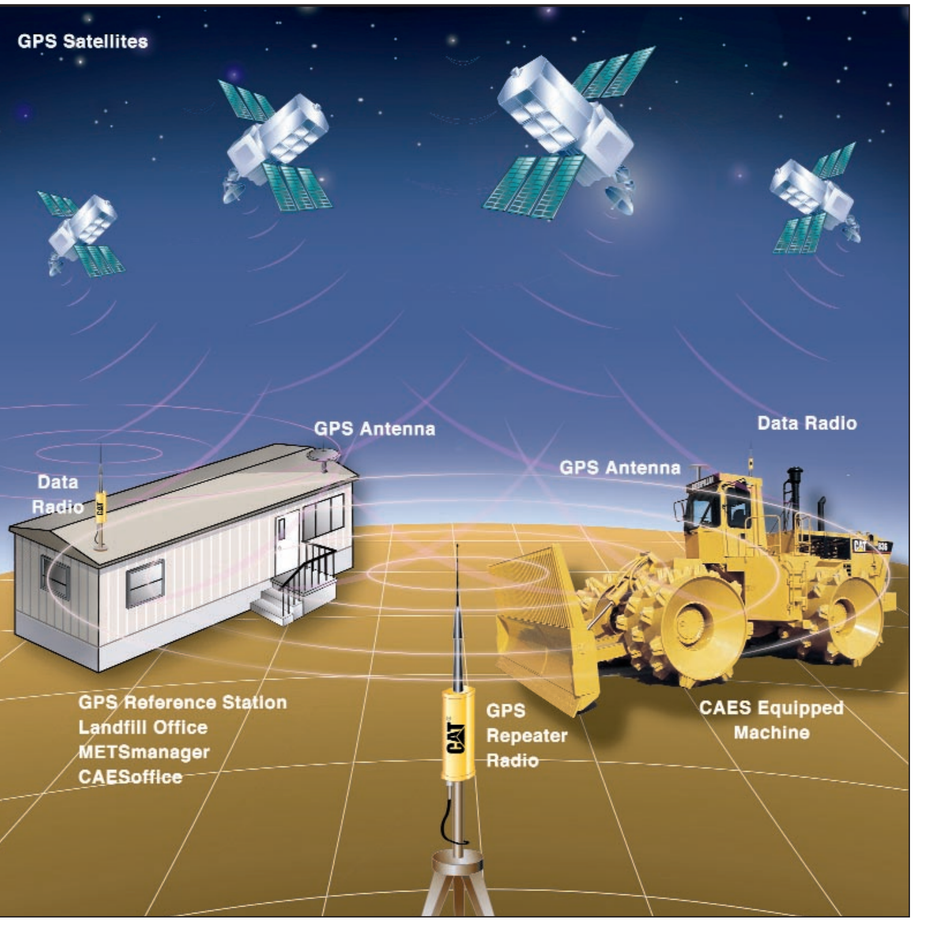
Off-Board Components

GPS Technology. Global Positioning System (GPS) technology uses 24+ satellites that orbit above the earth and constantly transmit their positions, identities and times of signal broadcasts to earth-based satellite sensors. The GPS receiver is an electronic box, which measures the distance to each visible satellite from an antenna on the ground. Through trilateralization, the receiver determines where the satellite is in respect to the center of the earth. The GPS receiver uses its own position and GPS satellite positions to calculate errors and corrections for computing exact location and precise positioning with centimeter accuracy.

GPS Reference Station. A GPS reference station is used to achieve the centimeter level accuracy needed in a landfill application. The reference station sends GPS information over a radio link to the GPS receiver on the CAES-enabled machine. The receiver combines the information with its own observations to compute precise positioning.

Radio Network. The radio network for CAES has two channels. GPS correction data is transmitted over one channel, while the other channel is used to send site planning and production data to the machine and from the machine back to the site office. By utilizing the same radio as a repeater the range can be extended to provide seamless coverage around local obstacles such as hills or large buildings. Up to four radio repeaters may be used to provide extended coverage.

Landfill Planning Software. Site planning and surveying begins with the landfill planning software. CAES is compatible with most third party CAD planning software packages. Data formats used between the CAES software and the planning software are industry standard .DXF and ASCII.



CAESoffice™. The powerful Caterpillar-designed CAESoffice software enables landfill management to monitor CAES-equipped machines and work progress throughout the site in near real-time. The data is stored in a database format for easy customized access, reporting and editing.

METSmanager. This software package allows for integration of the landfill planning system and the machine. It provides the user interface for CAES and controls all communications over the wireless radio network. METSmanager reads design files in standard .DXF formats, converts them to CAES format (.CAT), and sends the design files to the on-board display on the machine over the radio network. This program continually updates the site model by regularly requesting data transmissions from the machine to the office.

- **File Window.** Displays design files (.DXF) created using the site planning package, and holds application configuration files for GPS receivers and files converted from .DXF to the CAES on-board software format (.CAT).
- **Machines Window.** Shows icons of each machine equipped with CAES on-board software. Allows multiple machines to be monitored at the same time.
- **Messages Window.** Contains a list of recent error, warning, confirmation, or information messages generated by METSmanager.
- **Communications Queue Window.** Lists all file transmissions scheduled to occur over the radio network and displays transmission status for all files.

Specifications

TC900B Communications Radio
- Technology: Spread spectrum
- Modes: Base, repeater, rover
- Optimal Range: 10 km (6 miles), line-of-sight
- Typical Range: 3-5 km (2-3 miles) varies w/terrain and operating conditions. Repeaters may be used to extend range
- Frequency Range: 902-928 MHz
- Networks: Ten, user selectable
- Transmit Power: Meets FCC requirements, 1 watt max.
- License Free (U.S. and Canada)
- Wireless Data Rates: 128 Kbps[2]
- Operating Temperature: –40° C to 70° C (–40° F to 158° F)
- Storage Temperature: –40° C to 85° C (–40° F to 185° F)
- Humidity: 100%
- Sealing: Exceeds MIL-STD-810E, sealed to ±34.5 kPa (±5 psi), immersible to 1 m (39 in)
- Vibration: 8 gRMS, 20-2000 Hz
- Operational Shock: ±40 g, 10 msec
- Survival Shock: ±75 g, 6 msec
- Electrical Input: 10.5 to 20V DC
- Nominal Current: 250 mA (3 W)1
- Transmit Current: 1000 mA (12 W)1
- Protection: Reverse polarity
- Control Interface: SAE J1939 CAN
- Emissions and Susceptibility: CE compliant, exceeds ISO 13766
- Input Connector: 8-pin
- Network Connector: 8-pin
- Height: 250 mm (10 in)
- Width: 85 mm (3.4 in)
- Weight: 0.9 kg (2.0 lb)

Radios outside of U.S. and Canada operate on different frequencies. Please contact your Cat Dealer for specifics.

L1/L2 GPS Antenna
- Operating Temperature: –40° C to 70° C (–40° F to 158° F)
- Storage Temperature: –55° C to 85° C (–67° F to 185° F)
- Height: 151mm (6 in)
- Width: 330 mm (13 in)
- Depth: 72 mm (2.8 in)
- Weight: 1.695 kg (3.8 lb)

MS840 GPS Receiver
- Tracking: 9 channels L1 C/A code, L1/L2 full cycle carrier, fully operational during P-code encryption
- Signal Processing: Supertrak multibit technology, Everest multipath suppression
- Positioning Mode –
- Synchronized RTK: 1 cm + 2 ppm horizontal accuracy/2 cm + 2 ppm vertical accuracy, 300 ms latency, 5 Hz std. maximum rate
- Low Latency: 2 cm + 2 ppm horizontal accuracy/3 cm + 2 ppm vertical accuracy, <20 ms latency, 20 Hz maximum rate
- DPGS: <1m accuracy, <20 ms latency, 20 Hz maximum rate
- Range: Up to 20 km from base for RTK
- Communication: 3x RS-232 ports, baud rates up to 115,200
- Control Interface: SAE J1939 CAN
- Configuration: RS-232 Serial connection
- Operating Temperature: –20° C to 60° C (–4° F to 140° F)
- Storage Temperature: –30° C to 80° C (–22° F to 176° F)
- Humidity: 100%
- Operational Vibration: 3 gRMS
- Survival Vibration: 6.2 gRMS
- Operational Shock: ±40 g
- Survival Shock: ±75 g
- Electrical Input: 12/24V DC, 9 watts
- Height: 5.1 cm (2.0 in)
- Width: 14.5 cm (5.7 in)
- Depth: 23.9 cm (9.4 in)
- Weight: 1.0 kg (2.25 lb)

CAES Touch Screen Display
- LCD Display: 264 mm (10.4 in) 640 × 480 transflective color VGA
- Buttons: touch screen
- Touch Screen: 3.17 mm (0.125 in) resolution infrared high light rejection
- Back Light: 200 cd/m2, 200:1 dimming ratio
- Processor: Intel Pentium CPU
- Memory: 64 MB Ram
- Solid State Disk: Internal 128 MB, external compact flash
- Operating Environment: Embedded WinNT
- Operating Temperature: –20° C to 70° C (–4° F to 158° F)
- Storage Temperature: –50° C to 85° C (–58° F to 185° F)
- Sealing: IP68 sealed to ±5 psi
- Humidity: 100%
- Electrical Input: 9-32V DC
- Power Supply: 5 amp @ 40W load dump, reverse voltage, ESD, over voltage protection
- Connector: 70-pin
- Discrete I/O: 8 digital ports; 5 PMW inputs
- Mounting: bracket or panel
- Height: 261 mm (10.28 in)
- Width: 315 mm (12.4 in)
- Depth: 93 mm (3.66 in)
- Weight: 3.17 kg (8.5 lb)

CAESoffice/METSmanager PC Requirements
- Pentium II/III processor w/ 128 MB memory
- 21 in. monitor (SVGA color 1024 × 768 resolution) with 2MB video memory
- Windows NT 4.0 or higher with latest service pack
- Modem- internal or external (required for remote support)
- Required ports: serial (suggest 2 serial, 1 parallel)
- CD ROM drive
- 3.5 in disk drive
- Mouse or suitable pointing device
- Hard Drive Space: 200 MB min.

Customer Support. For over 25 years, Caterpillar has been providing electronic and electrical components and systems for the earthmoving industry – real world technology solutions that enhance the value of Cat products and make customers more productive and profitable. Your Cat Dealer is ready to assist you with matching machine systems to the application or obtaining responsible, knowledgeable support. For additional information, please contact us at LANDFILLGPS@CAT.com

Computer Aided Earthmoving System for Landfills

Landfill Compactors
Track-Type Tractors
Wheel Tractor Scrapers
Motor Graders

AEHQ5549 (9-03)

CATERPILLAR®

NM1-57

Revised Permit Application

June 2014

Volume 3, Part 2 of 3: Engineering Design and Calculations

NM1-57

Revised Permit Application

June 2014

Volume 3, Part 3 of 3:
Engineering Design
and Calculations

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Paraffin oil				20	+	+	O	+	+	+		-	+	+	+	O
				40	+			+	+	+			+	O	O	-
				60	O			+	O	+			+	O	-	
				80						+			O			
				100						+						
				120						+						
				140												
p-Dibromo benzene	$C_6H_5Br_2$		technically pure	20	-	-	-	O	O	+		-	+	-	-	-
				40						+						
				60						+						
				80						+						
				100						+						
				120												
				140												
Perchlorethylene (tetrachlorethylene)	$Cl_2C{=}CCl_2$	121	technically pure	20	-	-		O	O	+			+	O	-	-
				40						+			+	-		
				60						+			+			
				80						O						
				100						-						
				120												
				140												
Perchlorid acid (SpRB)	$HClO_4$		10%, aqueous	20	+	+	O	+	+	+		+	+	-	-	+
				40	+	+		+	+	+		O	+			+
				60	O	+		+	+	+			+			O
				80						+			O			-
				100						+						
				120												
				140												
Perchlorid acid (SpRB)			70%, aqueous	20	O	O	-	+	O	+		-	+	-	-	+
				40				O	-	+			+			+
				60				-		+			+			O
				80						+			O			
				100						+						
				120												
				140												
Petroleum			technically pure	20	+	-	-	+	+	+		-	+	+	O	-
				40				+	O	+			+	+	-	
				60				O	O	+			O	+		
				80						+						
				100						+						
				120						+						
				140												
Petroleum ether (SpRB)		40-70	technically pure	20	+	-	-	+	+	+		-	+	+	-	-
				40	+			O	O	+			+	O		
				60	+			O	O	+			O	-		
				80						+						
				100						+						
				120												
				140												
Phenol (SpRB)	$C_6H_5\text{-}OH$	182	up to 10%, aqueous	20	+	+	-	+	+	+			+	-	-	-
				40	O	+		+	+	+			+			
				60				O	+	+		+	+			
				80						+		O	O			
				100						+						
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Phenol (SpRB)			up to 5%	20	+	+	-	+	+	+		+	+	-	-	
				40		+		+	+	+		O	+			
				60				+	+	+			+			
				80						+			O			
				100						O						
				120												
				140												
Phenol (SpRB)	C_6H_5-OH		up to 90%, aqueous	20	O	-	-	+	+	+		-	+	-	-	-
				40				+	+	+			O		O	
				60				O	+	O			-			
				80												
				100												
				120												
				140												
Phenylhydrazine	C_6H_5-NH-NH_2	243	technically pure	20	-	-	-	O	O	O		-	+	-	-	-
				40						-			+			
				60									O			
				80												
				100												
				120												
				140												
Phenylhydrazine hydrochloride	C_6H_5-NH-NH_2.HCl		aqueous	20	O	O	-		+	+			+	O	O	+
				40					O	+			+	-	-	+
				60					O	+		O	O			O
				80									-			-
				100												
				120												
				140												
Phosgene (SpRB)	$COCl_2$	8	liquid, technically pure	20	-	-	-	-	-	-		-	+	O	+	+
				40												
				60												
				80												
				100												
				120												
				140												
Phosgene (SpRB)			gaseous, technically pure	20	+	-	-	O	O	+		+	+	+	+	+
				40	O					+			+	+	O	O
				60	O								O	+	-	
				80												
				100												
				120												
				140												
Phosphate disodique	see d'isodiumphosphate		saturated													
Phosphoric acid	H_3PO_4		up to 30%, aqueous	20	+	+	+	+	+	+		+	+	O	+	+
				40	+	+	+	+	+	+		+	+	O	+	+
				60	O	+	O	+	+	+		O	+	-	+	+
				80		+			+	+			+		O	O
				100						+			+			
				120						+						
				140												
Phosphoric acid			50%, aqueous	20	+	+	+	+	+	+		+	+	O	+	+
				40	+	+	+	+	+	+		O	+	-	+	+
				60	+	+	O	+	+	+			+		O	+
				80		+			+	+			+			+
				100						+			O			O
				120						+						
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Phosphoric acid			85%, aqueous	20	+	+	+	+	+	+		+	+	-	+	+
				40	+	+	+	+	+	+		+	+		+	O
				60	+	+	O	O	+	+		O	+		O	-
				80		+			+	+			+			
				100						+			O			
				120						+						
				140												
Phosphoric acid	H_3PO_4			20	+	+	-	+	+	+		O	+	-	-	-
				40	+	+		+	+	+			+			
				60		+				+			O			
				80						+						
				100												
				120												
				140												
Phosphoric acid	H_3PO_4			20	+	+	-	+	+	+			+			
				40	+	+		+	+	+			+			
				60	+	+			+	+			+			
				80		+			+	+			+			
				100						+			O			
				120												
				140												
Phosphoric acid tributyl ester	$(H_9C_4O)_3P{=}O$			20	-	-	-	+	+	-		+	-			
				40												
				60												
				80												
				100												
				120												
				140												
Phosphorous chlorides:		175	technically pure	20	-	-	-	+	O	-			+	-	-	+
- Phosphorous trichloride	PCl_3	162		40												
- Phosphorous pentachloride	PCl_5	105		60				O	O							
- Phosphorous oxichloride (SpRB)	$POCl_3$			80												
				100												
				120												
				140												
Photographic developer (SpRB)			usual commercial	20	+	+	+	+	+	+		+	+	O	+	+
				40	+	+	+	+	+	+		+	+	O	+	+
				60	O	+	O	O		+						
				80		O										
				100												
				120												
				140												
Photographic emulsions (SpRB)				20	+	+	+	+	+	+		+	+	O	+	+
				40	+	+	+	+	+	+		+	+		+	+
				60		O				+						
				80												
				100												
				120												
				140												
Photographic fixer (SpRB)			usual commercial	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+	O			+						
				80												
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Phthalic acid (SpRB)	$C_6H_5(COOH)_2$	Fp.*, 208	saturated, aqueous	20	+	-	-	+	+	+		+	-	-	+	+
				40	O			+	+	+		O			+	+
				60	-			+	+	+					O	
				80						+						
				100						+						
				120												
				140												
Phthalic acid dioctayl ester	$C_{24}H_{38}O_4$			20	-	-	-	+	+	-		+	-	-		
				40												
				60												
				80												
				100												
				120												
				140												
Picric acid (SpRB)	$C_6H_3N_3O_7$	FP. 122	1%, aqueous	20	+	-	-	+	+	+		+	+	O	O	+
				40						+		+	+	-	-	O
				60						+		+	+			-
				80						+		+	+			
				100						+		+	+			
				120												
				140												
Potash	see potassium carbonate		cold saturated, aqueous													
Potash lye	KOH		50%	20	+	+		+	+	-		+				
				40	+	+		+	+			+				
				60	+	+		+	+			+				
				80												
				100												
				120												
				140												
Potassium (SpRB)	$KMnO_4$		cold saturated, aqueous	20	+	+		+	+	+		+	+	O	O	+
				40	+	+		+	+	+			+	-	-	+
				60	O			O		+			+			+
				80						+			+			
				100						+						
				120												
				140												
Potassium acetate (SpRB)	CH_3COOK		saturated	20	+	+	+	+	+	+		+	-			
				40	+	+		+	+	+		+				
				60	+	+		+	+	+		+				
				80		+			+	+		+				
				100						+						
				120												
				140												
Potassium bichromate (SpRB)	$K_2Cr_2O_7$	107	saturated, aqueous	20	+	+	+	+	+	+		+	+	+	O	+
				40	+	+	+	+	+	+			+	O	-	+
				60	O	+	+	+	+	+			+			+
				80					+	+						
				100					+	+						
				120						+						
				140												
Potassium borate	K_3BO_3		10%, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+			+	+	+	+
				60	O	+	+	+	+	+			+	+	+	+
				80		+										
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Potassium bromate	$KBrO_3$		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+	+	O	+	+		+	+	+	+	+
				80		+			+	+			+	+	O	+
				100					+	+				+	O	+
				120						+						
				140												
Potassium bromide	KBr		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+	+	+	+	+		+	+	+	+	+
				80		+				+			+	O	+	+
				100						+			+	O	O	+
				120						+						
				140												
Potassium carbonate (potash)				20	+	+	+	+	+	O		+				
				40	+	+		+	+			+				
				60	+	+		+	+			+				
				80		+			+							
				100												
				120												
				140												
Potassium chlorate (SpRB)	$K\,ClO_3$		cold saturated, aqueous	20	+	+	+	+	+	O		+	+	+	+	+
				40	+	+	+	+	+	-		+	+	O	O	+
				60	+	+	+	+	+				+			+
				80		+							+			O
				100									+			
				120												
				140												
Potassium chloride	KCl		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	+	+	+	+	+	+		+	+	+	+	+
				80		+			+	+			+			+
				100					+	+			+			
				120						+						
				140												
Potassium chromate (SpRB)	K_2CrO_4		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+		+	+			+	O	+	+
				60	+	+	+		+	+			+	-	O	O
				80						+						
				100						+						
				120												
				140												
Potassium cyanide	KCN		cold saturated, aqueous	20	+	+	+	+	+	+			+	+	+	+
				40	+	+	+	+	+	+			O	+	+	+
				60	+	+	+	+	+	O		+	-	+	O	+
				80		+			+			+		+	-	+
				100												
				120												
				140												
Potassium dichromate	$K_2Cr_2O_7$		saturated	20		+		+	+	+		+	+			
				40		+		+	+	+						
				60		+		+	+	+						
				80		+				+						
				100						+						
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Potassium fluoride	KF		saturated	20	+	+		+	+	+			+			
				40	+	+		+	+	+						
				60	+	+		+	+	+						
				80		+			+	+						
				100						+						
				120												
				140												
Potassium Hexacyanoferrate -(II)	$K_4[Fe(CN)_6].3H_2O$			20	+	+		+	+	+		+	+			
				40	+	+		+	+	+						
				60	+	+		+	+	+						
				80		+			+	+						
				100						+						
				120												
				140												
Potassium hydrogen carbonate	$KHCO_3$		saturated	20	+	+		+	+	+		+	+			
				40	+	+		+	+	+		+				
				60	+	+		+	+	+		+				
				80		+			+							
				100												
				120												
				140												
Potassium hydrogen sulphate	$KHSO_4$		saturated	20	+	+		+	+			+				
				40	+	+		+	+			+				
				60	+	+		+	+			+				
				80		+			+			+				
				100												
				120												
				140												
Potassium iodide	KJ		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+			+	O	O	+
				60	+	+	+	+	+	+			+	-	-	+
				80		+				+			+			
				100						+						
				120												
				140												
Potassium nitrate	KNO_3		50%, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+			+	+	+	+
				60	+	+	+	+	+	+			+	+	+	+
				80		+				+						
				100						+						
				120												
				140												
Potassium perchlorate (SpRB)	$KClO_4$		cold saturated, aqueous	20	+	+		+		+		+	+	+	+	+
				40	+	+				+			+	O	O	+
				60	O	+				+			+			+
				80		+				+			+			O
				100												
				120												
				140												
Potassium persulphate (SpRB)	$K_2S_2O_8$		all, aqueous	20	+	+	+	+	+	+		+	+	-	+	+
				40	+	+	+	+	+	+		+	+		+	+
				60	O	+	+	+	+	+			+		+	+
				80						+			+		O	O
				100									+			
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Potassium sulphate	K_2SO_4		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+	+	+	+	+			+	+	+	+
				80		+				+			+			
				100						+			+			
				120												
				140												
Potassium sulphide	K_2S		saturated	20	+	+		+	+	O		+				
				40	+	+		+	+	O		+				
				60	+	+		+	+	O		+				
				80		+			+			+				
				100												
				120												
				140												
Potassium sulphite	K_2SO_3		saturated	20	+	+		+	+			+				
				40	+	+		+	+							
				60	+	+		+	+							
				80		+										
				100												
				120												
				140												
Potassium-aluminiumsulfate (alum)			50%	20	+	+		+	+	+		+				
				40	+	+		+	+	+		+				
				60	+	+		+	+	+		+				
				80		+			+	+		+				
				100						+						
				120												
				140												
Pottasium hexacyanoferrate -(III)	$K_3[Fe(CN)_6]$.			20	+	+		+	+	+		+	+			
				40	+	+		+	+	+						
				60	+	+		+	+	+						
				80		+			+	+						
				100						+						
				120												
				140												
Pottasium tartrat				20	+			+	+	+		+				
				40	+			+	+	+		+				
				60				+	+	+		+				
				80					+	+						
				100						+						
				120												
				140												
Pottasiumhydrogensulfite				20	+					+		+				
				40	+					+		+				
				60						+						
				80						+						
				100						+						
				120												
				140												
Pottasiumhypochlorite	KOCl			20	+	O		+	+	O		+	O			
				40												
				60												
				80												
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Pottasiumperoxodisulfate	$K_2S_2O_8$		saturated	20	+	+										
				40	+	+										
				60	+	+										
				80		+										
				100												
				120												
				140												
Pottasiumphosphate	KH_2PO_4 und $K_2H\ PO_4$		all, aqueous	20	+	+	O	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	O	O	+
				60	O	+		+	+	+			+	-	-	+
				80		+			+	+			+			O
				100									+			
				120												
				140												
Pottasiumphosphate				20	+				+							
				40	+				+							
				60	+				+							
				80					+							
				100												
				120												
				140												
Propane	C_3H_8	-42	technically pure, liquid	20	+	-	-	+	+	+		-	+	+	-	-
				40						+						
				60						+						
				80												
				100												
				120												
				140												
Propane			technically pure, gaseous	20	+	+	-	+	+	+		-	+	+	+	O
				40						+						
				60						+						
				80												
				100												
				120												
				140												
Propanol, n- and iso- (SpRB)	C_3H_7OH	97 bzw. 82	technically pure	20	+	-	-	+	+	+		+	+	+	+	+
				40	O			+	+	+		+	+	O	+	O
				60	O			+	+	+			+	-	+	O
				80						O						
				100												
				120												
				140												
Propargyl alcohol (SpRB)	$CH\equiv C-CH_2-OH$	114	7%, aqueous	20	+	-	-	+	+	+		+	+	+	+	+
				40	+			+	+	O		+	+	+	+	+
				60	+			+	+	O			+	+	O	O
				80												
				100												
				120												
				140												
Propionic acid (SpRB)	CH_3CH_2COOH	141	50%, aqueous	20	+	O	-	+	+	+		+	+	-	O	O
				40	+			+	+	+		+	+		-	-
				60	O			+	+	+			O			
				80												
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Propionic acid (SpRB)		141	technically pure	20	+	O	-	+	+	+		+	+	-	-	-
				40	O			O	O	+		O	+			
				60				O	O	+			+			
				80									O			
				100												
				120												
				140												
Propylene glycol (SpRB)	$C_3H_8O_2$	188	technically pure	20	+	-	O	+	+	+		+	+	+	+	+
				40	+			+	+	+		+	+	O	+	+
				60	+			+	+	+			O	-		+
				80												
				100												
				120												
				140												
Propylene oxide	C_3H_6O	35	technically pure	20	O		-	+	+	+		O	-	-	-	-
				40						O						
				60												
				80												
				100												
				120												
				140												
Pyridine	C_5H_5N	115	technically pure	20	-	-	-	+	O	+		O	-	-	-	-
				40				O	O	-						
				60				O	O							
				80												
				100												
				120												
				140												
Pyrogallol	$C_6H_3(OH)_3$		100%	20						+			+			
				40						+						
				60												
				80												
				100												
				120												
				140												
Ramsit fabric waterproofing agents			usual commercial	20	+			+	+	+		+	+	+	+	+
				40	+					+						
				60	+					+						
				80												
				100												
				120												
				140												
Salicylic acid	$C_6H_4(OH)COOH$		saturated	20	+	+	O	+	+	+		+	+	+	+	+
				40	+			+	+	+		+				
				60				+	+	+		+				
				80						+						
				100												
				120												
				140												
Sea water	see Brine															
Silicic acid	$Si(OH)_4$			20	+	+	+	+	+			+				
				40	+	+		+	+			+				
				60	+	+		+	+			+				
				80					+							
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Silicone oil				20	+	+	+	+	+	+		+	+	+	+	+
				40	O	+		+	+	+		+	+	+	+	+
				60	-			+	+	+		+	+	+	O	
				80					+	+						
				100						+						
				120												
				140												
Silver	AgCn		saturated	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80												
				100												
				120												
				140												
Silver salts	$AgNO_3$, AgCN, AgCl		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+	+	+	+	+		+	+	+	+	+
				80		+			+	+						
				100						+						
				120												
				140												
Silvercyanide				20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80												
				100												
				120												
				140												
Soap solution (SpRB)			all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+		+	+	+		+	+	+	+	+
				80						+						
				100						+						
				120												
				140												
Soda	see Sodium carbonate															
Sodium acetate	CH_3COONa		all, aqueous	20	+	+	+	+	+	+		+	-	+	+	O
				40	+	+		+	+	+		+		+	+	
				60	+	+		+	+	+		+				
				80		+			+	+						
				100					+	O						
				120												
				140												
Sodium aluminium sulfate				20	+			+	+							
				40	+			+	+							
				60	+			+	+							
				80					+							
				100												
				120												
				140												
Sodium arsenite	Na_3AsO_3		saturated	20	+	+		+	+			+				
				40	+	+		+	+			+				
				60	+	+		+	+			+				
				80		+			+							
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Sodium benzoate	C_6H_5-COONa		cold saturated, aqueous	20	+	+	-	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		O	+	+	+	+
				60	O	+		+	+	+			+			O
				80		+				+			O			
				100						O						
				120												
				140												
Sodium bicarbonate	$NaHCO_3$		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	+	+	+	+	+	+		+	+	+	+	+
				80					+	+		+	+			
				100						+						
				120												
				140												
Sodium bisulphate	$NaHSO_4$		10%, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		O	+	O	+	+
				60	O	+	+	+	+	+			+	-	O	
				80						+			+			
				100						+			+			
				120						+						
				140												
Sodium bisulphite	$NaHSO_3$		all, aqueous	20	+	+		+	+	+		+	O	O	+	+
				40	O	+		+	+	+		O	-	-	+	+
				60	-	+		+	+	+		-			O	+
				80						+					-	O
				100						+						
				120												
				140												
Sodium borate	Na_3BO_3		saturated	20	+	+		+	+			+	+			
				40	+	+		+	+			+	+			
				60	+	+		+	+			+	+			
				80		+			+							
				100												
				120												
				140												
Sodium bromate	$NaBrO_3$		all, aqueous	20	+	+		+	+	+		+	+	+	+	+
				40	O	+		O	O	+			+	O	+	+
				60						+			+	-	O	+
				80						+						
				100						+						
				120												
				140												
Sodium bromide	NaBr		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+			+	O	+	+
				60	O	+	+	+	+	+			+		O	O
				80		+			+	+			+			
				100						+						
				120						+						
				140												
Sodium carbonate	see soda		cold saturated, aqueous													
Sodium chlorate (SpRB)	$NaClO_3$		all, aqueous	20	+	+	+	+	+	O		+	+	+	+	+
				40	+	+	+	+	+			+	+	O	+	+
				60	O	+	+	+	+			+	+	-	O	+
				80		+						O	+		-	O
				100								-				
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Sodium chlorite (SpRB)	$NaClO_2$		diluted, aqueous	20	O	+		+	+	+		+	+	-	O	+
				40		+		O	O	O		+	+		-	+
				60		+				O		+	+			+
				80		+										
				100												
				120												
				140												
Sodium chromate (SpRB)	Na_2CrO_4		diluted, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+		+	+		+	+	O	+	+
				60	O		+			+		+	+	-	O	O
				80						+						
				100						+						
				120												
				140												
Sodium disulphite	$Na_2S_2O_5$		all, aqueous	20	+	+		+	+	+		+	+	O	+	+
				40	+	+				+		+	+	-	+	+
				60	O	+				+		+	+		+	O
				80						+						
				100						+						
				120												
				140												
Sodium dithionite	see hyposulphite		up to 10%, aqueous													
Sodium fluoride	NaF		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+			+		+	+	+	+	+
				60	+	+	+			+		+	+	O	+	+
Sodium hydroxide (see Caustic soda)				80						+						
				100						+						
				120												
				140												
Sodium hypochlorite (SpRB)	NaOCl		12,5% active chlorine, aqueous	20	+			O	O	O		+	+	-	-	+
				40	+			-	-							
				60	O											
Sodium iodide	NaI		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+			+			+	+	+	+
				60	O	+	+			+			+	O	+	O
				80		+				+						
				100						+						
				120												
				140												
Sodium nitrate	$NaNO_3$		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+		+	+	+		+	+	+	+	+
				80					+	+						
				100						+						
				120						+						
				140												
Sodium nitrite	$NaNO_2$		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+			+	O	+	+
				60	+	+		+	+	+			+	-	+	+
				80		+			+	+						
				100						+						
				120						+						
				140												

(Courtesy George Fischer Engineering Handbook)

Aggregate Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Sodium oxalate	$Na_2C_2O_4$		cold saturated, aqueous	20	+	+	+	+	+	+			+	+	+	+
				40	+	+				+						
				60	O	+				O						
				80												
				100												
				120												
				140												
Sodium perborate	$NaBO_3\ 4H_2O$		saturated	20	+	+		+	+	+		+	+			
				40	+	+				+						
				60	+	+				+						
				80		+										
				100												
				120												
				140												
Sodium perchlorate	$NaClO_4$		saturated	20	+	+		+	+			+	+			
				40	+	+										
				60	+	+										
				80		+										
				100												
				120												
				140												
Sodium persulphate (SpRB)	$Na_2S_2O_8$		cold saturated, aqueous	20	+			+	+	+		+	+	-	+	+
				40	+			+	+	+		+	+		+	+
				60	O			+	+	+			+		+	+
				80									+		O	O
				100									+			
				120												
				140												
Sodium phosphate	Na_3PO_4		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	+	+	+
				60	O	+		+	+	+		+	+	+	+	+
				80		+			+	O						
				100					+	-						
				120												
				140												
Sodium silicate	Na_2SiO_3		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+		+	+	O		+	+	+	+	+
				80					+	-						
				100												
				120												
				140												
Sodium Sulfide	Natriumsulfid															
Sodium sulphate	Na_2SO_4, $NaHSO_4$		cold saturated, aqueous	20	+	+	+	+	+	+		O	+	+	+	+
				40	+	+	+	+	+	+		O	+	+	+	+
				60	O	+	+	+	+	+		O	+	+	+	+
				80					+	+			+			
				100						+						
				120						+						
				140												
Sodium sulphide	Na_2S		cold saturated, aqueous	20	+	+	+	+	+	O		+	+	+	-	+
				40	+	+	+	+	+	O		+		+		+
				60	O	+	+	+	+	O		+		+		+
				80		+			+							
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Sodium sulphite	Na_2SO_3		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	O	+	+
				60	O	+		+	+	+		+	+	-	O	+
				80		+			+	+						
				100						+						
				120												
				140												
Sodium thiosulphate	$Na_2S_2O_3$		cold saturated, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40										O	+	+
				60										-	O	O
				80												
				100												
				120												
				140												
Sodiumchloride	NaCl		each, aqueous	20	+	+	+	+	+	+		+	+			
				40	+	+	+	+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+		+	+			
				100						+			+			
				120												
				140												
Sodiumcyanide	NaCN			20	+	+		+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+							
				100												
				120												
				140												
Sodiumdichromate	Na_2Cs2O_7			20	O	+		+	+			+	+			
				40		+		+				+	+			
				60		+							+			
				80		+										
				100												
				120												
				140												
Sodiumhydrogen-carbonate	$NaHCO_3$			20	+	+	+	+	+	+		+	+			
				40	+	+		+	+	+		+				
				60	+	+		+	+	+		+				
				80		+			+							
				100												
				120												
				140												
Sodiumhydrogensulfate	$NaHSO_4$			20	+	+	+	+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+						
				100						+						
				120												
				140												
Spindle oil				20	O	O	-	O	+	+		-	+	+	O	-
				40					O	+			O	+	-	O
				60				O	-	+			-	O		-
				80						+				-		
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Spinning bath acids containing carbon disulphide (SpRB)			100 mg CS_2/l	20	+			+	+	+			+	-	-	O
				40	+					+						
				60												
				80												
				100												
				120												
				140												
Spinning bath acids containing carbon disulphide (SpRB)			200 mg CS_2/l	20	O			+	+	+		-	+	-	-	-
				40						+						
				60												
				80												
				100												
				120												
				140												
Spinning bath acids containing carbon disulphide (SpRB)			700 mg CS_2/l	20	-			+	+	+		-	+	-	-	-
				40						+						
				60												
				80												
				100												
				120												
				140												
Stannous chloride	see Tin II chloride		cold saturated, aqueous													
Stannous chloride - Tin IV chloride	$SnCl_4$		cold saturated, aqueous	20				+	+							
				40				+	+							
				60				+	+							
				80					+							
				100												
				120												
				140												
Starch solution	$(C_6H_{10}O_5)n$		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	+	+		+	+	+		+	+	+	+	+
				80		+				+						
				100						+						
				120												
				140												
Starch syrup			usual commercial	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	+	+		+	+	+		+	+	+	+	+
				80		+			+	+		+	+	+	+	+
				100						+		+	+	+	+	+
				120												
				140												
Stearic acid (SpRB)	$C_{17}H_{35}COOH$	Fp. 69	technically pure	20	+	O	+	+	+	+		+	+	+	+	O
				40	+		+			+		+	+	+	+	O
				60	+			O	O	+		O	O	O	O	-
				80						+						
				100						+						
				120						+						
				140												
Styrol				20	-	-	-			+			+			
				40												
				60												
				80												
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Succinic acid	HOOC-CH_2-CH_2-COOH	Fp*., 185	aqueous, all	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	+	+	+
				60	+	+		+	+	+		+	+	+	+	+
				80									+			
				100												
				120												
				140												
Sugar syrup			usual commercial	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	O	+	+	+		+	+	+	+	+
				60	O	+		+	+	+		+	+	+	+	+
				80					+	+		+	+			+
				100					+	+						
				120						+						
				140												
Sulfur	S	Fp.*, 119	technically pure	20	O	O	-	+	+	+		+	+	-	+	+
				40	-			+	+	+			+			+
				60				+	+	+			+			+
				80					+	+			+			+
				100						+						
				120						+						
				140												
Sulfur dioxide	SO_2	-10	technically pure, anhydrous	20	+	+	-	+	+	O		+	+	-	-	O
				40	+	+		+	+	O		O	O			-
				60	+	+		+	+	-		-				
				80												
				100												
				120												
				140												
Sulfur dioxide	SO_2		technically pure, moist	20	-	-	-	-	-	-		-	O	-	-	O
				40												
				60												
				80												
				100												
				120												
				140												
Sulfur dioxide	SO_2		all, moist	20	+	+	-	+	+	+		+	+	-	-	O
				40	+	+		+	+	O		O	O			-
				60	O			+	+	-		-	-			
				80												
				100												
				120												
				140												
Sulfur trioxide	SO_3			20	-	-	-	-	-	-		-	-	-	-	-
				40												
				60												
				80												
				100												
				120												
				140												
Sulfuric acid saturated by Chlorine	H_2SO_4+Cl_2		60%	20						+						
				40						+						
				60						+						
				80						+						
				100						+						
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Sulfuric acid (see note 2.3.1 on jointing)	H_2SO_4	120	up to 40%, aqueous	20	+	+	+	+	+	+		+	+	O	+	+
				40	+	+	O	+	+	+		+	+	-	O	+
				60	O	+		+	+	+		+	+			O
				80		+				+		O	O			O
				100						+		-	-			-
				120						+						
				140												
Sulfuric acid (see note 2.3.1 on jointing) (SpRB)	H_2SO_4	140	up to 60%, aqueous	20	+	+	-	+	+	+		+	+	-	-	+
				40	+	+		+	O	+		+	+			O
				60	+	+		+	-	+		O	+			O
				80		+				+		-	O			-
				100						+						
				120						+						
				140												
Sulfuric acid (see note 2.3.1 on jointing) (SpRB)	H_2SO_4	195	up to 80%, aqueous	20	+	+	-	+	+	+		O	+	-	-	+
				40	+	+		+	+	+		O	+			O
				60	+	+		O	O	+		-	O			-
				80		+				+			-			
				100						+						
				120						O						
				140												
Sulfuric acid (see note 2.3.1 on jointing) (SpRB)	H_2SO_4	250	90%, aqueous	20	+	+	-	O	O	+		-	+	-	-	-
				40	+	+				+			+			
				60		+				+						
				80						+						
				100						O						
				120						O						
				140												
Sulfuric acid (see note 2.3.1 on jointing) (SpRB)	H_2SO_4		96%, aqueous	20	+	+	-	-	-	+		-	+	-	-	-
				40	+	+				+			+			
				60	O	+				-						
				80												
				100												
				120												
				140												
Sulfuric acid (see note 2.3.1 on jointing) (SpRB)	H_2SO_4		97%	20	+	+	-	-	-	O		-	+	-	-	-
				40		+										
				60												
				80												
				100												
				120												
				140												
Sulfuric acid (see note 2.3.1 on jointing) (SpRB)	H_2SO_4	340	98%	20	+	+	-	-	-	-		-	O	-	-	-
				40	O	+										
				60		O										
				80												
				100												
				120												
				140												
Sulfurous acid	H_2SO_3		saturated, aqueous	20	+	+	O	+	+	+		+	+	-	-	O
				40	+	+	+	+	+	+		-	+			O
				60	O			+	+	+			O			-
				80						+			-			
				100						+						
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Sulfuryl chloride	SO_2Cl_2	69	technically pure	20	-	-	-	-	-	O			+	-	O	+
				40												
				60												
				80												
				100												
				120												
				140												
Surfactants (SpRB)			up to 5%, aqueous	20	O	-	-	+	+	+		+	+	+	+	+
				40	O				O	O						
				60	O				O	O						
				80						O						
				100												
				120												
				140												
Surfactants (ESC)				20	O	O	O	O	O	O		O	O	O	O	
				40												
				60												
				80												
				100												
				120												
				140												
Tallow (SpRB)			technically pure	20	+	-	-	+	+	+		+	+	+	+	+
				40	+			+	+	+		+	+	+	+	+
				60	+			+	+	+		+	+	+	+	+
				80						+						
				100						+						
				120												
				140												
Tannic acid (SpRB)			all, aqueous	20	+	+	+	+	+				+	+	+	+
				40		+	+	+	+							
				60		+		+	+							
				80												
				100												
				120												
				140												
Tanning extracts form plants (SpRB)			usual commercial	20	+	+	+	+	+	+		+	+	+	+	+
				40		+	+									
				60												
				80												
				100												
				120												
				140												
Tartaric acid				20						+						
				40						+						
				60												
				80												
				100												
				120												
				140												
Tartaric acid	$HO_2C\text{-}CH(OH)\text{-}CH(OH)\text{-}CO_2H$		all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+		+	+	+	+		+	+	+	+	+
				60	O			+	+	+		+	+	O	+	+
				80						+						
				100						+						
				120						+						
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Tartaric acid up to 10%				20						+						
				40						+						
				60						+						
				80												
				100												
				120												
				140												
Tetrachlorethylene				20	-	-	-	-	-	+		-	+			
				40												
				60												
				80												
				100												
				120												
				140												
Tetrachloroethane	$Cl_2CH\text{-}CHCl_2$	146	technically pure	20	-	-	-	O	O	+		-	O	-	-	-
				40						+						
				60						O						
				80												
				100												
				120												
				140												
Tetrachloroethylene	see Perchloroethylene	121														
Tetraetylene lead (SpRB)	$(C_2H_5)_4Pb$		technically pure	20	+	+	-	+	+	+		O	+	+	O	+
				40						+						
				60						+						
				80						+						
				100						+						
				120						+						
				140												
Tetrahydrofurane	C_4H_8O	66	technically pure	20	-	-	-	O	O	-		O	-	-	-	-
				40												
				60												
				80												
				100												
				120												
				140												
Tetrahydronaphthalene	Teralin	207	technically pure													
Thionyl chloride	$SOCl_2$	79	technically pure	20	-	-	-	-	-	-		O	+	-	-	-
				40												
				60												
				80												
				100												
				120												
				140												
Tin (IV) -chloride				20	+	+				+		+	+			
				40	+	+				+		+	+			
				60	+	+				+		+	+			
				80		+				+			+			
				100						+						
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Tin-(II)-chloride	$SnCl_2$			20				+	+							
				40				+	+							
				60				+	+							
				80					+							
				100												
				120												
				140												
Toluene	C_6H_5-CH_3	111	technically pure	20	-	-	-	O	O	+		-	+	-	-	-
				40												
				60												
				80												
				100												
				120												
				140												
Triacetin (Glycerintriacetat)	$C_9H_{14}O_6$			20	-	-	-	+	+	+		+				
				40						+						
				60												
				80												
				100												
				120												
				140												
Tributylphosphate	$(C_4H_9)_3PO_4$	289	technically pure	20	-	-	-	+	+	+		+	-	-	-	-
				40				+	+							
				60				+	+							
				80												
				100												
				120												
				140												
Trichloroacetic acid	Cl_3C-COOH	196	technically pure	20	O	-	-	+	+	O		O	-	-	-	-
				40				O	+							
				60				-	O							
				80												
				100												
				120												
				140												
Trichloroacetic acid	Cl_3-C-COOH		50%, aqueous	20	+	-	-	+	+	+		O	-	-	-	-
				40	O			+	+	+						
				60				+	O	O						
				80						-						
				100												
				120												
				140												
Trichloroethane	Methylchloroform	74	technically pure													
Trichloroethylene	Cl_2C=CHCl	87	technically pure	20	-	-	-	-	O	+		-	+	-	-	-
				40						+						
				60						+						
				80						O						
				100												
				120												
				140												
Trichloromethane	Chloroform	61														

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Tricresyl phosphate (SpRB)	$H_3C\text{-}C_6H_5\text{-}O)_3PO_4$		technically pure	20	-	-	-	+	+			+	-	O	-	-
				40				+						-		
				60				+	O							
				80												
				100												
				120												
				140												
Triethanolamine (SpRB)	$N(CH_2\text{-}CH_2\text{-}OH)_3$	Fp. *21	technically pure	20	O	-	-	+	+	+		O	-	O	-	-
				40						+						
				60												
				80												
				100												
				120												
				140												
Triethylamine (SpRB)	$N(CH_2\text{-}CH_3)3$	89	technically pure	20	-	-	-	+	+	O		-	-	-	-	-
				40						-						
				60												
				80												
				100												
				120												
				140												
Trifluoro acetic acid (SpRB)	$F_3C\text{-}COOH$		up to 50%	20	-	-	-	+	+	+		O	-	-	-	-
				40						O						
				60												
				80												
				100												
				120												
				140												
Trioctyl phosphate (SpRB)	$(C_8H_{17})_3\ PO_4$		technically pure	20	-	-	-	+	+	O		+	-	O	-	-
				40				+	O							
				60												
				80												
				100												
				120												
				140												
Turpentine oil (SpRB)			technically pure	20	+	-	-	O	-	+		-	+	O	-	-
				40	O			O					+			
				60									+			
				80												
				100												
				120												
				140												
Urea (SpRB)	$H_2N\text{-}CO\text{-}NH_2$	Fp.*, 133	up to 30%, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+		+	+	+		+	+	+	+	+
				80		O				+						
				100						O						
				120												
				140												
Urine				20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+		+	+	+		+	+	+	+	+
				80						+						
				100						+						
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

Aggregssive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Vaseline			technically pure	20	O	O	-	+	O	+		-	+	+	-	-
				40	-					+			+	+		
				60				-	-	+			+	+		
				80						+			+	+		
				100						+			+	+		
				120						+			+			
				140												
Vegetable oils				20	O	-	-	+	+	+		-	+	+	O	O
				40						+						
				60						+						
				80												
				100												
				120												
				140												
Vegetable oils and fats (SpRB)				20	+	O	-	+	+	+		-	+	+	O	O
				40	O			O	+	+			+	+	O	O
				60					O	+			+	+	-	-
				80						+						
				100						+						
				120												
				140												
Vinegar	see wine vinegar															
Vinyl acetate	CH_2=CHOOCCH_3	73	technically pure	20	-	-	-	+	+	+		+	-	-	-	-
				40				+		-						
				60					O							
				80												
				100												
				120												
				140												
Vinyl chloride	CH_2=CHCl	-14	technically pure	20	-	-	-	-	-	+		-	+	-	-	
				40						+						
				60												
				80												
				100												
				120												
				140												
Viscose spinning solution				20	+	-	-	+	+	+			+	-	O	+
				40	+			+	+	+		+	+		O	+
				60	+			+	+	+		+	+		-	+
				80												
				100												
				120												
				140												
Waste gases containing - Alkaline				20	+	+		+	+	+		+	+	+	+	+
				40	+	+		+	+	O		+	+	+	+	+
				60	+	+		+	+	-		+	+	+	+	O
				80		+			+			+	O			-
				100									-			
				120												
				140												
Waste gases containing - Carbon oxides			all	20	+	+		+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	+	+	+
				60	+	+		+	+	+		+	+	+	+	+
				80		+			+	+		+	+			+
				100						+			+			
				120						+						
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Waste gases containing - Hydrochloric acid			all	20	+	+		+	+	+		+	+	O	+	+
				40	+	+		+	+	+		+	+	-	+	+
				60	+	+		+	O	+		+	+		+	+
				80		+				+		O	+			+
				100						+			+			
				120						+						
				140												
Waste gases containing - Hydrogen fluoride (SpRB)			traces	20	+	+		+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	O	+	+
				60	+	+		+	+	+		O	+	-	O	+
				80		+				+			+			
				100						+						
				120												
				140												
Waste gases containing - Nitrous gases			traces	20	+	+		+	+	+		+	+	O	+	+
				40	+	+		+	+	+		+	+	-	+	+
				60	+	+		+	O	+		+	+		O	+
				80		+				+		O	+			O
				100						+			O			
				120												
				140												
Waste gases containing - Sulphur dioxide			traces	20	+	+		+	+	+		+	+	O	+	+
				40	+	+		+	+	+		+	+	-	+	+
				60	+	+		+	+	+		+	+		+	+
				80		+			+	+		+	+			+
				100						+			+			
				120						+						
				140												
Waste gases containing - Sulphur trioxide (SpRB)			traces	20	+	+		+	+	+		+	+	O	+	+
				40	+	+		+	+	+		+	+	-	+	+
				60	+	+		+	O	+		+	+		+	+
				80		+				+		O	+			
				100						+						
				120												
				140												
Waste gases containing - Sulphuric acid			all	20	+	+		+	+	+		+	+	O	+	+
				40	+	+		+	+	+		+	+	-	+	+
				60	+	+		+	+	+		+	+		+	+
				80		+			O	+		O	+			+
				100						+			+			
				120						+						
				140												
Water - distilled - deionised	H_2O	100		20	+	+	+	+	+	+			+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	+	+	+	+	+	+		O	+	+	+	+
				80		+			+	+		-	+	+		+
				100					+	+			+	+		
				120						+			+			
				140												
Water, condensed				20	+	+	+	+	+	+			+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	O	+	+	+	+	+		O	+	+	+	+
				80		+			+	+			+	O		+
				100						+						
				120						+						
				140												

(Courtesy George Fischer Engineering Handbook)

Aggressive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Water, drinking, chlorinated				20	+	+	+	+	+	+			+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60	+	+	+	+	+	+		O	+	+	+	+
				80					+	+			+	O		+
				100					+	+			+			
				120						+						
				140												
Water, waste water without organic solvent and surfactants				20	+	+	+	+	+	+			+	+	+	+
				40	+	+	+	+	+	+		+	+	+	+	+
				60		+	+	+	+	+		O	+	+	+	+
				80		+			+	+			+	+		+
				100						+				O		
				120						+						
				140												
Wax alcohol (SpRB)	$C_{31}H_{63}OH$		technically pure	20	+	O	-	O	O	+		+	+	+	+	-
				40	+			-	-	+		+	+	+	+	
				60	+					+		+	+	+	+	
				80												
				100												
				120												
				140												
Wine vinegar (SpRB)			usual commercial	20	+	o	O	+	+	+		+	O	-	O	+
				40	+			+	+	+			-		-	O
				60	+			+	+	+						-
				80					+	+						
				100						+						
				120												
				140												
Wines, red and white			usual commercial	20	+	O	+	+	+	+		+	+	+	+	+
				40			+	+	+	+						
				60				+	+	+						
				80						+						
				100						+						
				120												
				140												
Xylene	$C_6H_4(CH_3)_2$	138? 144	technically pure	20	-	-	-	-	-	+		-	+	-	-	-
				40						+			O			
				60						O			-			
				80						-						
				100												
				120												
				140												
yeasts			all, aqueous	20	+	+	+	+	+	+		+	+	+	+	+
				40	+	+		+	+	+		+	+	+	+	+
				60		+		+	+	+						
				80		+				+						
				100												
				120												
				140												
Zinc salts	$ZnCl_2$, $ZnCO_3$, $Zn(NO_3)_2$, $ZnSO_4$		all, aqueous	20	+	+	+	+	+	+			+	+	+	+
				40	+	+	+	+	+	+		+	+	O	+	+
				60	O	+		+	+	+		+	+	-	+	+
				80						+						
				100						+						
				120						+						
				140												

(Courtesy George Fischer Engineering Handbook)

Aggregessive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
Zinccarbonate				20	+	+	+	+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+		+	+			
				100						+						
				120												
				140												
Zincchloride			saturated	20	+	+	+	+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+			+			
				100						+						
				120												
				140												
Zincnitrate	$Zn(NO_3)_2$		saturated	20	+	+	+	+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+			+			
				100						+						
				120												
				140												
Zincoxide			Suspension	20						+						
				40						+						
				60						+						
				80						+						
				100						+						
				120												
				140												
Zincphosphate			saturated	20	+	+	O	+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+			+			
				100						+						
				120												
				140												
Zincstearate			Suspension	20	-	-	-	+	+	+		+	O			
				40				+	+	+		+				
				60				+	+	+		+				
				80					+	+						
				100						+						
				120												
				140												
Zincsulfate	$ZnSO_4$			20	+	+		+	+	+		+	+			
				40	+	+		+	+	+		+	+			
				60	+	+		+	+	+		+	+			
				80		+			+	+		+	+			
				100						+			+			
				120												
				140												
1-Chloropentan	$C_5H_{11}Cl$			20	-	-	-									
				40												
				60												
				80												
				100												
				120												
				140												

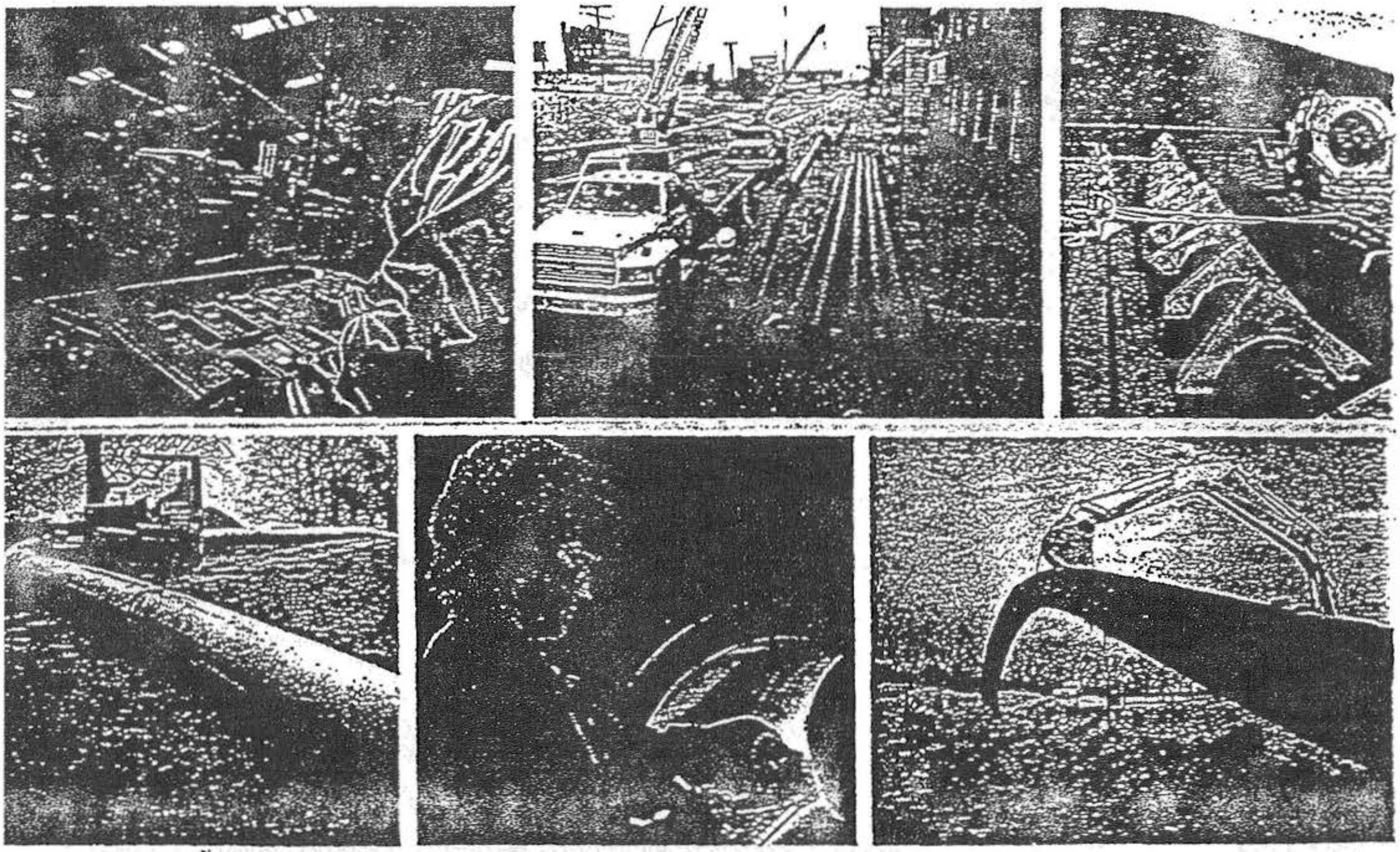
(Courtesy George Fischer Engineering Handbook)

Aggregive Media					Chemical Resistance											
Medium	Formula	Boiling point °C	Concentration	Temperature °C	PVC	CPVC	ABS	PE	PP-H	PVDF (SYGEF)		EPDM	FPM	NBR	CR	CSM
1,1,2-Trifluoro, 1,2,2-Trichloroethane (Freon 113) (SpRB)	$FCl_2C\text{-}CClF_2$	47	technically pure	20	+		-			+			+	+	+	+
				40	+											
				60												
				80												
				100												
				120												
				140												

(Courtesy George Fischer Engineering Handbook)

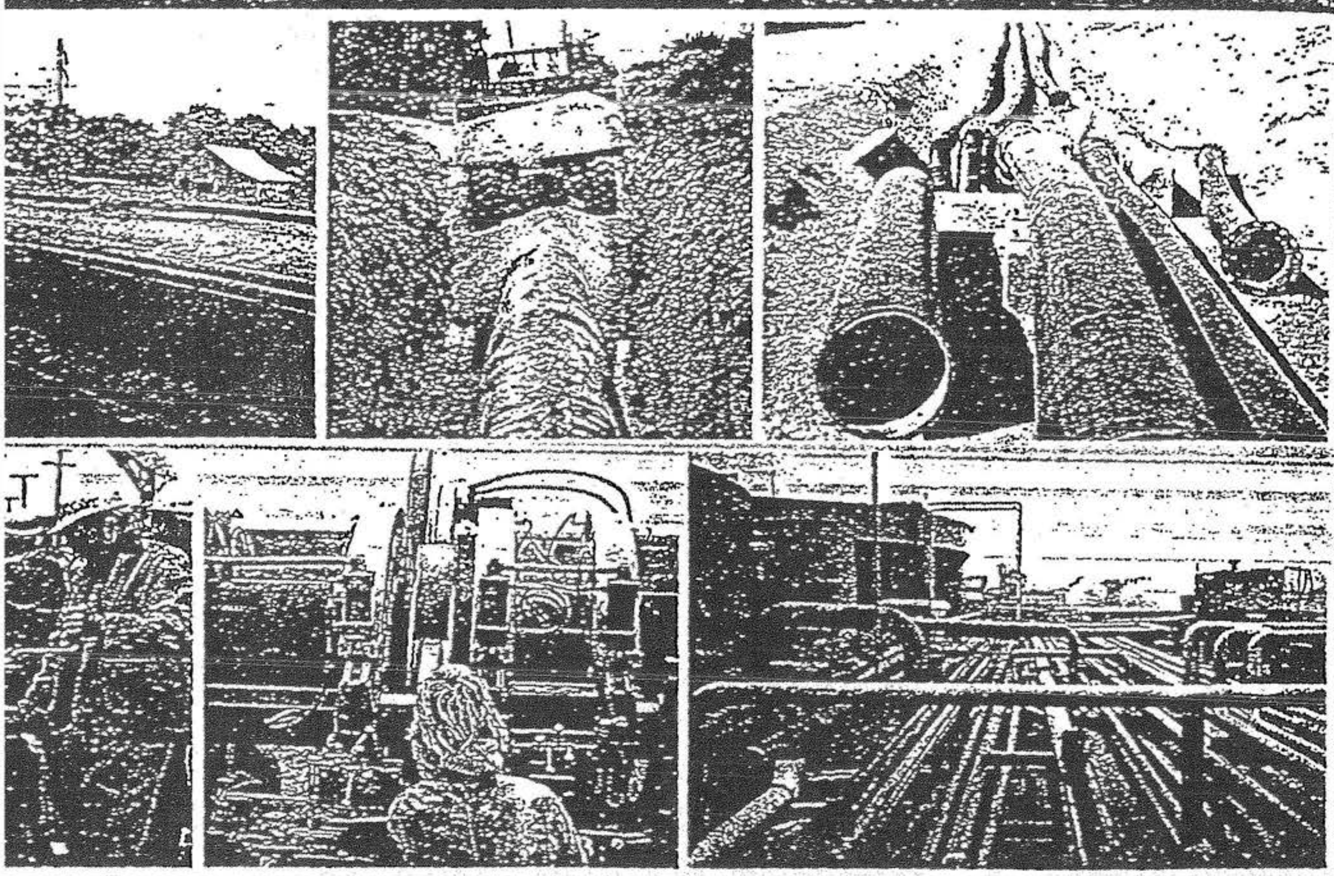
DRISCOPIPE

Engineering Characteristics



Contents

Photographs shown are typical Driscopipe installations.



Driscopipe® Engineering Characteristics

Introduction

Driscopipe high density polyethylene piping systems offer the modern engineer the opportunity to take advantage of the unusual characteristics of these materials and use them to solve many old problems and to design systems for applications where traditional materials are either unsuitable or too expensive. When compared to the older traditional piping materials, Driscopipe polyethylene piping systems offer a new freedom in environmental design, extended service life, significant savings for installation labor and equipment costs, and reduced maintenance for pipeline systems where operating conditions are within the pressure and temperature capabilities of the material.

This brochure outlines the Engineering Characteristics of Driscopipe high density polyethylene pipe and fittings and points out many of the advantages and benefits to be realized through the use of these systems. The discussion is directed primarily toward the large diameter (3" through 54") Driscopipe 8600 and Driscopipe 1000 Industrial and Municipal product lines. However, these engineering characteristics are also typical of other Driscopipe polyethylene product lines.

Physical Properties

Driscopipe 8600 is manufactured from Marlex M-8000 very high molecular weight high density PE 3408 resin. Pipe and fittings made from Marlex M-8000 are extremely tough and durable, and possess exceptional long term strength. Marlex M-8000 is a proprietary product and is extruded only by Phillips Driscopipe, Inc.

Driscopipe 1000 is manufactured from Marlex TR-480, a PE 3408 polyethylene pipe resin in a molecular weight range which permits the pipe to be extruded by conventional methods. In this respect, Driscopipe 1000 is comparable to other extra high molecular weight, high density, PE 3408 polyethylene pipes commercially available in North America.

Sheets detailing typical physical properties for Driscopipe 1000 and Driscopipe 8600 are available upon request.

Long Term Hydrostatic Strength

One of the outstanding engineering characteristics of Driscopipe high density polyethylene pipe is its long term hydrostatic strength under various thermal and environmental conditions. Life expectancy is conservatively estimated to be in excess of 50 years using the standard design basis. This strength is determined by standardized methods and procedures which the plastic pipe industry has used for many years to evaluate the long term strength of all types of plastic pipe.

Pipe hoop stress versus time to failure plots of long term hydrostatic pressure data for thermoplastic pipe have been studied and analyzed for many years. The mathematical equations used to evaluate the test data and extrapolate values to longer periods of time were chosen after careful evaluation of more than 1,000 sets of long term test data representing more than 400 plastic pipe compounds. Continued testing on new compounds and extended testing of older compounds have proven the validity of these test methods. Actual data from more than 11½ years (100,000 hours) of continuous testing shows the industry methods to be slightly conservative in that actual values are slightly higher than those calculated by the industry-accepted ASTM method.

The reduction in strength which occurs with time, as indicated by the stress-life curves, does not represent a strength degradation of the material but is more in the nature of a relaxation effect. Plastic pipe samples which have been on test for periods up to 70,000 hours have been de-pressurized and checked for permanent reduction of strength by using the quick-burst test. No loss has been found when compared to samples previously quick-burst from the same test lot.

All evidence confirms that the methods used to predict the long term strength of plastic pipe are sound methods. Through the years, these policies and procedures, used to develop recommended hydrostatic design strengths, have influenced manufacturers to research and develop improved piping products such as Driscopipe 8600 and Driscopipe 1000.

Typical calculated long term strengths are shown below:

Long Term Strength @ 73.4°F(23°C)

Time	Hoop Stress, psi
100,000 hrs. (11.43 yrs.)	1635
438,000 hrs. (50 yrs.)	1604
500,000 hrs. (57 yrs.)	1601
1,000,000 hrs. (114 yrs.)	1586

The 114-year long term strength has been included to show more about the nature of the method used by the industry to evaluate the long term strength of plastic pipe and to illustrate the very slow reduction in strength as time progresses.

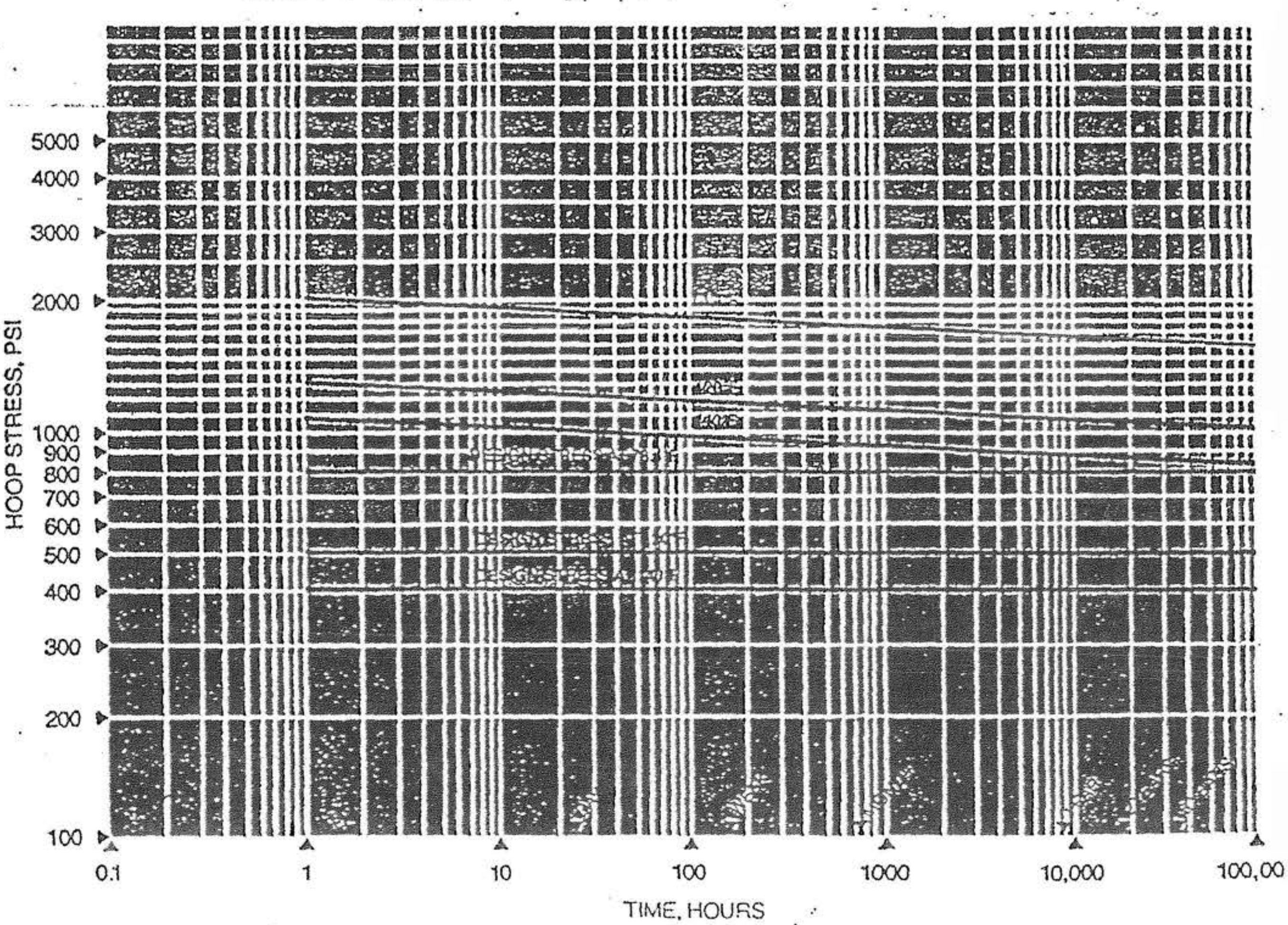
Long term hoop stresses for design purposes are normally selected at a level which is much lower than the long term strength of the materials. This ensures that the pipe is operating in a hoop stress range where creep (relaxation) of the materials is nil and assures service life in excess of 50 years. Design stress levels are discussed further in the next section.

The long term hydrostatic tests are conducted by using ASTM standard test procedures which may be applied to all types of plastic pipe (ASTM D 1598 Test for Time-to-Failure of Plastic Pipe Under Constant Internal Pressure). Stress-life tests are conducted by using numerous pipe samples which are filled with water (or other environmental fluids) and subjected to a controlled pressure at a controlled temperature. Samples are held on test until they fail. The pressure, temperature and time-to-failure data from all samples are used to calculate and plot stress-life curves for the particular type pipe being tested (ASTM D 2837 Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials). This data is then used to predict the probable safe life of the pipe at various stress levels (working pressures) and various temperatures. Because it is not practical to test at all temperature levels, these tests are generally conducted at temperatures of 73.4°F and one or more higher temperatures such as 100°F, 120°F and 140°F.

These stress-life curves give a relationship of the expected life span of the pipe when subjected to various internal stress levels (working pressures) at various temperatures. By comparing stress-life curves, one can compare relative long term performance ability of different plastic pipes. Stress-life curves for Driscopipe 8600 and Driscopipe 1000 are shown in Figure 1.

Figure 1

Stress-Life of Driscopipe® 8600 and Driscopipe® 1000



T- -se stress-life curves were obtained using water as st medium. However, years of laboratory testing a field experience have shown that these same curves may be used to design Driscopipe systems for natural gas, salt water, sewage and hundreds of other industrial and municipal fluids, mixtures and effluents. The long term strength of Driscopipe indicated by these curves must be de-rated in some environmental circumstances, such as in the presence of liquid hydrocarbons or abrasive fluids, although the pipe is very suitable for use in these environments. An outstanding engineering advantage of Driscopipe is its exceptionally long term service life in the presence of internal and external corrosive service conditions.

Design Pressure Ratings

Since plastic pipe was introduced in the late 50s, the safety factor for design of water systems at standard temperature has been 2 to 1. The 2:1 design factor which was officially adopted by the plastic pipe industry in 1963, was based on allowances for many sources of variation. The guiding principle has always been to make the selection on a conservative basis but not to be unreasonably conservative.

The sources of variation for which allowances are made include ... variation in test methods and procedures among laboratories ... variation among s of the same compound ... variation of lots of pipe from the compound in different plants and from different extruders ... variation in compounds of the same general class ... variations in handling and installation techniques ... variation in operating pressures (water hammer and surge) ... a strength-time allowance to give service life well beyond 50 years ... and, finally, the great unknown. Each of the factors was judged to reduce the 100,000 hour design strength by 5%-10% or 20% ... for a total of 100% ... or a design factor of 2:1. This is why polyethylene pipe, with a designated 100,000 hour strength of 1600 psi at 73.4°F, has a hydrostatic design strength of 800 psi hoop stress.

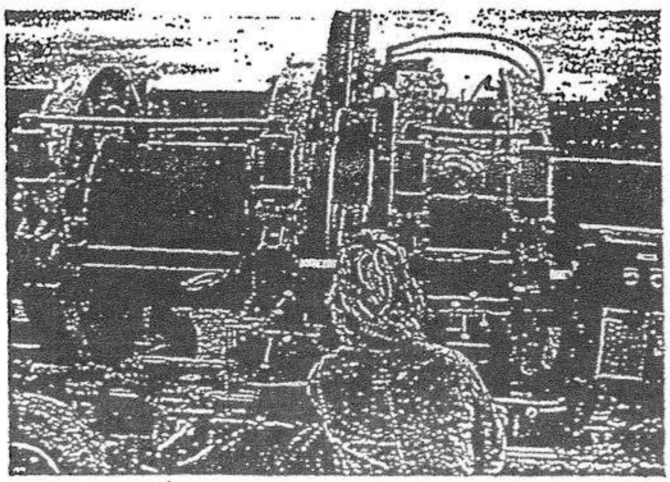
The design pressures for Driscopipe are determined by the following equation, adopted internationally by the industry for this purpose:

$$P = \frac{2S}{SDR-1} \times F \quad \text{or} \quad P = 2S\frac{t}{D-t} \times F$$

Where:
- D = Specified Outside Diameter, Inches
- P = Design Pressure, psi
- S = Long Term Hydrostatic Strength, psi, at the design temperature
- t = Minimum Wall Thickness, Inches
- F = Service Design Factor
- SDR = Standard Dimension Ratio of D/t

The traditional Service Design Factor for water at standard temperature (73.4°F) is one-half (.5). The Service Design Factor for oil or liquid hydrocarbons is 0.25 @ 73°F. The service design factor may be adjusted by the design engineer to reflect the particular conditions anticipated for the application. The temperature selected for design should consider both internal and external conditions. The design temperature should be based on the temperature of the pipe itself. For practical purposes, it is safer to design to the highest temperature.

The design service factor for water may also be used for solutions of inorganic salts, alkaline fluids, non-oxidizing acids, low concentrations of oxidizing acids and many other solutions. See the discussion on chemical resistance for more information.



All standard design pressure ratings shown in Driscopipe literature are based on water at 73.4°F temperature; ie, a safety factor of 2:1 based on the long term hydrostatic strength of the material. Driscopipe is applicable at pressures from 0 to 265 psi and temperatures from below 32°F up to 180°F. Standard Dimension Ratios (SDR) are available from SDR 32.5 to SDR 7.0

Flow Characteristics

Driscopipe polyethylene has excellent flow characteristics as compared to traditional materials. An extremely smooth interior surface offers low resistance to flow. It maintains these excellent flow properties throughout its service life in most applications due to the inherent chemical and abrasion resistance of the material. Because of smooth walls and the non-wetting characteristic of polyethylene, higher flow capacity and less friction loss is possible with Driscopipe. In many cases this higher flow capacity may permit the use of smaller pipe at a lower cost.

A "C" factor of 155 is commonly used in the Hazen-Williams formula for calculating flow in pressure applications. For gravity flow, an "n" factor of .009 is used in Manning's formula.

Experimental test data regarding pumping and pressure drop through Driscopipe is available upon request. This study compares the flow through 8" Driscopipe with and without internal fusion beads using clear water. It also includes flow data for some clay-water slurries and clay-water-sand slurries. Velocities up to 20 fps are studied. Data includes determination of Hazen-Williams "C" factor, Reynolds number, boundry drag, relative roughness, sand grain roughness and friction loss at various velocities.

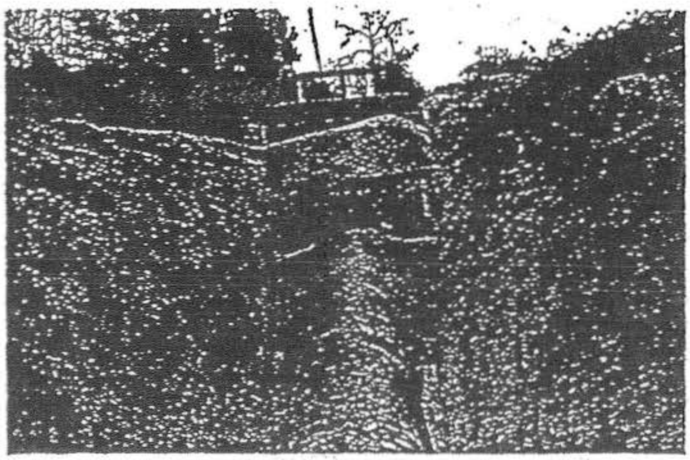
Lightweight – Flexible

The inherent light weight and flexibility of Driscopipe provides many cost saving benefits related to handling, storage, hauling, unloading, stringing, joining and installation. Because of its light weight, Driscopipe can be moved, handled and placed in the ditch with smaller and less expensive construction equipment. Usually, manpower requirements are also reduced.

Driscopipe weighs less than water; it has a specific gravity of .955-.957. Because it will float, it can be joined in long strings and easily towed into position on job sites where water is encountered. The combination of light weight and flexibility provides opportunity to fusion join the pipe in a convenient work area and pull it into position in difficult work areas where terrain or other obstacles present installation problems. The pipe can be joined above ground and rolled or lowered into the trench thus allowing the use of smaller trench widths and eliminating the necessity of placing men and equipment inside the trench. Such installation methods can dramatically reduce the time required for installation in many instances.

The flexibility of Driscopipe allows it to be curved over, under and around obstacles and to make elevation and directional changes, thus eliminating fittings and reducing installation costs. The pipe can be cold bent as it is installed to a radius of 20-40 times the pipe diameter. This flexibility and the butt fusion joining method make Driscopipe ideally suited for inserting it inside older piping systems to renew and renovate such systems at a much lower cost than would be possible otherwise.

Pipe flexibility and toughness also allow small diameter Driscopipe to be plowed-in or pulled-in with suitable equipment.



Toughness – "Ductile PE Pipe"

Overall "toughness" of Driscopipe is an important characteristic of the pipe which is derived from many of the chemical and physical properties of the material as well as the extrusion method. The pipe is ductile. It flexes, bends and absorbs impact loads over a wide temperature range of −180°F up to +180°F. This inherent resiliency and flexibility allow the pipe to absorb surge pressures, vibration and stresses caused by soil movement. Driscopipe can be deformed without permanent damage and with no adverse effect on long term service life. It is flexible for contouring to installation conditions. The toughness of Driscopipe is one of its outstanding engineering characteristics leading to innovative piping design.

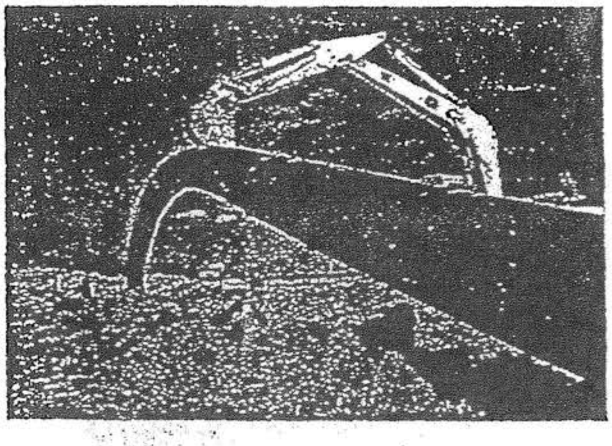
Even though "toughness" has become generally recognized by the industry as a highly desirable characteristic ... there is no standard test which can be used to directly compare the "toughness" among polyethylenes ... as well as among the different plastic materials which are considered suitable for piping.

A "toughness" test has not been devised is simply because it is influenced by so many of the physical and chemical properties of the material. The extreme toughness of Driscopipe has been noted as one of its outstanding features since its introduction to the industry ... yet to explain "toughness", many properties are discussed and demonstrated. To obtain a complete evaluation of the toughness of a plastic material, it is necessary to see demonstrations of tests and to conduct some tests in person in order to compare it with materials which are more familiar, such as cast iron, steel, cement, copper, etc.

Toughness is related to ... Environmental Stress Crack Resistance (ESCR) ... Notch sensitivity ... Resistance to secondary stresses from external loading ... Impact strength ... Tear strength ... Flexibility ... Kink resistance ... Abrasion and scratch resistance ... Flexural strength ... Elongation ... Chemical resistance ... Tensile strength ... Ductility ... Creep resistance ... Temperature resistance ... Density ... Molecular weight ... and the thermoplastic nature of the material. Part of the toughness of any polyethylene material can be attributed to its flexibility, flexural strength and impact resistance as compared to the more rigid thermoplastic materials such as PVC. Polyethylene is ductile and will elongate many times more than PVC. Consequently, it will absorb more impact without damage or failure. PE will flex or elongate and stress relieve itself rather than rupture. Generally, impact strength is greater for the higher molecular weight PE resins. Impact resistance is also important from the standpoint of a piping system being able to absorb energy imposed on it by external forces.

The expansive force of water freezing inside Driscopipe will not damage it.

ESCR is one of the properties closely related to "toughness" and has been studied as a possible means to define and measure toughness. The exceptional resistance of Driscopipe 8600 to environmental stress cracking as compared to other PE materials is discussed further in the next section.



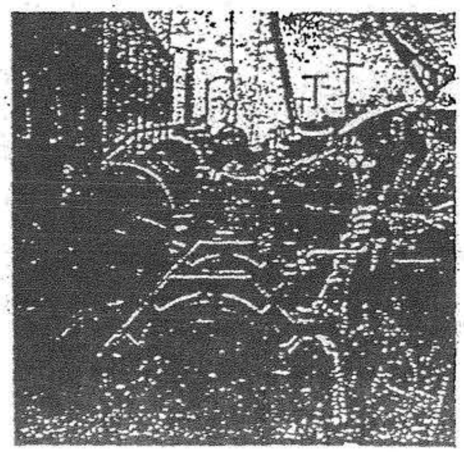
Driscopipe 8600 is unique and differs from Driscopipe 1000 and from all other polyethylene pipes. Driscopipe 8600 exhibits a superior toughness which gives the pipe the highest impact strength, highest tear strength and lowest notch sensitivity of any polyethylene pipe currently available. Driscopipe 8600 offers the highest resistance to cuts, scratches and abrasions which occur when handling and installing the pipe.

These properties are maintained throughout its temperature range without a loss of ductility or reduced resistance to notch sensitivity. Driscopipe has been successfully installed in numerous arctic applications. Some of these applications have included direct burial in the unstable arctic permafrost.

To learn more of the relative toughness of Driscopipe 8600, we encourage you to take a piece of pipe with a butt fusion joint and try to tear it up without using sharp tools. Pound it flat with a sledge hammer ... slam it against a corner of angle iron ... run over it with a truck ... then do the same with steel, copper, PVC, cast iron and the less rugged PEs. It's not very scientific ... but we believe you'll be convinced that Driscopipe 8600 has extremely high toughness. We have evaluated Driscopipe many times in laboratory and field test experiments to demonstrate and prove this toughness.

- One excellent indicator of the relative toughness of Driscopipe 8600, as compared to other polyethylene pipe materials, can be observed in the ASTM Standard Test for determination of flow rate of the thermoplastic materials.

  When Driscopipe 8600 is heated to 190°C (374°F) to measure the flow rate, it requires 432.5 pounds/sq. in. force, applied for 10 minutes, to flow 1½ grams of 8600 material through the orifice of the test unit! Other commercially available polyethylene pipe materials will flow 10 to 20 times this amount under the same conditions.
- When Driscopipe 8600 is heated to 475-500°F to melt it for fusion joining, it requires 150 pounds pressure per square inch of material to make the melted surfaces flow together. This is another indicator of toughness. Other commercially available polyethylene pipe materials require about one-half that amount of pressure and some competitive pipes require less than 25 psi!
- Driscopipe 8600 has been pressure tested for long periods at temperatures up to 140°F and performance requirements at these high temperatures can be used in purchase specifications to assure that the user is getting the highest performing polyethylene pipe.



Environmental Stress Crack Resistance

The most recent ASTM specification written to identify polyethylene plastic pipe and fittings materials is ASTM D 3350, "Polyethylene Plastics Pipe and Fittings Materials", adopted in 1974. This specification uses six (6) properties to classify PE material ... one of these is ESCR.

ASTM D 3350 lists three cell limits for ESCR classification which use the ESCR test outlined in ASTM D 1693, Test Method for Environmental Stress Cracking of Ethylene Plastics. The cell limits are:

Cell Classification Limit	Test Condition ASTM D 1693	Test Duration Hours	Percent of Failures Allowed	Test Temp. °C
1	A	48	50	50°
2	B	24	50	50°
3	C	192	20	100°

Minimum Notch for A is .020"; for B and C is .012". Minimum Thickness for A is .120"; for B and C is .070". A and B use a diluted aqueous solution reagent, C uses full strength reagent.

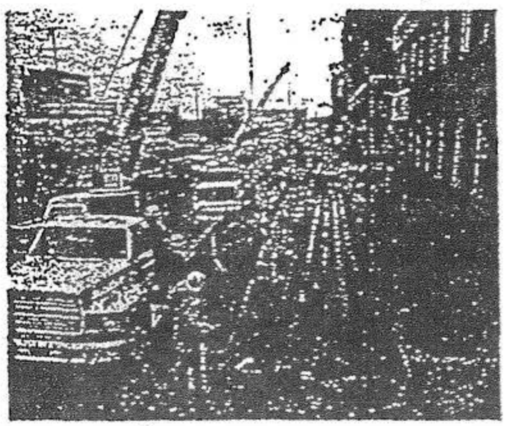
This method of testing for ESCR was first written in 1959 and was developed primarily to evaluate polyethylene as a jacketing material for power and communications cable. Although the method requires the use of laboratory compression molded specimens rather than pipe, it became the generally accepted method for evaluating ESCR of PE materials used for piping. Its wide use was responsible for its inclusion in ASTM D 3550 to describe one of the six primary properties of a PE pipe material.

The test method, ASTM D-1693, is an accelerated test method to determine the resistance of a polyethylene material to environmental stress cracking. It is a measure of the ability of the polyethylene to withstand secondary stress loadings. These loadings are typically thought of as low-level, long-term, external stresses which may act upon the polyethylene pipe in field installations.

Under conditions of the test, high local multiaxial stresses are developed through the introduction of a controlled imperfection (notch). The notched sample is subjected to an elevated temperature bath of a surface active agent. Environmental stress cracking has been found to occur most readily under such conditions.

A note in the test specifications states that, generally, low density (Type I) polyethylenes are tested under Condition A, medium and high density (Type II and Type III) polyethylenes are generally tested under Condition B and high density resins with high melt viscosity, such as pipe grade P34, are tested under Condition C.

As pipe grade polyethylenes have improved, the testing requirements of ASTM D-1693 have become less stringent for P34 pipe grade polyethylenes such as Driscopipe 8600 and Driscopipe 1000. As a result, a more severe stress crack resistance test has been developed to evaluate high density polyethylene pipe. The ASTM F-1248 stress crack resistance test method was developed by a gas distribution company for quality control purposes and is often referred to as Ring ESCR since it tests actual produced pipe ring samples rather than molded specimens.



ASTM F-1248 utilizes rings cut from a pipe sample. The rings are notched on one side and compressed between parallel plates until the distance between the plates is three times the specified pipe minimum wall thickness. The compressed ring samples are subjected to an elevated temperature bath of a surface active agent and visibly inspected for crack formation or propagation.

The Ring ESCR test provides useful information regarding the different polyethylene pipe grade materials. Driscopipe 8600 shows no tendency for sample failures when tested in excess of 10,000 hours. This further reinforces the unique ability of Driscopipe 8600 to provide the highest degree of resistance to the external stresses inherent to a pipeline installation.

Driscopipe 1000, an extra high molecular weight HDPE pipe, will exhibit a ring ESCR of $F_{50} > 1000$ hours. Other lower molecular weight pipes may exhibit lower F_{50} values.

Chemical Corrosion Resistance

The outstanding resistance of Driscopipe to attack by most chemicals makes it suitable to transport these chemicals or to be installed in an environment where these chemicals are present. Factors which determine the suitability and service life of each particular application include the specific chemical and its concentration, pressure, temperature, period of contact and service conditions which may introduce stress concentrations in the pipe or fittings.

Driscopipe is, for all practical purposes, chemically inert within its temperature use range. This advantageous engineering characteristic is one of the primary reasons for the wide use of Driscopipe in industrial applications. It does not rot, rust, pit, corrode or lose wall thickness through chemical or electrical reaction with the surrounding soil, whether acid, alkaline, wet or dry. It neither supports the growth of, nor is affected by, algae, bacteria or fungi and is resistant to marine biological attack. It contains no ingredients which make it attractive to rodents, gophers, etc.

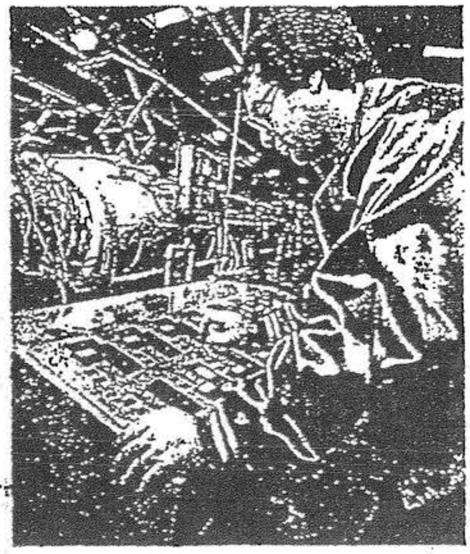
Information relative to the resistance of Driscopipe to a wide range of chemicals is shown in the following tables. This information is based on results of immersion tests (usually 3 months) at various temperatures. Changes in tensile strength and elongation are evaluated at a rapid strain rate to emphasize any strength decay in the material.

Most acids, bases and other chemicals can be transported by Driscopipe using the same design parameters as would apply to water, natural or manufactured gas and water solutions of inorganic salts. Strong oxidizing agents such as fuming sulfuric acid may adversely affect the pipe, depending upon concentration, temperature and period of contact. In many cases, such as gravity flow waste lines, these chemicals can be handled because of dilution and intermittent flow.

Some chemicals, such as all types of liquid hydrocarbons, will mechanically absorb into the wall of the pipe and cause a reduction in hoop stress but this does not degrade the material. This effect is temporary if exposure is intermittent. Where exposure is continuous, it is necessary to derate the pressure capability of the pipe for long term service. This includes such products as gasoline, ethyl alcohol, benzene, carbon tetrachloride, crude and refined oils, etc. Where 5-100% hydrocarbon liquids are continuously present in a pressure system, a service design factor of .25 should be used to calculate design pressures instead of the service design factor of .5 used with water.

$$P = \frac{2S}{SDR-1} \times F \quad \text{or} \quad P = 2S\frac{t}{D-t} \times F$$

Where:
- D = Outside Diameter, Inches
- P = Design Pressure, psi
- S = Long Term Hydrostatic Strength, psi, at the design temperature
- t = Minimum Wall Thickness, Inches
- F = Service Design Factor
- SDR = Standard Dimension Ratio of D/t



CHEMICAL RESISTANCE OF DRISCOPIPE

S – Satisfactory
U – Unsatisfactory
M – Marginal
N – Not known

All concentrations are 100% unless noted otherwise.

On reagents marked marginal, chemical attack will be recognized by a loss of physical properties of the pipe which may require a change in design factors.

Reagent	70°F (21°C)	140°F (60°C)
Acetic Acid 1-10%	S	S
Acetic Acid 10-60%	S	M
Acetic Acid 80-100%	S	M
Acetone	M	U
Acrylic Emulsions	S	S
Aluminum Chloride-Dilute	S	S
Aluminum Chloride Conc.	S	S
Aluminum Fluoride Conc.	S	S
Aluminum Sulfate Conc.	S	S
...ums (All Types) Conc.	S	S
Ammonia 100% Dry Gas	S	S
Ammonium Carbonate	S	S
Ammonium Chloride Sat'd	S	S
Ammonium Fluoride 20%	S	S
Ammonium Hydroxide 0.88 S.G.	S	S
Ammonium Metaphosphate Sat'd	S	S
Ammonium Nitrate Sat'd	S	S
Ammonium Persulfate Sat'd	S	S
Ammonium Sulfate Sat'd	S	S
Ammonium Sulfide Sat'd	S	S
Ammonium Thiocyanate Sat'd	S	S
Amyl Acetate	M	U
Amyl Alcohol 100%	S	S
Amyl Chloride 100%	N	U
Aniline 100%	S	N
Antimony Chloride	S	S
Aqua Regia	U	U
Barium Carbonate Sat'd	S	S
Barium Chloride	S	S
Barium Hydroxide	S	S
Barium Sulfate Sat'd	S	S
Barium Sulfide Sat'd	S	S
Beer	S	S
Benzene	M	U
Benzene Sulfonic Acid	S	S
Bismuth Carbonate Sat'd	S	S
...leach Lye 10%	S	S
...lack Liquor	S	S
Borax Cold Sat'd	S	S
Boric Acid Dilute	S	S

Reagent	70°F (21°C)	140°F (60°C)
Boric Acid Conc.	S	S
Bromic Acid 10%	S	S
Bromine Liquid 100%	M	U
Butanediol 10%	S	S
Butanediol 60%	S	S
Butanediol 100%	S	S
Butyl Alcohol 100%	S	S
Calcium Bisulfide	S	S
Calcium Carbonate Sat'd	S	S
Calcium Chlorate Sat'd	S	S
Calcium Chloride Sat'd	S	S
Calcium Hydroxide	S	S
Calcium Hypochlorite BL'GH Sol.	S	S
Calcium Nitrate 50%	S	S
Calcium Sulfate	S	S
Camphor Oil	N	U
Carbon Dioxide 100% Dry	S	S
Carbon Dioxide 100% Wet	S	S
Carbon Dioxide Cold Sat'd	S	S
Carbon Disulfide	N	U
Carbon Monoxide	S	S
Carbon Tetrachloride	M	U
Carbonic Acid	S	S
Castor Oil Conc.	S	S
Chlorine Dry Gas 100%	S	M
Chlorine Moist Gas	M	U
Chlorine Liquid	M	U
Chlorobenzene	M	U
Chloroform	M	U
Chlorosulfonic Acid 100%	M	U
Chrome Alum Sat'd	S	S
Chromic Acid 20%	S	S
Chromic Acid Up to 50%	S	S
Chromic Acid and Sulfuric Acid	S	M
Cider	S	S
Citric Acid Sat'd	S	S
Coconut Oil Alcohols	S	S
Cola Concentrates	S	S
Copper Chloride Sat'd	S	S
Copper Cyanide Sat'd	S	S
Copper Fluoride 2%	S	S
Copper Nitrate Sat'd	S	S
Copper Sulfate Dilute	S	S
Copper Sulfate Sat'd	S	S
Cottonseed Oil	S	S
Crude Oil*	S	M
Cuprous Chloride Sat'd	S	S
Cyclohexanol	S	S
Cyclohexanone	M	U
Detergents Synthetic	S	S
Developers, Photographic	S	S
Dextrin Sat'd	S	S
Dextrose Sat'd	S	S
Dibutylphthalate	S	M
Disodium Phosphate	S	S

Reagent	70°F (21°C)	140°F (60°C)
Diazo Salts	S	S
Diethylene Glycol	S	S
Diglycolic Acid	S	S
Dimethylamine	M	U
Emulsions, Photographic	S	S
Ethyl Acetate 100%	M	U
Ethyl Alcohol 100%	S	S
Ethyl Alcohol 35%	S	S
Ethyl Butyrate	M	U
Ethyl Chloride	M	U
Ethyl Ether	U	U
Ethylene Chloride	U	U
Ethylene Chlorohydrin	U	U
Ethylene Dichloride	M	U
Ethylene Glycol	S	S
Ferric Chloride Sat'd	S	S
Ferric Nitrate Sat'd	S	S
Ferrous Chloride Sat'd	S	S
Ferrous Sulfate	S	S
Fish Solubles	S	S
Fluoboric Acid	S	S
Fluorine	S	U
Fluosilicic Acid 32%	S	S
Fluosilicic Acid Conc.	S	S
Formaldehyde 40%	S	N
Formic Acid 0-20%	S	S
Formic Acid 20-50%	S	S
Formic Acid 100%	S	S
Fructose Sat'd	S	S
Fruit Pulp	S	S
Fuel Oil	S	U
Furfural 100%	M	U
Furfuryl Alcohol	M	U
Gallic Acid Sat'd	S	S
Gas Liquids*	S	M
Gasoline*	M	U
Gin	S	U
Glucose	S	S
Glycerine	S	S
Glycol	S	S
Glycolic Acid 30%	S	S
Grape Sugar Sat'd Aq.	S	S
Hexanol, Tert.	S	S
Hydrobromic Acid 50%	S	S
Hydrocyanic Acid Sat'd	S	S
Hydrochloric Acid 10%	S	S
Hydrochloric Acid 30%	S	S
Hydrochloric Acid 35%	S	S
Hydrochloric Acid Conc.	S	S
Hydrofluoric Acid 40%	S	S
Hydrofluoric Acid 60%	S	S
Hydrofluoric Acid 75%	S	S
Hydrogen 100%	S	S
Hydrogen Bromide 10%	S	S
Hydrogen Chloride Gas Dry	S	S

*HDPE Resin Service Design Factor for hydrocarbons per the formula on page 3 and 8 is F = 0.25 to compensate for hydrocarbon saturation effects on long term hydrostatic strength.

continued on page 10

continued from page 9

CHEMICAL RESISTANCE OF DRISCOPIPE

Reagent	70°F (21°C)	140°F (60°C)
Hydrogen Peroxide 30%	S	S
Hydrogen Peroxide 90%	S	M
Hydrogen Phosphide 100%	S	S
Hydroquinone	S	S
Hydrogen Sulfide	S	S
Hypochlorus Acid Conc.	S	S
Inks	S	S
Iodine (Alc. Sol.) Conc.	S	U
Lactic Acid 10%	S	S
Lactic Acid 90%	S	S
Latex	S	S
Lead Acetate Sat'd	S	S
Lube Oil*	S	M
Magnesium Carbonate Sat'd	S	S
Magnesium Chloride Sat'd	S	S
Magnesium Hydroxide Sat'd	S	S
Magnesium Nitrate Sat'd	S	S
Magnesium Sulfate Sat'd	S	S
Mercuric Chloride Sat'd	S	S
Mercuric Cyanide Sat'd	S	S
Mercurous Nitrate Sat'd	S	S
Mercury	S	S
Methyl Alcohol 100%	S	S
Methyl Bromide	M	U
Methyl Chloride	M	U
Methyl Ethyl Ketone 100%	M	U
Methylsulfuric Acid	S	S
Methylene Chloride 100%	M	U
Milk	S	S
Mineral Oils	S	U
Molasses Comm.	S	S
Nickel Chloride Sat'd	S	S
Nickel Nitrate Conc.	S	S
Nickel Sulfate Sat'd	S	S
Nicotine Dilute	S	S
Nicotinic Acid	S	S
Nitric Acid 0-30%	S	S
Nitric Acid 30-50%	S	M
Nitric Acid 70%	S	M
Nitric Acid 95-98%	U	U
Nitrobenzene 100%	U	U
Octyl Cresol	S	U
Oils and Fats*	S	M
Oleic Acid Conc.	S	U
Oleum Conc.	U	U
Orange Extract	S	S
Oxalic Acid Dilute	S	S
Oxalic Acid Sat'd	S	S
Ozone 100%	S	U
Perchloric Acid 10%	S	S
Petroleum Ether	U	U
Phenol 90%	U	U
Phosphoric Acid Up to 30%	S	S
Phosphoric Acid Over 30%	S	S
Phosphoric Acid 90%	S	S
Phosphorous (Yellow) 100%	S	N
Phosphorus Pentoxide 100%	S	N
Photographic Solutions	S	S
Pickling Baths		
Sulfuric Acid	S	S
Hydrochloric Acid	S	S
Sulfuric-Nitric	S	U
Plating Solutions		
Brass	S	S
Cadmium	S	S
Chromium	N	N
Copper	S	S
Gold	S	S
Indium	S	S
Lead	S	S
Nickel	S	S
Rhodium	S	S
Silver	S	S
Tin	S	S
Zinc	S	S
Potassium Bicarbonate Sat'd	S	S
Potassium Borate 1%	S	S
Potassium Bromate 10%	S	S
Potassium Bromide Sat'd	S	S
Potassium Carbonate	S	S
Potassium Chlorate Sat'd	S	S
Potassium Chloride Sat'd	S	S
Potassium Chromate 40%	S	S
Potassium Cyanide Sat'd	S	S
Potassium Dichromate 40%	S	S
Potassium Ferri/ Ferro Cyanide Sat'd	S	S
Potassium Fluoride	S	S
Potassium Hydroxide 20%	S	S
Potassium Hydroxide Conc.	S	S
Potassium Nitrate Sat'd	S	S
Potassium Perborate Sat'd	S	S
Potassium Perchlorate 10%	S	S
Potassium Sulfate Conc.	S	S
Potassium Sulfide Conc.	S	S
Potassium Sulfite Conc.	S	S
Potassium Persulfate Sat'd	S	S
Propargyl Alcohol	S	S
Propyl Alcohol	S	S
Propylene Dichloride 100%	U	U
Propylene Glycol	S	S
Rayon Coagulating Bath	S	S
Sea Water	S	S
Selenic Acid	S	S
Shortening	S	S
Silicic Acid	S	S
Silver Nitrate Sol.	S	S
Soap Solution Any Conc'n	S	S
Sodium Acetate Sat'd	S	S
Sodium Benzoate 35%	S	S
Sodium Bicarbonate Sat'd	S	S
Sodium Bisulfate Sat'd	S	S
Sodium Bisulfite Sat'd	S	S
Sodium Borate	S	S
Sodium Bromide Dilute Sol.	S	S
Sodium Carbonate Con.	S	S
Sodium Carbonate	S	S
Sodium Chlorate Sat'd.	S	S
Sodium Chloride Sat'd	S	S
Sodium Cyanide	S	S
Sodium Dichromate Sat'd	S	S
Sodium Ferricyanide	S	S
Sodium Ferrocyanide Sat'd	S	S
Sodium Fluoride Sat'd	S	S
Sodium Hydroxide Conc.	S	S
Sodium Hypochlorite	S	S
Sodium Nitrate	S	S
Sodium Sulfate	S	S
Sodium Sulfide 25%	S	S
Sodium Sulfide Sat'd Sol.	S	S
Sodium Sulfite Sat'd	S	S
Stannous Chloride Sat'd	S	S
Stannic Chloride Sat'd	S	S
Starch Solution Sat'd	S	S
Stearic Acid 100%	S	S
Sulfuric Acid 0-50%	S	S
Sulfuric Acid 70%	S	M
Sulfuric Acid 80%	S	U
Sulfuric Acid 96%	M	U
Sulfuric Acid 98%	M	U
Sulfuric Acid, Fuming	U	U
Sulfurous Acid	S	S
Tallow*	S	M
Tannic Acid 10%	S	S
Tanning Extracts Comm.	S	S
Tartaric Acid Sat'd	N	N
Tetrahydrofurane	N	U
Titanium Tetrachloride Sat'd	N	U
Toluene	M	U
Transformer Oil	S	M
Trisodium Phosphate Sat'd	S	S
Trichloroethylene	U	U
Urea Up to 30%	S	S
Urine	S	S
Vinegar Comm.	S	S
Vanilla Extract	S	S
Wetting Agents	S	S
Whiskey	S	N
Wines	S	S
Xylene	M	U
Yeast	S	S
Zinc Chloride Sat'd	S	S
Zinc Sulfate Sat'd	S	S

For additional chemical resistance listings, consult the P.P.I. technical report #TR 19/10-84, Table I and the ISO technical report #ISO/Data 8-1979, Tables I, II, III.

Temperature Characteristics

nce polyethylene is a thermoplastic material, many of its physical and chemical properties are dependent on temperature and will change as the temperature of the material is increased or decreased. However, the exposure of Driscopipe to temperature variations within the recommended operating range does not result in degradation of the material. As these temperature changes are reversed, the material properties also reverse to their original values.

You will note from the information on physical properties that Driscopipe has a brittleness temperature below −180°F and a softening temperature of +257°F. The recommended operating temperature is limited only on the higher temperature side to a range of 140-180°F, dependent upon the pressure of the application and other operating and installation considerations. On the lower temperature side, Driscopipe gains strength without becoming brittle and is ideal for use at sub-zero temperatures.

Driscopipe becomes molten at 400-500°F and temperatures in this range are used to fusion join the piping system. Pipe is extruded at about the same temperature. To protect the material against degradation at the higher temperature, it is chemically stabilized. This stabilizer protects the material against thermal degradation which might otherwise occur during manufacture, outside storage and installation.

Driscopipe has been tested for thousands of hours at elevated temperatures of 140°F and 180°F without thermal degradation. These long term pressure tests at the higher temperatures are used to obtain recommended design strengths for the pipe at these temperatures.

Since all thermoplastic piping materials are affected by temperature, it is a general practice to characterize these materials at ambient temperature of 23°C (73.4°F). Nearly all ASTM tests relating to physical, mechanical and chemical properties of thermoplastic materials are conducted at this temperature. If a test is conducted, or a property defined, at other than 73.4°F, it is always noted.

One example of the effect of temperature on Driscopipe is the change in long term strength of the material as shown on the stress-life curves. This type behavior is true for all thermoplastics but there are large differences between the performance of specific materials at the higher temperatures.

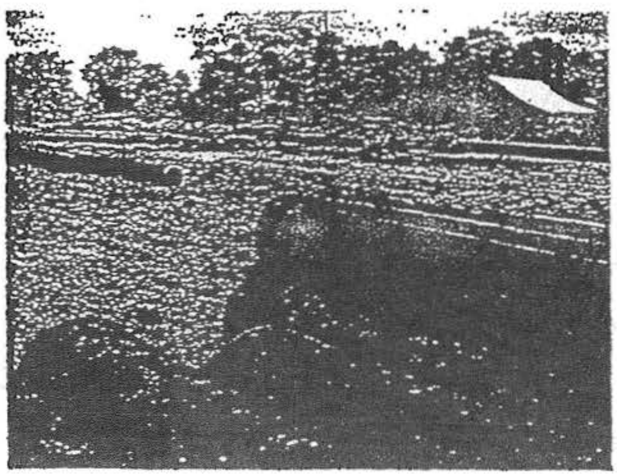
Knowledge of the long term strength of Driscopipe at the various temperatures allows selective design of a system. Accurate interpolations can be made for other temperatures between those which are known when data at three or more temperature levels is available.

Other properties of thermoplastic pipe which change with temperature and can affect system design and installation procedures include the following.

Burst strength – Short term (1 minute) burst tests on Driscopipe at various temperatures show these typical hoop stress values:

Temperature, °F	Hoop Stress, psi
73.4°	3250
32°	4300
0°	5290
−20°	5670
−40°	6385

Driscopipe will quick-burst at a pressure approximately four times greater than the rated operating pressure.



Chemical Resistance – The ability of most thermoplastics to resist degradation in the presence of corrosive chemicals is reduced as temperature increases. This is also true for Driscopipe but to a lesser extent because of its high density and high molecular weight. The effect of temperature on Driscopipe in the presence of various chemicals is shown in the chemical resistance tables.

Flexibility – As temperature is decreased, the flexibility of Driscopipe is also decreased. This has very little effect on installation except that at the lower winter temperatures, coiled pipe becomes more difficult, mechanically, to uncoil and stretch out in the ditch. Although Driscopipe becomes stiffer at low temperature, it can be bent, uncoiled or plowed in with sufficient mechanical power and no damage will occur to the pipe because of bending it at cold temperatures.

Other Physical Properties – There is a slight change with temperature of impact strength, notch sensitivity, flexural modulus, hardness and elongation ... but none are of such extent as to affect design parameters or installation procedures over the normal range of temperatures.

Modulus of Elasticity – Typical values for the variance in modulus of elasticity with temperature change is shown below.

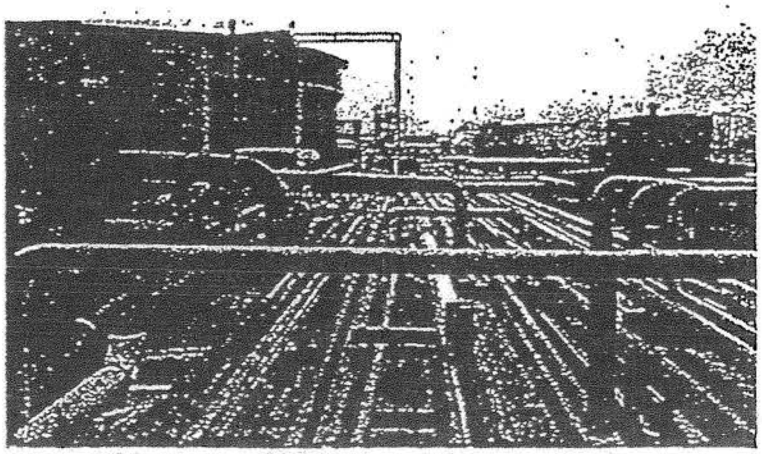
Temperature °F	Modulus of Elasticity, psi
−20°	300,000
0°	260,000
32°	200,000
75°	130,000
100°	105,000
140°	60,000

Thermal Expansion and Contraction – Polyethylene, like other thermoplastics, has a coefficient of expansion higher than metals. This coefficient is usually determined by a standard test method which employs the use of molded specimens. Measurements are made with a quartz dilatometer while the test specimen is held at elevated temperature. Typical coefficient values by this method range from $.75 \times 10^{-4}$ for Driscopipe 8600 to $.83 \times 10^{-4}$ for Driscopipe 1000.

The coefficient of linear expansion may also be determined by measuring the change in length of unrestrained pipe samples at different temperatures. The calculated coefficient is somewhat higher on extruded pipe than on molded test specimens. This appears to be true for all polyethylene pipe. The average coefficient calculated from measurements made on Driscopipe in the temperature range 0°F to 140°F, is 1.2×10^{-4} in/in/°F.

The circumferential coefficient of expansion and contraction for Driscopipe is approximately $.6 \times 10^{-4}$ in/in/°F in the range of 0° to 140°F ... or about ½ the linear coefficient. This circumferential change with temperature rarely presents any problems in system design. There may be need to consider this factor if compression fittings are used.

The expansion or contraction for Driscopipe can be stated in an easy rule of thumb ... the pipe will expand or contract approximately 1.4" per 100 feet for each 10°F change in temperature. Thus a 1000 foot unrestrained line which undergoes a 20°F increase in temperature change will increase in length 28 inches. The relatively large amount of expansion and contraction of plastic pipe generally presents no real problems in installation. The pipe has a relatively low elastic modulus and consequently there is less stress build-up. These stresses, caused by temperature change, are easily dissipated due to the thermoplastic nature of the material which relaxes and adjusts with time.



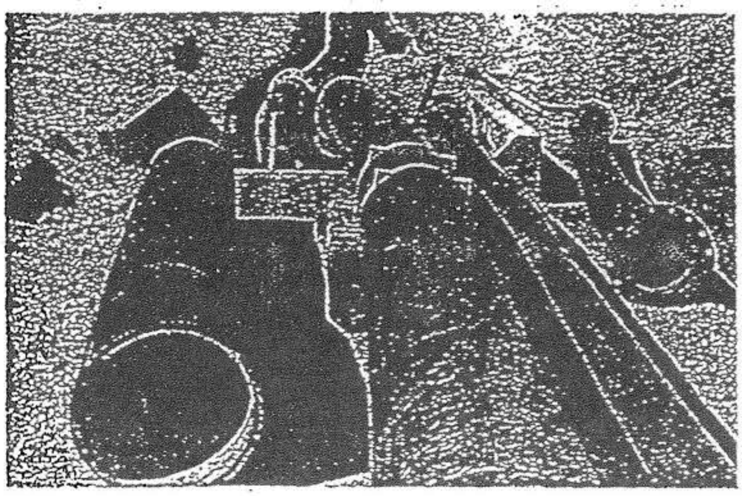
Tests have been conducted wherein the temperature of more than 100 feet of unrestrained pipe was changed 130°F in a period of a few minutes. The total force created by contraction was measured and proved to be about (½) one-half the theoretical calculated value. Thermoplastic materials are unique in their ability to stress-relieve themselves. Actual changes in temperature in most applications take place slowly over an extended period of time. The total stresses imposed will vary but are generally much lower than the calculated values.

Direct buried pipe will generally have ample soil friction and interference to restrain movement of the pipe under normal application temperature changes. It is a good idea to make the final tie-ins on a system at a temperature which is as close to operating temperature as possible. This is particularly true for insert liner systems where there is no soil restraint.

Normal good direct burial installation practices which include snaking the pipe in the ditch, proper backfill and compaction, making the tie-in at the proper temperature, etc. should be used at all times and will substantially reduce the possibility of pull out at tie-in connections on such installations. However, planning the transition tie-in becomes more important when Driscopipe is used for insert renewal inside another pipe because there is no restraint from earth loading. The contraction of the pipe due to reduction in temperature is freely transmitted to the transition connection and may result in pull-out if proper design precautions are not taken. In those cases, it may be necessary to provide additional anchoring at the terminations of the insert liner. Concrete anchors poured into undisturbed soil and cast around anchor projections in the Driscopipe line will restrict movement at the end of the line. Anchor projections on the Driscopipe liner can be made by fusing a blind tee into the line or by the use of two reducers, to the next larger size of pipe, fused together in the line.

Thermal Conductivity – This property of Driscopipe is lower than that for metals and can sometimes be exploited in the design of the system. It may eliminate or reduce the need for insulating pipe which carries water or other fluids through freezing temperatures. Thermal Conductivity of Driscopipe is 2.7 BTU per hour per sq. ft. per °F per inch of thickness. The slow heat transfer inhibits freezing and, if normal burial precautions are used, accidental freezing is usually eliminated. If the pipe does freeze, it does not fracture but fluid flow will be stopped. It will resume its function upon thawing. Direct application of intense heat should not be used to thaw a line. Antifreeze compounds such as methanol, isoproponol and ethylene glycol can be used without detrimental effect on the pipe.

Ignition Temperatures – The flash point for high density polyethylene using the Cleveland open cup method (ASTM D92) is 430°F. The flash ignition and self ignition temperatures using ASTM D1929 are 645°F and 660°F.



Weatherability

Two principal factors influence the weathering of plastic pipe in outside above ground applications ... temperature changes caused by seasonal variations and solar heating and solar radiation of ultraviolet rays. Effects of temperature variations on Driscopipe were discussed in the preceding section. Expansion and contraction of a line above ground, due to differential heating, will cause the line to move laterally, particularly if it is empty. This movement can easily be controlled within desired limits through the use of restraints.

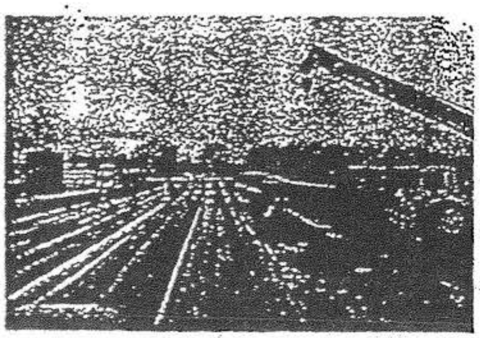
Driscopipe is also protected against degradation caused by ultraviolet rays when exposed to direct sunlight. The material contains 2½% of finely divided carbon black which also accounts for the black color of Driscopipe. Carbon black is the most effective single additive capable of enhancing the weathering characteristic of plastic materials. The protection even relatively low levels of carbon black impart to the plastic is so great that it is not necessary to use other light stabilizers or UV absorbers.

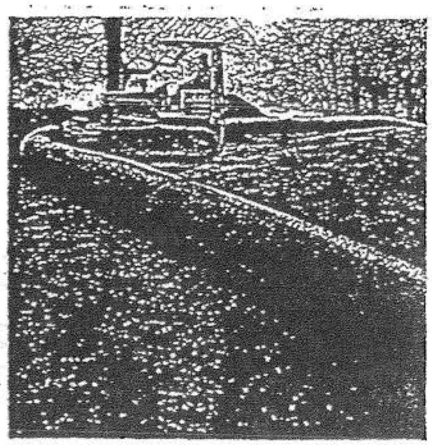
Weatherability tests indicate that Driscopipe can be safely used outside in most climates for periods of many years without danger of loss of physical properties due to UV exposure. Phillips has done extensive testing of polyethylene compounds containing 2 to 3% carbon black and compared these to other UV stabilizers to determine their effectiveness for protection against UV degradation in outdoor exposure. Samples were aged in outdoor exposure in three geographical locations: Phoenix, Arizona, Bartlesville, Oklahoma (Phillips 66 headquarters) and Akron, Ohio. From these actual tests, it was determined that one year exposure in Arizona was equivalent to at least two years in Bartlesville and greater than three and one-half years in Akron.

Weather-Ometer tests were run under standard conditions as set out in ASTM D 1499-64 and compared with the actual test samples in the three locations described above. From this test work, it was determined, conservatively, that 5000 hours (approximately 7 months) in the Weather-Ometer compares to greater than 42 months exposure in Arizona. Samples containing 2 to 3% carbon black and thermal stabilizers as used in Driscopipe have been tested for greater than 25,000 hours (2.85 years) in the Weather-Ometer without any brittleness or loss of physical properties. This is equivalent to over 17 years in Arizona and over 60 years in Akron, Ohio.

Permeability

The permeability of gases, vapors or liquids through a plastic membrane is generally considered to be an activated diffusion process. That is, the gas, vapor or liquid dissolves in the membrane and then diffuses to a position of lower concentration. The permeation rate is determined by the functional groups of the permeating molecules and by the density of the plastic ... the higher the density, the lower the permeability. Listed below are typical permeability rates for HDPE.





	Permeability Rate*
Carbon Dioxide	345
Hydrogen	321
Oxygen	111
Helium	247
Ethane	236
Natural Gas	113
Freon 12	95
Nitrogen	53

*Cubic centimeters per day per 100 sq. inches per mil thickness at atmospheric pressure differential.

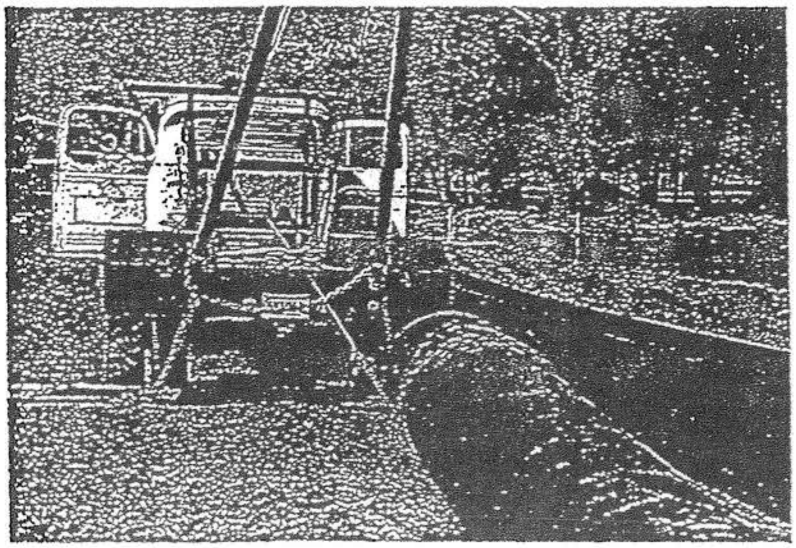
These permeation rates are considered very low. They result in negligible loss of product and create no hazard. For example, polyethylene piping systems are the predominant material used to construct new gas distribution systems and to renew old deteriorated systems. The permeation rate will vary in direct proportion to the differential pressure applied.

If the internal operating pressure is 60 psi, for example, the permeability rate would be approximately 4 times that shown above but volume losses would still be extremely low. Calculated volume loss in one mile of SDR 11 pipe (any size) in one day, for natural gas, would be ¼ of one cubic foot. At 120 psi, it would be ½ cubic foot per day.

Abrasion Resistance

One of the many outstanding characteristics of Driscopipe polyethylene is its resistance to abrasion. The inherent resilience and toughness of Driscopipe allows the mining industry to use this pipe in numerous surface applications where more conventional materials would be unsatisfactory, either because of the terrain encountered or the abrasiveness of the slurry to be moved. Quite often, a Driscopipe system offers substantial economic advantage as a means of transport over more conventional transportation methods used in the mining industry. Some of the more common applications include tailings lines and the transport of gypsum, limestone, sand, slimes and coal.

Due to its unique toughness, as indicated by low melt flow values, Driscopipe 8600 provides improved abrasion resistance over all other polyethylene piping materials. Controlled pipe loop pumping tests have demonstrated that Driscopipe can outlast steel pipe by as much as 4 to 1. One such test, performed by Williams Brothers Engineering, Tulsa, Oklahoma, compared Driscopipe to steel in pumping a coarse particle size magnetite iron ore slurry. At 13½ ft/sec velocity, Driscopipe was better by a factor of 4:1 and at 17 ft/sec by a factor of 3:1.



Heat Fusion Joining

The heat fusion joining technique has a long history of use for joining polyethylene pipe materials. The heat fusion method of joining PE pipe began shortly after the first commercial production of high density polyethylene in the early 1950s ... both developed by Phillips 66.

The integrity and superiority of heat fusion are now recognized universally. The modern day heat fusion joint is the same joint made in 1956 ... only the fusion equipment has evolved to gain efficiency, reliability and convenience. The principles learned on early equipment for making a successful joint are still in use today. Phillips designed, developed and built many models of heat fusion equipment from 1956 until the early 1970s. Since that time, Phillips has guided this development by others. The extensive line of high quality, efficient fusion equipment offered by McElroy Manufacturing, Inc., Tulsa, Oklahoma is one of the results of this long history of development. Phillips pioneered the idea and development of heat fusion and has used it exclusively in every high density polyethylene piping system sold by Phillips since 1956. There are millions of these joints in service today. In fact, 92% of all natural gas distribution pipe to homes, farms and factories is installed with polyethylene pipe and fittings. Heat fusion joints are industry accepted and field proven.

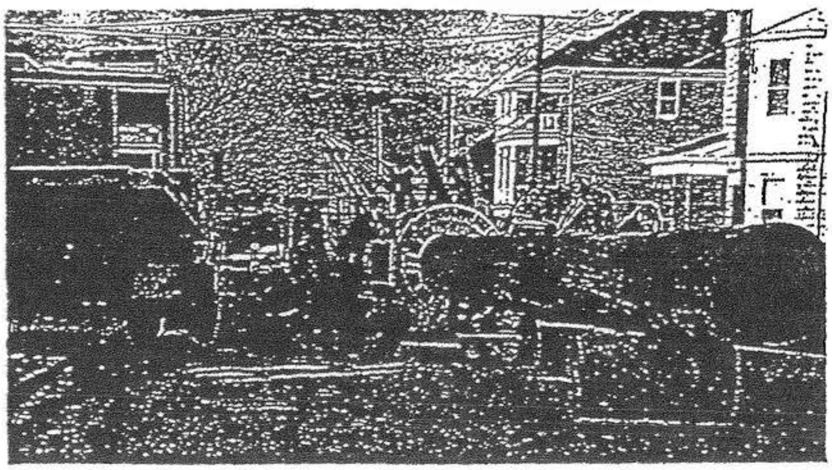
The heat fusion joining system has been so successful that it is the "standard" joining system for polyethylene. There are many reasons ... here are some.

Heat fusion joining ideally meets the requirements for a fast joining method to facilitate all phases of construction work in a safe and reliable manner.

The heat fusion joint is structurally superior to the socket fusion joint by configuration and, therefore, better meets the requirements of service. The heat joint configuration allows it to better disperse stresses initiated by pipe deflection and external loading. Stress concentration is minimized when the joint is placed in a strain and the joint is more "forgiving" when ground settlement occurs. In a socket joint, there is an extremely high ratio of "joint wall" to "pipe wall", resulting in stress intensification from external loading.

The Driscopipe heat fusion joining system is a simple, visual procedure with straight forward instructions. No "timing cycles" are necessary. The visual procedure allows the operator to concentrate on his work rather than a clock. Visually, he knows when the pipe ends have melted to the degree required to fuse them together. Visually, he observes and controls fusion pressure by observing the amount and configuration of the fusion bead as it is formed.

In the course of this work, the fusion operator is faced with a wide variety of job conditions. Changes in air temperature, material temperature, wind velocity, sun exposure, humidity, as well as condition of the terrain and the equipment all influence the joining requirements. Quality work under field conditions is more consistent with a simple, straight-forward, visual procedure.

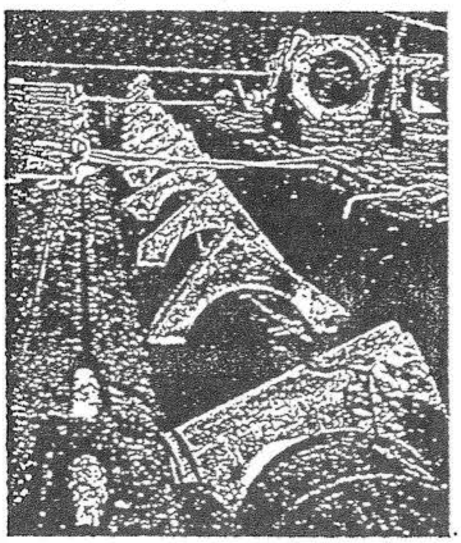


On^ heat fusion operator, with equipment, typically ^e whole operation himself, sometimes using a ... ound person as a helper. Pipe tolerances, ovality and curvature are no problem and "melt" is easily controlled by the visual procedure.

Heat fusion joints offer a large advantage over socket coupled joints for plow-in installation and for insert renewal applications. Socket coupled pipe requires larger size plow chutes and bore holes. Heat fused pipe one size larger can usually be handled and installed through bore holes and plow chutes selected for socket coupled pipe. Larger sizes of heat fused pipe can be used inside old mains for insert renewal because it does not require the extra space for the coupling.

Heat fusion joints may easily be cut out and re-done. This fact has a bearing on the quantity and quality of training necessary and favorably affects operator attitude toward quality in the field. These joints can be easily cut out and destructively tested in the field to check joining proficiency and equipment condition and it's inexpensive. There is no coupling to destroy and throw away.

The heat fusion joining system is especially effective with Driscopipe 8600. The melt of this material is very viscous and tough. The operator can apply ample pressure to form the heat fusion joint with little danger ...rcing the molten material from between the two ...ds of the joint, as can be done with the softer, less viscous, high density materials.



Driscopipe 8600 can be fusion joined to other polyethylene piping materials when necessary. Special joining techniques are required to achieve good joints. Phillips Driscopipe technical personnel are available to instruct and demonstrate the fusion joining procedure for joining Driscopipe to other polyethylene materials.

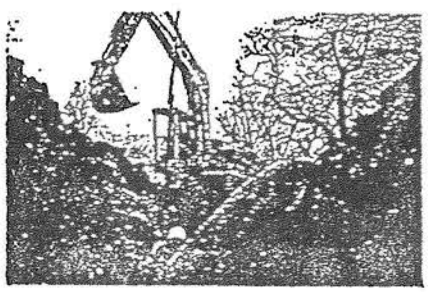
Fatigue Resistance

Driscopipe 8600 very high molecular weight, high density polyethylene has superior resistance to fatigue failure caused by cyclic loading. Independent laboratory tests were conducted to determine the suitability of Driscopipe 8600 for use as the cold water supply pipe and the barge mooring leg of the Mini-OTEC Project (Hawaii, 1979). In that application, 2150' of 24" 60 psi Driscopipe 8600 was deployed vertically in a deep ocean trench just offshore Keahole Point and was subject to cyclic distortion caused by wave action, current, and barge motion.

Cyclic tests showed that Driscopipe 8600 very high molecular weight PE could endure more than 100,000 cycles at a stress of 1800 psi without failure. Copies of this test report are avialable upon request.

Driscopipe 1000 offers good fatigue service life also, but not equal to 8600. Neither requires de-rating like PVC AWWA C-900 pipe. In fact, per AWWA C-906 for 4" to 63" HDPE pipe, no water hammer or fatigue de-rating factor need be applied to Driscopipe 8600 or Driscopipe 1000 ductile PE pipe.

The Driscopipe performance team offers you innovative solutions to your piping requirements. Contact your nearest Driscopipe Sales Representative. He'll give you personalized technical service, installation assistance and all the cost-saving advantages of a Driscopipe Piping System. Engineered for Performance!



DRISCOPIPE

Phillips Driscopipe, Inc.
A Subsidiary of Phillips 66 Company

To Secure Product Information or Leave a Message for a Sales Engineer or Technical Service Representative:

Mail:
Attn: Customer Service Department
P.O. Box 83-3866
2929 North Central Expressway
Suite 100
Richardson, Texas 75083

Phone:
U.S. Domestic Toll Free (800) 527-0662

TWX:
910-867-4818

Fax:
214-783-2689

PLASTICS WITH POWER TO WIN.®

1774-90 A 01

butane may condense and liquefy in the pipe. Such liquefied fuel gasses are known to permeate polyethylene pipe, and result in unreliable heat fusion or electrofusion joints.

In potable water applications, permeating chemicals could affect the pipe or water in the pipe. ANSI/AWWA standards provide the following guidance for potable water applications:

> "The selection of materials is critical for water service and distribution piping where there is likelihood the pipe will be exposed to significant concentrations of pollutants comprised of low molecular weight petroleum products or organic solvents or their vapors. Research has documented that pipe materials such as polyethylene, polybutylene, polyvinyl chloride, and asbestos cement, and elastomers, such as used in jointing gaskets and packing glands, may be subject to permeation by lower molecular weight organic solvents or petroleum products. If water pipe must pass through such a contaminated area or an area subject to contamination, consult with the manufacturer regarding permeation of pipe walls, jointing materials, and so forth, *before* selecting materials for use in that area."[1]

Chemical Attack

A direct chemical attack on the polymer will result in permanent, irreversible polymer damage or chemical change by chain scission, cross-linking, oxidation, or substitution reactions. Such damage or change cannot be reversed by removing the chemical.

Chemical Resistance Information

The following chemical resistance guide, Table 5-1 (next page), presents immersion test chemical resistance data for a wide variety of chemicals.

- ❑ This data may be applicable to gravity flow and low stress applications.
- ❑ It may not be applicable when there is applied stress such as internal pressure, or applied stress at elevated temperature.

Unless stated otherwise, polyethylene was tested in the relatively pure, or concentrated chemical.

It is generally expected that dilute chemical solutions, lower temperatures, and the absence of stress have less potential to affect the material. At higher temperature, or where there is applied stress, resistance may be reduced, or polyethylene may be unsuitable for the application. Further, combinations of chemicals may have effects where individual chemicals may not.

Testing is recommended where information about suitability for use with chemicals or chemical combinations in a particular environment is not available. PLEXCO cannot provide chemical testing services.

1 *ANSI/AWWA C906-90, Section 1.2; ANSI/AWWA C901-96, Section 4.1.*

Table 5-1 Chemical Resistance

Because the particular conditions of each application may vary, Table 5-1 information should be used only as a preliminary guide for PLEXCO and SPIROLITE polyethylene pipe materials. This information is offered in good faith, and is believed to be accurate at the time of publication, but it is offered without any warranty, expressed or implied. Additional information may be required, particularly in regard to unusual or special applications. Determinations of suitability for use in particular chemical or environmental conditions may require specialized laboratory testing.

Additional information on chemical compatibility may be found in PPI TR-19, *Thermoplastic Piping for the Transport of Chemicals.*

Chemical Resistance Key

Key†	*Meaning*
X	resistant (swelling <3% or weight loss <0.5%; elongation at break not substantially changed)
/	limited resistance (swelling 3 - 8% or weight loss 0.5 - 5%; elongation at break reduced by <50%)
—	not resistant (swelling > 8% or weight loss >5%; elongation at break reduced by >50%)
D	discoloration
*	aqueous solutions in all concentrations
**	only under low mechanical stress
† Where a key is not printed in the table, data is not available.	

Medium	*73°F*	*140°F*
Acetaldehyde, gaseous	X	/
Acetic acid (10%)	X	X
Acetic acid (100%) (Glacial acetic acid)	X	/D
Acetic anhydride	X	/D
Acetone	X	X
Acetylene tetrabromide	**/ to —	—
Acids, aromatic	X	X
Acrylonitrile	X	X
Adipic acid	X	X
Allyl alcohol	X	X
Aluminum chloride, anhydrous	X	X
Aluminum sulphate	*X	X
Alums	X	X
Ammonia, liquid (100%)	X	X
Ammonium chloride	*X	X
Ammonium flouride, aqueous (up to 20%)	X	X
Ammonium nitrate	*X	X
Ammonium sulphate	*X	X
Ammonium sulfide	*X	X
Amyl acetate	X	X
Aniline, pure	X	X
Anisole	/	—
Antimony trichloride	X	X
Aqua regia	—	—
Barium chloride	*X	X
Barium hydroxide	*X	X

Medium	73°F	140°F
Beeswax	X	**/ to —
Benzene	/	/
Benezenesulphonic acid	X	X
Benzoic acid	*X	X
Benzyl alcohol	X	X to /
Borax, all concentrations	X	X
Boric acid	*X	X
Brine, saturated	X	X
Bromine	—	—
Bromine vapor	—	
Butanetriol	X	X
Butanol	X	X
Butoxyl	*X	/
Butyl acetate	X	/
Butyl glycol	X	X
Butyric acid	X	/
Calcium chloride	*X	X
Calcium hypochlorite	*X	X
Camphor	X	/
Carbon dioxide	X	X
Carbon disulphide	/	
Carbon tetrachloride	**/ to —	—
Caustic potash	X	X
Caustic soda	X	X
Chlorine, liquid	—	—
Chlorine bleaching solution (12% active chlorine)	/	—
Chlorine gas, dry	/	—
Chlorine gas, moist	/	—
Clorine water (disinfection of mains)	X	
Chloroacetic acid (mono)	X	X
Chlorobenzene	/	—
Chloroethanol	X	XD
Chloroform	**/ to —	—
Chlorosulphonic acid	—	—
Chromic acid (80%)	X	—D
Citric acid	X	X
Coconut oil	X	/
Copper salts	*X	X
Corn oil	X	/
Creosote	X	XD
Creosol	X	XD
Cyclohexane	X	X
Cyclohexanol	X	X

Medium	73°F	140°F
Cyclohexanone	X	X
Decahydronaphthalene	X	/
Desiccator grease	X	/
Detergents, synthetic	X	X
Dextrin, aqueous (18% saturated)	X	X
Dibutyl ether	X to /	—
Dibutyl phthalate	X	/
Dichloroacetic acid (100%)	X	/D
Dichloroacetic acid (50%)	X	X
Dicliloroacetic acid methyl ester	X	X
Dichlorbenzene	/	—
Diclolorethane	/	/
Dicloroethylene	—	—
Diesel oil	X	/
Diethyl ether	X to /	/
Diisobutyl ketone	X	/ to —
Dimethyl formamide (100%)	X	X to /
Dioxane	X	X
Emulsifiers	X	X
Esters, aliphatic	X	X to /
Ether	X to /	/
Ethyl acetate	/	—
Ethyl alcohol	X	X
Ethyl glycol	X	X
Ethyl hexanol	X	X
Ethylene chloride (dichlorothene)	/	/
Ethylene diamine	X	X
Fatty acids ($>C^6$)	X	/
Feric chloride*	X	X
Fluorine	—	—
Fluorocarbons	/	—
Fluorosilic acid, aqueous (up to 32%)	X	X
Formaldehyde (40%)	X	X
Formamide	X	X
Formic acid	X	
Fruit juices	X	X
Fruit pulp	X	X
Furfuryl alcohol	X	XD
Gelatine	X	X
Glucose	*X	X
Glycerol	X	X
Glycerol chlorohydrin	X	X
Glycol (conc.)	X	X

Medium	73°F	140°F	Medium	73°F	140°F
Glycolic acid (50%)	X	X	Nitric acid (25%)	X	X
Glycolic acid (70%)	X	X	Nitric acid (50%)	/	—
Halothane	/	/	Nitrobenzene	X	/
Hydrazine hydrate	X	X	o-Nitrotoluene	X	/
Hydrobromic acid (50%)	X	X	Octyl cresol	/	—
Hydrochloric acid (all concentrations)	X	X	Oils, ethereal	/	/
Hydrocyanic acid	X	X	Oils, vegetable & animal	X	X to /
Hydrofluoric acid (40%)	X	/	Oleic acid (conc.)	X	/
Hydrofluoric acid (70%)	X	/	Oxalic acid (50%)	X	X
Hydrogen	X	X	Ozone	/	—
Hydrogen chloride gas, moist and dry	X	X	Ozone, aqueous solution (Drinking water purification)	X	
Hydrogen peroxide (30%)	X	X	Paraffin oil	X	X
Hydrogen peroxide (100%)	X		Perchloric acid (20%)	X	X
Hydrogen sulfide	X	X	Perchloric acid (50%)	X	/
Iodine, tincture of, DAB 7 (German Pharmacopoeia)	X	/D	Perchloric acid (70%)	X	—D
Isooctane	X	/	Petrol	X	X to /
Isopropanol	X	X	Petroleum	X	/
Isopropyl ether	X to /	—	Petroleum ether	X	/
Jam	X	X	Petroleum jelly	**X to /	/
Keotones	X	X to /	Phenol	X	XD
Lactic acid	X	X	Phosphates	*X	X
Lead acetate	*X	X	Phosphoric acid (25%)	X	X
Linseed oil	X	X	Phosphoric acid (50%)	X	X
Magnesium chloride	*X	X	Phosphoric acid (95%)	X	/D
Magnesium sulphate	*X	X	Phosphorus oxychloride	X	/D
Maleic acid	X	X	Phosphorus pentoxide	X	X
Malic acid	X	X			
Menthol	X	/	Phosphorus trichloride	X	/
Mercuric chloride (sublimate)	X	X	Photographic developers, commecial	X	X
Mercury	X	X	Phthalic acid (50%)	X	X
Methanol	X	X	Polyglycols	X	X
Methyl butanol	X	X	Potassium bichromate (40%)	X	X
Methyl ethyl ketone	X	/ to —	Potassium borate, aqueous (1%)	X	X
Methyl glycol	X	X	Potassium bromate, aqueous (up to 10%)	X	X
Methylene chloride	/	/	Potassium bromide	*X	X
Mineral oils	X	X to /	Potassium chloride	*X	X
Molasses	X	X	Potassium chromate, aqueous (40%)	X	
Monochloroacetic acid	X	X	Potassium cyanide	*X	X
Monochloroacetic ethyl ester	X	X	Potassium hydroxide (30% solution)	X	X
Monochloroacetic methyl ester	X	X	Potassium nitrate	*X	X
Morpholine	X	X	Potassium permanganate	X	XD
Naptha	X	/	Propanol	X	X
Naphthalene	X	/	Propionic acid (50%)	X	X
Nickel salts	*X	X	Propionic acid (100%)	X	/

Second Edition
©1998 Chevron Chemical Company LLC
Issued 12/98

VOLUME 1

Medium	73°F	140°F
Propylene glycol	X	X
Pseudocumene	/	/
Pyridine	X	/
Seawater	X	X
Silicic acid	X	X
Silicone oil	X	X
Silver nitrate	X	X
Soduim benzoate	X	X
Sodium bisulphite, weak aqueous solutions	X	X
Sodium carbonate	*X	X
Sodium chloride	*X	X
Sodium chlorite (50%)	X	/
Sodium hydroxide (30% solution)	X	X
Sodium hypochlorite (12% active chlorine)	/	—
Sodium nitrate	*X	X
Sodium silicate	*X	X
Sodium sulfide	*X	X
Sodium thiosulphate	X	X
Spermaceti	X	/
Spindle oil	X to /	/
Starch	X	X
Steric acid	X	/
Succinc acid (50%)	X	X
Sugar syrup	X	X
Sulfates	*X	X
Sulfur	X	X
Sulfur dioxide, dry	X	X
Sulfur dioxide, moist	X	X
Sulfur trioxide	—	—
Sulfuric acid (10%)	X	X
Sulfuric acid (50%)	X	X
Sulfuric acid (98%)	/	—
Sulfuric acid, fuming	—	—
Sulfurous acid	X	X
Sulfuryl chloride	—	
Tallow	X	X
Tannic acid (10%)	X	X
Tartaric acid	X	X
Tetrachloroethane	**X to /	—
Tetrahydrofurane	**X to /	
Tetetrahydronapthalene	X	/
Thionyl chloride	—	—
Thiophene	/	/
Toluene	/	—
Transformer oil	X	/
Tributyl phosphate	X	X
Trichloroacetic acid (50%)	X	X
Trichloroacetic acid (100%)	X	/ to —
Trichloroethylene	**X to /	—
Triethanolamine	X	X
Turpentine, oil of	x to /	/
Tween 20 and 90 (Atlas Chemicals)	X	X
Urea	*X	X
Vinegar (commecial conc.)	X	X
Viscose spinning solutions	X	X
Waste gases containing		
carbon dioxide	X	X
carbon monoxide	X	X
hydrocloric acid (all conc.)	X	X
hydrogen fluoride (traces)	X	X
nitrous vitriol (traces)	X	X
sulfur dioxide (low conc.)	X	X
sulphuric acid, moist (all conc.)	X	X
Water gas	X	X
Xylene	—	—
Yeast, aqueous preparations	X	X
Zinc chloride	*X	X

Second Edition

Issued 12/98

ATTACHMENT III.6.F

PVC PIPE REFERENCE DOCUMENTATION

CertainTeed

PVC Chemical Resistance

KEY — E = Excellent G = Good L = Limited U = Unsuitable O = No test

Chemical	PVC I 72°F.	PVC I 140°F.	PVC II 72°F.	PVC II 140°F.
Acetaldehyde	U	U	U	U
Acetamide	O	O	U	U
Acetate Solvents - Crude	U	U	U	U
Acetate Solvents - Pure	U	U	U	U
Acetic Acid 0-10%	E	E	G	L
Acetic Acid 10-20%	E	E	G	L
Acetic Acid 20-30%	E	G	G	L
Acetic Acid 30-60%	E	E	G	L
Acetic Acid 80%	G	L	L	L
Acetic Acid - Glacial	G	U	L	U
Acetic Acid - Vapors	E	E	G	G
Acetic Anhydride	U	U	U	U
Acetone	U	U	U	U
Acetylene	L	L	E	E
Adipic Acid	E	E	E	E
Alcohol - Allyl - 96%	G	L	U	U
Alcohol - Amyl	E	L	L	U
Alcohol - Buty	E	G	L	U
Alcohol - Ethyl	E	E	E	G
Alcohol - Methyl	E	E	E	E
Alcohol - Propargyl	E	E	E	E
Alcohol - Propyl	E	E	E	G
Allyl - Chloride	U	U	U	U
Alum	E	E	E	E
Alum, Ammonium	E	E	E	E
Alum, Chrome	E	E	E	E
Alum, Potassium	E	E	E	E
Aluminum Chloride	E	E	E	E
Aluminum Fluoride	E	E	E	E
Aluminum Hydroxide	E	E	E	E
Aluminum Oxychloride	E	E	E	E
Aluminum Nitrate	E	E	E	E
Aluminum Sulfate	E	E	E	E
Ammonia - Dry Gas	E	E	E	E
Ammonia, Aqua (10%)	E	E	E	E
Ammonia - Liquid	L	U	O	O
Ammonium Acetate	E	E	E	E
Ammonium BiFluoride	E	E	E	E
Ammonium Carbonate	E	E	E	E
Ammonium Chloride	E	E	E	E
Ammonium Fluoride - 25%	E	L	U	U
Ammonium Hydroxide - 28%	E	E	E	E
Ammonium Metaphosphate	E	E	E	E
Ammonium Monophosphate	E	E	E	E
Ammonium Nitrate	E	E	E	E
Ammonium Persulfate	E	E	E	E
Ammonium Phosphatel (Ammoniacal)	E	E	O	O
Ammonium Phosphate - Neutral	E	E	E	E
Ammonium Sulfate	E	E	E	E
Ammonium Sulfide	E	E	E	E
Ammonium Thiocyanate	E	E	E	E
Amyl Acetate	U	U	U	U
Amyl Chloride	U	U	U	U
Aniline	U	U	U	U
Aniline Chlorohydrate	U	U	U	U
Aniline Dyes	U	U	U	U
Aniline Hydrochloride	U	U	U	U
Anthraquinone	E	E	E	L
Anthraquinonesulfonic Acid	E	E	E	E
Anitimony Trichloride	E	E	E	E
Aqua Regia	E	L	U	U
Arsenic Acid - 80%	E	G	E	G
Arylsulfonic Acid	E	E	L	U
Asphalt	E	E	E	E
Barium Carbonate	E	E	E	E
Barium Chloride	E	E	E	E
Barium Hydroxide	E	E	E	E
Barium Sulfate	E	E	E	E
Barium Sulfide	E	E	E	E
Beer	E	E	E	E

Chemical	PVC I 72°F.	PVC I 140°F.	PVC II 72°F.	PVC II 140°F.
Beet - Sugar Liquor	E	E	E	E
Benzaldehyde	U	U	U	U
Benzene	U	U	U	U
Benzenesulfonic Acid - 10%	E	E	E	E
Benzenesulfonic Acid	U	U	U	U
Benzoic Acid	E	E	E	E
Benzol	U	U	U	U
Bismouth Carbonate	E	E	E	E
Black Liquor (Paper Industry)	E	E	E	E
Bleach - 12.5% Active CL_2	E	G	G	L
Borax	E	E	E	E
Borax Liquors	E	E	E	E
Boric Acid	E	E	E	E
Boron, TriFluoride	E	E	E	E
Breeder Pellets - Fish Deriv.	E	E	E	E
Brine	E	E	E	E
Bromic Acid	E	E	E	E
Bromine - Liquid	U	U	U	U
Bromine (Gas) - 25%	E	E	U	U
Bromine - Water	E	E	L	U
Butadiene	E	E	L	U
Butane	E	E	E	E
Butane, Buthylene	E	E	E	U
Butane, Diol	E	E	U	U
Butanol	E	U	U	U
Butanol - Primary	E	E	U	U
Butanol - Secondary	E	L	U	U
Buttermilk	E	E	E	E
Butyl Acetate	U	U	U	U
Butyl Phenol	E	U	L	U
Butylene	E	O	E	O
Butynediol (Erthritol)	E	U	U	U
Butyric Acid 20%	G	U	L	U
Butyric Acid	E	U	U	U
Calcium Bisulfide	E	E	E	E
Calcium Bisulfite	E	E	E	E
Calcium Carbonate	E	E	E	E
Calcium Chlorate	E	E	E	E
Calcium Chloride	E	E	E	E
Calcium Hydroxide	E	E	E	E
Calcium Hyposhlorite	E	E	E	E
Calcium Nitrate	E	E	E	E
Calcium Oxide	E	E	E	U
Calcium Sulfate	E	E	E	E
Cane Sugar Liquors	E	E	E	E
Carblic Acid	E	E	E	E
Carbon Bisulfide	U	U	U	U
Carbon Dioxide (Aqueous S.L.)	E	E	E	E
Carbon Dioxide Gas (Wet)	E	E	E	E
Carbon Monoxide	E	E	E	E
Carbon Tetrachloride	L	U	U	U
Carbonated Water	E	E	E	E
Carbonic Acid	E	E	E	E
Casein	E	E	E	E
Castor Oil	E	E	E	E
Caustic Potash	E	E	E	E
Caustic Soda	E	E	E	E
Cellosolve	G	L	L	U
Chloracetic Acid	E	L	E	U
Chloral Hydrate	E	E	E	E
Chloric Acid 20%	E	E	E	E
Chlorinated Solvents	U	U	U	U
Chlorine (Dry)	E	L	L	L
Chlorine Gas (Moist)	G	L	L	L
Chlorine Water	E	E	E	E
Chloroacetic Acid	E	E	E	U
Chlorobenzene	U	U	U	U
Chlorobenzyl Chloride	U	U	U	U
Chloro Form	U	U	U	U
Chlorosulfonic Acid (100%)	E	U	O	O
Chrome Alum	E	E	E	E

CertainTeed

Chemical	PVC I		PVC II	
	72 °F.	140 °F.	72 °F.	140 °F.
Chromic Acid 10%	E	E	E	E
Chromic Acid 25%	E	L	G	L
Chromic Acid 30%	E	L	G	U
Chromic Acid 40%	E	L	L	U
Chromic Acid 50%	E	L	L	U
Citric Acid	E	E	E	E
Coconut Oil	E	E	E	E
Coke Oven Gas	E	E	E	E
Copper Carbonate	E	E	E	E
Copper Chloride	E	E	E	E
Copper Cyanide	E	E	E	E
Copper Fluoride	E	E	E	E
Copper Nitrate	E	E	E	E
Copper Sulfate	E	E	E	E
Core Oils	E	E	E	E
Corn Oil	E	E	E	E
Corn Syrup	E	E	E	E
Cottonseed Oil	E	E	E	E
Cresol	U	U	U	U
Cresylic Acid 50%	E	E	L	U
Croton Aldehyde	U	U	U	U
Crude Oil - Sour	E	E	E	E
Crude Oil - Sweet	E	E	E	E
Cuprous Chloride	E	E	E	E
Cyclohexane	U	U	U	U
Cyclohexanol	U	U	U	U
Cyclohexanon	U	U	U	U
Demineralized Water	E	E	E	E
Dextrin	E	E	E	E
Dextrose	E	E	E	E
Diazo Salts	E	E	E	E
Diesel Fuels	E	E	E	U
Diethye Amine	U	U	U	U
Dioctylphthalate	U	U	U	U
Disodium Phosphate	E	E	E	E
Diethyl Ether	U	U	U	U
Diglycolic Acid	E	G	E	G
Dioxane - 1,4	O	O	O	O
Divinyl Benzene	O	O	O	O
Drying Oil	O	O	O	O
Ethers	U	U	U	U
Ethyl Acetate	U	U	U	U
Ethyl Acrylate	U	U	U	U
Ethyl Chloride	U	U	U	U
Ethyl Ether	U	U	U	U
Ethylene Bromide	U	U	U	U
Ethylene Chlorohydrin	U	U	U	U
Ethylene Dichloride	U	U	U	U
Ethylene Glycol	E	E	E	E
Ethylene Oxide	U	U	U	U
Fatty Acide	E	E	E	E
Ferric Chloride	E	E	E	E
Ferric Nitrate	E	E	E	E
Ferric Sulfate	E	E	E	E
Ferrous Nitrate	E	E	E	E
Fish Solubles	E	E	E	E
Fluorine Gas - Dry	L	U	U	U
Flourine Gas - Wet	L	U	U	U
Fluoroboric Acid - 25%	E	E	E	E
Fluorosilicic Acid	E	E	E	E
Formaldehyde	E	G	G	L
Food Products such.as Milk, Buttermilk, Molasses, Salad Oils, Fruit	E	E	E	E
Formic Acid	E	U	E	U
Freon - 12	E	G	E	G
Fructone	E	E	E	E
Fruit Pulps and Juices	E	E	E	E
Fuel Oil (containing H_2SO_4)	E	E	E	E
Furfural	U	U	U	U
Gallic Acid	E	E	E	E
Gas - Coke Oven	E	E	G	G
Gas - Manufactured	U	U	U	U
Gas - Natural (Dry)	E	E	E	E
Gas - Natural (Wet)	E	E	E	E
Gasoline (Leaded)	E	E	E	U
Gasoline (unleaded)	E	E	E	U
Gasoline - Refined	E		L	
Gasoline - Sour	E	E	E	E
Gelatine	E	E	E	E
Glucose	E	E	E	E
Glycerine (Glycerol)	E	E	E	E
Glycol	E	E	E	E
Glue	E	E	E	E
Glycolic Acid 30%	E	E	E	E
Green Liquor (Paper Industry)	E	E	E	E
Heptane	E	G	L	U
Hexane	E	L	U	U
Hexanol Tertiary	E	E	L	U
Hydrobromic Acid - 20%	E	E	E	G
Hydrochloric Acid - 0-25%	E	G	E	G
Hydrochloric Acid - 25-40%	E	E	E	G
Hydrocyanic Acid or Hydrogen Cyanide	E	E	E	E
Hydrofluoric Acid 4%	E	L	G	G
Hydrofluoric Acid 10%	E	L	E	G
Hydrofluoric Acid 48%	E	L	G	U
Hydrofluoric Acid 60%	E	L	G	U
Hydrofluoric Acid 100%	G	L	O	L
Hydrogen	E	E	E	G
Hydrogen Peroxide - 30%	E	E	E	G
Hydrogen Peroxide - 50%	E	E	E	L
Hydrogen Peroxide - 90%	E	E	U	U
Hydrogen Slurfide - Agueous Solution	E	E	E	E
Hydrogen Sulfide - Dry	E	E	E	E
Hydroquinone	E	E	E	E
Hydroxylamine Sulfate	E	E	E	E
Hypochlorous Acid	E	E	E	E
Hypo-(Sodium Thiosulfate)	E	E	E	E
Iodine	U	U	U	U
Iodine (in Alcohol)	U	U	U	U
Iodine Solution (10%)	U	U	U	U
Iodoform	O	O	O	O
Isopropylalcohol	E	E	E	G
Jet Fuels, JP4 & JP5	E	E	E	E
Kerosene	E	E	E	E
Ketones	U	U	U	U
Kraft Liquor (Paper Industry)	E	E	E	E
Lacquer Thinners	L	U	L	U
Lactic Acid 28%	E	E	E	E
Lard Oil	E	E	E	G
Lauric Acid	E	E	E	E
Lauryl Chloride	E	E	E	E
Lauryl Sulfate	E	E	E	E
Lead Acetate	E	E	E	E
Lime Sulfur	E	E	E	E
Linoleic Acid	E	E	E	E
Linseed Oil	E	E	E	E
Liquers	E	E	E	E
Liquors	E	E	E	E
Lithium Bromide	E	E	E	E
Lubricating Oil	E	E	E	E
Machine Oil	E	E	E	E
Magnesium Carbonate	E	E	E	E
Magnesium Chloride	E	E	E	E
Magnesium Citrate	E	E	E	E
Magnesium Hydroxide	E	E	E	E
Magnesium Nitrate	E	E	E	E
Magnesium Sulfate	E	E	E	E
Maleic Acid	E	E	E	E
Malic Acid	E	E	E	E
Mercuric Chloride	E	E	G	G
Mercuric Cyanide	E	E	G	G
Mercurous Nitrate	E	E	G	G
Mercury	E	E	G	G

CertainTeed

Chemical	PVC I 72°F.	PVC I 140°F.	PVC II 72°F.	PVC II 140°F.
Methane	E	E	E	E
Methyl Bromide	U	U	U	U
Methyl Cellosolve	U	U	U	U
Methyl Chloride	U	U	U	U
Methyl Chloroform	U	U	U	U
Methyl Ethyl Ketone	U	U	U	U
Methyl Iso-Butyl Ketone	U	U	U	U
Methyl Salicylate	E	E	E	E
Methyl Sulfate	E	L	E	L
Methyl Sulfonic Acid	E	E	E	E
Methyl Sulfuric Acid	E	E	E	E
Methylene Chloride	U	U	U	U
Milk	E	E	E	E
Mineral Oils	E	E	E	G
*Mixed Acids (H_2SO_4 & HNO_3)	E	E	E	L
Molasses	E	E	E	E
Monoethanolamine	U	U	U	U
Muriatic Acid	E	E	E	E
Naptha	E	E	E	U
Napthalene	U	U	U	U
Natural Gas, Dry & Wet	E	E	E	E
Nickel Acetate	E	E	E	E
Nickel Chloride	E	E	E	E
Nickel Nitrate	E	E	E	E
Nickel Sulfate	E	E	E	E
Nickel Sulphate	E	E	E	E
Nicotine	E	E	E	E
Nicotine Acid	E	E	E	G
Nitric Acid Anhydrous	U	U	U	U
Nitric Acid 10%	E	E	E	E
Nitric Acid 20%	E	L	G	L
Nitric Acid 35%	E	G	G	L
Nitric Acid 40%	E	G	G	L
Nitric Acid 60%	E	L	G	U
Nitric Acid 68%	G	U	L	U
Nitric Acid 70%	E	E	U	U
Nitric Acid 100%	E	U	U	U
Nitric Acid, Red Fuming	U	U	U	U
Nitrobenzene	U	U	U	U
Nitropropane	O	O	O	O
Nitrous Acid (10%)	E	E	E	E
Nitrous Oxide	E	E	E	E
Ocenol (Unsaturated Alcohol)	E	E	G	G
Oil and Fats	E	E	E	G
Oleic Acid	E	E	E	E
Oleum	U	U	U	U
Oxalic Acid	E	E	E	G
Oxygen	E	E	E	E
Ozone	G	L	U	U
Palmitic Acid 10%	E	E	E	E
Palmitic Acid 70%	E	U	L	U
Paraffin	E	E	E	E
Pentane	O	O	O	O
Paracetic Acid 40%	E	U	U	U
Perchloric Acid 10%	E	L	G	L
Perchloric Acid 15%	E	U	G	U
Perchloric Acid 70%	E	U	U	U
Perchloroethylene	O	O	O	O
Petrolatum	E	E	E	E
Phenol	L	U	U	U
Phenol (90%)	U	U	U	U
Phenylhydrazine	U	U	U	U
Phenylhydrazine Hydrochloride	E	U	L	U
Phosgene (Gas)	E	G	E	G
Phosgene (Liquid)	U	U	U	U
Phosphoric Acid 0-25%	E	G	E	G
Phosphoric Acid 25-50%	E	E	E	G
Phosphoric Acid 50-75%	E	E	E	G
Phosphoric Acid - 85%	E	E	E	G
Phosphorous (Yellow)	E	G	G	L
Phosphorous (Red)	E	E	E	U
Phosphorous Pentoxide	E	L	G	U
Phosphorous Trichloride	U	U	U	U
Photographic Chemicals	E	E	E	E

Chemical	PVC I 72°F.	PVC I 140°F.	PVC II 72°F.	PVC II 140°F.
Photographic Solutions	E	E	E	E
Phthalic Acid	O	O	O	O
Picric Acid	U	U	U	U
Plating Solutions:				
Brass	E	E	E	E
Cadium	E	E	E	E
Chromium	E	G	G	G
Copper	E	E	E	E
Gold	E	E	E	E
Iron	E	E	O	O
Judium	E	E	E	E
Lead	E	E	E	E
Nickel	E	E	E	E
Rhodium	E	E	E	E
Silver	E	E	E	E
Tin	E	E	E	E
Zinc	E	E	E	G
Potassium Acid Sulfate	E	E	E	E
Potassium Aluminum Sulfate	E	E	O	O
Potassium Alum	E	E	E	E
Potassium Antimonate	E	E	E	E
Potassium Bicarbonate	E	E	E	E
Potassium Bichromate	E	E	E	E
Potassium Bisulfite	E	E	E	E
Potassium Borate 1%	E	E	E	E
Potassium Borate	E	E	E	E
Potassium Bromate 10%	E	E	E	E
Potassium Bromate	E	E	E	E
Potassium Bromide	E	E	E	E
Potassium Carbonate	E	E	E	E
Potassium Chlorate (ag)	E	E	E	E
Potassium Chlorate	E	E	E	E
Potassium Chloride	E	E	E	E
Potassium Chromate (Aln)	E	E	E	E
Potassium Chromate (Neut.)	E	E	E	E
Potassium Chromate 40%	E	E	E	E
Potassium Cuprocyanide	E	E	E	E
Potassium Cyanide	E	E	E	E
Potassium Dichromate 40%	E	E	E	E
Potassium Dichromate	E	E	E	E
Potassium Dichrom (Alkaline)	E	E	E	E
Potassium Dichron (Neutral)	E	E	E	E
Potassium Diphosphate	E	E	E	E
Potassium Ferricyanide	E	E	E	E
Potassium Ferrocyanide	E	E	E	E
Potassium Fluoride	E	E	E	E
Potassium Hydroxide	E	E	E	E
Potassium Hypochlorite	E	G	G	L
Potassium Iodide	E	E	E	E
Potassium Nitrate	E	E	E	E
Potassium Perborate	E	E	E	E
Potassium Perchlorate	E	E	U	U
Potassium Perchlorite	E	E	E	E
Potassium Permanganate 10%	E	E	E	E
Potassium Permanganate 25%	G	L	G	L
Potassium Persulfate	E	E	E	E
Potassium Sulfate	E	E	E	E
Potassium Sulfide	E	E	E	E
Potassium Thiosulfate	E	E	E	E
Propane	E	E	E	E
Proplylene Dichloride	U	U	U	U
Proplylene Glycol	E	E	E	E
Pyrogallic Acid	O	O	O	O
Rayon Coagulating Bath	E	E	E	G
Rochelle Salts	E	E	E	E
Sea Water	E	E	E	E
Salenis Acid (Aqueous)	O	O	O	O
Salicylaldehyde	O	O	O	O
Salt Water	E	E	E	E
Selenic Acid	E	E	E	G
Sewage	E	E	E	E
Silicic Acid	E	E	E	E
Silver Cyanide	E	E	E	E
Silver Nitrate	E	E	E	E
Silver Sulfate	E	E	E	E
Soap Solution	E	E	E	E

*Use PVC 1120

CertainTeed

Chemical	PVC I 72°F.	PVC I 140°F.	PVC II 72°F.	PVC II 140°F.
Soaps	E	E	E	E
Sodium Acetate	E	E	E	E
Sodium Alum	E	E	E	E
Sodium Acid Sulfate	E	E	E	E
Sodium Aluminate	E	E	E	E
Sodium Antinonate	E	E	E	E
Sodium Arsenite	E	E	E	E
Sodium Benzoate	E	E	E	E
Sodium Bicarbonate	E	E	E	E
Sodium Bisulfate	E	E	E	E
Sodium Bisulfite	E	E	E	E
Sodium Borate	E	E	E	E
Sodium Bromide	E	E	E	E
Sodium Carbonate (Soda Ash)	E	E	E	E
Sodium Chlorate	E	G	G	L
Sodium Chloride	E	E	E	E
Sodium Chlorite	E	E	O	O
Sodium Cyanide	E	E	E	E
Sodium Dichromate	E	E	E	G
Sodium Dichromate (Neutral)	E	E	E	E
Sodium Ferricyanide	E	E	E	E
Sodium Ferrocyanide	E	E	E	E
Sodium Fluoride	E	E	E	E
Sodium Hydroxide 10%	E	E	E	E
Sodium Hydroxide 15%	E	E	E	E
Sodium Hydroxide 35%	E	E	E	E
Sodium Hydroxide 70%	E	E	O	O
Sodium Hydroxide (Satr)	E	E	E	E
Sodium Hypochlorite	E	E	E	E
Sodium Iodide	E	E	E	E
Sodium Nitrate	E	E	E	E
Sodium Nitrite	E	E	E	E
Sodium Perborate	E	E	O	O
Sodium Peroxide	E	E	E	E
Sodium Phosphate	E	E	E	E
Sodium Phosphate - Acid	E	E	G	G
Sodium Silicate	E	E	E	E
Sodium Sulfate	E	E	E	E
Sodium Sulfide	E	E	E	E
Sodium Sulfite	E	E	E	E
Sodium Thiosulfate (Hypo)	E	E	E	E
Sour Crude Oil	E	E	E	E
Stannic Chloride	E	E	E	E
Stannous Chloride (50%)	E	E	E	E
Stannous Chloride	E	G	E	G
Starch	E	E	E	E
Stearic Acid	E	E	E	E
Stoddards Solvent	E	E	U	U
Sulfated Detergents	E	E	E	E
Sulfur	E	E	E	E
Sulfur Dioxide Gas - Dry	E	E	E	E
*Sulfur Dioxide Gas - Wet	E	L	U	U
Sulfur Trioxide	E	E	E	G
Sulphur Dioxide - Liquid	G	U	L	U
Sulphuric Acid 0-10%	E	E	E	G
Sulphuric Acid 10-30%	E	E	E	G
Sulphuric Acid 30-50%	E	E	E	G

*Use PVC 1120

Chemical	PVC I 72°F.	PVC I 140°F.	PVC II 72°F.	PVC II 140°F.
Sulphuric Acid 50-75%	E	E	E	G
Sulphuric Acid 75-90%	E	E	L	L
Sulphuric Acid 95%	E	G	U	U
Sulphurous Acid	G	U	L	U
Tan Oil	E	E	E	E
Tannic Acid	E	E	E	E
Tanning Liquors	E	E	E	E
Tartaric Acid	E	E	E	E
Tetrachloroethane	O	O	O	O
Tetraethyl Lead	E	G	G	L
Tetrahydro Furane	U	U	U	U
Thionyl Chloride	U	U	U	U
Tepineol	G	L	G	L
Tin Chloride	E	E	E	E
Titanium Tetrachloride	E	U	E	U
Toluol or Toluene	U	U	U	U
Toxaphene (90%)	O	O	O	O
Tributyl Phosphate	U	U	U	U
Trichloroacetic Acid	E	E	E	E
Trichloroethylene	U	U	U	U
Tricresylphosphate	U	U	U	U
Triethanolamine	E	G	G	U
Triethylamine	E	E	G	L
Trimethyl Propane	E	G	L	U
Trisodium Phosphate	E	E	E	E
Turpentine	E	E	L	U
Urea	E	E	E	E
Urine	E	E	E	E
Vegetable Oil	E	E	E	E
Vinegar	E	E	E	U
Vinyl Acetate	U	U	U	U
Water - Acid Mine	E	E	E	E
Water - Distilled	E	E	E	E
Water - Fresh	E	E	E	E
Water - Salt	E	E	E	E
Water - Sewage	E	E	E	E
Whiskey	E	E	E	E
White Gasoline	E	E	E	E
White Liquor (Paper Industry)	E	E	E	E
Wines	E	E	E	E
Xylene or Xylol	U	U	U	U
Zinc Chloride	E	E	E	E
Zinc Chromate	E	E	E	E
Zinc Cyanide	E	E	E	E
Zinc Nitrate	E	E	E	E
Zinc Sulfate	E	E	E	E
Mixtures of Acids:				
Nitric 15% - Hydrofluoric 4%	E	E	E	G
Sodium Dichromate 13% - Nitric Acid 16 Water 71%	E	E	E	G

This information has been obtained from reliable sources and can be used as a guide to assist in the proper application of PVC pipe. CertainTeed, however, cannot warrant its accuracy. It is suggested that you run your own tests for critical applications.

Pipe & Plastics Group

CertainTeed Corporation
P.O. Box 860
Valley Forge, PA 19482
(610) 341-6820
(610) 341-6837 Fax

Code No. 40-10-29

Printed in U.S.A
0398

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ATTACHMENTS

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

1.0 Introduction

DNCS Environmental Solutions (DNCS Facility) is a proposed Surface Waste Management Facility for oil field waste processing and disposal services. The proposed DNCS Facility is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.36 NMAC, administered by the Oil Conservation Division (OCD). The Facility has been designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, DNCS Properties, LLC.

1.1 Description

The DNCS site is comprised of a 562-acre ± tract of land located south of NM 529 in portions of Section 31, Township 17 South, Range 33 East; and in the northern half of Section 6, Township 18 South, Range 33 East, Lea County, NM. A portion of the 562-acre tract is a drainage feature that will be excluded from development. The drainage feature includes a 500-ft setback and totals 67 acres ±. The DNCS Facility will include two main components; a liquid oil field waste Processing Area (177 acres ±), and an oil field waste Landfill (318 acres ±); therefore the DNCS Facility comprises 495 acres ±. Oil field wastes are anticipated to be delivered to the DNCS Facility from oil and gas exploration and production operations in southeastern NM and west Texas. The Site Development Plan provided in the **Permit Plans, Sheet 3**, identifies the locations of the Processing Area and Landfill facilities.

2.0 DESIGN CRITERIA

The liner system for the DNCS Landfill is designed to meet the requirements of the New Mexico Energy, Minerals and Natural Resource Department, Oil and Gas Rules (i.e., 19.15.36 NMAC). More specifically, 19.15.36.14.D.(1)(b) NMAC requires:

> *"Liners shall be able to withstand projected loading stresses, settling and disturbances from overlying oil field waste, cover materials and equipment operations."*

and further 19.15.36.14.D.(2)(b) NMAC requires:

> *"Geosynthetic material the operator installs on a slope greater than 25 percent shall be designed to withstand the calculated tensile forces acting upon the material. The design shall consider the maximum friction angle of the geosynthetic with regard to a soil-geosynthetic or geosynthetic-geosynthetic interface and shall ensure that overall slope stability is maintained."*

The interior (excavation) sideslopes of the DNCS Landfill are designed at 4H:1V, and the depth of waste is greater than 100 feet (ft). Tensile stresses in liner components were evaluated using guidelines provided in the following documents:

1. Koerner, Robert M. 2005. *Designing with Geosynthetics 5th Edition.* New Jersey:Pearson Prentice Hall (**Attachment III.7.A**).
2. Sharma, Hari D. and Lewis, Sangeeta, P. 1994. *Waste Containment Systems, Waste Stabilization and Landfills: Design and Evaluation*. New York: John Wiley and Sons (**Attachment III.7.B**).
3. Qian, Xuede; Koerner, Robert M.; and Gray, Donald H. 2002. *Geotechnical Aspects of Landfill Design and Construction.* New York: Pretence Hall (**Attachment III.7.C**).
4. CETCO® Lining Technologies, 2009. *Bentomat® GCL Direct Shear Database (TR-114BM)* (**Attachment III.7.D**).
5. Koerner, Robert M. and Koerner, George R. 2007. *Interpretation(s) of Laboratory Generated Interface Shear Strength Data for Geosynthetic Materials with Emphasis on the Adhesion Value.* GRI White Paper #11. Geosynthetic Institute (**Attachment III.7.E**).
6. Thiel, Richard. *A Technical Note Regarding Interpretation of Cohesion (or Adhesion) and Friction Angle in Direct Shear Tests*. Geosynthetics, April May 2009 Volume 27: Pages 10-19 (**Attachment III.7.F**).
7. Thiel, Richard. *Peak vs Residual Shear Strength for Landfill Bottom Liner Stability Analyses.* Thiel Engineering, Oregon House, CA, USA (**Attachment III.7.G**).
8. Bowles, Joseph E. 1977. *Foundation Analysis and Design, 2nd Edition.* United States: McGraw Hill Book Company (**Attachment III.7.H**).
9. Richardson, Clinton P., PhD., PE. 2009. *Municipal Landfill Design Calculations: An Entry Level Manual of Practice.* California: UBuildABook, LLC (**Attachment III.7.I**).
10. GSE Lining Technology, Inc., *GSE HD Textured Product Data Sheet* (**Attachment III.7.J**).

The liner design for the landfill sideslopes (**Figure III.7.1**), from top to bottom, consists of the following components below the waste:

- 24-inches (in.) protective soil layer (on-site soils)
- 60-mil double-sided textured high density polyethylene (HDPE) liner
- 200-mil geonet
- 60-mil double-sided textured HDPE liner
- Geosynthetic clay liner (GCL)
- 6-in. compacted subgrade

The liner design for the landfill floor (**Figure III.7.1**), from top to bottom, consists of the following components below the waste:

- 24-in. protective soil layer (on-site soils)
- 60-mil smooth HDPE liner
- 200-mil geonet
- 60-mil smooth HDPE liner
- Geosynthetic clay liner
- 6-in. compacted subgrade

3.0 CALCULATION OF TENSILE STRESSES IN GEOSYNTHETICS AND SIDESLOPE LINER STABILITY

External shear forces will develop on the 4H:1V sideslopes assuming the placement of an initial 2-ft lift of protective soil, and 8-ft lift of waste; assuming the lifts are unsupported and no adhesion (**Attachment III.7A, Attachment III.7.B**, **Attachment III.7.C** and **Attachment III.7.D**). The unbalanced forces, due to the assumed unsupported placement of the 2-ft protective soil layer and 10-ft waste layer, must be supported by the liner components above the interface with the least amount of frictional resistance. Based on the review of the six references listed in Section 2.0 above, **Tables III.7.1**, **III.7.2, III.7.3, III.7.4** and **III.7.5** present the interface friction angles and soil internal friction angles to be used to determine the tensile stresses in the geosynthetics that will be installed at the DNCS Landfill.

Interface friction angles (Φ) and adhesion (as determined by direct shear testing) for geosynthetics will vary depending on the normal load applied to the geosynthetics. For DNCS, the maximum normal load applied to the floor and sideslope varies. The interface friction angle and adhesion for the geosynthetic interfaces is determined for the sideslope and floor as follows:

TABLE III.7.1
Geosynthetic Interface Friction Angles and Adhesion – Sideslope Normal Load
DNCS Environmental Solutions

Normal Load	Thickness (ft)	Unit Weight (lbs/ft^3)	Total Weight (lbs/ft^2)	Range of Shear Testing Loads[1] per ASTM D 5321 (lbs/in^2)
1. Final Cover Soil	3	110	330	
2. Intermediate Cover Soils	1	110	110	0.25 (23.2) = 5.8
3. Oil Field Waste[2]	37.5	74	2,775	0.50 (23.2) = 11.6
4. Protective Soil Layer	2	110	220	1.0 (23.2) = 23.2
Design Vertical Load:		Total:	3,435 lbs/ft^2 (23.9 lbs/in^2)	
Design Normal Load: = [(23.9 lbs/in^2) (cos 14.04°)] = 23.2 lbs/in^2		Total:	44.3 lbs/in^2	5.8 11.6 23.2

Notes: 1. Shear testing loads based on ASTM D 5321 = 0.25 (maximum normal load); 0.5 (maximum normal load); 1.0 (maximum normal load)
2. Oil field waste on the sideslope varies from 0 to approximately 75 feet in depth; averaging 37.5 feet at the centroid of the sideslope waste mass.

TABLE III.7.2
Geosynthetic Interface Friction Angles and Adhesion – Floor Normal Load
DNCS Environmental Solutions

Normal Load	Thickness (ft)	Unit Weight (lbs/ft^3)	Total Weight (lbs/ft^2)	Range of Shear Testing Loads[1] per ASTM D 5321 (lbs/in^2)
1. Final Cover Soil	4	110	330	0.25 (87.6) = 21.9
2. Intermediate Cover Soils	1	110	110	0.50 (87.6) = 43.8
3. Oil Field Waste	160	74	11,840	1.0 (87.6) = 87.6
4. Protective Soil Layer	2	110	220	
Design Vertical/Normal Load:		Total:	12,610 lbs/ft^2 (87.6 lbs/in^2)	21.9 43.8 87.6

Note: 1. Shear testing loads based on ASTM D 5321 = 0.25 (maximum normal load); 0.5 (maximum normal load); 1.0 (maximum normal load)

TABLE III.7.3
Geosynthetic Interface Friction Angles and Adhesion[1] – Sideslope Liner System
DNCS Environmental Solutions

Geosynthetic to Geosynthetic Interface	Normal Stresses (lbs/in^2)	Mohr-Coulomb Failure Envelope[2]	
		Φ	Adhesion
Protective Soil Layer (SM)[2] to Double-Sided Textured HDPE FML	Reference 1	26°	ND
HDPE Geonet to Double-Sided Textured HDPE FML	Reference 2	7.0° – 25° Assume ¾ = 20°	ND
Double-Sided Textured HDPE FML to Nonwoven Geotextile of GCL	Reference 2	15° - 32° Average = 24°	ND
Nonwoven Geotextile of GCL to Subgrade Soil (undrained)	5.8 11.6 23.2 Reference 4	24.3°	92 lbs/ft^2

Notes: 1. *Values reported for Φ and Adhesion are based on review of available literature and are used to predict the performance of the liner system. Site specific shear strength testing should be conducted using actual liner system components and soils specified by the Engineer for the facility prior to construction.*
2. *Geotechnical laboratory testing of on-site soils show predominately SP-SC soils within the top 35 feet. For the purposes of these calculations, it was assumed these soils would behave similar to SM soils.*
3. *As recommended in Reference 7, the values for Φ and Adhesion (when available in the literature) represent "Residual Shear Strength" values.*
4. *ND = not determined*

TABLE III.7.4
Geosynthetic Interface Friction Angles and Adhesion[1] – Floor Liner System
DNCS Environmental Solutions

Geosynthetic to Geosynthetic Interface	Normal Stresses (lbs/in^2)	Mohr-Coulomb Failure Envelope[2]	
		Φ	Adhesion
Protective Soil Layer (SM) to Smooth HDPE FML	Reference 1	18°	ND
HDPE Geonet to Smooth HDPE FML	Reference 2	5° – 19° Average = 12°	ND
Smooth HDPE FML to Nonwoven Geotextile of GCL	Reference 2	8° – 12° Average = 10°	ND
Nonwoven Geotextile of GCL to Subgrade Soil (undrained)	21.9 43.8 87.6 Reference 4	32°	61 lbs/ft^2

Notes: 1. *Values reported for Φ and Adhesion are based on review of available literature and are used to predict the performance of the liner system. Site specific shear strength testing should be conducted using actual liner system components and soils specified by the Engineer for the facility prior to construction.*
2. *Geotechnical laboratory testing of on-site soils show predominately SP-SC soils within the top 35 feet. For the purposes of these calculations, it was assumed these soils would behave similar to SM soils.*
3. *As recommended in Reference 6, the values for Φ and Adhesion (when available in the literature) represent "Peak Shear Strength" values.*
4. *ND = not determined*

TABLE III.7.5
Soils Internal Friction Angle and Cohesion[1,2]
DNCS Environmental Solutions

Material	Density	Φ	Cohesion [Assumed]
Protective Soil Layer (Relative Density, Medium)	110 lbs/ft^3	33º	0 lbs/ft^2
Oil Field Stabilized Waste (Relative Density, Medium	74 lbs/ft^3	33º	0 lbs/ft^2
Compacted Subgrade (Relative Density, Medium to Dense)	112 lbs/ft^3	35º	0 lbs/ft^2
Natural Foundation Soils (Relative Density, Medium to Dense)	110 lbs/ft^3	35º	0 lbs/ft^2

Notes: 1. Values reported for Φ and Cohesion are based on review of available literature and are used to predict the performance of the liner system. Site specific shear strength testing should be conducted on soils specified by the Engineer for the facility prior to construction.

2. Geotechnical laboratory testing of on-site soils show predominately SP-SC soils within the top 35 feet. For the purposes of these calculations, the values of Φ are based on the "blow counts" recorded during the drilling of borings B-3 through B-5 (average range 27 – 45); and using information contained in Reference 8. No cohesion was assumed providing an additional factor of safety to these calculations.

Based on the sidelsope liner system design, the interface with the least amount of frictional resistance occurs at the geonet to double-sided textured interface (Φ = 20˚) [**Table III.7.3** as referenced in **Attachment III.7.B, p. 149**]. The unbalanced forces, due to the assumed unsupported oil field waste and protective soil layer, are based on the sideslope liner stability calculations presented in Reference 9; *Municipal Landfill Design Calculations: An Entry Level Manual of Practice* (Richardson, 2009) [**Attachment III.7.I**]:

Where given the following:

ß	=	slope angle for 4H:1V sideslope = 14.04˚
F_X	=	Shear forces that are equal to the product of the normal force (W_WCos ß) and the tangent of the friction angle between the two neighboring materials.
W_W	=	Weight of Waste.
T_W	=	Friction force on edge of waste.
W_{net}	=	Net weight of waste acting upon the liner system ($W_W – T_W$)
h_{waste}	=	Height of waste layer = 10 ft
h_{soil}	=	Height of protective soil layer = 2 ft
Φ_{waste}	=	Waste internal angle of friction = 33˚
Φ_{soil}	=	Soil Internal angle of friction = 33˚

Density of waste	=	74 lbs/ft^3
Density of protective soil	=	110 lbs/ft^3 dry density

A. Determine weight of waste and protective soil layer on sideslope:

Weight of waste and protective soil layer = [½ (base)(height)] x (density of material)]

$W_{waste/soil}$ = 0.5 (h_{waste}) [(h_{waste})(slope factor)] (density of waste) + 0.5 (h_{soil}) [(h_{soil})(slope factor)] (density of protective soil layer)

$W_{waste/soil} = 0.5\ (8\ ft)\ [(8\ ft)(4)]\ (74\ lbs/ft^3) + 0.5\ (2\ ft)\ [(2\ ft)(4)]\ (110\ lbs/ft^3)$

$W_{waste/soil} = 9{,}472.0\ lbs/ft + 880\ lbs/ft = 10{,}352.0\ lbs/ft$

B. Determine friction force on edge of waste and protective soil layer:

$T_W = (K_o)\ (\sigma_v)\ (\tan\ (\Phi_{waste})\ (h_{lift}) + (K_o)\ (\sigma_v)\ (\tan\ (\Phi_{soil})\ (h_{lift})$

Where:

K_o	=	$1 - \sin\ (\Phi_{waste}) = 1 - \sin\ (33\ ^o) = 0.455$
K_o	=	$1 - \sin\ (\Phi_{soil}) = 1 - \sin\ (33\ o) = 0.455$
σ_v	=	(0.5) (h_{waste}) (density of waste) = (0.5)(8 ft)(74 lbs/ft^3) = 296 lbs/ft^2
σ_v	=	(0.5) (h_{soil}) (density of soil) = (0.5)(2 ft)(110 lbs/ft3) = 110 lbs/ft^2
Φ_{waste}	=	Internal friction angle of waste = 33^o
Φ_{soil}	=	Internal friction angle of protective soil = 33^o
h_{waste}	=	height of lift of waste = 8 ft
h_{soil}	=	height of lift of soil = 2 ft

$T_W = (0.455)(296\ lbs/ft^2)(\tan\ (33^o))\ (8\ ft) + (0.455)(110\ lbs/ft^2\ )(\tan\ (33^o))\ (2\ ft)$

$T_W = 699.7\ lbs/ft + 65.0\ lbs/ft$

$T_W = 764.7\ lbs/ft$

C. Net weight of waste and protective soil layer

$W_{net} = W_{waste/soil} - T_W$

$W_{net} = 9{,}472\ lbs/ft - 764.7\ lbs/ft$

$W_{net} = 8{,}707.3\ lbs/ft$

D. Determine weight force component

$N_A = (W_{net}\ )\ (\cos\ (\text{slope angle}))$

Where N_A is the normal force perpendicular to the sideslope **(Figure III.7.2)**

$N_A = 8{,}707.3\ lbs/ft\ (\cos\ 14.04^o)$

N_A = 8,447.2 lbs/ft

E. Calculate shear forces on geosynthetics (Figure III.7.2)

Determine friction forces:

1. Interface friction angle between protective soil layer and double-sided, textured HDPE FML and, $\Phi = 26^\circ$.

   $F_1 = N_A$ (tan 26°)
   F_1 = 8,447.2 lbs/ft (0.487)
   F_1 = 4,113.8 lbs/ft

2. Interface friction angle between double-sided textured HDPE and the geonet, $\Phi = 20^\circ$

   $F_2 = N_A$ (tan 20°)
   F_2 = 8,447.2 lbs/ft (0.364)
   F_2 = 3,074.8 lbs/ft

   Geomembrane tension = 4,113.8 lbs/ft – 3,074.8 lbs/ft.
   Geomembrane tension = 1,039.0 lbs/ft = 86.5 lbs/in.

   $F_1 > F_2$, therefore the geomembrane is in tension.

The force difference must be carried by the geomembrane. The actual stress in the geomembrane is given by:

$\sigma_{actural} = (F_1 - F_2)/t_{geomembrane}$

$\sigma_{actural}$ = actual stress in geomembrane

$t_{geomembrane}$ = geomembrane thickness = 60 mil = 0.06in.

σ_{actual} = 86.5 lbs/in / 0.06 in

σ_{actual} = 1,441.7 lbs/in^2

The factor of safety for the geomembrane against failure in tension is:

$FS_{geomembrane} = \sigma_{yield} / \sigma_{actural}$

The tensile stress in the 60-mil geomembrane is 1,441.7 lbs/ft. This positive value indicates that the 60-mil geomembrane is in tension. The strength at yield for the geomembrane is 126 lbs/in-width (**Attachment III.7.J**) which results in a 60-mil geomembrane yield stress (σ_{yield}) of 2,100 lbs/in^2. Therefore a geomembrane with a strength at yield of 126 lbs/in or greater will not be adversely affected if a 8-ft lift of waste and 2-ft lift of PSL is placed on the sideslope as calculated below:

$FS_{geomembrane}$ = 2,100 lbs/in^2 / 1,441.7 lbs/in^2

$FS_{geomembrane}$ = 1.4

3. $F_3 = F_2$ = 3,074.8 lbs/ft for static no-slip condition.

4. Interface friction angle between double-side textured HDPE FML and geonet, $\Phi = 20^o$.

$F_4 = N_A$ (tan 20˚)
F_4 = 8,447.2 lbs/ft (0.364)
F_4 = 3,074.8 lbs/ft

Geonet tension = 3,074.8 lbs/ft – 3,074.8 lbs/ft
Geonet tension = 0 lbs/ft = 0 lbs/in.

$F_3 = F_4$, therefore the geonet is not in tension.

5. $F_4 = F_5$ = 3,074.8 lbs/ft for static no-slip condition.

6. Interface friction angle between geonet and double-side textured HDPE FML, $\Phi = 20^o$.

$F_6 = N_A$ (tan 20˚)
F_6 = 8,447.2 lbs/ft (0.364)
F_6 = 3,074.8 lbs/ft

Geomembrane tension = 3,074.8 lbs/ft – 3,074.8 lbs/ft
Geomembrane tension = 0 lbs/ft = 0 lbs/in.

$F_5 = F_6$, therefore the geomembrane is not in tension.

7. $F_6 = F_7$ = 3,074.8 lbs/ft for static no-slip condition.

8. Interface friction angle between double-side textured HDPE FML and nonwoven geotextile of GCL, $\Phi = 24^o$.

$F_8 = N_A$ (tan 24˚)
F_8 = 8,447.2 lbs/ft (0.435)
F_8 = 3,674.5 lbs/ft

Geomembrane tension = 3,074.8 lbs/ft – 3,674.5 lbs/ft
Geomembrane tension = - 599.7 lbs/ft = - 49.9 lbs/in.

$F_7 < F_8$, therefore the geomembrane is not tension.

9. $F_8 = F_9 = 3{,}674.5$ lbs/ft for static no-slip condition.

10. Interface friction angle between nonwoven geotextile of GCL and subgrade soils, $\Phi = 24.3^{o}$.

$F_{10} = N_A (\tan 24.3^{\circ})$
$F_{10} = 8{,}447.2$ lbs/ft (0.452)
$F_{10} = 3{,}818.1$ lbs/ft

GCL tension = 3,674.5 lbs/ft – 3,818.1 lbs/ft
GCL tension = - 143.6 lbs/ft = - 11.9 lbs/in.

$F_9 < F_{10}$, therefore the GCL is not tension.

F. Conclusion

The unbalanced forces due to the assumed unsupported placement of the 2-ft protective soil layer and 8-ft waste layer is supported by the 60-mil double-sided textured HDPE primary liner; the geosynthetics below the HDPE primary liner are not in tension. The stress in the primary geomembrane due to the unbalanced force is 1,441.7 lbs/in^2; and provides a factor of safety of 1.4 against failure in tension.

4.0 CALCULATION OF TENSILE STRESSES IN GEOSYNTHETICS DUE TO EQUIPMENT LOADING

A Caterpillar D6E dozer or equivalent will be used to place the protective soil layer up the sideslope a sufficient distance to accommodate an approximate 8 ft lift of waste placed on the floor of the landfill. The maximum unsupported length of protective soil to accommodate this lift will be 33 ft for a 4H:1V sideslope. Parameters to be used in the analysis include:

- Unit weight of protective soil = 110 lbs/ft^3 Dry Density.
- Internal friction angle of protective soil = 33 degrees .
- Critical liner interface friction angle occurs between the HDPE Geonet and the double-sided textured HDPE liner = 20˚ **(Table III.7.3)**.
- Equipment loading assuming a D6E dozer: (CAT Performance Handbook, Edition 29)
    - Weight = 32,000 lbs.
    - Track width = 22 in. = 1.83 ft.
    - Pressure distribution: Assume a 2H:1V distribution, therefore width acting on geomembrane = 9.83 ft.
- Tensile forces acting on Geomembrane:

  - Protective soil layer, F_{soil}
  - D6E dozer, F_{dozer}
- Total resisting forces:
  - Geonet interface friction, F_{geonet}
  - Soil buttress friction at toe of slope, $F_{buttress}$

The minimum interface friction angle for the liner system is 20º and occurs between the HDPE geonet and the double-sided textured geomembrane (**Table III.7.3**).

Tensile forces acting on geomembrane:

F_{soil} = h_{lift} (unsupported slope length) (unit weight of protective soil) (sin (slope angle))

F_{soil} = (2 ft) (33 ft) (110 lbs/ft^3) (sin (14.04˚))

F_{soil} = 1,761.3 lbs/ft

F_{dozer} = [0.5 (dozer weight) / (width acting on geocomposite)] (sin (14.04˚)

F_{dozer} = [0.5 (32,000 lbs) /9.83 ft] (sin (14.04˚)

F_{dozer} = [16,000 lbs/9.83 ft] (0.243)

F_{dozer} = 395.5 lbs/ft

Total tensile force acting on geocomposite = 1,761.3 lbs/ft + 395.5 lbs/ft = 2,156.8 lbs/ft

Total Resisting Forces acting on geomembrane:

$F_{geomembrane}$ = (Weight of protective soil + Weight of Dozer) (cos (slope angle)) (tan (interface friction angle))

$F_{geomembrane}$ = [(2 ft) (33 ft) (110 lbs/ft^3) + (16,000 lbs/9.83 ft)] (cos 14.04˚) (tan 20˚)

$F_{geomembrane}$ = (7,260.0 lbs/ft + 1,627.7 lbs/ft) (0.97) (0.364)

$F_{geomembrane}$ = 3,138.1 lbs/ft

$F_{buttress}$ = [[cos (internal friction angle of soil)] / [cos (internal friction angle of soil + slope angle)]] [[(Unit weight of soil) (thickness of soil)^2 / sin 2 (slope angle)] tan (internal friction angle of soil)]

$F_{buttress}$ = [[cos (33º) / cos (33º + 14.04˚)] [(110 lbs/ft^3 (2 ft)^2) / sin (2 (14.04˚))] [tan (33˚)]

$F_{buttress}$ = [0.839 / 0.682] [440 lbs/ft/0.471] [0.649]

$F_{buttress}$ = [1.23] [934.2] [0.649]

$F_{buttress}$ = 745.7 lbs/ft

Total resisting force acting on geomembrane = 3,138.1 lbs/ft + 745.7 lbs/ft = 3,883.8 lbs/ft

Tensile forces (2,156.8 lbs/ft) < Resisting forces (3,883.8 lbs/ft); therefore geomembrane is not in tension.

Summary

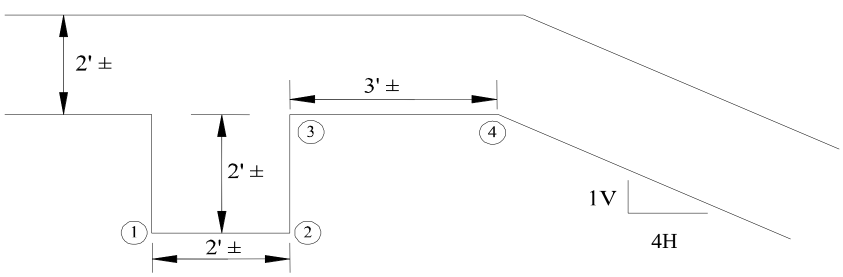
Tensile stress in the geomembrane = 2,156.8 lbs/ft – 3,883.8 lbs/ft = - 1.727.0 lbs/ft = - 143.9 lbs/in. The negative tensile stress indicates that the geocomposite is not in tension. Therefore, placing the protective soil layer 10 ft up the sideslope will not adversely impact the geomembrane.

Conclusion

The tensile stress upon the geocomposite due to equipment loading is – 143.9 lbs/in. This value is less than the tensile (yield) strength for the geocomposite of 270 lbs/in, as previously referenced.

5.0 ANCHOR TRENCH PULLOUT ANALYSIS

Anchor trench configuration:



The anchor trench consists of extending the geosynthetics along the trench bottom to increase resistance force. In order to establish the static equilibrium equation, two imaginary and frictionless pulleys are assumed at the top edge and the bottom corner of the anchor trench (**Attachment III.7.C, page 111, Equation 4-28**). The friction force above a runout geosynthetic is always neglected in the anchor trench. Based on the calculation in Section 3.0, the primary geomembrane is in tension and, the interface friction angle between the geonet and the double-sided textured geomembrane is the minimum interface friction angle of the liner system; therefore, any pull out will occur at this interface.

5.1 Geonet – Double-Sided Textured Geomembrane Interface

$\Sigma F_H = 0$ yields the following equation for the calculation of T (where T = geocomposite tensile force per unit width lbs/ft:

$$T = \frac{(\gamma_s)(d_{cs})(L_{ro})(\tan\delta_c) + [(1-\sin\theta)((\gamma_s)(d_{cs}+0.5d_{AT}))d_{AT} + \gamma_s(d_{cs}+d_{AT})L_{AT}](\tan\delta_c + \tan\delta_F)}{\cos\beta - (\sin\beta)(\tan\delta_c)}$$

Where:

γ_s	=	unit weight of cover and backfill soil = 110 lbs/cf dry density
d_{cs}	=	depth of cover soil = 2 ft
L_{ro}	=	runout length = 3 ft
δ_c	=	friction angle between the geomembrane and underlying HDPE geonet = 20°
θ	=	internal friction angle of compacted backfill soil in anchor trench = 35° **(Table III.7.5**)
d_{AT}	=	depth of anchor trench = 2 ft
L_{AT}	=	width of anchor trench = 2 ft
δ_F	=	interface friction angle between the geomembrane and the compacted backfill soil = 26°
β	=	sideslope angle, measured from horizontal = 14.04°

$$T = \frac{(110\,lbs/cf)(2')(3')(\tan 20°) + [(1-\sin 35°)((110\,lbs/cf)(2'+0.5(2'))(2') + 110\,lbs/cf\,(2'+2')2'](\tan 20° + \tan 26°)}{\cos 14.04° - (\sin 14.04°)(\tan 20°)}$$

$$T = \frac{(240.2\,lbs/ft) + [(0.426)(110\,lbs/cf)(3.0\,ft)(2\,ft) + 110\,lb/cf\,(8\,sf)](0.852)}{0.882}$$

$$T = \frac{240.2\,lbs/ft + [281.2\,lbs/ft + 880\,lbs/ft]0.852}{0.882}$$

$$T = \frac{240.2\,lbs/ft + 989.3\,lbs/ft}{0.882}$$

$$T = \frac{1{,}229.5\,lbs/ft}{0.882}$$

T = 1,394 lbs/ft = 116.2 lbs/in/0.06 in (Geomembrane Thickness) = 1,936.7 lbs/in^2

Ultimate Strength > Anchor Trench Resistance > Allowable Strength

Assume Allowable Strength = Ultimate Strength/Assumed Factor of Safety

Assumed Factor of Safety = 3

2,100 lbs/in^2 > 1,936.7 lbs/in^2 > 700 lbs/in^2

The results indicate that the anchor trench, as designed, provides sufficient capacity such that the anchor trench capacity lies between the geomembrane yield stress and allowable stress.

6.0 GEOSYNTHETIC SLIPPAGE ANALYSIS

In order to determine the factor of safety for slippage and subsequent tension in the liner geosynthetics, the method of active and passive wedges developed by Qian et al. (2002) was used (**Attachment III.7.C, pg. 521**). This calculation utilizes the passive wedge that supports the active wedge on the sideslope, consistent with actual conditions in the field. These calculations were performed along the geomembrane covered slope shown on the cross section (**Figure III.7.3**). To be conservative, the lowest interface friction angles (residual strength values) for the sideslope liner system; and peak strength values for the floor liner system were used. These values taken from **Table III.7.3** are $\delta_A = 20^\circ$, for the interface friction angle between the geonet and double-sided textured HDPE geomembrane on the sideslope; and $\delta_P = 10^\circ$ for the interface friction angle between the geonet and smooth HDPE geomembrane on the floor. The total height of the active wedge is the maximum height of waste over the sloped portion of liner system.

For the purposes of this calculation, the following assumptions and nomenclature (**Table III.7.6**) were used from the literature **(Attachment III.7.C, pg. 521)**:

TABLE III.7.6
Translational Failure Analysis
DNCS Environmental Solutions

W_P =	total weight of the passive wedge
N_P =	normal force acting on the bottom of the passive wedge
F_P =	Frictional force acting on the bottom of the passive wedge (parallel to the bottom of the passive wedge)
E_{HP} =	normal force from the active wedge acting on the passive wedge
E_{VP} =	frictional force acting on the side of the passive wedge
FS_P=	Factor of safety for the passive wedge
δ_P=	Minimum interface friction angle of multi-layer liner components beneath the passive wedge =10˚ (assumed interface friction angle between the geotextile of the GCL and the smooth HDPE geomembrane, from **Table III.7.4**)
Φ_S=	friction angle of the solid waste = 33^o
α=	angle of the waste slope, measured from horizontal
θ=	angle of the landfill cell subgrade, measured from horizontal = 1.15^o
W_A =	weight of the active wedge
W_T =	total weight of active and passive wedges
N_A =	normal force acting on the bottom of the active wedge
F_A=	Frictional force acting on the bottom of the active wedge (parallel to the bottom of the active wedge)
E_{HA} =	normal force from the active wedge acting on the active wedge, $E_{HA} = E_{HP}$
E_{VA}=	frictional force acting on the side of the active wedge, $E_{VA} = E_{VP}$
FS_A =	factor of safety for the active wedge
b =	Horizontal length of the Active Wedge (cell sideslope at its maximum depth) =200 ft
b_p =	Horizontal length of the Passive Wedge = 285 ft
h_t =	Total Height of the Wedges = 95 ft
δ_A=	minimum interface friction angle of multi-layer liner components beneath the active wedge = 20˚ (**Table III.7.3**)
β =	angle of sideslope, measured from the horizontal = 14.04^o
FS =	factor of safety for the entire solid waste mass

Figure III.7.4 also shows measured values for b, b_p, and h_t.

The active wedge is considered first:

$$W_A = \frac{1}{2}\left((b * h_a * \gamma) + (b * h_b * \gamma)\right)$$

$$W_A = \frac{1}{2}\left(200\,ft * 45\,ft * 74\left(\frac{lbs}{ft^3}\right) + 200\,ft * 50\,ft * 74\left(\frac{lbs}{ft^3}\right)\right) = 703{,}000\frac{lbs}{ft}$$

The passive wedge is then considered by multiplying the cross sectional area by the unit weight of waste.

$$W_P = \frac{1}{2}\left(b_P * h_t * \gamma\right) = W_P = \frac{1}{2}\left(285\,ft * 95\,ft * 74\left(\frac{lbs}{ft^3}\right)\right) = 1{,}001{,}775\frac{lbs}{ft}$$

$$W_T = 703{,}000\frac{lbs}{ft} + 1{,}001{,}775\frac{lbs}{ft} = 1{,}704{,}775\frac{lbs}{ft}$$

From **Attachment III.7.C, equation 13.62, pg. 524**, is used to determine the factor of safety.

$aFS^3 + bFS^2 + cFS + d = 0$

Where:

a	=	$W_A \sin\beta \cos\theta + W_P \cos\beta \sin\theta$
b	=	$(W_A \tan\delta_P + W_P \tan\delta_A + W_T \tan\varphi_s) \sin\beta \sin\theta - (W_A \tan\delta_A + W_P \tan\delta_P) \cos\beta \cos\theta$
c	=	$-[W_T \tan\varphi_s (\sin\beta \cos\theta \tan\delta_P + \cos\beta \sin\theta \tan\delta_A) + (W_A \cos\beta \sin\theta + W_P \sin\beta \cos\theta) \tan\delta_A \tan\delta_P]$
d	=	$W_T \cos\beta \cos\theta \tan\delta_A \tan\delta_P \tan\varphi_s$

and:

β	=	14.04^o – sideslope angle; $\sin 14.04^o = 0.243$, $\cos 14.04^o = 0.970$
θ	=	1.15^o – subgrade angle; $\sin 1.15^o = 0.020$, $\cos 1.15^o = 1.000$
δ_P	=	10^o – minimum friction angle of bottom liner system; $\tan 10^o = 0.176$
δ_A	=	20^o – minimum friction angle of sideslope liner system; $\tan 20^o = 0.364$
φ_s	=	33^o – friction angle of waste; $\tan 33^o = 0.649$

Compute values for a, b, c and d:

a = $W_A \sin\beta \cos\theta + W_P \cos\beta \sin\theta$

a = 703,000 lbs/ft (0.243)(1.000) + 1,001,775 lbs/ft (0.970)(0.020)

a = 170,829 lbs/ft + 19,434.4 = 190,263.4 lbs/ft

b = $(W_A \tan \delta_P + W_P \tan \delta_A + W_T \tan \varphi_s) \sin \beta \sin \theta - (W_A \tan \delta_A + W_P \tan \delta_P) \cos \beta \cos \theta$

b = [703,000 lbs/ft (0.176) + 1,001,775 lbs/ft (0.364) + 1,704,775 lbs/ft (0.649)] (0.243)(0.020) – [703,000 lbs/ft (0.364) + 1,001,775 lbs/ft (0.176)] (0.970) (1.000)

b = 1,594,773.1 lbs/ft (0.243)(0.020) – 432,204.4 lbs/ft (0.970)(1.000)

b = 7,750.6 lbs/ft – 419,238.3 lbs/ft = - 411,487.7 lbs/ft

c = $-[W_T \tan \varphi_s (\sin \beta \cos \theta \tan \delta_P + \cos \beta \sin \theta \tan \delta_A) + (W_A \cos \beta \sin \theta + W_P \sin \beta \cos \theta) \tan \delta_A \tan \delta_P]$

c = - [1,704,775 lbs/ft (0.649) [(0.243)(1.000)(0.176) + (0.970)(0.020)(0.364)] + [703,000 lbs/ft (0.970)(0.020) + 1,001,775 lbs/ft (0.243)(1.000)] (0.364)(0.176)]]

c = - [1,704,775 lbs/ft (0.649)[0.0428 + 0.0071] + [(13,638.2 lbs/ft + 243,431.3 lbs/ft) (0.364)(0.176)]]

c = - [1,106,399 lbs/ft [0.0499] + [257,069.5 lbs/ft (0.364)(0.176)]]

c = - [55,209.3 lbs/ft + 16,468.9 lbs/ft]

c = - 71,678.2 lbs/ft

d = $W_T \cos \beta \cos \theta \tan \delta_A \tan \delta_P \tan \varphi_s$

d = 1,704,775 lbs/ft (0.970)(1.000)(0.364)(0.176)(0.649)

d = 68,753.9 lbs/ft

$aFS^3 + bFS^2 + cFS + d = 0$

$190{,}263.4\ FS^3 - 411{,}487.7\ FS^2 - 71{,}678.2\ FS + 68{,}468.9 = 0$

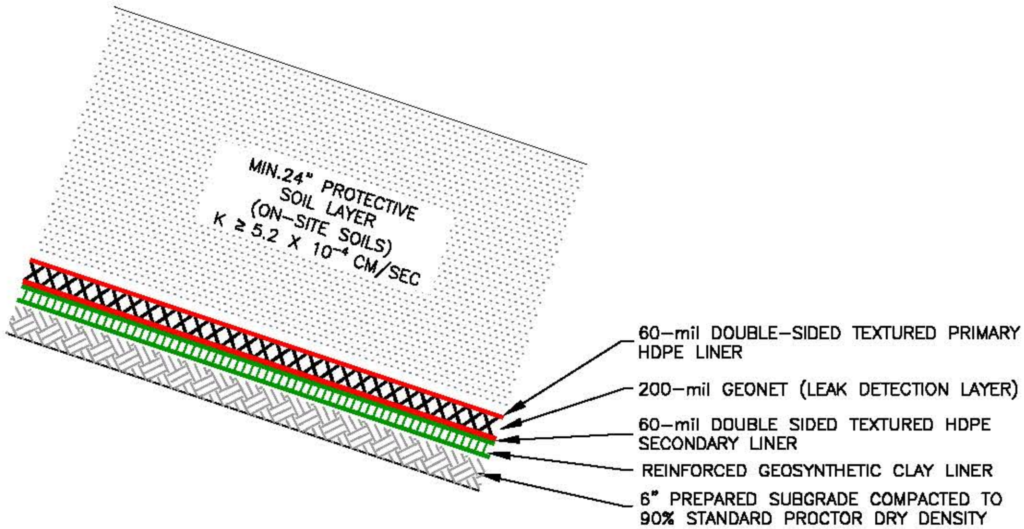
$190{,}263.4\ FS^3 + 68{,}468.9 = 411{,}487.7\ FS^2 + 71{,}678.2\ FS$

This equation is then solved by trial and error as provided in **Table III.7.7**.

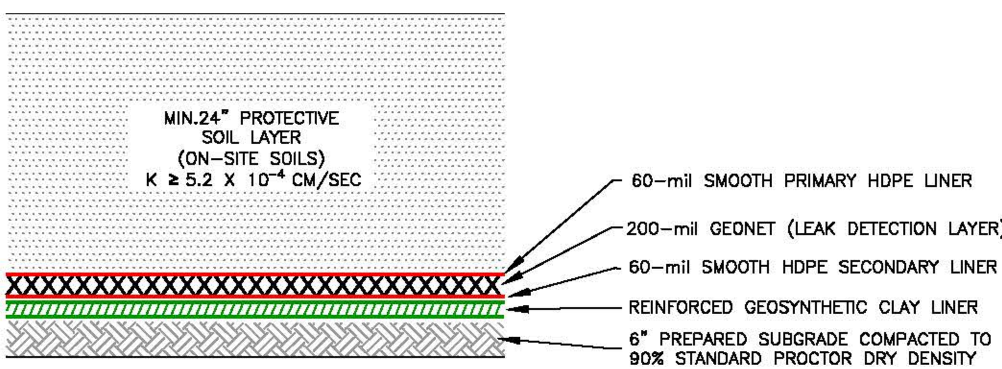
TABLE III.7.7
Geosynthetic Slippage Analysis Factor of Safety Summary
DNCS Environmental Solutions

Assumed FS	190,263.4 FS^3 + 68,468.9	411,487.7 FS^2+ 71,678.2 FS	Closure
(1)	(2)	(3)	(2) – (3)
2.0	1,590,576.10	1,789,307.20	-198,731.10
2.5	3,041,334.53	2,750,993.63	290,340.90
2.3	2,383,403.69	2,341,629.79	41,773.90
2.2	2,094,393.58	2,149,292.50	-54,898.92
2.25	2,235,687.94	2,244,432.43	-8,744.49
2.27	2,293,995.68	2,283,064.48	10,931.20

The factor of safety against translational geosynthetic failure considering active and passive soil wedges is 2.26, which indicates that the passive wedge will more than adequately support the active wedge on the sideslopes without slipping and the geosynthetic liner system is not in tension. Therefore, the proposed liner system design is compatible with calculated external forces.



SIDEWALL LINER SYSTEM



FLOOR LINER SYSTEM

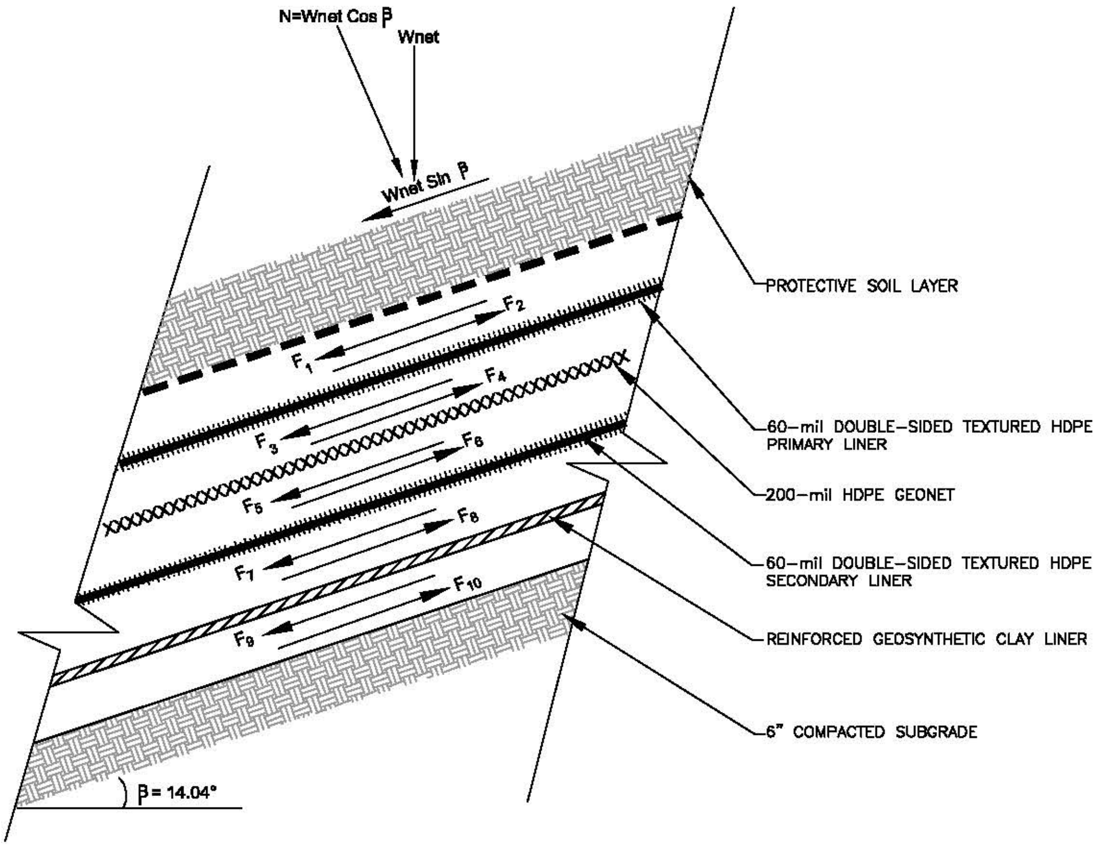
SIDEWALL AND FLOOR LINER SYSTEM DETAIL

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/22/2013	CAD: LINER SCHEM.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: DRT	FIGURE III.7.1
APPROVED BY: IKG	gei@gordonenvironmental.com	

Drawing:P:\acad 2003\542.01.01\PERMIT FIGURES\LINER SCHEM (2).dwg
Date/Time:Oct. 29, 2013-15:24:28



FREE BODY DIAGRAM

NOT TO SCALE

GEOSYNTHETIC FRICTION FORCES

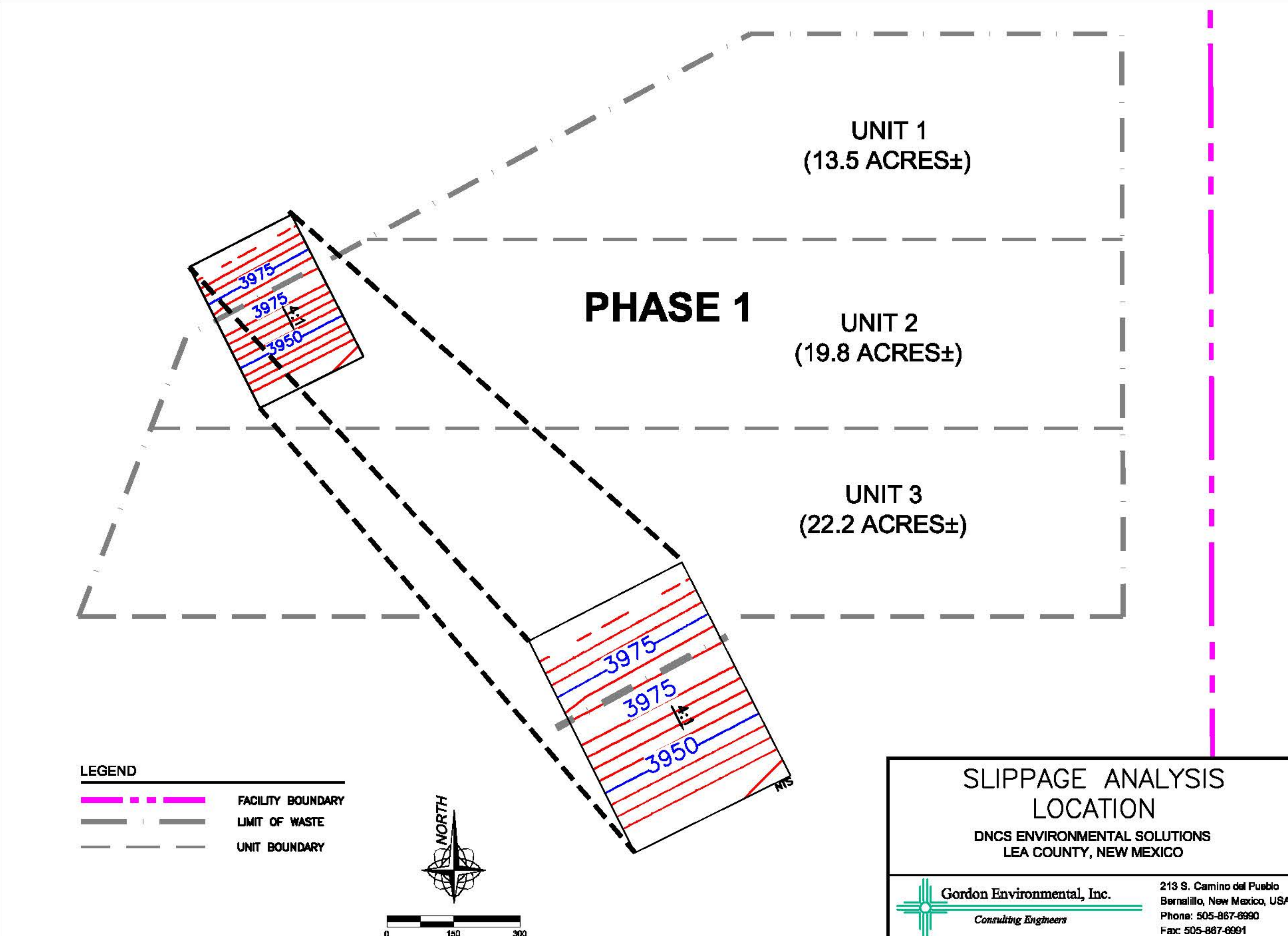
DNCS ENVIRONMENTAL SERVICES
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers

213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/22/2013	CAD: FRICTION.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	FIGURE III.7.2
APPROVED BY: IKG	gei@gordonenvironmental.com	

Drawing:P:\acad 2003\542.01.01\PERMIT FIGURES\FRICTION.dwg
Date/Time:Oct. 22, 2013-11:34:10



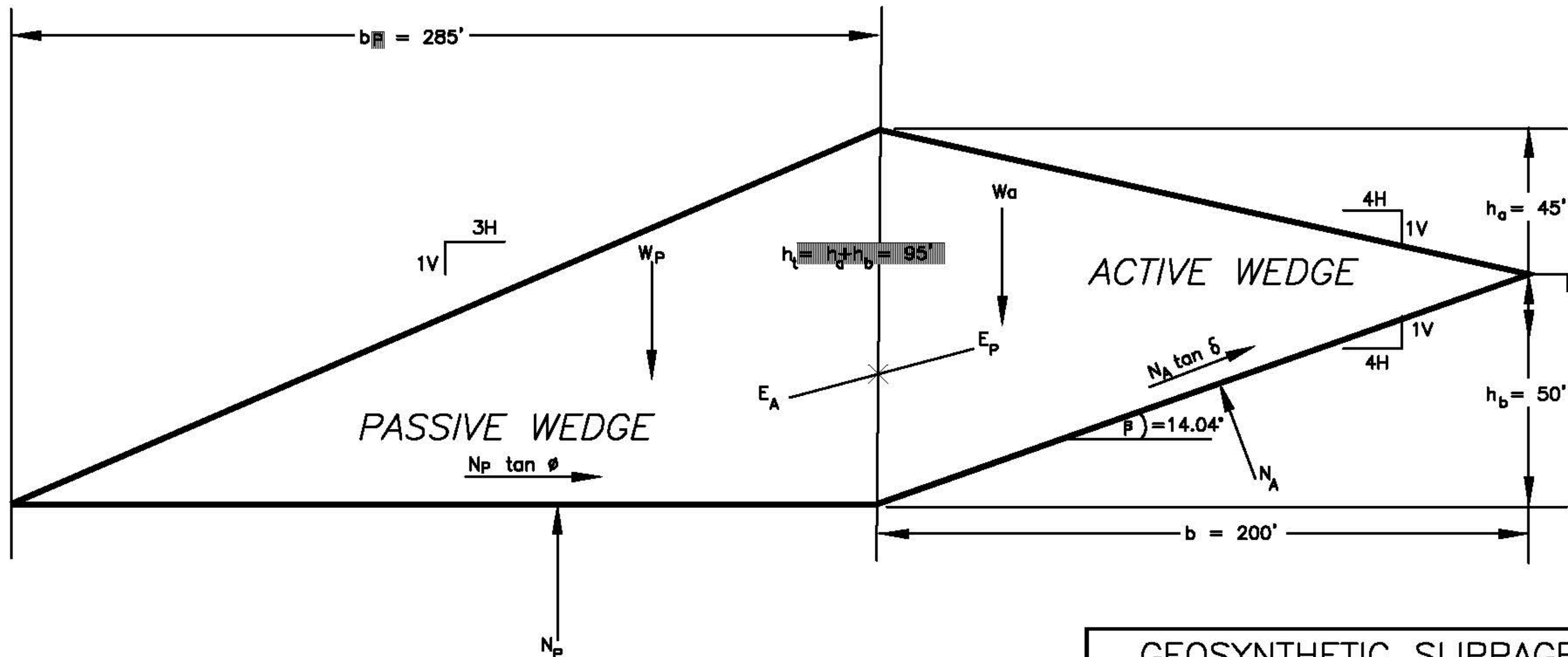
SLIPPAGE ANALYSIS LOCATION

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/29/2013	CAD: SLIP.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: GEI	FIGURE III.7.3
APPROVED BY: IKG	gei@gordonenvironmental.com	

Drawing:P:\acad 2003\542.01.01\PERMIT FIGURES\SLIP.dwg
Date/Time:Oct. 29, 2013-15:30:02 ; LAYOUT: A (LS)



GEOSYNTHETIC SLIPPAGE ANALYSIS

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers

213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 10/29/2013	CAD: GEO ANALYSIS.dwg	PROJECT #: 542.01.01
DRAWN BY: DMI	REVIEWED BY: MRH	FIGURE III.7.4
APPROVED BY: IKG	gei@gordonenvironmental.com	

NO SCALE

Drawing:P:\acad 2003\542.01.01\PERMIT FIGURES\GEO ANALYSIS.dwg
Date/Time:Oct. 29, 2013-15:27:12

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

ATTACHMENT III.7.A

KOERNER, ROBERT M. 2005.

DESIGNING WITH GEOSYNTHETICS, 5th EDITION

NEW JERSEY: PEARSON PRENTICE HALL.

DESIGNING WITH GEOSYNTHETICS

FIFTH EDITION

ROBERT M. KOERNER

TABLE 5.7 PEAK FRICTION VALUES AND EFFICIENCIES OF VARIOUS GEOSYNTHETIC INTERFACES*

(a) Soil-to-Geomembrane Friction Angles

Geomembrane	Soil type: Concrete Sand ($\phi = 30°$)		Ottawa Sand ($\phi = 28°$)		Mica Schist Sand ($\phi = 26°$)	
HDPE						
Textured	30°	(100%)	26°	(92%)	22°	(83%)
Smooth	18°	(56%)	18°	(61%)	17°	(63%)
PVC						
Rough	27°	(88%)	—	—	25°	(96%)
Smooth	25°	(81%)	—	—	21°	(79%)
CSPE-R	25°	(81%)	21°	(72%)	23°	(87%)

(b) Geomembrane-to-Geotextile Friction Angles

Geotextile	Geomembrane: HDPE Textured	HDPE Smooth	PVC Rough	PVC Smooth	CSPE-R Undulating
Nonwoven needle-punched	32°	8°	23°	21°	15°
Nonwoven heat-bonded	28°	11°	20°	18°	21°
Woven monofilament	19°	6°	11°	10°	9°
Woven slit-film	32°	10°	28°	24°	13°

(c) Soil-to-Geotextile Friction Angles

Geotextile	Soil type: Concrete Sand ($\phi = 30°$)		Ottawa Sand ($\phi = 28°$)		Mica Schist Sand ($\phi = 26°$)	
Nonwoven needle-punched	30°	(100%)	26°	(92%)	25°	(96%)
Nonwoven heat-bonded	26°	(84%)	—	—	—	—
Woven monofilament	26°	(84%)	—	—	—	—
Woven slit-film	24°	(77%)	24°	(84%)	23°	(87%)

*Efficiency percentages (in parentheses) are based on Equations (5.8) at (5.9).

Source: Extended from Martin et al. [18].

harder geomembranes being the lowest. A much more extensive and recent paper is by Narejo and Koerner [19].

The frictional behavior of geomembranes placed on clay soils is of considerable importance for composite liners containing solid or liquid wastes. The current requirements are for the clay to have a hydraulic conductivity equal to or less than 1×10^{-7} cm/s and for the geomembrane to be placed directly upon the clay. While an indication of the shear strength parameters has been investigated (e.g., Narejo and Koerner [19] and Koerner et al. [20]), the data are so sensitive to the variables discussed

ATTACHMENT III.7.B

SHARMA, HARI D. AND LEWIS, SANGEETA, P. 1994.

WASTE CONTAINMENT SYSTEMS, WASTE STABILIZATION,

AND LANDFILLS: DESIGN AND EVALUATION.

NEW YORK: JOHN WILEY AND SONS.

WASTE CONTAINMENT SYSTEMS, WASTE STABILIZATION, AND LANDFILLS: DESIGN AND EVALUATION

HARI D. SHARMA, Ph.D, P.E.
Chief Engieer and Director
EMCON Associates
San Jose, California

SANGETTA P. LEWIS, P.E.
Project Manager
CH_2M Hill
Oakland, California

A Wiley-Interscience Publication
JOHN WILEY & SONS, INC.
New York / Chichester / Toronto / Brisbane / Singapore

stress cracking may occur. The recommended elongation for shear test acceptance is greater than 50 percent (Rollin et al., 1991; Giroud and Peggs, 1990; Carlson et al., 1993).

Destructive testing procedures other than shear and peel tests are available to evaluate geomembrane seams, although their use has not yet been widely accepted. Several researchers (Peggs and Charron, 1989; Rollin et al., 1989, 1991; Halse et al., 1991b; Carlson et al., 1993) have suggested the use of microtomes (microscopic evaluation of thin geomembrane sections) to evaluate possible initiation of stress cracking in seams. Another reported method is impact testing (Rollin et al., 1993).

Geomembrane seams may also be tested using nondestructive test methods. These test methods do not measure the seam strength, but rather, detect whether holes exist in the seams. The most commonly used methods are the vacuum test, pressure test, and copper wire spark test. The vacuum test procedure involves placing a soapy solution over a seam approximately 1 to 2 feet in length. A vacuum box with a clear viewing window is placed over the seam length and a vacuum pressure of approximately 5 psi is applied. If a stream of soap bubbles is detected through the viewing window, a leak exists and must be repaired.

Pressure tests can be performed only on double-wedge weld seams. These tests are performed by sealing both ends of an unobstructed double-wedge weld length and then applying approximately 30 psi of air pressure in the channel between the welds through a fine needle. A pressure gage is attached to the needle, and the pressure is monitored for approximately 5 minutes. A reduction in pressure greater than 2 psi during the 5-minute period usually indicates that air is escaping through a leak in the seam. This leak must be located and repaired. In the copper wire spark test, a copper wire is welded into the seam. A current is passed through the copper wire, and any sparks indicate that a hole is present.

3.2 GEOTEXTILES

3.2.1 Types and Functions

Geotextiles are synthetic fabrics used in geotechnical engineering for various applications. The majority of geotextiles are composed of polypropylene or polyester fibers; a small percentage are composed of polyamide or polyethylene. Among the geosynthetics, geotextiles appear to have the most associated terminology and the widest ranging properties. This is due in part to the numerous types of fibers and geotextile manufacturing processes.

The types of fibers used in the manufacture of geotextiles include monofilament, staple, and slit[20] film. If fibers are twisted or spun together, they are known as a yarn. Monofilament fibers are created by extruding molten polymer through an apparatus containing several small-diameter holes, known as a spinnaret. The extruded polymer strings are then cooled and stretched to align the polymers and give

[20] Slit-film fibers are also known as split-film fibers.

the fiber increased strength. Staple fibers are also manufactured by extruding polymer through a spinnaret; however, the extruded strings are twisted together and cut into 1- to 4-inch lengths. The staple fibers are then spun into longer fibers known as staple yarns. Finally, slit-film fibers are manufactured by extruding a continuous sheet of polymer and cutting it into fibers by knives or lanced air jets. Slit-film fibers are rectangular in cross section rather than the circular cross sections of the monofilament and staple fibers.

The fibers or yarns are formed into geotextiles using either woven or nonwoven (spunbonded) methods. Woven geotextiles are formed using traditional weaving methods and a variety of weave types. Common terminology associated with woven geotextiles include machine direction, cross machine direction, selvage, warp, and weft. The machine direction refers to the direction in the plane of fabric parallel to the direction of manufacture, and conversely, the cross machine direction refers to the direction in the plane of fabric perpendicular to the direction of manufacture. The machine direction is also known as the warp, since warp yarns are those yarns placed lengthwise on the weaving loom; and the cross machine direction is known as the weft, since weft yarns are woven between and perpendicular to the warp yarns. The selvage is the finished area on both sides of the geotextile width that prevents the yarns from unraveling.

To create nonwoven geotextiles, the manufactured fibers are placed and oriented on a moving conveyor belt. The fibers are bonded by needle punching, melt bonding, or resin bonding. The needle-punching process consists of pushing numerous barbed needles through the fiber web. The fibers are thus mechanically interlocked into a stable configuration. As the name implies, the melt bonding process consists of melting and pressurizing fibers together at their crossover points. In resin bonding, an acrylic resin is applied to the fiber web to form the geotextile.

In waste containment facilities, geotextiles are most commonly used for filtration, separation, reinforcement, cushioning, and drainage. A relatively new application for geotextiles is an alternative daily cover over refuse. Typically, nonwoven geotextiles are used in waste containment facilities for filtration, separation, cushioning, and drainage. Woven geotextiles are usually used for reinforcement. Both woven and nonwoven geotextiles may be used for alternative daily cover.

3.2.2 Material Properties

As with geomembranes, there are numerous tests that may be performed on geotextiles. However, geotextiles have numerous different applications where geomembranes are used almost exclusively as a barrier material. In developing geotextile specifications, it is important that the designer understand the material tests and specify material properties important for the geotextiles' intended use. The following sections therefore indicate the geotextile application for which the material test is significant. Index or quality control tests are also discussed.

The material properties generally specified for waste containment system applications are thickness, mass per unit area, uniaxial tensile strength, multiaxial tensile strength, puncture resistance, trapezoid tear strength, apparent opening size, permittivity, transmissivity, and ultraviolet resistance. In specifying geotextile material properties, the designer should be aware that many reported material properties and test methods were borrowed from the textile industry. Many tests are therefore more applicable to evaluating fabric for clothing rather than for engineering fabrics. Most geotextile properties reported by manufacturers are index or quality control tests and are not intended for engineering design. Hopefully, as further research on geotextiles is performed, material tests to evaluate engineering properties will be developed.

Thickness (ASTM D 177,[21] D 5199). The average thickness of a geotextile is measured using a thickness gage under a gradually applied, specified pressure. The pressure to be applied depends on the material type. For geotextiles, a pressure of approximately 0.3 psi is typically used. The thickness of a geotextile alone is generally not critical for design. It is, however, related to other material properties, such as mass per unit area, tensile strength, puncture resistance, and tear resistance. Thickness is also important if the geotextile is used for cushioning and in calculating permeability coefficients.

Mass per Unit Area (ASTM D 5261[22]). The mass per unit area of a geotextile is determined by weighing several test specimens of known area, taken from various locations of the fabric sample. The calculated values are averaged to obtain the mean mass per unit area of the sample. Geotextiles, especially nonwoven geotextiles, are commonly referred to by an abbreviated form of their mass per unit area. For example, a nonwoven geotextile that is 8 ounces per square yard is commonly referred to as an 8-ounce geotextile. Although this is obviously incorrect, the problem is not as much in the terminology as it is in specifying the mass per unit area as a design value. Many specifiers attribute a certain mass per unit area to a certain set of mechanical and hydraulic properties, such as puncture resistance, tear resistance, apparent opening size, and tensile strength. While the mass per unit area is related to these properties, there is not a direct correlation. Therefore, geotextiles with a mass per unit area of 8 oz/yd^2 can have widely varying mechanical and hydraulic properties. A certain mass per unit area may be required, however, if the geotextile is to be used as a cushion.

Uniaxial Tensile Strength (ASTM D 4632,[23] D 4595[24]). The uniaxial tensile strength of geotextiles is measured in a tensile testing machine by applying a continually increasing load along the longitudinal length of a specimen. The specimen is grasped within clamps, specially designed to prevent slippage (Figure 3.33). The distance between clamps (called the gage dimension) and the specimen dimensions

[21] ASTM D 1777: Standard Method for Measuring Thickness of Textile Materials.

[22] ASTM D 5261: Standard Test Method for Measuring Mass per Unit Area of Geotextiles.

[23] ASTM D 4632: Standard Test Method for Breaking Load and Elongation of Geotextiles (Grab Method).

[24] ASTM D 4595: Standard Test Method for Tensile Properties by the Wide-Width Strip Method.

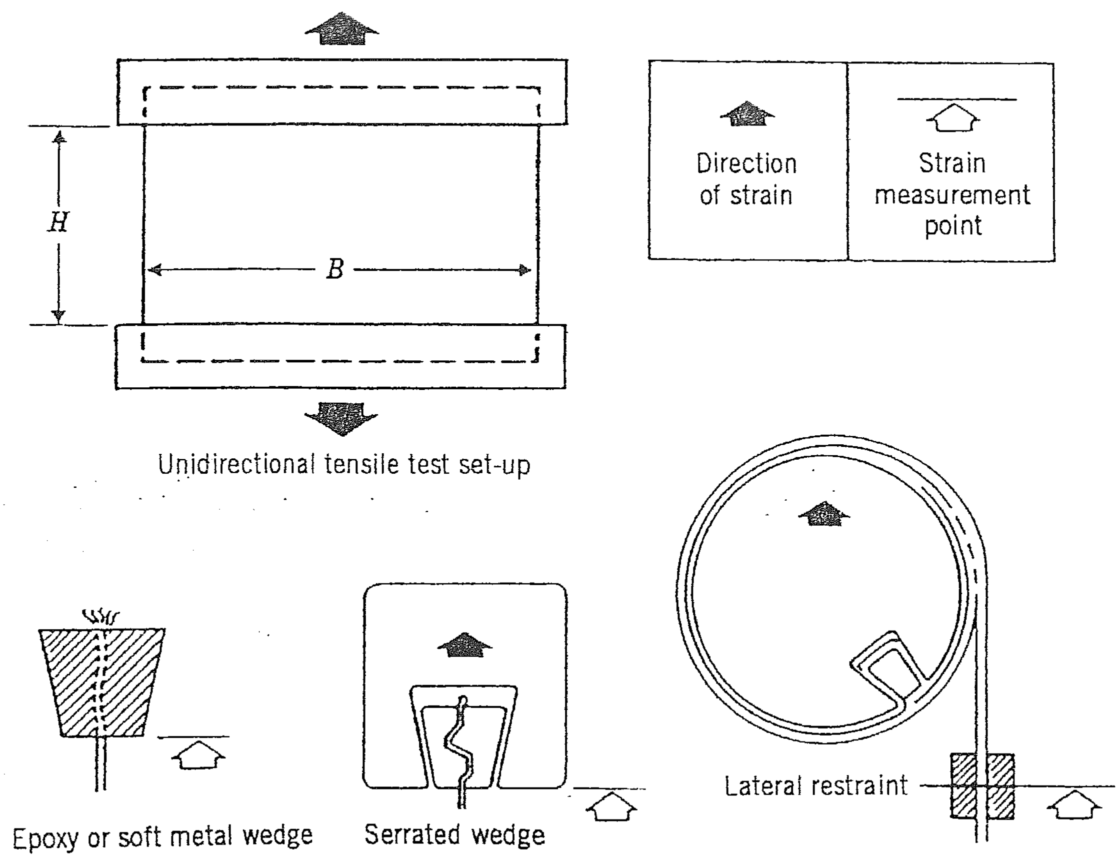


Figure 3.33 Clamping systems for uniaxial tension test. (From Myles, 1987.)

are standardized. While the test values typically reported are the breaking load (reported in pounds) and apparent elongation (reported as a percentage increase in length), a load elongation curve or a stress-strain curve can also be produced (Figure 3.34). The stress-strain curve is generated by dividing the load by the width and thickness of the geotextile specimen. Since the thickness of the geotextile typically decreases as tensile load is applied and is also variable throughout the specimen, the "stress" is often reported as the load divided by the specimen width (in lb/in.). This curve is important in assessing geotextile strength, particularly for strain compatibility in soil reinforcement applications.

Researchers throughout the world have studied the factors affecting the uniaxial tensile strength of geotextiles (Shrestha and Bell, 1982; Moritz and Murray, 1982; Richards and Scott, 1986; Rowe and Ho, 1986; Cazzuffi et al., 1986; Myles, 1987; deGroot et al., 1990; Anjiang et al., 1990; Wayne et al., 1993). These factors include specimen size, aspect ratio (width-to-length ratio), stain rates, gage length, clamping conditions, fabric type and construction, and anisotropic conditions. This research has led to the standardization of uniaxial tension testing procedures and the following general trends:

- The breaking force per unit width measured in a uniaxial tensile test is not affected significantly by the sample width (Moritz and Murray, 1982; Shrestha and Bell, 1982; Richards and Scott, 1986; Rowe and Ho, 1986; Cazzuffi et

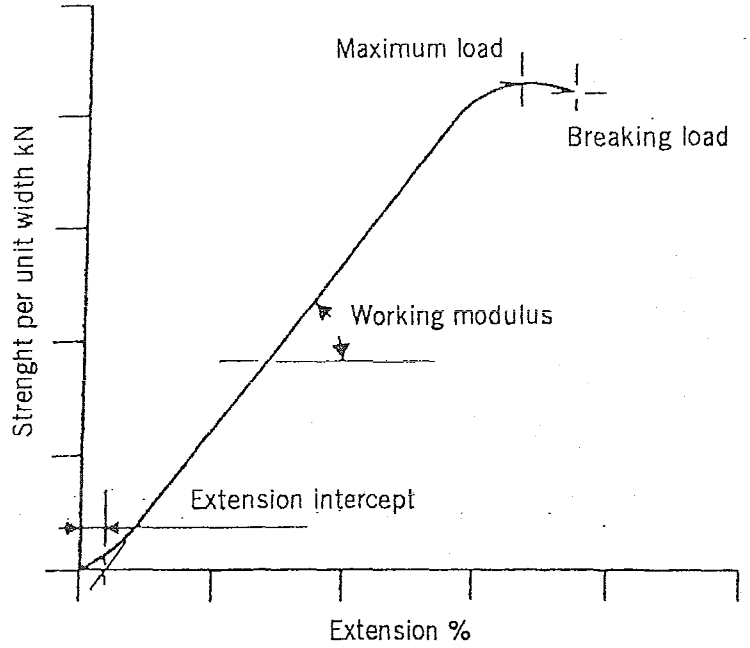


Figure 3.34 Strength per unit width versus extension curve for uniaxial tension test. (From Myles, 1987.)

al., 1986; Wayne et al., 1993) but may be influenced by the gage length[25] (Shrestha and Bell, 1982; Richards and Scott, 1986; Montalvo and Sickler, 1993).

- Depending on the type of geotextile, the modulus and elongation properties may vary with specimen width and gage length (Shrestha and Bell, 1982; Rowe and Ho, 1986; Richards and Scott, 1986; Wayne et al., 1993).
- Both woven and nonwoven geotextiles show anisotropic behavior. The anisotropic behavior in woven geotextile is expected due to the machine and cross directions. For nonwoven geotextiles, anisotropy is due to potential fluctuations and irregularity in the manufacturing process (Novais-Ferreira and Quaresma, 1982; Richards and Scott, 1986; Cazzuffi et al., 1986).
- Fabric structure has a significant influence on the stress-stain behavior. Woven and heat-bonded geotextiles show high strength and modulus and low elongation; needle-punched geotextiles have low strength and modulus and high elongation (Moritz and Murray, 1982; Shrestha and Bell, 1982; Richards and Scott, 1986).

Standard test methods have been developed for uniaxial geotextile tensile testing. The two commonly used standards include the grab (ASTM D 4632) and wide-width (ASTM D 4595) methods. The strip test is also often used and reported in the literature. Figure 3.35 shows various tensile test specimen sizes.

The strip and grab tensile tests utilize procedures originally established for the

[25]The gage length is defined as the length of the specimen between clamps.

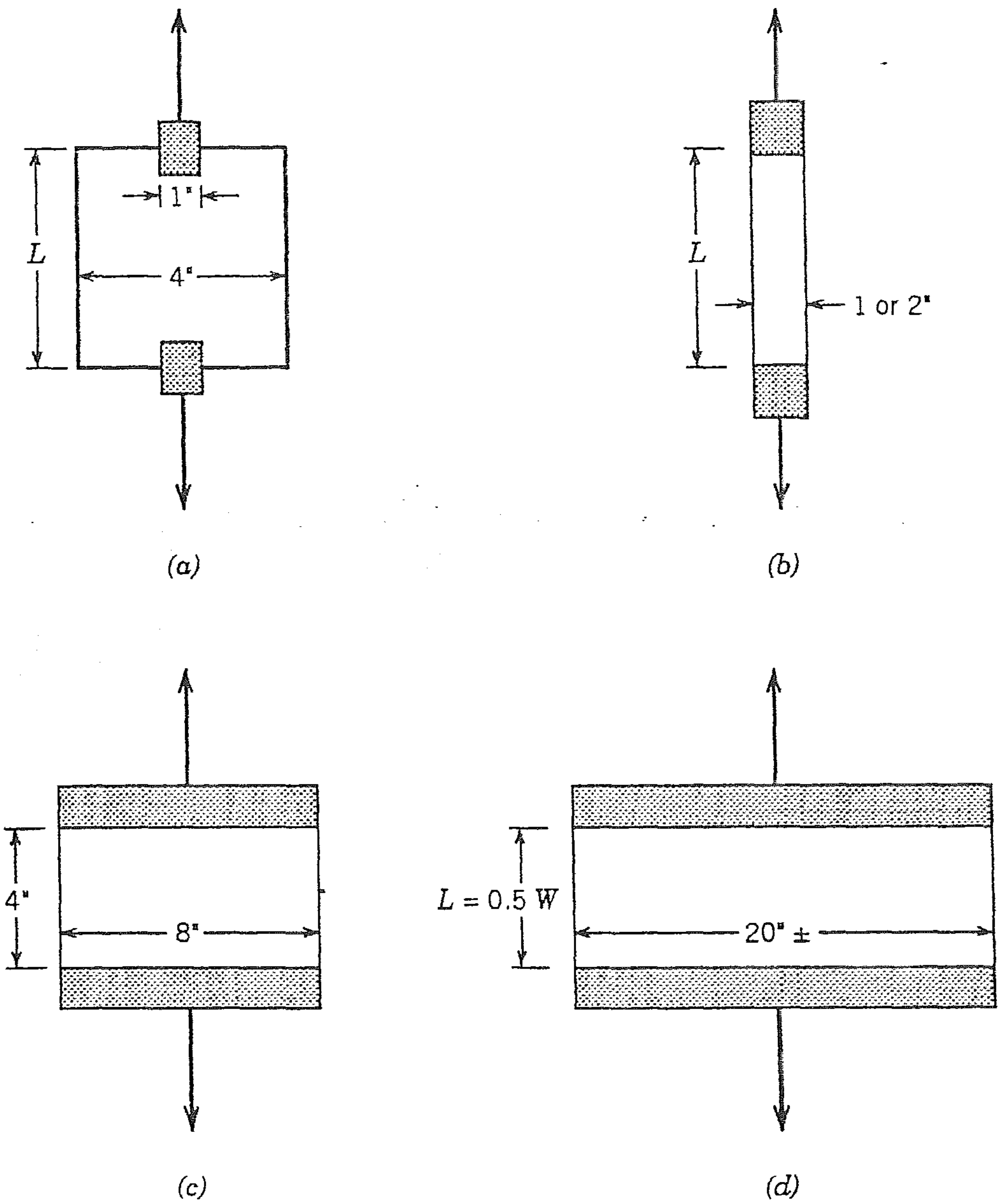


Figure 3.35 Various tensile test specimen sizes: *(a)* ASTM D4632 grab; *(b)* "narrow" strip; *(c)* ASTM D4595 wide width; *(d)* very wide width. (From Koerner, 1990.)

textile industry. The *strip tensile test* is typically performed on a 1- to 2-inch-wide specimen. As the tensile load is applied to this specimen, the specimen necks in its central region. These edge effects have significant influence on the tensile strength. In the *grab tensile test*, as shown in Figure 3.35, the clamps holding the specimen do not hold the entire width of the specimen. The grab method measures the "effective strength" of the geotextile, that is, the strength of the material in a specific width, together with the additional strength contributed by adjacent material. Both the grab and strip tests are useful as quality control or acceptance tests but have limited usefulness for design. Table 3.9 presents a range of typical grab tensile strength values for some nonwoven geotextiles.

The recommended tensile test for design is the *wide-width tensile test*, ASTM D 4595. This test was developed specifically for geotextiles and uses an 8-inch-wide

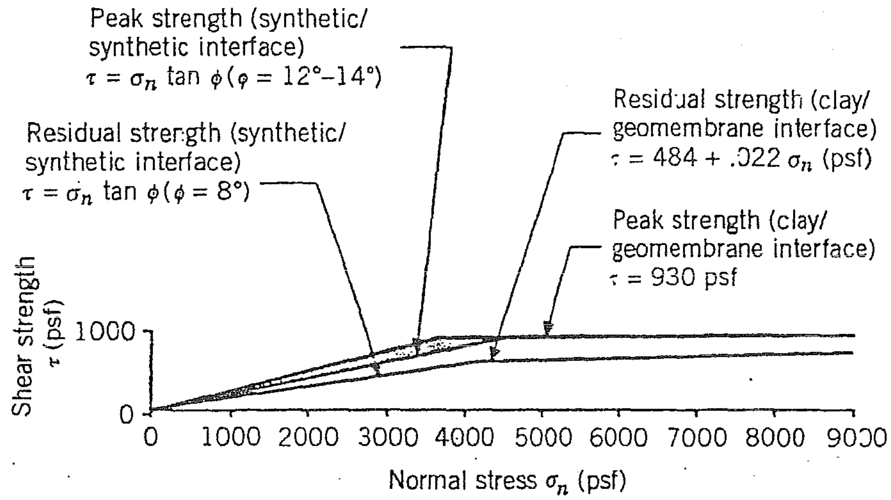


Figure 3.61 Liner strength relations. (From Byrne et al., 1992. Reproduced by permission of ASCE.)

1987; Soil and Material Engineers, 1987; Leach et al., 1987; Koutsourais et al., 1990; Swan et al., 1990; O'Rourke et al., 1990; Mitchell et al., 1990; Ojeshina, 1990; Druschel and O'Rourke, 1991; Somasundaram and Khilnani, 1991; Sharma and Hullings, 1993). The results are highly variable due to the large range of soil types and testing conditions. Both peak and residual values are included within the reported range. Table 3.14 also includes recommended soil geomembrane interface strengths.

As shown in Figure 3.61, the interface strength of clay–geomembrane exhibits a linear shear strength (τ) and normal stress (σ_n) relationship at lower normal stresses. The interface friction angles (δ) reported in Table 3.14 represent this behavior. At higher normal loads, the interface friction angle becomes very low and for all practical purposes τ tends to become independent of σ_n. The authors' experience on various low-plasticity (CL) and high-plasticity (CH) clays tested against both smooth and textured HDPE geomembrane confirms this τ–σ_n behavior. Recommended values presented in Table 3.14 should be used only as a guide in feasibility studies. Tests on site-specific materials and selected geomembranes should be conducted for final design purposes.

3.6.3 Geosynthetic-to-Geosynthetic Shear Strength

Several researchers have tested various geosynthetic-to-geosynthetic interfaces (Martin et al., 1984; Williams and Houlihan, 1986; Koutsourais et al., 1990; Mitchell et al., 1990; Lydick and Zagorski, 1990; Ojeshina, 1990; Somasundaram and Khilnani, 1991). The results of these studies are summarized in Table 3.15. The primary components of interface friction between multiple layers of geosynthetics are sliding between layers and dilation at the geosynthetic surface (Williams and Houlihan, 1986).

TABLE 3.15 Typical Range of Reported Geosynthetic to Geosynthetic Friction Angles (Degrees)

	PVC	HDPE Smooth	HDPE Textured	Geonet
Woven Geotextile	10–28	7–11	9–17	9–18
Nonwoven, needle-punched Geotextile	16–26	8–12	15–33	10–27
Nonwoven, resin/heat-bonded Geotextile	18–21	9–11	15–16	17–21
Geonet	11–24	5–19	7–25	—

The testing conditions may also have a significant effect on results. Mitchell et al. (1990) noted that polishing of geomembrane surfaces by geotextiles reduced interface friction. Also, the orientation of geonet strands can affect the interface strength between geonets and geomembranes (Geotek, 1987; Mitchell et al., 1990). Site-specific tests should therefore be performed using the actual materials and anticipated shear conditions.

3.6.4 Geosynthetic Clay Liner Shear Strength

Limited information is currently available on the internal shear strength of GCLs, due primarily to their relatively short history. The tests that have been performed are also difficult to compare, due to the numerous variations in test conditions. Many of these variations, such as strain rate, normal load, sample size, and consolidation conditions, are similar to the variations experienced when comparing shear strength testing of other geosynthetics. An additional variation of GCLs, however, is the hydrating conditions, including the hydrating liquid. Hydration can occur under free swell, constrained swell, or partially constrained swell, or the sample may be tested unhydrated. Even if hydrated under free-swell conditions, it may be difficult to assess whether full hydration has occurred since the bentonite may be restricted from free swell by the bonded geotextiles. Also, due to the large water absorption of bentonite, most shear strength test results will incorporate some immeasurable pore pressure effects unless the test is performed at extremely low displacement rates.

Table 3.16 presents the results of direct shear testing performed under various hydration conditions. The tests were performed at a strain rate of 9 mm/min and at normal stresses up to 60 kPa. Although these test results provide some information on the internal shear strength of GCLs, it is highly recommended that project specific testing be performed.

since creases in the geomembrane caused by sharp corners may lead to environmental stress cracking.

8.3.3.6 Placement of Soils over Geomembranes. As discussed in Section 8.3.3.2, soil should be "floated" over geomembranes such that a minimum 12 inches of this material exists between the construction equipment and the geomembrane at all times. This minimizes the possibility of geomembrane puncture and impact damage since the effective stress exerted by the construction equipment is reduced and the soil is not dumped on top of the geomembrane.

Soil placement over polyethylene geomembranes should occur in the early morning when there is adequate lighting and the geomembrane is contracted. By midday, wrinkles often develop in polyethylene geomembranes, making soil placement difficult. On days where the temperature exceeds 100°F, the wrinkles can be as large as 1 to 2 feet high. Even in the morning, 6-inch-high wrinkles can easily develop. If it cannot be avoided, soils may be placed over geomembrane wrinkles by placing the soil directly on top of the wrinkle such that it forms two smaller wrinkles. By continuously placing soil directly above the wrinkle, the wrinkle will eventually work itself out. Therefore, if possible, the geomembrane should not be permanently anchored until the soil overlying the geomembrane has been placed. In no situation should the geomembrane wrinkle be allowed to fold over under the weight of the overlying soil. These folds will crease the geomembrane and provide a preferential location for stress cracking and eventual leakage.

Placement of soils over geomembranes on slopes should occur from the bottom of slope upward. This will minimize the stresses on the geomembrane from construction equipment. Soils should be placed over geomembranes as soon as possible following geomembrane installation. This prevents UV degradation of the geomembrane and damage from ongoing construction activities, and also provides for good contact between the geomembrane and underlying material.

8.3.4 Structural Details

8.3.4.1 Anchorage. Anchor trenches are used at the top of side-slope liners to hold installed geosynthetics in place against applied loads and to prevent potential tears caused by wind intrusion beneath the geosynthetics. As shown in Figure 8.19, anchor trenches can generally be classified as flat, rectangular, or V-shaped. Selection of the appropriate anchor trench configuration for any particular site depends on the required holding capacity, access considerations, dimensional constraints, and available construction equipment. Often, a contractor may request that the anchor trench configuration be modified based on the equipment available. All such modifications should be checked and approved by the designer.

The holding capacity of anchor trenches is developed by the applied normal load of the soil placed above the geosynthetics, which creates frictional resistance between the geosynthetics and the underlying soil; there is minimal friction resistance developed between the upper soil and the geosynthetic since the soil above the

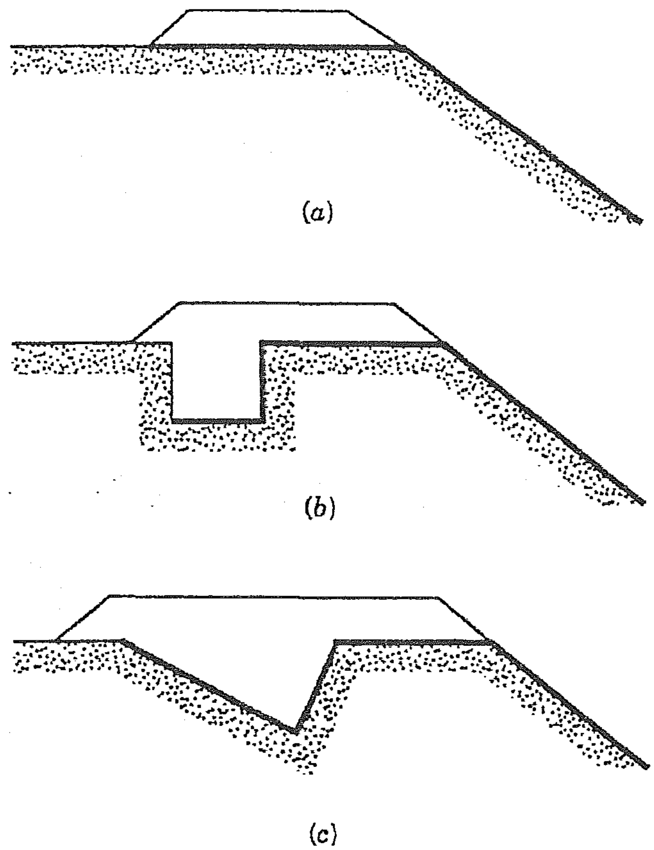


Figure 8.19 Typical anchor trench configurations: *(a)* flat anchor; and *(b)* rectangular anchor; and *(c)* V-shaped anchor.

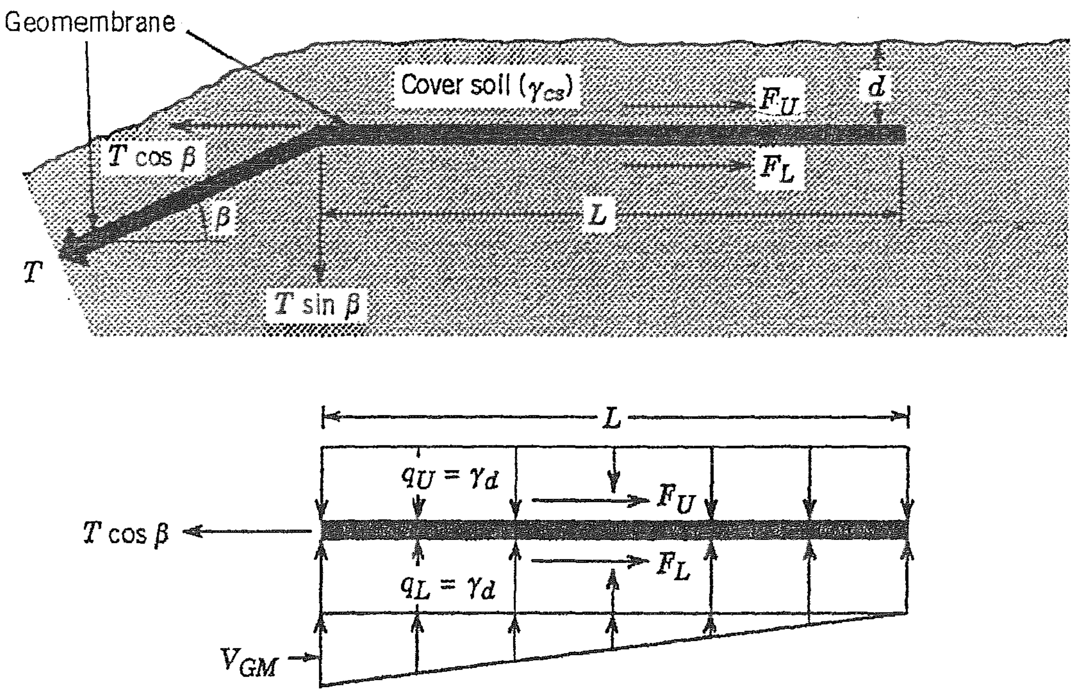
geosynthetic is likely to move with the geosynthetic. The soil depth, type of soil or other material underlying the geosynthetics, and geosynthetic anchorage length are therefore the key factors in developing the required anchor trench holding capacity.

The easiest anchor trench configuration to analyze is the flat anchor. The free-body diagram for the flat anchor and the development of equation (8.14) for anchorage length is shown in Figure 8.20.

$$L = \frac{T \cos \beta - T \sin \beta \tan \delta_L}{\gamma d \tan \delta_L} \tag{8.14}$$

There is no ideal solution for rectangular or V trenches. Koerner (1990) recommends that the problem be solved using imaginary, frictionless pulleys, as shown in Figure 8.21.

The anchor trench should be designed to resist pullout loads (T) caused by the self-weight of the geosynthetics. For geomembranes that may be exposed to severe temperature and wind loading conditions, stresses caused by these forces should also be evaluated. Ideally, the anchor trench should be designed to allow the geosynthetics to pull out slightly rather than cause tearing of the geosynthetics. The reasoning for this is that even if complete pullout occurred, it would usually be easier to replace pulled-out materials than to repair torn geosynthetics. The maxi-



$F_U = q_U \tan \delta_U (L)$ [neglected since cover soil moves with geomembrane]

$F_L = q_L + 0.5\, v_{GM} \tan \delta_L (L)$

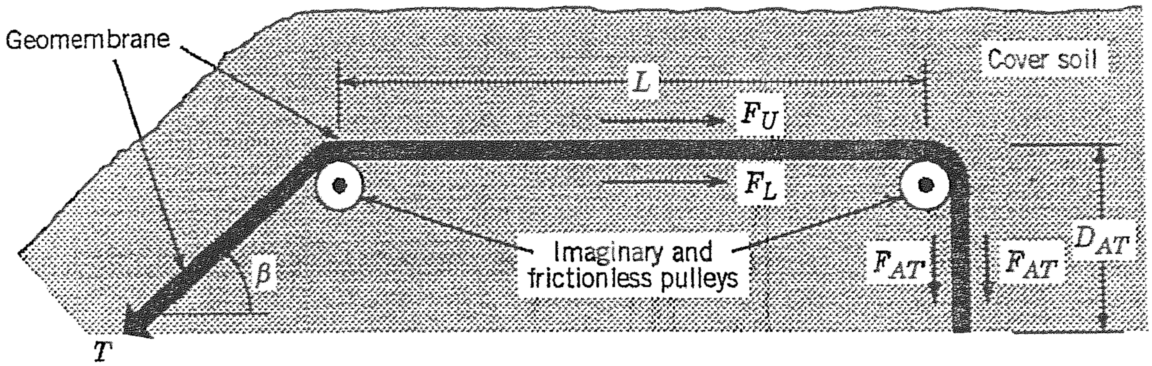
$$= \left[q_U + 0.5 \left(\frac{2\,T \sin\beta}{L} \right) \right] \tan \delta_L (L)$$

$T\cos\beta = q_L \tan \delta_L (L) + T\sin\beta \tan\delta_L$

$$L = \frac{T\cos\beta - T\sin\beta\tan\delta_L}{\gamma_d \tan\delta_L}$$

Where: V_{GM} = vertical force due to geomembrane
F_U = friction force above geomembrane
F_L = friction force below geomembrane
q_U = stress above geomembrane due to cover soil weight
q_L = stress below geomembrane due to cover soil weight
T = tensile force in geomembrane
β = slope angle
d =
= unit weight of cover soil
δ = interface fraction angle

Figure 8.20 Design of a flat anchor. (From Koerner, 1990.)



$T = F_U + F_L + 2F_{AT}$

Where: T = tensile stress in geomembrane

F_U = friction force above geomembrane (assumed to be negligible since cover soil likely moves with geomembrane)

$F_L = q \tan \delta(L)$

q = surcharge pressure = γd

d = depth of cover soil

γ = unit weight of cover soil

δ = interface friction angle

L = runout length

$F_{AT} = (\sigma_h \text{ ave}) \tan \delta (d_{AT})$

σ_h = average horizontal stress in anchor trench

$= k_o \sigma_V$

$\sigma_V = \gamma$ Have

Have = average depth of anchor trench (requires an estimate)

$k_o = 1 - \sin \phi$

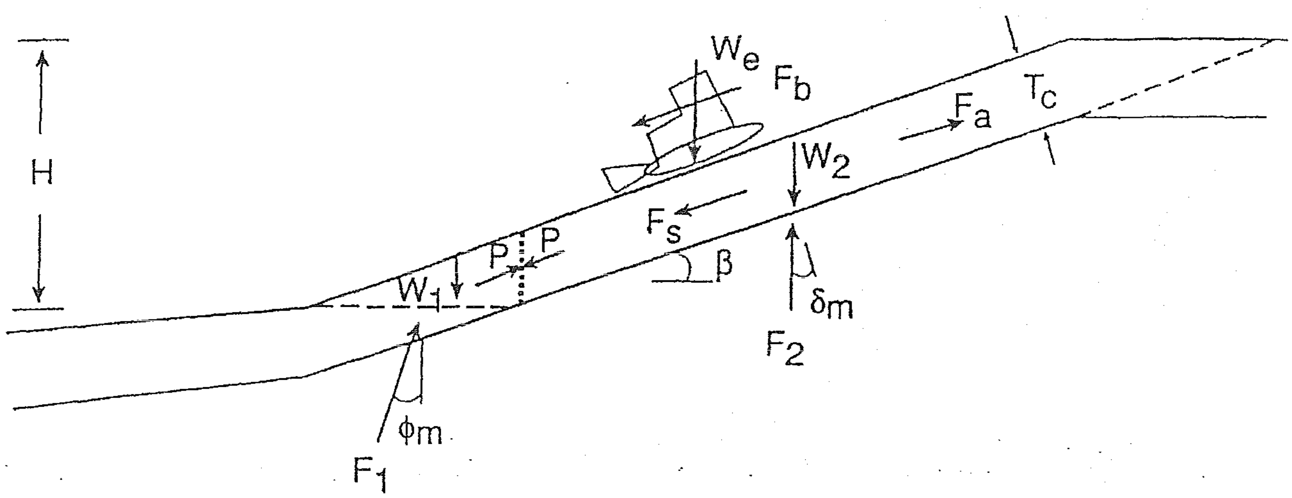
ϕ = angle of shearing resistance of backfill soil

D_{AT} = depth of anchor trench (unknown)

Figure 8.21 Design of a rectangular and V anchor trenches. (From Koerner, 1990.)

mum holding capacity of the anchor trench should therefore be slightly less than the ultimate tensile strength of the geosynthetic to be anchored, irrespective of the applied loads. If the applied loads are greater than the tensile strength of the geosynthetics, measures should be taken to reduce the applied loads or higher-strength geosynthetics should be used.

If soil materials are placed above side-slope geosynthetics, the load caused by soil, seepage forces, and construction equipment should be assessed. Often, a high-strength reinforcing geotextile or geogrid is required to hold the soil on the slopes. Druschel and Underwood (1993) used a force equilibrium method to assess the required anchorage force for these high-strength materials. The free-body and force vector diagram for this method are illustrated in Figures 8.22 and 8.23, respectively. As shown, the items[4] to be evaluated include the toe buttress resistance, soil



Note: P, F_s, F_a, and F_b, are assumed to be parallel to β

Figure 8.22 Free-body diagram of side-slope forces. (From Druschel and Underwood, 1993.)

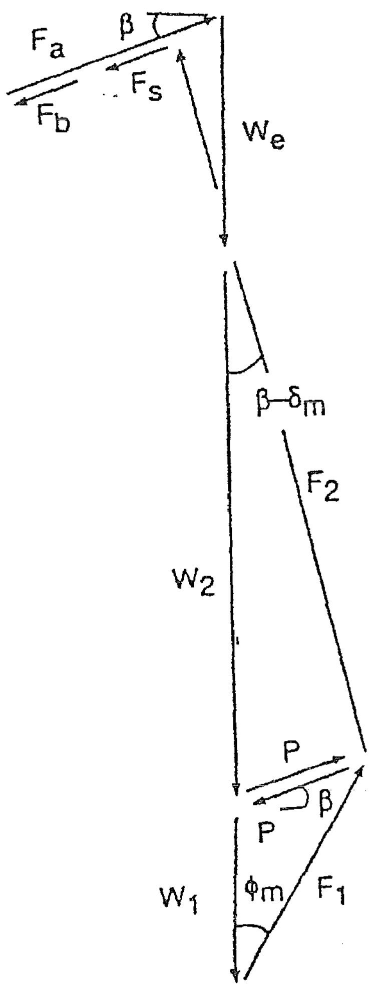


Figure 8.23 Force vector diagram. (From Druschel and Underwood, 1993.)

cover, equipment load, and seepage forces. The equation for the required anchorage force is

$$F_a = \frac{\gamma_w T_w^2}{2 \tan \beta}\left(\frac{\tan \phi_m}{\cos^2\beta} + \frac{2H \tan \delta_m}{\cos \beta} - \frac{\tan \delta_m}{\cos \beta}\right) + W_e\left[0.3 + \frac{\sin(\beta - \delta_m)}{\cos \delta_m}\right] - \frac{\gamma_c T_c^2 \sin(\beta - \delta_m)}{2 \sin \beta \cos \beta \cos \delta_m}\left[\frac{\sin \phi_m \cos \delta_m}{\cos(\beta + \phi_m) \sin(\beta - \delta_m)} + 1 - \frac{2H \cos \beta}{T_c}\right] \tag{8.15}$$

where H = side-slope height
T_c = cover soil thickness
β = side-slope angle
γ_w = unit weight of water
γ_c = unit weight of cover soil
δ = interface friction angle
δ_m = interface friction angle (mobilized)
ϕ = soil shear strength angle
ϕ_m = soil shear strength angle (mobilized)
W_2 = weight of side slope soil
W_1 = weight of toe buttress soil
W_e = weight of equipment on the sideslope (equipment weight divided by equipment width)
F_b = equipment braking force (approximately 30 percent of equipment's weight acting downslope and parallel to interface)
T_w = thickness of seepage
W_{w1} = weight of seepage water in toe buttress
W_{w2} = weight of seepage water in side-slope soil
F_a = geosynthetic anchorage force
F_s = seepage force
F_1 = toe buttress reaction force
F_2 = side-slope reaction force
P = side slope/toe buttress reaction force

Although this equation may seem complex, it is relatively straightforward and easily adaptable to a computer spreadsheet. Figures 8.24 and 8.25 present the variation in anchorage force with slope height assuming an interface friction angle of 9 and 12°, respectively. The reinforcing geotextile or geogrid selected should have a yield strength greater than the required anchorage force and should be able to attain the required anchorage force at a strain level of approximately 2 percent.

[4] Further discussion of these forces is provided in Chapter 10.

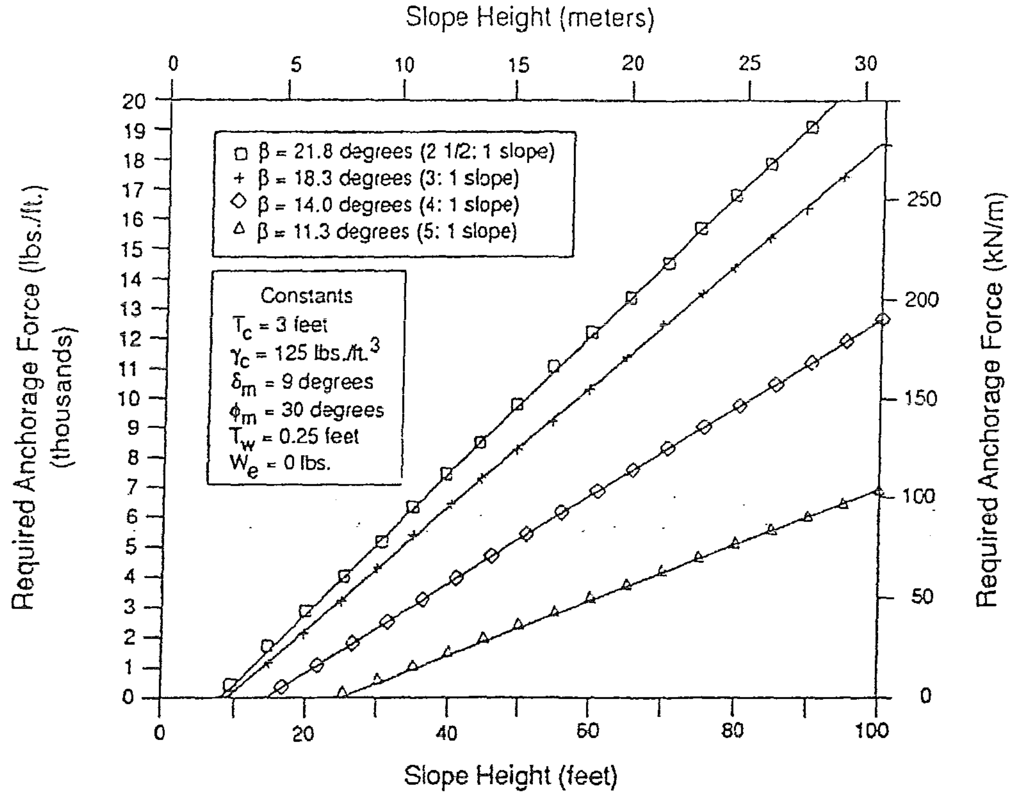


Figure 8.24 Anchorage force required for slope with 9° interface friction angle. (From Druschel and Underwood, 1993.)

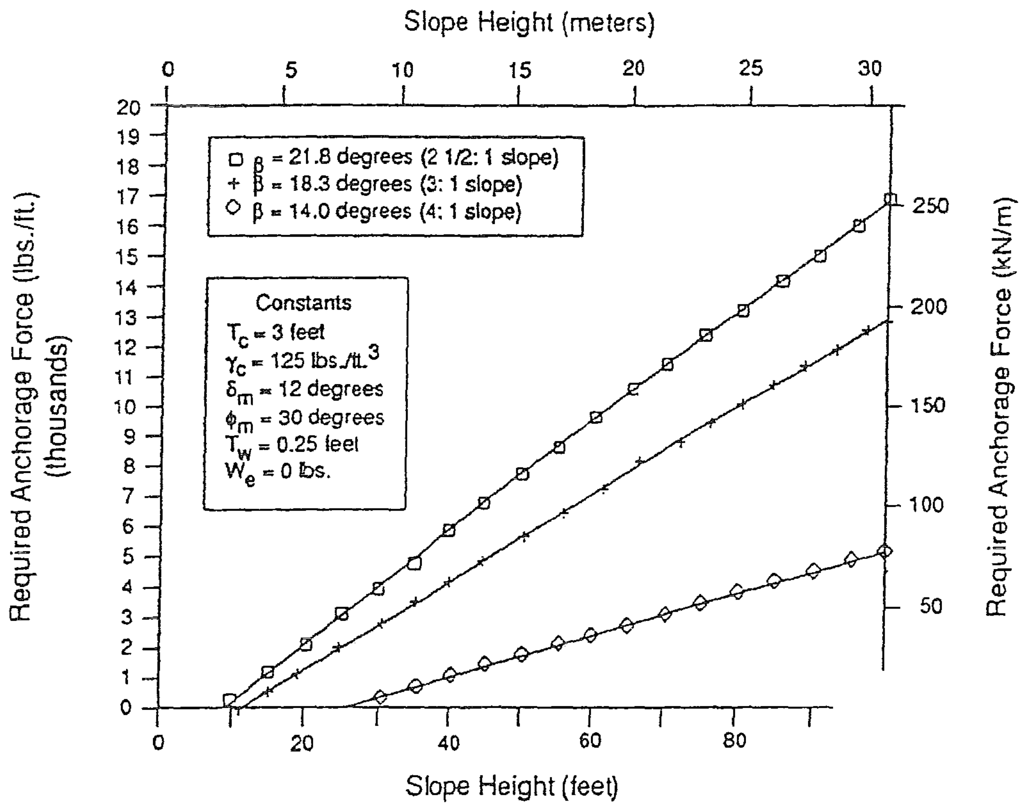


Figure 8.25 Anchorage force required for slope with 12° interface friction angle. (From Druschel and Underwood, 1993.)

Example 8.4. A 50-foot-high 3H:1V side slope is lined with 60-mil single sided textured HDPE (textured side down against underlying clay and smooth side facing up). Calculate various stresses in the liner and determine the anchor trench capacity assuming that it is 3 feet deep and 2 feet wide. At the base, a 3-foot thickness of soil, consisting of a 1-foot drainage layer and a 2-foot-thick operations layer, is already in place.

SOLUTION

A. *Forces on Geomembrane.* The forces on the geomembrane include those due to self-weight, temperature, and wind.

1. *Force (F_w) per foot width due to self-weight (W).*

$$F_w = W \sin\beta - F$$

where

$$W = L\,t\,\gamma = \frac{H}{\sin\beta}\,t\,\gamma$$

and where

$F = W \cos\beta \tan\beta$

H = exposed height of geomembrane = 50 − 3 = 47 ft

$\sin\beta = \sin[\tan^{-1}(1/3)] = \sin 18.3° = 0.314$

$\cos\beta = 18.3° = 0.95$

$t = \text{geomembrane thickness} = \dfrac{60}{1000 \times 12} = 0.005$ ft

$\gamma = \text{unit weight of geomembrane} = SG \cdot \gamma_w = (0.94)(62.4\ \text{lb/ft}^3) = 59\ \text{lb/ft}^3$

Therefore,

$$W = \frac{47}{0.314}(0.005)(59) = 44.1 \text{ lb/ft width}$$

and assuming that $\delta = 15°$ yields

$$F = (44.1)(0.95)(\tan 15°) = 11.23 \text{ lb/ft width}$$

and

$$F_w = 44.1(0.314) - 11.23$$
$$= 2.62 \text{ lb/ft width}$$

2. *Thermal forces (F_t) per foot width due to temperature change (ΔT).* Assume that the coefficient of thermal expansion $\mu = 1 \times 10^{-4}$/°F and the temperature fluctuations of the geomembrane during the day and the night are 120°F and 60°F, respectively. From equation (8.12),

$$\Delta L = \mu\ L\ \Delta T$$

which in terms of thermal strain may be written as

$$\epsilon_t = \mu\ \Delta T$$

Therefore,

$$\epsilon_t = 1 \times 10^{-4} \times (120 - 60) = 6 \times 10^{-3}$$

From the geomembrane stress-strain curve (test data sheet), σ corresponding to $\epsilon_t = 6 \times 10^{-3}$ is ~ 300 psi.

$$F_t = \sigma A = 300 \times 144 \times \frac{0.06}{12} = 216 \text{ lb/ft}$$

3. *Forces (F_{wind}) per foot width due to wind loading.* From equation (8.13)

$$q = 0.002556 V^2$$

Assuming that $V = 50$ miles/h, we have

$$q = 0.002556(50)^2 = 6.39 \text{ lb/ft}^2$$

Assuming that half of this force is supported by the drainage and operations layer and the other half is supported by the anchor trench gives us

$$F_{\text{wind}} = \tfrac{1}{2} qL = (6.39)(\tfrac{1}{2})(149.7) = 478 \text{ lb/ft width}$$

4. *Total design forces (F_d)*

$$\begin{aligned} F_d &= F_w + F_t + F_{\text{wind}} \\ &= 3 + 216 + 478 = 697 \text{ lb/ft width} \end{aligned}$$

B. *Anchor Trench Capacity.* From Figure 8.21.

$$\begin{aligned} T &= F_U + F_L + 2F_{\text{AT}} \\ &= 0 + \gamma d \tan \delta L + 2\sigma_{\text{have}} \tan \delta(d_{\text{AT}}) \end{aligned}$$

Assuming that $d = 3$ ft, $\delta = 15°$, $L = 3$ ft, $\phi = 30°$, $d_{\text{AT}} = 3$ ft yields

$$\sigma_{\text{have}} = k_0\left(\frac{\gamma h}{2}\right) = (1 - \sin\ \phi)\left(\frac{125 \times 3}{2}\right) = 94$$

$T = 125(2) \tan 15(3) + 2(94) \tan 15(3) = 352$ lb/ft width
additional resistance due to backfill soil = $(3+3) \times 2 \times 125$ (tan $20° +$ tan $15°) = 948$ lb/ft
total $T = 352 + 948 = 1300$ lb/ft

C. *Allowable Stress*

Minimum allowable stress at yield = 2000 psi:

$$F_{\text{all}} = \sigma t$$
$$= 2000(0.06) = 120 \text{ lb/in.} = 1440 \text{ lb/ft}$$

D. *Comparison of Various Forces*

$$F_d = \text{design force} = 697 \text{ lb/ft width}$$
$$T = \text{anchor trench capacity} = 1300 \text{ lb/ft width}$$
$$F_{\text{all}} = \text{allowable force} = 1440 \text{ lb/ft width}$$

The anchor trench should be designed to:

- Resist the design force = 697 lb/ft
- Allow the geomembrane to slip out before the allowable stress is reached

Therefore,

$$F_d < T < F_{\text{all}}$$
$$697 < 1300 < 1440 \text{ lb/ft width} \qquad \text{OK}$$
$$\text{FS against pullout} = \frac{T}{F_d} = \frac{1300}{697} = 1.87$$
$$\text{FS against geomembrane failure} = \frac{F_{\text{all}}}{F_d} = \frac{1440}{697} = 2.07$$

8.3.4.2 Connection/Termination. As discussed in Section 8.3.1, most landfill liners are constructed in phases. Adequate liner connection and termination details are therefore critical in maintaining liner continuity between phases. To provide satisfactory connection/termination details, the designer must first envision how the connection will be constructed, the required construction equipment access, and how much overlap is necessary between the lining systems. Typically a 4- to 5-foot overlap is sufficient for the clay liner and 2 to 3 feet for the geosynthetics. To avoid a preferential leachate flow path, the connection between clay liners should not be vertical but rather, stair-stepped at an angle (Figure 8.26). This requires some reworking of the existing clay liners but will lead to a continuous bond between the existing and future clay liners. For future connection of geomembrane liners, the edge of the existing geomembrane liner should be kept as clean as possible for proper seaming. This is often achieved by wrapping the final leading edge of the geomembrane with a nonwoven geotextile prior to placing any cover materials over the geomembrane.

Connection/termination details parallel to landfill sideslopes should also be considered, especially for geomembranes. Often the edge of a geomembrane is left

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

ATTACHMENT III.7.C

QIAN, XUEDE; KOERNER, ROBERT M.; AND GRAY, DONALD H. 2002.

GEOTECHNICAL ASPECTS OF LANDFILL DESIGN AND CONSTRUCTION.

NEW YORK: PRETENCE HALL.

GEOTECHNICAL ASPECTS OF LANDFILL DESIGN AND CONSTRUCTION

Xuede Qian
Geotechnical Engineering Specialist
Michigan Department of Environmental Quality

Robert M. Koerner
H. L. Bowman Professor of Civil Engineering, Drexel University
Director, Geosynthetic Research Institute

Donald H. Gray
Professor of Civil and Environmental Engineering
The University of Michigan

PRENTICE HALL
Upper Saddle River, New Jersey 07458

Solution:
Assume the runout resistance force is equal to the geomembrane allowable tensile force. From the design equations just presented,

$$T\cdot(\cos\beta) = 350(144)(0.030/12)\cos 18.4^\circ$$
$$= 120 \text{ lb/ft } (1.75 \text{ kN/m})$$
$$T\cdot(\sin\beta) = 39.8 \text{ lb/ft } (0.58 \text{ kN/m})$$
$$q_B = \gamma_s\cdot d_{CS} = (100)(1.0) = 100 \text{ lb/ft } (1.46 \text{ kN/m})$$

which, when substituted into Equation 4.11, gives

$$T\cdot(\cos\beta) = q_B\cdot\tan\delta_C(L_{RO}) + T\cdot\sin\beta\cdot\tan\delta_C$$
$$120 = 100(\tan 20^\circ)(L_{RO}) + 39.8(\tan 20^\circ) \quad (4.11)$$

$$120 = 36.4\cdot L_{RO} + 14.5$$

from which it follows that

$$L_{RO} = 2.9 \text{ ft } (0.88 \text{ m}); \text{ use } 3.0 \text{ ft (use 1 m)}$$

Note that the runout lengt\h is strongly dependent on the value of allowable stress used in the analysis. To mobilize the full strength of the geomembrane would require a longer runout length or an anchor trench. However, this might not be desirable. Pullout, without geomembrane failure, might be preferable to tensile rupture and separation of the geomembrane. Thus, the design runout or anchor resistance capacity should fall between the ultimate strength and allowable strength of a geosynthetic liner (Qian, 1995). That is,

$$\text{Ultimate Strength} > \text{Runout and/or Anchor Resistance Capacity} > \text{Allowable Strength}$$
$$\text{Runout and/or Anchor Resistance Capacity} = T/t$$
$$\sigma_{allow} = \sigma_{ult}/FS, \text{ and } T_{allow} = \sigma_{allow}\cdot t,$$

where T = geomembrane tensile force (i.e., runout or anchor resistance force) per unit width;
t = geomembrane thickness;
σ_{ult} = ultimate geomembrane stress (e.g., yield or break);
FS = factor of safety based on geomembrane strength;
σ_{allow} = allowable geomembrane stress; and
T_{allow} = allowable geomembrane force per unit width.

4.7.2 Design of Rectangular Anchor Trench

The situation with a rectangular anchor trench in place at the end of the runout section is illustrated in Figure 4.9. The configuration requires some important assumptions regarding the state of stress within the anchor trench and its resistance mechanism. In order to establish static equilibrium, an imaginary and frictionless pulley is assumed at

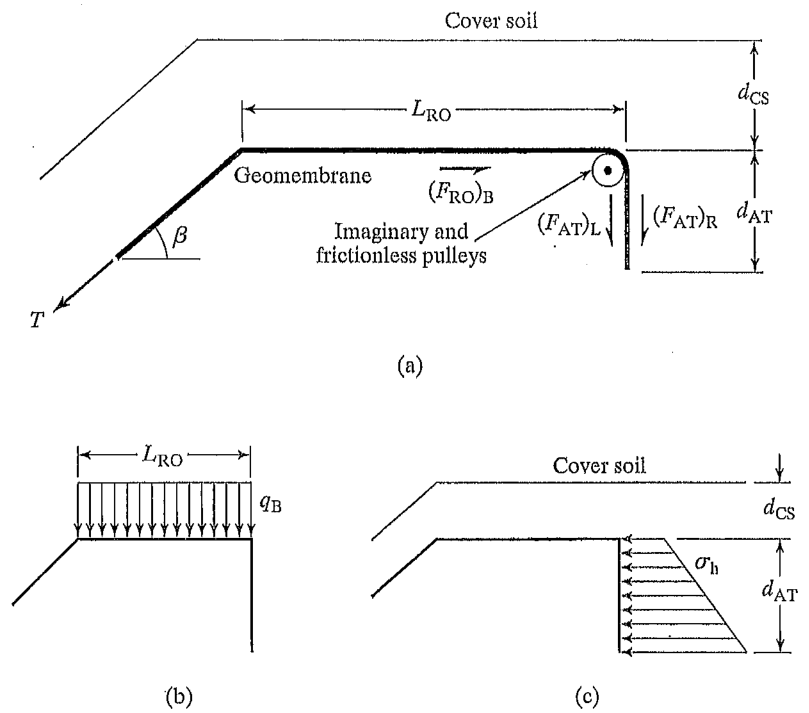


FIGURE 4.9 Cross Section of Geomembrane Runout Section with a Rectangular Anchor Trench and Related Stresses and Forces Involved

the top edge of the anchor trench, as shown in Figure 4.9 (Qian, 1995), which allows the geomembrane to be considered as a continuous member along its entire length.

From Figure 4.9, the following force summations lead to the appropriate design equations:

From $\sum F_V = 0$,

$$T \cdot (\sin\beta) = 0.5 \cdot V_{GM} L_{RO}$$

The cover soil pressure on the runout length is

$$q_B = \gamma_s \cdot d_{CS}$$

The lateral earth force acting on both sides of the geomembrane buried in the anchor trench is

$$P_L = P_R = K_o \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT}$$

The vertical force due to the geomembrane force is

$$V_{GM} = \frac{2 \cdot T \cdot \sin\beta}{L_{RO}}$$

The friction force above the runout geomembrane is always neglected in the anchor trench design, since the cover soil probably moves along with the geomembrane as it deforms.

From $\Sigma F_H = 0$,

$$T \cdot (\cos\beta) = (F_{RO})_B + (F_{AT})_L + (F_{AT})_R \tag{4.13}$$

and
$$\begin{aligned}(F_{RO})_B &= q_B \cdot L_{RO} \cdot \tan\delta_C + 0.5 \cdot V_{GM} \cdot L_{RO} \cdot \tan\delta_C \\ &= q_B \cdot L_{RO} \cdot \tan\delta_C + 0.5 \cdot (2 \cdot T \cdot \sin\beta / L_{RO}) \cdot L_{RO} \cdot \tan\delta_C\end{aligned}$$

or
$$(F_{RO})_B = q_B \cdot L_{RO} \cdot \tan\delta_C + T \cdot \sin\beta \cdot \tan\delta_C \tag{4.14}$$

Because $q_B = \gamma_s \cdot d_{CS}$, the friction force beneath the runout geomembrane is

$$(F_{RO})_B = \gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta_C + T \cdot \sin\beta \cdot \tan\delta_C \tag{4.15}$$

The friction force between the left side of the geomembrane and the side wall of the anchor trench is

$$(F_{AT})_L = (\sigma_h)_{ave} \cdot d_{AT} \cdot \tan\delta_C$$

The friction force between the right side of the geomembrane and the side wall of the anchor trench is

$$(F_{AT})_R = (\sigma_h)_{ave} \cdot d_{AT} \cdot \tan\delta_F$$

where $(\sigma_h)_{ave} = K_o \cdot (\sigma_v)_{ave}$

Because $K_o = 1 - \sin\phi$ and $(\sigma_v)_{ave} = \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT})$

$$(\sigma_h)_{ave} = (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 d_{AT}) \tag{4.16}$$

So
$$(F_{AT})_L = (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} \cdot \tan\delta_C \tag{4.17}$$

and
$$(F_{AT})_R = (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} \cdot \tan\delta_F \tag{4.18}$$

Substituting Equations 4.15, 4.17, and 4.18 into Equation 4.13 gives

$$\begin{aligned}T \cdot (\cos\beta - \sin\beta \cdot \tan\delta_L) = {}& \gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta_C + \\ & (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} \cdot (\tan\delta_C + \tan\delta_F)\end{aligned}$$

which leads to

$$T = \frac{\gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta_C + (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT} \cdot (\tan\delta_C + \tan\delta_F}{\cos\beta - \sin\beta \cdot \tan\delta_C} \tag{4.19}$$

or

$$T = \frac{q_B \cdot L_{RO} \cdot \tan\delta_C + K_o \cdot (\sigma_v)_{ave} \cdot d_{AT} \cdot (\tan\delta_C + \tan\delta_F)}{\cos\beta - \sin\beta \cdot \tan\delta_C} \tag{4.20}$$

When $\delta_C = \delta_F = \delta$, Equation 4.19 becomes

$$T = \frac{\gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta + 2 \cdot (1 - \sin\phi) \cdot \gamma_s + 0.5 \cdot d_{AT} \cdot \tan\delta}{\cos\beta - \sin\beta \cdot \tan\delta} \tag{4.21}$$

and Equation 4.20 becomes

$$T = \frac{q_B \cdot L_{RO} \cdot \tan\delta + 2 \cdot K_o \cdot (\sigma_v)_{ave} \cdot d_{AT} \cdot \tan\delta}{\cos\beta - \sin\beta \cdot \tan\delta} \tag{4.22}$$

where T = geomembrane tensile force (i.e., anchor trench resistance force) per unit width;
$(F_{RO})_B$ = friction force beneath runout geomembrane;
$(F_{AT})_L$ = friction force between the left side of the geomembrane and the side wall of the anchor trench;
$(F_{AT})_R$ = friction force between the right side of the geomembrane and the side wall of the anchor trench;
$(\sigma_h)_{ave}$ = average horizontal stress in anchor trench;
$(\sigma_v)_{ave}$ = average vertical stress in anchor trench;
H_{ave} = average depth of anchor trench;
K_o = coefficient of at-rest earth pressure;
L_{RO} = runout length;
d_{CS} = depth of cover soil;
d_{AT} = anchor trench depth;
γ_s = unit weight of cover and backfill soil;
ϕ = friction angle of backfill soil in anchor trench;
δ_C = friction angle between geomembrane and underlying soil;
δ_F = friction angle between geomembrane and backfill soil;
δ = friction angle between geomembrane and soil; and
β = sideslope angle, measured from horizontal.

Note that because this situation results in one equation with two unknowns, thus a choice of L_{RO} or d_{AT} is necessary to calculate the other.

EXAMPLE 4.4

A 60-mil (1.5-mm) HDPE geomembrane of allowable stress 840 lb/in^2 (5,800 kN/m^2) is placed on a 3(H) to 1(V) sideslope. There is a cover soil of 12 inches (0.3 m) placed over the geomembrane. The unit weight of cover soil and backfill soil in the anchor trench is 110 lb/ft^3 (17.3 kN/m^3). The friction angle between the geomembrane and the underlying soil is 18 degrees, and the friction angle between the geomembrane and the backfill soil in the anchor trench is 22 degrees. The friction of the backfill soil is 30 degrees. Determine the required runout length for a 24-inch-deep (0.6-meter-deep) anchor trench.

Solution:
Assume the anchor resistance force is equal to the geomembrane allowable tensile force. Using the previously developed design equation from Figure 4.9,

$$T \cdot (\cos\beta) = (F_{RO})_B + (F_{AT})_L + (F_{AT})_R \tag{4.13}$$

where $T = T_{allow} = \sigma_{allow} \cdot t$

From Equation 4.19, we have

$$T = \frac{\gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta_C + (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} \cdot (\tan\delta_C + \tan\delta_F)}{\cos\beta - \sin\beta \cdot \tan\delta_C} \tag{4.19}$$

and

$$\sigma_{\text{allow}} \cdot t \cdot (\cos\beta - \sin\beta \cdot \tan\delta_C) = \gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta_C + (1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} \cdot (\tan\delta_C + \tan\delta_F)$$

so that

$$\sigma_{\text{allow}} \cdot t = (840)(144)(0.060)/12 = 605 \text{ lb/ft } (8.83 \text{ kN/m}) \text{ and } (605)[(\cos 18.4°) - (\sin 18.4°)(\tan 18°)] = (110)(1)(\tan 18°)(L_{RO}) + (0.5)(110)(2)(2)(\tan 18° + \tan 22°)$$

or

$$(605)(0.846) = (35.74) \cdot L_{RO} + (220)(0.729) \text{ which yields } 512.83 = (35.74) \cdot L_{RO} + 160.38 \text{ or } L_{RO} = 9.86 \text{ ft } (2.96 \text{ m})$$

Thus, use the runout length $L_{RO} = 10$ ft (3 m).

The geomembrane can also be extended along the trench bottom to increase resistance force, which is called an L-shaped rectangular anchor trench. A typical layout in an L-shaped rectangular anchor trench, which is widely used in landfill projects, is shown in Figure 4.10. In order to establish the static equilibrium equation, two imaginary and frictionless pulleys are assumed at the top edge and the bottom corner of the anchor trench, as shown in Figure 4.10 (Qian, 1995). This assumption again allows the geomembrane to be considered as a continuous member.

The friction force above a runout geomembrane is always neglected in the anchor trench design, since the cover soil probably moves together with the geomembrane as it deforms.

From $\Sigma F_H = 0$

$$T \cdot (\cos\beta) = (F_{RO})_B + (F_{AT})_L + (F_{AT})_R + (F_{AB})_B + (F_{AB})_U \tag{4.23}$$

The friction force between the geomembrane and the underlying soil at the bottom of the anchor trench is

$$(F_{AB})_B = \sigma_{vB} \cdot L_{AT} \cdot \tan\delta_C \tag{4.24}$$

The friction force between the geomembrane and the overlying soil at the bottom of the anchor trench is

$$(F_{AB})_U = \sigma_{vB} \cdot L_{AT} \cdot \tan\delta_F \tag{4.25}$$

Because $\sigma_{vB} = \gamma_s \cdot (d_{CS} + d_{AT})$,

$$(F_{AB})_B = \gamma_s \cdot (d_{CS} + d_{AT}) \cdot L_{AT} \cdot \tan\delta_C \tag{4.26}$$

and

$$(F_{AB})_U = \gamma_s \cdot (d_{CS} + d_{AT}) \cdot L_{AT} \cdot \tan\delta_F \tag{4.27}$$

Substituting Equations 4.15, 4.17, 4.18, 4.26, and 4.27 into Equation 4.23 gives

$$T \cdot (\cos\beta - \sin\beta \cdot \tan\delta_L) = \gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta_C + \gamma_s \cdot (\tan\delta_C + \tan\delta_F) [(1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} + (d_{CS} + d_{AT}) \cdot L_{AT}]$$

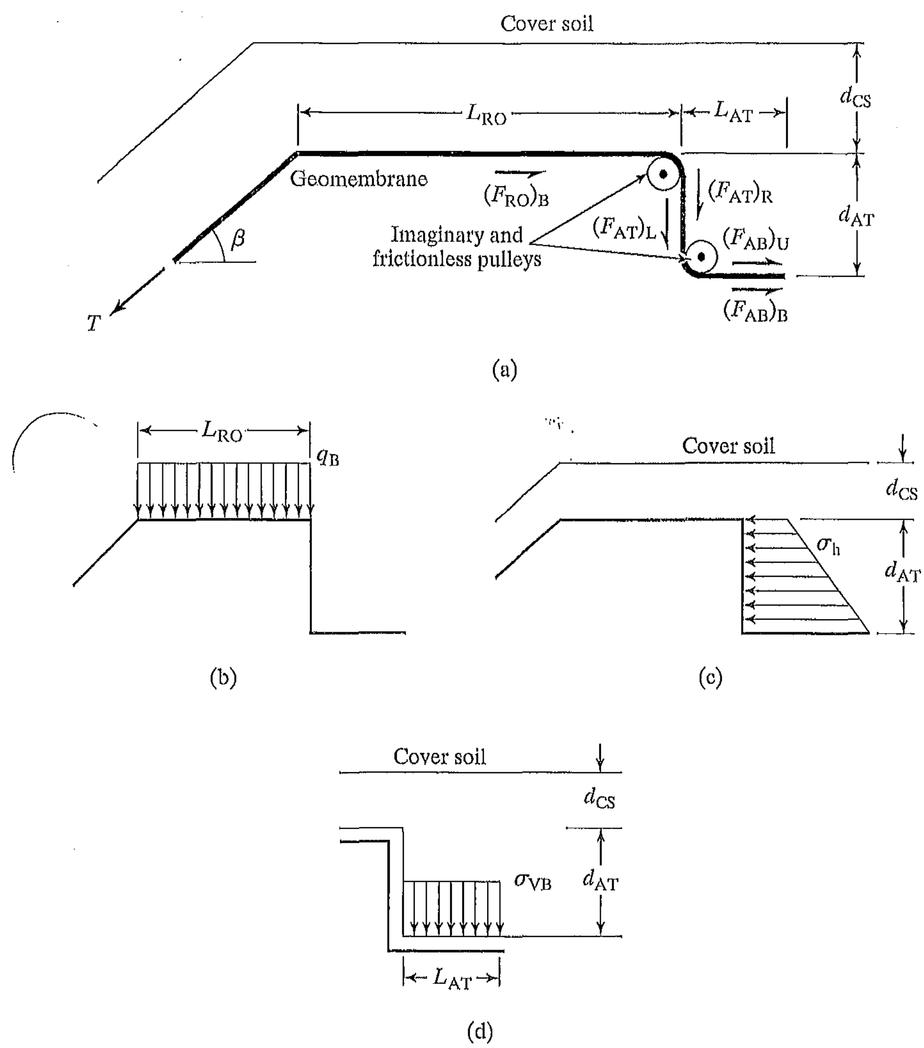


FIGURE 4.10 Cross Section of Geomembrane Runout Section with an L-Shaped Rectangular Anchor Trench and Related Stresses and Forces Involved

~~which leads to~~

$$\frac{{}_s\cdot d_{CS}\cdot L_{RO}\cdot \tan\delta_C + \gamma_s\cdot[(1-\sin\phi)\cdot\gamma_s\cdot(d_{CS}+0.5\cdot d_{AT})\cdot d_{AT} + (d_{CS}+d_{AT})\cdot L_{AT}](\tan\delta_C + \tan\delta_F)}{\cos\beta + \sin\beta\cdot\tan\delta_C} \tag{4.28}$$

or

$$T = \frac{q_B\cdot L_{RO}\cdot\tan\delta_C + [K_o\cdot(\sigma_v)_{ave}\cdot d_{AT} + \sigma_{vB}\cdot L_{AT}](\tan\delta_C + \tan\delta_F)}{\cos\beta - \sin\beta\cdot\tan\delta_C} \tag{4.29}$$

When $\delta_C = \delta_F = \delta$, Equation 4.28 becomes

$$T = \frac{\gamma_s \cdot d_{CS} \cdot L_{RO} \cdot \tan\delta + 2 \cdot \gamma_s \cdot [(1 - \sin\phi) \cdot \gamma_s \cdot (d_{CS} + 0.5 \cdot d_{AT}) \cdot d_{AT} + (d_{CS} + d_{AT}) \cdot L_{AT}] \cdot \tan\delta}{\cos\beta - \sin\beta \cdot \tan\delta} \tag{4.30}$$

and Equation 4.29 becomes

$$T = \frac{q_B \cdot L_{RO} \cdot \tan\delta + 2 \cdot [K_o \cdot (\sigma_v)_{ave} \cdot d_{AT} + \sigma_{vB} \cdot L_{AT}] \cdot \tan\delta}{\cos\beta - \sin\beta \cdot \tan\delta} \tag{4.31}$$

where
T = geomembrane tensile force (i.e., anchor trench resistance force) per unit width;
$(F_{RO})_B$ = friction force beneath runout geomembrane;
$(F_{AT})_L$ = friction force between the left side of the geomembrane and the side wall of the anchor trench;
$(F_{AT})_R$ = friction force between the right side of the geomembrane and the side wall of the anchor trench;
$(F_{AB})_B$ = friction force between the geomembrane and the underlying soil at the bottom of the anchor trench;
$(F_{AB})_U$ = friction force between the geomembrane and the overlying soil at the bottom of the anchor trench;
$(\sigma_v)_{ave}$ = average vertical stress in anchor trench;
K_o = coefficient of at-rest earth pressure;
L_{RO} = runout length;
d_{CS} = depth of cover soil;
d_{AT} = anchor trench depth;
γ_s = unit weight of cover and backfill soil;
ϕ = friction angle of backfill soil in anchor trench;
δ_C = friction angle between the geomembrane and the underlying soil;
δ_F = friction angle between the geomembrane and the backfill soil;
δ = friction angle between the geomembrane and the soil; and
β = sideslope angle, measured from horizontal.

The design of an anchor trench is considered to be adequate if mobilized stress lies between the yield stress and allowable stress of the geosynthetic components. It should be mentioned that many manufacturers specify 1.5-feet- (0.45-m)-deep anchor trenches and a 3.0-feet- (0.90-m)-long runout section.

EXAMPLE 4.5

Calculate the resistant capacity of a given geomembrane in a L-shaped rectangular anchor trench of known dimensions. The geomembrane is 60-mil (1.5-mm) HDPE with an ultimate strength (at yield) 2,100 lb/in^2 (14,500 kN/m^2) and an allowable strength 840 lb/in^2(5,800 kN/m^2).

The runout length is 3 feet (0.9 m). The cover soil is 1 foot (0.3 m). The anchor trench is 2 feet (0.6 m) wide and 2 feet (0.6 m) deep. The side slope angle is 18.4 degrees [3(H):1(V)]. The unit weight of soil is 110 lb/ft³ (17.3 kN/m³). The soil friction angle is 30 degrees. The friction angle between the soil and the geomembrane is 20 degrees.

Solution:
The resistance capacity of the geomembrane in the anchor can be calculated from Equation 4.31 as

$$T = \frac{q_B \cdot L_{RO} \cdot \tan\delta + 2 \cdot [K_o \cdot (\sigma_v)_{ave} \cdot d_{AT} + \sigma_{vB} \cdot L_{AT}] \cdot \tan\delta}{\cos\beta - \sin\beta \cdot \tan\delta}$$

where

$$\begin{aligned}
q_B &= \gamma_s \cdot d_{CS} = 110 \times 1 = 110 \text{ lb/ft}^2 \ (5.27 \text{ kN/m}^2) \\
K_o &= 1 - \sin\phi = 1 - 0.5 = 0.5 \\
(\sigma_v)_{ave} &= \gamma_s \cdot (d_{cs} + 0.5 \cdot d_{AT}) \\
&= 110 \times (1 + 0.5 \times 2) = 110 \times 2 = 220 \text{ lb/ft}^2 \ (10.53 \text{ kN/m}^2) \\
\sigma_{vB} &= \gamma_s \cdot (d_{cs} + d_{AT}) = 110 \times (1 + 2) = 330 \text{ lb/ft}^2 \ (15.80 \text{ kN/m}^2)
\end{aligned}$$

Substituting these calculated values into Equation 4.31 yields

$$\begin{aligned}
T &= \frac{q_B \cdot L_{RO} \cdot \tan\delta + 2 \cdot [K_o \cdot (\sigma_v)_{ave} \cdot d_{AT} + \sigma_{vB} \cdot L_{AT}] \cdot \tan\delta}{\cos\beta - \sin\beta \cdot \tan\delta} \\
&= \frac{(110)(2)(\tan 20°) + 2[(0.5)(220)(2) + (330)(2)](\tan 20°)}{\cos 18.4° - (\sin 18.4°)(\tan 20°)} \\
&= \frac{(110)(2)(0.364) + 2(220 + 660)(0.364)}{0.949 - (0.316)(0.364)} \\
&= \frac{80.08 + 640.64}{0.834} \\
&= \frac{720.72}{0.834} \\
&= 864 \text{ lb/ft } (12.61 \text{ kN/m})
\end{aligned}$$

So,

Anchor Resistance Capacity = 864 lb/ft = 72 lb/in ÷ 0.06 in = 1,200 lb/in² (8,270 kN/m²), which leads to the following inequalities:

Ultimate Strength	>	Anchor Resistance Capacity	>	Allowable Strength
2,100 lb/in²	>	1,200 lb/in²	>	840 lb/in²
(14,500 kN/m²	>	8,270 kN/m²	>	5,800 kN/m²)

The results of the calculation indicate the design anchor resistance capacity falls between the yield stress and allowable stress of a geosynthetic membrane liner. Therefore, the anchor trench dimensions are acceptable.

By using a model as presented here, any set of conditions can be used to analyze and arrive at an acceptable design solution. Even situations in which geotextiles and geonets or geocomposites are used in conjunction with a geomembrane can be analyzed in a similar manner.

be normally consolidated under the surcharge of about 4 m of fill. The soft clay layer, however, was underconsolidated below the fill layer. The excess pore pressures caused by the placement of the fill in the 1970s and 1980s had experienced very little dissipation—particularly between elevations of −10 and −20 m—at the time waste placement started. In the middle zone of the soft clay layer, the difference between the actual undrained strength and the one used in the stability analyses was of the order of 10 kN/m^2. The original short-term stability analysis did not consider the possibility of failure surfaces extending to the river (like the one that actually happened), where there was no fill layer over the soft clay, and, hence, the soft clay did not have the undrained strength assumed in the stability calculations.

As noted, this case history had a geosynthetic lining system that failed along with the rotational movement. However, the lining system could not (and was not) a contributing issue to the failure. The little reinforcement benefit that may have been provided by the geosynthetic layer is negligible in the context of this large of a waste mass. This, as with the previous two case histories, was completely a geotechnical-related failure of the classical rotational failure mode except now a portion of the failure surface passes through waste materials.

13.5.3 General Remarks

It should be obvious from these three case histories that proper site characterization during the design stage and well before waste placement is critical. Irrespective of the high shear strength of waste materials, if the soil foundation fails, it will eventually propagate through the waste mass and cause the entire system to fail. Once a crack is observed on the surface of the waste mass, the entire failure surface beneath it has been mobilized. Failure of the mass is then imminent.

The situation is obviously important when dealing with soft, fine-grained soils. Typically, but certainly not always, such soils are near rivers, harbors, and estuaries. Best available geotechnical practice must be followed (recall Section 13.3.3). Even beyond site investigation, laboratory testing, and design which lead to site-specific plans and specifications, one should consider field instrumentation. Piezometers placed in the subsoil and inclinometers placed at the toe of the waste slope (and beyond) could be most valuable in providing an instantaneous assessment of the landfill as waste is being placed. Unfortunately, such instrumentation is rarely provided, even for sensitive site situations.

13.6 WASTE MASS FAILURES

The relatively low interface shear strengths of components within liner systems can lead to translational failures of the type shown in Figure 13.1(f). However, failure can only occur if the toe of the waste mass is unsupported by an opposing slope or large soil berm. Unfortunately, unsupported toe conditions are often the case. Canyon landfills are very common in areas of mountainous or rolling topography. Even when an excavation is dug for a landfill, the waste mass during filling is generally left unsupported at its toe. This section deals with the instability of such situations.

13.6.1 Translational Failure Analysis

While the approach to translational failures is generally similar to that described in Section 13.5.1, the failure surface is not circular, but usually piecewise linear. Thus, the simplified Bishop method is not applicable. A translational (or two-wedge) failure analysis is used to calculate the factor of safety for the landfill against possible mass movement of the type of "translational (or wedge) failure along liner" [Figure 13.1(f)] in the interim filling condition.

The waste mass shown in Figure 13.24(a) can be divided into two discrete parts, one active wedge lying on the side slope and tending to cause failure, and another passive wedge lying on the cell bottom floor and tending to resist failure. The forces acting on the active and passive wedges are shown in Figure 13.24(a). The individual forces, friction angles, and slope angles involved in the analysis are listed as follows:

W_P = weight of the passive wedge;
N_P = normal force acting on the bottom of the passive wedge;
F_P = frictional force acting on the bottom of the passive wedge (parallel to the bottom of the passive wedge);
E_{HP} = normal force from the active wedge acting on the passive wedge (unknown in magnitude, but with the direction perpendicular to the interface of the active and passive wedges);

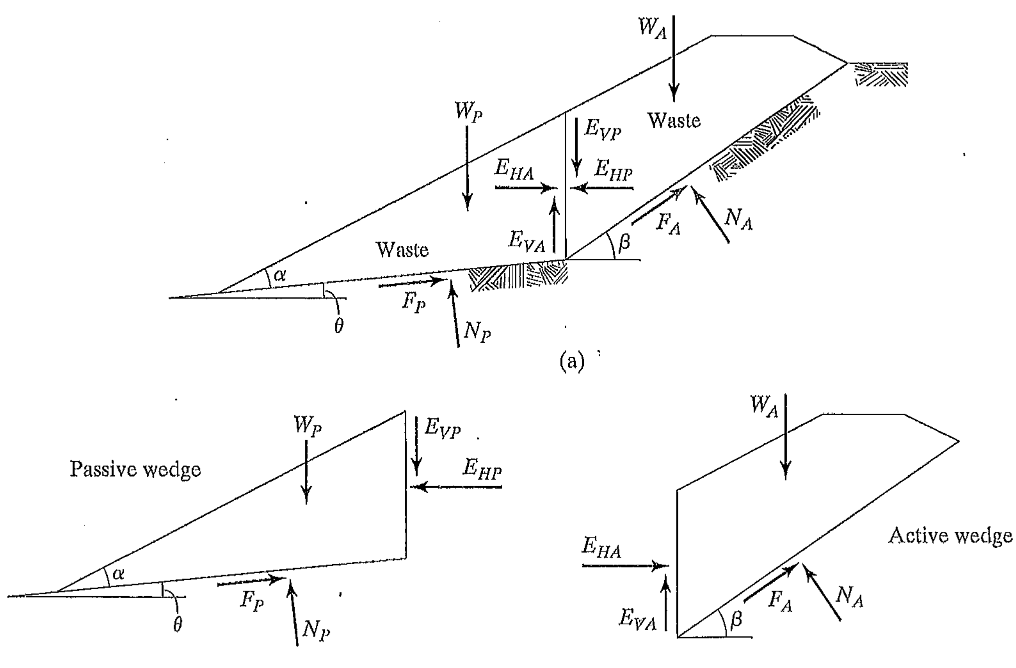


FIGURE 13.24 Forces Acting on Two adjacent Wedges for Solid Waste Filled in Landfill

E_{VP} = frictional force acting on the side of the passive wedge (unknown in magnitude, but with the direction parallel to the interface of the active and passive wedges);

FS_P = factor of safety for the passive wedge;

δ_P = minimum interface friction angle of multi-layer liner components beneath the passive wedge;

ϕ_s = friction angle of the solid waste;

α = angle of the solid waste slope, measured from horizontal, degrees;

θ = angle of the landfill cell subgrade, measured from horizontal, degrees;

W_A = weight of the active wedge;

W_T = total weight of the active and passive wedges;

N_A = normal force acting on the bottom of the active wedge;

F_A = frictional force acting on the bottom of the active wedge (parallel to the bottom of the active wedge);

E_{HA} = normal force from passive wedge acting on the active wedge (unknown in magnitude, but with the direction perpendicular to the interface of the active and passive wedges), $E_{HA} = E_{HP}$;

E_{VA} = frictional force acting on the side of the active wedge (unknown in magnitude, but with the direction parallel to the interface of the active and passive wedges), $E_{VA} = E_{VP}$;

FS_A = factor of safety for the active wedge;

δ_A = minimum interface friction angle of multi-layer liner components beneath the active wedge;

β = angle of the side slope, measured from horizontal, degrees;

FS = factor of safety for the entire solid waste mass.

Considering the force equilibrium of the passive wedge [Figure 13.24(b)], the forces acting on it are

$\Sigma F_Y = 0$:

$$W_P + E_{VP} = N_P \cdot \cos\theta + F_P \cdot \sin\theta \tag{13.47}$$

$$F_P = N_P \cdot \tan\delta_P / FS_P \tag{13.48}$$

$$E_{VP} = E_{HP} \cdot \tan\phi_s / FS_P \tag{13.49}$$

Substituting Equations 13.48 and 13.49 into Equation 13.47 gives

$$W_P + E_{HP} \cdot \tan\phi_s / FS_P = N_P \cdot (\cos\theta + \sin\theta \cdot \tan\delta_P / FS_P), \text{ and} \tag{13.50}$$

when $\Sigma F_X = 0$,

$$F_P \cdot \cos\theta = E_{HP} + N_P \cdot \sin\theta \tag{13.51}$$

Substituting Equation (13.48) into Equation (13.51) gives

$$N_P \cdot \cos\theta \cdot \tan\delta_P / FS_P = E_{HP} + N_P \cdot \sin\theta$$

$$N_P \cdot (\cos\theta \cdot \tan\delta_P / FS_P - \sin\theta) = E_{HP}$$

$$N_P = \frac{E_{HP}}{\cos\theta \cdot \tan\delta_P/FS_P - \sin\theta} \tag{13.52}$$

Substituting Equation 13.52 into Equation 13.50 gives

$$W_P + E_{HP} \cdot \tan\phi_s/FS_P = \frac{E_{HP} \cdot (\cos\theta + \sin\theta \cdot \tan\delta_P/FS_P)}{\cos\theta \cdot \tan\delta_P/FS_P - \sin\theta}$$

$$\begin{aligned} E_{HP} \cdot (\cos\theta + \sin\theta \cdot \tan\delta_P/FS_P) &= W_P \cdot (\cos\theta \cdot \tan\delta_P/FS_P - \sin\theta) \\ &\quad + E_{HP} \cdot (\cos\theta \cdot \tan\delta_P/FS_P - \sin\theta) \cdot \tan\phi_s/FS_P \end{aligned}$$

$$\begin{aligned} E_{HP} \cdot (\cos\theta + \sin\theta \cdot \tan\delta_P/FS_P - \cos\theta \cdot \tan\delta_P \cdot \tan\phi_s/FS_P^2 + \sin\theta \cdot \tan\phi_s/FS_P) \\ = W_P \cdot (\cos\theta \cdot \tan\delta_P/FS_P - \sin\theta) \end{aligned}$$

$$E_{HP} = \frac{W_P \cdot (\cos\theta \cdot \tan\delta_P/FS_P - \sin\theta)}{\cos\theta + (\tan\delta_P + \tan\phi_s) \cdot \sin\theta/FS_P - \cos\theta \cdot \tan\delta_P \cdot \tan\phi_s/FS_P^2} \tag{13.53}$$

Considering the force equilibrium of the active wedge [Figure 13.12(c)] yields

$\Sigma F_Y = 0$:

$$W_A = F_A \cdot \sin\beta + N_A \cdot \cos\beta + E_{VA} \tag{13.54}$$

$$F_A = N_A \cdot \tan\delta_A/FS_A \tag{13.55}$$

$$E_{VA} = E_{HA} \cdot \tan\phi_s/FS_A \tag{13.56}$$

Substituting Equations 13.55 and 13.56 into Equation 13.54 gives

$$W_A = N_A \cdot (\cos\beta + \sin\beta \cdot \tan\delta_A/FS_A) + E_{HA} \cdot \tan\phi_s/FS_A \tag{13.57}$$

$\Sigma F_X = 0$:

$$F_A \cdot \cos\beta + E_{HA} = N_A \cdot \sin\beta \tag{13.58}$$

Substituting Equation 13.55 into Equation 13.58 gives

$$E_{HA} = N_A \cdot (\sin\beta - \cos\beta \cdot \tan\delta_A/FS_A)$$

$$N_A = \frac{E_{HA}}{\sin\beta - \cos\beta \cdot \tan\delta_A/FS_A} \tag{13.59}$$

Substituting Equation 13.59 into Equation 13.57 gives

$$W_A = E_{HA} \cdot \frac{\cos\beta + \sin\beta \cdot \tan\delta_A/FS_A}{\sin\beta - \cos\beta \cdot \tan\delta_A/FS_A} + E_{HA} \cdot \tan\phi_S/FS_A$$

$$E_{HA} \cdot \frac{\cos\beta + \sin\beta \cdot \tan\delta_A/FS_A + \sin\beta \cdot \tan\phi_s/FS_A - \cos\beta \cdot \tan\delta_A \cdot \tan\phi_s/FS_A^2}{\sin\beta - \cos\beta \cdot \tan\delta_A/FS_A} = W_A$$

$$E_{HA} = \frac{W_A \cdot (\sin\beta - \cos\beta \cdot \tan\delta_A/FS_A)}{\cos\beta + (\tan\delta_A + \tan\phi_s) \cdot \sin\beta/FS_A - \cos\beta \cdot \tan\delta_A \cdot \tan\phi_s/FS_A^2} \tag{13.60}$$

Because $E_{HA} = E_{HP}$ and $FS_A = FS_P = FS$, Equation 13.60 must equal Equation 13.53, giving

$$\frac{W_A \cdot (\sin\beta - \cos\beta \cdot \tan\delta_A/FS)}{\cos\beta + (\tan\delta_A + \tan\phi_s) \cdot \sin\beta/FS - \cos\beta \cdot \tan\delta_A \cdot \tan\phi_s/FS^2} = \frac{W_P \cdot (\cos\theta \cdot \tan\delta_P/FS - \sin\theta)}{\cos\theta + (\tan\delta_P + \tan\phi_s) \cdot \sin\theta/FS - \cos\theta \cdot \tan\delta_P \cdot \tan\phi_s/FS^2}$$

$$\begin{aligned}
&W_A \cdot (\sin\beta - \cos\beta \cdot \tan\delta_A/FS)[\cos\theta + (\tan\delta_P + \tan\phi_s) \cdot \sin\theta/FS - \cos\theta \cdot \tan\delta_P \cdot \tan\phi_s/FS^2] \\
&= W_P \cdot (\cos\theta \cdot \tan\delta_P/FS - \sin\theta)[\cos\beta + (\tan\delta_A + \tan\phi_s) \cdot \sin\beta/FS - \cos\beta \cdot \tan\delta_A \cdot \tan\phi_s/FS^2]
\end{aligned}$$

$$\begin{aligned}
&(W_A \cdot \sin\beta - W_A \cdot \cos\beta \cdot \tan\delta_A/FS)[\cos\theta + (\tan\delta_P + \tan\phi_s) \cdot \sin\theta/FS - \cos\theta \cdot \tan\delta_P \cdot \tan\phi_s/FS^2] \\
&= (W_P \cdot \cos\theta \cdot \tan\delta_P/FS - W_P \cdot \sin\theta)[\cos\beta + (\tan\delta_A + \tan\phi_s) \cdot \sin\beta/FS - \cos\beta \cdot \tan\delta_A \cdot \tan\phi_s/FS^2]
\end{aligned}$$

$$\begin{aligned}
&W_A \cdot \sin\beta \cdot \cos\theta + W_A \cdot (\tan\delta_P + \tan\phi_s) \cdot \sin\beta \cdot \sin\theta/FS - W_A \cdot \sin\beta \cdot \cos\theta \cdot \tan\delta_P \cdot \tan\phi_s/FS^2 \\
&- W_A \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A/FS - W_A \cdot (\tan\delta_P + \tan\phi_s) \cdot \cos\beta \cdot \sin\theta \cdot \tan\delta_A/FS^2 \\
&+ W_A \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s/FS^3 = W_P \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_P/FS \\
&+ W_P \cdot (\tan\delta_A + \tan\phi_s) \cdot \sin\beta \cdot \cos\theta \cdot \tan\delta_P/FS^2 - W_P \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s/FS^3 \\
&- W_P \cdot \cos\beta \cdot \sin\theta - W_P \cdot (\tan\delta_A + \tan\phi_s) \cdot \sin\beta \cdot \sin\theta/FS + W_P \cdot \cos\beta \cdot \sin\theta \cdot \tan\delta_A \cdot \tan\phi_s/FS^2
\end{aligned}$$

$$\begin{aligned}
&(W_A \cdot \sin\beta \cdot \cos\theta + W_P \cdot \cos\beta \cdot \sin\theta) \cdot FS^3 + [W_A \cdot (\tan\delta_P + \tan\phi_s) \cdot \sin\beta \cdot \sin\theta \\
&+ W_P \cdot (\tan\delta_P + \tan\phi_s) \cdot \sin\beta \cdot \sin\theta - W_A \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A - W_P \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_P] \cdot FS^2 \\
&- [W_A \cdot (\tan\delta_P + \tan\phi_s) \cdot \cos\beta \cdot \sin\theta \cdot \tan\delta_A + W_P \cdot (\tan\delta_A + \tan\phi_s) \cdot \sin\beta \cdot \cos\theta \cdot \tan\delta_P \\
&+ W_A \cdot \sin\beta \cdot \cos\theta \cdot \tan\delta_P \cdot \tan\phi_S + W_P \cdot \cos\beta \cdot \sin\theta \cdot \tan\delta_A \cdot \tan\phi_s] \cdot FS \\
&+ (W_A \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s + W_P \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s) = 0
\end{aligned}$$

$$\begin{aligned}
&(W_A \cdot \sin\beta \cdot \cos\theta + W_P \cdot \cos\beta \cdot \sin\theta) \cdot FS^3 + [(W_A \cdot \tan\delta_P + W_P \cdot \tan\delta_A + W_T \cdot \tan\phi_s) \cdot \sin\beta \cdot \sin\theta \\
&- (W_A \cdot \tan\delta_A + W_P \cdot \tan\delta_P) \cdot \cos\beta \cdot \cos\theta] \cdot FS^2 - [W_T \cdot \tan\phi_s \cdot (\sin\beta \cdot \cos\theta \cdot \tan\delta_P \\
&+ \cos\beta \cdot \sin\theta \cdot \tan\delta_A) + (W_A \cdot \cos\beta \cdot \sin\theta + W_P \cdot \sin\beta \cdot \cos\theta) \cdot \tan\delta_A \cdot \tan\delta_P] \cdot FS \\
&+ W_T \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s = 0
\end{aligned} \tag{13.61}$$

Equation 13.61 is now solved as follows:

$$a \cdot FS^3 + b \cdot FS^2 + c \cdot FS + d = 0 \tag{13.62}$$

$$a = W_A \cdot \sin\beta \cdot \cos\theta + W_P \cdot \cos\beta \cdot \sin\theta$$

$$\begin{aligned}
b = &(W_A \cdot \tan\delta_P + W_P \cdot \tan\delta_A + W_T \cdot \tan\phi_s) \cdot \sin\beta \cdot \sin\theta \\
&- (W_A \cdot \tan\delta_A + W_P \cdot \tan\delta_P) \cdot \cos\beta \cdot \cos\theta
\end{aligned}$$

$$\begin{aligned}
c = &-[W_T \cdot \tan\phi_s \cdot (\sin\beta \cdot \cos\theta \cdot \tan\delta_P + \cos\beta \cdot \sin\theta \cdot \tan\delta_A) \\
&+ (W_A \cdot \cos\beta \cdot \sin\theta + W_P \cdot \sin\beta \cdot \cos\theta) \cdot \tan\delta_A \cdot \tan\delta_P]
\end{aligned}$$

$$d = W_T \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s$$

When the cell subgrade is very small (i.e., $\theta \approx 0$), $\sin\theta \approx 0$, and $\cos\theta \approx 1$, Equation 13.62 then becomes

$$a \cdot FS^3 + b \cdot FS^2 + c \cdot FS + d = 0 \tag{13.63}$$

where $a = W_A \cdot \sin\beta$

$b = -(W_A \cdot \tan\delta_A + W_P \cdot \tan\delta_P) \cdot \cos\beta$

$$c = -(W_T \cdot \tan\phi_s + W_P \cdot \tan\delta_A) \cdot \sin\beta \cdot \tan\delta_P$$
$$d = W_T \cdot \cos\beta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s$$

In the conventional translational (or two-wedge) failure analysis method, the direction of the resultant force E_P of E_{HP} and E_{VP} (or the resultant force E_A of E_{HA} and E_{VA}), which acts on the interface between the passive wedge and active wedge, is usually assumed to be parallel to waste filling slope. The effect of the waste property of the interface between the active and passive wedges (i.e., shear strength of the waste) on the stability is not considered for this assumption. Actually, the real direction of the resultant force E_A of E_{HA} and E_{VA} (or the direction of the interwedge force) should be calculated as

$$\begin{aligned} \tan\omega &= E_{VP}/E_{HP} \\ &= (E_{HP} \cdot \tan\phi_s/FS)/E_{HP} \\ &= \tan\phi_s/FS \\ \omega &= \tan^{-1}(\tan\phi_s/FS) \end{aligned} \tag{13.64}$$

where ω = inclination angle of the interwedge force (i.e., the resultant force of E_{HP} and E_{VP}), measured from horizontal, degrees;
ϕ_s = friction angle of solid waste;
FS = factor of safety for the entire solid waste mass.

Municipal solid waste usually settles a considerable amount during the filling operation. Review of field settlements from several landfills indicates that municipal solid waste landfills usually settle approximately 15 to 30% of the initial height because of placement and decomposition. The large settlement of the waste fill induces shear stresses in the liner system on the side slope, all of which tends to displace the liner downslope. The large settlement of the waste fill also causes the large deformation of the landfill cover to induce shear stresses in the final cover system. These shear stresses induce shear displacements along specific interfaces in the liner and cover systems that may lead to the mobilization of a residual interface strength. In addition, thermal expansion and contraction of the side slope liner and cover systems during construction and filling may also contribute to the accumulation of shear displacements and the mobilization of a residual interface shear strength in the liner system (Qian, 1994; Stark and Poeppel, 1994).

Earthquake loading can provide permanent displacements along landfill liner interfaces, resulting in a permanent reduction in their available shear resistance following the completion of the dynamic loading. Post-earthquake static stability must therefore be evaluated using shear strengths that are compatible with the shear displacements predicted to be experienced during the earthquake. In areas of high seismicity, this probably implies that the static stability of the final configuration of the landfill should be assured assuming the mobilization of full residual strength conditions (Byrne, 1994).

Landfill stability should be considered not only during construction and operation periods, but also for the duration of the closure period. Land development of closed landfills should be also considered in the future. Thus, the shear strengths (e.g., δ_P, δ_A, and ϕ_s) used in stability analysis must be carefully selected based on actual site-specific conditions.

EXAMPLE 13.8

Calculate the factor of safety for a landfill filling shown in Figure 13.25. Use a translational failure analysis and the following information:

Minimum interface friction angle of bottom liner system, $\delta_P = 20°$;
Minimum interface residual friction angle of side slope liner system, $\delta_A = 14°$;
Friction angle of solid waste, $\phi_s = 33°$;
Waste unit weight $= 10.2\ \text{kN/m}^3$;
Landfill subgrade is 2% [50(H) : 1(V)];
Waste filling slope is 25% [4(H) : 1(V)];
Side slope angle, $\beta = 18.4°$;
Height of side slope is 30 m;
Distance between the top edge of waste and the top edge of side slope is 20 m.

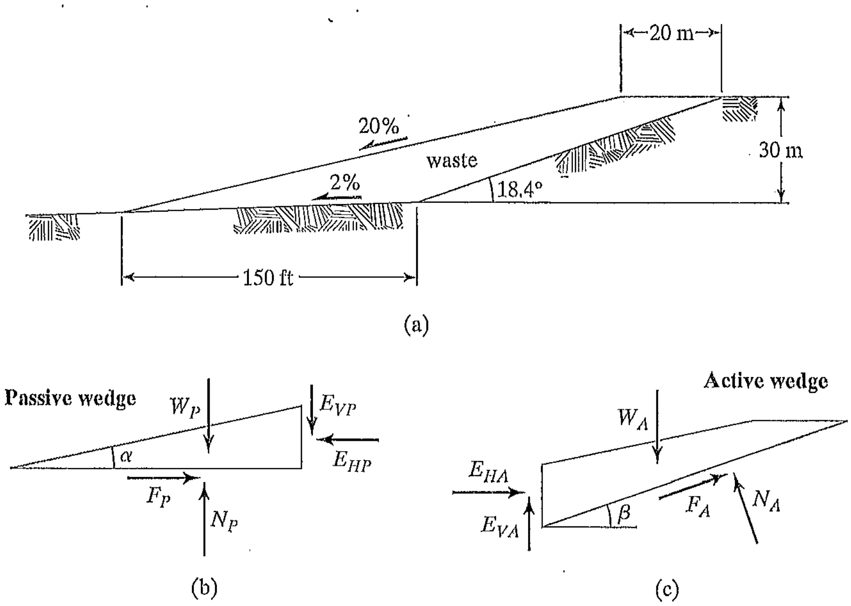


FIGURE 13.25 Cross Section of a Solid Waste Landfill during Filling Condition

Solution The forces acting on the solid waste mass are shown in Figure 13.25. The side slope angle is at 18.4° and the slope angle of cell subgrade is 1.15° according to a 2% slope; hence,

$$\sin\beta = \sin(18.4°) = 0.3162, \cos\beta = \cos(18.4°) = 0.9487,$$
$$\sin\theta = \sin(1.15°) = 0.0200, \cos\theta = \cos(1.15°) = 0.9998$$
$$\tan\delta_A = \tan(14°) = 0.2493, \tan\delta_P = \tan(20°) = 0.3640,$$
$$\tan\phi_s = \tan(33°) = 0.6494.$$

The total weight of solid waste mass is

$$W_T = 10{,}987 \text{ kN/m}$$

The weight of the passive wedge is

$$W_P = 3{,}465 \text{ kN/m}$$

The weight of the active wedge is

$$W_A = W_T - W_P = 10{,}987 - 3{,}465 = 7{,}522 \text{ kN/m}$$

Use Equation 13.62 to calculate FS.

Calculate the coefficients of a, b, c, and d in Equation 13.62:

$$\begin{aligned}
a &= W_A \cdot \sin\beta \cdot \cos\theta + W_P \cdot \cos\beta \cdot \sin\theta \\
&= 7{,}522 \times 0.3162 \times 0.9998 + 3{,}465 \times 0.9487 \times 0.0200 \\
&= 2{,}444 \text{ kN/m} \\
b &= (W_A \cdot \tan\delta_P + W_P \cdot \tan\delta_A + W_T \cdot \tan\phi_s) \cdot \sin\phi \cdot \sin\theta - (W_A \cdot \tan\delta_A + W_P \cdot \tan\delta_P) \cdot \cos\beta \cdot \cos\theta \\
&= (7{,}522 \times 0.3640 + 3.465 \times 0.2493 + 10{,}987 \times 0.6494) \times 0.3162 \times 0.0200 - \\
&\quad (7{,}522 \times 0.2493 + 3{,}465 \times 0.3640 \times 0.9487 \times 0.9998 \\
&= -2{,}907 \text{ kN/m} \\
c &= -[W_T \cdot \tan\phi_s \cdot (\sin\beta \cdot \cos\theta \cdot \tan\delta_P + \cos\beta \cdot \sin\theta \cdot \tan\delta_A) + \\
&\quad (W_A \cdot \cos\beta \cdot \sin\theta \cdot W_P \cdot \sin\beta \cdot \cos\theta) \cdot \tan\delta_A \cdot \tan\delta_P] \\
&= -[10{,}987 \times 0.6494 \times (0.3162 \times 0.9998 \times 0.3640 + 0.9487 \times 0.0200 \times 0.2493) + \\
&\quad (7{,}522 \times 0.9487 \times 0.0200 + 3{,}465 \times 0.3162 \times 0.9998) \times 0.2493 \times 0.3640] \\
&= -967 \text{ kN/m} \\
d &= W_T \cdot \cos\beta \cdot \cos\theta \cdot \tan\delta_A \cdot \tan\delta_P \cdot \tan\phi_s \\
&= 10{,}987 \times 0.9487 \times 0.9998 \times 0.2493 \times 0.3640 \times 0.6494 \\
&= 614 \text{ kN/m}
\end{aligned}$$

$$a \cdot FS^3 + b \cdot FS^2 + c \cdot FS + d = 0 \tag{13.62}$$
$$2{,}444 \cdot FS^3 - 2{,}907 \cdot FS^2 - 967 \cdot FS + 614 = 0$$
$$FS^3 - 1.189 \cdot FS^2 - 0.396 \cdot FS + 0.251 = 0$$
$$FS^3 + 0.251 = 1.189 \cdot FS^2 + 0.396 \cdot FS$$

which is solved by trial and error as in the following table:

Assumed FS	$FS^3 + 0.251$	$1.189 \cdot FS^2 + 0.396 \cdot FS$	Closure
(1)	(2)	(3)	(2) − (3)
1.5	3.626	3.269	0.357
1.4	2.995	2.885	0.110
1.3	2.448	2.524	−0.076
1.35	2.711	2.702	0.009
1.34	2.657	2.666	−0.009
1.345	2.684	2.684	0

Thus, $FS = 1.345$.
The direction of the resultant force of E_{HP} and E_{VP} (i.e., direction of the interwedge force) can be calculated from Equation 13.34 as

$$\begin{aligned} \tan\omega &= \tan\phi_s/FS \\ &= \tan(33°)/1.345 \\ &= 0.649/1.345 \\ &= 0.483 \\ \omega &= 25.8° \end{aligned} \tag{13.64}$$

Recall that the inclination of waste filling slope is 20%, which is only 11.3°. Thus, the direction of the resultant force of E_{HP} and E_{VP} is definitely not parallel to the waste filling slope as is often assumed in these types of calculations (Corps of Engineers, 1960).

13.6.2 Case Histories

Alternatively, for the analysis of the case histories that follow, which failed in a translational manner, the simplified Janbu method was used. (See Koerner and Soong, 2000.) This derivation is also readily available in the literature and leads to a similar equation for the FS-value, but it is now modified with an f_o-value. The resulting equation is

$$FS = (f_o) \cdot \frac{\sum_{i=1}^{n} [c \cdot \Delta b_i + (W_i - u_i \cdot \Delta b_i) \cdot \tan\phi]/m_i}{\sum_{i=1}^{n} W_i \cdot \sin\theta_i} \tag{13.65}$$

where m_i is defined in Equation 13.31, and f_o is a function of the curvature ratio of the failure surface and the type of soil. Since these surfaces are linear, however, the depth-to-length ratio is zero and the value of $f_o = 1.0$. The analysis becomes quite straightforward. (See Schuster and Krizek, 1978.)

To illustrate the seriousness of translational failures (they have represented the largest waste mass failures to date), three case histories are presented next.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

ATTACHMENT III.7.D

CETCO® LINING TECHNOLOGIES, 2009.

BENTOMAT® GCL DIRECT SHEAR DATABASE (TR-114BM)

BENTOMAT® DIRECT SHEAR TESTING SUMMARY

The following table summarizes the direct shear testing on Bentomat that has been performed by CETCO and other laboratories on a project-specific basis for the past several years. This data will give the designer some general information about the shear strength of commonly used GCL interfaces and should be the first step in evaluating a proposed liner system where slope stability is a concern.

The variables in any direct shear test are numerous, including specimen preparation; hydration pressures, liquids, and sequencing, and rate of shear, and others. Test results will vary accordingly, which is partially accountable for the wide range of data reported even for similar interfaces.

This data is for informational purposes only and is not intended to replace project-specific interface testing, which CETCO emphatically recommends. CETCO makes no warranty as to the usefulness of the data. Individual test reports for most of the summarized data can be provided upon request.

TR-114bm
Revised 9/09

800.527.9948 Fax 847.851.1899

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2] GCL	Interface Tested[2] Other	Testing Conditions: Normal Stresses (psi)	Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Mohr-Coulomb Failure Envelopes[6] Peak Angle (deg)	Peak adhesion (psf)	Large Displacement[7] Angle (deg)	Large Displacement[7] adhesion (psf)	Comments[8]
Internal Shear Results														
SGI	Oct-08	200R	Internal		75	200	24	48 hrs @ load	0.04	23 °	0	7 °	0	
SGI	Apr-09	ST	Internal		1.4	200	24	48 hrs @ load	0.004	73 °	0	--	--	sliding at gripping surface
PGL	Feb-08	ST	Internal		1.4	48 hrs @ load			0.004	77 °	0	--	--	sliding at gripping surface
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	27 °	0	7 °	0	
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	31 °	0	8 °	0	
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	38 °	0	9 °	0	
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	31 °	0	7 °	0	
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	42 °	0	9 °	0	
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	34 °	0	7 °	0	
SGI	Jun-06	ST	Internal		34.7	200	24	24 hrs	0.04	26 °	0	7 °	0	
SGI	Oct-06	ST	Internal		34.7	200	24	24 hrs	0.04	37 °	0	8 °	0	
PGL	Feb-03	ST	Internal		5 20 45	432	7 days	48 hrs @ load	0.004	22.7 °	1146	19.3 °	676	
SGI	Aug-01	ST	Internal		10 30 50	24 hrs @ load			0.001	32 °	1645	13 °	160	
SGI	Aug-01	ST	Internal		10 30 50	24 hrs @ load			0.001	39 °	1050	15 °	220	
SGI	Aug-01	ST	Internal		10 30 50	24 hrs @ load			0.001	38 °	1105	17 °	190	
SGI	Apr-09	ST	Internal		75	200	24	48 hrs @ load	0.004	32 °	0	8 °	0	

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2] GCL	Interface Tested[2] Other	Testing Conditions: Normal Stresses (psi)	Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Mohr-Coulomb Failure Envelopes[6] Peak Angle (deg)	Peak adhesion (psf)	Large Displacement[7] Angle (deg)	Large Displacement[7] adhesion (psf)	Comments[8]
SGI	Jan-09	ST	Internal		75	200	24	48 hrs @ load	0.04	32 °	0	7 °	0	
SGI	Feb-08	ST	Internal		75	200	24	48 hrs @ load	0.04	38 °	0	8 °	0	
SGI	Jan-07	ST	Internal		75	200	24	48 hrs @ load	0.004	33 °	0	11 °	0	
SGI	Oct-98	ST	Internal		36 75 145	167	6 days	step-load	0.00006	22 °	1545	6 °	731	
SGI	Jan-09	ST	Internal		150	200	24	48 hrs @ load	0.04	24 °	0	6 °	0	
SGI	Feb-01	ST	Internal		50 100 150	48 hrs @ load			0.04	15 °	1195	8 °	-310	
SGI	Feb-01	ST	Internal		150 250 400	48 hrs @ load			0.04	11 °	2875	5 °	1080	
SGI	Feb-01	ST	Internal		50 to 400 psi	48 hrs @ load			0.04	12 °	2095	6 °	275	
SGI	Apr-09	DN	Internal		1.4	200	24	48 hrs @ load	0.004	75 °	0	--	--	sliding at gripping surface
SGI	Feb-08	DN	Internal		1.4	24 hrs @ load			0.004	77 °	0	--	--	sliding at gripping surface
TRI	Apr-03	DN	Internal		0.7 1.7 3.5	24 hrs @ load			0.04	47.3 °	2813	26.6 °	392	
SGI	Jun-01	DN	Internal		1.0 2.6 6.5	72	120	step-load	0.004	46 °	215	42 °	120	
PGL	Jul-06	DN	Internal		7 21	200	48	24 hrs @ load	0.04	14.5 °	2326	0.5 °	1436	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	34 °	1155	7 °	425	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	33 °	1260	8 °	425	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	35 °	990	8 °	430	

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions					Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses	Hydration.[3]		Consol.[4]	SDR[5]	Peak		Large Displacement[7]		
			GCL	Other	(psi)	psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	32 °	1185	8 °	380	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	35 °	1120	7 °	385	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	33 °	1190	8 °	380	
SGI	Sep-08	DN	Internal		5 25 50	200	24	24 hrs	0.04	34 °	1150	7 °	410	
SGI	Sep-00	DN	Internal		10 25 50	24 hrs @ load			0.001	31 °	1000	12 °	770	GCL peel = 45 lbs
SGI	Sep-00	DN	Internal		10 25 50	24 hrs @ load			0.001	30 °	1155	10 °	170	GCL peel = 27 lbs
SGI	Mar-01	DN	Internal		15 30 60	48 hrs @ load			0.04	24 °	1655	7 °	180	
SGI	Apr-09	DN	Internal		75	200	24	48 hrs @ load	0.004	33 °	0	8 °	0	
SGI	Feb-08	DN	Internal		75	200	24	48 hrs @ load	0.04	40 °	0	8 °	0	
SGI	Jan-07	DN	Internal		75	200	24	48 hrs @ load	0.004	36 °	0	12 °	0	
SGI	Jun-08	DN	Internal		150	200	24	48 hrs @ load	0.04	28 °	0	7 °	0	
SGI	Sep-02	DN	Internal		34.7 150	As-received (21.6%)			0.04	23 °	1715	13 °	1100	
SGI	Apr-09	SDN	Internal		1.4	200	24	48 hrs @ load	0.004	76 °	0	--	--	sliding at gripping surface
GT	Nov-08	SDN	Internal		1.4	200	24	48 hrs @ load	0.004	74 °	0	--	--	
SGI	Aug-09	SDN	Internal		10 30 70	144	48	24 hrs @ load	0.004	34 °	1248	6 °	1020	
SGI	Apr-09	SDN	Internal		75	200	24	48 hrs @ load	0.004	37 °	0	8 °	0	

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions					Mohr-Coulomb Failure Envelopes[6]				Comments[8]
										Peak		Large Displacement[7]		
			GCL	Other	Normal Stresses (psi)	Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	Feb-08	SDN	Internal		75	200	24	48 hrs @ load	0.04	34 °	0	7 °	0	
SGI	Jan-07	SDN	Internal		75	200	24	48 hrs @ load	0.004	36 °	0	12 °	0	
TRI	Apr-08	SDN	Internal		90	24 hrs @ load			0.04	39.1 °	0	15.5 °	0	
SGI	Oct-06	SDN	Internal		5 20 90	115	24	step-load	0.004	22 °	755	5 °	435	
TRI	Oct-07	SDN	Internal		41.7 83.3 125	200	24	step-load	0.04	27.2 °	680	17.1 °	0	
SGI	Jun-03	SDN	Internal		150 250 400	48 hrs @ load			0.04	12 °	1390	5 °	1715	
SGI	Aug-08	STM	Internal		1.4	48 hrs @ load			0.004	73 °	0	--	--	sliding at gripping surface
SGI	Feb-01	CL	Internal		139	144	21 days	168 hrs @ load	0.04	25 °	0	7 °	0	
Interface Shear Results (with geomembranes)														
TRI	May-09	200R		40-mil smooth LLDPE	1 2 4	24 hrs @ load			0.04	11.2 °	4	10 °	4	
SGI	Mar-05	ST	W	60-mil text. HDPE	0.7	48 hrs @ load			0.04	34 °	0	25 °	0	co-extruded textured geomembrane
PGL	Feb-01	ST	white NW	40-mil text. LLDPE	0.35 0.69 1.39	50	24	24 hrs @ load	0.04	29 °	196	16 °	176	co-extruded textured geomembrane
SGI	Dec-08	ST	W	60-mil text. HDPE	2.8	200	24	48 hrs @ load	0.04	40 °	0	26 °	0	co-extruded textured geomembrane
SGI	Apr-07	ST	W	30-mil PVC	1 2 3	100	24	24 hrs @ load	0.04	16 °	5	15 °	5	smooth side
SGI	Apr-07	ST	NW	30-mil PVC	1 2 3	100	24	24 hrs @ load	0.04	14 °	0	14 °	0	smooth side
SGI	Jan-06	ST	W	30-mil PVC	1 2 3	200	48	--	0.04	15 °	5	15 °	0	faille side
SGI	Jan-96	ST	W	30-mil PVC	2 4 6	24 hrs @ load			0.04	17 °	24	17 °	24	

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2] GCL	Interface Tested[2] Other	Testing Conditions: Normal Stresses (psi)			Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Mohr-Coulomb Failure Envelopes[6] Peak Angle (deg)	Peak adhesion (psf)	Large Displacement[7] Angle (deg)	Large Displacement[7] adhesion (psf)	Comments[8]
PGL	Jun-01	ST	NW	40-mil text. LLDPE	1.3	2.6	6.3	72	72	step-load	0.001	24.8 °	230	18.9 °	203	embossed textured geomembrane
TRI	Apr-08	ST		60-mil text. HDPE	0.7	3.5	6.9	100	24	step-load	0.04	23.9 °	107	16.4 °	62	co-extruded textured geomembrane
TRI	Feb-06	ST	NW	60-mil text. HDPE	1.7	3.5	6.9	200	48	24 hrs @ load	0.04	26.7 °	0	23.9 °	0	co-extruded textured geomembrane
TRI	Sep-05	ST	NW	60-mil text. HDPE	2	5	10	24 hrs @ load			0.04	33.8 °	223	20.2 °	181	embossed textured geomembrane
TRI	Aug-06	ST	W	60-mil text. HDPE	3.5	6.9	13.9	48 hrs @ load			0.04	28 °	50	23.6 °	9	embossed textured geomembrane
TRI	Aug-09	ST	W	60-mil text. HDPE	6.9	13.9	20.8	24 hrs @ load			0.04	21.5 °	291	15.1 °	129	embossed textured geomembrane
PGL	Feb-03	ST	W	80-mil text. HDPE	5	20	45	432	7 days	48 hrs @ load	0.004	22.5 °	83	13.6 °	130	embossed textured geomembrane
PGL	Mar-06	ST	W	80-mil text. HDPE	5	20	45	432	7 days	48 hrs @ load	0.004	20 °	379	13.3 °	413	embossed textured geomembrane
PGL	Mar-07	ST	NW	60-mil text. HDPE	13.9	27.8	55.6	500	6 days	24 hrs @ load	0.04	18.1 °	70.5	12.2 °	222.5	
EMCON	Jun-05	ST	NW	60-mil text. HDPE	13.9	34.7	69.4	300	48	24 hrs @ load	0.04	20.6 °	426	8.1 °	738	embossed textured geomembrane
SGI	Jun-09	ST	W	60-mil text. HDPE		75		200	24	48 hrs @ load	0.04	24 °	0	10 °	0	co-extruded textured geomembrane
SGI	Jun-09	ST	W	60-mil text. HDPE		75		200	24	48 hrs @ load	0.04	23 °	0	11 °	0	co-extruded textured geomembrane
SGI	Dec-08	ST	W	60-mil text. HDPE		75		200	24	48 hrs @ load	0.04	22 °	0	11 °	0	co-extruded textured geomembrane
EMCON	Jul-05	ST	NW	60-mil text. HDPE	13.9	55.6	83.3	300	48	24 hrs @ load	0.04	17.8 °	404.9	6.4 °	463.6	
JLT	Oct-04	ST		60-mil text. HDPE	20	45	90	108	3 days	step-load	0.001	24.3 °	323	15.3 °	243	co-extruded textured geomembrane
TRI	Apr-08	ST		60-mil text. HDPE	6.9	69.4	139	100	24	step-load	0.04	18.9 °	0	7.6 °	192	co-extruded textured geomembrane

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions							Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)			Hydration.[3]		Consol.[4]	SDR[5]	Peak		Large Displacement[7]		
			GCL	Other				psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	2003	ST	W	60-mil text. HDPE	139			Hydrated			0.04	14.5 °	0	10.1 °	0	Encapsulated design
SGI	Sep-09	ST	NW	60-mil text. HDPE	13.9	139		200	24	step-load	0.04	21 °	550	8 °	590	co-extruded textured geomembrane
SGI	Sep-09	ST	W	60-mil text. HDPE	13.9	139		200	24	step-load	0.04	18 °	575	8 °	385	co-extruded textured geomembrane
VE	Jun-09	ST		60-mil text. LLDPE	39	78	156	96 hrs @ load			0.04	18 °	990	4 °	1600	embossed textured geomembrane
SGI	2003	ST	W	60-mil text. HDPE	208			Hydrated			0.04	13.7 °	0	9.8 °	0	Encapsulated design
GA	Oct-08	ST		60-mil smooth LLDPE	75	150	300	24 hrs @ load			0.04	15 °	662	4.2 °	3355	GCL internal failure @ 300 psi
SGI	Mar-09	DN	white NW	60-mil text. HDPE	1	2	3	240	48	24 hrs @ load	0.04	33 °	65	27 °	30	embossed textured geomembrane
SGI	Mar-09	DN	white NW	60-mil text. HDPE	1	2	3	240	48	24 hrs @ load	0.04	36 °	50	26 °	45	embossed textured geomembrane
SGI	Mar-09	DN	white NW	60-mil text. HDPE	1	2	3	240	48	24 hrs @ load	0.04	35 °	60	27 °	40	embossed textured geomembrane
SGI	Jan-06	DN	black NW	30-mil PVC	1	2	3	200	48	--	0.04	15 °	0	15 °	0	faille side
PGL	Jun-01	DN	black NW	Textured HDPE	1.3	2.6	6.3	216	72	step-load	0.001	21.4 °	225	18.5 °	184	2-inch displacement
PGL	Jun-01	DN	black NW	Textured HDPE	1.3	2.6	6.3	72	72	step-load	0.001	24.8 °	230	18.9 °	203	2-inch displacement
SGI	May-01	DN	black NW	40-mil text. LLDPE	1	2.6	6.5	72	120	step-load	0.004	32 °	5	28 °	5	embossed textured geomembrane
PGL	Mar-08	DN	white NW	60-mil text. HDPE	5	7	9	24 hrs @ load			0.04	22.5 °	309	22.5 °	305	
EMCON	May-03	DN		Textured HDPE	13.9			Partially hydrated b/w 2 GMs with 0.3" holes			0.04	18.8 °	0	14.3 °	0	Encapsulated b/w GMs with 0.3" holes
GT	Aug-07	DN	black NW	60-mil text. HDPE	18			Hydrated			0.04	26.6 °	0	18.5 °	0	embossed textured geomembrane

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions							Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)			Hydration.[3]		Consol.[4]	SDR[5]	Peak		Large Displacement[7]		
			GCL	Other				psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	Feb-00	DN	black NW	60-mil text. HDPE	7	14	35	72	72	step-load	0.0016	29 °	370	18 °	375	
PGL	Jul-05	DN		60-mil text. HDPE	13.9	27.8	41.7	48 hrs @ load			0.04	17.2 °	359	15.4 °	275	
SGI	Jul-03	DN	black NW	60-mil text. HDPE	10.4	20.8	41.7	48 hrs @ load			0.04	27 °	60	18 °	25	co-extruded textured geomembrane
SGI	Feb-08	DN	black NW	60-mil text. HDPE	15	30	50	1440	48	24 hrs @ load	0.04	27 °	530	16 °	390	co-extruded textured geomembrane
PGL	Jan-05	DN	white NW	80-mil text. HDPE	15	30	50	1440	48	24 hrs @ load	1	17.2 °	151	8.5 °	303	
PGL	Feb-07	DN		60-mil text. HDPE	10	30	60	24 hrs @ load			0.02	24 °	254	22.6 °	65	
PGL	Dec-06	DN		60-mil text. HDPE	10	30	60	24 hrs @ load			0.02	19.2 °	155	15.5 °	147	
PGL	Dec-06	DN		60-mil text. HDPE	10	30	60	24 hrs @ load			0.02	18.5 °	342	18.6 °	108	
SGI	Jul-02	DN	white NW	60-mil text. HDPE	6.9	34.7	69.4	125	24	48 hrs @ load	0.04	23 °	520	12 °	380	co-extruded textured geomembrane
SGI	Jun-03	DN		40- and 60-mil textured HDPE	69.4			Partially hydrated b/w 2 GMs with 0.25" holes			0.04	29 °	0	21 °	0	Encapsulated b/w GMs with 0.25" holes
EMCON	Jun-03	DN		Textured HDPE	69.4			Partially hydrated b/w 2 GMs with 0.3" holes			0.04	19.6 °	0	6.5 °	0	Encapsulated b/w GMs with 0.3" holes
SGI	Feb-08	DN	white NW	60-mil text. HDPE	25	50	75	1440	48	24 hrs @ load	0.04	23 °	570	10 °	420	co-extruded textured geomembrane
SGI	Feb-08	DN	white NW	60-mil text. HDPE	25	50	75	1440	48	24 hrs @ load	0.04	28 °	345	13 °	415	co-extruded textured geomembrane
PGL	Mar-08	DN	black NW	60-mil text. HDPE	25	50	75	24 hrs @ load			0.04	23.6 °	0	22.2 °	0	
SGI	Apr-09	DN	black NW	60-mil text. HDPE	75			200	24	step-load	0.04	30 °	0	14 °	0	embossed textured geomembrane
TRI	Oct-07	DN	black NW	60-mil textured HDPE	25	50	75	24 hrs @ load			0.04	22.7 °	52	11.9 °	409	co-extruded textured geomembrane

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2] GCL	Interface Tested[2] Other	Testing Conditions: Normal Stresses (psi)	Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Peak Angle (deg)	Peak adhesion (psf)	Large Displacement[7] Angle (deg)	Large Displacement[7] adhesion (psf)	Comments[8]
TRI	Oct-07	DN	black NW	60-mil textured HDPE	25 50 75	24 hrs @ load			0.04	10.8 °	1516	5.4 °	1194	co-extruded textured geomembrane
TRI	Oct-07	DN	black NW	60-mil textured HDPE	25 50 75	24 hrs @ load			0.04	20.4 °	455	9.6 °	644	co-extruded textured geomembrane
PGL	Mar-07	DN	white NW	60-mil text. LLDPE	25 50 75	24 hrs @ load			0.04	23 °	0	22 °	0	embossed textured geomembrane
PGL	Mar-06	DN	white NW	60-mil text. LLDPE	25 50 75	24 hrs @ load			0.04	20 °	334	8.6 °	1216	embossed textured geomembrane
GA	Mar-02	DN	black NW	80-mil text. LLDPE	20.8 41.7 83.3	288	24	10 minutes	0.04	21.7 °	789	11.7 °	559	co-extruded textured geomembrane
GA	Mar-02	DN	black NW	60-mil text. LLDPE	20.8 41.7 83.3	288	24	10 minutes	0.04	21.5 °	361	6.7 °	880.5	embossed textured geomembrane
PGL	Apr-07	DN		60-mil text. HDPE	20.8 41.7 83.3	48 hrs @ load			0.04	20.9 °	0	12.3 °	545	
JLT	May-07	DN	black NW	60-mil text. HDPE	20 45 90	115	4 days	step-load	0.005	22.1 °	77	13 °	239	co-extruded textured geomembrane
SGI	May-08	DN	black NW	60-mil text. HDPE	1.4 100	200	24	48 hrs @ load	0.04	24 °	130	12 °	80	co-extruded textured geomembrane
TRI	Jul-08	DN		60-mil text. HDPE	139	144	24	step-load	0.04	22 °	0	10.2 °	0	co-extruded textured geomembrane
VE	May-03	DN		40- and 60-mil text. HDPE	13.9 27.8 55.6 111	250	48	16 hrs @ load	0.04	24 °	260	10 °	650	Encapsulated design
SGI	Jul-09	DN	black NW	60-mil text. HDPE	13.9 27.8 55.6 111	144	24	24 hrs @ load	0.04	22 °	560	11 °	585	co-extruded textured geomembrane
VE	May-03	DN		40- and 60-mil text. HDPE	27.8 111	As-received (25% moisture)			0.04	26 °	0	16 °	140	Encapsulated design
EMCON	Nov-02	DN		60-mil text. HDPE	27.8 55.6 111	48 hrs @ load			0.04	26 °	0	16.8 °	0	co-extruded textured geomembrane
SGI	2003	DN		40- and 80-mil HDPE	5 20 80 120	wetted conditions (not fully hydrated)			0.04	27 °	150	19 °	95	Encapsulated design (slip b/w 80-mil + GCL)
SGI	2003	DN		40- and 80-mil HDPE	5 20 80 120	wetted conditions (not fully hydrated)			0.04	29 °	270	19 °	120	Encapsulated design (slip b/w 80-mil + GCL)

Mohr-Coulomb Failure Envelopes[6]

BENTOMAT GCL DIRECT SHEAR DATABASE
TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions								Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)				Hydration.[3]		Consol.[4]	SDR[5]	Peak		Large Displacement[7]		
			GCL	Other					psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	2003	DN		40- and 80-mil HDPE	5	20	80	120	wetted conditions (not fully hydrated)			0.04	28 °	140	20 °	20	Encapsulated design (slip b/w 80-mil + GCL)
SGI	2003	DN		40- and 80-mil HDPE	5	20	80	120	wetted conditions (not fully hydrated)			0.04	29 °	145	19 °	50	Encapsulated design (slip b/w 80-mil + GCL)
SGI	2003	DN		40- and 80-mil HDPE	5	20	80	120	wetted conditions (not fully hydrated)			0.04	27 °	580	20 °	70	Encapsulated design (slip b/w 80-mil + GCL)
SGI	2003	DN		40- and 80-mil HDPE	5	20	80	120	wetted conditions (not fully hydrated)			0.04	27 °	235	19 °	95	Encapsulated design (slip b/w 80-mil + GCL)
SGI	Jun-08	DN	black NW	60-mil text. HDPE	41.7	83.3	125		24 hrs @ load			0.04	26 °	105	15 °	620	2-inch displacement
SGI	Jun-08	DN	black NW	60-mil text. HDPE	41.7	83.3	125		24 hrs @ load			0.04	25 °	165	13 °	870	2-inch displacement
SGI	Jun-08	DN	black NW	60-mil text. HDPE	41.7	83.3	125		24 hrs @ load			0.04	26 °	110	16 °	485	2-inch displacement
SGI	Jun-08	DN	black NW	60-mil text. HDPE	41.7	83.3	125		24 hrs @ load		24 hrs @ load	0.04	26 °	20	16 °	350	2-inch displacement
SGI	Jun-08	DN	black NW	60-mil text. HDPE	41.7	83.3	125		24 hrs @ load			0.04	26 °	50	15 °	165	2-inch displacement
SGI	Jul-08	DN	black NW	60-mil text. HDPE	125				24 hrs @ load		24 hrs @ load	0.04	25.1 °	0	16.4 °	0	2-inch displacement
SGI	Aug-03	DN	white NW	60-mil text. HDPE	41.7	83.3	125		0	24	48 hrs @ load	0.04	22 °	835	15 °	40	2-inch displacement
SGI	Aug-03	DN	white NW	60-mil text. HDPE	41.7	83.3	125		0	24	48 hrs @ load	0.04	25 °	315	16 °	255	2-inch displacement
TRI	Jun-09	DN		60-mil text. HDPE	20.8	55.6	104	139	125	20	24 hrs @ load	0.04	24.9 °	0	8.7 °	617	embossed textured geomembrane
GTX	Apr-07	DN		HDPE	34.7	69.4	104	139	48 hrs @ load			0.04	26 °	588	12 °	398	
SGI	Feb-00	DN	black NW	60-mil text. HDPE	7 to 150 psi				72	72	step-load	0.0016	22 °	760	11 °	710	
SGI	Oct-02	DN		80-mil text. HDPE	15	25	100	150	1440	48	24 hrs @ load	0.04	23 °	120	14 °	330	co-extruded textured geomembrane

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions: Normal Stresses (psi)				Hydration.[3]		Consol.[4]	SDR[5]	Mohr-Coulomb Failure Envelopes[6]: Peak		Large Displacement[7]		Comments[8]
			GCL	Other					psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	Nov-02	DN		80-mil text. HDPE	25	100	150		As-received (25% moisture)			0.04	24 °	335	18 °	120	co-extruded textured geomembrane
SGI	Feb-00	DN	black NW	60-mil text. HDPE	35	100	150		72	72	step-load	0.0016	21 °	1305	9 °	1105	
GTX	Jul-05	DN	white NW	60-mil text. HDPE	69.4	111	167		24 hrs @ load			0.04	16 °	102	5 °	707	
SGI	Apr-09	DN	black NW	60-mil text. HDPE	75	150	250	400	200	24	step-load	0.04	18 °	2450	5 °	2220	embossed textured geomembrane
SGI	Jul-09	DN	black NW	60-mil text. HDPE	150	250	400		200	24	step-load	0.04	17 °	3705	4 °	3435	GCL internal failure @ 400 psi
TRI	Mar-07	SDN	black NW	40-mil text. LLDPE	0.7	2.8	4.9		100	24	24 hrs @ load	0.04	32.6 °	148	22.5 °	83	embossed textured geomembrane
TRI	Mar-07	SDN	black NW	60-mil text. HDPE	0.7	2.8	4.9		24 hrs @ load			0.04	39.3 °	31	26.7 °	44	embossed textured geomembrane
TRI	Mar-07	SDN	black NW	50-mil text. LLDPE	0.7	2.8	4.9		24 hrs @ load			0.04	44.3 °	97	44.5 °	0	structured GM/Drainage Liner
TRI	Mar-07	SDN	black NW	40-mil text. LLDPE	0.7	2.8	4.9		100	24	24 hrs @ load	0.04	32.6 °	148	22.5 °	83	embossed textured geomembrane
SGI	May-03	SDN	black NW	40-mil text. HDPE	0.7	3.5	6.9		100	24	24 hrs @ load	0.04	30 °	25	19 °	20	co-extruded textured geomembrane
TRI	Jul-08	SDN	Black NW	60-mil text. HDPE	3.5	13.9	31.3	62.5	200	24	step-load	0.04	15.8 °	243	6.5 °	303	co-extruded textured geomembrane
TRI	May-07	SDN		60-mil text. HDPE	6.9	41.7	83.3		250	24	step-load	0.04	23.8 °	467	10.6 °	365	embossed textured geomembrane
SGI	Oct-06	SDN	white NW	60-mil text. HDPE	5	20	90		115	24	step-load	0.04	23 °	695	8 °	425	co-extruded textured geomembrane
PGL	Apr-04	SDN		60-mil text. HDPE	25	60	100		24 hrs @ load			0.04	24.7 °	308	14.1 °	155	
PGL	Sep-04	SDN		60-mil text. HDPE	25	60	100		24 hrs @ load			0.04	22.6 °	0	14.5 °	203	
PGL	Sep-04	SDN		60-mil text. HDPE	25	60	100		24 hrs @ load			0.04	18.9 °	387	15.2 °	333	

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions							Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)			Hydration.[3]		Consol.[4]	SDR[5]	Peak		Large Displacement[7]		
			GCL	Other				psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
PGL	Sep-04	SDN		60-mil text. HDPE	25	60	100	24 hrs @ load			0.04	26.4 °	0	24.1 °	0	
PGL	Sep-04	SDN		60-mil text. HDPE	25	60	100	24 hrs @ load			0.04	22.6 °	0	14.5 °	203	
PGL	Sep-04	SDN		60-mil text. HDPE	25	60	100	24 hrs @ load			0.04	18.9 °	387	15.2 °	333	
PGL	Sep-04	SDN		60-mil text. HDPE	25	60	100	24 hrs @ load			0.04	26.4 °	0	24.1 °	0	
EMCON	Dec-02	SDN	white NW	60-mil text. HDPE	27.8	55.6	111	220	24	24 hrs @ load	0.04	21.2 °	0	11.4 °	0	co-extruded textured geomembrane
TRI	Oct-07	SDN	black NW	60-mil text. HDPE	41.7	83.3	125	200	24	step-load	0.04	22.7 °	0	10.5 °	0	embossed textured geomembrane
GA	Oct-08	SDN		60-mil smooth LLDPE	75	150	300	24 hrs @ load			0.04	18.3 °	662	12.4 °	2246	
SGI	Jun-03	SDN		80-mil text. LLDPE	150	250	400	48 hrs @ load			0.04	11 °	540	7 °	325	co-extruded textured geomembrane
TRI	Jun-07	STM	white NW	60-mil text. LLDPE	100			200	24	step-load	0.04	20.1 °	0	11.5 °	0	co-extruded textured geomembrane
SGI	May-07	STM	white NW	40-mil text. LLDPE	100			200	24	48 hrs @ load	0.04	24 °	0	10 °	0	co-extruded textured geomembrane
SGI	Aug-09	STM	white NW	60-mil text. LLDPE	39	78	156	96 hrs @ load			0.04	21 °	720	9 °	1185	embossed textured geomembrane
Interface Shear Results (with soil)																
ARD	Aug-01	ST	W	SOIL	2.3	3	3.75	24 hrs @ load			0.04	38.7 °	0	38.7 °	0	CIDCO Pit sand
ARD	Aug-01	ST	NW	SOIL	2.3	3	3.75	24 hrs @ load			0.04	36.5 °	0	36.5 °	0	CIDCO Pit sand
ARD	Aug-01	ST	W	SOIL	2.3	3	3.75	24 hrs @ load			0.04	38.1 °	0	38.1 °	0	Michigan Pit sand
ARD	Aug-01	ST	NW	SOIL	2.3	3	3.75	24 hrs @ load			0.04	36.7 °	0	35.6 °	0	Michigan Pit sand
STS	Jan-00	ST	W	SOIL	1	2	4	48 hrs @ load			0.04	28.6 °	293	28 °	241	Topsoil: 62 pcf, 15%

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions								Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)				Hydration.[3]		Consol.[4]	SDR[5]	Peak		Large Displacement[7]		
			GCL	Other					psf	hrs		(in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
TRI	Nov-03	ST	NW	SOIL	1.4	3.6	7.1		24 hrs @ load			0.04	17.7 °	139	18.2 °	135	Soil: 99 pcf, 17%
TRI	Oct-05	ST	W	SOIL	2	5	10		24 hrs @ load			0.04	23.2 °	134	19.9 °	117	Soil: 114 pcf, 14%
TRI	Aug-09	ST	NW	SOIL	7.4	15.4	23.5		24 hrs @ load			0.04	28.1 °	5	25.9 °	0	
PGL	Mar-07	ST	W	SOIL	13.9	27.8	55.6		500	6 days	24 hrs @ load	0.04	21.4 °	279	8.7 °	926	Soil: 110 pcf, 15.2%
TRI	Jul-08	ST	NW	SOIL	3.5	13.9	55.6		24 hrs @ load			0.04	28.7 °	176	16.1 °	474	Soil: 94 pcf, 14.2%
TRI	Nov-06	ST	NW	SOIL	8.1	27.8	55.7		24 hrs @ load			0.04	21.6 °	0	21.6 °	0	Soil: 110 pcf, 12.4%
SGI	Jul-04	ST	W	SOIL	1	20	40	60	24 hrs @ load			0.04	23 °	145	22 °	120	
SGI	Aug-08	ST	NW	SOIL	10	35	60		100	24	24 hrs @ load	0.04	7 °	475	7 °	360	
SGI	Feb-04	ST	W	SOIL	20.8	52.1	79.9		72	7 days	step-load	0.0016	9.9 °	930	6.7 °	500	Clay
SGI	Feb-04	ST	W	SOIL	20.8	52.1	79.9		72	7 days	step-load	0.0016	10 °	1025	7 °	590	Clay
EMCON	Jul-05	ST	W	SOIL	13.9	55.6	83.3		300	48	24 hrs @ load	0.04	15.6 °	561.1	15.6 °	435.8	
NTH	2005	ST	NW	SOIL	25	50	100		144	24	--	0.04	11.9 °	0	7.9 °	0	Clay: 95 pcf, 8%
SGI	Apr-06	ST	W	SOIL	20.8	79.9	139		72	7 days	step-load	0.004	12 °	905	--	--	Clay; GCL internal failure at 139 psi load
JLT	Jan-03	DN		SOIL	0.3	0.7	1.4		24 hrs @ load			0.04	36.3 °	2	29 °	1	Angular gravel
CETCO	Mar-00	DN	black NW	SOIL	0.7	1.4	2.1		24 hrs @ load			0.04	25.2 °	315	--	--	SP, 108 pcf, 11%
GTX	Jul-05	DN	black NW	SOIL	0.7	1.4	2.8		24 hrs @ load		24 hrs @ load	0.04	31 °	60	18 °	27	

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions					Mohr-Coulomb Failure Envelopes[6]				Comments[8]
										Peak		Large Displacement[7]		
					Normal Stresses (psi)	Hydration.[3]		Consol.[4]	SDR[5] (in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
			GCL	Other		psf	hrs							
TRI	Nov-08	DN	white NW	SOIL	0.8 1.6 2.9	24 hrs @ load			0.04	41.1 °	0	28.4 °	29	Soil: 105 pcf, 13.5%
SGI	Nov-08	DN	black NW	SOIL	1 2 3	240	48	24 hrs @ load	0.04	32 °	25	31 °	5	
SGI	Nov-08	DN	black NW	SOIL	1 2 3	240	48	24 hrs @ load	0.04	31 °	25	31 °	5	
TRI	Nov-08	DN	black NW	SOIL	0.7 1.5 3	24 hrs @ load			0.04	18.9 °	70	10.9 °	82	Soil: 105 pcf, 14.1%
PGL	Jun-01	DN	white NW	SOIL	1.3 2.6 6.3	72	72	step-load	0.001	21.2 °	207	21.6 °	184	2-inch displacement; soil: 103 pcf, 17%
PGL	Jun-01	DN	white NW	SOIL	1.3 2.6 6.3	216	72	step-load	0.001	23.2 °	206	20.8 °	194	2-inch displacement; soil: 103 pcf, 17%
SGI	Jun-01	DN	white NW	SOIL	1.0 2.6 6.5	72	120	step-load	0.004	35 °	65	34 °	40	
PGL	Mar-08	DN	black NW	SOIL	5 7 9	24 hrs @ load			0.04	33.6 °	342	33.6 °	337	Soil: 107 pcf, 13.4%
ARD	Oct-05	DN	white NW	SOIL	2 5 9.9	48 hrs @ load			0.04	28.2 °	64	28.4 °	47	Medium to fine silty sand: 117 pcf, 9.5%
ARD	Oct-05	DN	black NW	SOIL	2 5 9.9	48 hrs @ load			0.04	29.3 °	42	29.4 °	38	Medium to fine silty sand: 117 pcf, 9.5%
SGI	Apr-01	DN	black NW	SOIL	1 5 10	48 hrs @ load			0.04	36 °	35	35 °	10	Soil: 124 pcf, 9 %
GT	Aug-07	DN	white NW	SOIL	3 5 10 18	Hydrated			0.04	25.8 °	81	24.3 °	92	Soil: 100 pcf, 19.4%
GT	Aug-07	DN	white NW	SOIL	3 5 10 18	Hydrated			0.04	25.1 °	96	16.1 °	135	Soil: 93 pcf, 20.9%
SGI	Jul-03	DN	white NW	SOIL	10.4 20.8 41.7	48 hrs @ load			0.04	28 °	40	26 °	10	
SGI	Mar-01	DN	white NW	SOIL	55.6	1000	24	24 hrs @ load	0.04	26 °	0	23 °	0	
PGL	Dec-06	DN		SOIL	10 30 60	24 hrs @ load			0.02	32.5 °	491	7.5 °	1319	Soil: 92 pcf, 17.5%

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions				Mohr-Coulomb Failure Envelopes[6]				Comments[8]
									Peak		Large Displacement[7]		
			GCL	Other	Normal Stresses (psi)	Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
PGL	Dec-06	DN		SOIL	10 30 60	24 hrs @ load			0.02	36.9 °	305	23.2 °	751	
PGL	Aug-04	DN	white NW	SOIL	6.9 41.7 69.4	125	20	16 hrs @ load	0.04	28.6 °	312	15.6 °	854	Soil: 120 pcf, 12%
PGL	Aug-04	DN	white NW	SOIL	6.9 34.7 69.4	125	20	16 hrs @ load	0.04	20.8 °	177	17.3 °	190	Soil: 114 pcf, 14.9%
PGL	Aug-04	DN	white NW	SOIL	6.9 41.7 69.4	125	20	16 hrs @ load	0.04	28.6 °	312	15.6 °	854	Soil: 120 pcf, 12%
PGL	Aug-04	DN	white NW	SOIL	6.9 34.7 69.4	125	20	16 hrs @ load	0.04	20.8 °	177	17.3 °	190	Soil: 114 pcf, 14.9%
PGL	Mar-06	DN	black NW	SOIL	25 50 75	24 hrs @ load			0.04	32 °	61	32 °	0	Soil: 109 pcf, 14.9%
PGL	Apr-07	DN		SOIL	20.8 41.7 83.3	48 hrs @ load			0.04	32.2 °	0	31.9 °	0	
PGL	Jul-03	DN		SOIL	3.5 20.8 41.7 83.3	125	24	16 hrs @ load	0.04	22.3 °	320	19 °	322	Soil: 91 pcf, 22%; GCL internal failure at 83 psi
GTX	Apr-07	DN		SOIL	34.7 69.4 104 139	48 hrs @ load			0.04	20 °	1940	-3 °	3247	Brown silty gravel
GTX	Jul-05	DN	black NW	SOIL	69.4 111 167	24 hrs @ load			0.04	11 °	1833	4 °	975	Brown clay with silt: 69 pcf, 45%
OSU	Jan-05	SDN	white NW	SOIL	0.8	Dry			0.04	40.5 °	0	33.2 °	0	Topsoil: 93 pcf, 18%
OSU	Jan-05	SDN	white NW	SOIL	0.8	2 days @ load			0.04	36.1 °	0	25.5 °	0	Topsoil: 93 pcf, 37.8%
OSU	Jan-05	SDN	black NW	FGD	0.8	Dry			0.04	44.8 °	0	41.5 °	0	FGD: 93 pcf, 68.4%
OSU	Jan-05	SDN	black NW	FGD	0.8	2 days @ load			0.04	38.3 °	0	35.3 °	0	FGD: 93 pcf, 68.4%
OSU	Jan-05	SDN	white NW	SOIL	0.8	2 days @ load			0.04	36.3 °	0	14.3 °	0	Topsoil: 93 pcf, 38.2%
JLT	Feb-07	SDN		SOIL	0.7 2.1	12 hrs @ load			0.04	27 °	44	17 °	41	Soil: 116 pcf, 16.4%

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions					Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)	Hydration.[3]		Consol.[4]	SDR[5] (in/min)	Peak		Large Displacement[7]		
			GCL	Other		psf	hrs			Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
SGI	2/205	SDN	black NW	COAL REFUSE	0.7 2.8	144	24	24 hrs @ load	0.04	32 °	40	31 °	40	Coal Refuse
SGI	Jul-06	SDN	black NW	SOIL	0.7 1.4 2.8	24 hrs @ load			0.04	34 °	5	33 °	0	Gravel (34R)
SGI	Jul-06	SDN	white NW	SOIL	0.7 1.4 2.8	24 hrs @ load			0.04	32 °	30	31 °	10	Fine brown sand
TRI	Apr-07	SDN	white NW	SOIL	0.9 3.0 5.2	100	24	24 hrs @ load	0.04	25.3 °	108	23.6 °	117	Soil: 103 pcf, 19.6%
ARD	Jul-03	SDN	white NW	SOIL	2 3.8 5.9	24 hrs @ load			0.04	28.5 °	72	27.7 °	79	Fine brown sand with silt
ARD	Jul-03	SDN	black NW	SOIL	2 3.8 5.9	24 hrs @ load			0.04	33.5 °	43	33.5 °	43	Fine brown sand with silt
TRI	Jul-08	SDN	Black NW	SOIL	3.5 13.9 31.3 62.5	100	24	step-load	0.04	19.3 °	587	19.1 °	561	Soil: 112 pcf, 17%
SGI	2/205	SDN	white NW	SOIL	83	144	24	24 hrs @ load	0.04	27 °	0	22 °	0	Compacted Subgrade
SGI	2/205	SDN	white NW	SOIL	13.9 34.7 55.6 83.3	144	24	24 hrs @ load	0.04	23 °	365	18 °	485	Compacted Subgrade
SGI	Oct-06	SDN	black NW	SOIL	5 20 90	115	24	step-load	0.04	17 °	245	9 °	140	Compacted clay
TRI	May-07	SDN		SOIL	9.3 52.3 91.6	250	24	step-load	0.04	21.6 °	317	6.6 °	1270	Soil: 102 pcf, 12.9%
EMCON	Dec-02	SDN	white NW	SOIL	27.8 55.6 111	220	24	24 hrs @ load	0.04	26.8 °	1320	2.7 °	3140	Sand
TRI	Oct-07	SDN	white NW	SOIL	41.7 83.3 125	200	24	step-load	0.04	28.8 °	0	5.8 °	2935	Soil: 100 pcf, 12.9%
SGI	Feb-02	CL	smooth plastic	SOIL	0.7 1.4 2.8	24 hrs @ load			0.04	20 °	50	20 °	40	Graded Aggregate Base
SGI	Feb-02	CL	smooth plastic	SOIL	0.7 1.4 2.8	24 hrs @ load			0.04	18 °	40	16 °	40	Silty sand
SGI	Feb-02	CL	smooth plastic	SOIL	0.7 1.4 2.8	24 hrs @ load			0.04	19 °	70	18 °	70	Clay

BENTOMAT GCL DIRECT SHEAR DATABASE

TR-114BM

Lab[1]	Report Date	GCL Tested	Interface Tested[2]: GCL	Interface Tested[2]: Other	Testing Conditions: Normal Stresses (psi)	Hydration.[3] psf	Hydration.[3] hrs	Consol.[4]	SDR[5] (in/min)	Mohr-Coulomb Failure Envelopes[6] Peak: Angle (deg)	Peak: adhesion (psf)	Large Displacement[7]: Angle (deg)	Large Displacement[7]: adhesion (psf)	Comments[8]
PGL	Dec-05	CL	smooth plastic	SOIL	0.7 1.4 2.8	Interface sprayed with water			0.04	29.6 °	67	24.4 °	54	Clayey sand: 113 pcf, 14%
PGL	Dec-05	CL	smooth plastic	SOIL	0.7 1.4 2.8	Interface sprayed with water			0.04	37 °	14	33 °	8	Silty sand: 115 pcf, 11.5%
PGL	Dec-05	CL	smooth plastic	SOIL	0.7 1.4 2.8	Interface sprayed with water			0.04	25 °	66	18.5 °	71	CL: 102 pcf, 17.5%
PGL	Dec-05	CL	smooth plastic	SOIL	0.7 1.4 2.8	Interface sprayed with water			0.04	22.9 °	78	21.8 °	49	CH: 92.8 pcf, 22.6%
PGL	Dec-05	CL	smooth plastic	SOIL	0.7 1.4 2.8	Interface sprayed with water			0.04	22.9 °	57	22.5 °	58	SP: 106.5 pcf, 5%
SGI	May-00	CL	W	SOIL	0.5 1.0 2.1	24 hrs @ load			0.04	36 °	10	36 °	10	
CETCO	Mar-00	CLT	W	SOIL	0.7 1.4 2.1	24 hrs @ load			0.04	24.9 °	278	--	--	SP, 108 pcf, 11%
CETCO	Feb-00	CLT	20-mil text. HDPE	SOIL	0.7 1.4 2.1	24 hrs @ load			0.04	41.7 °	108	--	--	SP, 108 pcf, 11%
SGI	Mar-01	CLT	20-mil text. HDPE	SOIL	55.6	1000	24	24 hrs @ load	0.04	24 °	0	21 °	0	
Interface Shear Results (with drainage geocomposites, geonets, and geotextiles)														
GT	Dec-00	ST	W	drainage geocomposite	1.4 2.8 4.2	100	24	--	0.04	25 °	0	20.7 °	1	
EMCON	Jul-05	ST	W	drainage geocomposite	13.9 55.6 83.3	300	48	24 hrs @ load	0.04	19.7 °	0	8.3 °	331	
PGL	Sep-03	ST	NW	drainage geocomposite	5 19.4 60 83.3	144	48	24 hrs @ load	0.04	19.8 °	129	13.6 °	164	
PGL	Jul-06	ST	W	geonet	1.5 3 6	24 hrs @ load			0.04	23.5 °	33.5	23.6 °	29	
GT	Dec-00	DN	black NW	drainage geocomposite	1.4 2.8 4.2	100	24	--	0.04	28 °	0	21.9 °	0	
TRI	Sep-06	DN		drainage geocomposite	1.4 2.8 5.6	24 hrs @ load			0.04	30.1 °	14	27.2 °	0	
PGL	Sep-09	DN	white NW	drainage geocomposite	0.35 2.78 6.94	24 hrs @ load			0.04	21.7 °	96	13.8 °	68	

Lab[1]	Report Date	GCL Tested	Interface Tested[2]		Testing Conditions								Mohr-Coulomb Failure Envelopes[6]				Comments[8]
					Normal Stresses (psi)			Hydration.[3]		Consol.[4]	SDR[5]		Peak		Large Displacement[7]		
			GCL	Other				psf	hrs		(in/min)		Angle (deg)	adhesion (psf)	Angle (deg)	adhesion (psf)	
GTX	Oct-00	DN	black NW	drainage geocomposite	10	30	70	144	72	24 hrs @ load	0.04		22 °	144	18 °	0	
GT	Aug-08	DN	white NW	drainage geocomposite	20.8	41.7	83.3	200	24	48 hrs @ load	0.04		28.7 °	152	16.5 °	515	
PGL	Dec-06	DN		Nonwoven geotextile	2	3.5	5	24 hrs @ load			0.04		20.7 °	160	6.3 °	167	
GT	Dec-04	SDN	black NW	drainage geocomposite	0.7	1.4	2.8	Hydrated			0.04		21.6 °	9	17.2 °	10	
TRI	Jun-07	SDN		drainage geocomposite	6.9	41.7	83.3	250	24	step-load	0.04		21.4 °	0	9.5 °	278	
TRI	Oct-07	SDN	white NW	drainage geocomposite	41.7	83.3	125	200	24	step-load	0.04		27.5 °	0	21.6 °	0	
SGI	Jul-06	SDN	white NW	Nonwoven geotextile	0.7	1.4	2.8	24 hrs @ load			0.04		27 °	35	20 °	20	
TRI	Jun-07	CL	smooth plastic	drainage geocomposite	0.7	1.4	2.8	200	24	24 hrs @ load	0.04		19.2 °	33	10.8 °	46	
ATT	Dec-98	CL	smooth plastic	drainage geocomposite	1	2	3	72	48	--	0.04		14 °	72	11.6 °	72	
SGI	Mar-01	CLT	20-mil text. HDPE	drainage geocomposite	55.6			1000	48	24 hrs @ load	0.04		23 °	0	19 °	0	

Notes:

(1) Laboratories:
ARD = Ardaman and Associates, Orlando FL
ATT = Advanced Terra Testing, inc. Lakewood, CO
CETCO = CETCO, Hoffman Estates, IL
EMCON = Emcon Assoc. (now Shaw Group), Mahwah, NJ
GA = Golder Associates, Atlanta, Georgia
GT = Geotechnics, East Pittsburgh, PA
GTX = Geotesting Express, Boxborough, MA
JLT = J&L Testing, Canonsburg, PA
OSU = Ohio State University, Columbus, OH
PGL= Precision Laboratory, Orange, CA
SGI = SGI Testing Services LLC, Atlanta, GA (formerly GeoSyntec)
STS = STS Consultants, Ltd., Vernon Hills, IL
TRI = TRI Laboratory, Austin, TX
VE = Vector Engineering, Grass Valley, CA

(2) Internal = Failure forced within the GCL (between the geotextiles).
NW = Non-woven geotextile of Bentomat.
W = Woven geotextile of Bentomat.

(3) Hydrated = specimen was soaked under the specified load for the specified duration prior to testing. Hydration methods may vary
Dry = specimen was tested in the as-received moisture (typically 25-30 percent).
Wetted = specimen was partially hydrated.

(4) Consolidation. If the hydration load does not equal the ultimate normal load for shearing, the normal load is increased in steps.

(5) SDR = Shear Displacement Rate.

(6) Mohr-Coulomb failure envelope, $\tau = c_a + \sigma \tan \phi$, determined by a least-squares, "best-fit" straight line through the shear strength-normal stress test results. Two shear strength components are shown: c_a = adhesion and ϕ = friction angle. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered. Refer to TR-264 for discussion of cohesion (or adhesion) and friction angle in direct shear tests.

(7) Measured at 3" displacement, unless otherwise noted.

(8) Including information on: geomembrane type; soil type, density, and moisture content; observed GCL internal failure during interface shearing; and any other unique testing conditions.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

ATTACHMENT III.7.E

KOERNER, ROBERT M. AND KOERNER, GEORGE R. 2007.

INTERPETATION(S) OF LABORATORY GENERATED INTERFACE SHEAR STREGTH DATA FOR GEOSYNTHETIC MATERIALS WITH EMPHISIS ON THE ADHESION VALUE.

GRI WHITE PAPER #11. GEOSYNTHETICS INSTITUTE

Geosynthetic Institute
475 Kedron Avenue
Folsom, PA 19033-1208 USA
TEL (610) 522-8440
FAX (610) 522-8441

GRI White Paper #11

Interpretation(s) of Laboratory Generated Interface Shear Strength Data for Geosynthetic Materials With Emphasis on the Adhesion Value

by

Robert M. Koerner and George R. Koerner
Geosynthetic Institute
475 Kedron Avenue
Folsom, PA 19033 USA

Phone (610) 522-8440
Fax (610) 522-8441

E-mail:
robert.koerner@coe.drexel.edu
gkoerner@dca.net

September 11, 2007

Interpretation(s) of Laboratory Generated Interface Shear Strength Data for Geosynthetic Materials With Emphasis on the Adhesion Value

The beginning point of this White Paper is based on the assumption that a designer has a credible set of laboratory generated shear stress versus shear displacement curves on the desired geosynthetic-to-geosynthetic or geosynthetic-to-soil interface tested per ISO 12957 or ASTM D5321, or ASTM D6243 if geosynthetic clay liners are involved. In this regard we are considering having such data as shown in Figure 1. It is clearly seen that many behavioral trends are possible.

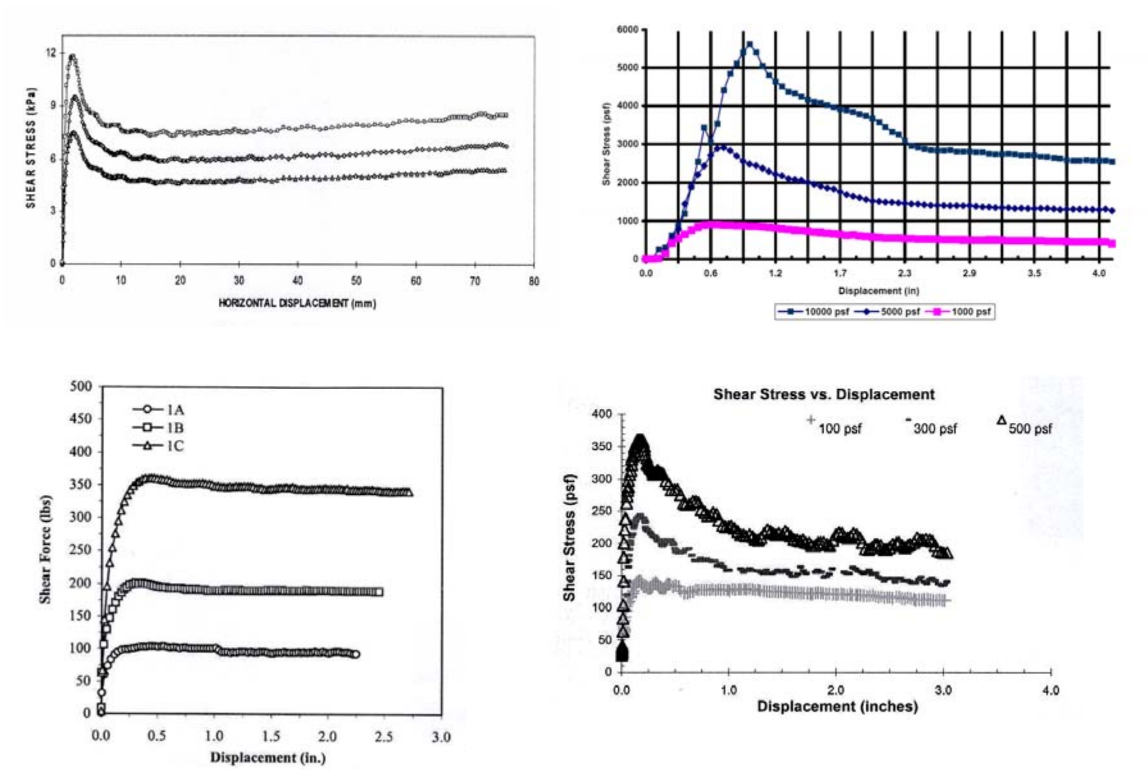


Figure 1 – Various stress versus displacement curves for different geosynthetic materials. (Data compliments of TRI, Golder, Precision and SGI Laboratories)

Either the designer or the testing laboratory will have to generate the Mohr-Coulomb failure envelope from these curves by selecting one point on each normal stress curve and plotting the results on a normal stress versus shear stress curve as shown in Figure 2a. A least squares fit of the data point produces the failure envelope. Even further, one might have more than one such failure envelopes; peak, large displacement and/or residual. Please note, however, that this White Paper is not about the selection of peak, large displacement or residual values and the technical literature is abundant on that subject.

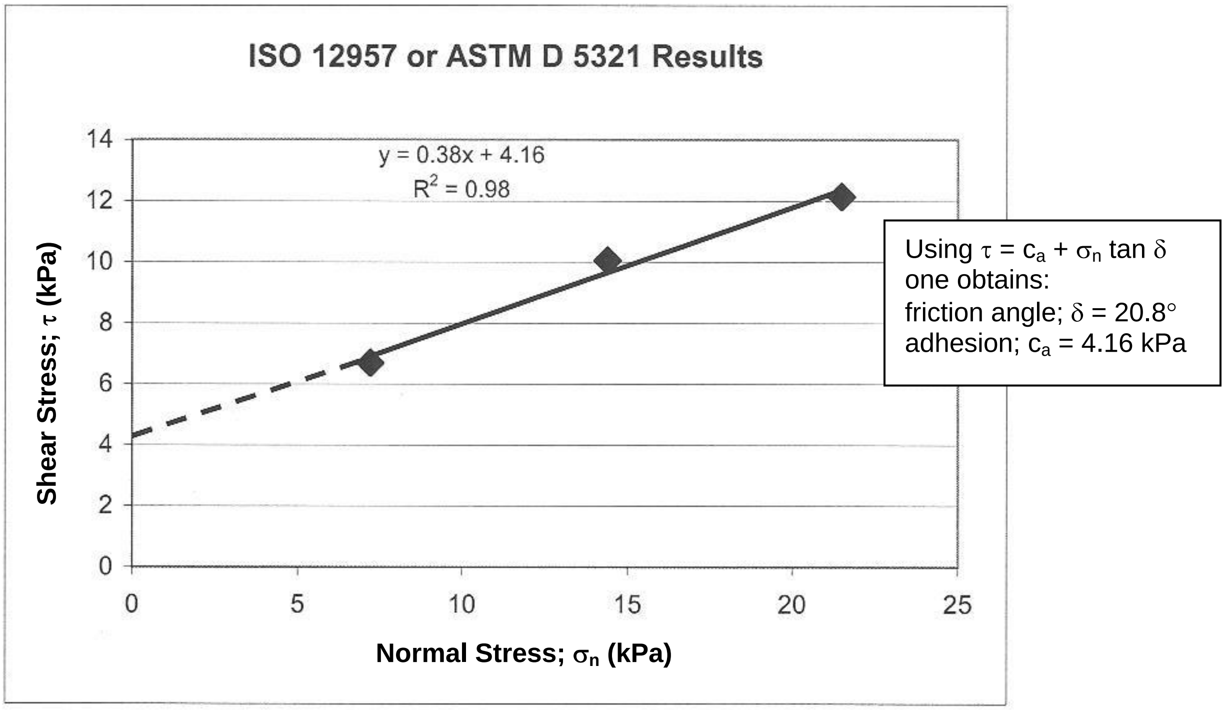


Figure 2a – Three point laboratory data leading to the drawing of a failure envelope and subsequent measurement of friction angle and shear strength intercept (or adhesion) values.

At any rate, to begin the present discussion on the interpretation of the selected failure envelope, the designer is confronted with something like that shown Figure 2a. Here the data points are clearly identified and the failure envelope is usually generated by a least squares fitting procedure. The dashed extension to the y-axis is often the general assumption particularly for low normal stresses as indicated. Note that there are indeed exceptions to this situation such as curved failure envelopes within the normal stress range tested, or zero normal stress tests. They are special cases and will be discussed later.

<u>Interpretation #1 – Use of full “c_a” and full “δ” values</u>

Assuming that the previous failure envelope is based on credible laboratory procedures, properly simulated insofar as representative samples, normal stress selection, moisture conditions, strain rate, etc., our recommended approach is to use the shear strength parameters directly in your slope stability analysis and, if found to be adequate, for your materials specification criteria as well. For landfill cover veneer stability problems all GSI Members and Associate Members should have our spread sheet calculation program which is extremely easy to use. For others, there are many computer codes available. For a hypothetical veneer slope stability example using the two shear strength parameters (c_a and δ) from Figure 2a, the input information is as follows:

- cover soil thickness h = 0.3 m
- slope angle $\beta = 18.4°$ (3-to-1)
- length of slope L = 30.0 m
- unit weight of cover soil $\gamma = 18.0$ kN/m^3
- friction angle of cover soil $\phi = 30.0$ deg
- cohesion of cover soil c = 0.0 kN/m^2
- friction angle of interface $\delta = 20.8$ deg
- adhesion of interface $c_a = 4.16$ kPa (= 87 psf)

By using the program just mentioned or similar procedure, the resulting slope factor-of-safety value is; FS = 3.62. This is a relatively high value and would generally be considered quite conservative. One point worth mentioning, however, is the strong influence of the adhesion value on factor-of-safety. To illustrate this, we now vary the c_a-value between zero and ten while holding everything else the same. This procedure results in the following table; clearly illustrating the sensitivity of the FS-value to this particular parameter.

Adhesion; "c_a"		Resulting FS-value
kPa	lb/ft^2	
0	0	1.18
2	42	2.35
4	84	3.53
6	125	4.70
8	167	5.80
10	209	7.05

Presented now is the heart of this White Paper concerning the *issue of how reliable is this laboratory generated c_a-value?* The ultimate decision is yours as the designer, but our opinions on different geosynthetic materials and related interfaces are as follows:

(a) For textured geomembranes against geotextiles or soil, the asperities (be they manufactured as structured, blown film, or impinged) are on the material giving rise to the high adhesion values, so we recommend using the adhesion value accordingly. Only by continuously rubbing the surfaces against one another can asperity reorientation occur and we feel this is an artifact of aggressive laboratory testing as has been done (and reported) using the ring shear testing device in particular. Alternatively, concern has been expressed when testing at very high normal stresses. The thought in both instances is that if you eliminate adhesion from textured geomembranes you are essentially assuming smooth geomembrane sheet. This is a designer's prerogative, but be prepared to have very gentle slopes in so doing.

(b) For smooth geomembranes against other geosynthetics or soil, a small adhesion is often observed. This is particularly the case for LLDPE, fPP, EPDM, and PVC. Each of these geomembranes are less hard than HDPE, and thus an indentation can be visualized (particularly dealing with soil) which is clearly a function of the applied normal stress. Assuming that the appropriate normal stresses were used in the direct shear test, we feel that one is generally justified in its use.

(c) For geotextiles thermally bonded to geonets or other types of drainage cores, we feel that the full value of adhesion should be used. Most of these geocomposites can barely be "delaminated" in the conducting of the test and we have never heard of a field delamination problem from a properly manufactured geocomposite interface in this regard.

(d) For the internal shear strength of reinforced GCLs, the fibers would have to pull-out or break (or both) for a loss of adhesion. While you can force this to happen in the lab, we have no evidence of this occurring in the field. Test results invariably show high adhesion values. Furthermore, longevity (durability) of the fibers in a hydrated bentonite atmosphere promises 100-year lifetime, or longer. We have a creep-related paper in this regard. Thus, we see no reason not to use the laboratory generated value of adhesion for reinforced GCLs manufactured by either needlepunching or stitching. Of course, the upper and lower interfaces of the GCLs must be independently evaluated.

(e) For certain geosynthetic-to-soil interfaces, the interface shear behavior may force the failure plane into the soil. This results in the identification of the soil's shear strength and if there is a shear strength intercept it is a cohesion value and can be used accordingly.

Thus, if adhesion from short-term testing is indicated by the failure envelope and the long-term permanence of the physical or mechanical mechanism giving rise to this adhesion is logical to anticipate, its use in a stability analysis and subsequent material's specification is felt to be generally justified.

Interpretation #2 – Use of zero "c_a" and full "δ" value

For the situation where an adhesion is indicated by the failure envelope and you as the designer feel that its long-term existence is not justified, the most conservative approach you can take is to simply translate the entire failure envelope in a parallel manner down by the amount of adhesion indicated on the original data-generated graph; see Figure 2b.

The effect of this very conservative approach on the FS-value of the slope is substantial. The shear strength is now represented by a friction angle alone and the site-specific result will be very flat slopes. For example, the 3-to-1 slope in the hypothetical example given previously with an adhesion of zero, now has a FS = 1.18 using this approach. For the interfaces mentioned previously, we do not recommend this approach.

Alternatively, one could also decrease the adhesion slightly, but not entirely. That said, we really don't know how to comment on this type of "compromise" situation?

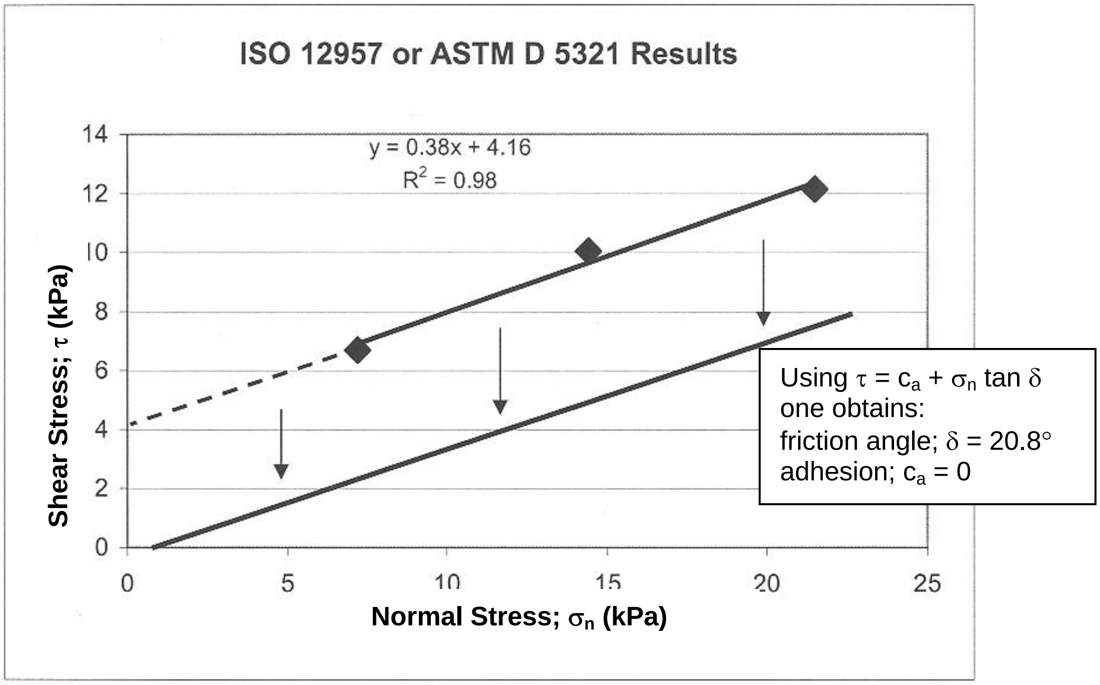


Figure 2b – Parallel translation downward of the entire laboratory generated failure envelope by an amount equal to the y-axis intercept, i.e., the adhesion.

Interpretation #3 – Use of zero "c_a" at zero normal stress only

A hybrid interpretation somewhere between the interpretations just presented is sometimes suggested, but its logic is somewhat difficult to fathom. In essence, the adhesion is lost only at zero normal stress but not at higher normal stresses. Thus, the failure envelope is forced through the origin but thereafter it is based on a least squares fit of the laboratory tested points as they were generated. Figure 3 illustrates the situation where the resulting friction angle is seen to be 32.2°. For our hypothetical example, this results in FS = 1.93. Alternatively, and equally difficult to fathom, is when only one laboratory point is generated and the failure envelope is forced through it and the origin. Both approaches are the least conservative of those mentioned in this White Paper giving rise to a rotation of the failure envelope and the highest friction angle possible. The angle resulting from this practice has been variously called "secant friction angle", "secant angle", or "modulus angle". Of the group, secant angle is probably the best description for this interpretation since it shouldn't be confused with the Mohr-Coulomb friction angle, and modulus brings with it completely other test procedures like tension testing.

We generally do not recommend such approaches for the reason that adhesion should be an intrinsic property of the interface involved and not be arbitrarily eliminated or used on the basis of a particular normal stress, or stresses. (That stated, if the interface is tested at zero normal stress and found to have zero adhesion, the origin is a valid point and should then be used accordingly).

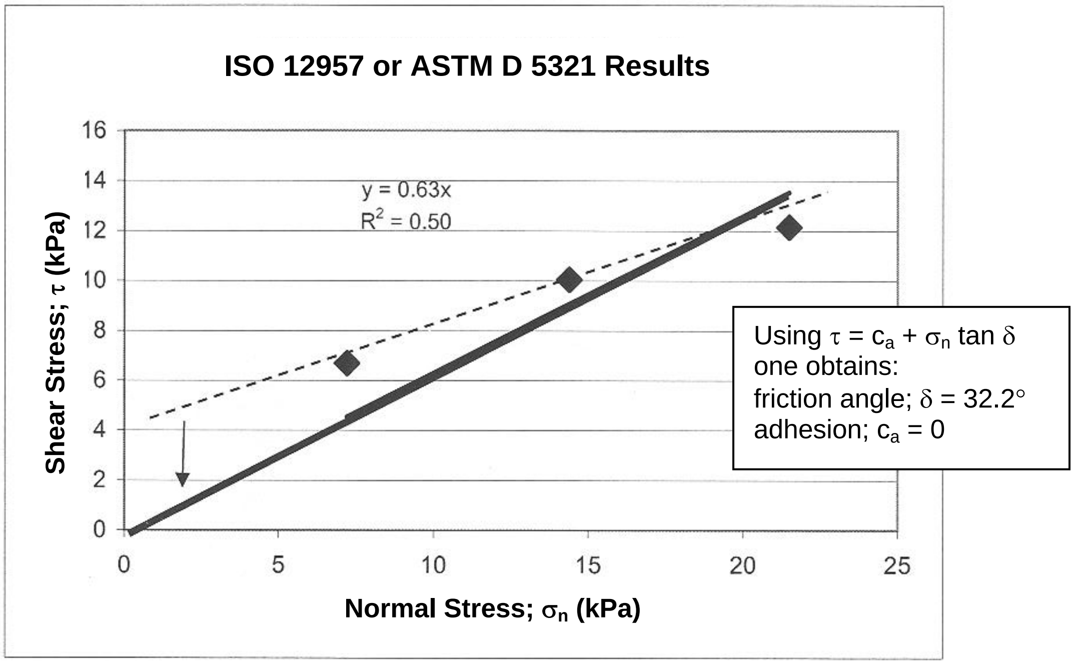


Figure 3 – Elimination of adhesion at zero normal stress but not at any of the three laboratory measured data points.

Interpretation #4 – Use of the total shear strength at a particular normal stress

A very straightforward approach to a specification value is to require a certain shear strength value at a particular normal stress. This is particularly the case if the failure envelope is curved as mentioned previously. In so doing, a specifier is requiring a single point to be taken from the failure envelope which is targeted at the expected field normal stress. Figure 4 suggests that if the field normal stress is 17.2 kPa it results in a required shear strength of 10.7 kPa, or greater. The shear strength value is thereby reflective of both a frictional component and adhesion, neither of which are specifically identified.

In so doing one avoids specifying individual "c_a" and "δ" values and much of the previous discussion is altogether avoided. The method can be extended to give two, or more, values of shear strength (or even the equation of the failure envelope) at different normal stresses in the form of a "required" table.

This approach has been used by a select few designers but is far from common practice. There is nothing of a fundamental nature which says it cannot be done and it would avoid some of the other complications inherent with different approaches.

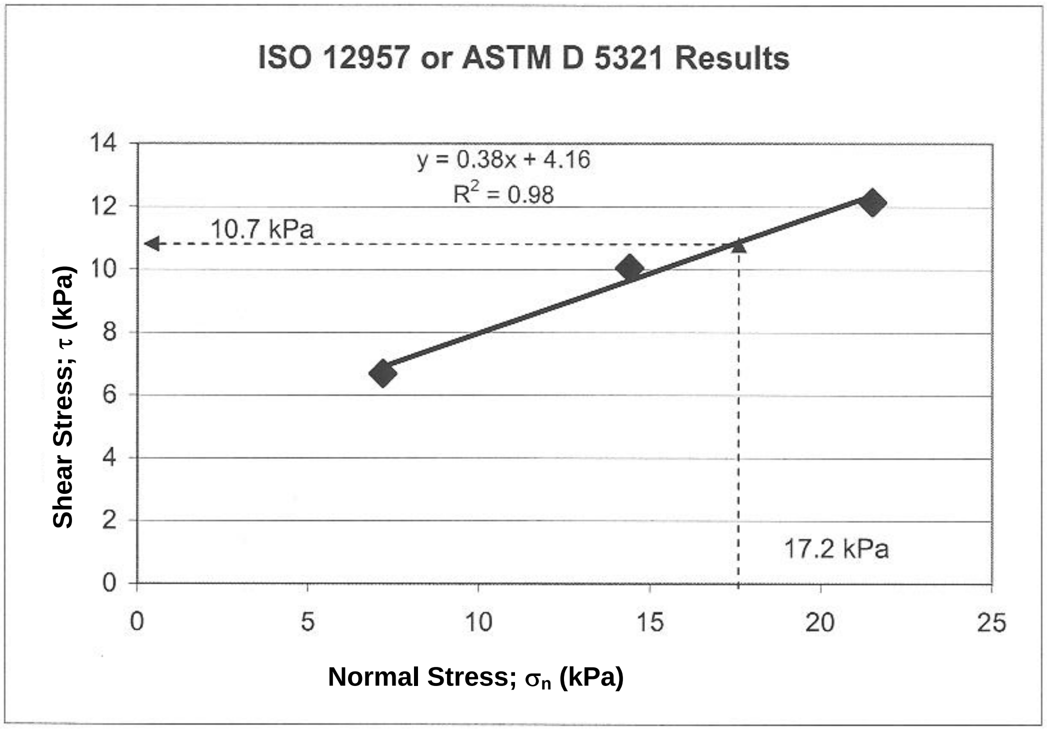


Figure 4 – Use of a laboratory generated failure envelope by specifying a site-specific normal stress and requiring a minimum value of shear strength taken directly off of the y-axis.

In summary, there are probably other or intermediate interpretations of an interface shear strength failure envelope for use in design and then a subsequent specification, but those presented here are felt to be the most common.

ATTACHMENT III.7.F

THIEL, RICHARD. *A TECHNCIAL NOTE REGARDING INTERPRETATION OF COHESION (OR ADHESION) AND FRICTION ANGLE IN DIRECT SHEAR TESTS.* GEOSYNTHETICS, APRIL MAY 2009 VOLUME 27: PAGES 10-19.

A technical note regarding interpretation of cohesion (or adhesion) and friction angle in direct shear tests

By Richard Thiel

Introduction

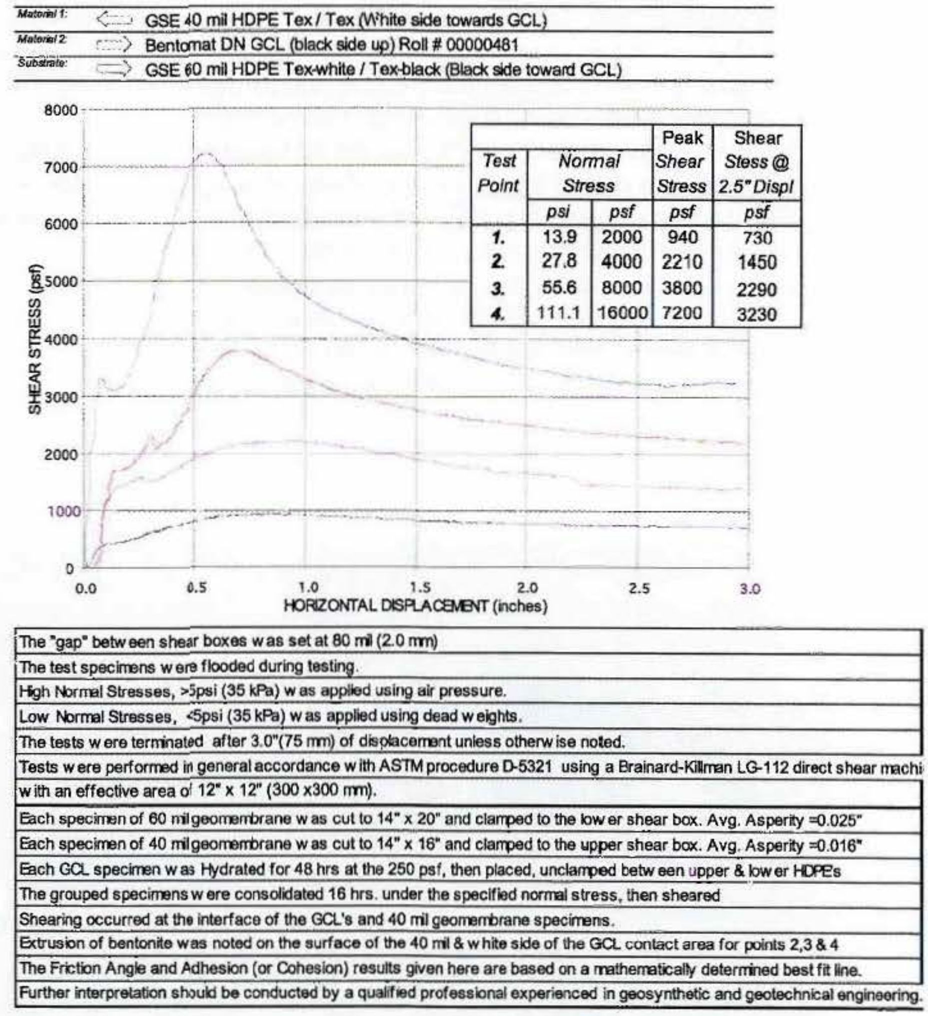
Direct shear testing with geosynthetics is generally performed in accordance with ASTM D5321, *Standard Test Method for Determining the Coefficient of Soil to Geosynthetic or Geosynthetic to Geosynthetic Friction by the Direct Shear Method.* There is also a related standard, D6243, *Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method.* This technical note applies to both equally.

Interpreting lab results

There is often confusion expressed in the industry regarding how laboratory results should be interpreted, specifically: whether one should use both the friction angle and cohesion (or adhesion) parameters; whether cohesion should be ignored; whether secant friction angles are more appropriate; what to do if the data are nonlinear; and how the data should be interpolated or extrapolated.

The goal of this technical note is to provide some guidance to take the mystery out of these questions. In the end, all data should be evaluated by an experienced practitioner qualified to use the test results properly.

What this note will not do is go into the subtleties of requesting, setting up, calibrating, and performing a direct shear test. That would be the subject of additional articles.



Test Point	Normal Stress		Peak Shear Stress	Shear Stess @ 2.5" Displ
	psi	psf	psf	psf
1.	13.9	2000	940	730
2.	27.8	4000	2210	1450
3.	55.6	8000	3800	2290
4.	111.1	16000	7200	3230

This article will also not definitively describe how direct shear test data should be interpreted. That is the responsibility of a professional with specific expertise, and one article could never presume to cover all of the considerations that might apply to any unique design problem that might arise. That is why professionals are trained and mentored in basic geotechnical principles: so they can appropriately account for the various factors affecting a design and make appropriate decisions regarding test data interpretations.

The typical sequence of events related to direct shear testing includes the following:

1. An engineer requests a direct shear test series to obtain data to help solve a problem. The request should be very specific with regard to all the necessary details regarding

| Richard Thiel is a senior project manager at Vector Engineering Inc. in Grass Valley, Calif.

The Designer's Forum column is refereed by Greg Richardson, Ph.D., P.E., a principal at RSG & Associates, Raleigh, N.C., www.rsgengineers.com sampling, specimen preparation and setup in the testing device, and test execution in accordance with both project-specific conditions and industry standards.

2. A competent and certified laboratory performs the test series in accordance with the request and the industry standard test method (e.g., ASTM D5321 or D6243). The laboratory reports results to the engineer.

3. The engineer interprets and applies the results to the project design.

What we are measuring in the direct shear test is shear strength as a function of normal load. The test does not measure "friction" or "cohesion," as these are simply mathematical parameters derived from the laboratory test results.

Ideally the engineer who originally specified and required the shear test would be the same one who reviews and interprets the results. Sometimes, such as in a third-party construction quality assurance (CQA) project, an engineer other than the original designer will commission and review the testing. Interactions with test laboratories and other engineers over time have shown that there are often misconceptions and misunderstandings related to the interpretation of direct shear test data. Thus, this article is intended to serve the purpose of helping project participants avoid confusion. The key point of this article is that what we are measuring in the direct shear test is shear strength as a function of normal load. The test does not measure "friction" or "cohesion," as these are simply mathematical parameters derived from the laboratory test results.

Figure 1 presents shear test results of a 4-point test for an interface between a textured geomembrane and a reinforced GCL. Three shear points, each at a different normal stress, are the most common number of points used to run a test series, but the number of points could vary from as few as one, to perhaps as many as six points, depending on many factors beyond the scope of this article. The figure shows: (a) a table of the normal stresses vs. peak and large-displacement shear strengths measured at 2.5in. of displacement, (b) graphs of the shear stress vs. displacement measurements, and (c) notes describing test conditions and observations.

There is adequate information in this figure for a trained practitioner to evaluate and use the data. The laboratory has performed its duty, which is to measure and report the shear strength under specified normal stresses (we are simplifying the discussion here by not elaborating on other factors such as hydration, consolidation, etc.), showing how the shear strength changed with displacement of the two surfaces, and providing descriptive and observational notes.

Figure 2 shows additional information that can be provided by a laboratory in the form of a graph of the peak and large-displacement strengths plotted as a function of normal stress. Best-fit straight lines, called Mohr-Coulomb strength envelopes, named after the gentlemen who first publicized the relationship between shear strength and normal stress, have been drawn through the two sets (peak and large-displacement) of data points.

Equations can be written for these lines, as we learned in first-year algebra class, in the form of $y = mx + b$. In this case we define y as the shear strength (S); m as the slope of the line that we call the "coefficient of friction" and whose angle is *phi* (ϕ), which we call the "friction angle" (and thus $\tan[\phi]$ is the slope of the line); x is the normal stress (N); and b is the y-intercept of the line that we call either "adhesion" (a, usually used for geosynthetics-only tests) or "cohesion" (c, usually used for tests involving soils, which will be used for the remainder of this article).

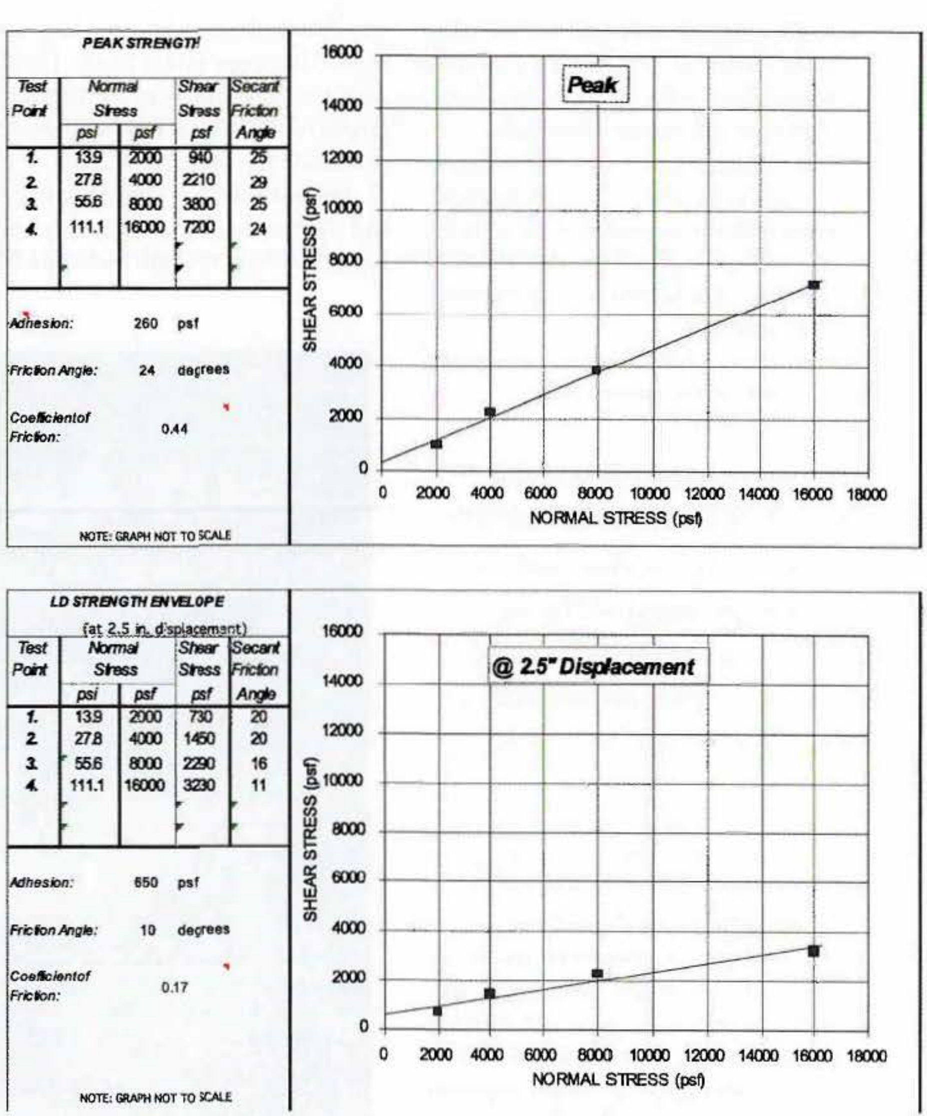


Figure 2 | Example of supplemental data interpretation provided by the laboratory.

Mohr-Coulomb

In geotechnical engineering, we write the Mohr-Coulomb equation for these lines as:

$S = N \cdot \tan(\phi) + c$

This equation is written for peak, large-displacement, or residual shear strength conditions. The fundamental points in this article regarding the presentation of the data in **Figure 2** include the following:

1. The Mohr-Coulomb envelope should not be extrapolated beyond the limits of the normal stresses under which the testing was conducted. To do so would never be conservative and, in fact, may be significantly nonconservative. The reason that simple extension-extrapolations of the Mohr-Coulomb envelope are nonconservative is presented in **Figure 3**. Most shear strength envelopes are truly curved (nonlinear). This tendency for a curved failure envelope is exaggerated in **Figure 3**, but can clearly be identified for the real-life strength envelopes presented in **Figure 2**, in particular for large-displacement conditions.

The Mohr-Coulomb model is merely a linear simplification of a portion of the entire envelope over a limited range of normal stresses. If testing were performed over a large enough range of normal stresses the curvature would become more apparent. True shear strength envelopes are found to be most accurately described by hyperbolic functions. Giroud et al. (1993) provides a good method to describe hyperbolic strength envelopes.

2. The values of *phi* and *c* should be considered nothing more than mathematical parameters to describe the shear strength vs. normal stress *over the normal-load range the test was conducted.* It is perhaps better not to think of "friction" and "cohesion" as real material properties, but simply as mathematical parameters to describe the failure envelope.

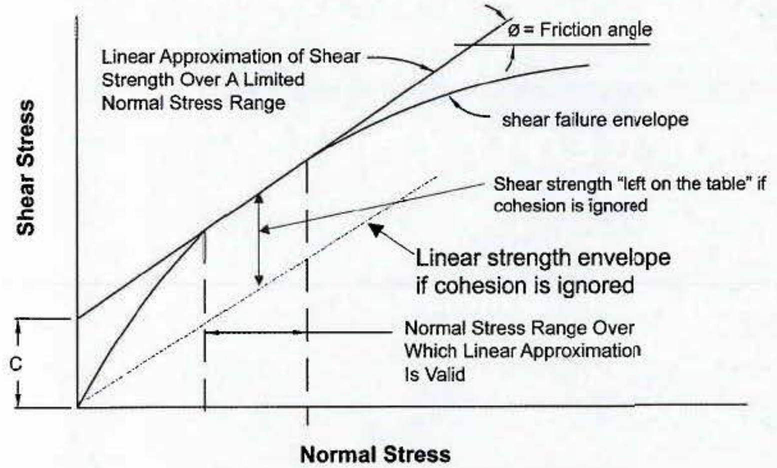


Figure 3 | Exaggerated schematic of true curvilinear shear strength envelope, linear interpretation over a selected normal stress range, and the penalty for ignoring cohesion.

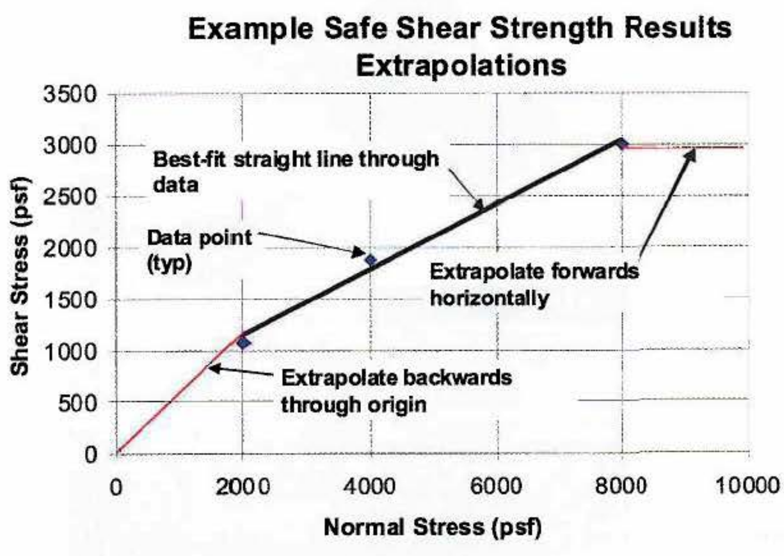


Figure 4 | Example of safe shear strength extrapolation.

In geotechnical practice with soils, there are situations and examples where the cohesion parameter is evaluated separately from the friction parameter, but these are sophisticated considerations that involve very project-specific materials and conditions and should only be done by experienced professionals.

For many geosynthetic interfaces and in the context of many types of projects, there is absolutely no reason to dissociate the slope of the line from its y-intercept, and the shear strength should be taken as a whole in those cases. Other situations may occur, however, where it is appropriate, but those considerations are beyond the scope of this article.

3. In many, if not most, cases with geosynthetics where there is no reason to ignore the cohesion value, it is important to re-emphasize that shear strength should only be defined within the range of normal stresses for which the Mohr-Coulomb envelope was derived. Ignoring the cohesion may be unjustifiably penalizing the shear strength values that were measured in the test, as illustrated in **Figure 3**.

Using the cohesion value at normal stresses extrapolated below the range of testing, however, could have dire consequences on the safety of a design project. This problem may occur when designers consider only the operational or final build-out of a facility and they ignore the construction condition. Several failures have occurred during construction because of this. For example, an embossed geomembrane against a geotextile may perform well under high normal loads by providing a good friction angle and a modest y-intercept for operating and final build-out conditions. However, under the low normal loads experienced during construction of a thin soil veneer on a steep sideslope, testing might reveal that the adhesion extrapolated from the high-normal load results do not exist at low normal loads. In this case, a more aggressive texturing that exhibits a "Velcro®-effect" type of adhesion, or a very high friction angle, at low normal loads may be needed and should be verified at the proper normal loads.

4. Figures 1 and **2** also report *secant* friction angles for each point. These are the angles of the straight lines from each point drawn back to the origin. A key concept regarding secant friction angles is that you should never extrapolate a secant angle line beyond the normal load for which it is measured. Secant values are conservative as long as the secant values are derived from a test whose normal stress was greater than the normal stresses of the design. They can quickly become nonconservative if the same friction angle is used for higher normal loads.

5. If users wish to extrapolate shear strength data, **Figure 4** illustrates the only "safe" way to accomplish this. Going from the low end of the Mohr-Coulomb envelope and extrapolating backward, the data can be extrapolated by drawing a straight line back to the origin. Going from the high end of the Mohr-Coulomb envelope and extrapolating forward, the data can be extrapolated by drawing a straight line

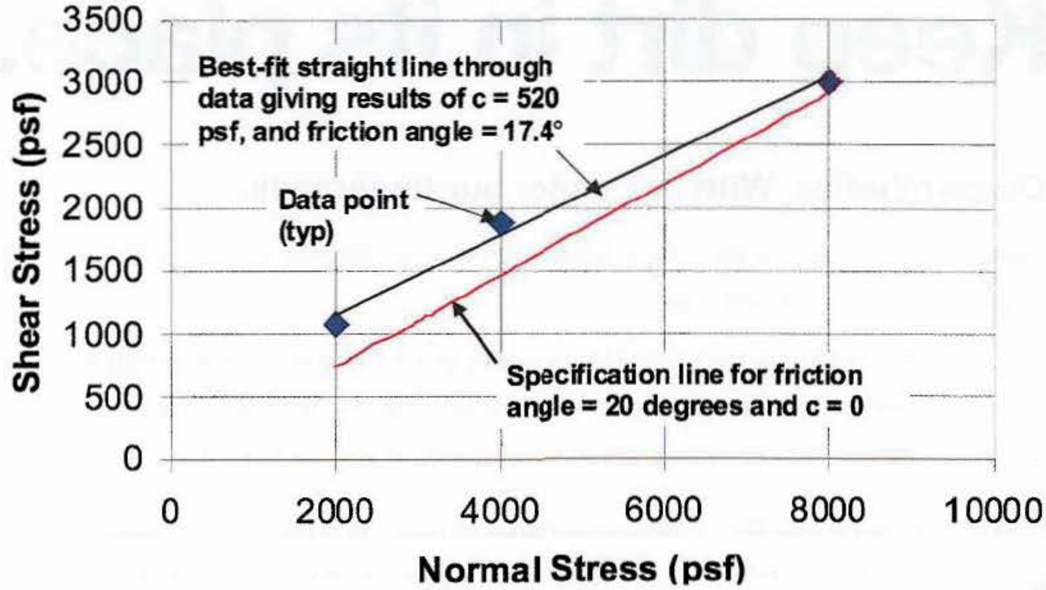


Figure 5 | Example project results where interpretation of test data results in lower friction angle than specified value, even though shear strength results are higher than the failure envelope implied by the specifications.

horizontally forward. This extrapolation rule is safe only when considering a single interface. When multiple interfaces are involved, it is not safe to extrapolate a multi-layered system on the high side of the Mohr-Coulomb envelope.

From the discussion above, we can now look at the ASTM standard D5321 with more understanding and critical thought. The first thing to note is that the title of that standard is poorly worded. The title is "*Determining the Coefficient of...Friction...*" This is somewhat misleading because it implies that the designer is simply after a coefficient of friction. In fact, what designers need is a relationship between shear strength and normal stress. Therefore, a more appropriate title for this method would be "*Determining the Relationship between Shear Strength and Normal Stress for Soil-to-Geosynthetic or Geosynthetic-to-Geosynthetic Interfaces Using the Direct Shear Method.*" Note that ASTM D6243 has already rectified this problem in its title.

Another misleading element in ASTM D5321 is the definition of *adhesion* (which applies equally to cohesion), which it states as: "The shearing resistance between two adjacent materials *under zero normal stress* (emphasis added). Practically, this is determined as the y-intercept to a straight line relating the limiting value of shear stress that resists slippage between two materials and the normal stress across the contact surface of the two materials."

This is actually *two separate definitions,* which are most likely not the intent of the standard. The first part of this definition, which defines the adhesion as the shear strength at zero normal stress, is not applicable relative to the test method. It *could be true* if we proposed to test the interface at zero normal load, but that is rarely done and generally of no use. The industry would be better served by deleting the first part of the definition. In reality, the second part of the definition is the controlling aspect of the definition, and the "y-intercept" concept is the true nature of the adhesion value which, as stated above, is simply a mathematical parameter.

Note that ASTM D6243 has a different set of definitions, and it is not clear if those definitions are unique to that standard, or are intended to be industry norms. ASTM D6243 suggests that adhesion is the true shear strength when there is truly zero normal load, and that cohesion is the mathematical parameter of the y-intercept obtained from the Mohr-Coulomb envelope. In the author's opinion these definitions are acceptable as stated, but the audience should know that the definition of *adhesion* may conflict with other definitions put forward in the industry. Also, other authors have introduced other terms for the measurable shear strength under zero normal load, such as Lambe and Whitman's (1969) "*true cohesion.*" Interested readers can research ASTM D6243 and the literature and judge for themselves.

Example problem 1

The following situation illustrates a common example of a problem that occurs with shear test data interpretation:

- A specification is written that requires a certain minimum interface friction angle to be achieved between a textured geomembrane and a GCL. For purposes of this example, the requirement is 20° peak shear strength for normal loads tested between 2,000 and 8,000 pounds per square foot (psf).
- The laboratory results, shown as an

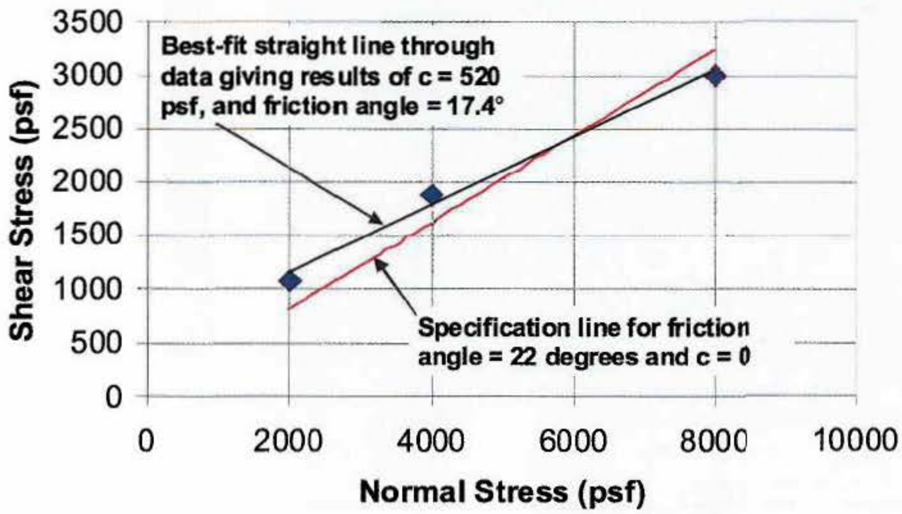


Figure 6 | Example project results where the two lower points are above the specification and the upper point is below the specification.

example in **Figure 5**, report a best-fit Mohr-Coulomb peak strength envelope with shear strength parameters of 500 psf cohesion and 15° friction. **Figure 5** also shows the line representing the minimum project specification.

Inspection of **Figure 5** shows that the shear strengths achieved in the direct shear test plot above the shear strength envelope required by the specification. Even though the plot appears to clearly indicate that the minimum required shear strength is achieved by the products tested, the author has experienced several projects where one of the project parties (e.g., the design engineer or perhaps a regulator) have declared the test a failure because the reported Mohr-Coulomb friction angle was less than the specified friction angle.

In the author's opinion, in many cases involving this particular interface, there is no reason to consider this a failing test.

This example illustrates the confusion that might arise when specification is written in terms of a shear-strength *parameter*, when the real objective is to achieve a certain value of absolute shear strength. Even though the materials provided the shear strength required by the specification, there is some confusion because one of the strength *parameters* did not meet the specified value for that parameter.

It is possible that the original specifier had taken into account the potential for cohesion, and had wished to discount cohesion, and really wanted a true minimum friction angle of 20°. If the specifier were truly that sophisticated and had such complex reasoning, then more than likely the specification would have also been more sophisticated in explaining these constraints on the test results.

In the author's experience it is rare that other designers and specifiers are discounting cohesion with geosynthetic interfaces, and usually it is simply a matter of proper interpretation and communication of the design intent compared to the actual test results. Nevertheless, as stated at the beginning of this article, it is not the intent of this article to provide guidance and suggestions on interpreting test results. Rather, the intent is to shed light on some common misunderstandings.

Example problem 2

The following problem has the same laboratory shear strength results as Problem 1, but the specification requirement is increased to 22° peak shear strength.

The relationship between the test results and the specification is shown in **Figure 6**. In this example, the two lower-normal load shear strength test results plot above the specification line, while the upper-normal load shear strength test result plots below the specification line. Based on the failing result of the upper-normal load test, most reviewers would initially say that this is a noncompliant test result and fails to meet the specification.

In the author's experience, curved failure envelopes are common, and the tendency for the highest normal-load result to fall beneath a straight-line friction-based specification is not unusual.

In this case, a more detailed review by the design engineer might reveal that the shear strength results provide an acceptable factor of safety for the intended purpose. It may be that the additional strength capacity provided in the lower normal load range that is above the specification more than offsets the reduced strength capacity in the upper normal load range that is below the specification. Clearly, the only person who can evaluate this issue, and who carries the requisite authority and responsibility, is the design engineer.

The following lessons can be gleaned from this example:

- Design engineers often attempt to specify a unique set of shear strength parameters as a minimum requirement for a given design. In reality, there may be an infinite combination of shear strength variations over the applicable range of normal loads that may satisfy the stability and shear resistance requirements, and many of these combinations may have a portion of their failure envelopes that fall below the specification.
- The tendency for natural and geosynthetic interfaces to yield curved failure envelopes can present a challenge to engineers, owners, and manufacturers who wish to optimize a design using simple straight-line shear strength specifications.
- A learned interpretation of direct shear testing data by an experienced practitioner may allow acceptance of apparently failing test results. This can occur because overly simplistic specification parameters may not ac-

count for other combinations of shear strength results that could provide acceptable overall shear resistance.

Summary

The direct shear test measures shear strengths as a function of normal stress. Period.

The test does not measure "friction angle" or "cohesion," as these values are parameters that are derived from the test results. Consideration of "friction angle" and "cohesion" simply as mathematical parameters used to describe shear strength data is of great benefit to practitioners for the following four reasons:

1. Interpretation of laboratory shear strength data should not be confused with the mathematical parameters used to describe it.

2. Proper data interpretation may avoid unnecessary penalization of the results by arbitrarily reducing the measured values.

3. This understanding can improve a designer's sensitivity to how important it is that shear strength is measured within the range of normal stresses that represent the design. Thus, the only defendable extrapolation of data should be: (a) back through the origin from the lowest normal stress, and (b) horizontally from the highest normal stress.

4. Laboratory shear strength data should be interpreted by a qualified practitioner experienced in the use and application of the results.

Often of much more importance than deciding whether to include or omit the cohesion (or adhesion) parameter is the decision of whether to use peak, post-peak, or residual shear strength. This discussion is beyond the scope of this technical note, and anyone commissioning and interpreting shear strength testing should be well versed in the issues surrounding this topic, as well.

Acknowledgements

The author would like to thank Richard Erickson and Chris Athanassopoulos for their review of this article.

References

Giroud, J.P., Darrasse, J., and Bachus, R.C., (1993). "Hyperbolic Expression for Soil-Geosynthetic or Geosynthetic-Geosynthetic Interface Shear Strength", *Geotextiles and Geomembranes*, Vol. 12, No.3, pp. 275–286.

Lambe, T.W. and Whitman, R.V., (1969). *Soil Mechanics*. John Wiley & Sons, New York, NY.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 7: TENSILE STRESS ANALYSIS

ATTACHMENT III.7.G

THIEL, RICHARD. *PEAK VS RESIDUAL SHEAR STRENGTH FOR BOTTOM LINER STABILITY ANALYSES.*

THIEL ENGINEERING. OREGON HOUSE, CALIFORNIA, USA

PEAK VS RESIDUAL SHEAR STRENGTH FOR LANDFILL BOTTOM LINER STABILITY ANALYSES

Richard Thiel
Thiel Engineering, Oregon House, CA, USA

ABSTRACT

The decision whether to use peak or residual shear strengths for a stability analysis must be made in the context of a specific design situation. Yet even when the specific situation is defined, the decision of whether to use peak or residual shear strength is often unclear. In general, if there are potential construction, operation, or design conditions that might cause relative displacement between layers, then a post-peak or residual shear strength for the layer having the lowest peak strength is appropriate. If seismic analyses predict deformation on a given interface, then the design should use the post-peak or residual shear strength for that interface. For bottom liner systems, where stress distribution along the liner system is very complex, it is advisable to verify that the slope stability has a factor of safety greater than unity for residual shear strength conditions along the critical interface.

INTRODUCTION

This paper is concerned with the forces that support a landfill on its liner system, and the shear strength of geosynthetic interfaces that keep the mass from sliding. Figure 1 schematically portrays the shear forces that work to keep the waste mass from sliding. If sliding occurs, the surface along which sliding would occur is called the critical surface, or potential slip plane. Bottom liner systems that use geosynthetics often have their critical surface along one of the geosynthetic interfaces. The shear strength of these interfaces can usually be measured by means of laboratory testing. These interfaces often realize their peak shear strength within a small amount of relative displacement (on the order of 25 mm), after which their shear strength decreases. Typically, after 50 to 300 mm of relative displacement, the shear strength is reduced to a steady minimum value, which is called the residual shear strength of that interface. Figure 2 shows a typical shear stress-displacement curve for a geosynthetic interface.

Over the life of a landfill the following activities occur: the liner system is built; waste is placed; settlement occurs; a final cover system is installed; and settlement and degradation of the waste continues. Each of these phases of the landfill's life produces different combinations of normal and shear stresses on the liner system. Landfill leachate and gas, which can create destabilizing pore pressures, are by-products of the landfill, and are removed with varying degrees of efficiency. The primary questions addressed in this paper are:

- Should a designer use peak or residual shear strengths, something in between, or a combination of peak and residual strengths, when evaluating a landfill design?
- What does the profession really know about the mobilized shear stresses? (This paper will focus on bottom liner systems.)
- Should the same choice whether to use peak or residual shear strengths be applied along the entire lining system, or should slopes and base liners be treated differently?
- Is there a preferred design approach?
- What factors of safety are appropriate for design?

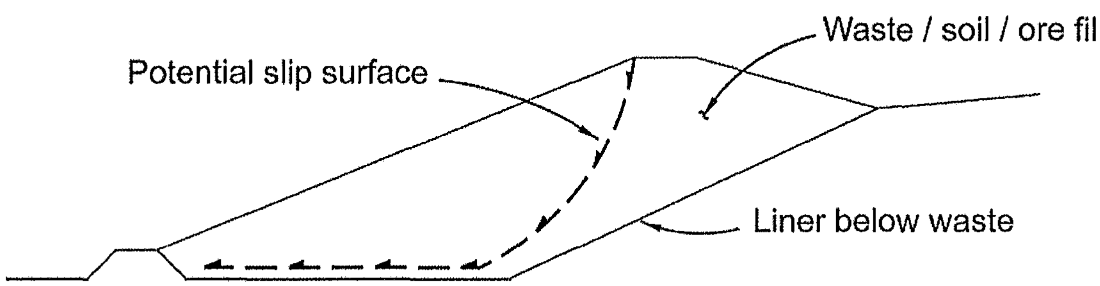


Figure 1 – Schematic of Shear Forces Along Critical Slip Plane

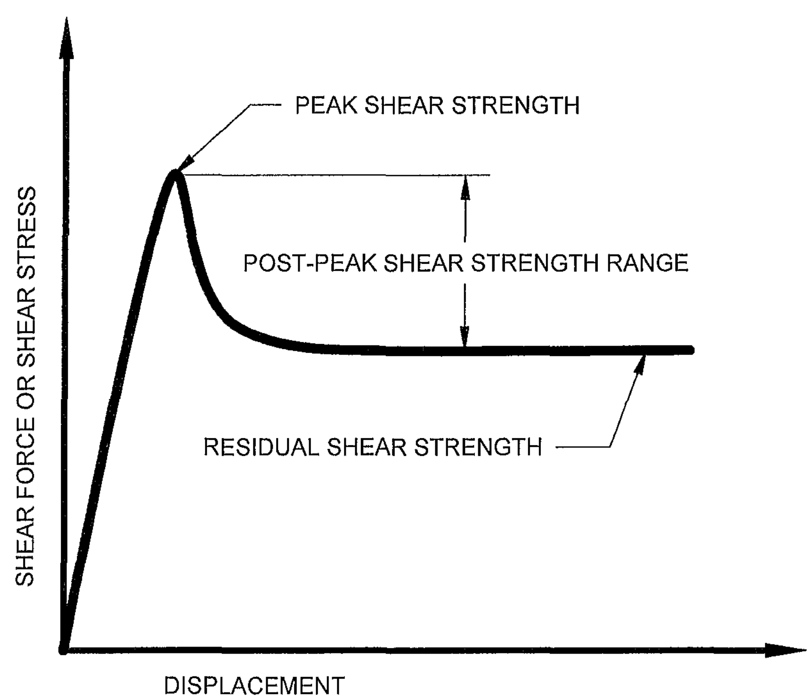


Figure 2 – Example Graph of Shear Force vs. Deformation for Geosynthetic Interface

ORGANIZATION OF THIS PAPER

Part 1 of the paper describes general considerations in performing slope stability analyses. It begins with a discussion of different types of slope stability analyses, including limit equilibrium, finite element, and 2-dimensional (2-D) vs. 3-dimensional (3-D) analyses. Understanding how the state-of-the-practice has developed, and the limitations of the analytical approach, both contribute strongly to making the right selection of appropriate shear strengths and factors of safety.

2-D limit-equilibrium analyses are by far the most common approach for evaluating slope stability. Part 1 discusses practical guidelines and common pitfalls that affect the results of these analyses, especially the selection of the critical shear plane on which the peak or residual shear strength will be modeled. Part 1 also discusses how pore pressures might cause a surface to exceed its peak shear strength and induce progressive failure. Selecting the appropriate shear strength requires an understanding of the effective normal stress range. Also, commissioning direct shear testing from a laboratory requires that one understand the proper testing parameters needed to obtain appropriate peak and/or residual shear strength values.

Part 2 of the paper directly addresses the question of peak vs. residual shear strength, and begins by discussing ductile vs. brittle behavior. Progressive failure, which occurs with brittle materials, then emerges as the chief concern of this paper. The discussion that follows considers conditions that could cause a brittle material to exceed its peak strength in the context of a landfill bottom liner, followed by a brief summary of field observations in this regard.

Part 3 discusses possible design approaches in terms of the selection of peak strength, residual strength, and hybrid approaches, and then considers the appropriate factors of safety for these different approaches.

Part 4 then presents conclusions reached from the preceding discussions. It also provides recommendations for practical design approaches based on the author's experience, as well as recommendations for further research.

This paper surveys the key considerations one employs when deciding whether to use peak or residual shear strength for bottom liner systems in landfills. It does not presume to make that decision, but rather seeks to outline and discuss all considerations that are necessary and pertinent to that process. Although many of the considerations this paper presents may be general enough to apply to cover (veneer) systems, it has been written solely with bottom liner systems in mind, and does not consider the long-term issues related to cover systems.

PART 1 – GENERAL CONSIDERATIONS

LIMIT-EQUILIBRIUM VS FINITE-ELEMENT ANALYSES

Limit-equilibrium analyses, whether 2-D or 3-D, are the most common methods of assessing slope stability. These methods can be performed by hand or, more commonly, by using a computer program. Such analyses evaluate the force and moment equilibrium of a slope on an assumed slip plane given assumed shear strength, unit weight, and pore pressure parameters. The result of these analyses is then presented as a factor of safety (*FS*) defined as:

$$FS = \frac{\text{Shear strength along the slip surface}}{\text{Shear stress along the slip surface}}$$

One defining characteristic of the limit-equilibrium approach is that it presumes that the factor of safety is the same everywhere along the slip plane. Therefore, the mobilized shear stress distribution along the slip plane is simplistically assumed to be a constant ratio of the shear strength along that plane. Such analyses also do not take into account elastic or plastic deformation. These are both significant considerations when deciding whether to use peak or residual shear strength.

Finite-element analyses attempt to calculate the stress distribution and deformations in a soil mass. In addition to considering force and moment equilibrium, these analyses also typically consider the materials' elastic modulus and Poisson's ratio, and some models can also calculate the change in shear strength with displacement for various materials. The result of these analyses is usually presented as a distribution of mobilized shear stress and displacements.

At first glance it would seem that finite-element analyses offer more of what we wish from a slope stability analysis as opposed to limit-equilibrium analyses. So much so, that we might even ask ourselves why we continue to bother with limit-equilibrium analyses. The fact remains, however, that the limit-equilibrium approach has been and will continue to be the basis of standard practice in the industry. The reasons for this, some of which also appear in the next section that considers 2-D vs. 3-D, are:

- Limit-equilibrium approaches have been performed and "calibrated" through industry experience for the past 80 years. Properly performed limit-equilibrium analyses have been proven to be adequate.
- Finite-element analyses are sophisticated and complicated to perform. The average design practitioner often is not adequately trained to perform such analyses, and the low frequency of projects that require their use do not justify the

resources needed to keep an engineer qualified to perform them on every landfill-design firm's staff.

- In the past few years the author has peer-reviewed a number of slope stability analyses. On four major landfill projects for which calculations had been prepared by separate reputable nationwide and local design firms, the author found fundamental errors in 2-D limit-equilibrium analyses. Some of these projects had already been built and were, in the author's opinion, at serious risk of large-scale failure. If such fundamental errors continue to be made with analyses as simple as 2-D limit-equilibrium, the prospects of universalizing a finite-element approach for the solid waste industry is not very promising. Finite-element analyses epitomize the expression "garbage-in garbage-out", so strict quality control and quality assurance is in order whenever they are employed.

2-D vs. 3-D ANALSYES

One issue that is periodically debated in the literature and at professional gatherings is the use of 2-D as opposed to 3-D analyses. Soong et al. (1998) question whether 2-D analyses are appropriate for landfills, and suggest it would be more appropriate to use 3-D analyses with residual strengths. From a pragmatic point of view, the everyday stability analysis has been, and will continue to be, 2-D in actual practice. There are three main reasons for this, clearly laid out by Duncan (1996):

- Inherent Conservatism. Properly performed 2-D analyses always give a factor of safety that is equal to or less than those given by 3-D analyses. 2-D analyses, therefore, are more conservative.

- Ease of Application. The average professional consulting engineer is interested in the amount of time it will take to arrive at an answer, the frequency of projects that will require special attention, and the effort it will take to organize the results in a final report. 3-D applications are simply not as easy to use as 2-D.

- Avoidance of Errors. As illustrated above, analyses are prone to errors, and 3-D analyses are more complicated than 2-D analyses. The author believes that the emphasis in the profession needs to be on performing solid, fundamental engineering, rather than on increased sophistication that invites more errors.

3-D analyses have mostly been used for forensic studies, and for those few complex situations that involve a very unusual geometry and/or distribution of shear strengths in the potential sliding mass. Examples of these can be found in Stark and Eid (1998). In the author's 16 years of experience performing stability analyses on dams, embankments, cut slopes, and landfills, there were only three situations where a 3-D analysis was warranted during design, and all three were satisfactorily accomplished using multiple 2-D sections. One of these projects was given as an example in the Stark and Eid (1998) paper. In that case Stark and Eid (1998) felt that a 2-D slope stability analysis could not anticipate the combined effects of the project's complicated geometry and shear strength zones. After discussion of the project's complexity, they reported a minimum 3-D factor of safety of 1.65 using a 3-D analysis program. In fact, the original design team, of which the author was a part, had two years earlier calculated a factor of safety of 1.60 using weighted averages of several 2-D cross-sections. Thus, even in this circumstance that had unusually complicated geometry and shear strength conditions, a modified-2-D approach gave results one would expect relative to the 3-D analysis results.

Notwithstanding the reservations given above, 3-D analyses will well serve those who have the time and budget to perform them.

To summarize, the refinements in accuracy offered by 3-D analyses are rarely matched by the average practitioner's understanding of basic slope stability mechanics, much less the level of confidence ordinarily offered by assumed shear-strength and pore-pressure parameters. Most often, the differences in shear strength and pore-pressure assumptions made by different engineers will substantially outweigh the refinements obtained by favoring 3-D over 2-D analyses. Compare, for example, the different conclusions reached by Schmucker and Hendron (1998) versus Stark et al. (2000) regarding the cause of a major landfill failure; or the difference in 2-D vs. 3-D comparisons for a landfill failure described by Soong et al. (1998), from those made by Stark et al. (1998). These case histories, recently published by experienced professionals, do not provide a compelling argument that 3-D analyses should be preferred. They do, however, reinforce the notion that the major factors contributing to uncertainty in a slope's performance are shear strengths and fluid pressures, and that this is where our attention should be focused. The purpose of this paper is to focus specifically on one of these issues, namely, when it is appropriate to use residual vs. peak shear strength for geosynthetic interfaces at the base of a waste containment facility.

GENERAL DISCUSSION OF 2-D ANALYSIS APPROACH

Method of Analysis

Slope stability analyses are most commonly assessed using computer programs that evaluate the limit equilibrium of a 2-D cross-section. Less sophisticated limit equilibrium analyses can be performed using hand-calculation methods or charts. Hand calculations are an effective analysis tool because they often provide a clearer understanding of the critical aspects of the problem, and mistakes in geometry and assumed failure planes are less likely. A common approach is to perform a hand check on the most critical surface that has been analyzed by a computer program. A good summary of slope stability approaches using hand calculations is provided by Abramson et al. (1996).

Limit-equilibrium analyses of varying complexity that have been developed are available to design practitioners. One of the first approaches was the Ordinary Method of Slices developed by Fellenius. Later refinements were presented by Bishop, Janbu, Morgenstern and Price, Spencer, and others. A review of these methods is beyond the scope of this paper, and the reader is referred to Abramson et al. (1996) and Duncan (1996) as a starting place for a comparison of the various limit-equilibrium methods. The author would, however, offer three points from his own practice as to which method to use for performing stability analyses of bottom liner systems:

- The Bishop method is generally not applicable when analyzing bottom liner system geometries because it was developed for circular failure surfaces. The critical slip plane for liner systems is often a translational block that is non-circular.
- Spencer's method, which is now commonly available in computer codes, is considered more rigorous and complete in its analysis than the simplified Janbu method, which is commonly used for block analyses. Spencer's method is computationally more intensive, however, and may be difficult to use for random searches for a critical failure surface, even with modern computers. It is also less stable and can yield incorrect results unless the line of thrust results are checked by the user. Therefore, a good practice is to search for the critical surface using Janbu's simplified approach, and then perform a final check on the stability using Spencer's method. Usually, but not always, Janbu's method will result in a slightly higher factor of safety.
- The approach developed by NAVFAC (1982) for translational block analyses is often a good and appropriate method for performing a hand-check on the computer results for a 2-D translational block failure along a bottom liner system.

Identification of Critical Slip Plane

The most typical requirement for static stability is to meet a specified factor of safety. Just what constitutes an appropriate factor of safety will be discussed later in this paper. The idea is that if the stability analysis is performed correctly with the proper input variables, the factor of safety should provide a level of confidence that the slope will in fact be stable.

The essential operative words in the above paragraph relating to stability analyses is that they are *"performed correctly"*. The safety margin in a factor of safety exists to account for unknown or unpredicted deviations from the original design assumptions. It is not, however, supposed to account for errors in the analysis, or incorrect geometric and material property assumptions.

When performing a correct analysis the critical slip plane for analysis must be identified correctly. An experienced geotechnical engineer is usually required in order to select the critical cross-sections for analysis of a slope. Even for experienced practitioners, though, it is not always obvious which section is the most critical, and several trials generally need to be performed. For very complicated geometries, as described in the previous section, multiple 2-D sections may need to be weighted in order to simulate a 3-D analysis, or the more complex 3-D analysis can actually be performed.

In addition to selecting the proper cross-section, it is also important to search for and select the correct critical slip plane within that cross-section. In peer-reviewing slope stability analyses performed by others, the author has found errors in which the designer had correctly identified the critical cross-section, but incorrectly identified the critical slip plane within that cross-section. He found others, too, in which the designer had conceptually identified the correct slip plane, but failed to code the computer program to correctly place the slip plane at the correct interface within the liner system. The effects of such errors was to drop from an ignorantly-blissful factor of safety of 2 to 3, to an uncomfortable factor of safety of less than 1.1.

When the critical slip plane is along the liner system, the critical surface is always the one that has the lowest peak strength. If residual strengths are used in the analysis, they should reflect the surface that has the lowest peak shear strength, because that is the one that will govern deformations.

Pore Pressures

Next to gravity, pore pressures (most pervasively those caused by liquid as opposed to gas) are the single most prevalent factor contributing to slope stability failures. They are also among the most overlooked elements in slope stability analyses. Schmucker and Hendron (1998) illuminate this problem when they state that "Very little is known at this time regarding the generation and distribution of pore pressures in MSW landfills."

The one area where evaluating the influence of pore pressures on slope stability has been well focused has been in the design of dams. For this reason there have been few dam failures due to the neglect of pore pressures, with dam failures in the past century generally being caused by other factors (e.g. liquifaction or piping). Pore pressures are not commonly included in landfill analyses. Yet most (or at least many) of the dramatic landfill failures reported in the industry can be attributed to pore pressures that built up either in the foundation, due to waste loading, or in the waste itself, due to leachate buildup or leachate injection. Examples are the Rumpke landfill failure (see Schmucker and Hendron, 1998, who attributed the failure in part to leachate buildup caused by an ice dam at the toe), and the Dona Juana landfill failure (see Hendron et al., 1999, who attributed the failure to high-pressure leachate injection).

When performing slope stability analyses, designers should consider the potential for unanticipated pore pressures. Unanticipated conditions may occur in landfills due to clogging of the leachate collection systems, or aggressive leachate recirculation in the waste mass. Additional discussion of this issue is provided by Koerner and Soong (2000). Further discussion later in this paper describes how pore pressures could lead to a localized exceedence of peak strength, leading ultimately to a progressive failure.

Selecting and Measuring Material Shear Strengths

Shear Strength Definition. Figure 3 illustrates a non-linear shear strength envelope, which is typical for many soil and geosynthetic interfaces. Sometimes the non-linearity is slight, and a straight-line approximation over the entire load range under consideration can be valid. This is often true for very narrow load ranges such as those considered for cover veneer systems. At other times this non-linearity is quite significant, especially when shear strength characteristics are evaluated over the broad range of normal loads indicative of bottom lining systems.

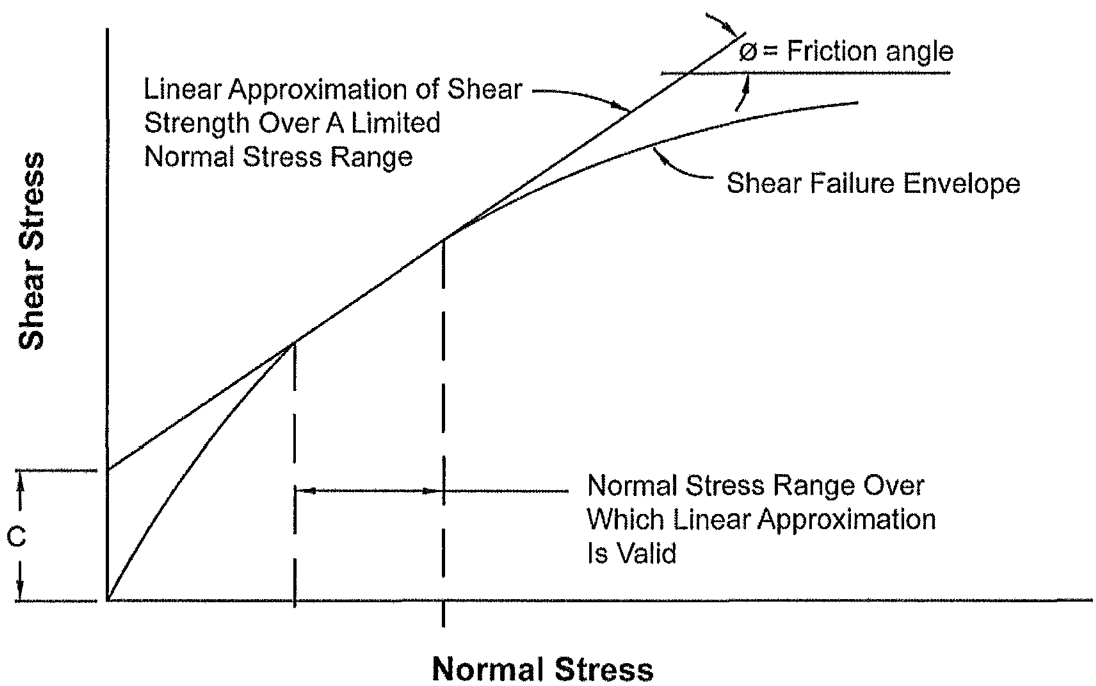


Figure 3 - Typical Shear Failure Envelope for Soil and Geosynthetic Materials.

If the shear strength curve of the evaluated materials is non-linear with respect to normal load, then special consideration should be given to defining the shear strength parameters within a specific normal load range. Many computer programs only allow the input of linear shear strength parameters. These parameters are normally identified as a friction parameter *(ϕ)* and a cohesion (or adhesion) parameter *(c)*. It is useful to recognize that these are often only mathematical parameters that describe the shear strength of a material or interface over a specific normal load range. The shear strength parameters are demonstrated in Figure 3.

Draft European Standards, and other publications (e.g. Koerner and Daniel, 1997) suggest that the apparent cohesion of a shear strength envelope can be ignored. As stated by Jones and Dixon (1998): "This assumption can have a significant effect in that the shear strength for any particular normal stress will be quoted as being lower than measured... It is possible that the failure envelope may curve to the origin at very low normal stresses, in which case ignoring the apparent cohesion will result in over conservative results." If we recognize that the values of the parameters ϕ and c are only mathematical tools used to describe the measured or estimated shear strength over a given normal load range, we can discount statements that advocate that cohesion can be ignored.

The friction parameter (ϕ) is related to the slope of the line (slope = $\tan\phi$), the cohesion parameter (c) is the y-intercept, and the normal load range is the abscissa range over which the straight-line approximation of the shear strength envelope is valid. Use of the shear strength parameters outside of the normal load range for which they were defined is generally non-conservative, as illustrated in Figure 3.

If the computer program only allows the consideration of linear shear strength envelopes, the shear strength envelope for non-linear materials should be discretized into a series of straight-line approximations for different normal load ranges. Furthermore, where the critical slip surface runs through a material or interface that exhibits a non-linear strength envelope, the designer should either use a computer code that allows input of a non-linear shear strength envelope, or assign different strength parameters to different zones of the material or interface according to the normal loading it theoretically experiences. For computer codes that do not allow non-linear shear strength envelopes, the delineation of different normal-load zones for non-linear materials is usually calculated by hand. This procedure is outlined in detail by Thiel et al. (2001).

Shear Strength Measurement. For geosynthetic lining systems, the internal and interface shear strength is normally determined by using the direct shear test in accordance with ASTM D 5321. For GCL internal and interface shear strength evaluation, direct shear testing is conducted in accordance with ASTM D 6243. In these direct shear tests, the geosynthetic material and one or more contact surfaces, such as soil or other geosynthetics, are placed within a direct shear box. The specimens are hydrated, consolidated, and placed under a constant normal load in accordance with the ASTM procedures, along with any project-specific testing clarifications/instructions from the design engineer. A tangential (shear) force is applied to the materials, causing one section of the box to move in relation to the other section. The shear force needed to cause movement is recorded as a function of horizontal displacement.

The test is normally performed for several different normal loads. Typically a series of at least three individual tests are performed at specified normal load conditions. The normal load and shear forces are converted to stresses by the given area over which shear occurred, typically a 12 in x 12 in (300 mm x 300 mm) sample. The peak and post-peak (or residual, if deformation is taken far enough) shear strengths are plotted on a graph, and a best-fit straight line or curve is fit through the data to represent the shear strength envelope. Several factors can influence the interface shear strength of geosynthetics. The most important of these are discussed below.

Valid Testing Technique. While not offering any endorsements, the author can state that he trusts very few laboratories in the nation to provide high quality direct shear test data. Initial ASTM round-robin testing of even the most simple interface (nonwoven geotextile against a smooth HDPE geomembrane) produced a shot-gun scatter of results with very poor correlation. Unless the initial test data has integrity, most of the further considerations offered in this paper become meaningless. It is imperative that the designer screen the testing laboratory in order to obtain test data of assured accuracy.

Rate of Shear Displacement. The typical default shear rate for direct shear testing with geosynthetics as presented in ASTM D 5321 is 0.04 in/min (1.0 mm/min). For testing hydrated GCLs, ASTM D 6243 provides guidance on attaining consolidated drained conditions that should preclude the build-up of excess pore pressures.

In general the rate of shear displacement affects peak strength more than residual strength. Depending on the interface being tested, the strain rate of the test should be slow enough to give results representative of long-term (slow) shear conditions.

Hydration. The moisture content, degree of saturation, and degree of consolidation of adjacent soils and geosynthetics can all exert an influence on the shear strength results. It is important to direct the testing laboratory as to the sequence of hydration and consolidation. With clay soils adjacent to geosynthetics, it is generally more conservative to hydrate under low normal loads before consolidating. Thus far, the type of hydrating fluid has not been reported in the literature as affecting shear strength results, especially in regard to typical landfill leachates.

Normal Stress. The most common strength-related errors in computer slope stability analyses stem from using strength parameters that do not correspond to the normal load conditions at the surface being analyzed (Lambe et al., 1989). It is generally unconservative to extrapolate linear strength envelopes beyond the limits for which they were defined. It is, therefore, important that shear test data be acquired under normal loading conditions that are representative of the conditions being analyzed. For base liners this is zero to full height of the waste mass.

Utilization of Representative Materials. Designers often tend to use either published literature values or previously obtained test results for shear strengths. In such cases, their experience and judgment may assist them in selecting shear strength parameters for the purposes of preliminary design. It is highly recommended, however, that material-specific testing be performed to assist in preparing the final construction specifications, and/or to verify the actual materials delivered as part of a CQA program. The reason for this is that the variation in geosynthetic manufacturing parameters from job to job can have a significant effect on shear strength. The most significant of these is the degree of texturing on coextruded geomembranes. Figure 4 presents a graph showing the difference in peak and post-peak shear strengths obtained with two different degrees of texturing. Designers can use this concept to their advantage, as will be discussed later. Designers unaware of this issue may test a manufacturer's sample and obtain passing results, and then use GRI-GM 13 as a texturing specification. This would provide an extremely low-level requirement for texturing that may not achieve the same interface shear strength as the nice sample provided for initial testing by the manufacturer. The same principle may hold for geotextile-based products, whose fiber denier size, fiber type, degree of needling, etc. can influence its interface shear strength properties. The only way to be sure is to test the actual materials provided for construction.

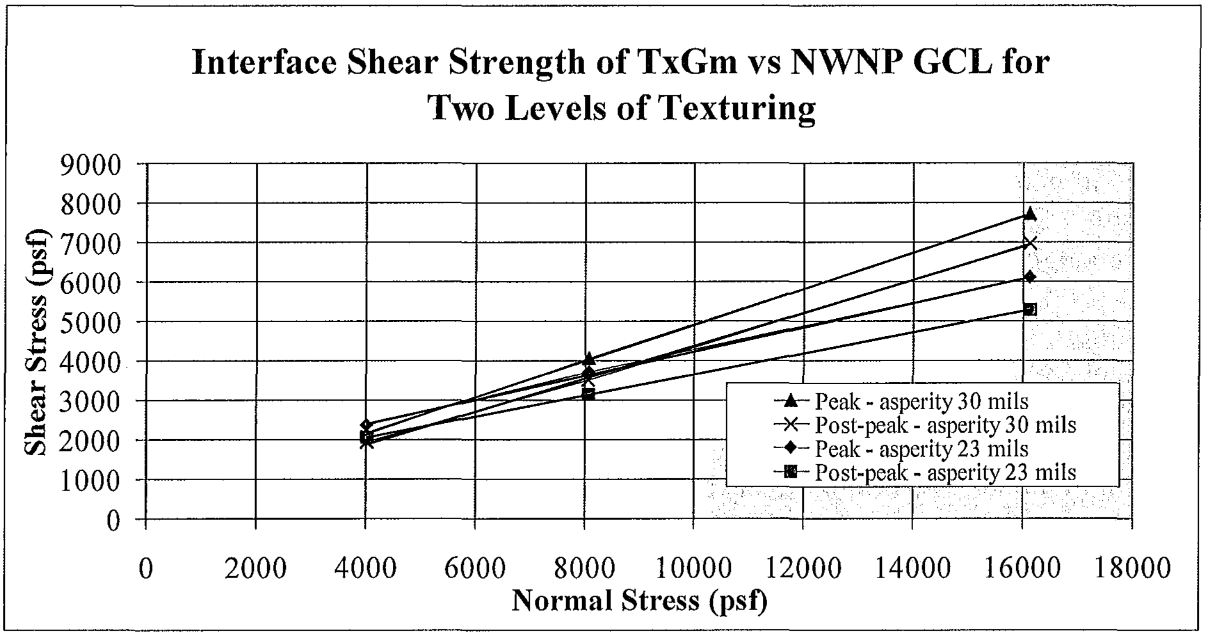


Figure 4 – Variation of Interface Shear Strength with Different Degrees of Geomembrane Texturing

Adjacent Materials and Consolidation Time. Using representative materials for direct shear testing refers not just to the materials for the interface being tested, but also to the adjacent materials. The use of realistic adjacent soil materials will typically provide slightly higher interface shear strengths than will, for example, the use of steel plates. In the same vein, Breitenbach and Swan (1999) show that longer load consolidation times result in a significant increase in interface shear strengths, apparently due to micro-scale load-induced deformation of the interface materials. Jones and Dixon (1998) question the used of the ring-shear apparatus for testing, because the narrow specimen of limited surface area on hard, smooth boundaries may not be representative of field conditions. These factors can affect both the peak and post-peak shear strength results.

Peak vs. Post-Peak vs. Residual Shear Strength. The highest level of shear strength measured in a direct shear test under a given normal load is defined as the peak strength. With continued shear displacement there is typically a loss of strength. The shear strength at any given displacement past the point of peak strength is referred to as "post-peak strength". The strength at which there is no further strength loss with continued displacement is called the "residual strength". Many of the most common direct shear devices do not allow enough displacement to occur that would enable true residual strength to be measured (e.g., see Stark et al., 1996). Therefore, in some cases it is not technically correct to refer to end-of-test conditions as representing the "residual" strength, but rather, to refer to "post-peak" strength while also specifying the amount of displacement. For the purposes of this paper, the lowest expected shear strength after significant deformation (typically more than 3-6 inches [70-150 mm]) is described as the residual shear strength. Shear strengths between the peak and residual shear strength are referred to as post-peak. This brings us then, to the main focus of this paper, which is whether it is appropriate to use peak or residual shear strengths (or something in between).

PART 2 – PEAK vs. RESIDUAL: THEORETICAL AND PRACTICAL CONSIDERATIONS

BACKGROUND DISCUSSION ON BRITTLE MATERIALS AND PROGRESSIVE FAILURE

Many, but not all, geosynthetic interfaces are strain softening. This highlights the essence of the peak vs. residual question. With a relatively short amount of deformation (typically less than 25 mm), the materials pass beyond peak strength into a lower post-peak shear strength, ultimately becoming what we call residual. In geotechnical engineering these shear strength characteristics are also sometimes called 'brittle' – brittle meaning that the material substantially decreases in strength after it is "broken", that is, has gone past peak strength. (Note that this has nothing to do with the tensile behavior of the material.) This behavior is in contrast to a ductile shear interface, which continues to deform after reaching its peak strength, but retains its strength close to the peak. An example of a brittle geosynthetic interface is an HDPE textured geomembrane against a geotextile, which produces a dramatic drop in strength after the peak strength is exceeded. An example of a ductile geosynthetic interface is a smooth PVC geomembrane against a geotextile (see data published by Hillman and Stark, 2001). Also, MSW waste is generally considered a ductile material in terms of shear strength (Kavazanjian, 2001).

As a progressive failure develops, the shear stresses are redistributed within the slope. This often involves the slow deformation of the failing mass over time, followed by an abrupt slide. If the critical plane supporting a slope is brittle, and for some reason part of it is stressed past its peak strength, then that part quickly becomes significantly weaker, which means it can carry less of the load. That in turn puts more of the load on other parts of the critical plane, which may in turn cause another part of that plane to become overstressed and exceed its peak strength. The continuation of this process is called progressive failure. At some point the entire system becomes overstressed and an abrupt failure occurs. This is the concern when there is a brittle interface.

Progressive failures have been characteristically noted for stiff clays, as described by LaRochelle (1989): "We have come to realize that we cannot count on the peak strength in this strain-softening material either for short- or long-term stability." Past landfill failures have been attributed to this same phenomenon (Schmucker and Hendron, 1998; Mazzucato et al., 1999; Stark et al., 2000), which holds significant potential for future failures (Gilbert and Byrne, 1996).

POTENTIAL CONDITIONS THAT MAY LEAD TO PROGRESSIVE FAILURE

Several reasons are provided below which explain why the peak strength of a bottom liner interface might unexpectedly be exceeded.

Non-Uniform Stress Distribution and Strain Incompatibility

Perhaps one of the most compelling reasons to be concerned about progressive failure in liner systems is that the stress distribution along the liner interface is not known. "It is impossible to obtain all of the necessary information in most cases" to perform a rigorous analysis of a progressive failure process (Tiande et al. 1999). "It is difficult to determine the available shear resistance along an interface exhibiting strain-softening behavior. It may be unsafe to assume that peak strength is available, while it may be excessively conservative and costly to assume that only the residual strength is available" (Gilbert and Byrne, 1996).

The complexities of stress distribution are affected by the type of loading and by pore pressures. According to Li and Lam (2001) ".. the development of progressive failure will also be different depending on whether failure is triggered by a rise in water table *[insert by author: namely, leachate]* or an increase in external loading *[insert by author: namely, continued waste stacking]*".

Reddy et al. (1996) present a most interesting finite-element modeling study that evaluates the stress distribution and deformations along a landfill liner system for an assumed landfill geometry. Their study compares smooth and textured interfaces for different stiffnesses of waste. Although their analysis did not model strain-softening behavior of the interfaces, the results provide valuable insight into stress and strain distribution. Some of the conclusions from their study are:

- The stiffness of the waste influences the distribution of interface stress and shear displacements. Stiffer waste puts more stress and strain on side slopes (especially the lower part of the slope). Softer (more compressible) waste puts more stress on the base liner below the highest part of the waste, and more strain accumulation towards the toe. The overall factor of safety, however, is not affected by the waste stiffness, assuming that no strain-softening of the interface shear strength occurs.

- The smooth interface with 11° friction reached its peak strength in a number of places along the interface in their example, even though the global factor of safety was 1.5. The textured interface did not approach its peak strength anywhere along the interface in their example, but had a factor of safety of over 4. This means that a typical stability evaluation that results in a factor of safety of 1.5 may actually result in areas of the critical interface achieving their peak strength and possibly going into a reduced post-peak strength.

A finite element study was performed by Filz et al. (2001) who reached conclusions similar to those obtained by Reddy et al. (1996). Filz et al. (2001) provided a compelling demonstration that a smooth clay-geomembrane interface exhibiting strain-softening characteristics might be inappropriate to analyze based on peak shear strengths. They showed that the distribution of mobilized shear stresses was not uniform along the base and side slope, and would result in progressive exceedence of peak strength. Their comparative analyses demonstrated that whereas a limit-equilibrium analysis based on peak strengths might result in FS = 1.6, the finite-element analysis would suggest impending failure (i.e. FS = 1.0). The same problems analyzed using residual shear strengths in limit-equilibrium analyses resulted in an average FS = 0.94. Furthermore, for a finite-element analysis to show FS = 1.5, the limit-equilibrium analysis based on peak strengths needed to show a FS of about 2.2, and the limit-equilibrium analyses using residual shear strength resulted in FS = 1.3.

Differences in the relative stiffnesses of the overlying waste as compared to that of the liner interface are also cited by Gilbert and Byrne (1996) as a significant potential cause of deformations along the liner interface that could lead to residual shear strengths.

Similar suppositions are made by Stark et al. (2000), who postulate that strain incompatibility between MSW and underlying interfaces can lead to progressive failure, as they believe was the underlying cause of the Rumpke landfill failure. The weaker lower interfaces may achieve post-peak strengths before the MSW ever achieves peak strength. After peak strength of the interfaces is achieved, the peak strength of the MSW may be mobilized at a time when the strength of the interfaces is reduced to the residual value. They state: "The greater the difference between the stress-strain characteristics of the MSW and the foundation soil or geosynthetic interfaces, the smaller the percentage of [peak] strength mobilized in the MSW and underlying materials." [1]

Unexpected Increases in Pore Pressure

The typical effect of pore pressures is to decrease the effective normal stress, which in turn decreases the effective shear strength, even as the shear stress that is driving instability remains unchanged. When pore pressures are introduced, the effective shear strength may be reduced to the point that the peak shear strength at that location is exceeded, at which point progressive failure can begin. This was what Schmucker and Hendron (1998) concluded was the triggering mechanism for the Rumpke landfill failure.

Seismic Loading

With seismic loading there is certainly the potential for deformation to occur along the critical failure plane, which can reduce the strength of the critical interface below its peak strength. In this regard the design practitioner needs to assess the potential for this type of deformation and, if the design earthquake is expected to produce deformation greater than about 20 mm, then the residual strength of that interface must be considered.

Construction Deformation

Construction conditions frequently result in temporary stability conditions with lower factors of safety than the completed fill scenario. To the author's knowledge, the effect of preliminary interface deformation at low normal loads on the subsequent shear strength at higher normal loads has only been documented in one recent study by Esterhuizen et al. (2001). They showed that for a smooth clay-geomembrane interface, deformations at low normal loads would partially, but not fully, reduce the peak strength of the interface at higher normal loads. They provide a very interesting "work-softening" model to describe this behavior in a manner that can be used in a finite-element analysis. Although their model fits the data very well, it is only applicable to the specific clay and geomembrane used for their study, and it is not know at this time how well their approach would work for other interfaces. This is an area for further research.

---

[1] For years now the author has heard the statement that the strain incompatibility between waste and liner systems could be a major consideration in selecting appropriate shear strengths. It is interesting, however, that some of the literature reports surprisingly low amounts of deformation required to reach the peak strength of the waste; on the order of only 40 mm for rigid-body deformation. See, for example, Eid et al. (2000), Stark et al. (1998), Mazzucato et al. (1999). Also Kavazanjian (2001) states his belief that strain compatibility with MSW is not nearly as significant an issue as has generally been supposed, based on direct- and simple-shear test results that show that the strains and deformations required to reach peak strength are comparable to those required for most soils.

Waste and Foundation Settlement

Over time there is substantial deformation and settlement of the waste that may cause unknown redistribution of stresses. The settlement of waste adjacent to a sideslope has often been noted as a source of downdrag forces, which may become great enough to exceed the peak strength of one of the slope liner interfaces. This phenomenon was cited by Stark and Poeppel (1994) as a mechanism contributing to the Kettleman Hills landfill failure, and is echoed in Gilbert and Byrne's (1996) theoretical study: "...it is more likely that the residual strength will be mobilized along the side slope rather than the buttress [bottom liner]", and they even go so far as to say "...it is unlikely that an average stress greater than the residual value could be mobilized along a typical side slope in a containment system." Likewise, foundation settlement has the potential to cause differential movements of the liner system.

Aging and Creep

Geosynthetic durability has been the subject of many papers and studies which address the ability of geosynthetics to maintain their physical properties as containment barriers, and to some extent as tensile reinforcement. Little has been published, however, regarding the long-term durability of shear interfaces such as, for example, the long-term dependence on the strength of geotextile fibers at interfaces with textured geomembranes, or within reinforced GCLs. Quantitative predictions regarding the long-term aging and creep potential of geosynthetic interfaces are certainly beyond the author's capacity, but are noted as an additional potential mechanism whereby the assumed peak strength of an interface might be reduced.

FIELD OBSERVATIONS

From the author's experience and his informal polling of industry representatives, two general field observations that have been made regarding deformations along geosynthetic interfaces on slopes:

- Slopes that were designed with robust interfaces using textured geomembrane or granular materials against geosynthetics, have not been observed to undergo tension or deformation.

- Slopes that had less brittle, but also less strong interfaces, such as a geotextile over a smooth geomembrane, have been observed to result in tension in the upper geosynthetic, presumably due to slippage along the interface which occurred as a result of downdrag forces.

It is worthwhile to note in the Gilbert and Byrne (1996) model that strain softening on the slope would generally only occur if the slope angle was greater than the peak friction angle of the lining material. Although unverified by the author, this may be a general guideline for estimating whether or not peak or residual shear strength would occur on a slope (excluding seismic forces). For example, on a 3(H):1(V) slope, perhaps a peak interface strength of 18° or more would maintain its peak strength, and an interface strength of less than that would have a higher potential for going into residual.

Given the large number of landfills constructed with geosynthetic bottom liner systems, it is quite surprising how few failures have actually been reported. Furthermore, none of the reported failures, to the author's knowledge, involved the progressive failure of a substantially brittle geosynthetic interface. Most of those failures have involved soil (including bentonite failures associated with unreinforced GCLs, which are ductile relative to shear strength). The best example of a pure geosynthetic failure that involved some degree of strain softening is the notorious Kettleman Hills failure, but the interfaces in that failure were fairly weak to begin with (all against smooth HDPE), and the initial factor of safety, even assuming peak strengths of the interfaces as they existed, was low, and below standard industry guidelines.

The conclusion of industry observations is that actual industry experience has not shown degradation of peak strength (i.e. progressive failure) to be a pervasive problem. Nonetheless, it definitely presents a potential problem that has on occasion bloomed into an unfortunate reality. It is, therefore, worth taking it into account by means of design and analysis considerations, which are discussed in the next section.

PART 3 - DESIGN APPROACHES

THE PEAK vs. RESIDUAL ISSUE IN THE CONTEXT OF THE DESIGN PROCESS

Many elements of a landfill are not designed, per se, but are largely dictated either by the owner's desires or by regulatory constraints. For example, the geometry of a landfill (boundaries, slopes, height, etc.) is often governed by an attempt to maximize the resource (i.e. volume) while meeting the constraints presented by conditional use permits, property line setbacks, maximum slope regulations and the like. Furthermore, the liner system is usually prescribed by regulation, at least in its fundamental requirements, and oftentimes by a default regulatory configuration.

In many cases then, the two major elements that influence a stability analysis are largely predetermined. That is, both the preferred landfill geometry and the liner system are more or less given to the "designer", who is charged with producing the "final design". From the point of view of slope stability, what is there left to do? Obviously the slope stability should be checked and verified. What does this mean and how is it done?

The first step in performing a slope stability analysis is to define the basis of the analysis. This is often documented in the project files as a Design Basis Memorandum (DBM), in which the following kinds of determinations are made:

- Will the analysis look at only the final configuration, or at interim operational configurations as well? (The latter option is highly recommended for risk management.)
- What unit weight will be assumed for the waste?
- What material strength values will be assumed for the different materials, and how will they be determined?
- Which pore-pressure scenarios will be evaluated?
- What will be the minimum acceptable factors of safety?
- Are seismic analyses required? If so, what approach will be used? How is the design earthquake defined? If a deformation approach is used, what is the maximum allowable deformation?

The results of the slope stability analyses will be:

- A static factor of safety (for each configuration analyzed).
- If a seismic analysis is required, the results will present either a potential magnitude of deformation along the critical slip plane, or a factor of safety for a simplified pseudo-static analysis.
- A description of the minimum required interface shear strength properties for the liner system construction.

It is this last point that makes slope stability analyses a design function rather than a mere geotechnical engineering exercise. It is essential that a clear linkage be made between the slope stability calculations and the ultimate project specifications, to ensure that the proper materials are provided during construction to meet the slope stability requirements. If the analysis results do not meet expectations, iterations of laboratory testing and/or alterations in slope geometry and/or liner materials may be required in order to achieve an acceptable design that can be adequately specified.

The design aspect of slope stability analyses becomes even more interesting when an additional constraint is put on the design criteria, namely to position the critical slip surface above the primary geomembrane. This is a common practice in Germany that is also employed by several design practitioners in the United States (and likely in other places as well, given the author's limited knowledge of practices worldwide). This design approach helps to ensure that, if for any reason slippage does occur, the barrier liner system will remain intact. Ensuring that the slip plane is above the primary geomembrane is not necessarily a simple matter; laboratory shear testing programs and iterations of slope stability analyses are often required in order to achieve acceptable results.

Implicit in the slope stability design and analysis process is the need to decide whether peak or residual shear strengths should be used. Though this is not generally an issue for waste materials, which are usually considered ductile, it is often a significant issue for liner system interfaces. This decision will significantly influence the calculated factor of safety. For seismic analyses, the influence is often less significant, because if the seismic analysis indicates deformation will occur, a prudent designer will use a post-peak shear strength (even as the question remains whether to use a deformation-based post-peak strength, or a true residual strength).

WHAT IS AN APPROPRIATE FACTOR OF SAFETY?

The author previously co-authored a paper whose title posed this same question concerning cover systems (Liu et al., 1997). That paper discussed assessing the degree of confidence in each of the variables that went into assessing the factor of safety, and assessing the potential risk and cost of a failure. This approach is espoused by Gilbert (pers. comm.) who believes that the factor of safety should be based on "uncertainties, assumptions, and the consequences of failure."

It is common in the literature to see geotechnical references that reiterate the idea that the greatest degree of uncertainty in performing slope stability analyses is the shear strength of the materials (e.g. Liu et al, 1997; Stark and Poeppel, 1994; Duncan, 1996). Given that the factor of safety is a reflection of uncertainty, it should logically reflect the degree of uncertainty in the shear strength properties. This was clearly noted by Terzaghi and Peck (1948, pg. 106):

> "The practical consequences of the observed differences between real soils and their ideal substitutes must be compensated by adequate factors of safety."

A commonly accepted value for the factor of safety in geotechnical engineering slope stability analyses is FS $\geq$ 1.5. Many engineers blindly accept this value while remaining ignorant of its basis. The origin of this value was the empirical result of analyzing the relative success and failure of dams that have been constructed over the past century. Experience proved that when an analysis was performed correctly, assuming reasonable and prudent material properties, an earthen structure with a factor of safety of 1.5 can be expected to remain stable even when some of its structural geometry and material properties have varied from those assumed in the analysis. Similarly, other values for an acceptable factor of safety have been established as general industry practice for other types of problems, such as bearing capacity (required FS generally between 2 and 5) or drainage applications (FS generally ranging from 1 to 20 depending on the problem).

It is also fundamental to the establishment of generally accepted factors of safety that analyses are performed correctly, and are based on prudent assumptions regarding material properties, geometry, unit weights, and pore pressures. Factors of safety are not intended to compensate for engineering errors or omissions. Indeed, the author has evaluated failures where the design factor of safety exceeded 1.5, which means that the original design neglected to take into account one or more critical factors.

With containment lining systems we meet a unique opportunity. We have a greater ability to know where the potential critical slip plane is, and can measure its shear strength characteristics more accurately than we can in a number of traditional geotechnical problems. We have far more knowledge of the geometry and shear strengths than when we are confronted with a natural slope, for example. Knowing where slippage is most likely to occur, we have to assess the implications for deformation. As described previously in this paper, we often don't really know if some deformation will occur, but experience from many analogous failures, along with the process of deduction, tells us that it *could* occur. Knowing this, we should at least be prepared to use the post-peak shear strength of the surface having the lowest peak strength.

SPECIFIC APPROACHES

Some specific design approaches, which the author has himself employed, are summarized below. This does not imply that others approaches do not exist, but simply that this paper is based on the author's experience.

1. <u>The Most Conservative Approach – Force the Slip Plane Above the Geomembrane and Use Residual Shear Strengths Everywhere the Slip Plane Occurs in the Liner System</u>. A simple and common way of achieving this objective is to use single-side textured geomembrane for the primary liner, and then cover it with a geotextile or geonet product. In nearly every case the author has been involved with (save a few inevitable exceptions), single-sided textured geomembrane (textured side down, of course) always caused whatever slippage occurred to take place on the top surface of the geomembrane, if it was covered with another geosynthetic. Even when directly covered by a granular material, it was often possible to make the bottom (textured) interface stronger than the smooth geomembrane/granular soil interface. In our experience there is often not a large difference between the peak and residual shear strength on smooth geomembrane interfaces with either other geosynthetics or granular soils, and these interfaces would not be considered very brittle. There may be some exceptions, such as a smooth HDPE geomembrane against a wet clay as described by Filz et al. (2001) for the Kettleman Hills failure analysis.

Some designs may need greater shear strength for interim construction and operational conditions than can be provided by a smooth geomembrane surface, so a double-sided textured geomembrane may be required. In this case the design condition of having the weak interface above the primary geomembrane may still be achieved by specifying a more aggressive texturing on the lower side of the geomembrane (see shear data presented in Figure 4).

If a designer is able to use the residual shear strength of the upper geomembrane interface and achieve acceptable factors of safety, this design can be very safe from the point of view of both stability and environmental containment. This approach is favored by Hullings and Sansome (1997), who recommend: "If possible, provide a slip plane and a stress-free geomembrane."

If true residual shear strengths are used for the analysis, and those strengths are measured with a degree of confidence that they represent worst case for the liner system interfaces, it follows that a lower-than-typical factor of safety can be allowed. Gilbert and Byrne (1996) suggest that a factor of safety simply greater than unity may be an adequate design criterion for analyses that assume residual shear strengths are the only strengths mobilized along the entire slip surface. Part of Gilbert's rationale (personal communication, 2001) is that even if a failure were induced for a slope analyzed with this criterion, things could not degenerate quickly, presuming the analysis were properly performed. The slope could subsequently be monitored and measures taken to reduce the deformation rate, if deemed necessary.

A similar recommendation is given by Stark et al. (1998): "...strain incompatibility can facilitate the development of slope instability because the geosynthetic interface may mobilize a post-peak or residual strength while the waste is mobilizing a strength that is significantly below the peak strength. This can be incorporated into a design by assigning a residual strength to the critical interface or slip surface and requiring a factor of safety, FS>1...Because field interface displacements and *effect(s) of progressive failure are not known [emphasis by author]*, a factor of safety, FS>1 with a ring shear residual interface strength assigned to all potential slip surfaces should be satisfied in addition to meeting regulatory requirements."

Filz et al. (2001) suggest that if true residual shear strengths are used for the analysis, then whatever factor of safety would normally be deemed appropriate for a given project could be reduced by the following reduction factor (*RF*):

$$RF = \tau_r / [\tau_r + 0.1(\tau_p - \tau_r)]$$

Where τ_r = residual shear strength, and τ_p = peak shear strength. They imply that the normally appropriate factor of safety would be determined based on considerations of uncertainty and consequences as described by Duncan (2000). Also, it should be noted that their discussion and recommendations were restricted to smooth-geomembrane/clay interfaces.

2. Safe Approach – Use Residual Shear Strength of the Interface with the Lowest Peak Strength. This approach could be the same as the above approach if the interface having the lowest shear strength happens to be above the primary geomembrane. If, due to overall slope stability constraints, the interface with the lowest peak strength is below the primary geomembrane (e.g. weak subgrade interface), this approach will still result in a very safe design relative to slope stability. It could, however, be less conservative in terms of environmental containment should deformation occur, causing a tear in the primary geomembrane. This approach is recommended by Gilbert and Byrne (1996) who "strongly recommended that the potential for instability be explored in a limit equilibrium analysis using residual strengths along all interfaces….It is strongly recommended that a factor of safety greater than one be achieved in all containment system slope designs, assuming residual strengths are mobilized along the entire slip surface."

   The same degree of factor of safety for this approach would apply as for Approach # 1 above. Holley et al. (1997) reported using residual shear strengths for a critical surface below the primary geomembrane in a steep canyon landfill, and obtaining operating factors of safety of 1.2 and an ultimate factor of safety of 1.4 for the final build-out. It is not clear if these were their minimum design criteria, or simply the results that they accepted.

3. Brute Strength Approach – This approach would employ very aggressive texturing to achieve high interface strengths, although the assumed strengths may be prorated by some factor to account for variability. The need to occasionally use this approach is suggested by Hullings and Sansome (1997): "Overall slope stability conditions often do not allow low interface strengths, so the interface strengths above the geomembrane cannot be much lower than the interface strength on the underside of the geomembrane."

   If the approach of high interface strength is used everywhere, and seismic analysis shows no deformation, an acceptable design basis may be to use peak shear strength with an adequately high factor of safety. How high is adequate is difficult to say, because the theoretical possibility of progressive failure still exists. The finite-element study performed by Filz et al. (2001) indicates that FS > 2 should be required for analyses based on peak strength of smooth-geomembrane/clay interfaces.

We have only the record of successful designs that were constructed based on peak strength to testify that the brute strength approach may be valid, but this does not demonstrate that it is conservative. The analysis should account for potential leachate build-up under worst case assumptions, for example after a post-closure maintenance period with substantial leachate still being generated, and the operations or leachate-collection layer completely clogged. Check that a submerged condition at the toe does not result in a reduction in shear strength (due to reduction in effective normal stresses) to the point that it fails the peak strength at the toe, which could lead to progressive failure through the rest of the fill (such as that discussed by Schmucker and Hendron, 1998).

4. Hybrid Approaches

   a) *Use Residual on the Side Slope and Peak on the Base.* To the author's knowledge, this approach was first documented in the literature by Stark and Poeppel (1994) in their review of the notorious Kettleman Hills failure. As they so aptly stated: "...it appears that peak and residual interface strengths should be assigned to the base and sideslopes, respectively, for design purposes." This was later echoed by Jones and Dixon (1998) from the U.K., who stated: "In some instances residual values may be appropriate on the side slope where large displacements are anticipated, used together with peak values on the base." In the author's opinion, this approach is a strong qualifier for accepting a traditional factor of safety in the range of 1.5 for ultimate build-out conditions (assuming unexpected pore-pressure scenarios are included in the evaluation), and 1.3 for operations.

   b) *Use Post-Peak Strength Values that Anticipate a Limited Amount of Deformation.* Shear strength reductions may occur due to relative deformations during construction, landfill operations, and waste settlement, but these deformations may be less than those which would lead to the minimum residual shear strength conditions. Also, based on their observation of numerous apparently successful facilities, design practitioners may consider peak shear strengths with an adequate factor of safety to be valid designs, while still wishing to incorporate an additional degree of conservatism by reducing the measured peak strength of the geosynthetic interfaces. These strength reductions would be applied to the side slope as well as the base. Use of this approach is suggested by Filz et al. (2001), who suggest using a mobilized strength that is higher than the residual by about 10% of the increment from residual to peak strength, and applying an appropriate factor of safety to this based on reliability concepts as described by Duncan (2000).

c) *Use Lower Waste Shear Strengths.* From the observation of trends published in the literature, shear strengths of 30° or more are commonly used for municipal solid waste. This level of shear strength has been documented as being generally conservative (e.g. Kavazanjian, 2001), but may require some amount of strain to become fully mobilized. As an approach to stability analyses designers may wish to reduce the mobilized strength of the waste material to more closely match the strain compatibility of the liner system.

The author has used all the above approaches in his own practice, which over the years has been based on improved levels of understanding. Currently (subject to change!) the author employs a combination of Approach #1 and #4 as his standard practice. That is, he usually defines a "design condition" which he believes will be the actual long-term conditions that interface shear strengths will experience. The decision as to what long-term shear strengths he selects is project-specific (there are many variations), and a complete discussion of this is beyond the scope of this paper. Suffice it to say that the decision is usually related to the criteria described for Approach #4. Next, the author follows the advice of Gilbert and Byrne (1996) and checks that the stability under the worst-case shear strength conditions (e.g. hydrated residual shear strength) results in FS > 1.0. This latter test is often the more significant.

A good example of the above approach is for bottom liner designs that involve the encapsulation of unreinforced bentonite between two geomembranes. The design scenario argues that most of the bentonite will remain dry for at least several centuries, and the basic slope stability analysis is performed on this basis. A second analysis is performed, however, to verify that the stability factor of safety is greater than unity even when all of the bentonite is under fully hydrated residual shear strength conditions. This example is more fully described in Thiel et al. (2001).

PART 4 – CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

- Many geosynthetic interfaces are highly strain-softening (i.e. "brittle"). The most common example is a textured geomembrane against some form of geotextile (whether it be a cushion, part of a geonet composite, or a GCL).

- There are mechanisms that can lead to exceedence of peak strength even though a correctly-performed slope stability analysis predicts a factor of safety greater than one. Examples of these mechanisms include:

  - Non-uniform mobilized stress distribution.

- Relative differences in stiffness between waste and liner materials.
- Unexpected pore pressures.
- Seismic loading.
- Deformation during construction.
- Waste settlement.
- Foundation settlement.
- Aging and creep of the geosynthetics.

➢ Exceedence of peak strength in a brittle interface can result in progressive failure.

➢ Based on field observation, most facilities designed with aggressive interface shear strengths are not experiencing post-peak shear strength, which means that the working shear stress is probably less than or equal to the peak strength. Only a few examples of progressive failure along geosynthetic interfaces have occurred in the industry, and these have not been along highly brittle interfaces, which means that the projects did not have high factors of safety to begin with, even assuming peak interface strengths.

➢ Several design approaches have been used over the years and the standard-of-practice is evolving. In the United States a preferred approach has not yet clearly emerged.

RECOMMENDATIONS FOR PRACTICE

➢ Designers and CQA firms should conduct material-specific testing of interfaces to verify that the materials specified and/or supplied for a project are realistic and meet the design requirements. Whoever commissions the testing should possess a skilled familiarity with the design objectives as well as the testing technique.

➢ Designers should attempt to position the critical slip plane above the primary geomembrane to the extent feasible for a given project. If a double-sided textured geomembrane is required for construction or operational stability, attempt to specify more aggressive texturing on the under side of the geomembrane.

➢ Using peak shear strengths on the landfill base, and residual shear strengths on the side slopes appears to be a successful state-of-the-practice in many situations.

➢ Designers should consider evaluating all facilities for stability using the residual shear strength along the geosynthetic interface that has the lowest peak strength. This would be an advisable risk-management practice for designers, even if the FS under these conditions is simply greater than unity.

- Regardless of the design assumptions, specify soil spreading by pushing up-slope only, and require close monitoring of LCRS and operations soil placement on slopes during construction to verify that relative shear displacement does not occur during construction. Exceptions to this practice should be allowed only with field tests and CQA verification.

- If LCRS or operations soils are placed as part of landfill operations, designers should assume the worst and automatically assume residual side-slope shear strength conditions will occur (and extra leakage rates as well). The reason for this is that construction by landfill operators is usually not controlled and monitored closely.

- Check stability for a potential leachate buildup, especially near the toe of the landfill.

RECOMMENDATIONS FOR FURTHER RESEARCH

- More finite element analyses at an academic level, such as those performed by Reddy et al. (1996) and Filz et al. (2001) would be warranted, to gain a better understanding of the threshold beyond which localized stress distributions might cause exceedence of peak shear resistance. Refinements in the analyses would include modeling the strain-softening behavior of the geosynthetic interfaces, and checking different types of interfaces and geometries. The results of these analyses might prove useful for establishing guidelines as to when peak strengths might be exceeded and when they might be maintained. Ultimately, the author envisions correlations between the FS determined by limit equilibrium analyses, ratios of peak interface strengths to waste fill strengths, and relative stiffnesses (somewhat as proposed by Gilbert and Byrne (1996), but more specific and less general), being used to estimate when and where peak vs. post-peak strengths would be reached at the interfaces.

- The monitoring of slope deformation on geosynthetic interfaces that are being buried by waste is recommended. One fairly easy way to do this would be to use the simple tell-tale technique employed for the Cincinnati cover demonstration project (Koerner et al., 1996), though this would require participation by landfill owners and operators. This avenue of research echoes that suggested by Gilbert and Byrne (1996), who state: "Future research should focus on measuring deformations and mobilized shear resistances in existing waste containment facilities."

- The monitoring of pore pressures in the LCRS above liner systems, with the reporting of the worst-case conditions, would provide valuable information regarding long term conditions in landfills. Unfortunately, any high pressures would likely result in a permit violation at many facilities, so it is improbable that an existing owner will voluntarily monitor high pressures, much less report them. We are therefore left with only orphan or Superfund sites as a possible basis for monitoring. Because of this limitation, participation in international waste conferences is increasingly valuable.

- Additional laboratory testing, conducted on various types of interfaces, would be useful to assess the impact of interface deformations at low normal loads on the peak strength reductions at higher normal loads.

REFERENCES

Abramson, L.W., Lee, T.S., Sharma, S., and Boyce, G.M. (1996). Slope Stability and Stabilization Methods. John Wiley & Sons, Inc. New York.

Breitenbach, A.J. (1997) "Overview Study of Several Geomembrane Liner Failures Under High Fill Load Conditions." Proc. of Geosynthetics '97, IFAI, Vol. 2, pp. 1045-1061.

Breitenbach, A.J. and Swan, R.H. (1999) "Influence of High Load Deformations on Geomembrane Liner Interface Strengths." Proc. of Geosynthetics '99, IFAI, Vol. 1, pp. 517-529.

Brink, D., Day, P.W. and DuPreez, L. (1999) "Failure and Remediation of Bulbul Drive Landfill: Kwazulu-Natal, South Africa." Proc. Sardinia '99 Seventh International Waste Management and Landfill Symposium, CISA, Vol. III, pp. 555-562.

Duncan, J.M. (1996) "State of the Art: Limit Equilibrium and Finite-Element Analyses of Slopes." J. of Geotechnical Engineering, ASCE, Vol. 122, No. 7, May, pp. 577-596.

Duncan, J.M. (2000) "Factors of Safety and Reliability in Geotechnical Engineering" J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 4, Apr., pp. 307-316.

Eid, H.T., Stark, T.D., Evans, W.D. and Sherry, P.E. (2000) "Municipal Solid Waste Slope Failure. I: Waste and Foundation Soil Properties." J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 5, May, pp. 397-407.

Esterhuizen, J.B., Filz, G.M., and Duncan, J.M. (2001) "Constitutive Behavior of Geosynthetic Interfaces" J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 10, Oct., pp. 834-840.

Filz, G.M., Esterhuizen, J.B., and Duncan, J.M. (2001) "Progressive Failure of Lined Waste Impoundments" J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 10, Oct., pp. 841-848.

Gilbert, R.B. and Byrne, R.J. (1996) "Strain-Softening Behavior of Waste Containment System Interfaces." Geosynthetics International, IFAI, Vol. 3, No. 2, pp. 181-203.

Gilbert, R.B. (2001) Personal communication with the author.

Hendron, D.M., Fernandez, G., Prommer, P.J., Giroud, J.P., and Orozco, L.F. (1999) "Investigation of the Cause of the 27 September 1997 Slope Failure at the Dona Juana Landfill" Proc. Sardinia '99 Seventh International Waste Management and Landfill Symposium, CISA, Vol. III, pp. 545-554.

Hillman, R.P. and Stark, T.D. (2001) "Shear Strength Characteristics of PVC Geomembrance-Geosynthetic Interfaces." Geosynthetics International, IFAI, Vol. 8, No. 2, pp. 135-162.

Holley, K., Richardson, J., and Sadlier, M. (1997) "Design and Construction to Optimise Landfill Stability at the Nent Landfill, Hong Kong." Proc. Sardinia '99 Seventh International Waste Management and Landfill Symposium, CISA, Vol. III, pp. 565-574.

Hullings, D.E. and Sanome, L.J. (1997) "Geomembrane Anchor Trenches." J. of Geotextiles and Geomembranes, Elsevier, Vol. 15, Nos. 4-6, pp. 403-417.

Jones, D.R.V. and Dixon, N. (1998) "Shear Strength Properties of Geomembrane/Geotextile Interfaces." J. of Geotextiles and Geomembranes, Elsevier, Vol. 16, Nos. 1, pp. 45-71.

Kavazanjian, E. (2001) "Mechanical Properties of Solid Waste." Proc. Sardinia '01 Eighth International Waste Management and Landfill Symposium, CISA, Vol. III, pp. .

Koerner, R.M., Carson, D.A., Daniel, D.E., and Bonaparte, R. (1996) "Current Status of the Cincinnati GCL Test Plots" Proceedings of the 10th GRI Conference, Field Performance of Geosynthetics and Geosynthetic Related Systems. Geosynthetic Research Institute, Drexel University, Philadelphia, PA, pp. 147-175.

Koerner, R.M. and Daniel, D.E. (1997) Final Covers for Solid Waste Landfill and Abandoned Dumps. ASCE Press, Reston, VA.

Koerner, R.M. and Soong, T.Y. (2000) "Leachate in Landfills: The Stability Question." Geotextiles and Geomembranes, Elsevier, Vol. 18, pp. 293-309.

LaRochelle, P. (1989) "Problems of Stability: Progress and Hopes." The Art and Science of Geotechnical Engineering. Ed. by Corning et al., Prentice Hall, N.J., pp. 269-290.

Li, K.S. and Lam, J. (2001) Discussion of "Evolution of Progressive Failure of Landslides." J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 1, Jan., pg. 98.

Liu, C.N., Gilbert, R.B., Thiel, R.S., and Wright, S.G. (1997). "What is an Appropriate Factor of Safety for Landfill Cover Slopes?" Conference Proceedings from Geosynthetics '97. IFAI, Roseville, MN, pp. 481-496.

Mazzucato, A., Somonini, P. and Colombo, S. (1999) "Analysis of Block Slide in a MSW Landfill." Proc. Sardinia '99 Seventh International Waste Management and Landfill Symposium, CISA, Vol. III, pp. 537-544.

NAVFAC (1982) Soil Mechanics. Design Manual 7.1, Department of the Navy, Naval Facilities Engineering Command, Alexandria, VA.

Reddy, K.R., Kosgi, S. and Motan, S. (1996) "Interface Shear Behavior of Landfill Composite Liner Systems: A Finite Element Analysis." Geosynthetics International, IFAI, Vol. 3, No. 2, pp. 247-275.

Schmucker, B.O. and Hendron, D.M. (1998) "Forensic Analysis of the 9 March 1996 Landslide at the Rumpke Sanitary Landfill, Hamilton County, Ohio." Proc. of the 12th GRI Conference, Lessons Learned from Geosynthetic Case Histories, Geosynthetic Institute, Folsom, PA, pp. 269-295.

Soong, T.Y., Hungr, O., and Koerner, R.M. (1998) "Stability Analyses of Selected Landfill Failures by 2-D and 3-D Methods" Proc. of the 12th GRI Conference, Lessons Learned from Geosynthetic Case Histories, Geosynthetic Institute, Folsom, PA, pp. 296-329.

Stark, T.D. and Eid, H.T. (1998) "Performance of Three-Dimensional Slope Stability Methods in Practice" J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No.11, Nov., pp. 1049-1060.

Stark, T.D., Arellano, D., Evans, W.D., Wilson, V.L., and Gonda, J.M. (1998) "Unreinforced Geosynthetic Clay Liner Case History." Geosynthetics International, IFAI, Vol. 5, No. 5, pp. 521-544.

Stark, T.D., Eid, H.T., Evans, W.D. and Sherry, P.E. (2000) "Municipal Solid Waste Slope Failure. II: Stability Analyses." J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 5, May, pp. 408-419.

Stark, T.D., and Poeppel, A.R. (1994) "Landfill Liner Interface Strengths from Torsional Ring Shear Tests." J. of Geotechnical Engineering, ASCE, Vol. 120, No. 3, March, pp. 597-615.

Terzaghi, K. and Peck, R. (1948) Soil Mechanics in Engineering Practice. John Wiley & Sons, New York, NY.

Thiel, R., Daniel, D.E., Erickson, R., Kavazanjian, E., and Giroud, J.P. (2001) *GundSeal GCL Design Manual.* Published by GSE, Houston, TX.

Tiande, M., Chongwu, M, and Shengzhi, W. (1999) "Evolution Model of Progressive Failure of Landslides." J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 125, No. 10, Oct,, pp. 827-831.

ATTACHMENT III.7.H

BOWLES, JOSEPH E. 1977. *FOUNDATION ENGINEERING ANALYSIS AND DESIGN, 2ND EDITION.* UNITED STATES: MCGRAW HILL BOOK COMPANY

FOUNDATION ANALYSIS AND DESIGN

Joseph E. Bowles

Table 3-2. Standard designation and sizes for drill rods and casing

Drill rod	OD, in	Casing and core barrel	Core-barrel-bit OD, in	Approx. diam of borehole,* in	Diam of core sample, in
E	$1\frac{5}{16}$	EX	$1\frac{7}{16}$	$1\frac{1}{2}$	$\frac{7}{8}$
A	$1\frac{5}{8}$	AX	$1\frac{7}{8}$	2	$1\frac{1}{8}$
B	$1\frac{7}{8}$	BX†	$2\frac{3}{8}$	$2\frac{1}{2}$	$1\frac{5}{8}$
N	$2\frac{3}{8}$	NX	$1\frac{15}{16}$	3	$2\frac{1}{8}$

* Diameter of borehole is very nearly the ID of the casing.
† In soft or fractured rock, BX or larger cores are preferred.

The SPT was originally developed for cohesionless soils so that samples would not have to be taken. The test has evolved to the current practice of routinely determining N for all soils. In the zones of particular interest from about 2.5 ft or 1 m below ground surface to considerable depth below the estimated base of the foundation the test is performed every 2.5 ft or 1 m depth increment. At considerable depths where the boring becomes more informational the depth increment for testing is often increased to 5 ft or 2 m.

Empirical correlations between N and various soil properties have been attempted for cohesionless soils (Table 3-3). Table 3-3 should be used cautiously; for example, a "loose" soil with a range of D_r between 15 and 35 percent places rather arbitrary numbers on a rather tenuous description of a soil.

Table 3-3. Empirical values for ϕ, D_r, and unit weight of granular soils based on the standard penetration number with corrections for depth and for fine saturated sands

Description		Very loose		Loose		Medium		Dense		Very dense	
Relative density D_r*	0		0.15		0.35		0.65		0.85		1.00
Standard penetration no. N			4		10		30		50		
Approx. angle of internal friction ϕ°†	25°–30°		27–32°		30–35°		35–40°		38–43°		
Approx. range of moist unit weight, (γ) pcf (kN/m^3)		70–100‡ (11–16)		90–115 (14–18)		110–130 (17–20)		110–140 (17–22)		130–150 (20–23)	

* USBR [Gibbs and Holtz (1957)].
† After Meyerhof (1956). $\phi = 25 + 25D_r$ with more than 5 percent fines and $\phi = 30 + 25D_r$ with less than 5 percent fines. Use larger values for granular material with 5 percent or less fine sand and silt.
‡ It should be noted that excavated material or material dumped from a truck will weigh 70 to 90 pcf. Material must be quite dense and hard to weigh much over 130 pcf. Values of 105 to 115 pcf for nonsaturated soils are common.

ATTACHMENT III.7.I

RICHARDSON, CLINTON P., PHD., PE 2009.

MUNICIPAL LANDFILL DESIGN CALCULATIONS: AN ENTRY LEVEL MANUAL OF PRACTICE. CALIFORNIA: UBUILDABOOK, LLC.

Municipal Landfill Design Calculations

An Entry Level Manual of Practice

Clinton P. Richardson, PhD. PE.

Chapter 28 Side-slope Liner Stability

Problem Statement

Liner stability or side-slope slippage is complicated for multi-layered liner and collection system. A unit load of waste gravitationally induces shear stress and a portion of stress is transmitted by means of friction to the geosynthetics components beneath. The difference between frictional components must be carried by the particular component in the form of tensile stress and then compared to the component's yield stress for the resulting factor of safety. The portion transmitted to upper component is then propagated to the next component in the multilayered sequence. An unbalanced portion is eventually transmitted to the subgrade soil beneath the lower geosynthetic. If mass failure is going to occur, it will seek the interface with the lowest friction angle. The liner stability method is simply a resolution of shear stresses Koerner, 1994).

Design Objective

Calculate the tensile stresses and shear stresses carried by the upper and lower geosynthetic components and estimate the factor of safety.

Design Equations

Figure 1 shows a schematic of a multi-layered liner and resolution of forces assuming a single waste lift thickness.

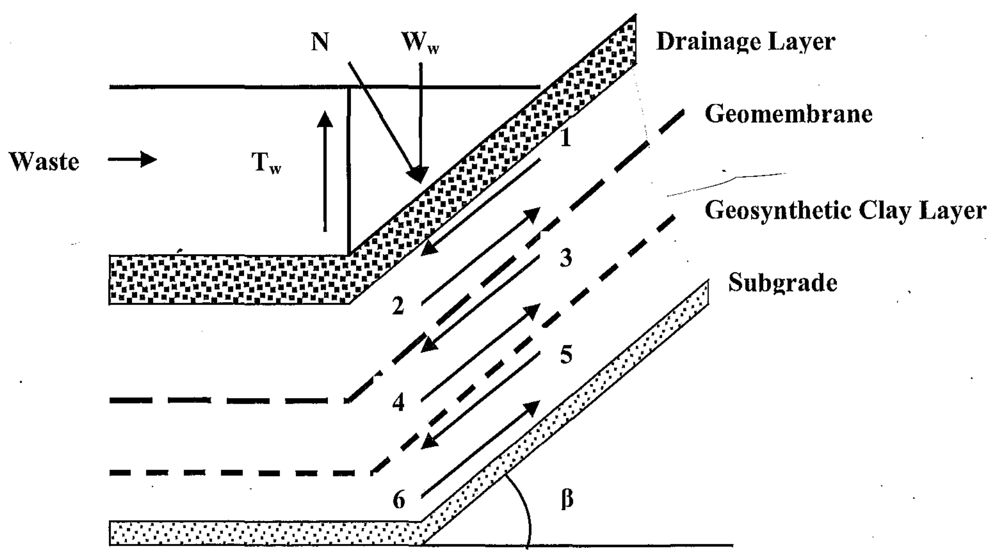


Figure 1: Resolution of Shear Forces in A Multi-layered Landfill Barrier Liner (adapted from Koerner, 1990).

The simple barrier system consists of a geomembrane underlain by a geosynthetic clay liner (GCL). The procedure may be extended to any number of interfaces, such a geotextile, geomembrane, clay liner, etc. Time is assumed to be sufficiently long between waste lifts that system readjustment will occur and either equilibrium or failure will exist. A unit width is assumed. The numbers 1 through 6 shown in the figure represent the forces that must be resolved sequentially.

The weight of a unit width of compacted waste is given by

$$W_w = \frac{1}{2}\gamma_w H \frac{H}{\tan\beta} \qquad \text{Eq. 1}$$

where

W_w = weight of waste per unit width (lb_f/ft or kN/m)

H = lift height (ft or m)

β = slope angle (°)

γ_w = unit weight of waste (lb_f/ft^3 or kN/m^3)

The frictional resistance along the waste edge is given by

$$T_w = \sigma_h \tan\phi_w H = K_o \sigma_v \tan\phi_w H \qquad \text{Eq. 2}$$

$$K_o = (1 - \sin\phi_w) \qquad \text{Eq. 3}$$

$$\sigma_v = \frac{1}{2}\gamma_w H \qquad \text{Eq. 4}$$

where

T_w = frictional resistance force per unit width (lb_f/ft or kN/m)

σ_h = horizontal stress of waste lift (lb_f/ft^2 or kN/m^2)

ϕ_w = waste fiction angle (°)

K_o = coefficient of earth pressure at rest (unitless)

σ_h = vertical stress of waste lift (lb_f/ft^2 or kN/m^2)

The net weight of the waste is the difference between the downward acting waste weight and the upward acting resistance force, or

$$W_{net} = W_w - T_w \qquad \text{Eq. 5}$$

The net weight can now be resolved into its two components: a normal force component acting perpendicular to the slope and a parallel force component acting downslope, or

$$N = W_{net}\cos\beta \qquad \text{Eq. 6}$$

$$P = W_{net} \sin \beta \qquad \text{Eq. 7}$$

where

N = normal force component of net weight (lb_f/ft or kN/m)

P = parallel force component of net weight (lb_f/ft or kN/m)

This latter force component is assumed to be dissipated through the drainage layer (Koerner, 1990). The forces that must be determined are a function of the normal force and the frictional resistance provided by the respective interface; for example, in the first force couple, the following relationships hold:

$$F_1 = N \tan \delta_1 = (W_{net} \cos \beta) \tan \delta_1 \qquad \text{Eq. 8}$$

$$F_2 = N \tan \delta_2 = (W_{net} \cos \beta) \tan \delta_2 \qquad \text{Eq. 9}$$

where

δ_1 = drainage layer friction angle with respect to the upper geomembrane surface (°)

δ_2 = lower geomembrane surface friction angle with respect to the upper GCL surface (°)

If F_1 exceeds F_2, then the geomembrane is in tension. The force difference must be carried by the geomembrane. The actual stress in the geomembrane is given by

$$\sigma_{\text{actual}_{\text{geomembrane}}} = \left(\frac{F_1 - F_2}{t_{geo}} \right) \qquad \text{Eq. 10}$$

where

$\sigma_{\text{actual geomembrane}}$ = actual stress in geomembrane (lb_f/ft^2 or kN/m^2)

t_{geo} = geomembrane thickness (ft or m)

The factor of safety for the geomembrane against failure in tension is

$$FS_{\text{geomembrane}} = \frac{\sigma_{\text{yield}}}{\sigma_{\text{actual}_{\text{geomembrane}}}} \qquad \text{Eq. 11}$$

where

σ_{yield} = allowable geomembrane stress at yield (lb_f/ft^2 or kN/m^2)

The allowable geomembrane stress at yield is usually given in terms of lb_f/in^2 or kN/m^2 or kPa based on a wide-width tensile test (ASTM D 4885-01 Determining Performance Strength of Geomembranes by the Wide Width Strip Tensile Method).

The frictional shear force acting on the lower geomembrane surface, or F_2, is equal and opposite to the frictional shear force above the GCL surface, or F_3; thus,

$$F_2 = N \tan \delta_2 = F_3 \qquad \text{Eq. 12}$$

The frictional shear force acting on the lower GCL is given by

$$F_4 = N \tan \delta_4 \qquad \text{Eq. 13}$$

where

δ_4 = friction angle between the lower GCL surface and the subgrade soil

The difference between F_3 and F_4 determines the tensile force carried by the GCL. If negative, the GCL is not in tension. If positive, then the GCL is in tension and a factor of safety must be evaluated based on the wide width strength test (ASTM D 6768-04 Standard Test Method for Tensile Strength of Geosynthetic Clay Liners). The force difference must be carried by the geomembrane. The actual stress in the GCL is given by

$$\sigma_{\text{actual}_{\text{GCL}}} = \left(\frac{F_3 - F_4}{t_{GCL}} \right) \qquad \text{Eq. 14}$$

where

$\sigma_{\text{actual GCL}}$ = actual stress in GCL (lb_f/ft^2 or kN/m^2)

t_{geo} = GCL thickness (ft or m)

The factor of safety for the GCL against failure is

$$FS_{GCL} = \frac{\sigma_{\text{yield}}}{\sigma_{\text{actual}_{\text{GCL}}}} \qquad \text{Eq. 15}$$

where

σ_{yield} = allowable GCL stress at yield (lb_f/ft^2 or kN/m^2)

If $\delta_2 = \delta_4$, then $F_4 = F_2 = F_3$. If the lower frictional shear force exceeds the upper frictional shear force for a given interface, then the factor of safety is infinite and only a value of the upper frictional shear force will be mobilized at the upper surface of the next interface below. This procedure is repeated for multiple interfaces until the lower most interface is encountered, i.e. a compacted subgrade or compacted clay. For compacted clay, special attention must be paid to its short-term friction angle *versus* its long-term friction angle with respect to the interface above. Compacted clay can consolidate with overburden stress and expel moisture, which can reduce the friction between it and the contact surface above, potentially placing the upper geosynthetic in tension.

Design Example #1

Evaluate the maximum stresses, if any, in the landfill liner system described in Figure 1 consisting of a textured 60 mil HDPE/non-woven, needle-punched Bentomat® GCL/USCS SP compacted subgrade sequence. The following data may be assumed:

$H = 10$ ft (3.0 m)

$\beta = 18.43\,^{\circ}$ (3H:1V)

$\gamma_W = 60$ lb_f/ft^3 or (9.4 kN/m^3)

$\phi_W = 20\,^{\circ}$

$\delta_1 = 18\,^{\circ}$

$\delta_2 = 16\,^{\circ}$

$\delta_4 = 30\,^{\circ}$

$\sigma_{allow\ geomembrane} = 2100$ lb_f/in^2 (14,478 kN/m^2)

$T_{GCL} = 100$ lb_f/in (17.5 kN/m)

$t_{GCL} = 0.25$ in (6.4 mm)

Solution:

The critical interface lies between the HDPE geomembrane and the GCL based on the magnitude of the respective friction angles. The following parameters are calculated:

$W_w = 9.0 \times 10^3$ lb_f/ft (131 kN/m)	Eq. 1
$K_o = 0.658$	Eq. 3
$\sigma_v = 300$ lb_f/ft^2 (14.4 kN/m^2)	Eq. 4
$\sigma_h = 197$ lb_f/ft^2 (9.4 kN/m^2)	Eq. 2
$T_W = 718$ lb_f/ft (10.5 kN/m)	Eq. 2
$W_{net} = 8282$ lb_f/ft (120.9 kN/m)	Eq. 5
$N = 7857$ lb_f/ft (114.7 kN/m^2)	Eq. 6
$F_1 = 2553$ lb_f/ft (37.3 kN/m)	Eq. 8

ATTACHMENT III.7.J

GSE LINING TECHNOLOGY, INC.,

GSE HD TEXTURED PRODUCT DATA SHEET

Appendix B - Minimum Testing Frequencies and Properties for GSE Geomembranes

MINIMUM PROPERTIES FOR GSE HD TEXTURED

TESTED PROPERTY	TEST METHOD	FREQUENCY	MINIMUM VALUE				
Product Code			HDT 030G000	HDT 040G000	HDT 060G000	HDT 080G000	HDT 100G000
Thickness, (minimum average) mil (mm)	ASTM D 5994	every roll	29 (0.73)	38 (0.96)	57 (1.45)	76 (1.93)	95 (2.41)
Lowest individual for 8 out of 10 values			27 (0.69)	36 (0.91)	54 (1.40)	72 (1.80)	90 (2.30)
Lowest individual for any of the 10 values			26 (0.66)	34 (0.86)	51 (1.30)	68 (1.73)	85 (2.16)
Density, g/cm^3	ASTM D 1505	200,000 lb	0.94	0.94	0.94	0.94	0.94
Tensile Properties (each direction)(1)	ASTM D 6693, Type IV	20,000 lb					
Strength at Break, lb/in-width (N/mm)	Dumbell, 2 ipm		45 (8)	60 (11)	90 (16)	120(21)	150 (27)
Strength at Yield, lb/in-width (N/mm)			63 (11)	84 (15)	126 (22)	168 (29)	210 (37)
Elongation at Break, %	G.L. = 2.0 in (51 mm)		100	100	100	100	100
Elongation at Yield, %	G.L. = 1.3 in (33 mm)		12	12	12	12	12
Tear Resistance, lb (N)	ASTM D 1004	45,000 lb	21 (93)	28 (125)	42 (187)	56 (249)	70 (311)
Puncture Resistance, lb (N)	ASTM D 4833	45,000 lb	45 (200)	60 (267)	90 (400)	120 (534)	150 (667)
Carbon Black Content, %	ASTM D 1603*/4218	20,000 lb	2.0	2.0	2.0	2.0	2.0
Carbon Black Dispersion	ASTM D 5596	45,000 lb	+Note 1	+Note 1	+Note 1	+Note 1	+Note 1
Asperity Height	GRI GM 12	second roll	+Note 2	+Note 2	+Note 2	+Note 2	+Note 2
Notched Constant Tensile Load(2), hr	ASTM D 5397, Appendix	200,000 lb	300	300	300	300	300
REFERENCE PROPERTY	**TEST METHOD**	**FREQUENCY**	**NOMINAL VALUE**				
Oxidative Induction Time, min	ASTM D 3895, 200° C; O_2, 1 atm	200,000 lb	>100	>100	>100	>100	>100
Roll Length(3) (approximate), ft (m)	Standard Textured		830 (253)	700 (213)	520 (158)	400 (122)	330 (101)
Roll Width(3), ft (m)			22.5 (6.9)	22.5 (6.9)	22.5 (6.9)	22.5 (6.9)	22.5 (6.9)
Roll Area, ft^2 (m^2)			18,674 (1,735)	15,750 (1,463)	11,700 (1,087)	9,000 (836)	7,425 (690)

NOTES:
- +Note 1: Dispersion only applies to near spherical agglomerates. 9 of 10 views shall be Category 1 or 2. No more than 1 view from Category 3.
- +Note 2: 10 mil average. 8 of 10 readings ≥7 mils. Lowest individual ≥ 5 mils.
- GSE HD Standard Textured is available in rolls weighing about 4,000 lb (1,800 kg).
- (1)The combination of stress concentrations due to coextrusion texture geometry and the small specimen size results in large variation of test results. Therefore, these tensile properties are minimum average values.
- (2)NCTL for HD Textured is conducted on representative smooth membrane samples.
- All GSE geomembranes have dimensional stability of ±2% when tested with ASTM D 1204 and LTB of <-77° C when tested with ASTM D 746.
- (3)Roll lengths and widths have a tolerance of ± 1%.
- *Modified.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 8: EROSION CALCULATIONS

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ATTACHMENTS

III.8.E	NATURAL RESOURCES CONSERVATION SERVICE. 2002. FIGURE 7 - FLAT SMALL GRAIN EQUIVALENTS OF UNGRAZED BLUE GRAMA AND BUFFALOGRASS IN *NATIONAL AGRONOMY MANUAL*, 190-V-NAM, THIRD EDITION, OCTOBER 2002, PART 502, WIND EROSION. WASHINGTON, D.C.: UNITED STATES DEPARTMENT OF AGRICULTURE.
III.8.F	NATURAL RESOURCES CONSERVATION SERVICE. 1998. SUBPART G – EXHIBITS (C=150, I=134, K=1.0) IN *NATIONAL AGRONOMY MANUAL*, 190-V-NAM, THIRD EDITION, JANUARY 1998. WASHINGTON, D.C.: UNITED STATES DEPARTMENT OF AGRICULTURE.

APPLICATION FOR PERMIT
DNCS ENVIRONMENTAL SOLUTIONS

VOLUME III: ENGINEERING DESIGN AND CALCULATIONS
SECTION 8: EROSION CALCULATIONS

1.0 INTRODUCTION

DNCS Environmental Solutions (DNCS Facility) is a proposed Surface Waste Management Facility for oil field waste processing and disposal services. The proposed DNCS Facility is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.36 NMAC, administered by the Oil Conservation Division (OCD). The Facility has been designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, DNCS Properties, LLC.

1.1 Description

The DNCS site is comprised of a 562-acre ± tract of land located south of NM 529 in portions of Section 31, Township 17 South, Range 33 East; and in the northern half of Section 6, Township 18 South, Range 33 East, Lea County, NM. A portion of the 562-acre tract is a drainage feature that will be excluded from development. The drainage feature includes a 500-ft setback and totals 67 acres ±. The DNCS Facility will include two main components; a liquid oil field waste Processing Area (177 acres ±), and an oil field waste Landfill (318 acres ±); therefore the DNCS Facility comprises 495 acres ±. Oil field wastes are anticipated to be delivered to the DNCS Facility from oil and gas exploration and production operations in southeastern NM and west Texas. The Site Development Plan provided in the **Permit Plans, Sheet 3**, identifies the locations of the Processing Area and Landfill facilities.

2.0 DESIGN CRITERIA

The purpose of the Erosion Calculations is to determine potential soil losses due to wind and rainfall erosion for the DNCS Facility Landfill during operations and following final cap installation. Erosion calculations project that the soil loss from rainfall is approximately 4.96 tons per acre per year, which is below the established criterion of 5.0 tons/acre/year. The wind erosion loss from the site is estimated at 1.2 tons per acre per year, which is also below the established criterion of 2.5 tons/acre/year. The total soil loss from the site potentially caused by water and wind erosion is calculated at 6.16 tons per acre per year.

The attached calculations were used to assess the potential for wind and rainfall erosion at the DNCS Facility. These conservative calculations were also used to determine if additional erosion control measures are required. Evaluation of erosion of the final cover surface was based on the following design criteria:

1. The New Mexico Energy, Minerals, and Natural Resources Department Oil and Gas Rules, 19.15.36 NMAC, Surface Waste Management Facilities Closure and Post-Closure Requirements. More specifically, 19.15.36.18.D.(2)(a) NMAC states:

   "*The operator shall properly close landfill cells, covering the cell with a top cover pursuant to Paragraph (8) of Subsection C of 19.15.36.14 NMAC, with soil contoured to promote drainage of precipitation; side slopes shall not exceed a 25 percent grade (four feet horizontal to one foot vertical), such that the final cover of the landfill's top portion has a gradient of two percent to five percent, and the slopes are sufficient to prevent the ponding of water and erosion of the cover material.*"
2. The final cover crown of the landfill consists of a maximum 5% slope.
3. The sideslopes of the landfill consist of a 4H:1V slope with drainage benches.
4. The longevity of any temporary erosion protection shall be a minimum of 24 months for the 5% slope and 36 months for the 4H:1V slope.
5. The design erosion rate shall not exceed the 12-inch soil thickness of the landfill erosion/vegetative layer of the final cover.
6. The final cover has been conservatively assumed to have poor vegetation (50% coverage) established.
7. A soil loss tolerance target erosion rate is established at 5.0 tons/acre/year for rainfall erosion; and 2.5 tons/acre/year for wind erosion. The target values represent the erosion at which a management system is or is not sustainable. The target values are typical for non-farm application of erosion calculations (NRCS, 1962).
8. The Operations, Inspection, and Maintenance Plan (**Volume II.1**) provides routine corrective measures to address cover erosion when the site is under construction. The Closure/Post-closure Plan details specific plans to address potential erosion of the final cap.

3.0 RAINFALL EROSION LOSS CALCULATIONS

North American Green, Inc. Slope Erosion Protection Module **(Attachment III.8.A)** was used to model the soil erosion rate from the DNCS Landfill final cover due to rainfall. The City of Alamogordo database was selected based on its similar climate to the DNCS site. This program uses the Revised Universal Soil Loss Equation (RUSLE). The equation is as follows:

A = R x K x LS x C

Where:

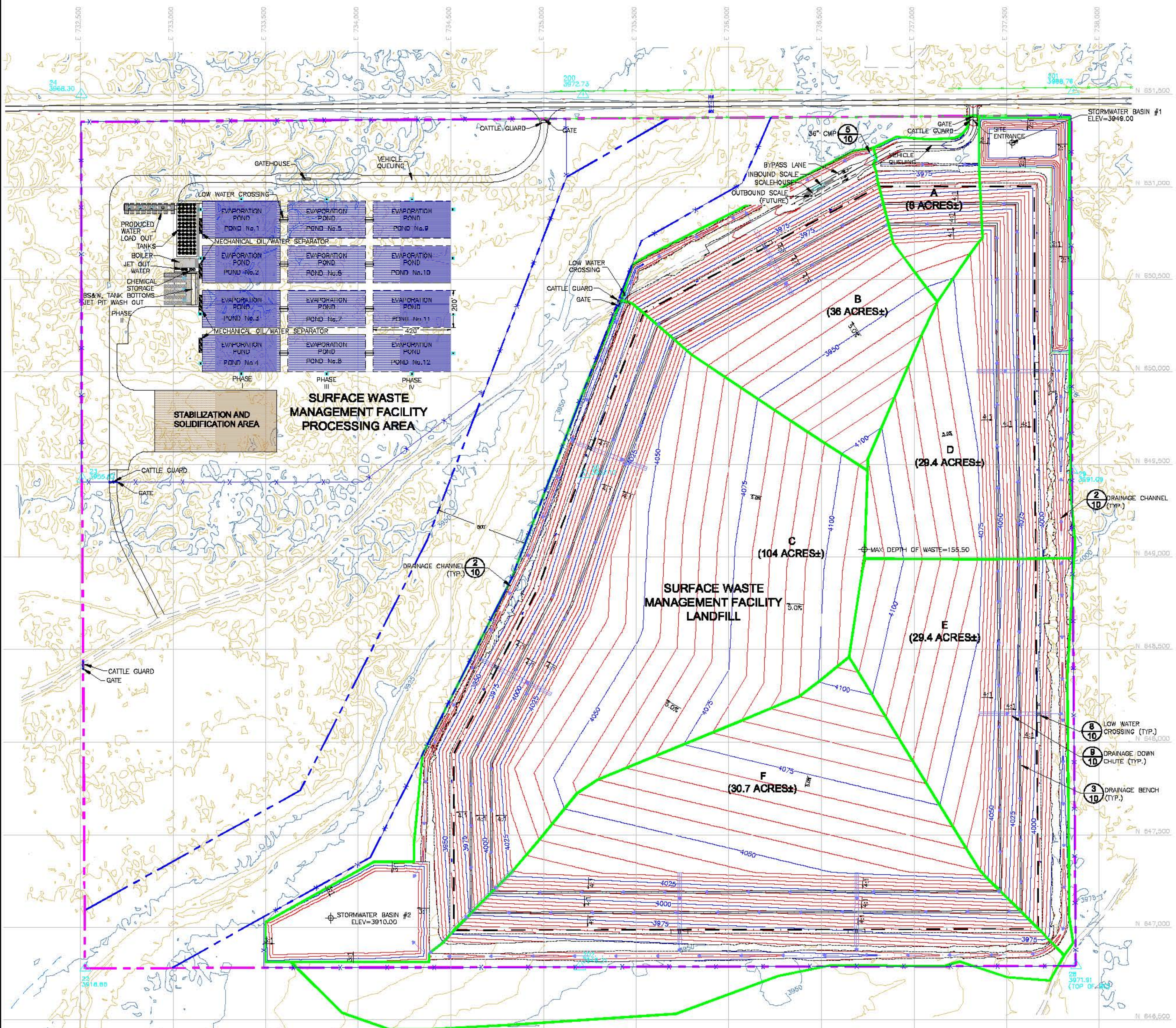
A is the soil loss per unit area, typically in tons per acre per year.
R is the rainfall/runoff factor which varies with location and climate.
K is the soil erodibility factor, which depends on the soil type
LS is the topographic factor which accounts for the site slope gradient and slope length.
C is the cover factor that accounts for ground cover (bare slope=1).
NOTE: The Slope Erosion Protection Module calculates these factors based on the assumptions input.

The RUSLE was used to determine the loss of soil from each drainage area **(Figure III.8.1)** of the final cover. The values of final cover erosion and their sum are provided on **Table III.8.1**:

TABLE III.8.1
Rainfall Erosion Losses
North American Green Output
DNCS Environmental Solutions

Area ID	Area (ac)	Slope Length (ft)	Average Slope (ft/ft)	Slope Gradient (H:1)	Average Soil Loss with Vegetation (in)	Tons/year with Vegetation
A	8.0	761	0.16	6.25	0.029	46.3
B	36.0	1462	0.11	9.1	0.025	165.3
C	104.0	1579	0.10	10	0.023	519.1
D	43.0	1072	0.13	7.7	0.027	231.8
E	39.0	1076	0.13	7.7	0.027	210.2
F	89.0	1645	0.10	10	0.023	408.7
Sum	**319.0**				**0.154**	**1,581.7**

Conclusion: When a 50% vegetative cover is considered, the soil loss is 4.96 tons per acre per year.



LEGEND

- SITE BOUNDARY (562 ACRES±)
- WATER FEATURE SETBACK (67 ACRES±)
- LIMIT OF WASTE
- LANDFILL PHASE BOUNDARY
- LANDFILL UNIT BOUNDARY
- EXISTING FENCE
- PROPOSED FENCE
- 3975 — 25' EXISTING CONTOUR
- 5' EXISTING CONTOUR
- 3975 — 25' DESIGN CONTOUR
- 5' DESIGN CONTOUR
- TOP/TOE OF SLOPE
- PAVED ROAD AND SHOULDER (NM 529)
- EXISTING UNPAVED ROAD/TRAIL
- PROPOSED FACILITY ACCESS ROAD
- DIRECTION OF STORMWATER FLOW
- LEACHATE EXTRACTION RISER PIPES
- LEACHATE CLEANOUT RISER PIPES
- A (8 ACRES±) — DRAINAGE AREA
- 3991.09 — SURVEY CONTROL POINT
- EXISTING CULVERT
- NEW CULVERT
- HYDROGEN SULFIDE MONITORING STATION
- ROAD SIGN
- 8/10 — DETAIL NUMBER / SHEET NUMBER
- N 650,500 / E 738,000 — SITE GRID

NOTES:

1. BASE MAP PROVIDED BY DALLAS AERIAL SURVEYS, INC
2. FIELD SURVEY PROVIDED BY PETTIGREW & ASSOCIATES PA (12/13/2012)
3. DATE OF AERIAL PHOTOGRAPHY: 02-28-2013
4. SITE GRID BASED ON NEW MEXICO STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAVD 88.
5. THE DNCS SURFACE WASTE MANAGEMENT FACILITY COMPRISES A TOTAL OF 495 ACRES ± (i.e., the processing area (177 acres ±) and the landfill (318 acres ±).

STORMWATER DISCHARGE

DRAINAGE ID	DRAINAGE AREA (ACRES)	FLOW RATE (CFS)	VOLUME (ACRE-FT)
A	8	42	1.5
B	36	103	6.6
C	104	183	19.1
D	43	142	7.9
E	39	103	7.2
F	89	196	16.3

RETENTION BASIN CAPACITIES

BASIN ID	CONTRIBUTING DRAINAGE AREAS	DISCHARGE VOLUME (ACRE-FT)	BASIN CAPACITY W/ 1 FT. FREEBOARD (ACRE-FT)	BASIN MAX. CAPACITY W/O 1 FT. FREEBOARD (ACRE-FT)	FACTOR OF SAFETY
1	D+NE RUN-ON	55.2	61.0	65.3	1.2
2	A+B+C+E+F+SE RUN-ON	58.1	61.5	68.6	1.2

NORTH

0 200' 400'

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

LANDFILL COMPLETION DRAINAGE PLAN

DNCS ENVIRONMENTAL SOLUTIONS
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 06/13/2014	CAD: 07 COMPLETION PLAN.dwg	PROJECT #: 542.01.01
DRAWN BY: JMC	REVIEWED BY: MRH	FIGURE III.8.1
APPROVED BY: IKG	gei@gordonenvironmental.com	

NOT FOR CONSTRUCTION
Drawing: P:\acad 2005\542.01.01\RAI 1\DRAINAGE PLAN.dwg
Date/Time: Jun. 13, 2014-11:18:18; LAYOUT: D (LS)