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LEACHATE COLLECTION SYSTEM

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APPENDIX I.1

LEACHATE DESIGN COMPATIBILITY

POLY-FLEX, INC.

An Engineering Approach to Groundwater Protection

REFERENCE MANUAL

POLY·FLEX
POLYETHYLENE GEOMEMBRANES

CHEMICAL COMPATIBILITY OF POLY-FLEX LINERS

Chemical compatibility or resistance, as applied to geomembranes, is a relative term. Actual compatibility would mean that one material dissolves in the other such as alcohol in water or grease in gasoline. An example of incompatibility would be oil and water. In liners it is undesirable to have the chemicals dissolve in the liner, hence the term compatibility is the reverse of what is normally meant in the chemical industry. In the strictest sense and from a laboratory perspective, chemical compatibility, as the term applies to this industry, would imply that the chemical has no effect on the liner. On the other hand, from an engineering perspective, chemical compatibility means that a liner survives the exposure to a given chemical even though the chemical could have some effect on the performance of the liner, but not enough to cause failure. Therefore, one must understand and define chemical compatibility for a specific project.

Generally polyethylene is effected by chemicals in one of three ways.

1. No effect—This means that the chemical in question and the polyethylene do not interact. The polyethylene does not gain (lose) weight or swell, and the physical properties are not significantly altered.
2. Oxidizes (cross linking)—Chemicals classed as oxidizing agents cause the polyethylene molecules to cross link and cause irreversible changes to the physical properties of the liner. Basically they make the liner brittle.
3. Plasticizes—Chemicals in this classification are soluble in the polyethylene structure. They do not change the structure of the polyethylene itself but act as a plasticizer. In doing so, the liner experiences weight gain of 3-15%, may swell by up to 10%, and has measurable changes in physical properties (e.g. the tensile strength at yield may decrease by up to 20%). Even under these conditions the liner maintains its integrity and is not breached by liquids, provided the liner has not been subjected to any stress. These effects are reversible once the chemicals are removed and the liner has time to dry out.

Aside from the effect that chemicals have on a liner is the issue of vapor permeation through the liner. Vapor permeation is molecular diffusion of chemicals through the liner. Vapor transmission for a given chemical is dependent primarily on liner type, contact time, chemical solubility, temperature, thickness, and concentration gradient, but not on hydraulic head or pressure. Transmission through the liner can occur in as little as 1-2 days. Normally, a small amount of chemical is transmitted. Generally HDPE has the lowest permeation rate of the liners that are commercially available.

As stated above chemical compatibility is a relative term. For example, the use of HDPE as a primary containment of chlorinated hydrocarbons at a concentration of 100% may not be recommended, but it may be acceptable at 0.1% concentration for a limited time period or may be acceptable for secondary containment. Factors that go into assessment of chemical compatibility are type of chemical(s), concentration, temperature and the type of application. No hard and fast rules are available to make decisions on chemical compatibility. Even the EPA 9090 test is just a method to generate data so that an opinion on chemical compatibility can be more reliably reached.

A simplified table on chemical resistance is provided to act as a screening process for chemical containment applications.

CHEMICAL RESISTANCE INFORMATION

POLY·FLEX
POLYETHYLENE GEOMEMBRANES

CHEMICAL CLASS	CHEMICAL EFFECT	PRIMARY CONTAINMENT (LONG TERM CONTACT) HDPE	PRIMARY CONTAINMENT (LONG TERM CONTACT) LLDPE	SECONDARY CONTAINMENT (SHORT TERM CONTACT) HDPE	SECONDARY CONTAINMENT (SHORT TERM CONTACT) LLDPE
CARBOXYLIC ACID	1				
- Unsubstituted (e.g. Acetic acid)		B	C	A	C
- Substituted (e.g. Lactic acid)		A	B	A	A
- Aromatic (e.g. Benzoic Acid)		A	B	A	A
ALDEHYDES	3				
- Aliphatic (e.g. Acetaldehyde)		B	C	B	C
- Hetrocyclic (e.g. Furfural)		C	C	B	C
AMINE	3				
- Primary (e.g. Ethylamine)		B	C	B	C
- Secondary (e.g. Diethylamine)		C	C	B	C
- Aromatic (e.g. Aniline)		B	C	B	C
CYANIDES (e.g. Sodium Cyanide)	1	A	A	A	A
ESTER (e.g. Ethyl acetate)	3	B	C	B	C
ETHER (e.g. Ethyl ether)		C	C	B	C
HYDROCARBONS	3				
- Aliphatic (e.g. Hexane)		C	C	B	C
- Aromatic (e.g. Benzene)		C	C	B	C
- Mixed (e.g. Crude oil)		C	C	B	C
HALOGENATED HYDROCARBONS	3				
- Aliphatic (e.g. Dichloroethane) +A4		C	C	B	C
- Aromatic (e.g. Chlorobenzene)		C	C	B	C
ALCOHOLS	1				
- Aliphatic (e.g. Ethyl alcohol)		A	A	A	A
- Aromatic (e.g. Phenol)		A	C	A	B
INORGANIC ACID					
- Non-oxidizers (e.g. Hydrochloric acid)	1	A	A	A	A
- Oxidizers (e.g. Nitric Acid)	2	C	C	B	C
INORGANIC BASES (e.g. Sodium hydroxide)	1	A	A	A	A
SALTS (e.g. Calcium chloride)	1	A	A	A	A
METALS (e.g. Cadmium)	1	A	A	A	A
KETONES (e.g. Methyl ethyl ketone)	3	C	C	B	C
OXIDIZERS (e.g. Hydrogen peroxide)	2	C	C	C	C

Chemical Effect (see discussion on Chemical Resistance)

1. No Effect—Most chemicals of this class have no or minor effect.
2. Oxidizer—Chemicals of this class will cause irreversible degradation.
3. Plasticizer—Chemicals of this class will cause a reversible change in physical properties.

Chart Rating

A. Most chemicals of this class have little or no effect on the liner.
Recommended regardless of concentration or temperature (below 150° F).

B. Chemicals of this class will affect the liner to various degrees.
Recommendations are based on the specific chemical, concentration and temperature.
Consult with Poly-Flex, Inc.

C. Chemicals of this class at high concentrations will have significant effect on the physical properties of the liner.
Generally not recommended but may be acceptable at low concentrations and with special design considerations.
Consult with Poly-Flex, Inc.

The data in this table is provided for informational purposes only and is not intended as a warranty or guarantee. Poly-Flex, Inc. assumes no responsibility in connection with the use of this data. Consult with Poly-Flex, Inc. for specific chemical resistance information and liner selection.

Chemicals Resistance Table
Low Density and High Density Polyethylene

INTRODUCTION
The table in this document summarises the data given in a number of chemical resistance tables at present in use in various countries, derived from both practical experience and test results.

Source: ISO/TR 7472, 7474; Carlowitz: "Kunststofftabellen-3. Auflage".

The table contains an evaluation of the chemical resistance of a number of fluids judged to be either aggressive or not towards low and high density polyethylene. This evaluation is based on values obtained by immersion of low and high density polyethylene test specimens in the fluid concerned at 20 and 60°C and atmospheric pressure, followed in certain cases by the determination of tensile characteristics.

A subsequent classification will be established with respect to a restricted number of fluids deemed to be technically or commercially more important, using equipment which permits testing under pressure and the determination of the ìcoefficient of chemical resistanceî for each fluid. These tests will thus furnish more complete indications on the use of low and high density polyethylene products for the transport of stated fluids, including their use under pressure.

SCOPE AND FIELD APPLICATION
This document establishes a provisional classification of the chemical resistance of low and high density polyethylene with respect to about 300 fluids. It is intended to provide general guidelines on the possible utilisation of low and high density polyethylene:

– at temperatures up to 20 och 60°C
– in the absence of internal pressure and external mechanical stress (for example flexural stresses, stresses due to thrust, rolling loads etc).

DEFINITIONS, SYMBOLS AND ABBREVIATIONS
The criteria of classification, definitions, symbols and abbreviations adopted in this document are as follows:

S = Satisfactory

The chemical resistance of low or high density polyethylene exposed to the action of a fluid is classified as "satisfactory" when the results of test are acknowledged to be ìsatisfactoryî by the majority of the countries participating in the evaluation.

L = Limited

The chemical resistance of low or high density polyethylene exposed to the action of a fluid is classified as "limited" when the results of tests are acknowledged to be "limited" by the majority of the countries participating in the evaluation.

Also classified as "limited" are the resistance to the action of chemical fluids for which judgements "S" and "NS" or "L" are pronounced to an equal extent.

NS = Not satisfactory

The chemical resistance of low or high density polyethylene exposed to the action of a fluid is classified as "not satisfactory" when the results of tests are acknowledged to be "not satisfactory" by the majority of the countries participating in the evaluation.

Also classified as "not satisfactory" are materials for which judgements "L" and "NS" are pronounced to an equal extent.

Sat.sol	Saturated aqueous solution, prepared at 20°C
Sol	Aqueous solution at a concentration higher than 10 %, but not saturated
Dil.sol	Dilute aqueous solution at a concentration equel to or lower than 10 %
Work.sol	Aqueous solution having the usual concentration for industrial use

Solution concentrations reported in the text are expressed as a percentage by mass.
The aqueous solutions of sparingly soluble chemicals are considered, as far as chemical action towards low or high density polyethylene is concerned, as saturated solutions.

In general, common chemical names are used in this document.

The table is made as a first guideline for user of polyethylene. If a chemical compound is not to be found or if there is an uncertainty on the chemical resistance in an application, please contact Borealis for advise and proposal on testing.

Chemical resistance of low density and high density polyethylene, not subjected to mechanical stress, to various fluids at 20 and 60°C

Chemical or product	Concentration	LD °C 20	60	HD °C 20	60
Acetaldehyde	100 %	L	NS	S	L
Acetanilide	–			S	S
Acetic acid	10 %	S	S	S	S
Acetic acid	60 %	S	L	S	S
Acetic acid, glacial	Greater than 96 %	L	NS	S	L
Acetic anhydride	100 %	L	NS	S	L
Acetone	100 %	L	NS	L	L
Acrylnitrile	–	S	S	S	S
Acetylsilicacid	–	S	S	S	S
Adipic acid	Sat.sol	S	S	S	S
After shave	–	NS	NS	NS	NS
Aliphatic hydocarbons	–	L	NS	L	L
Allyl acetate	–	S	L	S	L
Allyl alcohol	100 %	L	NS	–	–
Allyl alcohol	96 %	–	–	S	S
Allyl chloride	–	L	NS	L	NS
Aluminium chloride	Sat.sol	S	S	S	S
Aluminium fluoride	Sat.sol	S	S	S	S
Aluminium hydroxide	Sat.sol	S	S	S	S
Aluminium nitrate	Sat.sol	S	S	S	S
Aluminium oxychloride	Sat.sol	S	S	S	S
Al/potassium sulphate	Sat.sol	S	S	S	S
Aluminium sulphate	Sat.sol	S	S	S	S
Alums	Sol	S	S	S	S
Aminobenzoic acid	–	S	S	S	S
Ammonia, dry gas	100 %	S	S	S	S
Ammonia, liquid	100 %	L	L	S	S
Ammonia, aqueous	Dil.sol	S	S	S	S
Ammonium acetate	–	S	S	S	S
Ammonium carbonate	Sat.sol	S	S	S	S
Ammonium chloride	Sat.sol	S	S	S	S
Ammonium fluoride	Sol	S	–	S	S
Ammonium hexafluorosilicate	Sat.sol	S	S	S	S
Ammonium hydrogen carbonate	Sat.sol	S	S	S	S
Ammonium hydroxide	10 %	S	S	S	S
Ammonium hydroxide	30 %	S	S	S	S

Chemical or product	Concentration	LD °C 20	LD °C 60	HD °C 20	HD °C 60
Ammonium metaphosphate	Sat.sol	S	S	S	S
Ammonium nitrate	Sat.sol	S	S	S	S
Ammonium oxalate	Sat.sol	S	S	S	S
Ammonium phosphate	Sat.sol	S	S	S	S
Ammonium persulphate	Sat.sol	S	S	S	S
Ammonium sulphate	Sat.sol	S	S	S	S
Ammonium sulphide	Sol	S	S	S	S
Ammonium thiocyanate	Sat.sol	S	S	S	S
Amyl acetate	100 %	NS	NS	L	L
Amyl alcohol	100 %	L	L	S	L
Amyl chloride	100 %	NS	NS	–	–
Amyl phthalate	–	L	L	S	L
Aniline	100 %	NS	NS	S	L
Anilinchlorohydrate	–	L	–	–	–
Antimony (III) chloride	90 %	–	–	S	S
Antimony (III) chloride	Sat.sol	S	S	S	S
Antimony trichloride	Sol	S	S	S	S
Apple juice	Sol	–	–	S	L
Aqua regia	$HCl/HNO_3 = 3/1$	NS	NS	NS	NS
Aromaitic hydrocarbons	–	NS	NS	NS	NS
Arsenic acid	Sat.sol	S	S	S	S
Asorbic acid	10 %	S	S	S	S
Barium bromide	Sat.sol	S	S	S	S
Barium carbonate	Sat.sol	S	S	S	S
Barium chloride	Sat.sol	S	S	S	S
Barium hydroxide	Sat.sol	S	S	S	S
Barium sulphate	Sat.sol	S	S	S	S
Barium sulphide	Sat.sol	S	S	S	S
Beer	–	S	S	S	S
Benzaldehyde	100 %	L	NS	S	L
Benzene	100 %	NS	NS	L	L
Benzoic acid	Sat.sol	S	S	S	S
Benzoylchloride	–	S	L	S	L
Benzyl alcohol	–	S	L	S	S
Benzylsulphonic acid	10 %	S	S	S	S
Bismuth carbonate	Sat.sol	S	S	S	S
Bitumen	–	S	L	S	S
Bleach lye	10 %	S	S	S	S

Chemical or product	Concentration	LD °C 20	LD °C 60	HD °C 20	HD °C 60
Borax	Sat.sol	S	S	S	S
Boric acid	Sat.sol	S	S	S	S
Boron trifluoride	–	L	NS	L	NS
Brake fluid	–	L	NS	L	NS
Brine	–	S	S	S	S
Bromine, dry gas	100 %	NS	NS	NS	NS
Bromine, liquid	100 %	NS	NS	NS	NS
Bromoform	100 %	NS	NS	NS	NS
Butandiol	10 %	S	S	S	S
Butandiol	60 %	S	S	S	S
Butandiol	100 %	S	S	S	S
Butane, gas	100 %	–	–	S	S
Butanol	100 %	S	L	S	S
Butter	–	S	S	S	S
Butyl acetate	100 %	S	L	S	L
Butyl alcohol	100 %	S	S	S	S
Butyl chloride	–	S	–	S	–
Butylene glycol	10 %	S	S	S	S
Butylene glycol	60 %	S	S	S	S
Butylene glycol	100 %	S	S	S	S
Butyraldehyde	–	–	–	S	L
Butyric acid	100 %	L	L	S	L
Calcium arsenate	–	S	S	S	S
Calcium benzoate	–	S	S	S	S
Calcium bisulphide	–	S	S	S	S
Calcium bromate	10 %	S	S	S	S
Calcium bromide	Sat.sol	S	S	S	S
Calcium carbonate	Sat.sol	S	S	S	S
Calcium chlorate	Sat.sol	S	S	S	S
Calcium chloride	Sat.sol	S	S	S	S
Calcium chromate	40 %	S	S	S	S
Calcium cyanide	–	S	S	S	S
Calcium hydrosulphide	Sol	S	S	S	S
Calcium hydroxide	Sat.sol	S	S	S	S
Calcium hypochlorite	Sol	S	S	S	S
Calcium nitrate	Sat.sol	S	S	S	S
Calcium oxide	Sat.sol	S	S	S	S
Calcium perchlorate	1 %	S	–	S	S

Chemical or product	Concentration	LD °C 20	LD °C 60	HD °C 20	HD °C 60
Calcium permanganate	20 %	S	S	S	S
Calcium persulphate	Sol	S	S	S	S
Calcium sulphate	Sat.sol	S	S	S	S
Calcium sulphide	Dil.sol	–	–	L	L
Camphor oil	–	NS	NS	L	L
Carbon dioxide, dry gas	100 %	–	–	S	S
Carbon dioxide, wet	–	S	S	S	S
Carbon disulphide	100 %	NS	NS	L	NS
Carbon monoxide	100 %	S	S	S	S
Carbon tetrachloride	100 %	NS	NS	L	NS
Carbonic acid		S	S	S	S
Castor oil	Sol	S	S	S	S
Chlorine, water	2 % Sat.sol	L	L	S	S
Chlorine, aqueous	Sat.sol	NS	NS	L	NS
Chlorine, dry gas	100 %	NS	NS	L	NS
Chloroacetic acid	Sol	–	–	S	S
Chlorobenzene	100 %	NS	NS	NS	NS
Chloroethanol	100 %	S	S	S	S
Chloroform	100 %	NS	NS	NS	NS
Chloromethane, gas	100 %	L	–	L	–
Chlorosulphonic acid	100 %	NS	NS	NS	NS
Chloropropene	–	NS	–	L	–
Chrome alum	Sol	S	S	S	S
Chromic acid	Sat.sol	S	S	–	–
Chromic acid	20 %	–	–	S	L
Chromic acid	50 %	–	–	S	L
Chromium VI oxide	Sat.sol	S	S	S	S
Cider	–	S	S	S	S
Citric acid	Sat.sol	S	S	S	S
Citric acid	10 %	S	S	S	S
Citric acid	25 %	S	S	S	S
Coconut oil alcoholic	–	S	S	S	S
Coffee	–	S	S	S	S
Copper (II) chloride	Sat.sol	S	S	S	S
Copper cyanide	Sat.sol	S	S	S	S
Copper (II) fluoride	Sat.sol	S	S	S	S
Copper (II) fluoride	2 %	S	S	S	S
Copper (II) nitrate	Sat.sol	S	S	S	S
Copper (II) sulphate	Sat.sol	S	S	S	S

Chemical or product	Concentration	LD °C 20	60	HD °C 20	60
Corn oil	–	S	S	S	S
Cottonseed oil	–	S	S	S	S
Cresylic acid	Sat.sol	–	–	L	–
Crotonaldehyde	Sat.sol	L	–	–	–
Cyclanone	–	S	S	S	S
Cyclohexane	–	NS	NS	NS	NS
Cyclohexanol	Sat.sol	L	NS	–	–
Cyclohexanol	100 %	–	–	S	S
Cyclohexanone	100 %	NS	NS	S	L
Decahydronaphthalene	100 %	L	NS	S	L
Decane	–	NS	NS	L	NS
Decalin	100 %	–	–	S	L
Detergents, synthetic	–	S	S	S	S
Developers (photographic)	Work.conc	–	–	S	S
Dextrin	Sol	S	S	S	S
Dextrose	Sol	S	S	S	S
Diacetone alcohol	–	L	L	L	L
Diazo salts	–	S	S	S	S
Dibutyl amine	–	NS	NS	L	NS
Dibuthyl ether	–	NS	NS	L	–
Dibutylphthalate	–	L	L	S	L
Dichlorobenzene	–	NS	NS	NS	NS
Dichloroethylene	–	NS	NS	NS	NS
Dichloropropylene	–	NS	NS	NS	NS
Diesel oil	–	S	NS	S	L
Diethyl ether	100 %	NS	NS	L	–
Diethyl ketone	–	L	NS	L	L
Diethylene glycol	–	S	S	S	S
Diglycolic acid	–	S	S	S	S
Diisobutylketone	100 %	S	L	S	L
Dimethyl amine	100 %	NS	NS	–	–
Dimethyl formamid	–	S	L	S	S
Dioctyl phthalate	100 %	L	NS	S	L
Dioxan	100 %	–	–	S	S
Dipentene	–	NS	NS	NS	NS
Disodium phosphate	–	S	S	S	S
Drano, plumbing cleaner	–	S	S	S	S

Chemical or product	Concentration	LD °C 20	LD °C 60	HD °C 20	HD °C 60
Emulsions, photographic	–	S	S	S	S
Ethandiol	100 %	S	S	S	S
Ethanol	40 %	S	L	S	L
Ethanol	96 %	L	L	–	–
Ethyl acetate	100 %	L	NS	S	NS
Ethyl acrylate	100 %	NS	NS	L	NS
Ethyl alcohol	35 %	S	S	S	S
Ethyl alcohol	100 %	S	S	S	S
Ethyl benzene	–	NS	NS	NS	NS
Ethyl chloride	100 %	NS	NS	NS	NS
Ethylene chloride	100 %	NS	NS	NS	NS
Ethylene diamine	100 %	S	L	S	S
Ethyl ether	–	NS	NS	NS	NS
Ethylene glycol	100 %	S	S	S	S
Ethyl mercaptan	–	NS	NS	NS	NS
Ferric chloride	Sat.sol	S	S	S	S
Ferric nitrate	Sat.sol	S	S	S	S
Ferric sulphate	Sat.sol	S	S	S	S
Ferrous chloride	Sat.sol	S	S	S	S
Ferrous sulphate	Sat.sol	S	S	S	S
Fish solubles	Sol	S	S	S	S
Fluoboric acid	–	S	S	S	S
Fluorine gas	100 %	L	NS	NS	NS
Fluorine gas, dry	100 %	NS	NS	NS	NS
Fluorine gas, wet	100 %	NS	NS	NS	NS
Fluorosilic acid	Conc	S	L	S	L
Fluorosilic acid	40 %	S	S	S	S
Formaldehyde	40 %	S	S	S	S
Formic acid	40 %	S	S	S	S
Formic acid	98 to 100 %	S	S	S	S
Fructose	Sat.sol	S	S	S	S
Fruit pulps	Sol	S	S	S	S
Furfural	100 %	NS	NS	NS	NS
Furfuryl alcohol	100 %	L	NS	S	L
Gallic acid	Sat.sol	S	S	S	S
Gasoline, petrol	–	L	NS	L	L
Gelatine	–	S	S	S	S

Chemical or product	Concentration	LD °C 20	LD °C 60	HD °C 20	HD °C 60
Glucose	Sat.sol	S	S	S	S
Glycerine	100 %	S	S	S	S
Glycerol	100 %	S	S	S	S
Glycolic acid	30 %	S	L	–	–
Glycolic acid	Sol	–	–	S	S
n-Heptane	100 %	NS	NS	L	NS
Hexachlorobenzene	–	S	S	S	L
Hexachlorophene	–	NS	NS	L	L
Hexamethylenetriamine	40 %	S	–	S	–
Hexane	–	S	L	S	L
Hexanol, tertiary	–	S	S	S	S
Hydrobromic acid	50 %	S	S	S	S
Hydrobromic acid	Up to 100 %	S	S	S	S
Hydrochloric acid	Up to 36 %	S	S	S	S
Hydrochloric acid	Conc	S	S	S	S
Hydrochlorous acid	Conc	S	S	S	S
Hydrocyanic acid	10 %	S	S	S	S
Hydrocyanic acid	Sat.sol	S	S	S	S
Hydrofluoric acid	40 %	S	S	S	S
Hydrofluoric acid	60 %	S	L	S	L
Hydrogen	100 %	S	S	S	S
Hydrogen chloride	Dry gas	S	S	S	S
Hydrogen peroxide	30 %	S	L	S	S
Hydrogen peroxide	90 %	S	NS	S	NS
Hydrogen sulphide gas	100 %	S	S	S	S
Hydroquinone	Sat.sol	S	S	–	–
Hydroxylamine	up to 12 %	S	S	S	S
Inks	–	S	S	S	S
Iodine (in potassium sol)	–	L	NS	NS	NS
Iodine (in alcohol)	–	NS	NS	NS	NS
Iron (II) chloride	Sat.sol	S	S	S	S
Iron (II) sulphate	Sat.sol	S	S	S	S
Iron (III) chloride	Sat.sol	S	S	S	S
Iron (III) nitrate	Sol	S	S	S	S
Iron (III) sulphate	Sat.sol	S	S	S	S
Iso octane	100 %	S	NS	S	L
Iso pentane	–	NS	NS	NS	NS

Chemical or product	Concentration	LD °C 20	60	HD °C 20	60
Isopropanol	–	S	S	S	S
Isopropyl amine	–	NS	NS	NS	NS
Isopropyl ether	100 %	L	NS	S	NS
Kerosene	–	NS	NS	NS	NS
Lactic acid	10 %	S	S	S	S
Lactic acid	28 %	S	S	S	S
Lactic acid	up to 100 %	S	S	S	S
Latex	–	S	S	S	S
Lead acetate	Dil.sol	S	S	S	S
Lead acetate	Sat.sol	S	S	S	S
Lead arsenate	–	S	S	S	S
Lubricating oil	–	S	S	S	S
Lysol	–	NS	NS	L	NS
Magnesium carbonate	Sat.sol	S	S	S	S
Magnesium chloride	Sat.sol	S	S	S	S
Magnesium hydroxide	Sat.sol	S	S	S	S
Magnesium nitrate	Sat.sol	S	S	S	S
Magnesium sulphate	Sat.sol	S	S	S	S
Maleic acid	Sat.sol	S	S	S	S
Mercury	–	S	S	S	S
Mercury (I) nitrate	Sol	S	S	S	S
Mercury (II) chloride	Sat.sol	S	S	S	S
Mecury (II) cyanide	Sat.sol	S	S	S	S
Mercury	100 %	S	S	S	S
Methanol	100 %	S	L	S	S
Methyl alcohol	100 %	S	L	S	S
Methyl benzoic acid	Sat.sol	NS	NS	L	–
Methyl bromide	100 %	NS	NS	NS	NS
Methyl chloride	100 %	NS	NS	NS	NS
Methylcyclohexane	–	L	NS	L	NS
Methyl ethyl ketone	100 %	–	–	S	L
Methylene chloride	–	NS	NS	NS	NS
Methoxybutanol	100 %	S	L	S	L
Milk	–	S	S	S	S
Milk of Magnesia	–	S	L	S	L
Mineral oils	–	L	NS	S	L

Chemical or product	Concentration	LD °C 20	60	HD °C 20	60
Molasses	Work.conc	S	S	S	S
Motor oil	–	S	L	S	S
Naphtha	–	L	NS	L	NS
Naphtahalene	–	NS	NS	L	–
Nickel chloride	Sat.sol	S	S	S	S
Nickel nitrate	Sat.sol	S	S	S	S
Nickel sulphate	Sat.sol	S	S	S	–
Nicotine	Dil.sol	S	S	S	S
Nicotinic acid	Dil.sol	L	L	S	–
Nitric acid	25 %	S	S	S	S
Nitric acid	50 %	S	L	S	L
Nitric acid	70 %	S	L	S	L
Nitric acid	95 %	NS	NS	NS	NS
Nitric acid	100 %	NS	NS	NS	NS
Nitrobenzene	100 %	NS	NS	NS	NS
Nitroethane	100 %	S	NS	S	NS
Nitromethane	100 %	S	–	S	–
Nitrotoluene	–	NS	NS	NS	NS
n-Octane	–	S	S	S	S
Octyl alcohol	–	S	NS	S	NS
Oil and fats		L	NS	S	L
Oleic acid	100 %	L	NS	S	S
Oleum (H_2SO_4 + 10 % SO_3)		NS	NS	NS	NS
Oleum (H_2SO_4 + 50 % SO_3)		NS	NS	NS	NS
Olive oil	–	S	NS	S	NS
Orthophosphoric acid	50 %	S	S	S	S
Orthophosphoric acid	95 %	S	L	S	L
Oxalic acid	Sat.sol	S	S	S	S
Oxygen	100 %	S	–	S	L
Ozone	100 %	NS	NS	L	NS
Paraffin oil	–	S	L	S	S
n-Pentane	–	NS	NS	NS	NS
Pentane-2	–	NS	NS	NS	NS
Perchloric acid	20 %	S	S	S	S
Perchloric acid	50 %	S	L	S	L
Perchloric acid	70 %	S	NS	S	NS

Chemical or product	Concentration	LD	°C	HD	°C
		20	60	20	60
Perchloroethylene	–	NS	NS	NS	NS
Phenol	Sol	L	NS	S	S
Phosphine	100 %	S	S	S	S
Phosphoric acid	up to 25 %	S	S	S	S
Phosphoric acid	25 to 50 %	S	S	S	S
Phosphoric (III) chloride	100 %	S	L	S	L
Phosphorous (II) chloride	100 %	–	–	S	L
Phosphorous pentoxide	100 %	S	S	S	S
Phosphorous trichloride	100 %	S	L	S	L
Photographic solutions	–	S	S	S	S
Phtalic acid	50 %	S	S	S	S
Picric acid	Sat.sol	S	L	S	–
Plating solutions	–	S	S	S	S
Potassium acetate	–	S	S	S	S
Potassium aluminium sulphate	Sat.sol	S	S	S	S
Potassium benzoate	–	S	S	S	S
Potassium bicarbonate	Sat.sol	S	S	S	S
Potassium borate	Sat.sol	S	S	S	S
Potassium bromate	Sat.sol	S	S	S	S
Potassium bromide	Sat.sol	S	S	S	S
Potassium carbonate	Sat.sol	S	S	S	S
Potassium chlorate	Sat.sol	S	S	S	S
Potassium chloride	Sat.sol	S	S	S	S
Potassium chromate	Sat.sol	S	S	S	S
Potassium cyanide	Sol	S	S	S	S
Potassium dichromate	Sat.sol	S	S	S	S
Potassium fluoride	Sat.sol	S	S	S	S
Potassium hexacyanoferrate (III)	Sat.sol	S	S	S	S
Potassium hexacyanoferrate (II)	Sat.sol	S	S	S	S
Potassium hexafluorosilicate	Sat.sol	S	S	S	S
Potassium hydrogen carbonate	Sat.sol	S	S	S	S
Potassium hydrogen sulphate	Sat.sol	S	S	S	S
Potassium hydrogen sulphide	Sol	–	–	S	S
Potassium hydroxide	10 %	S	S	S	S
Potassium hydroxide	Sol	S	S	S	S
Potassium hypochlorite	Sol	S	L	S	L
Potassium iodate	10 %	S	S	S	S
Potassium iodide	Sat.sol	S	S	S	S
Potassium nitrate	Sat.sol	S	S	S	S

Chemical or product	Concentration	LD 20	°C 60	HD 20	°C 60
Potassium orthophosphate	Sat.sol	S	S	S	S
Potassium oxalate	Sat.sol	S	S	S	S
Potassium perchlorate	Sat.sol	S	S	S	S
Potassium permanganate	20 %	S	S	S	S
Potassium persulphate	Sat.sol	S	S	S	S
Potassium phosphate	Sat.sol	S	S	S	S
Potassium sulphate	Sat.sol	S	S	S	S
Potassium sulphide	Sol	S	S	S	S
Potassium sulphite	Sat.sol	S	S	–	–
Potassium thiocyanate	Sat.sol	S	S	S	S
Potassium thiosulphate	Sat.sol	S	S	S	S
Propargul alcohol	–	S	S	S	S
n-Propyl alcohol	–	S	S	S	S
Propionic acid	50 %	–	–	S	S
Propionic acid	100 %	–	–	S	L
Propylene dichloride	100 %	NS	NS	NS	NS
Propylene glycol	–	S	S	S	S
Pyridine	100 %	–	–	S	L
Quinol (hydroquinone)	Sat.sol	S	S	S	S
Resorcinol	Sat.sol	S	S	S	S
Salicylic acid	Sat.sol	S	S	S	S
Sea water	–	S	S	S	S
Selenic acid	–	S	S	S	S
Silicon oil	–	S	S	S	S
Silver acetate	Sat.sol	S	S	S	S
Silver cyanide	Sat.sol	S	S	S	S
Silver nitrate	Sat.sol	S	S	–	–
Soap solution	100 %	S	S	S	S
Sodium acetate	Sat.sol	S	S	–	–
Sodium antimonate	Sat.sol	S	S	S	S
Sodium arsenite	Sat.sol	S	S	S	S
Sodium benzoate	Sat.sol	S	S	S	S
Sodium bicarbonate	Sat.sol	S	S	S	S
Sodium bisulphate	Sat.sol	S	S	S	S
Sodium bisulphite	Sat.sol	S	S	S	S
Sodium borate	–	S	S	S	S
Sodium bromide	Sat.sol	S	S	S	S
Sodium carbonate	Sat.sol	S	S	S	S

Chemical or product	Concentration	LD 20	°C 60	HD 20	°C 60
Sodium chlorate	Sat.sol	S	S	S	S
Sodium chloride	Sat.sol	S	S	S	S
Sodium chlorite	Sat.sol	L	–	–	–
Sodium cyanide	Sat.sol	S	S	S	S
Sodium dichromate	Sat.sol	S	S	S	S
Sodium fluoride	Sat.sol	S	S	S	S
Sodium hexacyanoferrate (III)	Sat.sol	–	–	S	S
Sodium hexacyanoferrate (II)	Sat.sol	–	–	S	S
Sodium hexafluorosilicate	Sat.sol	S	S	S	S
Sodium hydrogen carbonate	Sat.sol	S	S	S	S
Sodium hydrogen sulphate	Sat.sol	S	S	S	S
Sodium hydrogen sulphite	Sol	S	S	S	S
Sodium hydroxide	40 %	S	S	S	S
Sodium hydroxide	Sol	–	–	S	S
Sodium hypochloride	–	L	NS	S	S
Sodium hypochlorite	15 %	–	–	S	S
	available Cl	–	–	S	S
Sodium iodate	10 %	S	S	S	S
Sodium iodide	Sat.sol	S	S	S	S
Sodium nitrate	Sat.sol	S	S	S	S
Sodium nitrite	Sat.sol	S	S	S	S
Sodium ortophosphate	Sat.sol	S	S	S	S
Sodium oxalate	Sat.sol	S	S	S	S
Sodium phosphate	Sat.sol	S	S	S	S
Sodium silicate	Sol	S	S	S	S
Sodium sulphate	Sat.sol	S	S	S	S
Sodium sulphide	Sat.sol	S	S	S	S
Sodium sulphite	Sat.sol	S	S	S	S
Sodium thiocyanate	Sat.sol	S	S	S	S
Stannic chloride	Sat.sol	S	S	S	S
Stannous chloride	Sat.sol	S	S	S	S
Starch solution	Sat.sol	S	S	S	S
Stearic acid	Sat.sol	S	L	S	–
Styrene	Sol	L	NS	L	NS
Sulphur dioxide, dry	100 %	S	S	S	S
Sulphur trioxide	100 %	NS	NS	NS	NS
Sulphur acid	10 to 50 %	S	S	S	S
Sulphuric acid	10 %	S	S	S	S
Sulphuric acid	50 %	S	S	S	S

Chemical or product	Concentration	LD °C 20	LD °C 60	HD °C 20	HD °C 60
Sulphuric acid	70 %	S	L	S	L
Sulphuric acid	80 %	S	NS	S	NS
Sulphuric acid	98 %	L	NS	S	NS
Sulphuric acid	Fuming	NS	NS	NS	NS
Sulphurous acid	30 %	S	S	S	S
Sulphurous acid	Sol	S	S	S	S
Tallow	–	S	L	S	L
Tannic acid	Sol	S	S	S	S
Tartaric acid	Sat.sol	S	S	S	S
Tartaric acid	Sol	–	–	S	S
Tetrachloroethylene	100 %	NS	NS	NS	NS
Tetrachloromethane	100 %	NS	NS	L	NS
Tetradecane		NS	NS	NS	NS
Tetrahydrofuran	–	NS	NS	NS	NS
Tetrahydronaphthalene	100 %	L	NS	S	L
Thionyl chloride	100 %	NS	NS	NS	NS
Tin (II) chloride	Sat.sol	S	S	S	S
Tin (IV) chloride	Sol	S	S	S	S
Tin (IV) chloride	Sat.sol	–	–	S	S
Titanium tetrachloride	Sat.sol	NS	NS	NS	NS
Toluene	100 %	NS	NS	L	NS
Tribromomethane	–	NS	NS	NS	NS
Trichloroacetaldehyde	–	S	–	S	–
Trichlorobenzene	–	NS	NS	–	–
Trichloroethylene	100 %	NS	NS	NS	NS
Triethanolamine	100 %	S	–	S	–
Triethanolamine	Sol	–	–	S	L
Triethylene glycol	–	S	S	S	S
Trisodium phosphate	Sat.sol	S	S	–	–
Turpentine	–	NS	NS	NS	NS
Urea	up to 30 %	S	S	S	S
Urea	Sol	S	S	S	S
Urine	-	S	S	S	S
Vanilla extract	–	S	S	S	S
Vaseline	–	S	L	S	S
Vegetables oils		S	L	S	S
Vinegar		S	S	S	S
Water		S	S	S	S
Wetting agents	–	S	S	S	S
Wines and spirits		S	S	S	S
Chemical or product	**Concentration**	**LD °C 20**	**LD °C 60**	**HD °C 20**	**HD °C 60**

Xylene	100 %	NS	NS	L	NS
Yeast	Sol	S	S	S	S
Zinc bromide	Sat.sol	S	S	S	S
Zinc carbonate	Sat.sol	–	–	S	S
Zinc chloride	Sat.sol	S	S	S	S
Zinc oxide	Sat.sol	S	S	S	S
Zinc stearate	–	S	S	S	S
Zinc sulphate	Sat.sol	S	S	S	S
o-Zylene		NS	NS	NS	NS
p-Zylene	–	NS	NS	NS	NS

Document ID: EDF-ER-278
Revision ID: 1
Effective Date: 05/14/02

Engineering Design File

Liner/Leachate Compatibility Study

Idaho National Engineering and Environmental Laboratory

Form 412.14
07/24/2001
Rev. 03

ENGINEERING DESIGN FILE

1. Title: Liner/Leachate Compatibility Study

2. Project File No.: NA

3. Site Area and Building No.: NA | 4. SSC Identification/Equipment Tag No.: NA

5. Summary:

This study evaluates the compatibility of the liner materials with the leachate generated by the waste disposed in the INEEL CERCLA Disposal Facility. The liner system is composed of both natural and synthetic materials including compacted clay, geosynthetic clay liner, high-density polyethylene, and polypropylene products. This study will determine whether these materials are compatible with the leachate, based on experience at similar landfills and published literature.

6. Review (R) and Approval (A) and Acceptance (Ac) Signatures:
(See instructions for definitions of terms and significance of signatures.)

	R/A	Typed Name/Organization	Signature	Date
Performer		Phillip Crouse/ Montgomery Watson	Philip E. Crouse	05/14/02
Checker	R	(Same as Independent Peer Reviewer)		05/14/02
Independent Peer Reviewer	A	Marty Doornbos/ BBWI	Marty Doornbos	05/14/02
Approver	A	Thomas Borschel/ BBWI	Thomas F. Borschel	05/14/02
Requestor	Ac	Don Vernon/ BBWI	D. Vernon	05/14/02

7. Distribution: (Name and Mail Stop) M. Doornbos, MS 3930; D. Vernon, MS 3930; T. Borschel, MS 3930

8. Records Management Uniform File Code (UFC):

Disposition Authority: | Retention Period:

EDF pertains to NRC licensed facility or INEEL SNF program?: ☐ Yes ☐ No

9. Registered Professional Engineer's Stamp (if required)

2. EXISTING STUDIES OF LINER/LEACHATE COMPATIBILITY

2.1 EPA Method 9090

In 1992, EPA published Method 9090, 'Compatibility Tests for Wastes and Membrane Liners,' to set the standard that liners must meet to be protective of human health and the environment. This test has been used throughout the industry to demonstrate that liners are compatible with numerous leachate compositions from municipal and hazardous waste landfills, and surface impoundments. The results of these studies have been documented and are readily available. The manufacturers of the liners now supply limitations of the products based on these tests. The results are commonly accepted as reliable and complete. Since the ICDF leachate contains no unusual or excessive constituents, the industry results for these liners is sufficient to demonstrate compatibility.

The compatibility of GCL and SBL materials are usually demonstrated by permeating the material with leachate to determine its permeability. Method 9090 consists of immersing small sample specimens of a liner material in leachate and periodically measuring changes in the physical properties. The specimens are removed after 30, 60, 90, and 120 days, then tested to determine changes to the physical dimensions and mechanical properties. Acceptance criteria for defining compatibility tend to vary. Compatibility has been defined as geomembrane properties remaining above the minimum suggested property value or an allowable small percentage of change in properties (e.g., less than 15%) to maintain the integrity of the liner.

GCL and SBL are tested for compatibility by permeating the material with a leachate solution to determine effects on the hydraulic performance of the material. Typically, solutions with high concentrations of contaminants or pure products are allowed to permeate a sample under confining pressure to determine the saturated permeability of the material using ASTM methods such as ASTM D5084. A saturated permeability exceeding 1×10^{-7} cm/sec would indicate incompatibility.

The HDPE geomembrane and GCL materials planned for the ICDF are considered to be the most chemically inert liner materials commercially available for waste disposal facilities. Numerous studies using EPA Method 9090 and permeability tests, among other testing procedures, have been performed for waste disposal facilities and in the laboratory providing a good understanding of the compatibility behavior of these liner materials.

2.2 Published Studies

2.2.1 Comparison with Other Geomembrane 9090 Compatibility Studies

Relevant compatibility studies have been performed at DOE's Hanford facility near Richland, Washington. These projects include the Liquid Effluent Retention Facility (LERF), W-025 landfill, and the Grout Facility. Other relevant studies include the Kettleman Hills landfill located in northern California. The results of these published studies indicate that a HDPE geomembrane will function well as a liner beneath the landfill waste or liquid waste in the evaporation pond. The published geomembrane compatibility studies for the Hanford facility are listed in Section 6 Bibliography of this report.

A comparison between the anticipated ICDF landfill leachate and that used in compatibility tests for other facilities is summarized in Table 2-1.

Table 2-1. EPA test method 9090 compatibility studies comparison.

Compatibility Study[a]	Type of Material Tested	General Composition of Leachate	9090[b] Test Concentrations or Radiation Exposure that Demonstrated Compatibility in Each Study	ICDF[c] Leachate Concentration/ Absorbed Radiation
Hanford LERF	60-mil smooth HDPE from four manufacturers	Organics	16.25 mg/L	70 mg/L
Hanford W-025 Landfill	60-mil smooth HDPE	Inorganics	204,210 mg/L	18,400[g] mg/L
		Organic Leachate and Radiation Exposure	50,000 rads	12,000 rads (landfill) 100,000 rads (evaporation pond)
		pH	9.2	8.0
Hanford Grout Facility	60-mil smooth HDPE	Inorganics	368,336 mg/L	18,400 mg/L
		Organic Leachate and Radiation[e] Exposure	37,000,000 rads	
		Organic Leachate and Radiation[f] Exposure	16,000,000 rads	12,000 rads (landfill) 100,000 rads (evaporation pond)
		pH	>14	8.0
Kettleman Hills Landfills	60-mil smooth HDPE	Organics	93,040 mg/L	70 mg/L
		Inorganics	250,000 mg/L	18,400 mg/L
		pH	>12	8.0
Unidentified Landfill Study	Textured HDPE	Organics	154 mg/L	70 mg/L

a. Detailed compatibility test information is provided in *Evaluation of Liner/Leachate Chemical Compatibility for the Environmental Restoration Disposal Facility report* (USACE 1995).

b. EPA Test Method 9090 "Compatibility Test for Wastes and Membrane Liners" (EPA 1992).

c. Values reported represent values at which the test was run, showing no unacceptable effects. They do not represent an allowable limit.

d. Values based on the "Leachate/Contaminate Reduction Time Study" (EDF-ER-274).

e. A slight reduction in strength and elasticity of the HDPE liner occurred at the highest doses used in the testing.

f. No measurable changes in the HDPE liner material properties were observed after the testing.

g. Reported as total inorganics.

HDPE is chemically resistant to inorganic salt solutions and can be incompatible with some organic solutions at high concentrations (i.e., pure products). Actual compatibility tests from other landfills show that HDPE is chemically resistant to much higher concentrations of organics in the leachate than what is expected in the ICDF leachate. The organic concentration in the Kettlemen Hills Landfill leachate is almost four orders of magnitude higher than what is expected in the ICDF landfill leachate. The use of general categories of chemicals rather than individual constituents has been accepted by the EPA for the Environmental Restoration Disposal Facility at Hanford and provide a worst-case scenario due to possible synergistic effects of mixed compounds.

The EPA Method 9090 tests performed on HDPE geomembrane liner planned for the Grout Facility included high temperatures and doses of large amounts of radiation. The leachate solution temperature was increased to 194°F, which is significantly above the standard test temperatures of 73° and 122°F required in Method 9090. Additionally, the samples were irradiated at doses up to 37,000,000 rads prior to the testing, significantly decreasing the strength and elasticity (i.e., greater than 25%) of the geomembrane specimens (USACE 1995). Geomembrane samples tested for the W-025 facility did not produce measurable changes in the HDPE liner properties when irradiated for 120 days with a total dose of 50,000 rads. HDPE geomembranes are manufactured with additives to improve ductility and durability such as carbon black and antioxidants. The literature also indicates that these additives allow higher doses than standard HDPE material alone (Kircher and Bowman 1964). The literature indicates that thin films (i.e., 0.002 in.) of different types of HDPE material alone can become brittle when irradiated at doses between 4,400,000 and 78,000,000 rads. Studies performed using polymer materials show that properties typically begin to change at a total radiation dose of between 1,000,000 and 10,000,000 rads (Koerner et al. 1990).

The landfill and evaporation pond HDPE geomembrane liners are expected to receive a dose from the leachate of 12,000 and 100,000 rads, respectively. This is a conservatively high dose since it assumes that concentrations of radionuclides are constant in the leachate over the 15-year operational life of the landfill. Even though conservatively high, the total dose is below the dose found in other studies (i.e., 1,000,000 rads) that may affect the properties of the geomembrane.

2.2.2 Geosynthetic Clay and Soil Bentonite Liners

Based on review of the published studies listed in Section 6 (Bibliography), SBL and GCL perform well unless exposed to high concentrations of divalent cations, very acidic or basic solutions, or solutions with a low dielectric constant (such as gasoline). The leachate expected at the ICDF will have a pH of 8, slightly above neutral. The studies further demonstrate that, when confined, as is the case in the ICDF landfill, or pre-hydrated, SBLs and GCLs will perform well when exposed to high divalent cation concentrations.

Several studies were found that evaluated the impact of SBL permeability with various organic and inorganic materials. The majority of them used very concentrated compounds, which is not the typical composition of landfill leachates and when compared with ICDF leachate exceeded concentrations by as much as an order of magnitude. One study was found that addressed the issue of when leachate constituent concentrations impact SBL permeability. For this study, four different types of organic compounds were used as permeants. They included methanol, acetic acid, heptane, and trichloroethylene (TCE). The results indicate that soil permeability was not affected by methanol until a concentration of 80% by volume was used. The acetic acid actually reduced the soil permeability due to dissolution and reprecipitation of the soil. Heptane and TCE had no effect on permeability when used up to their solubility limit in water. However, when used in pure form, they increased the soil permeability significantly (250 to 1,000 times). In addition to the concentration of the permeant used, changes in hydraulic permeability are also governed by the mineralogy of the soil (Borders 1986). Although only low concentrations of TCE are predicted in the ICDF leachate, the study demonstrates that high concentrations of organic constituents are required to affect permeability.

No studies were identified that considered the long-term effects of radiation on the physical properties of the SBL or GCL materials. Since long-term studies cannot be conducted, conservative radiation limitations have been employed. Low-permeability soils have been used at multiple DOE facilities containing radioactive waste. The only potential adverse reaction that could occur with the SBL or GCL would be high heat that could dry out these materials, however, it is anticipated that the radioactive material placed in the ICDF will not generate any thermal gradients across the liner system.

The concentration of organic material is expected to be approximately 70 mg/L. This is significantly below the concentration of a highly concentrated solution so it will not increase the permeability of the SBL and GCL. The amount of radioactivity will be low in the ICDF landfill waste and will not generate a significant amount of heat that can desiccate the compacted clay. Additionally, the operations layer will provide a 3-ft buffer between the liner system and waste.

2.3 Manufacturers' Data

2.3.1 HDPE Geomembrane

The manufacturers of the geosynthetic products proposed for the ICDF landfill have published maximum allowable concentrations of various chemical compounds that can contact the HDPE geomembrane without adversely affecting its performance. The most recent recommended maximum concentrations of chemicals were obtained from the manufacturer. A list of the manufacturers' maximum allowable concentrations for specific leachate constituents on HDPE material is provided in Appendix C. In addition, the effects of radiation exposure with respect to the geomembrane physical properties are also presented.

2.3.2 Geosynthetic Clay and SBLs

The GCL underlying the geomembrane in the ICDF landfill and evaporation pond liner consists of processed sodium bentonite clay sandwiched between two geotextile fabrics. The SBL underlying the geosynthetic liners also consists of 5% by weight of processed bentonite amendment. Sodium bentonite is an ore comprised mainly of the montmorillonite clay mineral with broad, flat, negatively charged platelets that attract water hydrating the bentonite. The swelling provides the ability to seal around penetrations, giving the GCL its self-healing properties. A GCL product with Volclay® type sodium bentonite manufactured by CETCO will be installed in the landfill and evaporation pond.

The GCL manufacturer allows the use of GCL with few restrictions on maximum chemical concentrations. The manufacturer does recommend that treated bentonite should be used when directly exposed to liquids with high concentration of salts (divalent cations) such as in seawater (CETCO 2001). The concentration of salts in typical seawater is on the order of 35,000 mg/L (USGS 1989). The ICDF total inorganic leachate concentration is on the order of 17,000 mg/L, approximately 2 times lower than that of seawater. The same compatibility limitation is found in the literature as described in Section 2.1.2. The bentonite added to the soil for the bentonite liner will have the same limitation, however, to a lesser extend since only a small percentage (i.e., 5%) is comprised of bentonite. Based on this assessment, the exposed salts in the brackish leachate will be compatible with the GCL and SBL underlying the geomembrane. Notably, this assumes that the overlying HDPE geomembranes must leak before leachate can come in contact with the GCL or SBL.

3. WASTE ACCEPTANCE CRITERIA

3.1 Landfill

Individual constituents in the ICDF landfill design inventory were evaluated to determine maximum allowable ICDF landfill waste concentrations, that if placed in the landfill would generate leachate compatible with the liner system. Many of the individual design inventory constituents have not been included in the composition of leachate used for published compatibility studies. However, the constituents used in the published studies are in similar chemical groups as the constituents in the ICDF design inventory and therefore, would react similarly with the liner materials. Moreover, the use of general chemical categories rather than individual constituents provide a worst-case scenario due to possible synergistic effects of mixed compounds.

Table 3-1 provides the recommended maximum concentration of chemical categories that, if in the landfill leachate, may be incompatible with the polymeric or earthen material comprised of the ICDF landfill and evaporation pond liner systems. These limits are based on review of the published liner compatibility studies and manufacturers' recommendations. The maximum allowable concentration for HDPE geomembrane, GCL, and SBL were compared to determine the highest acceptable value. The lowest of all three values was selected as the suggested maximum concentration. The concentrations based on the design inventory of waste constituents are also provided in Table 3-1. Where available, the recommended maximum allowable concentration with regard to liner compatibility for individual constituents is provided in Tables D-1, D-2, and D-3 in Appendix D for specific organic, inorganic, and radionuclide constituents, respectively.

Table 3-1. Maximum allowable concentrations in leachate by chemical category.

Chemical Category	Compatible Concentration for HDPE	Compatible Concentration for GCL and Clay	Suggested ICDF Maximum Concentration or Value	Design Inventory Concentration Dose or Value
Organics	500,000[a] mg/L	500,000[b] mg/L	500,000 mg/L	70 mg/L
Acids and Bases	750,000[a] mg/L	500,000[b] mg/L	500,000 mg/L	0[d] mg/L
Inorganic	500,000[a] mg/L	500,000[b] mg/L	500,000 mg/L	17,100 mg/L
Dissolved Salts	No Limit	35,000 mg/L	35,000 mg/L	8,000 mg/L[c]
Strong Oxidizers	1,000 mg/L	No limit	1,000 mg/L	0[d] mg/L
Radionuclides	1,000,000[b] rads	No limit	1,000,000 rads	12,000 rads (15 yr) 800,000 rads (1000 yr)
pH	0.5 - 13.0[a]	0.5 - 13.0	0.5 - 13.0	8.0

a. Based on the manufacturers' maximum concentration of the list of constituents tested by the manufacturers. The manufacturers' recommendations are provided in Appendix C.

b. Based on reported literature values.

c. Based on the maximum sodium concentration determined in the Geochemical Evaluation.

d. Strong acids, bases, or oxidizing compounds were not reported in the design inventory.

HDPE Liner Manufacturer's Compatibility Data

LINER COMPATIBILITY

1. **Identify the manufacturer and the type of liner that will be used in the landfill which will contain the form R wastes.**

   MANUFACTURER: GSE Lining Technology, Inc.
   LINER TYPE: 60 mil HDPE

2. **Describe how the following types of chemicals will affect the liner to be used to contain the form R waste:**

   aromatic halogenated hydrocarbons - SEE ATTACHED SHEET

   aliphatic halogenated hydrocarbons - SEE ATTACHED SHEET

   aromatic hydrocarbons - SEE ATTACHED SHEET

   aliphatic hydrocarbons - SEE ATTACHED SHEET

   volatile and semi-volatile organics - SEE ATTACHED SHEET

   oil and grease - SEE ATTACHED SHEET

   strong oxidizers - GENERALLY NO SIGNIFICANT EFFECT

   acids - GENERALLY NO SIGNIFICANT EFFECT

   bases - GENERALLY NO SIGNIFICANT EFFECT

   dissolved metals, salts and nutrients - GENERALLY NO EFFECT

3. **Give an acceptable compatibility limit for each of the compounds on the following pages and certificate liner manufacturer:**

Signature of Liner Manufacturer:

____
Matthew W. Adams
Technical Support Chemist

____
Date

FORM-R/PAGE 1

Aromatic Halogenated Hydrocarbons

Aromatic Halogenated Hydrocarbons tend to be absorbed into polyethylene over long periods of time where they may function as a plasticizer. As a result, the polyethylene may swell and become softer and more elastic. These effects are generally reversible if the exposure is terminated.

Since polyethylene consists of a range of molecular weight molecules and somewhat different branching arrangements, some lower density polyethylenes may contain fractions that are extractable. Some types of chemical stabilizers and processing aids may also be extractable.

These above noted effects increase with increasing temperature. Softening, swelling and increased elasticity may rapidly reduce the usefulness of polyethylene as a structural component such as for use as a pressure pipe. Generally, these effects do not seriously affect the performance of polyethylene as a containment membrane.

GSE HyperFlex® polyethylene geomembranes are manufactured from a narrow molecular weight range resin designed to minimize the possibility of extractable fractions and maximize the resistance to stress cracking.

Aliphatic Halogenated Hydrocarbons

Similar effects as for Aromatic Halogenated Hydrocarbons but generally less severe. Some materials have little or no effect.

Aromatic Hydrocarbons *

Again similar to Aromatic Halogenated Hydrocarbons but generally less severe. Many materials have no significant effect.

Aliphatic Hydrocarbons

Again similar, but with further reductions of general severity. Most materials have no significant effect.

Volatile and Semivolatile Organics

These are mostly covered by the previously noted comments about hydrocarbons.

Oil and Grease

Mineral, vegetable and animal oils, fats or grease generally have no significant effect.

Strong Oxidizers - Generally no significant effect.

Acids - Generally no significant effect.

Dissolved Metals, Salts and Nutrients - Generally no effect.

FORM R

LINER COMPATABILITY

PARAMETER CLASSIFICATION	PARAMETER	MANUFACTURER'S LINER/LEACHATE LIMIT mg/l
Aromatic Halogenated Hydrocarbons	polychlorinated biphenyl *	(2000)
	aldrin	(2000)
	dichlorobenzene	(2000)
	hexachlorobenzene	(2000)
	pentachlorobenzene	(2000)
	trichlorobenzene	(2000)
	tetrachlorobenzene	(2000)
	2-chloronaphthalene	(2000)
	chloronaphthalene	(2000)
	chlorobenzene	(2000)
	4,4-DDT	(2000)
	4,4-DDE	(2000)
	4,4-DDD	(2000)
Aliphatic Halogenated Hydrocarbons	bromoform	(2000)
	carbon tetrachloride	(2000)
	chlorodibromomethane	(2000)
	chloroethane	(2000)
	chloroform	(2000)
	dichlorobromomethane	(2000)
	dichlorodifluoromethane	(2000)
	dichloroethane	(2000)
	dichloropropane	(2000)
	dichloroethene	(2000)
	ethylene chloride	(2000)
	ethylene dichloride	(2000)
	hexachloroethane	(2000)
	methyl bromide	(2000)
	methyl chloride	(2000)
	methylene chloride	(2000)
	tetrachloroethane	(2000)
	tetrachloroethene	(2000)
	trichloroethane	(2000)
	trichloroethene	(2000)
	trichlorofluoromethane	(2000)
	vinyl chloride	(2000)

TEMPERATURE 70 °F

FORM R
LINER COMPATABILITY

PARAMETER CLASSIFICATION	PARAMETER	MANUFACTURER'S LINER/LEACHATE LIMIT mg/l
Aromatic Hydrocarbons	acenapthene	(2000)
	acenaphthylene	(2000)
	anthracene	(2000)
	benzene	(2000)
	benzo(a)anthracene	(2000)
	benzo(a)pyrene	(2000)
	benzo(g,h,i)perylene	(2000)
	benzo(k)fluoranthene	(2000)
	3,4-benzoflouranthene	(2000)
	chrysene	(2000)
	dibenzo(a,h)anthracene	(2000)
	ethyl benzene	(2000)
	flouranthene	(2000)
	flourene	(2000)
	ideno(1,2,3,c,d)pyrene	(2000)
	naphthalene	(2000)
	phenanthrene	(2000)
	pyrene	(2000)
	styrene	(5000)
	toluene	(5000)
	xylene	(5000)
Aliphatic Hydrocarbons	heptane	(500,000)
	hexane	(500,000)
	octane	(500,000)

TEMPERATURE 70 °F

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FORM R

LINER COMPATABILITY

PARAMETER CLASSIFICATION	PARAMETER	MANUFACTURER'S LINER/LEACHATE LIMIT mg/l
Volatile & Semivolatile Organics	acrolein	(200,000)
	acrylonitrile	(200,000)
	acetone	(200,000)
	amyl acetate	(200,000)
	benzidine	(200,000)
	butyl alcohol	(500,000)
	bis(2-chloroethoxy)methane	(2,000)
	bis(2-chloroethosy)ether	(2,000)
	bis(2-chloroisopropy)ether	(2,000)
	bis(2-etylhexyl)pthphalate	(2,000)
	4-bromophenyl phenyl ether	(2,000)
	butyl benzyl phthalate	(200,000)
	cresol	(100,000)
	chlordane	(2,000)
	alpha-BHC	(2,000)
	beta-BHC	(2,000)
	gamma-BHC	(2,000)
	delta-BHC	(2,000)
	dieldrin	(2,000)
	dichlorobenzidine	(2,000)
	diethyl phthalate	(100,000)
	dibutyl phthalate	(100,000)
	dimethyl phthalate	(100,000)
	isobutyl alchohol	(500,000)
	isopropyl alcohol	(500,000)
	methyl alcohol	(500,000)
	2-chloroethyl vinyl ether	(2,000)
	2-chlorophenol	(2,000)
	dichlorophenol	(2,000)
	dimethyl phenol	(2,000)
	dinitro-o-cresol	(2,000)
	dinitrophenol	(2,000)
	dinitrotoluene	(2,000)
	diphenylhydrazine	(2,000)
	ethyl acetate	(100,000)
	ethyl ether	(2,000)
	ethyl glycol	(500,000)
	endosulfan	(2,000)
	endrin	(2,000)
	formaldehyde	(200,000)
	heptachlor	(2,000)
	hexachlorocyclopentadiene	(2,000)
	hexachlorobutadiene	(2,000)
	isophorone	(2,000)
	methyl ethyl ketone	(200,000)

TEMPERATURE 70 °F

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FORM R
LINER COMPATABILITY

PARAMETER CLASSIFICATION	PARAMETER	MANUFACTURER'S LINER/LEACHATE LIMIT mg/l
Volatile & Semivolotile Organics (cont.)	methyl isobutyl ketone	(500,000)
	nitrophenol	(100,000)
	N-nitrosodimethylamine	(100,000)
	N-nitrosodi-n-propylamine	(100,000)
	nitrobenzene	(100,000)
	pentachlorophenol	(100,000)
	phenol	(100,000)
	pyridine	(100,000)
	toxaphene	(100,000)
	trichlorophenol	(100,000)
	2,4,5-TP(silvex)	(?)

TEMPERATURE 70 °F

FORM R

LINER COMPATABILITY

PARAMETER CLASSIFICATION	PARAMETER	MANUFACTURER'S LINER/LEACHATE LIMIT mg/l
Acids & Bases	acetic acid	(500,000)
	chromic acid	(100,000)
	citric acid	(500,000)
	hydrobromic acid	(100,000)
	hydrochloric acid	(350,000)
	hydrocyanic acid	(100,000)
	hydrofluoric acid	(750,000)
	nitric acid	(500,000)
	picric acid	(500,000)
	phosphoric acid	(500,000)
	perchloric acid	(500,000)
	sulfuric acid	(500,000)
	potassium hydroxide	(500,000)
	sodium hydroxide	(500,000)
Products & Various Substances	antifreeze	(500,000)
	asphalt	(500,000)
	cresols	(100,000)
	crude oil	(500,000)
	diesel fuel	(500,000)
	fatty acids	(500,000)
	freon	(500,000)
	fuel oil	(500,000)
	gasoline	(500,000)
	hydraulic oil	(500,000)
	kerosene	(500,000)
	lacquers	(500,000)
	lubricating oil	(500,000)
	mineral spirits	(500,000)
	naphtha	(500,000)
	paraffin	(500,000)
	transformer oil	(500,000)
Miscellaneous	pH	(0.5-13.0 pH unit)
	strong oxidizers*	(1000-500,000)
	metals, salts, nutrients	(500,000)

*potassium permaganate, potassium dichromate, chlorine, peroxides

TEMPERATURE 70 °F

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GSE®

Technical Note

Chemical Resistance Chart

GSE is the world's leading supplier of high quality, polyethylene geomembranes. GSE polyethylene geomembranes are resistant to a great number and combinations of chemicals. Note that the effect of chemicals on any material is influenced by a number of variable factors such as temperature, concentration, exposed area and duration. Many tests have been performed that use geomembranes and certain specific chemical mixtures. Naturally, however, every mixture of chemicals cannot be tested for, and various criteria may be used to judge performance. Reported performance ratings may not apply to all applications of a given material in the same chemical. Therefore, these ratings are offered as a guide only. This information is provided for reference purposes only and is not intended as a warranty or guarantee. GSE assumes no liability in connection with the use of this information.

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
A			
Acetic acid	100%	S	L
Acetic acid	10%	S	S
Acetic acid anhydride	100%	S	L
Acetone	100%	L	L
Adipic acid	sat. sol.	S	S
Allyl alcohol	96%	S	S
Aluminum chloride	sat. sol.	S	S
Aluminum fluoride	sat. sol.	S	S
Aluminum sulfate	sat. sol.	S	S
Alum	sol.	S	S
Ammonia, aqueous	dil. sol.	S	S
Ammonia, gaseous dry	100%	S	S
Ammonia, liquid	100%	S	S
Ammonium chloride	sat. sol.	S	S
Ammonium fluoride	sol.	S	S
Ammonium nitrate	sat. sol.	S	S
Ammonium sulfate	sat. sol.	S	S
Ammonium sulfide	sol.	S	S
Amyl acetate	100%	S	L
Amyl alcohol	100%	S	L
Aniline	100%	S	L
Antimony trichloride	90%	S	S
Arsenic acid	sat. sol.	S	S
Aqua regia	HCl-HNO3	U	U
B			
Barium carbonate	sat. sol.	S	S
Barium chloride	sat. sol.	S	S
Barium hydroxide	sat. sol.	S	S
Barium sulfate	sat. sol.	S	S
Barium sulfide	sol.	S	S
Benzaldehyde	100%	S	L
Benzene	—	L	L
Benzoic acid	sat. sol.	S	S
Beer	—	S	S
Borax (sodium tetraborate)	sat. sol.	S	S
Boric acid	sat. sol.	S	S
Bromine, gaseous dry	100%	U	U
Bromine, liquid	100%	U	U
Butane, gaseous	100%	S	S
1-Butanol	100%	S	S
Butyric acid	100%	S	L
C			
Calcium carbonate	sat. sol.	S	S
Calcium chlorate	sat. sol.	S	S
Calcium chloride	sat. sol.	S	S
Calcium nitrate	sat. sol.	S	S
Calcium sulfate	sat. sol.	S	S
Calcium sulfide	dil. sol.	L	L
Carbon dioxide, gaseous dry	100%	S	S
Carbon disulfide	100%	L	U
Carbon monoxide	100%	S	S
Chloracetic acid	sol.	S	S
Carbon tetrachloride	100%	L	U
Chlorine, aqueous solution	sat. sol.	L	U
Chlorine, gaseous dry	100%	L	U
Chloroform	100%	U	U
Chromic acid	20%	S	L
Chromic acid	50%	S	L
Citric acid	sat. sol.	S	S

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
Copper chloride	sat. sol.	S	S
Copper nitrate	sat. sol.	S	S
Copper sulfate	sat. sol.	S	S
Cresylic acid	sat. sol.	L	—
Cyclohexanol	100%	S	S
Cyclohexanone	100%	S	L
D			
Decahydronaphthalene	100%	S	L
Dextrine	sol.	S	S
Diethyl ether	100%	L	—
Dioctylphthalate	100%	S	L
Dioxane	100%	S	S
E			
Ethanediol	100%	S	S
Ethanol	40%	S	L
Ethyl acetate	100%	S	U
Ethylene trichloride	100%	U	U
F			
Ferric chloride	sat. sol.	S	S
Ferric nitrate	sol.	S	S
Ferric sulfate	sat. sol.	S	S
Ferrous chloride	sat. sol.	S	S
Ferrous sulfate	sat. sol.	S	S
Fluorine, gaseous	100%	U	U
Fluorosilicic acid	40%	S	S
Formaldehyde	40%	S	S
Formic acid	50%	S	S
Formic acid	98-100%	S	S
Furfuryl alcohol	100%	S	L
G			
Gasoline	—	S	L
Glacial acetic acid	96%	S	L
Glucose	sat. sol.	S	S
Glycerine	100%	S	S
Glycol	sol.	S	S
H			
Heptane	100%	S	U
Hydrobromic acid	50%	S	S
Hydrobromic acid	100%	S	S
Hydrochloric acid	10%	S	S
Hydrochloric acid	35%	S	S
Hydrocyanic acid	10%	S	S
Hydrofluoric acid	4%	S	S
Hydrofluoric acid	60%	S	L
Hydrogen	100%	S	S
Hydrogen peroxide	30%	S	L
Hydrogen peroxide	90%	S	U
Hydrogen sulfide, gaseous	100%	S	S
L			
Lactic acid	100%	S	S
Lead acetate	sat. sol.	S	—
M			
Magnesium carbonate	sat. sol.	S	S
Magnesium chloride	sat. sol.	S	S
Magnesium hydroxide	sat. sol.	S	S
Magnesium nitrate	sat. sol.	S	S
Maleic acid	sat. sol.	S	S
Mercuric chloride	sat. sol.	S	S

- Continued -

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
Mercuric cyanide	sat. sol.	S	S
Mercuric nitrate	sol.	S	S
Mercury	100%	S	S
Methanol	100%	S	S
Methylene chloride	100%	L	—
Milk	—	S	S
Molasses	—	S	S
N			
Nickel chloride	sat. sol.	S	S
Nickel nitrate	sat. sol.	S	S
Nickel sulfate	sat. sol.	S	S
Nicotinic acid	dil. sol.	S	—
Nitric acid	25%	S	S
Nitric acid	50%	S	U
Nitric acid	75%	U	U
Nitric acid	100%	U	U
O			
Oils and Grease	—	S	L
Oleic acid	100%	S	L
Orthophosphoric acid	50%	S	S
Orthophosphoric acid	95%	S	L
Oxalic acid	sat. sol.	S	S
Oxygen	100%	S	L
Ozone	100%	L	U
P			
Petroleum (kerosene)	—	S	L
Phenol	sol.	S	S
Phosphorus trichloride	100%	S	L
Photographic developer	cust. conc.	S	S
Picric acid	sat. sol.	S	—
Potassium bicarbonate	sat. sol.	S	S
Potassium bisulfide	sol.	S	S
Potassium bromate	sat. sol.	S	S
Potassium bromide	sat. sol.	S	S
Potassium carbonate	sat. sol.	S	S
Potassium chlorate	sat. sol.	S	S
Potassium chloride	sat. sol.	S	S
Potassium chromate	sat. sol.	S	S
Potassium cyanide	sol.	S	S
Potassium dichromate	sat. sol.	S	S
Potassium ferricyanide	sat. sol.	S	S
Potassium ferrocyanide	sat. sol.	S	S
Potassium fluoride	sat. sol.	S	S
Potassium hydroxide	10%	S	S
Potassium hydroxide	sol.	S	S
Potassium hypochlorite	sol.	S	L
Potassium nitrate	sat. sol.	S	S
Potassium orthophosphate	sat. sol.	S	S
Potassium perchlorate	sat. sol.	S	S
Potassium permanganate	20%	S	S
Potassium persulfate	sat. sol.	S	S
Potassium sulfate	sat. sol.	S	S
Potassium sulfite	sol.	S	S
Propionic acid	50%	S	S
Propionic acid	100%	S	L
Pyridine	100%	S	L
Q			
Quinol (Hydroquinone)	sat. sol.	S	S
S			
Salicylic acid	sat. sol.	S	S

Medium	Concentration	Resistance at: 20 °C (68 °F)	60 °C (140 °F)
Silver acetate	sat. sol.	S	S
Silver cyanide	sat. sol.	S	S
Silver nitrate	sat. sol.	S	S
Sodium benzoate	sat. sol.	S	S
Sodium bicarbonate	sat. sol.	S	S
Sodium biphosphate	sat. sol.	S	S
Sodium bisulfite	sol.	S	S
Sodium bromide	sat. sol.	S	S
Sodium carbonate	sat. sol.	S	S
Sodium chlorate	sat. sol.	S	S
Sodium chloride	sat. sol.	S	S
Sodium cyanide	sat. sol.	S	S
Sodium ferricyanide	sat. sol.	S	S
Sodium ferrocyanide	sat. sol.	S	S
Sodium fluoride	sat. sol.	S	S
Sodium hydroxide	40%	S	S
Sodium hydroxide	sat. sol.	S	S
Sodium hypochlorite	15% active chlorine	S	S
Sodium nitrate	sat. sol.	S	S
Sodium nitrite	sat. sol.	S	S
Sodium orthophosphate	sat. sol.	S	S
Sodium sulfate	sat. sol.	S	S
Sodium sulfide	sat. sol.	S	S
Sulfur dioxide, dry	100%	S	S
Sulfur trioxide	100%	U	U
Sulfuric acid	10%	S	S
Sulfuric acid	50%	S	S
Sulfuric acid	98%	S	U
Sulfuric acid	fuming	U	U
Sulfurous acid	30%	S	S
T			
Tannic acid	sol.	S	S
Tartaric acid	sol.	S	S
Thionyl chloride	100%	L	U
Toluene	100%	L	U
Triethylamine	sol.	S	L
U			
Urea	sol.	S	S
Urine	—	S	S
W			
Water	—	S	S
Wine vinegar	—	S	S
Wines and liquors	—	S	S
X			
Xylenes	100%	L	U
Y			
Yeast	sol.	S	S
Z			
Zinc carbonate	sat. sol.	S	S
Zinc chloride	sat. sol.	S	S
Zinc (II) chloride	sat. sol.	S	S
Zinc (IV) chloride	sat. sol.	S	S
Zinc oxide	sat. sol.	S	S
Zinc sulfate	sat. sol.	S	S

Specific immersion testing should be undertaken to ascertain the suitability of chemicals not listed above with reference to special requirements.

NOTES:

(S) Satisfactory: *Liner material is resistant to the given reagent at the given concentration and temperature. No mechanical or chemical degradation is observed.*

(L) Limited Application Possible: *Liner material may reflect some attack. Factors such as concentration, pressure and temperature directly affect liner performance against the given media. Application, however, is possible under less severe conditions, e.g. lower concentration, secondary containment, additional liner protections, etc.*

(U) Unsatisfactory: ***Liner material is not resistant to the given reagent at the given concentration and temperature. Mechanical and/or chemical degradation is observed.***

(–) Not tested

sat. sol. = *Saturated aqueous solution, prepared at 20°C (68°F)*

sol. = *aqueous solution with concentration above 10% but below saturation level*

dil. sol. = *diluted aqueous solution with concentration below 10%*

cust. conc. = *customary service concentration*

TN032 ResistChart R03/17/06

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Middle East	GSE Lining Technology-Egypt	The 6th of October City, Egypt		202 2 828 8888	Fax: 202 2 828 8889

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Appendix D

Suggested Maximum Leachate Concentrations for Individual Constituents

Table D-1. Suggested maximum leachate concentrations for organic constituents for liner compatibility.

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/L)	Compatible Concentration For GCL (mg/L)	Compatible Concentration For Clay (mg/L)	Suggested Maximum Leachate Concentration[c] (mg/L)
1,1,1-Trichloroethane	0.0609	-[d]	-[d]	20[e]	20
1,1,2,2-Tetrachloroethane	0.0002	-	-	-	-
1,1,2-Trichloroethane	0.0013	-	-	-	-
1,1-Dichloroethane	0.0105	-	-	-	-
1,1-Dichloroethene	0.0004	-	-	-	-
1,2,4-Trichlorobenzene	0.0113	-	-	-	-
1,2-Dichlorobenzene	0.0734	-	-	-	-
1,2-Dichloroethane	0.0001	-	-	-	-
1,2-Dichloroethene (total)	0.0003	-	-	-	-
1,3-Dichlorobenzene	0.0071	2,000[f]	-	-	2,000
1,4-Dichlorobenzene	5.1303	-	-	-	-
1,4-Dioxane	0.0000	-	-	-	-
2,4,5-Trichlorophenol	0.0441	-	-	-	-
2,4,6-Trichlorophenol	0.0427	-	-	-	-
2,4-Dichlorophenol	0.0371	-	-	-	-
2,4-Dimethylphenol	0.3041	-	-	-	-
2,4-Dinitrophenol	0.1705	-	-	-	-
2,4-Dinitrotoluene	0.0488	-	-	-	-
2,6-Dinitrotoluene	0.2903	-	-	-	-
2-Butanone	0.0063	200,000[f]	-	-	200,000
2-Chloro-phthalene	0.0108	2,000[g]	-	-	2,000
2-Chlorophenol	0.1867	2,000[g]	-	-	2,000
2-Hexanone	0.0001	-	-	-	-
2-Methyl-phthalene	1.7772	-	-	-	-

Table D-1. (continued).

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/L)	Compatible Concentration For GCL (mg/L)	Compatible Concentration For Clay (mg/L)	Suggested Maximum Leachate Concentration[c] (mg/L)
2-Methylphenol	0.2014	-	-	-	-
2-Nitroaniline	0.1728	-	-	-	-
2-Nitrophenol	0.0098	-	-	-	-
3,3'-Dichlorobenzidine	0.1896	-	-	-	-
3-Methyl Buta-l	0.0022	-	-	-	-
3-Nitroaniline	0.0165	-	-	-	-
4,6-Dinitro-2-methylphenol	0.0010	-	-	-	-
4-Bromophenyl-phenylether	0.0615	2,000[g]	-	-	2,000
4-Chloro-3-methylphenol	0.0810	-	-	-	-
4-Chloroaniline	0.0052	-	-	-	-
4-Chlorophenyl-phenylether	0.0288	-	-	-	-
4-Methyl-2-Pentanone	0.1131	-	-	-	-
4-Methylphenol	0.3766	-	-	-	-
4-Nitroaniline	0.1728	-	-	-	-
4-Nitrophenol	0.0029	-	-	-	-
Ace-phthene	0.0399	2,000[g]	-	-	2,000
Ace-phthylene	0.3366	2,000[g]	-	-	2,000
Acetone	6.2674	200,000[g]	-	-	100,000
Acetonitrile	0.0002	-	-	-	-
Acrolein	0.0001	200,000[g]	-	-	200,000
Acrylonitrile	0.0000	200,000[g]	-	-	200,000
Anthracene	0.0083	2,000[g]	-	-	2,000
Aramite	0.0000	-	-	-	-
Aroclor-1016	0.0000	-	-	-	-

Table D-1. (continued).

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/L)	Compatible Concentration For GCL (mg/L)	Compatible Concentration For Clay (mg/L)	Suggested Maximum Leachate Concentration[c] (mg/L)
Aroclor-1254	0.0002	-	-	-	-
Aroclor-1260	0.0087	-	-	-	-
Aroclor-1268	0.2891	-	-	-	-
Benzene	1.3491	2,000[g]	-	-	1,000
Benzidine	0.0000	200,000[g]	-	-	200,000
Benzo(a)anthracene	0.0001	2,000[g]	-	-	2,000
Benzo(a)pyrene	0.0000	2,000[g]	-	-	2,000
Benzo(b)fluoranthene	0.0000	2,000[g]	-	-	2,000
Benzo(g,h,i)perylene	0.0000	-	-	-	-
Benzo(k)fluoranthene	0.3024	-	-	-	-
Benzoic acid	0.1162	-	-	-	-
bis(2-Chloroethoxy)methane	0.0455	2,000[g]	-	-	2,000
bis(2-Chloroethyl)ether	0.0535	2,000[g]	-	-	2,000
bis(2-Chloroisopropyl)ether	0.0000	2,000[g]	-	-	2,000
bis(2-Ethylhexyl)phthalate	0.5714	2,000[g]	-	-	2,000
Butane,1,1,3,4-Tetrachloro-	0.0001	-	-	-	-
Butylbenzylphthalate	0.0080	200,000[g]	-	-	200,000
Carbazole	0.1856	-	-	-	-
Carbon Disulfide	0.0734	-	-	-	-
Chlorobenzene	0.0679	2,000[g]	-	-	2,000
Chloroethane	0.0000	-	-	-	-
Chloromethane	0.0000	2,000[g]	-	-	2,000
Chrysene	4.4199	2,000[g]	-	-	2,000

Table D-1. (continued).

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/L)	Compatible Concentration For GCL (mg/L)	Compatible Concentration For Clay (mg/L)	Suggested Maximum Leachate Concentration[c] (mg/L)
Decane, 3,4-Dimethyl	0.0004	-	-	-	-
Diacetone alcohol	0.0005	-	-	-	-
Dibenz(a,h)anthracene	0.0006	2,000[g]	-	-	2,000
Dibenzofuran	0.4156	-	-	-	-
Diethylphthalate	0.1897	100,000[g]	-	-	100,000
Dimethyl Disulfide	0.0127	-	-	-	-
Dimethylphthalate	0.0001	100,000[g]	-	-	100,000
Di-n-butylphthalate	0.0000	100,000[f]	-	-	100,000
Di-n-octylphthalate	0.4370	-	-	-	-
Eicosane	0.0472	-	-	-	-
Ethyl cyanide	0.0000	-	-	-	-
Ethylbenzene	0.0705	2,000[g]	-	-	2,000
Famphur	0.0000	-	-	-	-
Fluoranthene	0.0221	2,000[g]	-	-	2,000
Fluorene	3.0594	2,000[g]	-	-	2,000
Heptadecane, 2,6,10,15-Tetra	0.0000	-	-	-	-
Hexachlorobenzene	0.0001	2,000[g]	-	-	2,000
Hexachlorobutadiene	0.0000	2,000[g]	-	-	2,000
Hexachlorocyclopentadiene	0.0025	2,000[g]	-	-	2,000
Hexachloroethane	0.0000	2,000[g]	-	-	2,000
Indeno(1,2,3-cd)pyrene	0.1585	2,000[g]	-	-	2,000
Isobutyl alcohol	0.0001	500,000[g]	-	-	500,000
Isophorone	0.1829	2,000[g]	-	-	2,000

Table D-1. (continued).

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/L)	Compatible Concentration For GCL (mg/L)	Compatible Concentration For Clay (mg/L)	Suggested Maximum Leachate Concentration[c] (mg/L)
Isopropyl Alcohol/2-propanol	0.0000	500,000[g]	-	-	500,000
Kepone	0.2511	-	-	-	-
Mesityl oxide	1.2939	-	-	-	-
Methyl Acetate	0.0057	-	-	-	-
Methylene Chloride	0.0165	2,000[g]	-	20[e]	20
-phthalene	1.9193	2,000[g]	-	-	2,000
Nitrobenzene	0.0948	100,000[g]	-	-	100,000
N-Nitroso-di-n-propylamine	0.0035	100,000[g]	-	-	100,000
N-Nitrosodiphenylamine	0.1896	100,000[g]	-	-	100,000
Octane,2,3,7-Trimethyl	0.0027	-	-	-	-
o-Toluenesulfo-mide	0.0033	-	-	-	-
Pentachlorophenol	0.0046	100,000[g]	-	-	100,000
Phe-nthrene	8.8500	2,000[g]	-	-	2,000
Phenol	0.1370	100,000[g]	-	-	100,000
Phenol,2,6-Bis(1,1-Dimethyl)	0.0674	-	-	-	-
p-Toluenesulfo-mide	0.0000	-	-	-	-
Pyrene	3.2501	2,000[g]	-	-	2,000
RDX	0.0000	5,000[g]	-	-	5,000
Styrene	0.0000	2,000[g]	-	-	2,000
Tetrachloroethene	0.0235	5,000[g]	-	20[h]	20
Toluene	16.3666	-	-	-	-
Tributylphosphate	1.2292	2,000[g]	-	1,100[e]	1,100

Table D-1. (continued).

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/L)	Compatible Concentration For GCL (mg/L)	Compatible Concentration For Clay (mg/L)	Suggested Maximum Leachate Concentration[c] (mg/L)
Trichloroethene	1.1526	-	-	-	-
Trinitrotoluene	0.0000	-	-	-	-
Undecane,4,6-Dimethyl-	0.0003	5,000[g]	-	-	5,000
Xylene (ortho)	0.0071	-	-	-	-
Xylene (total)	6.2805	-	-	-	-

Notes
a. Constituent reported in the "INEEL CERCLA Disposal Facility Design Inventory (EDF-ER-264).
b. Predicted leachate concentration in the first year of the ICDF landfill operation (EDF-ER-274).
c. The suggested maximum concentration selected for the ICDF liner system is based on the lowest of the concentrations listed for HDPE, GCL, and clay materials and are applicable for the leachate in the landfill and the waste liquids in the evaporation ponds.
d. "–" indicates that a specific test value was not available, compatibility issues are not anticipated.
e. The TCE solubility limit in water is 1,100 mg/l. A minimum of 2 pore volumes of permeant liquid was passed through the clay sample or until the concentration of total organic carbon in the influent and effluent were the same (Bowders and Daniel 1988). No significant change in permeability was observed.
f. From "Evaluation of Liner/Leachate Chemical Compatibility for the Environmental Restoration Disposal Facility," BHI-00359.
g. From manufacturer specifications.
h. 20 mg/l is the typical concentration of leachate found in muncipal landfills. No change in clay permeability was observed at this concentration (Kim, Tuncer, and Park 1999).

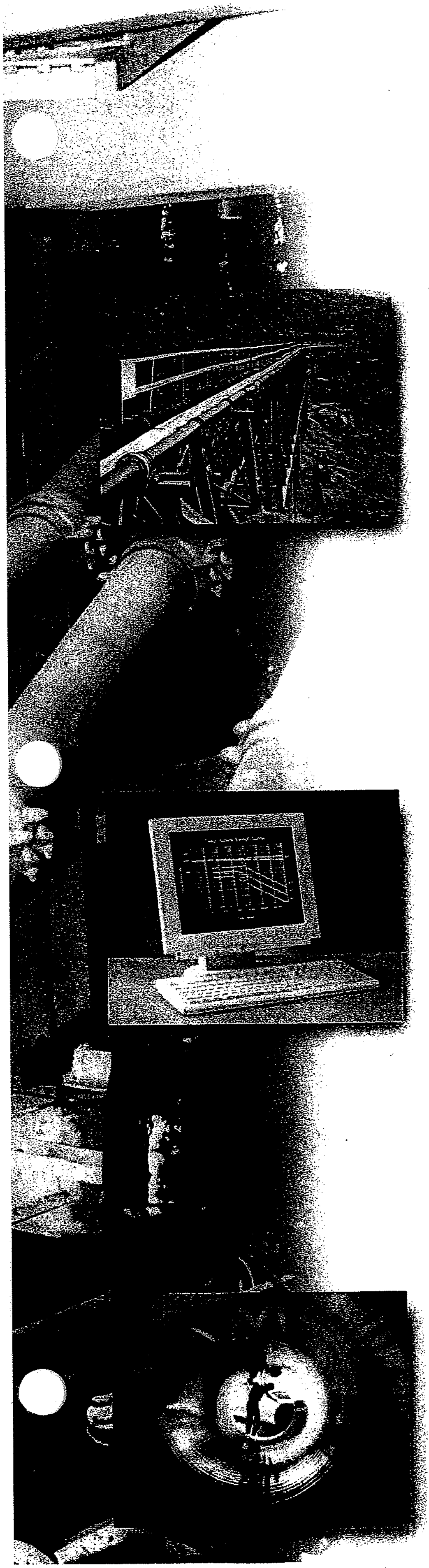
Table D-2. Suggested maximum leachate concentrations for inorganic constituent for liner compatibility.

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/l)	Compatible Concentration For GCL (mg/l)	Compatible Concentration For Clay (mg/l)	Suggested Maximum Leachate Concentration[c] (mg/l)
Aluminum	28.3029				
Antimony	0.1165				
Arsenic	1.8470	500,000[d]	-	-[e]	500,000
Barium	3.5848				
Beryllium	0.0011				
Boron	36.4728	500,000[d]	-	-[e]	500,000
Cadmium	0.5917	500,000[d]	-	-[e]	500,000
Calcium	4035.0217	500,000[d]	-	-[e]	500,000
Chloride	31.1061				
Chromium	1.3691				
Cobalt	0.5999	500,000[d]	-	-[e]	500,000
Copper	1.4906	500,000[d]	-	-[e]	500,000
Cyanide	4.0932	500,000[d]	-	-[e]	500,000
Dysprosium	0.2472				
Fluoride	64.4341				
Iron	46.5528				
Lead	0.5753				
Magnesium	883.9838	500,000[d]	-	-[e]	500,000
Manganese	4.1300				
Mercury	49.6286				
Molybdenum	1.0117	500,000[d]	-	-[e]	500,000
Nickel	0.1964				
Nitrate	65.4429				
Nitrate/Nitrite-N	3.6979				
Nitrite	0.1414				
Phosphorus	19.2492	500,000[d]	-	-[e]	500,000
Potassium	74.8819	500,000[d]	-	-[e]	500,000
Selenium	0.2084	500,000[d]	-	-[e]	500,000
Silver	0.1092				
Sodium	2.7716				
Strontium	1.5094	500,000[d]	-	-[e]	500,000

Table D-2. (continued).

Constituent[a]	Predicted Concentration in Leachate[b] (mg/L)	Compatible Concentration For HDPE (mg/l)	Compatible Concentration For GCL (mg/l)	Compatible Concentration For Clay (mg/l)	Suggested Maximum Leachate Concentration[c] (mg/l)
Sulfate	342.1180				
Sulfide	12641.8391				
Terbium	2.3867				
Thallium	0.0037				
Va-dium	3.5063	500,000[d]	-	-[e]	500,000
Ytterbium	0.8124				
Zinc	12.9486	500,000[4]	-	-[e]	500,000
Zirconium	0.1151				
Total Inorganic	18367.1936				

a. Constituent reported in the "INEEL CERCLA Disposal Facility Design Inventory (EDF-ER-264).
b. Predicted leachate concentration in the first year of the ICDF landfill operation (EDF-ER-274).
c. The suggested maximum concentration selected for the ICDF liner system is based on the lowest of the concentrations listed for HDPE, GCL, and clay materials and are applicable for the leachate in the landfill and the waste liquids in the evaporation ponds.
d. From manufacturer specifications
e. From manufacturer specifications



high density polyethylene pipe

Sclairpipe®

Systems Design

KWH PIPE

Chemical Resistance

INTRODUCTION

Outstanding resistance to both internal and external chemical attack has made the SCLAIRPIPE system the material of choice for the transport of lower temperature (below 150°F) fluids in adverse chemical environments. High-density polyethylene is chemically inert to a wide range of industrial chemicals.

The chemical, its concentration in the fluid, its temperature, its contact time with the piping material and other service conditions, determines the suitability and expected service life of the SCLAIRPIPE system for the application. For most bases, acids, inorganic salts and other chemicals, you usually apply the same design parameters as considered for water service conditions. Chemical attack of SCLAIRPIPE may be divided into three categories: OXIDATION, STRESS-CRACKING and PLASTICIZATION.

OXIDIZERS are the only group of materials which are capable of chemically degrading the SCLAIRPIPE system. Some strong oxidizers have only a gradual effect on the pipe, therefore short-term effects are not measurable. If continuous exposure is expected, chemical effects should be defined. The following oxidizers are unsuitable for long-term contact with the SCLAIRPIPE system: Nitric acid (fuming), Sulphuric acid (fuming), Aqua Regia, wet chlorine gas and liquid bromine. However, weaker solutions of mineral acids, such as battery acid or reagent nitric acid, do not attack the pipe. Other common oxidizing agents, such as hydrochloric acid, hydrofluoric acid, hydrobromic acid and hydrogen peroxide have been shown to have no measurable effects on SCLAIRPIPE after 3 or 4 years' exposure.

STRESS CRACKING AGENTS are chemicals that accelerate the cracking of polyethylene when subjected to stress, but have no chemical effect on the material itself. Although some polyethylenes are extremely sensitive to brittle fracture, SCLAIRPIPE is highly resistant to this type of failure.

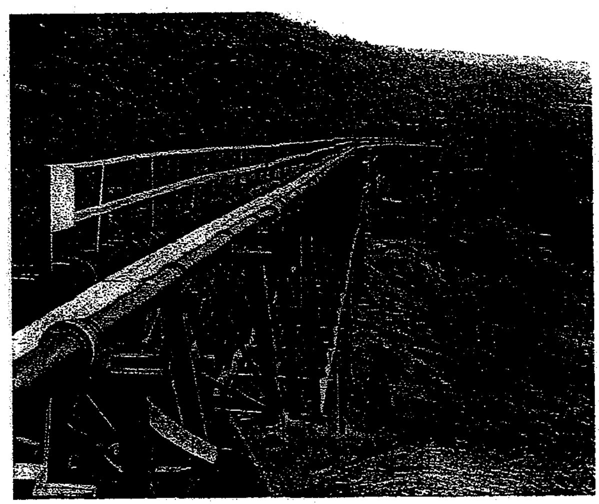
PLASTICIZERS are chemicals that can be absorbed to varying degrees by polyethylene, causing softening, some loss of yield strength and some gain in impact strength. These plasticizing materials cause no chemical degradation of polyethylene and they are not solvents for the material. SCLAIRPIPE is designed to give high resistance to this absorption and consequent weakening, but if it is to be exposed continuously to these environments, an added safety factor should be applied. Some of these materials are sufficiently volatile that when they are removed, the pipe will "dry out" and return to its original strength. Intermittent exposure to these materials, therefore, has little or no effect on SCLAIRPIPE.

GENERAL GUIDE TO RESISTANCE OF SCLAIRPIPE TO VARIOUS CHEMICALS

This chemical resistance chart is a comprehensive listing of chemicals, concentrations and pipe resistance at two temperatures. In all cases, SCLAIRPIPE at higher temperatures should be considered to have variable resistance. Contact your KWH Pipe representative for design assistance in these applications.

CODE:
- R = Resistant
- VR = Variable resistance, depending on conditions*
- NR = Not resistant
- O = Oxidizer
- P = Plasticizer
- SC = Potential stress-cracker

*The classification "variable resistance" is very broad. Depending on the nature of the chemical, its concentration, the service temperature and pressure and the time of exposure, SCLAIRPIPE can be either very resistant or very susceptible to attack. Therefore, when SCLAIRPIPE is said to have variable resistance to a chemical, it is strongly recommended that caution be exercised and that the specific application be discussed with a technical representative of KWH Pipe.



Installation of this 8 inch series 125 SCLAIRPIPE tailings line called for it to be supported on a trestle to maintain grade. Note the guides located at regular intervals to hold the pipe on the trestle during thermal expansion and contraction situations.

Polyethylene's abrasion resistance and chemical inertness were prime considerations in the decision to specify SCLAIRPIPE in this process pipe application.

Index

		73°F	120°F
Acetic acid, 20%	SC	R	R
Acetic acid, 80%	SC	VR	NR
Acetone	SC	NR	NR
Alcohol, ethyl		R	VR
Alcohol, isopropyl		R	R
Alcohol, methyl		R	R
Aluminum salts		R	R
Alums		R	R
Ammoniacal liquor		R	R
Amyl acetate		VR	NR
Aniline		R	R
Aqua Regia	O	NR	NR
Arsenic acid, 80%		R	R
Barium salts		R	R
Beer		R	R
Benzone (benzol)	P	NR	NR
Benzoic acid		R	R
Bleach plant wastes		R	R
Bleach 12.5% active chlorine		R	NR
Bleach 5.5% active chlorine		R	NR
Boric acid		R	R
Bromine, liquid	O	NR	NR
Bromic acid		NR	NR
Brine		R	R
Butadiene		R	VR
Butane		R	R
Butylene		R	R
Calcium salts		R	R
Calcium hydroxide		R	R
Calcium hypochlorite		R	R
Carbon disulfide	P	NR	NR
Carbon tetrachloride	P	N	NR
Chloric acid, 20%		R	NR
Chlorinated water		R	R
Chlorine (gas or liquid)	O	NR	NR
Chlorobenzene	P	NR	NR
Chloroform		NR	NR
Chromic acid, 50%		R	R
Copper salts		R	R
Corn Oil		R	VR
Cresol	P	NR	NR
Creosote, coatings	P	NR	NR
Cyclohexane	P	R	VR
Cyclohexanol	P	NR	NR
Detergent, synthetic	SC	R	R
Developers, photographic		R	R
Dextrin		R	R
Dichloroacetic acid		R	R
Dichlorobenzene	P	VR	NR
Dichloroethylene	P	NR	NR
Diesel fuels	P	R	VR
Diethylene glycol		R	R
Dimethylamine		VR	VR
Ethers		NR	NR
Ethylene glycol		R	R
Ethylene dichloride		NR	NR
Fatty acids		NR	NR
Ferric salts		R	R
Ferrous salts		R	R
Flourine, aqueous		VR	NR
Formaldehyde		R	R
Formic acid	O	R	NR
Fuel oil	P	VR	NR
Furfural		NR	NR
Gas, natural methane		R	R
Gasoline	P	NR	NR
Gelatin		R	R
Glycerine		R	R
Glycols		R	R
Glycolic acid		R	R
Heating oil	P	VR	VR
Hexane		R	VR
Hydrobromic acid, 20%		R	R
Hydrochloric acid, 30%		R	VR
Hydrofluoric acid, 10%		R	R
Hydrogen peroxide, 90%		R	NR

		73°F	120°F
Hydrogen sulfide		R	R
Hypochlorous acid		R	R
Iodine, alc. sol.		NR	NR
Isooctane	P	VR	VR
Kerosene	P	NR	NR
Ketones		R	VR
Lactic acid, 25%		R	R
Lead acetate		R	R
Linseed Oil		NR	NR
Lubricating oils	P	VR	NR
Magnesium salts		R	R
Maganese sulface		VR	VR
Mercury		R	R
Methyl bromide		NR	NR
Methyl chloride		NR	NR
Methyl cyclohexane	P	VR	VR
Methyl ethyl ketone		R	R
Mineral oils	P	VR	NR
Mixed acids (sulfuric & Nitric)	P	NR	NR
Mixed acids (sulfuric & phosphoric)	P	R	VR
Molasses		R	R
Monochlorobenzene		NR	NR
Naphtha	P	VR	NR
Nitric acid, 0 - 50%		R	VR
Nitric acid, 60%	O	VR	NR
Nitric acid, fuming	O	NR	NR
Nitrous acid		R	NR
Oil, animal & vegetable	P	NR	NR
Oleic acid		NR	NR
Oleum		NR	NR
Oxalic acid		R	R
Paraffin		VR	NR
Perchloric acid, 10 - 70%		R	R
Petroleum, crude asphaltic		NR	NR
Petroleum, crude paraffinic		NR	NR
Phenol		VR	NR
Phosgene, gas		VR	VR
Phosgene, liquid		NR	NR
Potassium salts		R	R
Potassium permanganate, 25%		VR	VR
Propylene glycol		R	R
Pulp-mill wastes (red & black liquor)		R	R
Sea water		R	R
Sewage, residential		R	R
Silicic acid		R	R
Silicone oil		R	VR
Silver salts		R	R
Soap solution (concentrated)		R	R
Sodium salts		R	R
Sodium chlorite		VR	NR
Sodium chlorate		R	VR
Sodium hydroxide (caustic soda)		R	R
Sodium hypochlorite		R	R
Stannous chloride		R	R
Starch solution		R	R
Stearic acid		R	R
Sulfite liquor		R	R
Sulfur dioxide		R	R
Sulfuric acid, 0 - 90%		R	NR
Sulfuric acid, 90 - 100%	O	NR	NR
Sulfurous acid		R	R
Tannic acid		R	R
Tartaric acid		R	R
Tetrabromoethane	P	NR	NR
Tetrachloroethane	P	NR	NR
Tetrahydrofuran	P	NR	NR
Toluene	P	NR	NR
Transformer oil	P	VR	VR
Trichloroethylene		NR	NR
Turpentine	P	VR	NR
Urea		R	R
Vinegar		R	R
Whiskey		R	R
Xylene		NR	NR
Zinc salts		R	R

Design Considerations Related to Environment

BIOLOGICAL REACTIONS

Polyethylene is inert to biological degradation. It is indigestible, has no food value and will not support the growth of organisms of any kind.

Algae and Marine Growths:
The smooth surface of SCLAIRPIPE polyethylene pipe, particularly on the inside, discourages the adherence of algae growths. Under essentially static conditions of flow, algae may deposit on the inside walls, but they flush off readily at low velocities of flow. Barnacles, limpets and other similar types of marine growth are not attracted to the surface of SCLAIRPIPE; where they have become established, their size of growth and thickness of encurstation have been significantly smaller than those associated with other materials.

Termites, etc.:
SCLAIRPIPE is not attacked by termites, ants or other burrowing insects, or by marine worms such as teredos.

Rodents:
SCLAIRPIPE can be damaged by rodents but is not preferentially attacked by them. In ground infested by gophers or groundhogs, pipe should be placed more than 30 inches below the surface.

Toxicity:
The resin compound used in the manufacture of SCLAIRPIPE contains nothing which can be extracted by prolonged contact with water. It imparts no taste or odours to potable water. The antioxidant added to the compound to prevent thermal degradation during processing is of a type and in a quantity approved by Food and Drug control administrations in Canada, the United States, Great Britain and most European countries, for contact with food and potable water.

SUNLIGHT AND WEATHER

SCLAIRPIPE contains finely divided and thoroughly dispersed carbon black which gives virtually permanent protection against ultra-violet light. However, if pipe is intended for installation above ground, particularly in desert locations, it should be remembered that other problems may arise related to temperature differentials rather than simple degradation. These are discussed below under "Temperature".

Exposure to conditions of alternating wetness and dryness or freezing and thawing does not require any special precaution.

TEMPERATURE

Operating Temperatures:
As with all homogeneous thermoplastic piping, polyethylene pipe loses stiffness and tensile strength as its temperature increases. SCLAIRPIPE is not normally recommended for use at temperatures in excess of 140°F. For systems where the service temperature frequently exceeds 73°F, the rated allowable working pressure of the pipe should be decreased.

The required pipe DR rating can be selected by using the minimum pressure rating, determined from Figure 4. The pipe chosen should have a long-term pressure rating at least as high as that determined from the following relationship:

$$\text{REQUIRED PIPE PRESSURE RATING} = \frac{\text{MAXIMUM OPERATING PRESSURE}}{\text{THERMAL SERVICE FACTOR}}$$

The required pipe DR rating is the closest available pressure rating above the calculated pressure rating.

The graph in Figure 4 is based on the same information as is used to derive the service life and margin of safety recommended for pipe operating at 73°F. The broken section of the curve is based on limited extrapolations of laboratory data. Care should be taken to ensure that pressure ratings determined by using this section are based on the worst possible conditions of temperature and pressure.

THERMAL SERVICE FACTOR

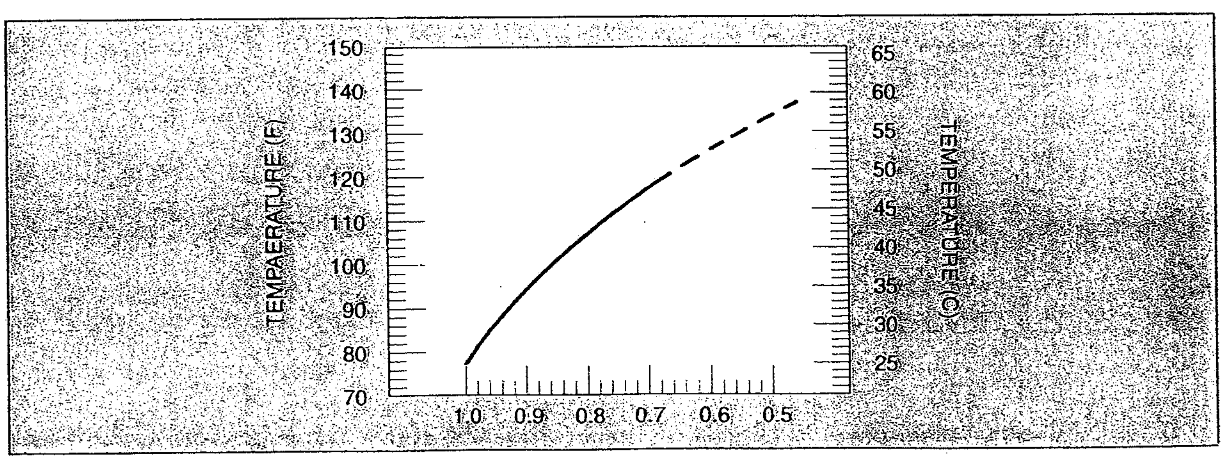


Figure 4: Thermal Service Factors for SCLAIRPIPE used at Service Temperatures higher than 73°F.

For gravity pipe in which the internal pressure is effectively zero, the service temperature should not exceed 150°F. Where there is high external stress on the pipe, the pipe DR rating may have to be selected by using the same thermal service factor as is used for pressure pipe. If the service requirements warrant a reduced service life or a different margin of safety, a qualified representative of KWH Pipe should be consulted for assistance.

Thermal Expansion:
The coefficient of thermal expansion for SCLAIRPIPE under completely unrestrained conditions is 8 x 10^{-5} in./in./°F. (14 x 10^{-5}cm./cm.°C). However, in most conditions of installation, some restraint is automatically provided. With pipes of 4 inch nominal diameter or greater, simple burial under 2 feet or more of soil usually provides ample restraint. Under these conditions, expansion or contraction due to temperature changes does not occur and no design considerations are required to provide restraint. Pipe installed in a trench should be at the temperature of the trench bottom before backfilling is started. The temperature differences after backfilling will not have any contraction or expansion effects because of the friction between the soil and pipe.

Smaller diameter pipes, i.e. $^1/_2$" to 3", should be snaked during installation in the trench, regardless of the burial depth, to increase the restraint available from friction with the soil.

If unrestrained, a pipeline installed above ground will tend to move laterally as a result of temperature changes, especially if the line is empty. If space is limited, or if the line is installed on a pipe bridge, restraining supports must be provided. When lateral movement is restricted, expansion will take place in either length or diameter, whichever is less restrained. (See Construction brochure, Surface Installation Section, for further details).

Of particular importance in design is the condition in which pipe passes from an area of adequate restraint into an area of poor restraint. Failures can result if the pipe and connections do not have adequate support at points of transition from large fixed structures to less restricting conditions. (See Construction brochure, Buried Installation Section, for further details).

Thermal Conducitivity:
Polyethylene is a relatively poor conductor of heat compared to metals. The coefficient of thermal conductivity for SCLAIRPIPE is approximately 2.5 BTU/hr/ft^2/°F. per inch of thickness. As a result, temperatures which are unevenly applied do not dissipate readily and thermal effects can be localized.

This property can be used to advantage in water systems in cold climates. The slow heat transfer inhibits freezing and, if the usual precautions are taken with respect to depth of burial, accidental freezing is practically eliminated. If the pipe does freeze, it does not burst and will resume its function upon thawing. Cyclical freezing, as in lines used for summer service only, is well tolerated but it is recommended that such lines be depressurized at shutdown. Irrigation lines have been operated in this way for many seasons without damage.

Localization of heated areas can cause noticeable deformation of the pipe. Solar heat, absorbed on one side of the pipe, is not readily conducted to the other side. Lines installed on the surface of the ground, unprotected from solar exposure, will require extensive anchoring to confine and control movement. The principle of design for such systems is to ensure that the movement is controlled over short lengths and is confined within a convenient plane where room to accommodate the movement can be provided.

INCIDENTAL DAMAGE

Despite its toughness and resilience, SCLAIRPIPE may be scuffed or scratched on the outside surface during handling. This does not affect its serviceability unless severe gouging or cutting takes place. In general, specifications should call for repair or removal of pipe which is gouged to depths greater than 10% of the wall thickness. V-shaped cuts of any depth occurring on the inside of the pipe must be removed.

At Kirkland Lake in Northern Ontario, a gravity sanitary sewer of 18 inch SCLAIRPIPE is installed in a rock tunnel. Pipe is laid above ground, tied down to wooden sleepers and secured with rock anchors.

In this mine tailings applications, 36 inch series 60 SCLAIRPIPE is installed at grade on a prepared right of way. The pipe routing incorporates a number of gentle bends to accommodate thermal expansion and contraction forces, minimizing the overall stresses on the pipe.

The durability of HDPE geomembranes

By L.G. Tisinger and J.P. Giroud

Excellent papers have been written on the durability of high density polyethylene (HDPE) geomembranes. Since the subject is very complex, however, many of these papers can be understood only by polymer scientists. Because information on the durability of HDPE geomembranes is very important, such information needs to be presented to the wide range of geomembrane users.

In this article, aspects of materials' durability that relate to the composition and the structure of the material used in geomembrane will be discussed. Mechanical actions, including stress cracking, and aspects related to the durability of the geomembrane seams will not be addressed.

From low to high density

Polyethylene is a polymer. A polymer is a molecule that has many units (from the Greek, poly, which means many, and meros, which means part). In contrast, a monomer is a single unit (from the Greek monos, which means single). Polymers are made from monomers through a reaction called polymerization.

For example, a polyethylene polymer results from the polymerization reaction of the ethylene monomer (Seymour and Carraher, 1981).

Production of polyethylene began in the mid-1930s from a process using high pressure and high temperature (Brydson, 1982). In the mid 1950s, new reaction conditions were introduced in which polyethylene was produced at lower pressures and lower temperatures than before.

As a result, a new variety of polyethylene was made that had a higher softening point, a higher density and more rigidity than earlier types.

This new variety of polyethylene was appropriately named high density polyethylene, while the name low density polyethylene (LDPE) became used to designate the type of polyethylene produced with the early process.

Anatomy of HDPE

The high density of HDPE results from the presence of many crystals of polyethylene molecules within its structure. Crystals are regions in which matter is ordered and densely packed.

The crystalline regions are connected by less organized, or amorphous regions, hence the terminology semicrystalline structure. The amount of crystalline regions in a material is typically expressed as crystallinity, a ratio that varies between 0 percent for a totally amorphous material and 100 percent for a totally crystalline material. Crystallinity, measured by differential scanning calimetry, is the ratio of the energy required to melt a given HDPE to the energy required to melt a totally crystalline HDPE.

Because they are composed of densely packed matter, crystals are essentially impermeable to liquids and chemicals. Clearly, a relationship exists between the number of crystals, the density of polyethylene and the impermeability of the geomembrane.

HDPE used to produce geomembranes is made not only from ethylene. It also contains some comonomer (a monomer in addition to ethylene at a proportion of approximately 1 percent to 3 percent), such as butene, hexene or octene. Comonomers result in more branching on the polyethylene molecules of HDPE, which notably improves HDPE materials' flexibility and environmental stress cracking resistance (Bourgeois and Blackett, 1990).

As more branching slightly increases the distance between parallel long-chain molecules, however, it increases HDPE material permeability and reduces its chemical resistance, but by amounts that are generally considered insignificant.

HDPE geomembranes are not made from HDPE only. They also contain additives, such as carbon black and antioxidants. The resulting material is called the HDPE compound and it contains approximately 97 percent HDPE, 2.5 percent carbon black, and 0.5 percent antioxidants. Note that HDPE geomembranes do not contain plasticizers.

Chemical reactions

HDPE is chemically resistant for two reasons. First, as all members of the polyethylene family, HDPE is essentially inert. Second, as discussed earlier, because of its high density, HDPE has a low permeability, therefore, it resists penetration by chemicals. Under certain conditions, however, HDPE can react with chemicals. A chemical reaction between a material and a chemical occurs when the chemical modifies the structure of the molecules making up the material.

Reaction of HDPE with chemicals is generally limited to oxidizing agents, such as nitric acid and oxygen. In other words, oxidation is the predominant mechanism of chemical reaction of HDPE. Oxidation is a step-wise process.

The polymer first absorbs energy, provided by heat, UV radiation and/or high-energy radiation (radioactivity). This absorption excites the polymer molecules, causing them to break, forming highly reactive fragments referred to as radicals. This mechanism is called chain scission. The radicals then react with oxygen, forming even more radicals.

As the process proceeds, an increasing number of radicals are formed. The process is terminated only when the radicals either react with antioxidants or recombine, or when energy is no longer supplied (Brydson, 1982; Rodriguez, 1970; and Seymour and Carraher, 1981). If oxidation occurs, it causes the molecular weight of molecules to decrease, making the HDPE material soften and embrittle, thereby becoming subject to stress crack-

g. Oxidation occurs only if two conditions are present.

The first condition is a high concentration of the oxidizing agent. The second condition is that the material must receive a sufficient supply of energy to activate the reaction.

When the conditions are not present—which is often the case—HDPE is not attacked. This is confirmed by reported cases of EPA 9090 tests conducted to evaluate the chemical compatibility between HDPE geomembranes and municipal waste or hazardous waste leachates from modern waste disposal facilities, which indicate no detectable deterioration of the properties of HDPE geomembranes (Ojeshina et al., 1984; and Dudzik and Tisinger, 1990).

Physical interaction

Another potential mechanism of HDPE degradation is physical interaction. Physical interaction of HDPE with a chemical occurs when HDPE, without undergoing change in the structure of its molecules, absorbs the chemical, usually organic. Organic chemicals can interact with HDPE, because like HDPE, they are nonpolar, and therefore, have similar intermolecular forces (cohesive forces) holding adjacent molecules together. The most typical mechanism of physical interaction involving HDPE is solvation.

Solvation Solvation is a physical process by which solvent molecules are absorbed into a material. Solvation causes a polymeric material to swell (which increases its permeability) and to soften, a process often referred to as plasticization. A limited degree of swelling and softening is, to some extent, reversible: The geomembrane more or less retrieves its original dimensions and properties if the solvent is removed by evaporation. The ultimate degree of solvation is dissolution, where the molecules of the initially solid material are dispersed in the solvent. Of course, this mechanism is not reversible.

Typical solvents that may cause solvation of HDPE are aromatic solvents, such as benzene, toluene, xylene and halogenated solvents, such as chloroform, methylene chloride and trichloroethylene. These solvents cause some degree of solvation of HDPE at ordinary temperature. Dissolution of HDPE by these solvents,

> A USEPA ad hoc committee has concluded that polymeric landfill lining materials should maintain their integrity in waste diposal environments in "terms of hundreds of years."

however, will not occur at ambient temperature.

In fact, no known solvents can dissolve HDPE at room temperature. Typical waste disposal facility temperatures should not exceed 50 C, which is significantly below 80 C, the temperature at which some solvents may begin to dissolve HDPE. These solvents should, therefore, not cause complete dissolution of HDPE geomembranes under waste disposal facility conditions.

Moreover, the solvents must be present at very high concentration to affect HDPE, a condition that is not observed in waste disposal facilities.

Extraction Extraction is a mechanism of physical interaction between polymeric compounds and chemicals. It is a process by which chemicals and heat cause additives, such as plasticizers and antioxidants, to leach out of the polymeric compounds.

HDPE compounds used to produce geomembranes do not contain plasticizers; however, their antioxidants can be extracted. Such an extraction typically requires a very high concentration of chemical, a condition typically not present in a waste disposal facility. Moreover, most modern antioxidants have a high molecular weight and are physically entangled among the polyethylene molecules. Such physical entanglement greatly reduces the ability of chemicals to extract antioxidants. As a result, HDPE geomembranes do not undergo significant loss of antioxidants by extraction.

Energy and environment

In all the potential mechanisms of degradation described above, energy plays a crucial role. In geomembrane applications, the most typical sources of energy are heat and ultraviolet (UV) radiation; both conditions often occur through direct exposure to sunlight. Also, exposure to high-energy radiation (radioactivity) can induce reaction of HDPE with oxidizing agents. High-energy radiation also may cause HDPE to crosslink, that is, to form chemical bonds between adjacent polyethylene molecules. As a result, HDPE may harden and become brittle. Again, for this to happen, HDPE would have to be exposed to large doses of high-energy radiation (Whyatt and Farnsworth, 1990).

In the absence of either oxygen or energy, oxidation, the predominant mechanism of chemical reaction of HDPE, cannot occur. Typical waste disposal facility environments are anaerobic, eliminating the possibility for oxidative degradation of HDPE geomembranes once they are buried (Haxo and Haxo, 1989).

In addition, the supply of energy is limited, because there is no light and because geomembranes are usually protected by a layer of soil, which insulates them from heat generated by decomposition of waste.

Some oxidation of HDPE geomembranes can occur as the result of their exposure to sun during installation. Such oxidation is limited and superficial, however, because carbon black, which is an additive used in most HDPE geomembranes, absorbs sunlight, preventing it from penetrating the geomembrane (Whitney, 1988).

Furthermore, the effects of oxidation should be limited, because HDPE geomembranes contain antioxidants, additives that stabilize radicals generated by HDPE's absorption of energy. Information on the durability of HDPE geomembranes that are permanently exposed can be obtained from experience gained in observing the performance of existing facilities.

If not attacked, could HDPE simply age?

Aging refers to changes that occur in materials when they are subjected to the type of temperate conditions in which a human could survive (but would age)—no contact with liquid chemicals, moderate ambient temperature, no exposure to UV radiation or radioactivity, no supply of oxygen beyond that naturally present in air, etc. Studies have indicated that the effect of such conditions on HDPE materials is very slow.

For example, test results obtained from polyethylene films stored in a ventilated box exposed to desert, temperate and tropical environments for 15 years, have shown negligible changes in crystallinity and minimal evidence of oxidation (Moskes, 1976).

Resistance to aging is best evaluated by observations of actual performance in service. Polyethylene has a long track record of successful uses. Polyethylene was first synthesized in 1933, and became commercially available in 1937.

The use of polyethylene for cable sheathing began in 1942 (Gilroy, 1985). Since then, polyethylene has been the material of choice for the protection of transatlantic cables.

The first HDPE geomembranes were used in 1973 in Europe (Knipschild, 1984) and in 1974 in the United States. To date, HDPE geomembranes have been used, exposed or buried, for 20 years. [illegible] they have been properly protected against mechanical failures (including stress cracking), HDPE geomembranes have performed satisfactorily. The performance of HDPE geomembranes for 20 years confirms the successful performance of HDPE in other outdoor applications, such as cable sheathing and buried pipes, for more than 40 years.

How long will geomembranes last?

A question frequently asked about geosynthetics and geomembranes in particular is, "How long will they last?" To answer this question, some clear conclusions can be drawn from the facts presented earlier.

Experience has shown that exposed HDPE materials, including geomembranes, can perform satisfactorily for decades if they are protected from mechanical aggressions.

In waste disposal facility environ[illegible] buried, only little energy should be acting on them, and in addition, the supply of oxygen should most likely be very low. In the absence of an aggressive environment, therefore, HDPE geomembranes should last for a very long time in waste disposal facilities.

A U. S. Environmental Protection Agency (USEPA) ad hoc committee on the durability of polymeric landfill lining materials has concluded that the polymeric landfill lining materials should maintain their integrity in waste disposal facility environments in "terms of hundreds of years" (Haxo and Haxo 1988). This conclusion is consistent with durability evaluations made using the Arrhenius model (Koerner et al. 1990). One can conclude, then, that in properly designed and constructed facilities, HDPE geomembranes should be able to protect ground water from leachate for hundreds of years, which is long after leachate generation has stopped.

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Acknowledgements

The authors acknowledge the technical assistance of J.F. Beech, F. Struve and C.E. Carraher in the preparation of this article.

APPENDIX I.2

GROUNDWATER SEEPAGE

Shaw®

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill No. 3, Inc.
Project: Clinton LF. No. 3 Chemical Waste Unit
Calculated By: PCT **Date:** 10/02/2007
Title: GROUNDWATER SEEPAGE **Checked By:** JPV **Date:** 10/08/2007

Problem Statement:

Calculate inward leakage through the composite liner (CSL) design based on Giroud et al. (1989).

Given:

1) Inward gradient with 27 feet of head on the liner during operational and post-closure periods (assumed maximum potentiometric elevation of 691.4 ft.MSL (measured in Well EX-4 in Nov. 2004) and top of leachate drainage layer at lowest elevation equal to 664 ft.MSL (assumes drainage layer completely saturated, see Drawing D6)
   $\rightarrow \therefore$ 691.4 ft. - 664 ft. = 27.4 ft.
2) Poor contact between the geomembrane and the compacted earth liner is considered.
3) The rate of leakage was calculated using the equation: where Q = (0.0008) * ($X^{0.9176}$); source for the equation is *Giroud and Bonaparte (1989): Leakage Through Liner Constructed With Geomembrane-Part II*, (VI, pp. 71-111)

Solution:

The rate of leakage through geomembrane defect (Q) expressed in m^3/s/ acre =

$$Q = (0.0008) \bullet (X^{0.9176})$$ (Source: IEPA based on Giroud and Bonaparte (1989))

Where,

X = Leachate depth on top of top of the geomembrane (m)

Q = Flow rate or leakage rate (m/yr) = (0.0008) • ($X^{0.9176}$)

Calculation:

X = Leachate depth on top of top of the geomembrane = 27.4 ft. = 8.3515 m

Q = Rate of leakage = (0.0008) • ($X^{0.9176}$) = 6.29E-3 m/yr
= 0.0206 ft/yr
= 0.2476 in/yr

APPENDIX I.3

LEACHATE HEAD DETERMINATION

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT **Date:** 10/9/07

Checked By: JPV **Date:** 10/9/07

TITLE: LEACHATE HEAD (LEACHATE COLLECTION DESIGN)

Problem Statement:

Determine the leachate head on the landfill liner system. The leachate collection system is designed to maintain a maximum one (1) foot of head of leachate on the liner.

Given:

1. Richardson, G., *Design of Waste Containment and Final Closure Systems.* ASCE Publication, April 2001. (Please see attached pages.)

2. Landfill cellular design presented in the design drawings.

Assumptions:

1. Giroud's Approximate Numerical Solution used to calculate leachate head on a liner.

$$t_{max} = (j)\left[\frac{\sqrt{\tan^2(\beta) + 4\frac{q_h}{k}} - \tan(\beta)}{2\cos(\beta)}\right](L)$$

Where:

t_{max} = leachate head on landfill liner (ft)

β = slope angle (degrees)

q_h = leachate generation rate (ft/yr)

k = hydraulic conductivity of drainage material (ft/yr)

L = maximum horizontal drainage distance (ft)

j = numerical modifying factor given as:

$$j = 1 - 0.12\exp\left\{-\left[\log\left(\frac{8\left(\frac{q_h}{k}\right)}{5\tan^2(\beta)}\right)^{5/8}\right]^2\right\}$$

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: PCT Date: 10/9/07
Checked By: JPV Date: 10/9/07

TITLE: LEACHATE HEAD (LEACHATE COLLECTION DESIGN)

2. The leachate collection system was analyzed as follows:
   – The maximum flow length (L) to a leachate collection pipe is 170 feet.

3. q_L = 1.872 ft/yr = Estimated maximum leachate generation rate due to percolation of moisture through the waste during the operational and closure periods of the proposed landfill. It was conservatively assumed equal to the Peak Daily Value from the HELP Model Intermediate Cover Scenario results — 0.06154 inches/day = 1,671 gallons/acre-day (refer to Appendix I.12).

4. q_s = 0.0206 ft/yr

5. $q_h = q_L + q_s$ = 1.8926 ft/yr

6. $$k = \frac{3.0 * 10^{-2}\text{ cm}}{\text{sec}} * \left(\frac{1\text{ in}}{2.54\text{ cm}}\right) * \left(\frac{1\text{ ft}}{12\text{ in}}\right) * \left(\frac{86{,}400\text{ sec}}{1\text{ day}}\right) * \left(\frac{365\text{ days}}{1\text{ year}}\right) = \frac{31{,}039.4\text{ ft}}{\text{year}}$$

7. Slope of liner = 2.66%, therefore, tan(β) = 0.0266

8. Chemical Waste Unit is at field capacity

9. The final cover is in place

Calculations:

$$j = 1\text{-}0.12\exp\left\{-\left[\left[\log\left(\frac{8\left(\frac{q_h}{k}\right)}{5\tan^2(\beta)}\right)^{5/8}\right]^2\right]\right\}$$

$$j = 1\text{-}0.12\exp\left\{-\left[\left[\log\left(\frac{8\left(\frac{1.8926}{31{,}039}\right)}{5(0.0266)^2}\right)^{5/8}\right]^2\right]\right\} = 0.9101$$

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Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT Date: 10/9/07

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TITLE: LEACHATE HEAD (LEACHATE COLLECTION DESIGN)

$$t_{max} = (j)\left[\frac{\sqrt{\tan^2(\beta) + 4\frac{q_h}{k}} - \tan(\beta)}{2\cos(\beta)}\right](L)$$

$$t_{max} = (0.9101)\left[\frac{\sqrt{\left((0.0266)^2 + 4\frac{(1.8926)}{(31{,}039)}\right)} - 0.0266}{2\cos\left(\tan^{-1}(0.0266)\right)}\right](170)$$

$$t_{max} = 0.329 \text{ feet}$$

Results:

The leachate drainage and collection system has been designed to maintain less than one (1) foot of head above the liner.

Maximum Flow Length "L" (ft)	Head "t_{max}" (ft)
170	0.329

APPENDIX I.4

CAPACITY OF
LEACHATE COLLECTION PIPE

Shaw®

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT **Date:** 10/4/07

Checked By: JPV **Date:** 10/9/07

TITLE: CAPACITY OF LEACHATE COLLECTION SYSTEM PIPING

Problem Statement:

Determine the following to verify that the proposed 6-inch diameter leachate collection system piping has sufficient capacity to accommodate the anticipated leachate flow volumes.

1. Maximum allowable flow through a 6-inch diameter leachate collection pipe.

2. Anticipated leachate flow volume through the leachate collection piping system based on the estimated maximum leachate generation rate due to percolation of moisture through waste (refer to Appendix I.12).

Given:

1. Calculation "Leachate Storage Tanks(s)" contained within this application (refer to Appendix I.6).

2. HELP Model results contained within this application (refer to Appendix I.12)

3. Calculation "Earthloads on the Leachate Collection System" contained in this application (refer to Appendix I.7).

4. KWH Sclairpipe® product information (refer to attached pages).

5. Landfill design located in the design drawings contained in this application.

Assumptions:

1. Formula used to calculate the maximum allowable flow for the design pipe:

$$Q_{max} = \frac{1.486}{n} AR^{(2/3)} S^{(1/2)} \quad \text{Manning's Equation}$$

$$A = \frac{\pi D^2}{4}$$

$$R = \frac{D}{4}$$

Shaw® Shaw Environmental, Inc.

Client:	Clinton Landfill, Inc		
Project:	Clinton Landfill No. 3 Chemical Waste Unit		
Proj. #:	128017		
Calculated By:	PCT	Date:	10/4/07
Checked By:	JPV	Date:	10/9/07

TITLE: CAPACITY OF LEACHATE COLLECTION SYSTEM PIPING

Where,

Q_{max} = Maximum allowable flow (ft^3/sec)
n = Manning's roughness coefficient
A = Pipe flow area (ft^2)
D = Inside pipe diameter (ft)
R = Hydraulic radius, for pipes flowing full
S = Channel slope (ft/ft)

2. Formula used to calculate anticipated leachate flow volumes:

$$Q = qA$$

Where,

Q = Leachate flow volume
q = Leachate generation rate
A = Surface area drained by pipe trench

3. n = 0.010 for HDPE pipe

4. $D_{(6\text{-inch SDR11})}$ = 5.348 inches = 0.446 ft (reference KWH Sclairpipe® / Earthloads calculation)

5. S_1 = 0.0055 ft/ft (conservatively assumed for settlement)

6. q_L = 1.872 ft/yr = Estimated maximum leachate generation rate due to percolation of moisture through the waste during the operational and closure periods of the proposed landfill. It was conservatively assumed equal to the Peak Daily Value from the HELP Model Intermediate Cover Scenario results — 0.06154 inches/day = 1,671 gallons/acre-day (refer to Appendix I.12). The area contributing to the flow in the leachate collection pipe is assumed to be the plan view area of the cell. The largest area will be Cell CWU1 (approximately 11.4 acres).

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT **Date:** 10/4/07

Checked By: JPV **Date:** 10/9/07

TITLE: CAPACITY OF LEACHATE COLLECTION SYSTEM PIPING

Calculations:

Maximum Allowable Flow for an SDR-11, 6-inch Pipe, Q_{max}

$$Q_{max} = \frac{1.486}{n} AR^{(2/3)} S_1^{(1/2)} = \frac{1.486}{n}\left(\frac{\pi D^2}{4}\right)\left(\frac{D}{4}\right)^{(2/3)} S_1^{(1/2)}$$

$$= \frac{1.486}{0.010}\left(\frac{\pi (0.446)^2}{4}\right)\left(\frac{0.446}{4}\right)^{(2/3)} (0.0055)^{(1/2)}$$

$$= 0.399 \frac{ft^3}{s}\left(\frac{7.48 gal}{ft^3}\right)\left(\frac{60 sec}{min}\right)$$

Q_{max} = 179.07 gpm (leachate collection pipe and leachate header pipe)

Leachate Flow Volume

Convert q to feet per minute:

$$q = 1.872 \frac{ft}{yr}\left(\frac{year}{52 weeks}\right)\left(\frac{week}{7 days}\right)\left(\frac{day}{1440 min}\right) = 35.71 \times 10^{-7} \frac{ft}{min}$$

Calculate the area of the largest cell:

$$A = 11.4 \text{ acres} = 496{,}584 \text{ ft}^2$$

Calculate leachate flow volumes for an SDR-11, 6-inch diameter pipe:

$$Q = qA$$

$$= \left(35.71 \times 10^{-7} \frac{ft}{min}\right)\left(496{,}584 \text{ ft}^2\right)\left(\frac{7.48 \text{ gal}}{ft^3}\right)$$

$$Q = 13.26 \text{ gpm}$$

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Client: Clinton Landfill, Inc

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT **Date:** 10/4/07

Checked By: JPV **Date:** 10/9/07

TITLE: CAPACITY OF LEACHATE COLLECTION SYSTEM PIPING

Results:

Based on the results summarized below, SDR-11, 6-inch diameter leachate collection system piping has sufficient capacity to accommodate the maximum anticipated leachate flow volumes.

$$Q < Q_{max} \text{ (gpm)}$$

$$13.26 \text{ gpm} < 179.07 \text{ gpm} \therefore \text{Ok}$$

APPENDIX I.5

LAMINAR FLOW IN LEACHATE COLLECTION PIPE

Shaw®

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill No. 3, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 122150
Calculated By: PCT **Date:** 10/3/07
Checked By: JPV **Date:** 10/4/07

TITLE: LAMINAR FLOW IN THE LEACHATE COLLECTION SYSTEM

Problem Statement:

Determine if the leachate drainage layer will maintain laminar flow in accordance with 35 Ill. Admin. Code Section 811.307 (d), by calculating the Reynold's number, R_e.

Given:

1. Freeze and Cherry, *Groundwater*, pp. 73, 96-97 (refer to attached pages).

2. Streeter and Wylie, *Fluid Mechanics,* 8th Ed., page 111 (refer to attached pages).

3. Landfill design specifications contained in this application.

Assumptions:

1. Formula used to calculate the Reynold's number, R_e.

$$R_e = \frac{\rho\, vD}{\mu}$$

$$v = -K\frac{dh}{dl}$$

Where,
ρ = fluid density (g/cm^3)
μ = absolute viscosity (g/cm-sec)
D = mean sand diameter (cm)
v = specific discharge (cm/s)
K = hydraulic conductivity (cm/s)
(dh/dl) = hydraulic gradient (or frictional resistance)

2. Flow through granular media is laminar if Reynold's number does not exceed "some value between 1 and 10." Therefore assume a conservative value of R_e = 1.0, as a division between laminar and turbulent flow (refer to Freeze and Cherry).

3. Temperature range = 40°F to 140°F

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill No. 3, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 122150
Calculated By: PCT Date: 10/3/07
Checked By: JPV Date: 10/4/07

TITLE: LAMINAR FLOW IN THE LEACHATE COLLECTION SYSTEM

4. ρ = ρ_w at a specific temperature
= 1.0000 g/cm^3 (40°F)
= 0.98320 g/cm^3 (140°F)

5. μ = μ_w at a specific temperature
= 0.015676 g/cm-sec (40°F)
= 0.004690 g/cm-sec (140°F)

6. D = 20 mm = 2 cm. "D" ranges from 0.075 to 20 mm for sand/gravel. Assume D = 20 mm to be conservative.

7. K = 3.0 x 10^{-2} cm/sec

8. (dh/dl) = 2.66 % (0.0266 cm/cm)

Calculations:

Calculate the Reynold's number at: T=40°F and T=140°F

At T = 40°F

$$R_e = \frac{\rho K\left(-\frac{dh}{dl}\right)D}{\mu} = \frac{(1.0000)(0.03)(0.0266)(2)}{0.015676} = 0.0510$$

$$= 0.0510 < 1 \therefore \text{ Laminar Flow}$$

At T = 140°F

$$R_e = \frac{\rho K\left(-\frac{dh}{dl}\right)D}{\mu} = \frac{(0.98320)(0.03)(0.0266)(2)}{0.004690} = 0.3346$$

$$= 0.3346 < 1 \therefore \text{ Laminar Flow}$$

Results:

Based on the calculated Reynolds numbers, the leachate drainage layer will maintain laminar flow in accordance with 35 Ill. Admin. Code Section 811.307 (d).

LAMINAR FLOW   Freeze & Cherry

permeability k. Bear (1972) summarizes the experimental evidence with the statement that "Darcy's law is valid as long as the Reynolds number based on average grain diameter does not exceed some value between 1 and 10" (p. 126). For this range of Reynolds numbers, all flow through granular media is laminar.

Flow rates that exceed the upper limit of Darcy's law are common in such important rock formations as karstic limestones and dolomites, and cavernous volcanics. Darcian flow rates are almost never exceeded in nonindurated rocks and granular materials. Fractured rocks (and we will use this term to refer to rocks rendered more permeable by joints, fissures, cracks, or partings of any genetic origin) constitute a special case that deserves separate attention.

Flow in Fractured Rocks

The analysis of flow in fractured rocks can be carried out either with the *continuum* approach that has been emphasized thus far in this text or with a *noncontinuum* approach based on the hydraulics of flow in individual fractures. As with granular porous media, the continuum approach involves the replacement of the fractured media by a representative continuum in which spatially defined values of hydraulic conductivity, porosity, and compressibility can be assigned. This approach is valid as long as the fracture spacing is sufficiently dense that the fractured media acts in a hydraulically similar fashion to granular porous media. The conceptualization is the same, although the representative elementary volume is considerably larger for fractured media than for granular media. If the fracture spacings are irregular in a given direction, the media will exhibit trending heterogeneity. If the fracture spacings are different in one direction than they are in another, the media will exhibit anisotropy. Snow (1968, 1969) has shown that many fracture-flow problems can be solved using standard porous-media techniques utilizing Darcy's law and an anisotropic conductivity tensor.

If the fracture density is extremely low, it may be necessary to analyze flow in individual fissures. This approach has been used in geotechnical applications where rock-mechanics analyses indicate that slopes or openings in rock may fail on the basis of fluid pressures that build up on individual critical fractures. The methods of analysis are based on the usual fluid mechanics principles embodied in the Navier-Stokes equations. These methods will not be discussed here. Wittke (1973) provides an introductory review.

Even if we limit ourselves to the continuum approach there are two further problems that must be addressed in the analysis of flow through fractured rock. The first is the question of non-Darcy flow in rock fractures of wide aperture. Sharp and Maini (1972) present laboratory data that support a nonlinear flow law for fractured rock. Wittke (1973) suggests that separate flow laws be specified for the linear-laminar range (Darcy range), a nonlinear laminar range, and a turbulent range. Figure 2.28 puts these concepts into the context of a schematic curve of specific discharge vs. hydraulic gradient. In wide rock fractures, the specific discharges and Reynolds numbers are high, the hydraulic gradients are usually less

Fundamental considerations of the nature of flow in porous media have led investigators to conclude that Darcy's law of proportionality of macroscopic velocity and hydraulic gradient is an accurate representation of the "law of flow" as long as velocities are low. Although it is generally concluded that the range of validity cannot be definitely established, Darcy's law is considered widely to be infinitely superior to methods which, though adhering strictly to basic laws, become so complex as to be beyond practical application.

Muskat (1937a), Taylor (1948), Leonards (1962), and others have presented excellent discussions of permeability and Darcy's law. Taylor (1948a) points out that in soils there is a slow transition from purely laminar flow to a slightly turbulent state and concludes that under a hydraulic gradient of 100%, uniform soils with a grain size of 0.5 mm or less always have laminar flow. For a gradient of 800% the diameter is 0.25 mm. This admittedly is a conservative approximation, based on a Reynold's number of 1.0.

Jacob (1950) concludes from experiments with natural and artificial sands of nearly uniform spherical grains that the transition from laminar to turbulent flow in sands requires at least a thousandfold increase in velocity to reach the limit of fully established turbulence. He states that, ". . . a tenfold increase above the approximate critical velocity results in about 50 percent error in the hydraulic gradient as predicted by Darcy's law."

Fishel (1935) reports experiments with very low heads indicate that, ". . . for the material tested (Fort Caswell sand) the rate of flow varies directly as the hydraulic gradient, down to a gradient of 2 or 3 inches to the mile and there are indications that Darcy's law holds for indefinitely low gradients."

In the analysis of seepage in coarse sands and gravels Darcy's law is not strictly applicable. Forchheimer (1902) found the frictional resistance of pervious gravel to be

$$\frac{\Delta h}{\Delta l} = \frac{1.77}{10^3} V + \frac{3.18}{10^4} V^2 \qquad (3.4)$$

In Eq. 3.4, V is the velocity in meters per day.

The general form of Eq. 3.4 is

$$\frac{\Delta h}{\Delta l} = aV + bV^2 \qquad (3.5)$$

According to Eqs. 3.4 and 3.5, head losses in gravels are greater than indicated by Darcy's law. If permeability tests can be made under conditions similar to those that will exist in a prototype, the errors will tend to be neutralized; however, this is not always possible. It is, therefore, desirable to allow liberal factors of safety in the design of drainage systems containing coarse, clean aggregates where semi-turbulent or turbulent flow may develop.

Applications of Darcy's Law

Applications of Darcy's law to permeability determinations are described in Chapter 2.

The validity of Darcy's law is an essential assumption in the following soil mechanics theories and methods.

1. The theory of consolidation of clays.
2. Quantitative theory of laminar flow of homogeneous fluids through porous media.
3. Practical solutions to Laplace equations by flow nets.

The validity of Darcy's law is an essential assumption for all seepage solutions presented in this text, including:

1. Flow nets for steady seepage through earth cross sections of one or more different permeabilities, for both isotropic and anisotropic conditions (Darcy's law enters into the derivation of the basic differential equation).
2. Calculations involving the *velocities* of masses of water in porous media under steady seepage conditions. (These computations involve the *seepage velocity* $v_s = ki/n_e$.) The *seepage velocity* of moving groundwater can be used as an index of permeability (Sec. 2.7); its magnitude in any water-bearing material or drainage layer is a useful criterion of the rate of movement of water.
3. Approximate nonsteady seepage applications of the flow net to moving saturation lines. (These computations involve the additional use of the *seepage velocity* $v_s = ki/n_e$, which depends on Darcy's law.)
4. Calculations for seepage quantities through saturated soil and rock formations and other porous media. (These determinations involve the *discharge velocity* $v_d = ki$, determined from Darcy's law.)
5. Determination of the discharge capacities of porous aggregate drains, chimneys, sand-filled wells, etc. (These determinations also make use of the *discharge velocity*, defined in 4.)

The relationships represented by Darcy's law, though very simple, represent some of the most powerful tools available to the soils engineer and the drainage engineer. Unfortunately their great benefits

The thickness of the unsaturated zone varies depending on the amount of water in storage. After significant rainfall and infiltration, the top of the saturated level will be high. During dry periods, as stored groundwater drains to streams or other water bodies, the saturated level will generally fall.

Below the groundwater table, water generally moves very slowly. As shown in Fig. 3-2, water can flow either into or out of streams, springs, or lakes depending on the gradient of the groundwater surface. Springs occur wherever the groundwater table intersects the earth's surface. Springs can be intermittent if the water table rises and falls above and below the spring's elevation, since the amount of rainfall and infiltration vary throughout the year. In some locations, streams gain water from groundwater inflow during the rainy season and lose water to the groundwater zone during the dry season.

Water within the saturated zone fills the natural voids that occur within the solid material, which may be soil, gravel, or rock depending on the local geology. If voids make up a large percent of the bulk volume of the solid material, a greater volume of water can be contained. Thus, an aquifer of saturated gravel contains a much greater volume of water in a unit volume of the solid material than does an equal volume of hard rock. All natural materials are porous to some degree. Table 3-1 lists typical values of *porosity* (volume of voids divided by total volume) for several natural materials. The equation used to calculate porosity (n) is

$$n = \frac{V_v}{V_t} \qquad (3\text{-}1)$$

where V_v is the volume of the voids, and V_t is the total volume.

Using the definition of porosity, the volume of water V_w contained in a saturated total volume of material V_t is

$$V_w = nV_t \qquad (3\text{-}2)$$

Table 3-1 Typical Values of Mechanical Properties for Various Natural Materials (6)

Material	Porosity	Specific Yield	Specific Retention	Hydraulic Conductivity (m/day)
Soil	0.55	0.40	0.15	10^{-3}–5
Clay	0.50	0.02	0.48	10^{-7}–10^{-4}
Sand	0.25	0.22	0.03	0.06–120
Gravel	0.20	0.19	0.01	100–7000
Limestone	0.20	0.18	0.02	10^{-4}–5000
Sandstone	0.11	0.06	0.05	10^{-5}–0.5
Basalt	0.11	0.08	0.03	10^{-8}–1000
Granite	0.001	0.0009	0.0001	10^{-8}–5

FLUID MECHANICS

Eighth Edition

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APPENDIX

C

Physical Properties of Fluids

Table C.1 Physical properties of water in SI units

Temp, °C	Specific weight γ, N/m^3	Density ρ, kg/m^3	Viscosity $\mu \times 10^3$, $N \cdot s/m^2$	Kinematic viscosity $\nu \times 10^6$, m^2/s	Surface tension $\sigma \times 10^2$, N/m	Vapor-pressure head p_v/γ,† m	Bulk modulus of elasticity $K \times 10^{-7}$, N/m^2
0	9806	999.9	1.792	1.792	7.62	0.06	204
5	9807	1000.0	1.519	1.519	7.54	0.09	206
10	9804	999.7	1.308	1.308	7.48	0.12	211
15	9798	999.1	1.140	1.141	7.41	0.17	214
20	9789	998.2	1.005	1.007	7.36	0.25	220
25	9778	997.1	0.894	0.897	7.26	0.33	222
30	9764	995.7	0.801	0.804	7.18	0.44	223
35	9749	994.1	0.723	0.727	7.10	0.58	224
40	9730	992.2	0.656	0.661	7.01	0.76	227
45	9711	990.2	0.599	0.605	6.92	0.98	229
50	9690	988.1	0.549	0.556	6.82	1.26	230
55	9666	985.7	0.506	0.513	6.74	1.61	231
60	9642	983.2	0.469	0.477	6.68	2.03	228
65	9616	980.6	0.436	0.444	6.58	2.56	226
70	9589	977.8	0.406	0.415	6.50	3.20	225
75	9560	974.9	0.380	0.390	6.40	3.96	223
80	9530	971.8	0.357	0.367	6.30	4.86	221
85	9499	968.6	0.336	0.347	6.20	5.93	217
90	9466	965.3	0.317	0.328	6.12	7.18	216
95	9433	961.9	0.299	0.311	6.02	8.62	211
100	9399	958.4	0.284	0.296	5.94	10.33	207

† γ = 9806 N/m^3.

Note: 1N sec/m² = 1kg/m.sec

APPENDIX I.6

LEACHATE STORAGE TANK(S)

Shaw® Shaw Environmental, Inc.

Client:	Clinton Landfill, Inc.
Project	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	122150
Calculated By: PCT	Date: 10/4/07
Checked By: JPV	Date: 10/9/07

TITLE: LEACHATE STORAGE TANK SIZING

Problem Statement

Determine the required storage tank capacity on site to safely store a minimum of 1-days worth of accumulated leachate based on the estimated maximum leachate that would be generated from the proposed Chemical Waste Unit.

Given

1. Product information for the existing leachate storage tanks (refer to attached pages).
2. HELP Model results contained in this application.
3. Landfill cellular design contained in the design drawings.

Assumptions

1. $q_{L\,(open)}$ = 1,671 gallons/acre-day = estimated maximum leachate generation rate due to percolation of moisture through the waste during the operational periods. It was conservatively assumed equal to the Peak Daily Value from the HELP Model Intermediate Cover Scenario results — 0.06154 inches/day = 1,671 gallons/acre-day (refer to Appendix I.12).

2. $q_{L(closed)}$ = 1,422 gallons/acre-day = estimated maximum leachate generation rate due to percolation of moisture through the waste during post closure period. It was conservatively assumed equal to the Peak Daily Value from the HELP Model Post-Closure Years 1 - 30 Scenario results — 0.05236 inches/day = 1,422 gallons/acre-day (refer to Appendix I.12).

3. A_{open} = 7.5 acres = maximum area of the proposed Chemical Waste Unit footprint that will be open during operations:

$$\left(\frac{1}{3} \times 22.5\ \text{acres}\right) = 7.5\ \text{acres}$$

4. A_{closed} = 15 acres = area of the Chemical Waste Unit footprint that will closed with final cover installed:

$$\left(22.5\ \text{acres} - 7.5\ \text{acres}\right) = 15\ \text{acres}$$

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.

Project Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 122150

Calculated By: PCT Date: 10/4/07

Checked By: JPV Date: 10/9/07

TITLE: LEACHATE STORAGE TANK SIZING

Calculations:

Calculate storage volume necessary for 1 day's storage.

Results:

$$V_{\text{1-day total}} = \left[\left(\frac{1{,}671\,\text{gal}}{\text{ac} \times \text{day}} \times 7.5\,\text{ac}\right) + \left(\frac{1{,}422\,\text{gal}}{\text{ac} \times \text{day}} \times 15\,\text{ac}\right)\right]$$
$$= 33{,}863\ \text{gallons}$$

One initial tank of at least 35,000 gallons will be installed for the initial development of the Chemical Waste Unit. This tank will be similar to the 35,000 gallon leachate storage tank that exists on the northern portion of the Clinton Landfill No. 3 site. Additional tanks will be added depending on 1) the number of treatment systems which are permitted, 2) the area permitted for waste disposal, and 3) the actual leachate generation amounts.

APPENDIX I.7

EARTHLOADS ON LEACHATE COLLECTION SYSTEM

Shaw®

Client:	Clinton Landfill No. 3, Inc
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
Calculated By:	PCT Date: 10/3/07
Checked By:	JPV Date: 10/3/07

Shaw® Shaw Environmental, Inc.

TITLE: EARTHLOADS ON THE LEACHATE COLLECTION SYSTEM

Problem Statement:

Determine the maximum earthload (W) on the leachate collection system, considering two scenarios:

1. W_{FL} = Loading on pipe due to landfill at final grade.
2. W_{IL} = Loading on pipe due to initial cell lift of 5 feet of chemical waste, 1 foot leachate drainage layer, and compactor concentrated load.

Given:

1. *Gravity Sanitary Sewer Design and Construction*, ASCE Manuals and Reports on Engineering Practice - No. 60, pp. 166-191 (refer to attached pages).
2. *Caterpillar Performance Handbook*, Edition 37 (refer to attached pages).
3. KWH Sclairpipe® product information (refer to attached pages).
4. Maximum waste thickness (MSW and chemical waste) overlying leachate collection piping taken from geotechnical analyses contained in this application (Appendix H).
5. Landfill design specifications contained in this application.
6. Laboratory test data contained in this application (Appendix H).

Assumptions:

Final Landform

1. Marston's formula utilized to calculate the prism load (reference ASCE No. 60):

$$W_c = C_c \omega B_c^2$$

Where,

W_c= Load on pipe (psf)
C_c = Load coefficient, obtained from figure 9-7 of ASCE No. 60
w = Unit weight of overlying fill (pcf)
B_c = Outer diameter of pipe (ft)
H = Height of fill above the top of the pipe (ft)

Client:	Clinton Landfill No. 3, Inc
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017

Shaw® Shaw Environmental, Inc.

Calculated By:	PCT	Date:	10/3/07
Checked By:	JPV	Date:	10/3/07

TITLE: EARTHLOADS ON THE LEACHATE COLLECTION SYSTEM

Assumed embankment conditions over a positive projecting pipe since the pipe is located in a wide trench and the top of the pipe is near the surface of compacted soil. Therefore, Marston's formula equals:

$$W_c = HwB_c$$

2. B_c = 6.63 in = 0.55 ft for a 6-inch SDR 11 pipe (reference KWH Sclairpipe®)

3. Moist unit weight of final cover soil is based on the average dry density and water content values determined from Standard Proctor data (refer to Appendix H).

$$\gamma_m = \gamma_d(1+w)$$

4. The chemical waste will have a thickness of 108.2 feet (refer to Appendix H.3).

5. The MSW waste will have a thickness of 59.5 feet (refer to Appendix H.3).

6. Unit weight of the chemical waste is conservatively assumed to be 90 pcf

7. Unit weight of the MSW is conservatively assumed to be 75 pcf.

Concentrated Load

1. D.L. Holl's integration of Boussinesq's formula utilized to calculate the load on the pipe due to a superimposed concentrated load (corresponding to a landfill compactor, reference ASCE No. 60):

$$W_{sc} = C_s \frac{PF}{L}$$

Where,

W_{sc} = Load on pipe (lb/ft)
P = Concentrated load (lb)
F = Impact Factor
C_s = Load Coefficient, a function of $B_c/2H$
H = Height of fill above top of pipe (ft)
B_c = Outer diameter of pipe (ft)
L = Effective length of pipe (ft)

Client:	Clinton Landfill No. 3, Inc
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
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Shaw® Shaw Environmental, Inc.

TITLE: EARTHLOADS ON THE LEACHATE COLLECTION SYSTEM

2. P = Total weight of compactor divided by 2 axles = 118,348 lb/2 = 59,174 lb (reference Caterpillar (CAT 836H))

3. F = 1.0 (recommend per ASCE No. 60 for H > 3 ft)

4. H = 1 ft of drainage layer material + 5 ft of waste (initial lift) = 6 ft

5. B_c = 6.63 in = 0.55 ft (reference)

6. L = 3 ft (recommended per ASCE No. 60 for pipe lengths > 3 ft)

7. C_s = 0.037 (reference ASCE No. 60, Table 9-4, based on the following ratios)

$$\frac{B_c}{2H} = \frac{0.55}{2(6)} = 0.046$$

$$\frac{L}{2H} = \frac{3}{2(6)} = 0.25$$

9. Marston's equation used to calculate load due to initial 5-foot lift of chemical waste.

Calculations:

Loading on Pipe due to Landfill at Final Grade (W_{FL})

MAXIMUM LOAD ON LEACHATE COLLECTION PIPE - FINAL GRADE			
Layer	**Thickness, t (ft)**	**Density, γ (pcf)**	**t x γ (psf)**
Final Cover	4	128	512
MSW Waste	59.5	75	4,462.5
Chemical Waste	108.2	90	9,738
Granular Drainage Material	1	130	130
Total Thickness =	**172.7 feet**	**∑ (t x γ) =**	**14,842.5 psf**
		∑(t x γ) / total thickness = Average Density (w) =	**85.94 pcf**

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Client:	Clinton Landfill No. 3, Inc		
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Checked By:	JPV	Date:	10/3/07

TITLE: EARTHLOADS ON THE LEACHATE COLLECTION SYSTEM

For a 6-inch SDR 11 Pipe:

$W_{FL} = H*w*B_c = (172.7 \text{ ft}) (85.94 \text{ pcf}) (0.55 \text{ ft}) = 8{,}163 \text{ lb/ft} = 680.3 \text{ lb/in}$

Loading on Pipe due to Initial Lift of Waste and Concentrated Compactor Load (W_{IL})

AVERAGE LOAD ON LEACHATE COLLECTION PIPE - INITIAL LIFT OF WASTE			
Layer	**Thickness, t (ft)**	**Density, γ (pcf)**	**t x γ (psf)**
Chemical Waste	5	90	450
Granular Drainage Material	1	130	130
Total Thickness =	**6 feet**	**∑(t x γ) =**	**580 psf**
∑(t x γ) / total thickness = Average Density (w) =			**96.7 pcf**

For a 6-inch SDR 11 Pipe:

$$W_c = H * w * B_c = (6)(96.7)(0.55) = 319.1 \text{ lb/ft} = 26.6 \text{ lb/in(initial lift of waste)}$$

$$W_{sc} = C_s \frac{PF}{L} = (0.037)\frac{(59{,}174 \text{ lb})(1.0)}{3\text{ft}} = 729.8 \text{ lb/ft} = 60.8 \text{ lb/in (compactor load)}$$

$$W_{IL} = W_c + W_{sc} = 26.6 + 60.8 = 87.4 \text{ lb/in}$$

Results:

The maximum load on the leachate collection pipe (W) was determined to be 680.3 lb/in for a 6-inch SDR 11 pipe. The maximum load corresponds to the load imposed by the weight of the landfill after final cover has been placed, which was determined to be greater than the load imposed during compaction of the initial lift of waste. The results are summarized below.

For 6-inch SDR 11 Pipe:

$$W = W_{FL} = 680.3 \text{ lb/in} > W_{IL} = 87.4 \text{ lb/in}$$

ASCE—MANUALS AND REPORTS ON ENGINEERING PRACTICE—NO. 60

WPCF—MANUAL OF PRACTICE—NO. FD-5

Gravity Sanitary Sewer Design and Construction

AMERICAN SOCIETY of CIVIL ENGINEERS

WATER POLLUTION CONTROL FEDERATION

CHAPTER 9

STRUCTURAL REQUIREMENTS

A. INTRODUCTION

The structural design of a sanitary sewer requires that the supporting strength of the installed sewer pipe, divided by a suitable factor of safety, must equal or exceed the loads imposed on it by the combined weight of soil and any superimposed loads.

This chapter presents generally accepted criteria and methods for determining combined loads and supporting strength of the sewer pipe, as well as procedures for combining these elements with the application of a factor of safety to produce a safe and economical design.

Methods are presented for estimating probable maximum loads caused by soil forces and for both static and moving superimposed loads. Where so noted, the methods apply to rigid and flexible conduits in the three most common conditions of installation: In a trench in natural ground; in an embankment; and in a tunnel.

The design of rigid and flexible pipes is treated separately. There are no specific design procedures given for flexible pipes of intermediate stiffness. For such cases, design procedures such as computer analysis based on soil-structure interaction or the designs for rigid or flexible pipes may be used (not interchangeably) for conservative results.

The supporting strength of a buried sewer pipe is a function of installation conditions as well as the strength of the sewer pipe itself. Structural analysis and design of the sewer line are problems of soil-structure interaction. This chapter presents procedures for determining the field or installed supporting strength of rigid sewer pipe based on its established relationship to the laboratory test strength. It also presents methods of predicting approximate field deflections for flexible pipe, based on empirical methods.

Since installation conditions have such an important effect on both load and supporting strength, a satisfactory sewer construction project requires accurate assumed design conditions from the job site. Therefore, this chapter also includes a section on recommendations for construction and field observations to achieve this goal.

This chapter does not include information on reinforced concrete design of rigid sewer pipe sections. Reference should be made to standard textbooks and to ACI/ASTM Specifications or industry handbooks for such design data.

B. LOADS ON SEWERS CAUSED BY GRAVITY EARTH FORCES

1. General Method — Marston Theory

Marston developed methods for determining the vertical load on buried conduits caused by soil forces in all of the most commonly encountered construction conditions (1,2). His methods are based on both theory and experiment and have achieved acceptance as being useful and reliable.

More recent analysis and actual observation of field performance have shown that designs based on the Marston Theory yield satisfactory results especially for small-diameter conduits in narrow trenches. For la[illegible] [illegible]meter conduits the results are conservative.

In general, the theory states that the load on a buried pipe is eq[illegible] to the weight of the prism of soil directly over it, called the interior prism, plus or minus the frictional shearing forces transferred to that prism by the adjacent prisms of soil — the magnitude and direction of these frictional forces being a function of the relative settlement between the interior and adjacent soil prisms. The theory makes the following assumptions:

(a) The calculated load is the load which will develop when ultimate settlement has taken place.

(b) The magnitude of the lateral pressures which induce the shearing forces between the interior and adjacent soil prisms is computed in accordance with Rankine's theory.

(c) Cohesion is negligible except for tunnel conditions.

The general form of Marston's equation is:

$$W = CwB^2 \tag{9.1}$$

in which W is the vertical load per unit length acting on the sewer pipe because of gravity soil loads; w is the unit weight of soil; B is the trench width or sewer pipe width, depending on installation conditions; and C is a dimensionless coefficient that measures the effect of the following variables:

- The ratio of the height of fill to width of trench or sewer pipe;
- the shearing forces between interior and adjacent soil prisms;
- the direction and amount of relative settlement between interior and adjacent soil prisms for embankment conditions.

2. Types of Loading Conditions

Although the general form of Marston's equation includes all the factors necessary to analyze all types of installation conditions, it is convenient to classify these conditions, write a specialized form of the equation, and prepare separate graphs and tables of coefficients for each.

The accepted system of classification is shown diagrammatically in Fig. 9-1 and is described briefly below.

Trench conditions are defined as those in which the sewer pipe is installed in a relatively narrow trench cut in undisturbed ground and covered with soil backfill to the original ground surface.

Embankment conditions are defined as those in which the sewer pipe is covered above the original ground surface or when a trench in undisturbed soil is so wide that trench wall friction does not affect the load on the sewer pipe. The embankment classification is further subdivided into two major subclassifications — positive projecting and negative projecting. Sewer pipe is defined as positive projecting when the top of the sewer pipe is above the adjacent original ground surface. Negative projecting sewer pipe is that installed with the top of the sewer pipe below the adjacent original ground surface in a trench which is narrow with respect to the size of pipe and the depth of cover (Fig. 9-1), and when the native material is of sufficient strength that the trench shape can be maintained dependably during placement of the embankment.

A special case, called the induced trench condition, may be employed to minimize the load on a conduit under an embankment of unusual height.

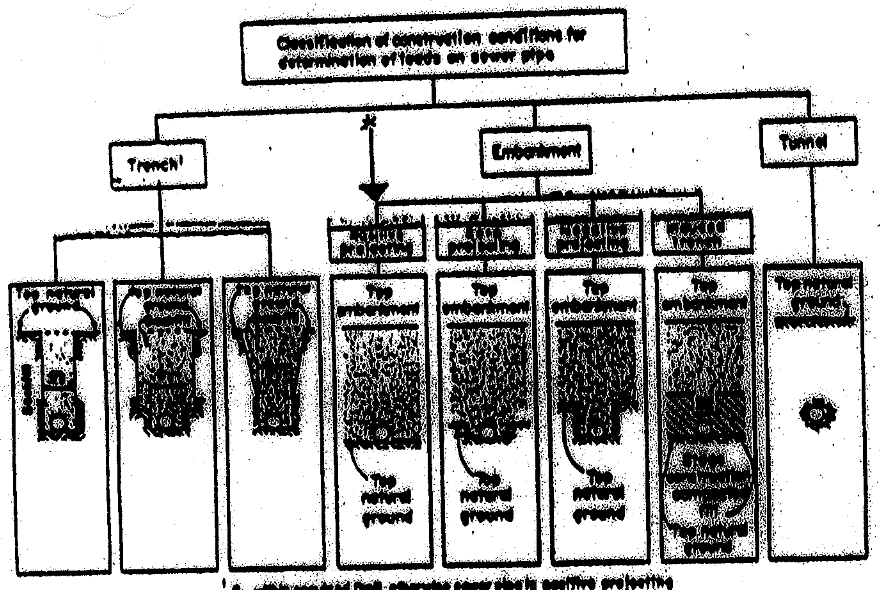


Fig. 9-1. Classification of construction conditions.

3. Loads for Trench Conditions

Sewers usually are constructed in ditches or trenches which are excavated in natural or undisturbed soil, and then covered by refilling the trench to the original ground line. This construction procedure often is referred to as "cut and cover," "cut and fill," or "open cut."

The vertical soil load to which a sewer pipe in a trench is subjected is the resultant of two major forces. The first is produced by the mass of the prism of soil within the trench and above the top of the sewer pipe. The second is the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench.

The backfill soil has a tendency to settle in relation to the undisturbed soil in which the trench is excavated. This downward movement or tendency for movement induces upward shearing forces which support a part of the weight of the backfill. Thus, the resultant load on the horizontal plane at the top of the sewer pipe within the trench is equal to the weight of the backfill minus these upward shearing forces (Fig. 9-2).

Unusual conditions may be encountered in which poor natural soils may effect a change from trench to embankment conditions with considerably increased load on the sewer pipe. This is covered in the next subsection.

a. Use of Marston's Formula

Marston's formula for loads on rigid sewer pipe in trench conditions is:

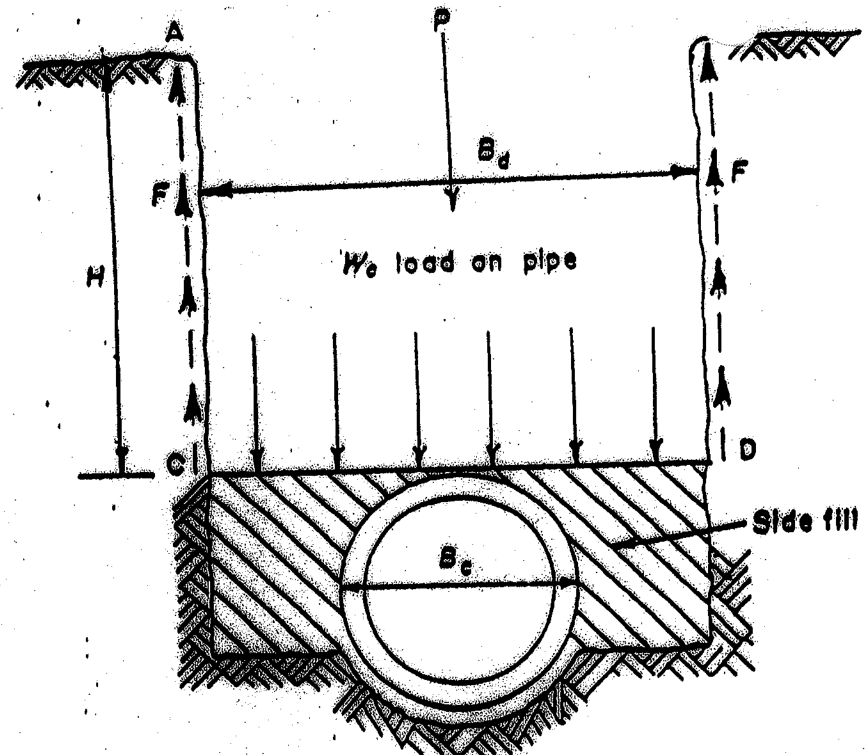


Fig. 9-2. Load-producing forces: P = weight of backfill ABCD; F = upward shearing forces on AC and BD; and, $W_e = P - 2F$.

In which W_e is the load on the sewer pipe, in newtons per meter (pounds per foot); w is the density of backfill soil, in kilograms per cubic meter (pounds per cubic foot); B_d is the width of trench at the top of the sewer pipe, in meters (feet); C_d is a dimensionless load coefficient which is a function of the ratio of height of fill to width of trench and of the friction coefficient between the backfill and the sides of the trench. The load coefficient, C_d, is computed as follows:

$$C_d = \frac{1 - e^{-2\kappa\mu' \frac{H}{B_d}}}{2\kappa\mu'} \tag{9.3}$$

In which e is the base of natural logarithms and κ is Rankine's ratio of lateral pressure to vertical pressure:

$$\kappa = \frac{\sqrt{\mu^2 + 1} - \mu}{\quad} = \frac{1 - \sin\phi}{\quad} \tag{9.4}$$

Sanit... ...wers which are to be constructed in sloping-sided trenches with the slopes extending to the invert, or to any plane above the invert but below the top of the sewer, should be designed for loads computed by using the actual width of the trench at the top of the sewer pipe, or by the projecting-sewer formula (covered in the next subsection), whichever gives the least load on the sewer pipe.

If for any reason the trench becomes wider than that specified and for which the sewer pipe was designed, the load on the sewer pipe should be checked and a stronger sewer pipe or higher class of bedding used, if necessary.

b. Soil Characteristics-Trench Conditions

The load on a sewer pipe is influenced directly by the density of the soil backfill. This value varies widely for different soils, from a minimum of about 1,600 kg/m³ (100 lb/cu ft) to a maximum of about 2,200 kg/m³ (135 lb/cu ft). The average maximum unit weight of the soil which will constitute the backfill over the sewer pipe may be determined by density measurements in advance of the structural design of the sewer pipe. A design value of not less than 1,900 or 2,000 kg/m³ (120 or 125 lb/cu ft) is recommended if such measurements are not made.

The load also is influenced by the coefficient of friction between the backfill and the sides of the trench and by the coefficient of internal friction of the backfill soil. Ordinarily these two values will be nearly the same and may be so considered for design purposes, as in Fig. 9-3. However, in special cases this may not be true. For example, if the backfill is sharp sand and the sides of the trench are sheeted with finished lumber, μ may be substantially greater than μ'. Unless specific information to the contrary is available, values of the products $k\mu$ and $k\mu'$ may be assumed to be the same and equal to 0.130. If the backfill soil is a "slippery" clay and there is a possibility that it will become very wet shortly after being placed, $k\mu$ and $k\mu'$ equal to 0.110 (maximum for saturated clay, Fig. 9-3) should be used.

Sample Calculations:

Example 9-1. Determine the load on a 24-in. diameter rigid sewer pipe under 14 ft of cover in trench conditions.

Assume that the sewer pipe wall thickness is 2 in.; B_c = 24 + 4 = 28 in. = 2.33 ft; B_d = 2.33 + 2.00 = 4.33 ft; and w = 120 lb/cu ft for saturated top soil backfill. Then H/B_d = 14/4.33 = 3.24; C_d (from Fig. 9-3) = 2.1; and W_c = 2.1 × 120 × (4.33)² = 4,720 lb/ft (68,880 N/m).

Example 9-2. Determine the load on the same size sewer laid on a concrete cradle and with trench sheeting to be removed.

Assume that the wall thickness is 2 in.; the cradle projection outside of the sewer pipe is 8 in. (4 in. on each side); and the maximum clearance between cradle and outside of sheeting is 14 in. Then B_d = 24 + (2 × 2 in.) + 8 + (2 × 14) = 64 in. = 5.33 ft.

As this seems to be an extremely wide trench, a check should be made on the transition width of the trench: B_c = 2.33 ft; H = 14 ft; $r_{sd}p$ = 0.5; and H/B_c = 14/2.33 = 6.0.

From Fig. 9-4, B_d/B_c = 2.39 (the ratio of the width of the trench to the width of the sewer at which loads are equal by both ditch-sewer theory and projecting-sewer theory); B_d = 2.33 × 2.39 = 5.57 > 5.33; H/B_d = 14/5.33 = 2.63; C_d (from Fig. 9-3) = 1.85; and W_c = 1.85 × 120 × (5.33)² = 6,300 lb/ft (91,700 N/m).

left in place.

B_d becomes 4 in. less = 5.00 ft; H/B_d = 14/5.00 = 2.8; C_d (from F... 9-3) = 1.92; and W_c = 1.92 × 120 × (5.00)² = 5,750 lb/ft (84,040 N/m).

Example 9-4. Determine the load on a 30-in. diameter flexible sewer pipe installed in a trench 4 ft 6 in. wide at a depth of 12 ft.

Assume the soil is clay weighing 120 lb/cu ft. and that it will be well compacted at the sides of the sewer pipe. Then H = 12 ft; B_d = 4.5 ft; B_c = 2.5 ft; H/B_d = 2.67; C_d = 1.9; and W_c = 1.9 × 120 × 4.5 × 2.5 = 2,565 lb/ft (37,450 N/m).

For conservative design, the prism load should be determined. The prism load on flexible sewer pipe will be W = 2.5 × 12 × 120 = 3,600 lb/ft (52,460 N/m).

4. Loads for Embankment Conditions

A sewer pipe is described as a projecting sewer pipe when it is installed in a wide trench or in such a manner that the top of the sewer pipe is at or above the natural ground surface or the surface of thoroughly compacted soil and subsequently is covered with an embankment. If the top of the sewer pipe projects above the natural ground surface or if it is installed in a wide trench, it is a positive projecting sewer pipe. There are, however, other methods of installing sewer pipe under embankments which have the favorable effect of minimizing the load on the sewer pipe. In these cases, the installation is classified as a negative projecting sewer pipe or an induced trench sewer pipe (Fig. 9-1).

These variations of embankment conditions will be treated separately for convenience in computation.

a. Positive Projecting Sewer Pipe

The load on a positive projecting sewer pipe is equal to the weight of the prism of soil directly above the structure, plus (or minus) vertical shearing forces which act on vertical planes extending upward into the embankment from the sides of the sewer pipe. For an embankment installation of sufficient height, these vertical shearing forces may not extend to the top of the embankment, but terminate in a horizontal plane at some elevation above the top of the sewer pipe known as the "plane of equal settlement" as shown in Fig. 9-6. The shear increment acts downward when $(s_m + s_g) > (s_f + d_c)$ and vice versa. In this expression s_m is the compression of the columns of soil of height pB_c; s_g is the settlement of the natural ground adjacent to the sewer pipe; s_f is the settlement of the bottom of the sewer pipe; and d_c is the deflection of the sewer pipe.

The location of the plane of equal settlement is determined by equating the total strain in the soil above the pipe to that in the side fill plus the settlement of the critical plane. When the plane of equal settlement is an imaginary plane above the top of the embankment, i.e., shear forces extend to the top of the embankment, the installation is called either "complete trench condition" or "complete projection condition," depending on the direction of the shear forces. When the plane of equal settlement is located within the embankment (Fig. 9-6), the installation is called "incomplete trench condition," or "incomplete projection condition" (Fig. 9-7).

In computing the settlement values, the effect of differential settlement caused by any compressible layers below the natural ground surface also must be considered. An exceptional situation for a sewer pipe in a trench can be

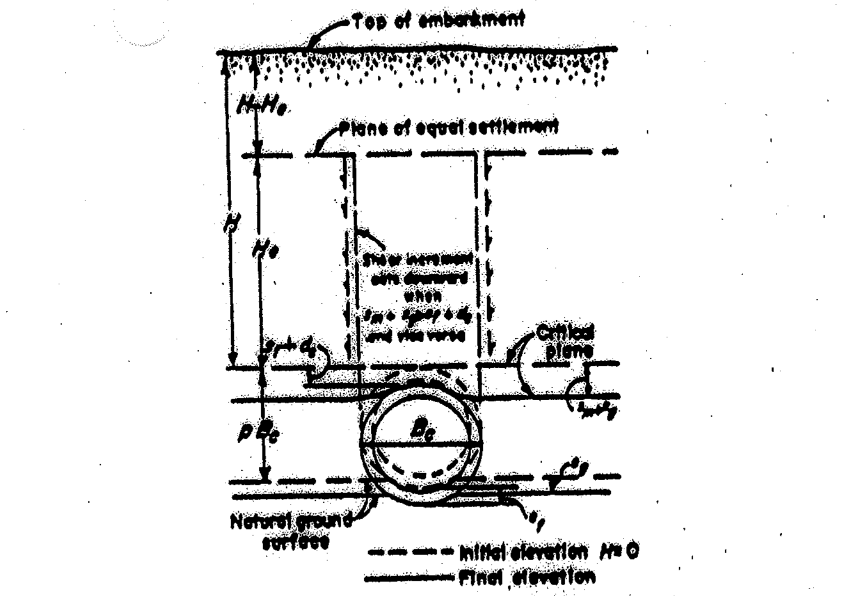
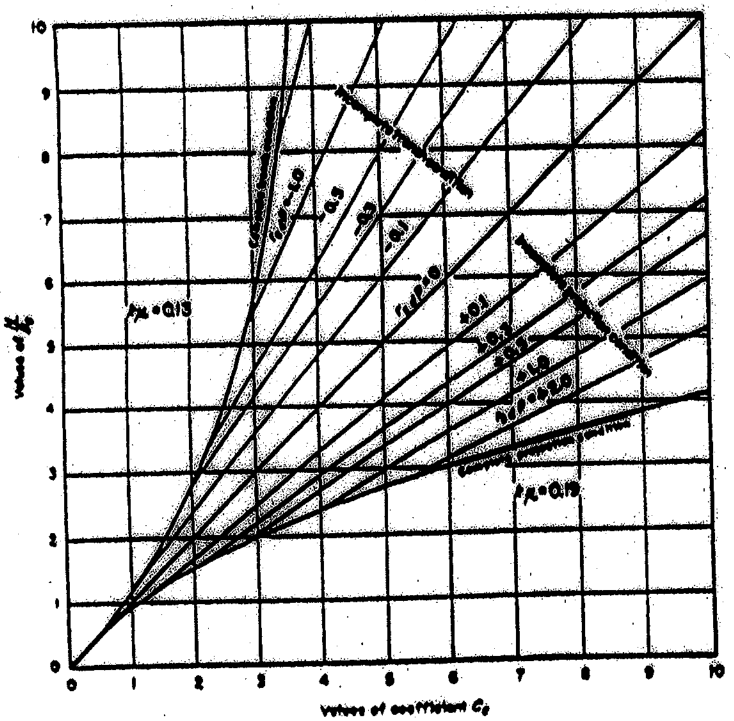


Fig. 9-6. Settlements that influence loads on positive projecting sewer pipe: s_g = settlement of natural ground adjacent to sewer pipe, s_m = compression of columns of soil of height pB_c, d_c = deflection of sewer pipe, and s_f = settlement of bottom of sewer pipe.



as where the natural soils are organic or peat, and the tre... ...kfill is relatively incompressible compacted fill. A more common situatio... is where the sewer pipe is pile-supported in organic soils. In such cases, the load on the sewer pipe is greater than that of the prism above the pipe, and down drag loads should be considered in the design of the piles.

Marston's Formula

Marston's formula for loads on rigid positive projecting sewer pipe is written:

$$\boxed{W_c = C_c w B_c^2} \tag{9.6}$$

in which W_c is the load on the sewer pipe, in newtons/meter (pounds per foot); B_c is the outside width of the sewer pipe, in meters (feet); and C_c is the load coefficient. Values of C_c may be obtained from Fig. 9-7. In this diagram, H is the height of fill above the top of the sewer pipe, in meters (feet); B_c is the outside width of sewer pipe, in meters (feet); p is the projection ratio; and r_{sd} is the settlement ratio (the latter two terms are defined in the next subsection).

Influence of Environmental Factors

The shear component of the total load on a sewer pipe under an embankment depends on two factors associated with the conditions under which the sewer pipe is installed. These are the projection ratio and the settlement ratio.

The projection ratio, p, is defined as the ratio of the distance that the top of the sewer pipe projects above the adjacent natural ground surface, or the top of thoroughly compacted fill, or the bottom of a wide trench, to the vertical outside height of the sewer pipe. It is a physical factor that can be determined in advanced stages of planning when the size of the sewer pipe and its elevation have been established.

The settlement ratio, r_{sd}, indicates the direction and magnitude of the relative settlements of the prism of soil directly above the sewer pipe and of the prisms of soil adjacent to it.

In computing the settlement, the influence of any compressible layers below the sewer pipe also must be considered.

These relative settlements generate the shearing forces which combine algebraically with the weight of the central prism of soil to produce the resultant load on the sewer pipe. The settlement ratio is the quotient obtained by taking the difference between the settlement of the horizontal plane in the adjacent soil which was originally level with the top of the sewer pipe (the critical plane) and the settlement of the top of the sewer pipe, and dividing the difference by the compression of the columns of soil between the natural ground surface and the level of the top of the sewer pipe. The formula for the settlement ratio is:

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \tag{9.7}$$

in which r_{sd} is the settlement ratio; s_g is the settlement of the natural ground adjacent to the sewer pipe; s_m is the compression of the columns of soil of

Table 9-1 ...mmended Design Values of r_{sd}.

Type of Sewer Pipe (1)	Soil Conditions (2)	Settlement Ratio, r_{sd} (3)
Rigid	Rock or unyielding foundation	+1.0
Rigid	Ordinary foundation	+0.5 to +0.8
Rigid	Yielding foundation	0 to +0.5
Rigid	Negative projecting installation	−0.3 to −0.5
Flexible	Poorly compacted side fills	−0.4 to 0
Flexible	Well compacted side fills	0

the sewer pipe, that is, the shortening of its vertical dimension; s_f is the settlement of the bottom of the sewer pipe; and $(s_f + d_c)$ is the settlement of the top of the sewer pipe.

The elements of the settlement ratio are shown in Fig. 9-6. When the settlement ratio is positive, the shearing forces induced along the sides of the central prism of soil are directed downward, and the load on the sewer pipe is greater than the weight of the central prism. When the settlement ratio is negative, the shearing forces act upward and the load is less than the weight of the central prism.

The numerical magnitude of the product of the projection ratio and the settlement ratio, $r_{sd}\,p$, is an indicator of the relative height of the plane of equal settlement and, therefore, of the magnitude of the shear component of the load. The plane of equal settlement is at the top of the sewer pipe when this product is equal to zero. There are no induced shearing forces in this case, and the load is equal to the weight of the central prism (the "prism load").

It is not practical to predetermine a value of the settlement ratio by estimating the magnitude of its various elements except in very general terms. Rather, it should be treated as an empirical factor. Recommended design values of r_{sd}, based on measured settlements of a number of actual installations, are given in the Table 9-1.

The last three cases (Table 9-1) presume soil conditions immediately under the sewer pipe to be the same as those in the adjacent areas outside the trench. In these cases, the settlement ratio may be conservatively assumed as zero in locations with highly fluctuating water tables above the pipe or plastic native trench soils. This results in designing for the "prism load," or the weight of the prism of soil above the pipe. In such cases, C_c is equal to H/B_c and Marston's formula for the prism load becomes:

$$\boxed{W_c = HwB_c} \tag{9.8}$$

The prism load may also be expressed in terms of soil pressure, P, in pascals (pounds per square foot) at depth H as:

$$P = wH = \frac{W_c}{\;} \tag{9.9}$$

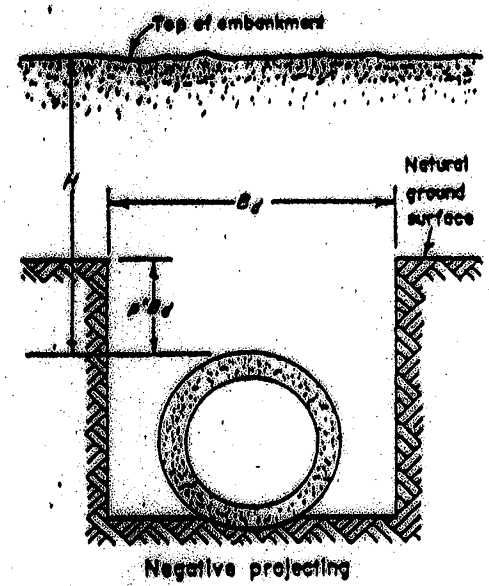


Fig. 9-8. Negative projecting sewer pipe.

Embankment Soil Characteristics

The load on a projecting sewer pipe is influenced directly by the density of the embankment soil. If the soil is to be compacted to a specified dry density, the corresponding wet density under normal moisture conditions should be used in calculating the load. A design value of not less than 1,900 or 2000 kg/m³ (120 or 125 lb/cu ft) is recommended if specific information relative to soil density is not available.

The load also is influenced by the coefficient of internal friction of the embankment soil. Recommended values of the product $k\mu$ (Fig. 9-7) are:

for a positive settlement ratio, $k\mu = 0.19$;
for a negative settlement ratio, $k\mu = 0.13$.

b. Negative Projecting and Induced Trench Sewer Pipes

A negative projecting sewer pipe (Fig. 9-8) is one installed in a relatively shallow trench with its top at some elevation below the natural ground surface. The trench above the sewer pipe is refilled with loose, compressible material, and the embankment is constructed to finished grade by ordinary methods.

Sometimes straw, hay, cornstalks, sawdust, or similar materials may be added to the trench backfill to augment the settlement of the interior prism. The greater the value of the negative projection ratio, p', and the more compressible the trench backfill over the sewer pipe, the greater will be the settlement of the interior prism of soil in relation to the adjacent fill material. In using this technique, the plane of equal settlement must fall below the top of the finished embankment. This action generates upward shearing forces which relieve the load on the sewer pipe.

An induced trench sewer pipe (Fig. 9-9) first is installed as a positive projecting sewer pipe. The embankment then is built up to some height above

Table 9-3 "A[illegible] Pressure Coefficients

Soil Type (1)	"At-Rest" Coefficient (2)
Granular soils	0.5 to 0.67
Cohesive soils, medium to hard	0.67 to 0.88
Cohesive soils, soft	0.75 to 1.0

sewer pipe are automatically generated from the specified boundary conditions, the material properties, and the constitutive relationships of material behavior. Most solutions consider elastic behavior of the materials. Elasto-plastic behavior and nonlinear analysis are also available.

The arch analysis method requires specification of vertical and lateral loads. The vertical loads can be determined by the Marston method, as described in preceding sections, and distributed uniformly over the full width of the sewer pipe. Lateral loads depend on the soil type and geologic history of the soil deposit. Design parameters should be obtained from a soils consultant knowledgeable of the subsurface conditions in the area. For sewer pipe installed in tunnel or in a trench with properly compacted backfill, the recommended design lateral pressures are those corresponding to "at-rest" conditions. Where the backfill on the sides of the sanitary sewer may be loosely placed or insufficiently compacted, "active" pressure coefficients should be used to determine the lateral pressures. For preliminary analysis, the "at-rest" pressure coefficients in Table 9-3 are suggested. Since active and passive earth pressures are the result of lateral strain in the soil mass, the at-rest condition refers to the lateral pressures existing in a large soil mass not subject to horizontal forces or strains except those resulting from its own weight.

C. SUPERIMPOSED LOADS ON SANITARY SEWERS

1. General Method

Two types of superimposed loads are encountered commonly in the structural design of sanitary sewers, concentrated load and distributed load. Loads on sewer pipe caused by these loadings can be determined by application of Boussinesq's solution for stresses in a semi-infinite elastic medium through the convenience of an integration developed by D.L. Holl for concentrated loads and tables of influence coefficients developed by Newmark for distributed loads (26).

Other methods, such as that given in the AASHTO Code, can be used to determine loads on sewer pipe from superimposed loads (27). The AASHTO method is intended for use with wheel loads directly over the pipe and may not be conservative or applicable for other types of loads, such as those from adjacent building foundations. Empirical studies indicate the difficulties of accurately predicting the actual loads on the pipe. Therefore, the method presented in this text is based on the more general and theoretically correct Boussinesq equations.

In the design of buried sewer pipe systems, proper consideration of construction loads is necessary. Construction loads resulting from heavy equipment and reduced backfill heights can produce loads on the sewer pipe that exceed final design loads.

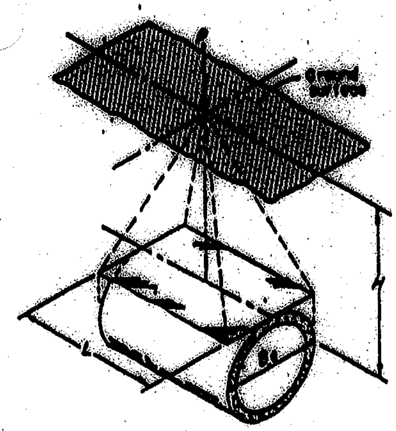


Fig. 9-14. Concentrated superimposed load vertically centered over sewer pipe.

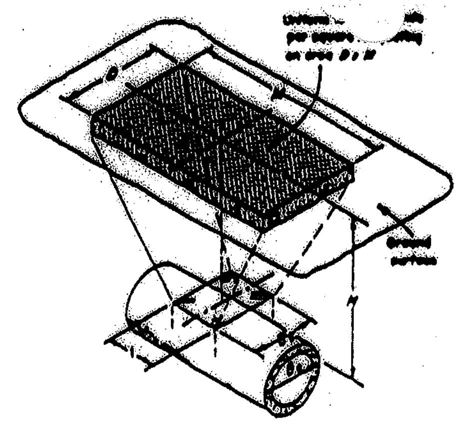


Fig. 9-15. Distributed superimposed load vertically centered over sewer pipe (psf × 47.9 = Pa).

2. Concentrated Loads

The formula for load caused by a superimposed concentrated load, such as a truck wheel (Fig. 9-14), is given the following form by D.L. Holl's integration of Boussinesq's formula:

$$\boxed{W_{sc} = C_s \frac{PF}{L}} \qquad (9.13)$$

in which W_{sc} is the load on the sewer pipe, in newtons per unit length (pounds per unit length); P is the concentrated load, in newtons (pounds); F is the impact factor; C_s is the load coefficient (Table 9-4), a function of $B_c/(2H)$ and $L/(2H)$; H is the height of fill from the top of sewer pipe to ground surface, in m (ft); B_c is the width of sewer pipe, in m (ft); and L is the effective length of sewer pipe, in m (ft).

The effective length of a sewer pipe is defined as the length over which the average load caused by surface traffic wheels produces nearly the same stress in the sewer pipe wall as does the actual load which varies in intensity from point to point. Little research information is available on this subject. Tentative recommendations are to use an effective length equal to 1.0 m (3 ft) for sewer pipe greater than 1.0 m (3 ft) long. The actual length should be used for sewer pipe shorter than 1.0 m (3 ft).

If the concentrated load is displaced laterally and longitudinally from a vertically centered location over the section of sewer pipe under construction, the load on the pipe can be computed by adding algebraically the effect of the concentrated load on various rectangles each with a corner centered under the concentrated load. Values of C_s in Table 9-4 divided by 4 equal the load coefficient for a rectangle whose corner is vertically centered under the concentrated load.

3. Impact Factor

le 9-4 Values of Load Coefficients, C_s, for Concentrated and Distributed Superimposed Loads Vertically Centered over Sewer Pipe[a]

$\frac{D}{H}$ or $\frac{B_c}{H}$ (1)	$\frac{M}{2H}$ or $\frac{L}{2H}$: 0.1 (2)	0.2 (3)	0.3 (4)	0.4 (5)	0.5 (6)	0.6 (7)	0.7 (8)	0.8 (9)	0.9 (10)	1.0 (11)	1.2 (12)	1.5 (13)	2.0 (14)	5.0 (15)
0.1	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112	0.117	0.121	0.124	0.128
0.2	0.037	0.072	0.103	0.131	0.155	0.174	0.189	0.202	0.211	0.219	0.229	0.238	0.244	0.248
0.3	0.053	0.103	0.149	0.190	0.224	0.252	0.274	0.292	0.306	0.318	0.333	0.345	0.355	0.360
0.4	0.067	0.131	0.190	0.241	0.284	0.320	0.349	0.373	0.391	0.405	0.425	0.440	0.454	0.460
0.5	0.079	0.155	0.224	0.284	0.336	0.379	0.414	0.441	0.463	0.481	0.505	0.525	0.540	0.548
0.6	0.089	0.174	0.252	0.320	0.379	0.428	0.467	0.499	0.524	0.544	0.572	0.596	0.613	0.624
0.7	0.097	0.189	0.274	0.349	0.414	0.467	0.511	0.546	0.584	0.597	0.628	0.650	0.674	0.688
0.8	0.103	0.202	0.292	0.373	0.441	0.499	0.546	0.584	0.615	0.639	0.674	0.703	0.725	0.740
0.9	0.108	0.211	0.306	0.391	0.463	0.524	0.574	0.615	0.647	0.673	0.711	0.742	0.768	0.784
1.0	0.112	0.219	0.318	0.405	0.481	0.544	0.597	0.639	0.673	0.701	0.740	0.774	0.800	0.816
1.2	0.117	0.229	0.333	0.425	0.505	0.572	0.628	0.674	0.711	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.440	0.525	0.596	0.650	0.703	0.742	0.774	0.820	0.861	0.894	0.916
2.0	0.124	0.244	0.355	0.454	0.540	0.613	0.674	0.725	0.766	0.800	0.849	0.894	0.930	0.956

[a] nfluence coefficients for solution of Holl's and Newmark's integration of the Boussinesq equation for vertical stress.

Table 9-5 Suggested Values of Impact Factor, F

Traffic Type (1)	F (2)
Highway	1.30
Railway	1.40
Airfield runways (for taxiways, consult FAA)	1.00

traffic at the ground surface. Suggested values for various kinds of traffic are shown in Table 9-5.

The impact effect decreases with increasing cover. The AASHTO (highway) Code (27) recommends a reduction to 1.00 where depth of cover exceeds 1 m (3 ft) or the pipe outside diameter, whichever is lower. The AREA (railway) Code (30) recommends 10 ft (3 m) of cover for the elimination of impact effect. In design of airfield pavements, it is customary not to design for impact on runways because of the counterbalancing effect of the lift provided by aircraft wings. Similarly, for taxiways the slower speed reduces the lift, but it also is considered to reduce impact to a negligible amount in most cases. Since airfield pavement design involves empirical procedures, the design engineer should exercise judgment as to the amount of impact to be included in the design of buried sewer pipes. Common practice is to use an impact factor of 1.0 for runways and 1.5 for taxiways, aprons, hardstands, and run-up pads.

4. Distributed Loads

For the case of a superimposed load distributed over an area of considerable extent (Fig. 9-15), the formula for load on the sewer pipe is:

$$W_{sd} = C_s p F B_c \tag{9.14}$$

in which W_{sd} is the load on the sewer pipe, in newtons per unit length (pounds per unit length); p is the intensity of distributed load, in pascals (pounds per square foot); F is the impact factor; B_c is the width of the sewer pipe, in meters (feet); C_s is the load coefficient, a function of $D/(2H)$ and $(M/2H)$ from Table 9-4; H is the height from the top of the sewer pipe to the ground surface, in meters (feet); and D and M are the width and length, respectively, of the area over which the distributed load acts, in meters (feet).

For the case of a uniform load offset from the center of the sewer pipe, the loads per unit length of the sewer pipe may be determined by a combination of rectangles. For determination of the stress below a point such as A in Fig. 9-16, as a result of the loading in the rectangle BCDE, the area may be considered to consist of four rectangles: AJDF−AJCG−AHEF+AHBG. Each of these four rectangles has a corner at point A. By computing $D/2H$ and $M/2H$ for each rectangle, the load coefficient for each rectangle can be taken from Table 9-4. Since point A is at the corner of each rectangle, the load coefficients from Table 9-4 should be divided by 4. A combination of the stresses from the four rectangles, with signs as indicated above, gives the desired stress

Caterpillar Performance Handbook

Edition 37

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MODEL	816F2		826H		836H	
Flywheel Power	189 kW	**253 hp**	264 kW	**354 hp**	372 kW	**499 hp**
Operating Weight*	23 744 kg	**52,364 lb**	36 967 kg	**81,498 lb**	53 682 kg	**118,348 lb**
Engine Model	**C9 ACERT**		**C15 ACERT**		**C18 ACERT**	
Rated Engine RPM	**2100**		**1800**		**1800**	
No. Cylinders	**6**		**6**		**6**	
Displacement	8.8 L	**537 in³**	15.2 L	**928 in³**	18.1 L	**1105 in³**
Speeds:						
Forward	**2**		**2**		**2**	
Reverse	**2**		**2**		**2**	
Turning Radius with Straight Blade						
Inside Blade Corner	3.5 m	**11'6"**	3.2 m	**10'6"**	4.3 m	**14'11"**
Outside Blade Corner	6.5 m	**21'2"**	7.3 m	**23'9"**	9.0 m	**29'6"**
Fuel Tank Refill Capacity	464 L	**122.6 U.S. gal**	640 L	**169.1 U.S. gal**	795 L	**210 U.S. gal**
WHEELS:	**PLUS TIP**		**PLUS TIP**		**PLUS TIP**	
Each Drum Width	1.02 m	**3'4"**	1.2 m	**3'11"**	1.4 m	**4'7"**
Diameters, over Tips	1.7 m	**5'10"**	1.9 m	**6'6"**	2.0 m	**6'9"**
Drum only	1.3 m	**4'3"**	1.53 m	**5'0"**	1.62 m	**5'8"**
Tips per Wheel	**20**		**25**		**35**	
Tip Height	158 mm	**6.5"**	158 mm	**6.5"**	158 mm	**6.5"**
Chopper Blades per Wheel	**20**		**24**		**28**	
Blade Height	152 mm	**6"**	158 mm	**6"**	158 mm	**6"**
Width of Two Pass Coverage	4.5 m	**14'9"**	4.78 m	**15'8"**	5.67 m	**18'7"**
GENERAL DIMENSIONS:						
Height (Overall)	3.8 m	**12'8"**	4.2 m	**13'7"**	4.5 m	**14'9"**
Height (Top of Cab)	3.4 m	**11'3"**	3.8 m	**12'8"**	4.1 m	**13'6"**
Wheel Base	3.35 m	**11'0"**	3.7 m	**12'2"**	4.55 m	**14'11"**
Overall Length with Dozer	7.85 m	**25'7"**	8.27 m	**27'2"**	10.18 m	**33'5"**
Width over Drums	3.33 m	**10'11"**	3.8 m	**12'8"**	4.18 m	**14'1"**
Ground Clearance	456 mm	**1'5"**	489 mm	**1'6"**	697 mm	**2'3"**
LANDFILL BULLDOZER:						
Width	3.65 m	**12'0"**	4.5 m	**14'9"**	5.19 m	**17'0"**
Height**	1.91 m	**6'3"**	1.91 m	**6'3"**	2.22 m	**7'3"**

*Operating Weight includes coolant, full hydraulics, full fuel tank, all heaviest options and 82 kg (180 lb) operator.
**Height (stripped top) — without ROPS cab, exhaust, seat back or other easily removed encumbrances.

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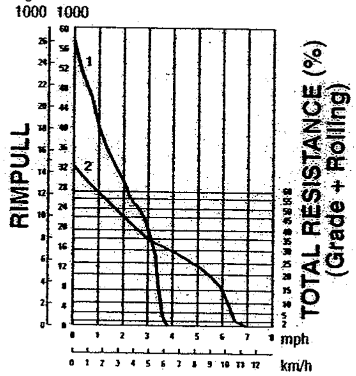

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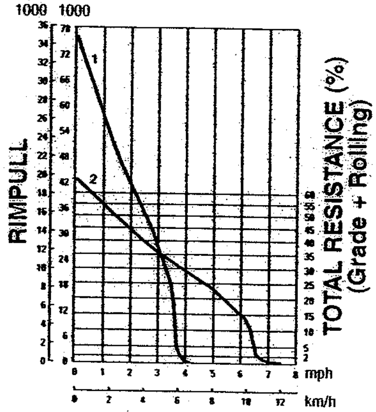

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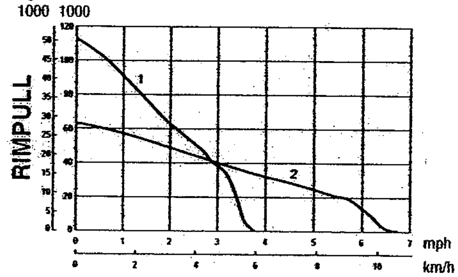
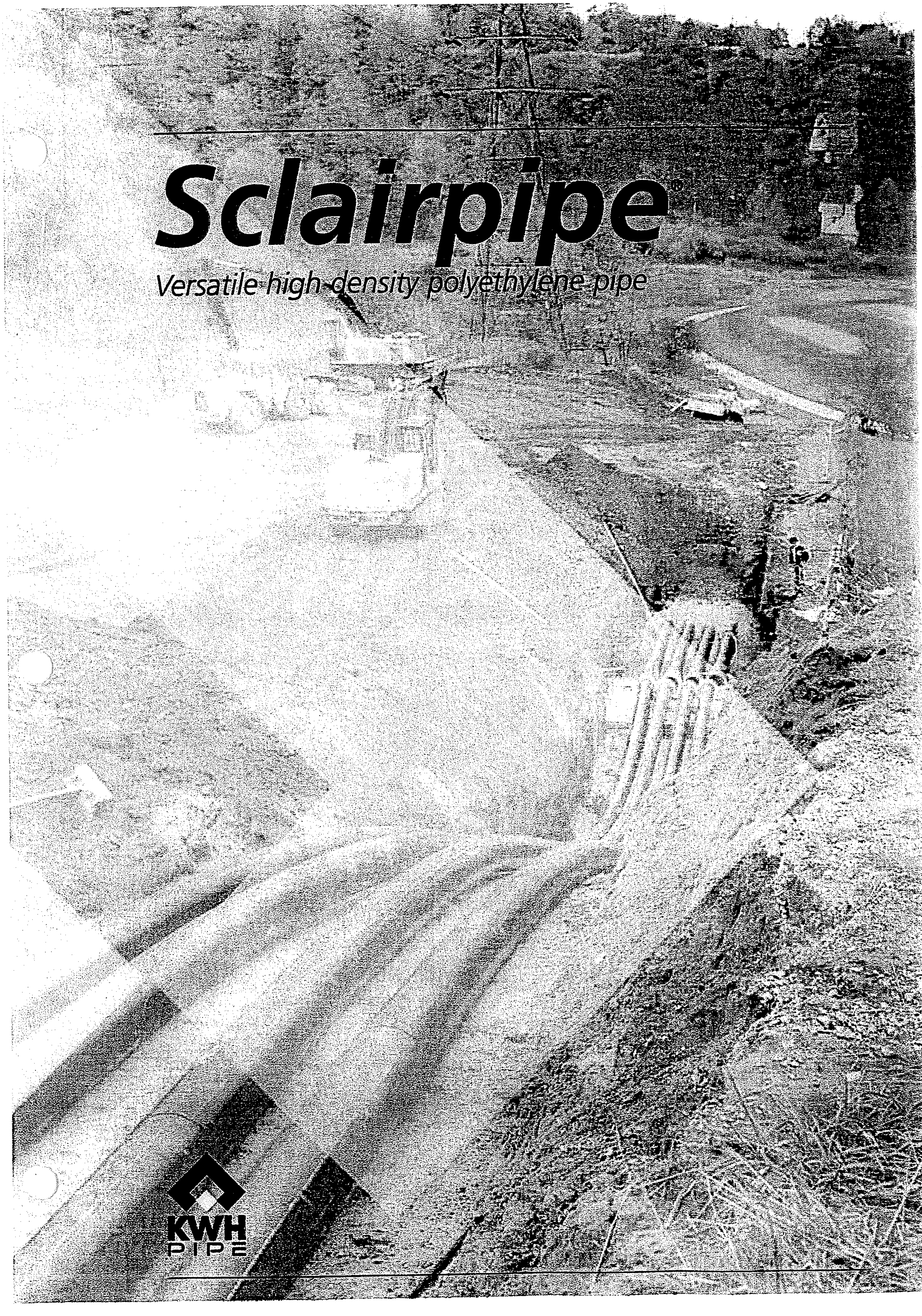


KEY
1 — 1st Gear
2 — 2nd Gear

Sclairpipe®

Versatile high-density polyethylene pipe



Sclairpipe®

KWH PIPE

Choose the size that's right for you

Sclairpipe is available in standard Dimensional Ratio's (DR's), in sizes ranging from 3/4" to 63" in diameter. The Dimensional Ratio relates the minimum wall thickness of the pipe to its outside diameter, and is important to define the pressure rating of a particular pipe. Coiled pipe diameters range from 3/4" and 1" Copper Tube Size (CTS) and 3/4" to 3" nominal outside diameters in Iron Pipe Sizes (IPS). The maximum continuous operating pressure stated is based on a PE3408/3608 (Per ASTM D3350)[1] material for the indicated DR and is based on water service at 73°F.

[1]Sclairpipe is also available from higher density bi-modal resins, having enhanced performance characteristics.

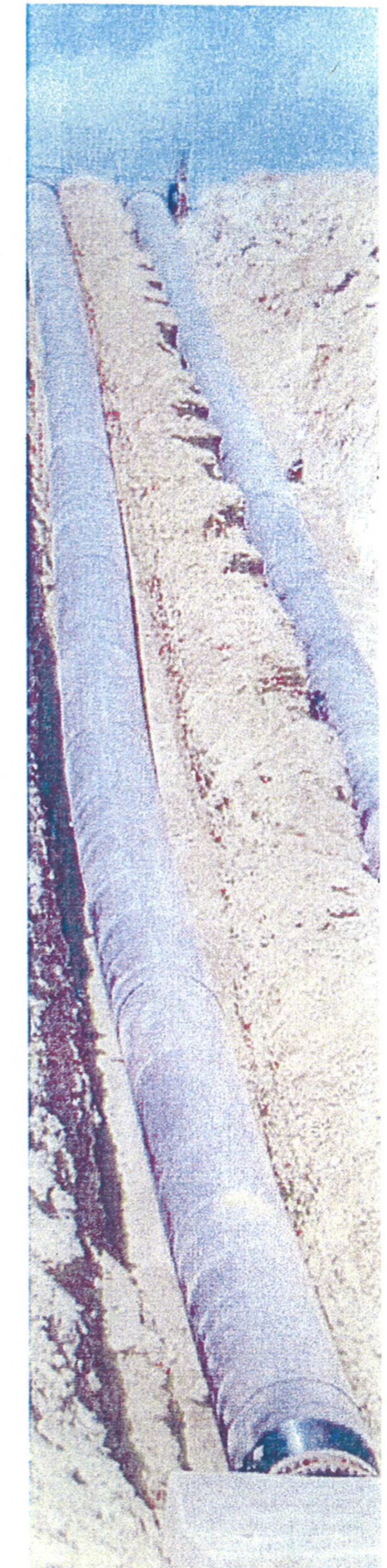
The standard stocked length of Sclairpipe pipe is 50 feet, in sizes above 2" in diameter with longer lengths available on request. Coiled pipe is stocked in diameters up to 3" and is available in 250 and 500 foot lengths. Larger pipe sizes and coil lengths are available on special request.

Please visit our web site (www.kwhpipe.ca) and use our online design tools to determine the pipe size best suited to your specific application.

Nominal Pipe Size	Average Outside Diameter	DR32.5 (50 psi)			DR26 (64 psi)			DR21 (80 psi)			DR17 (100 psi)			DR11 (160 psi)			DR9 (200 psi)			DR7.3 (254 psi)		
		Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)	Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)	Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)	Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)	Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)	Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)	Average Inside Diameter	Minimum Wall Thickness	Average Weight (lbs/ft)
3	3.50	–	–	–	–	–	–	3.147	0.167	0.76	3.064	0.206	0.93	2.825	0.318	1.38	2.676	0.389	1.65	2.484	0.479	1.97
4	4.50	4.206	0.138	0.83	4.133	0.173	1.03	4.046	0.214	1.26	3.939	0.265	1.54	3.633	0.409	2.29	3.440	0.500	2.73	3.193	0.616	3.26
5	5.56	5.200	0.171	1.27	5.109	0.214	1.57	5.001	0.265	1.93	4.869	0.327	2.35	4.491	0.506	3.50	4.253	0.618	4.17	3.947	0.762	4.99
6	6.63	6.193	0.204	1.80	6.085	0.255	2.23	5.956	0.315	2.73	5.799	0.390	3.33	5.348	0.602	4.96	5.064	0.736	5.92	4.701	0.908	7.07
7	7.13	6.660	0.219	2.08	6.544	0.274	2.58	6.406	0.339	3.16	6.236	0.419	3.85	5.752	0.648	5.74	5.447	0.792	6.85	5.056	0.976	8.18
8	8.63	8.062	0.265	3.05	7.922	0.332	3.78	7.754	0.411	4.63	7.549	0.507	5.65	6.963	0.784	8.41	6.593	0.958	10.03	6.120	1.182	11.98
10	10.75	10.049	0.331	4.73	9.873	0.413	5.87	9.665	0.512	7.19	9.409	0.632	8.77	8.678	0.977	13.06	8.218	1.194	15.59	7.628	1.473	18.62
12	12.75	11.918	0.392	6.66	11.710	0.490	8.25	11.463	0.607	10.11	11.160	0.750	12.34	10.293	1.159	18.37	9.747	1.417	21.92	9.047	1.747	26.19
13	13.38	12.503	0.412	7.33	12.284	0.514	9.08	12.025	0.637	11.13	11.707	0.787	13.58	10.797	1.216	20.22	10.224	1.486	24.13	9.491	1.832	28.82
14	14.00	13.087	0.431	8.03	12.858	0.538	9.95	12.587	0.667	12.19	12.254	0.824	14.87	11.302	1.273	22.15	10.702	1.556	26.43	9.934	1.918	31.58
16	16.00	14.956	0.492	10.48	14.695	0.615	12.99	14.385	0.762	15.93	14.005	0.941	19.43	12.916	1.455	28.93	12.231	1.778	34.53	11.353	2.192	41.24
18	18.00	16.826	0.554	13.27	16.532	0.692	16.45	16.183	0.857	20.16	15.755	1.059	24.59	14.531	1.636	36.62	13.760	2.000	43.70	12.773	2.466	52.20
20	20.00	18.695	0.615	16.38	18.369	0.769	20.30	17.981	0.952	24.88	17.506	1.176	30.35	16.145	1.818	45.21	15.289	2.222	53.95	14.192	2.740	64.44
22	22.00	20.565	0.677	19.82	20.206	0.846	24.57	19.779	1.048	30.11	19.256	1.294	36.73	17.760	2.000	54.70	16.818	2.444	65.28	15.611	3.014	77.98
24	24.00	22.434	0.738	23.59	22.043	0.923	29.24	21.577	1.143	35.83	21.007	1.412	43.71	19.375	2.182	65.10	18.347	2.667	77.68	17.030	3.288	92.80
26	26.00	24.304	0.800	27.68	23.880	1.000	34.31	23.375	1.238	42.05	22.758	1.529	51.30	20.989	2.364	76.41	19.876	2.889	91.17	18.449	3.562	108.91
28	28.00	26.174	0.862	32.11	25.717	1.077	39.80	25.173	1.333	48.77	24.508	1.647	59.50	22.604	2.545	88.61	21.404	3.111	105.74	–	–	–
30	30.00	28.043	0.923	36.86	27.554	1.154	45.68	26.971	1.429	55.99	26.259	1.765	68.30	24.218	2.727	101.72	22.933	3.333	121.38	–	–	–
32(M)	31.59	29.533	0.972	40.88	29.018	1.215	50.67	28.405	1.504	62.10	27.654	1.858	75.75	25.505	2.872	112.82	24.152	3.510	134.62	–	–	–
36	36.00	33.652	1.108	53.08	33.065	1.385	65.79	32.366	1.714	80.62	31.511	2.118	98.35	29.062	3.273	146.48	–	–	–	–	–	–
40(M)	39.47	36.894	1.214	63.80	36.251	1.518	79.07	35.485	1.879	96.91	34.547	2.322	118.22	–	–	–	–	–	–	–	–	–
42	42.00	39.260	1.292	72.24	38.575	1.615	89.54	37.760	2.000	109.74	36.762	2.471	133.86	–	–	–	–	–	–	–	–	–
48(M)	47.38	44.291	1.458	91.94	43.519	1.822	113.96	42.599	2.256	139.67	41.473	2.787	170.37	–	–	–	–	–	–	–	–	–
54	54.00	50.478	1.662	119.42	49.597	2.077	148.02	48.549	2.571	181.41	47.266	3.176	221.29	–	–	–	–	–	–	–	–	–
55(M)	55.30	51.688	1.701	125.22	50.786	2.127	155.20	49.713	2.633	190.21	48.399	3.253	232.03	–	–	–	–	–	–	–	–	–
63(M)	63.21	59.086	1.945	163.62	58.055	2.431	202.81	56.828	3.010	248.55	–	–	–	–	–	–	–	–	–	–	–	–

Standard Inventory Product.

- Pipe dimensions are in accordance with ASTM F714 and AWWA C906.
- Pressure ratings are for water at 73.4 deg F.
- Some of the pipe sizes and/or DRs above are only available upon request. Check with your KWH contact for availability.
- Other diameters and DRs not listed may be available upon special request.
- All dimensions are in inches unless otherwise specified.
- Weights are calculated by the methodology established in PPI's TR7.



APPENDIX I.8

RING DEFLECTION OF LEACHATE COLLECTION PIPE

Shaw®

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill No. 3, Inc
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: PCT Date: 10/3/07
Checked By: JPV Date: 10/3/07

TITLE: RING DEFLECTION OF LEACHATE COLLECTION PIPE

Problem Statement

Determine the ring deflection of the leachate collection pipe to demonstrate that an adequate cross-sectional area results to allow cleaning in accordance with 35 Ill. Admin. Code 811.308 (c).

Given

1. Calculation *Earthloads on the Leachate Collection System* contained in this application (Appendix I).

2. KWH Sclairpipe® product information (refer to attached pages)

3. ASTM Standard F 1962-99, *Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings* (refer to attached pages).

4. Harrison, Steven and Watkins, Reynold K., *HDPE Leachate Collection Pipe Design By Fundamentals of Mechanics*, presented at the Nineteenth International Madison Waste Conference, September 25-26, 1996.

5. Leachate collection system design contained in Section 3 of this application.

Assumptions

1. Modified Iowa formula was conservatively used to calculate the deflection of a cylindrical horizontal pipe under earth load (reference KWH Sclairpipe®):

$$\Delta y = \frac{D_I W K_x r^3}{EI + 0.061E' r^3}$$

Where,

Δy = Vertical deflection of pipe (in)
D_l = Deflection lag factor
W = Earthload on pipe (lb/in)
K_x = Bedding constant
r = Mean pipe radius (in) = (O.D. - t_{min})/2
O.D. = Pipe outer diameter (in)
t_{min} = Minimum wall thickness of pipe (in)
E = Modulus of elasticity of polyethylene (psi)

Shaw® Shaw Environmental, Inc.

Client:	Clinton Landfill No. 3, Inc		
Project:	Clinton Landfill No. 3 Chemical Waste Unit		
Proj. #:	128017		
Calculated By:	PCT	Date:	10/3/07
Checked By:	JPV	Date:	10/3/07

TITLE: RING DEFLECTION OF LEACHATE COLLECTION PIPE

I = Moment of inertia of pipe wall = $t_{min}^3/12$
E' = Modulus of soil reaction (psi)

An equivalent form of the equation is derived as follows:

$$\Delta y = \frac{D_I W K_x r^3}{EI + 0.061E' r^3} = \frac{D_I W K_x}{\frac{EI}{r^3} + 0.061E'}$$

$$Where \quad \frac{I}{r^3} = \frac{\frac{t_{min}^3}{12}}{\left(\frac{O.D.-t_{min}}{2}\right)^3} = \frac{2}{3}\left(\frac{1}{\frac{O.D.}{t_{min}}-1}\right)^3 = \frac{2}{3}\left(\frac{1}{SDR-1}\right)^3$$

$$\therefore \Delta y = \frac{D_I W K_x}{\frac{2E}{3(SDR-1)^3} + 0.061E'}$$

2. Leachate collection system design specifies a 6-inch SDR 11 pipe for the proposed Chemical Waste Unit.

3. SDR = DR = (Standard) Dimension Ratio = (Pipe outer diameter)/(pipe wall thickness)

4. D_I = 1.5 (reference KWH Sclairpipe®)

5. W = 680.3 lb/in for a 6-inch SDR 11 Pipe (reference "Earthloads" calculation)

6. K_x = 0.083 (reference KWH Sclairpipe®)

7. E = 30,000 psi (reference KWH Sclairpipe®)

8. E' = 3,000 psi (reference KWH Sclairpipe®)

9. O.D. = 6.63 inches for a 6-inch SDR 11 Pipe (reference KWH Sclairpipe®)

10. Landfill design literature recommends a maximum allowable pipe deflection of 7% of the pipe diameter to allow for cleaning (refer to Harrison and Watkins in attached pages).

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill No. 3, Inc

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT **Date:** 10/3/07

Checked By: JPV **Date:** 10/3/07

TITLE: RING DEFLECTION OF LEACHATE COLLECTION PIPE

Calculation

For a 6-inch SDR 11 Pipe the deflection is calculated as follows:

$$Dy = \frac{D_l W K_x}{\frac{2E}{3(SDR-1)^3} + 0.061E'} = \frac{(1.5)(680.3)(0.083)}{\frac{2(30,000)}{3(11-1)^3} + 0.061(3,000)} = 0.417\,\text{in}$$

$$\%Deflection = \frac{0.417\,\text{in}}{6.63\,\text{in}}(100\%) = 6.29\%$$

Results

The calculated ring deflection is based on the worst-case loading conditions at the landfill, but still allows enough cross-sectional area to clean the leachate collection pipes using a 6-inch hydraulic jet.

Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings[1]

This standard is issued under the fixed designation F 1962; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This guide describes the design, selection considerations, and installation procedures for the placement of polyethylene pipe or conduit below ground using maxi-horizontal directional drilling equipment. The pipes or conduits may be used for various applications including telecommunications, electric power, natural gas, petroleum, water lines, sewer lines, or other fluid transport.

1.2 Horizontal directional drilling is a form of trenchless technology. The equipment and procedures are intended to minimize surface damage, restoration requirements, and disruption of vehicular or maritime traffic with little or no interruption of other existing lines or services. Mini-horizontal directional drilling (min-HDD) is typically used for the relatively shorter distances and smaller diameter pipes associated with local utility distribution lines. In comparison, maxi-horizontal directional drilling (maxi-HDD) is typically used for longer distances and larger diameter pipes common in major river crossings. Applications that are intermediate to the mini-HDD or maxi-HDD categories may utilize appropriate "medi" equipment of intermediate size and capabilities. In such cases, the design guidelines and installation practices would follow those described for the mini- or maxi-HDD categories, as judged to be most suitable for each situation.

1.3 The values stated in inch-pound units are to be regarded as the standard. The values given in parentheses are for information purposes only.

1.4 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of the regulatory limitations prior to use.* Section 6 contains general safety information related to the use of maxi-horizontal directional drilling equipment.

2. Referenced Documents

2.1 *ASTM Standards:*

D 420 Guide to Site Characterization for Engineering, Design, and Construction Purposes[2]

D 422 Test Method for Particle-Size Analysis of Soils[2]

D 1586 Test Method for Penetration Test and Split-Barrel Sampling of Soils[2]

D 1587 Practice for Thin-Walled Tube Geotechnical Sampling of Soils[2]

D 2113 Practice for Diamond Core Sampling for Site Investigations[2]

D 2166 Test Method for Unconfined Compressive Strength of Cohesive Soil[2]

D 2435 Test Method for One-Dimensional Consolidation Properties of Soils[2]

D 2447 Specification for Polyethylene (PE) Plastic Pipe, Schedules 40 and 80 Based on Controlled Outside Diameter[3]

D 2513 Specification for Thermoplastic Gas Pressure Pipe, Tubing, and Fittings[3]

D 2657 Practice for Heat-Joining of Polyolefin Pipe and Fittings[3]

D 2850 Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression[2]

D 3035 Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Controlled Outside Diameter[3,4]

D 4186 Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading

D 4220 Practices for Preserving and Transporting Soil Samples[2]

D 4318 Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils[2]

D 4767 Test Method for Consolidated-Undrained Triaxial Compression Test on Cohesive Soils[2]

D 5084 Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter[5]

F 714 Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter[3]

F 1804 Practice for Determining Allowable Tensile Load

[1] This guide is under the jurisdiction of ASTM Committee F-17 on Plastic Piping Systems and is the direct responsibility of Subcommittee F17.67 on Trenchless Plastic Pipeline Technology.

Current edition approved April 10, 1999. Published August 1999.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

[3] *Annual Book of ASTM Standards*, Vol 08.04.

[4] *Annual Book of ASTM Standards*, Vol 08.02.

[5] *Annual Book of ASTM Standards*, Vol 04.09.

for Polyethylene (PE) Gas Pipe during Pull-In Installation[3]

2.2 *Other Standards:*

ANSI Preferred Number Series 10

ANSI/EIA/TIA-590 Standard for Physical Location and Protection of Below-Ground Fiber Optic Cable Plant[6]

OSHA-3075 Controlling Electrical Hazards[7]

TR-NWT-000356 Generic Requirements for Optical Cable Innerduct[8]

3. Terminology

3.1 *Definitions:*

3.1.1 *horizontal directional drilling, HDD, n*—a technique for installing pipes or utility lines below ground using a surface-mounted drill rig that launches and places a drill string at a shallow angle to the surface and has tracking and steering capabilities.

3.1.1.1 *Discussion*—The drill string creates a pilot bore hole in an essentially horizontal path or shallow arc which may subsequently be enlarged to a larger diameter during a secondary operation which typically includes reaming and then pullback of the pipe or utility line. Tracking of the initial bore path is accomplished by a manually operated overhead receiver or a remote tracking system. Steering is achieved by controlling the orientation of the drill head which has a directional bias and pushing the drill string forward with the drill head oriented in the direction desired. Continuous rotation of the drill string allows the drill head to bore a straight path. The procedure uses fluid jet or mechanical cutting, or both, with a low, controlled volume of drilling fluid flow to minimize the creation of voids during the initial boring or backreaming operations. The drilling fluid helps stabilize the bore hole, remove cuttings, provide lubricant for the drill string and plastic pipe, and cool the drill head. The resultant slurry surrounds the pipe, typically filling the annulus between the pipe and the bored cavity.

3.1.2 *maxi-horizontal directional drilling, maxi-HDD, n*—a class of HDD, sometimes referred to as directional drilling, for boring holes of up to several thousand feet in length and placing pipes of up to 48 in. (1¼ m) diameter or greater at depths up to 200 ft (60 m).

3.1.2.1 *Discussion*—Maxi-HDD is appropriate for placing pipes under large rivers or other large obstacles (Fig. 1). Tracking information is provided remotely to the operator of the drill rig by sensors located towards the leading end of the drill string. Cutting of the pilot hole and expansion of the hole is typically accomplished with a bit or reamer attached to the drill pipe, which is rotated and pulled by the drilling rig.

3.1.3 *mini-horizontal directional drilling, mini-HDD, n*—a class of HDD, sometimes referred to as guided boring, for boring holes of up to several hundred feet in length and placing pipes of typically 12 in. (300 mm) or less nominal diameter at depths typically less than 25 ft (7 m).

3.1.3.1 *Discussion*—Mini-HDD is appropriate for placing local distribution lines (including service lines or laterals) beneath local streets, private property, and along right-of-ways. The creation of the pilot bore hole and the reaming operations are typically accomplished by fluid jet cutting or the cutting torque provided by rotating the drill string, although mud motors powered by the drilling fluid are sometimes used for hard or rocky soil conditions. The use of such mud motors would only be applicable for the larger mini-HDD machines. The locating and tracking systems typically require a manually operated overhead receiver to follow the progress of the initial pilot bore. The receiver is placed above the general vicinity of the drill head to allow a determination of its precise location and depth, indicate drill head orientation for determining steering information to be implemented from the drill rig.

3.1.4 *pipe dimension ratio, DR, n*—the average specified diameter of a pipe divided by the minimum specified wall thickness.

3.1.4.1 *Discussion*—For pipes manufactured to a controlled outside diameter (OD), the DR is the ratio of pipe outer diameter to minimum wall thickness. The standard dimension ratio (SDR) is a specific ratio of the outside diameter to the minimum wall thickness as specified by ANSI Preferred Number Series 10.

NOTE 1—Lower DR values correspond to thicker, stronger pipes.

4. Preliminary Site Investigation

4.1 *General Considerations*—A maxi-HDD project, such as that associated with a river crossing, is a major event that will

[6] Available from the Electronics Industries Association, 2001 Pennsylvania Ave., N.W., Washington, DC, 20006.

[7] Available from the Occupational Health and Safety Administration, 200 Constitution Ave. N.W. Washington, DC 20210.

[8] Available from Bellcore, 60 New England Ave., Room 1B252, Piscataway, NJ, 08854-4196.

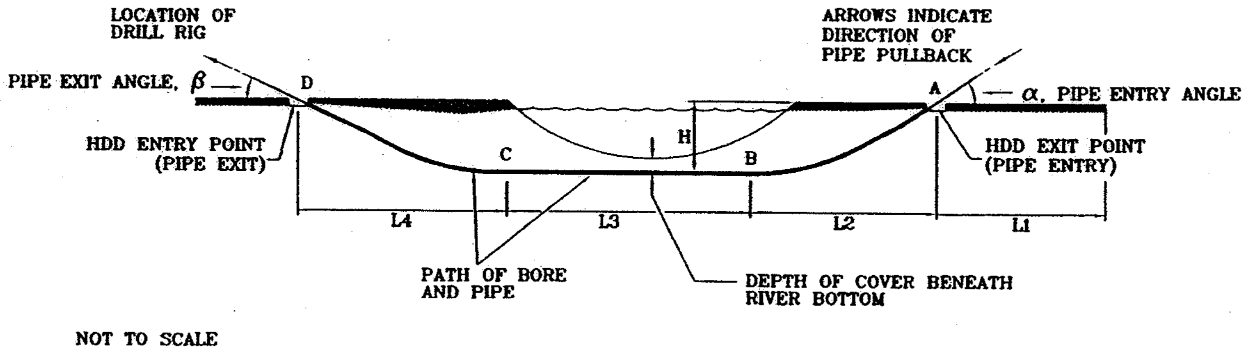


FIG. 1 Maxi-HDD for Obstacle (for example, River) Crossing

require extensive and thorough surface and subsurface investigations. Qualified geotechnical engineers should perform the work for the owner in preparation for planning and designing of the bore route. The information should also be provided to the potential contractors to provide guidance for the bidding stage and subsequent installation. The contractor may perform additional investigations, as desired. Since typical maxi-HDD projects represent river crossings, the following procedures are described in terms of the specific investigations and issues arising in such cases. The general procedures, however, may be appropriately interpreted to also apply to non-river crossings, such as under land-based obstacles including highways, railways, etc.

4.2 *Surface Investigation* **(1, 2)**[9]

4.2.1 *Topographic Survey*—A survey should be conducted to accurately define the working areas described in 4.1 for the proposed crossing site. Horizontal and vertical references must be established for referencing hydrographic and geotechnical data. The survey should typically include overbank profiles on the anticipated path center-line, extending about 150 ft (75 m) landward of the bore entry point to the length of the (pre-fabricated) pull section landward of the bore exit point. The survey information should be related to topographical features in the vicinity of the proposed crossing. Existing topographical information may be available from the U.S. Geological Survey, or Federal, state, or county publications. Aerial photographs or ordnance surveys may be useful, especially for crossing land-based obstacles in urban areas, since these may indicate the presence of demolished buildings and the possibility of old foundations, as well any filled areas **(3)**. It is also necessary to check available utility records to help identify the precise location of existing below-ground facilities in the vicinity, including electric power, natural gas, petroleum, water, sewer, or telecommunications lines. The presence of existing pipelines, support pilings, etc., containing significant steel mass should be noted since this may cause interference with magnetically sensitive equipment guidance or location instrumentation.

4.2.1.1 *Drill Rig (Bore Entry) Side*—The available area required on the side of the drill rig must be sufficient for the rig itself and its ancillary equipment. In general, the size of the required area on the rig side will depend upon the magnitude of the operation, including length of bore and diameter of pipe to be placed. Typically, a temporary workspace of approximately 150 ft (45 m) width by 250 ft (75 m) length will be sufficient. These dimensions may vary from 100 by 150 ft (30 by 45 m) for shorter crossings of 1000 ft (300 m) or less, to 200 by 300 ft (60 by 90 m) for medium or long crossings.

4.2.1.2 *Water Supply*—Water storage and facilities for mixing, storing, and pumping drilling fluid will require significant space. Although it is standard practice to draw fresh water found at the location for mixing the drilling fluid, alternate water supplies may be required to obtain proper drilling fluid characteristics. Hard or salty water is undesirable, although additives may be used to create the proper pH value. It may be necessary to provide access for trucks to transport water or to provide for the installation of a relatively long surface pipe or hose connecting a remote hydrant.

4.2.1.3 *Pipe (Bore Exit) Side*—Assuming the pipe to be placed is too large a diameter to be supplied on a reel (for example, larger than 6 in. (150 mm)), sufficient space is required at the side opposite that of the drill rig, where the bore will exit and the pipe be inserted, to accommodate a continuous straight length of pre-fabricated pipe. The space for the straight length will begin approximately 50 to 100 ft (15 to 30 m) from the anticipated bore exit and extend straight landward at a width of 35 to 50 ft (10 to 15 m), depending upon the pipe diameter. In the immediate vicinity of the bore exit (pipe entry), an area of typically 50 ft (15 m) width by 100 ft (30 m) length is required; for relatively large diameter pipes (larger than 24 in. (600 mm), or in cases of difficult soil conditions, an area of 100 ft (30 m) width by 150 ft (45 m) length should be provided.

4.2.2 *Hydrographic/Potamological Survey*—For crossing significant waterways, a survey should be conducted to accurately describe the bottom contours and river stability to establish suitability for the design life of the pipeline. Typically, depths should be established along the anticipated center-line, and approximately 200 ft (60 m) upstream and downstream; closer readings may be required if it is necessary to monitor future river activity. Consideration should be given to future changes in river bank terrain. Washouts, bank migrations, or scour can expose pipe.

4.2.3 *Drilling Fluid Disposal*—The means for disposal of the drilling fluid wastes must be considered. The volume of drilling fluid used will depend upon the soil characteristics but is typically on the order of 1 to 3 times the volume of removed soil. Most drilling fluids use bentonite or polymer additives which are not generally considered to be hazardous. However, local regulations should be followed regarding disposal.

4.2.3.1 *Drilling Fluid Recirculation*—Occasionally, drilling fluid recirculation is used to reduce overall material and disposal costs. If drilling fluid recirculation is contemplated, a means must be considered for transporting any fluid exhausted from the opposite (bore exit) side, during the pullback operation, to the rig side. This may be accomplished by truck, barge, or a temporary recirculation pipe line on the bottom of the waterway (for river-crossings). The recirculation line must be adequate to prevent accidental discharge into the waterway.

4.3 *Subsurface Investigation*—The overall technical and economic feasibility of the maxi-HDD process is highly dependent upon the properties of the soil formation through which the penetration will be accomplished. Thus, an accurate and thorough geotechnical investigation must be performed by a qualified engineer, including review of existing information and site specific studies for the proposed location. This information will be used to produce design drawings (including final bore route, pipe design, and bore design), construction specifications, and permit applications as well as to provide information for the contractors upon which to select appropriate tools and methods for the actual construction. While the guidelines given in the following sections point out general procedures or types of information, or both, which could be

[9] The boldface numbers in parentheses refer to the list of references at the end of this standard.

developed, unforeseeable site-specific variables make the thoroughness and accuracy of any site characterization study directly dependent on the skill, experience, and inquisitiveness of the investigating engineer. Therefore, the investigator should define the configuration, extent, and constituency of the investigation. Site characterization information must go beyond just defining soil conditions along the bore path to include a forecast of future conditions (that is, river meanders and scours) and to anticipate the affect of the maxi-HDD process on site conditions.

4.3.1 *Preliminary Study*—The subsurface investigation should begin with a review of existing data such as may be obtained from published soil reports (for example, Soil Conservation Service Report, U.S. Geological Survey, U.S. Army Corps of Engineers reports, etc.) or records from previous construction projects. In particular, data from nearby pipe or cable river-crossings, or bridge foundation construction should be examined. The results of this study will be used to define the initially recommended bore penetration profile path.

4.3.2 *Test Borings* **(1,2,4)**—Site-specific data must be obtained to fully characterize and verify the conditions through which the proposed bore path will be created. Refer to Guide D 420, Test Method D 1586, Test Method D 1587, Test Method D 2213 and Practice D 4220. Data collection should be aimed at identifying earth materials at the site and at exploring subsurface stratification (including identification of the boundary between rock and other strata, presence of cobbles or boulders and other anomalies such as old tree stumps and fill debris). The location, depth, and number of borings should be determined by the engineer based on the preliminary study, anticipated future changes in site conditions (river meanders, scours, etc.), and modifications of soil conditions during construction. These borings should be located at a sufficient lateral distance (to either side) from the proposed bore path to avoid boring into the test hole, and the holes should be sealed with grouting to avoid potential leakage paths for drilling fluid during the actual installation. Following completion of the detailed route design (Section 7), additional test borings may be desirable at critical points such as bends.

NOTE 2—In environmentally sensitive areas, possible restrictions may exist on the location or number of test borings.

4.3.3 In addition to test borings, dynamic cone testing or developing non-intrusive techniques such as ground penetrating radar or sonar may be used to identify stratification and areas with anomalies. Such probing techniques may be applied in the proximity of known conditions determined by a boring to obtain proper calibration, and then extended towards untested areas at relatively close intervals to identify irregularities between borings. If needed, additional borings may then be made at intermediate points of interest **(3,4)**.

4.3.4 *Soil Analysis* **(2,5,6)**—The geotechnical study should evaluate several parameters, including soil classifications, (Refer to Test Methods D 4318 and D 4220.) strength and deformation properties, (Refer to Test Methods D 1586, D 2166, D 2435, D 2850, D 4186, and D 4767.) and groundwater table behavior. (Refer to Test Method D 5084.) Although some field evaluation and in-situ testing should be included, the geotechnical investigation should emphasize laboratory testing in order to obtain more accurate and meaningful quantitative results. If rock is encountered, the borings should penetrate sufficiently to verify whether or not it is bedrock. The relevant soil testing methods listed in Section 2 should be followed. In general, the following specific data should be obtained from the borings:

4.3.4.1 Standard classification of soils, (Refer to Test Method D 4318),

4.3.4.2 Gradation curves for granular soils, as described in Test Method D 4220,

4.3.4.3 Standard penetration test values, as described in Test Method D 1586,

4.3.4.4 Cored samples of rock with rock quality designation (RQD) and percent recovery,

4.3.4.5 Unconfined compressive strength, as described in Test Method D 2166,

4.3.4.6 Moh's hardness for rock samples,

4.3.4.7 Possible contamination (hazardous waste),

4.3.4.8 Groundwater location, type, and behavior, and

4.3.4.9 Electrical resistivity or mineralogical constituents.

4.3.5 For river crossings, the results from the preliminary study and site specific tests should be combined in a comprehensive report describing the geotechnical subsurface conditions beneath the river bottom plus the stream's potential for meandering and scouring. The results must then be considered by the owner, the engineer, and potential contractors, with regard to compatibility with the state-of-the-art of directional drilling technology for cost-effectively completing the task. If necessary, the crossing location may be altered to a more favorable crossing site. In this case, many of the surface and subsurface investigations may have to be repeated for the new proposed crossing location and bore path.

4.3.6 *Feasibility*—Soil conditions are a major factor affecting the feasibility and cost of using maxi-HDD in a given geographic area. Table 1 indicates the suitability of horizontal directional drilling as a function of the general characteristics of the soil conditions in the area and depths of interest **(3,5)**. The "generally suitable" category presumes knowledgeable, experienced contractors or personnel using appropriate equipment. Such contractors are assumed to have a minimum of one year field experience and completed approximately 30 000 ft (10 km) of construction in related projects. The size and type machines considered appropriate for particular installations are a function of bore length, final hole diameter, and soil conditions. Various type drill heads, mud motors, reamers, and drilling fluid capabilities are available for various ground conditions. The conditions under which "difficulties may occur" may require modifications of routine procedures or equipment, such as the use of special purpose drill heads or optimized drilling fluids. Some cases will entail "substantial problems" and may not be economically feasible for directional drilling using present technology. The potential for problems to occur increases with the presence of gravels, boulders, or cobbles or with transitions from non-lithified material into solid rock. In such cases, other drilling locations or construction alternatives should be considered unless special circumstances dictate the need for directional drilling at the present location, even at high costs associated with special rock

TABLE 1 Soil Conditions and Suitability of Horizontal Directional Drilling[A]

Soil Conditions	Generally Suitable	Difficulties May Occur	Substantial Problems
Soft to very soft clays, silts, and organic deposits		X	
Medium to very stiff clays and silts	X		
Hard clays and highly weathered shales	X		
Very loose to loose sands above and below the water table (not more than 30 % gravel by weight)		X	
Medium to dense sands above or below the water table (not more than 30 % gravel by weight)	X		
Very loose to dense gravelly sand, (30 % to 50 % gravel by weight)		X	
Very loose to dense gravelly sand (50 % to 85 % gravel by weight)			X
Very loose to very dense gravel			X
Soils with significant cobbles, boulders, and obstructions			X
Weathered rocks, marls, chalks, and firmly cemented soils	X		
Slightly weathered to unweathered rocks		X	

[A]For additional information, see Ref. (5).

drilling techniques, etc.

5. Safety and Environmental Considerations

5.1 *General Considerations*—Injury to personnel may result from the mechanical and hydraulic machine operations directly related to the drilling operation or from striking of electric power lines or buried pipelines. In addition, the scale of maxi-HDD operations may involve additional equipment and accessories required for the lifting and handling of heavy drill rods, drill heads, reamers, etc., as well as the product pipe or conduit. Additional precautions relating to specific auxiliary equipment must be followed, but is beyond the scope of this standard. Non-essential personnel and bystanders should not be allowed in the immediate vicinity of the maxi-HDD equipment. Barriers and warnings should be placed a minimum of 30 ft (10 m) from the edge of the equipment or associated hardware. Safety precautions are to be followed by all personnel and at both ends of the bore path. Inadvertent contact with electric power, natural gas, or petroleum lines may result in hazards to personnel or contamination. If possible, any in-service pipeline in the proximity of the bore should be de-activated during the construction. In general, the possibility of injury or environmental impact caused by damage to working or powered subsurface facilities or pipelines during the initial boring or backreaming operations is reduced by appropriate adherence to regulations and damage prevention procedures, as outlined in Section 6.

5.2 *Work Clothing*—**Caution:** Loose clothing or jewelry should not be worn since they may snag on moving mechanical parts. Safety glasses or OSHA approved goggles, or both, and OSHA approved head gear should be worn at all times. Protective work shoes and gloves must be worn by all personnel.

5.3 *Machine Safety Practices*—Contractors must comply with all applicable OSHA, state, and local regulations, and accepted industry practices. All personnel in the vicinity of the drill rig or at the opposite end of the bore must be properly trained and educated regarding the potential hazards associated with the maxi-HDD equipment. For electrical hazards, see OSHA 3075. Personnel shall be knowledgeable of safe operating procedures, safety equipment, and proper precautions. Courses and seminars are available in the industry, including training provided by the equipment suppliers.

5.3.1 The operation of the drill rig requires rotation and advancement or retraction of the drill rods. Drill rig operation is typically accomplished using chain drives, gear systems, and vises which may potentially lead to personal injury due to the moving mechanical components. All safety shields or guards must be properly mounted. The equipment must be checked at the beginning of each work day to verify proper operation.

5.3.2 *Hydraulic Fluid*—The hydraulic oil lines powering the drill rig operate under pressures of several thousand psi (hundreds of bars). The hoses and connectors must be properly maintained to avoid leaks.

5.3.2.1 **Caution:** If a leak is suspected, it should be checked by using a piece of cardboard or other object, but not hands or any other part of the body. The high pressure hydraulic fluid can penetrate the skin, burn, or cause blood poisoning. Before disconnecting any hydraulic lines, the system pressure should be relieved.

5.3.3 *Drilling Fluid*—Drilling fluid pressures will vary depending upon the equipment design and operator preference; pressures of several thousand psi (hundreds of bars) are possible. The hoses and connections must be properly maintained to avoid leaks.

5.3.3.1 **Caution:** Suspected leaks should be checked by using a piece of cardboard or other object. Avoid the use of hands or any other part of the body to check for a leak. Before individual drill rods are inserted or removed from the drill string, it must be verified that the drilling fluid pressure has been shut off and allowed to decrease; otherwise, high pressure fluid will squirt from the joint and possibly cause injury to personnel. The drilling fluid pressure gage must be checked to verify the pressure has been relieved before disconnecting any rods.

NOTE 3—If the pressure does not decrease in a short interval following pressure shut off, the fluid jet openings at the drill head may be clogged. Special care must then be made when disconnecting the rod. It may be necessary to retract the drill string or expose the drill head to clear the jets before continuing the operation. To avoid injury from the drill head and drilling fluid, all personnel should maintain a safe distance from the exit point of the bore as the drill head surfaces. The pressure should be shut off as soon as the drill head exits.

5.4 *Construction Effects on Site*—It is assumed that the preliminary site investigations included analyses to verify the stability of embankments, roads, or other major features to be traversed. It is necessary to ensure that the maxi-HDD operation will not negatively impact the site upon completion. In many cases, it will be appropriate to use grouting to seal the final bore path hole or the end portions of the hole following the installation of the pipe to prevent future flow or environmental contamination. Particularly sensitive areas include statutorily designated areas, such as wetlands, natural and scenic waterways, or contaminated or waste disposal sites. If the bore will pass through, or in close proximity to, a contaminated area, special spoils monitoring and disposal procedures must be followed, consistent with applicable Federal, state, or local regulations.

5.4.1 *Drilling Fluid*—The most common drilling fluid additive is bentonite, a naturally occurring clay. When added to water, the resulting fluid provides desired properties including viscosity, low density, and lubricity. The bentonite material used should be National Sanitation Foundation (NSF) certified. Disposal should be in accordance with local laws and regulations. The bentonite-water slurry is not a hazardous material unless it becomes mixed with toxic pollutants. The waste material is usually considered as typical excavation spoils and can be disposed or by means similar to other spoils. If other additives are of concern or hazardous material disposal is required, it may be necessary to de-water the spoils, transport the solids to an appropriate disposal site, and treat the water to meet disposal requirements.

5.4.2 The utility access pits which may be present at both ends of the bore are convenient receptacles for collecting used drilling fluid. If not present for utility access, small pits should be provided at both ends to serve as such receptacles. Depending upon soil permeability, the pits may be lined with an appropriate material or membrane. The pits should be emptied as necessary. Some maxi-HDD systems use drilling fluid recirculating systems to reduce the volume of spoils. If the geotechnical investigation revealed the existence of soil conditions conductive to fluid migration, such as through pre-fractures in surrounding clay or soil mass permeability, this condition must be anticipated and accounted for in the drilling operation.

6. Regulations and Damage Prevention

6.1 *General Considerations*—The owner of the proposed pipeline should obtain any required drilling permits and is responsible for obtaining approvals from the Federal, state, or local jurisdictions or other agencies that may be affected by the work. The preliminary investigations (Section 4) should identify appropriate site locations and paths, including safe separations from other facilities such as electric power, natural gas, or petroleum lines. If the constraints for a particular maxi-HDD bore are such as to be in the vicinity of known facilities, the affected owners must be contacted and strict procedures for location and marking followed. If a maxi-HDD bore interconnects points under the jurisdiction of several states or governing bodies, then the regulations of all parties must be considered, including relevant permits. Special restrictions may exist, including restoration regulations, in environmentally sensitive habitat areas.

6.2 *Environmental, Health, and Safety Plan*—When required, each contractor that will work on the project must submit an environmental, health, and safety plan. Items to consider are the responsibilities of the plan, reporting, employee training, MSDS sheets for materials being used, emergency telephone numbers for police, fire department, and medical assistance, fire prevention, sanitation, and industrial hygiene.

6.3 *Environmental and Archaeological Impact Study*—Most projects using maxi-HDD will require procurement of various environmental permits. When an environmental permitting plan must be prepared, it should include a list of required permits (for example, USAE, USEPA), the time needed to prepare permits, and an estimated date of issuance. Items to consider are solid and hazardous materials and waste management, wetlands, burial grounds, land use, air pollution, noise, water supply and discharge, traffic control and river and railroad transportation.

6.4 *Waterways* (see ANSI/EIA/TIA-590)—The U.S. Army Corps of Engineers (USAE) regulates activities involving interstate bodies of water, including marshes and tributaries, as well as intrastate waters which could affect interstate or foreign commerce. The organization is responsible for work affecting such waterways, including to the headwaters of freshwater streams, wetlands, swamps and lakes. The Regional District Engineer of the USAE will advise applicants of the types of permits required for such proposed projects. In addition, a state or local, or both, agency environmental review and permit may be required.

6.5 *Railroad Crossings* (see ANSI/EIA/TIA-590)—The chief engineer of the railroad should be consulted for the approved methods of crossing the railroad line. For spur tracks or sidings, the tract owner should be consulted. Railroads normally require cased pipes at crossings to prevent track washouts or damage in the event of pipeline rupture. (At the time of writing of this standard, an American Railway Engineering Association (AREA) committee is studying the use of HDD for uncased and cased crossing of railroads for both plastic and steel gas pipelines.)

7. Bore Path Layout and Design

7.1 *General Considerations*—For maxi-HDD projects, such as river crossings, the bore path should be designed and specified by the engineer representing the owner prior to the contractor bidding process. Based upon the preliminary surface and subsurface investigations, the path will be selected to place the pipe within stable ground and isolated from river activities for the design life of the utility line. The ground through which the path will traverse must be compatible with maxi-HDD technology. In general, for maxi-HDD projects, the design path will lie within a vertical plane. If necessary, lateral curvature is possible, consistent with the capabilities of the equipment and the product pipe. The path should be clearly designated in an integrated report summarizing the results of the surface and subsurface investigations, and should be used for pricing, planning, and executing the operation.

7.2 *Steering and Drill Rod Constraints*—The planned path must be consistent with the steering capability of the drill string and the allowable radius of curvature of the steel drill rods based upon the corresponding bending stresses in the steel rods and joints. Although some soil conditions will inhibit sharp steering maneuvers, path limitations will often be based upon fatigue strength considerations of the rods. A given rod may be ble to withstand a single bend cycle corresponding to a relatively sharp radius of curvature, but the rotation of the rod during the boring operation results in flexural cycles which may eventually cause cumulative fatigue failure. The diameter of the drill rod is an important parameter affecting its stiffness, steering capability, and the allowable bend radii. A conservative industry guideline indicates the minimum bend radius should be approximately:

$$(R_{rod})_{min} = 1200\, D_{rod} \tag{1}$$

where:

$(R_{rod})_{min}$ = medium recommended bend radius of drill rod, in. (mm), and

D_{rod} = nominal diameter of drill rod, in. (mm).

This applies to bends in horizontal (plan) or vertical (profile) planes.

7.3 The proposed path should avoid unnecessary bends. Such trajectories may be difficult to follow and may lead to oversteering and excessive bends, resulting in increased stresses in the drill rods and greater required pulling forces during the installation of the pipe. The local radius of curvature of the path at any point may be estimated by:

$$R = \frac{\Delta S}{\Delta \phi} \tag{2}$$

where:

R = local radius of curvature along path segment, ft (m),

ΔS = distance along path, ft (m), and

$\Delta \phi$ = angular change in direction, rad.

NOTE 4—The angle in radians is equal to the angle in degrees × 0.0175. (One radian equals 57.3°.)

Thus, if ΔS is selected to be equal to 30 ft (10 m) (for example, one rod length for some maxi-HDD machines) a change of 0.1 rad (6°) corresponds to a radius of curvature of 300 ft (100 m).

7.4 *Bore Paths Profile (Vertical Plane) Trajectory* (**1,2**)—A typical obstacle crossing, such as that represented by a river is illustrated in Fig. 1.

7.4.1 The following parameters must be specified in defining the bore path:

7.4.1.1 Bore entry (pipe exit) point,

7.4.1.2 Bore exit (pipe entry) point,

7.4.1.3 Bore entry (pipe exit) angle,

7.4.1.4 Bore exit (pipe entry) angle,

7.4.1.5 Depth of path, (for example, depth of cover of pipe beneath river bottom), and

7.4.1.6 Path curvatures.

7.4.2 *Bore Entry (Pipe Exit)*—The bore entry point must be accurately specified consistent with the pipe route, equipment requirements, and preliminary topographical investigations. Bore entry angles should be in the range of 8 to 20° (0.15 to 0.35 rad) from the ground surface, preferably 12 to 15° (0.20 to 0.25 rad) from the ground surface. These angles are compatible with typical equipment capabilities.

7.4.3 *Bore Exit (Pipe Entry)*—The bore exit point must also be accurately specified consistent with the pipe length and topographical investigations. Bore exit angles should be relatively shallow, preferably less than 10° (0.15 rad). A shallow angle will facilitate the insertion of the pipe into the bore hole while maintaining the minimum radius of curvature requirements. Relatively steep angles will require greater elevation of the pipe to maintain the required bend radii.

7.4.4 *Path Profile*—The proposed path should optimally lay within a vertical plane including the bore entry and exit points. The arcs of the bore path and straight sections (that is, after achieving desired depth) must be defined, including the radii of curvature and approximate points of tangency of curved and straight segments. The curvatures must be compatible with both the steel drill rods (Eq 1) and the PE pipe or conduit (Section 8). It should be noted that even larger bend radii (lower curvatures) will further reduce lateral flexural bending loads on the pipe and drill rods as they traverse the route, thereby helping avoid additional increases in tensile loads associated with their stiffness effects. Typically, the path should ensure a minimum depth of cover of 15 ft (5 m) beneath the river bottom as projected over the design life of the pipe line, including allowance for scouring (**2,4**). This will overcome buoyancy effects and help overcome the tendency for the drill head to rise towards the free surface, thereby complicating the steering operation.

NOTE 5—The Directional Crossing Contractors Associations (DCCA) (**7**) recommends a minimum depth of 20 ft beneath the river bottom.

7.4.4.1 *Average Radius of Curvature*—The average radius of curvature for a path segment (that is, A-B or C-D in Fig. 1) reaching to or from a depth required to pass beneath an obstacle, may be estimated from the bore exit or entry angle, respectively, and the depth of the bore:

$$R_{avg} = \frac{2H}{\theta^2} \tag{3}$$

where:

R_{avg} = average radius of curvature along path segment, ft (m),

θ = bore exit or entry angle to surface, rad, and

H = depth of bore beneath surface, ft (m).

The corresponding horizontal distance required to achieve the depth or rise to the surface may be estimated by:

$$L = \frac{2H}{\theta} \tag{4}$$

where:

L = horizontal transition distance, ft (m).

It must be noted that departures from a uniform radius will result in locally smaller radii.

7.4.4.2 The resultant path will determine the stresses to be exerted upon the pipe during the installation and service life. The product pipe design must therefore be analyzed based upon the final selected path, following the pipe design and selection procedures given in Section 8.

8. Pipe Design and Selection Considerations

8.1 *General Guidelines:*

8.1.1 Maxi-HDD applications typically require detailed analysis of the pipe or conduit in relation to its intended application. Due to the large anticipated pulling loads and potentially high external pressure, a careful analysis of the PE pipe must be performed, subject to the route geometry, to verify or determine an appropriate DR (or pipe wall thickness). The analysis should consider both the installation forces occurring during pull-back and the long-term operational loads.

8.1.2 *PE Pipe*—Pipes made from either high density polyethylene (HDPE) or medium density polyethylene (MDPE) are suited for directional drilling. PE pipe specifications include Specifications D 2447, D 2513, D 3035, and F 714. If such pipe is provided in short segments, the individual units should be joined using a butt-fusion technique in accordance with Practice D 2657. This will allow the inherent strength of the PE pipe to be maintained during the placement process and when subjected to other operational stresses. Small diameter pipe of continuous length may be provided on reels. Table X1.1 gives modulus and strength values for typical pressure-rated HDPE and MDPE resins.

8.1.3 *Cable Conduit Applications*—For cable conduit applications, including electric power and telecommunications, small diameter pipe may be supplied on a continuous reel including internal pull line or the cable itself, as pre-installed by the manufacturer. In addition, the pipe may be provided with the interior surface pre-lubricated. Such features will be in accordance with that specified by the owner or engineer. Requirements for telecommunications applications, including HDPE pipe with various internal surface profiles, including smoothwall or ribbed are specified in TR-NWT-000356.

8.2 *Pipe Loading*:

8.2.1 *Operational and Installation Loads*—The pipe will be subject to loads during its long-term operation and during the installation process. It is the responsibility of the owner (or the owner's contractor or engineer) to determine the design and selection of the pipe to serve the function intended and withstand the operational stresses at the directionally drilled section as well as at other sections along the pipe line. This practice deals primarily with the loads imposed during the directional drilling process and earth and groundwater loads during operation (post-installation).

8.2.2 *Internal (Operational) Pressure Loads*—It is the responsibility of the owner (or owner's contractor or engineer) to determine the nominal diameter and wall thickness appropriate for the intended application. For example, if the pipe will be used for the pressurized flow of liquids or gases, it is necessary to determine the nominal diameter based on flow capacity requirements and the minimum wall thickness (or DR) to withstand the corresponding circumferential stresses on a long term basis. Specification D 2513, D 3035, or F 714 may be used to determine an initial estimate of the corresponding maximum dimension ratio (DR) for PE pipe.

8.2.3 *External (Operational) Hydraulic and Earth Loads*—The pipe will be subjected to hydrostatic external pressure due to the height of water or drilling fluid (or slurry) above the maximum depth of placement relative to the entry or exit point, and earth loads and liveloads due to load transfer through the deformation of the soil around the borehole (**8**). If borehole deformation is minimal (such as in rock) or does not deform the pipe, the only loading applied to the pipe is the hydrostatic external pressure. When earth load does reach the pipe, load reductions from the geostatic stress (arching) may be anticipated. The reductions may be significant when the in situ soil is normally- or over-consolidated. On the other hand, in under-consolidated soils such as river deposits, the earth load on the pipe may equal the prism load (adjusted for buoyancy in the case of a river crossing). The external pressure applied to the pipe equals the total stress, that is, it is the sum of the effective earth pressure, reduced for arching, and the hydrostatic pressure. In some cases, the mud-slurry pressure will offset the earth pressure. As the earth load applied to directional drilled pipe is dependent on the depth of cover, borehole diameter, mud-slurry properties, drilling and back-reaming techniques, and the in situ soil properties, among other things, a geotechnical engineer should be consulted. See X2.2 for a discussion earth load calculations. Liveload pressure can be transmitted to shallow directional drilled pipe. For shallow applications, it is likely that the pipe is subjected to the same liveload and earth pressures as an entrenched pipe.

8.2.3.1 *Net External Pressure*—The net external pressure, P_{net}, is the differential pressure between the inside and outside of the pipe. The external operational load applied to the pipe may be decreased or totally off-set by internal pressure occurring within the pipe. Likewise, the external load may increase with the occurrence of negative pressure (vacuum) inside the pipe. The net external pressure may vary at different times in the life of the pipeline. For instance, during pressurized flow, the net external pressure may be zero but during a shut-down or prior to service, considerable external pressure may be applied. An analysis should be made of all potential external loadings, internal pressurization or vacuum events, and of their duration of occurrence, so that the net external pressure and its duration is determined for each cycle of the pipeline's service life.

8.2.4 *Pipe Resistance to External Loads*—The pipe must be of sufficient thickness (or DR ratio) to withstand the net external pressure without collapsing or deflecting unduly during each cycle of the operational life of the pipeline. (The effects of external hydrosatic loads applied during the installation phase are discussed in 8.2.8.2.)

NOTE 6—Spangler's Iowa Formula is typically not applicable to directional drilled pipes as the mud-slurry (unless cemented) on setting develops only the consistency of a soft clay which will not provide significant side-support for the pipe.

8.2.4.1 *Pipe Deflection (Ovality)*—Deflection reduces the pipe's resistance to external collapse pressure. Earth loads, longitudinal bending (bore path curvature), and buoyancy forces during installation will produce ring deflection in the pipe. Formulas for calculating earth load deflection, buoyancy deflection, and curvature-induced deflection along with permissible deflection limits are given in Appendix X2. When bore path curvature is limited to the guidelines given in Note 7 and the DR is 21 or less, ovality due to longitudinal bending can generally be ignored. Filling the pipe with water during the placement operation will reduce the buoyancy force (see 8.2.6) and greatly eliminate the possible short-term collapse. The effective external pressure would then be equal to that corresponding to the actual external differential pressure due to the head of drilling slurry minus the internal pressure due to that of the water inside the pipe.

8.2.4.2 *Unconstrained Collapse*—The following version of Levy's equation may be used to determine the allowable external pressure for directional drilled pipe:

$$P_{ua} = \frac{2E}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N} \quad (5)$$

where:
P_{ua} = allowable external collapse pressure, psi (kPa),
E = apparent (time-corrected) modulus, psi (kPa), for the grade of material used to manufacture the pipe, and time and temperature of interest,
μ = Poisson's Ratio (long term loading = 0.45, short term loading = 0.35),
DR = dimension ratio (OD/t),
f_o = ovality compensation factor (see Fig. 2), and
N = safety factor, generally 2.0 or higher.

For design, the allowable collapse pressure, P_{ua}, *must equal or exceed the net effective pressure, P_{net}. The modulus of elasticity and Poisson's ratio are a function of the duration of the anticipated load. Modulus values are given in Table X1.1. If the safety factor in Levy's equation is set equal to one, the equation gives the critical collapse (buckling) pressure. Table X1.3 gives the critical collapse pressure for different DR's of HDPE pipe. For design purposes, the critical collapse pressure must be reduced by a safety factor and by ovality compensation to obtain an allowable stress, P_{ua}. When using Table X1.3 for determining pipe's resistance to buckling during pull-back, an additional reduction for tensile stresses is required. In general, the resulting DR value is lower than that determined by the initial selection criteria based upon internal pressure considerations, the lower value must be used as corresponding to a required thicker, stronger pipe.*

8.2.4.3 For a pipe that will be supported by grouting, the allowable external collapse pressure increases (is enhanced) by a factor of approximately 4 (**1**). Accordingly, the allowable pressure obtained from Levy's Equation, Eq 5, can be increased by a factor of 4. However, the enhancement will not apply to unsupported pipe until the grouting is fully effective. A period of 1 week may be conservatively assumed.

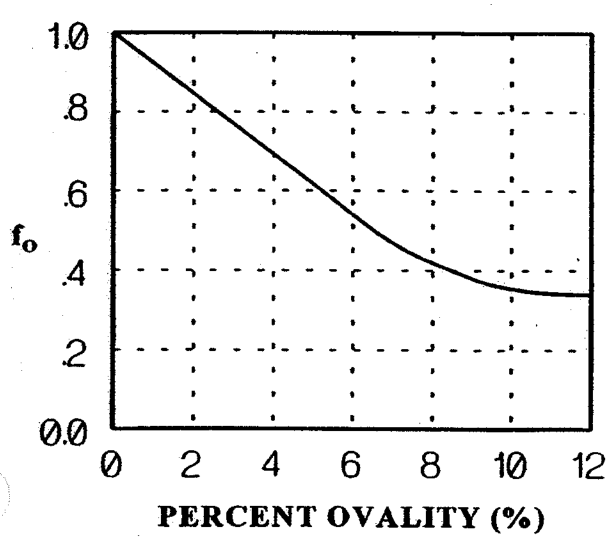


FIG. 2 Ovality Compensation Factor

8.2.5 *Axial Bending Stress*—The radii of curvature for segments of the bored path, as indicated in Fig. 1, must be sufficiently large to ensure minimal bending strains and stresses within the pipe or conduit. The recommended minimum bend radius may be provided by the manufacturer, and corresponds to the following peak axial strain level:

$$\epsilon_a = \frac{D}{2R} \quad (6)$$

where:
ϵ_a = peak axial strain, in./in. (mm/mm),
D = outer diameter of pipe, in. (mm), and
R = local radius of curvature, in. (mm).

The corresponding axial bending stresses may be calculated by:

$$\sigma_a = E_a \epsilon_a \quad (7)$$

where:
σ_a = peak axial stress, psi (kPa),
E_a = apparent modulus of elasticity, psi (kPa) (see Table X1.1).

NOTE 7—Some PE pipe manufacturers recommend an allowable bending radius to diameter ratio of approximately 40 or 50 to 1 during pull-back to minimize the effect of ovaling due to tensile loads.

See X2.5 for calculating ovality induced by bending curvature.

8.2.5.1 *PE Pipe*—In general, the relatively stiff drill rods will require considerably larger bending radii than the flexible PE pipe. The resulting path radii for passing beneath a major obstacle, such as a river, are typically at least an order of magnitude greater than the minimum recommended for the plastic pipe. The corresponding bending strains and stresses are therefore usually not of major significance. However, the curvature required for the pipe to enter or exit the bore hole may be more severe and must be externally controlled to avoid excessive strains or stresses in these areas.

8.2.6 *Pulling Force*—The pipe pullback operation is illustrated in Fig. 1, which shows the geometry of the path including the depth, entry and exit curves, and the possibly straight interim segment beneath the river or obstacle to be crossed. The required tensile force at the leading end of the product pipe will vary during the operation and is, in general, less than that experienced at the drill rig due to the additional load on the balance of the drill string still within the bore hole and that due to any simultaneous reaming operation. The tensile forces on the pipe result from the fractional drag forces acting on the sides of the pipe due to the weight or buoyancy forces as it is pulled into and along the hole, force amplifications due to pulling the pipe around the curves, and resistance due to the pipe stiffness. The resultant forces will depend upon whether the pipe is empty or deliberately weighted (for example, filled with ballast) to reduce the buoyancy. For the purposes of estimating the peak force on the product pipe, the load is calculated at the 4 transition points, A, B, C, D shown in Fig. 1 (**1**). The greatest load on the pipe would typically be at point D. The corresponding loads may be estimated by the following equations:

$$T_A = \exp(v_a\alpha)(v_a w_a (L_1 + L_2 + L_3 + L_4)) \quad (8)$$

$$T_B = \exp(v_b\alpha)(T_A + v_b|w_b|L_2 + w_b H - v_a w_a L_2 \exp(v_a\alpha)) \quad (9)$$

$$T_C = T_B + v_b|w_b|L_3 - \exp(v_b\alpha)(v_a w_a L_3 \exp(v_a\alpha)) \quad (10)$$

$$T_D = \exp(v_b\beta)(T_C + v_b|w_b|L_4 - w_b H - \exp(v_a\alpha)(v_a w_a L_4 \exp(v_a\alpha))) \quad (11)$$

where:

T_A = pull force on pipe at point A, lbf (N),
T_B = pull force on pipe at point B, lbf (N),
T_C = pull force on pipe at point C, lbf (N),
T_D = pull force on pipe at point D, lbf (N),
L_1 = additional length of pipe required for handling and thermal contraction, ft (m),
L_2 = horizontal distance to achieve desired depth, ft (m),
L_3 = additional distance traversed at desired depth, ft (m),
L_4 = horizontal distance to rise to surface, ft (m),
H = depth of bore hole from ground surface, ft (m),
$\exp(X)$ = e^x, where e = natural logarithm base (e = 2.71828),
v_a = coefficient of friction applicable at the surface before the pipe enters bore hole,
v_b = coefficient of friction applicable within the lubricated bore hole or after the (wet) pipe exits,
w_a = weight of empty pipe, lbf/ft (N/m),
w_b = net upward buoyant force on pipe in bore hole, lbf/ft (N/m),
α = bore hole angle at pipe entry (or HDD exit, at side opposite drill rig), rad, and
β = bore hole angle at pipe exit (or HDD entry, at same side as drill rig), rad.

The exponential factors correspond to the capstan effect, reflecting increased bearing pressure caused by the pipe pulled against the inside surface of the bend.

NOTE 8—Although the actual value of L_1 may be considered to be approximately 100 ft (30 m) to allow for handling at both ends of the bore, including possible thermal contraction, it is recommended that a larger value of L_1(for example, 200 to 250 ft (60 to 75 m)) be used in Eq 8 to account for the actual path length along the arc. In some cases, L_3 may be equal to zero.

8.2.6.1 If additional pipe length (to accommodate subsequent elastic, viscoelastic, or thermal contractions) is pulled through the bore hole by using a pulling force applied in a horizontal direction at the drill rig side, resulting in an additional bend of angle β at the surface, there may be a further increase in the pull force T_D. The total force would correspond to that of multiplying the value of T_D, as calculated by Eq 11, by the additional factor $\exp(v_b\beta)$. Furthermore, depending upon the total force magnitude and the local bend radius at this point, the corresponding sidewall bearing pressure at the inside of the bend may cause collapse of the pipe or conduit. This procedure should therefore be avoided in preference to pulling additional pipe length in a direction along the pipe exit (bore entry) angle.

8.2.6.2 *Pipe Stiffness*—The equations in 8.2.6 do not explicitly account for the resistance due to the pipe stiffness at curves along the bore path. This effect will be reduced for sufficiently large radii and greater clearance within the bore hole, but may still represent a significant contribution. Thus, Eq 8-11 and associated calculations should be considered primarily as estimates for the purposes of investigating the overall feasibility of the installation and providing an understanding of the effect of the other parameters. The operational procedures (Section 9) include methods for limiting the actual pulling force applied to the pipe to provide confidence in the integrity of the installed pipeline.

8.2.6.3 *Coefficient of Friction*—The coefficient of friction depends on the characteristics of the surfaces bearing against each other, the presence of any lubrication, and whether there is relative motion between the surfaces. The degree of friction immediately prior to slippage is generally greater than the level during subsequent sliding. Although brief interruptions in the placement process are necessary during the removal of the drill rods during the pullback operation, it is important to attempt to complete the operation without extensive interruptions, which may allow the bore hole to collapse or the pipe to become embedded in the surrounding soil. The value for v_b represents the lubricated value for the pipe in the bore hole as surrounded by drilling fluid and mud slurry assuming minimal interruptions. It is recommended that the pipe external to the bore hole be supported such as to provide as low a coefficient of friction v_a as possible.

NOTE 9—Suggested design values for the frictional coefficients v_a and v_b are 0.5 and 0.3, respectively (1). Where pipe is placed on rollers, v_a is typically considered equal to 0.1.

8.2.6.4 *Multiple Pipes*—If more than one pipe (that is, a bundle of small diameter pipes) is simultaneously pulled into the hole, higher overall loads will result due to the greater weight or buoyancy of the combination as well as an effectively amplified coefficient of friction v_b within the hole. The degree of amplification will depend upon the relative pipe and hole diameters and will be minimized for greater clearance within the borehole.

8.2.6.5 *Effective Weight and Buoyancy Forces*—The weight of the vacant pipe or conduit may be obtained from the manufacturer, or may be calculated by the following formula:

$$w_a = \pi D^2 \frac{(DR-1)}{DR^2} \rho_w \gamma_a \quad (12)$$

where:

w_a = weight of empty pipe, lbf/in. (N/mm),
γ_a = specific gravity of pipe material (for example, 0.955 for PE),
ρ_w = weight density of water times length unit conversion factor, lbf/in.^{3}(N/mm^3), and
D = outside diameter of pipe, in. (mm).

NOTE 10—The density of water is 3.61×10^{-2} lbf/in.^{3} (9.80×10^{-6} N/mm^3).

The net (upward) buoyant force on the vacant pipe surrounded by a drilling fluid or mud slurry may be calculated by:

$$w_b = \frac{\pi D^2}{4} \rho_w \gamma_b - w_a \quad (13)$$

$$w_b = \pi\frac{D^2}{4}\rho_w\left(\gamma_b - \frac{4\gamma_a(DR-1)}{DR^2}\right) \quad (14)$$

where γ_b equals specific gravity of mud slurry.

NOTE 11—The specific gravity of the mud slurry may be conservatively assumed to be 1.5 (see 8.2.3).

If the pipe is filled with water or fluid to serve as ballast, the buoyant force is reduced and is given by either:

$$w_b = \pi\frac{D^2}{4}\rho_w\left(\gamma_b - \gamma_c\left(1 - \frac{2}{DR}\right)^2\right) - w_a \quad (15)$$

$$w_b = \pi\frac{D^2}{4}\rho_w\left(\gamma_b - \gamma_c\left(1 - \frac{2}{DR}\right)^2 - \frac{4\gamma_a(DR-1)}{DR^2}\right) \quad (16)$$

where γ_c equals specific gravity of ballast fluid.

If the pipe is filled with water, then $\gamma_c = 1$; if the pipe is filled with mud slurry (that is, if an open-ended pulling grip is used that allows the drilling fluid or slurry to enter the pipe), then $\gamma_c = \gamma_b$, and the above formula becomes:

$$w_b = \pi D^2\rho_w(\gamma_b - \gamma_a)\frac{(DR-1)}{DR^2} \quad (17)$$

For PE pipe, these procedures will typically result in a lower required pull force as calculated by Eq 8-11.

8.2.6.6 *Hydrokinetic Pressure*—A pressure gradient exists during the pipe pullback operation corresponding to that required to exhaust the drilling fluid out of the hole, towards the pipe entry area. Additional pressure surges are possible due nonuniform pulling rates **(1,2)**. The flow of the drilling fluid along the length of the pipe results in a drag force which may be estimated by considering a balance of the forces acting on the fluid annulus in the bore hole due to the hydrokinetic pressure and the lateral shear forces acting on the pipe and walls of the bore hole:

$$\Delta T = \Delta P\frac{\pi}{8}(D_{hole}^2 - D^2) \quad (18)$$

where:

ΔT = pulling force increment, lbf (N),
ΔP = hydrokinetic pressure, psi (kPa $\times$ 10^{-3}), and
D_{hole} = backreamed hole diameter, in. (mm).

NOTE 12—ΔP is estimated to be 10 psi (70 kPa) **(1,6)**.

The term ΔT may be added to the pulling forces calculated by Eq 8-11 to obtain the total pull force at each corresponding point of the installation. This is shown explicitly in Eq 19.

NOTE 13—For a bundle of pipes, the term D^2 in Eq 18 is replaced by an equivalent sum of the corresponding quantities (diameters squared) for the individual pipes.

8.2.7 *Axial Tensile Stress*—The average axial stress acting on the pipe cross-section at point A, B, C, or D, including the increment for hydrokinetic pressure, is given by:

$$\sigma_i = (T_i + \Delta T)\frac{1}{\pi D^2}\left(\frac{DR^2}{DR-1}\right) \quad (19)$$

where:

T_i = T_A, T_B, T_C, or T_D, lbf (N), and
σ_i = corresponding stress, psi (kPa $\times$ 10^{-3}).

The highest average axial stress will occur at the pulling head. However, depending on the curvature of the borepath, the peak tensile stress may not occur at the pulling head, but in a curve. In the curve, the maximum tensile stress due to bending occurs in the outer fibers of the pipe. For each curve, the maximum tensile stress equals the sum of the bending stress, as in Eq 7, due the curvature and the average axial stress at that point due to pulling. The maximum tensile stress for each curve should be determined and compared with the average axial stress at the pulling head to determine the peak tensile stress, σ_p, occurring in the pipe:

$$\sigma_{pi} = \sigma_i + \sigma_{ai} \quad (20)$$

where:

σ_{pi} = peak tensile stress at i-th point (where i = A, B, C, or D), psi (kPa),
σ_i = average axial tensile pull stress i-th point (where i = A, B, C, or D), psi (kPa), and
σ_{ai} = outerfiber tensile stress (Eq 7) at i-th point (where i = A, B, C, or D), psi (kPa).

8.2.7.1 *Allowable Tensile Stress*—The peak tensile stress, σ_p, should be compared to the allowable stress at the anticipated installation temperature. Thus, it is required that:

$$\sigma p \leq SPS \quad (21)$$

where *SPS* equals safe pull tensile stress, psi (kPa $\times$ 10^{-3}) at the anticipated installation temperature. Under continuous load, polyethylene undergoes creep deformation. Therefore, the safe pull stress values are time and temperature dependent. See Table X1.1 for typical SPS values. The time interval for the installation depends upon the length and rate of pullback of the pipe. Pullback rates are on the order of several feet per minute, depending upon the soil conditions. If it is anticipated that the back-reaming process will be slow and difficult (see Section 9), it is recommended that a separate pre-reaming operation be used to allow a subsequent faster pipe pullback and shorter time interval for installation pull forces to be applied.

8.2.7.2 If necessary, the stress on the PE pipe or conduit may be reduced by increasing the pipe wall thickness (that is, lower SDR value) or, possibly, reducing the net buoyant force by filling the pipe with fluid ballast (as described in 8.2.7.1).

8.2.8 *Torsional Stress*—Torsional stresses are eliminated or minimized by the use of a swivel at the leading end of the pipe. Section 9 provides information for the selection of an appropriate swivel.

8.2.9 *Combined Loads During Installation*—The calculations allow a preliminary selection of the pipe DR consistent with the anticipated application, installation, and path characteristics. It is necessary, however, to finally consider the overall installation stresses due to the combination of loads which many be present simultaneously. If the combined stresses are not within the desired overall design margin, it may be necessary to select a thicker wall pipe or modify the installation parameters to relieve the resultant stresses.

8.2.9.1 *Reduced PE Collapse Strength*—For PE pipe, the presence of an axial tensile load will have a tendency to reduce the pipe's short-term resistance to collapse under external pressure, as otherwise estimated from Eq 5 **(1)**. In addition, the hydrokinetic pressure increment at the leading end of the pipe also increases the external hydrostatic pressure during this riod. The modified equation to account for these effects is:

$$P_{pba} = \frac{2E}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_O f_R}{N} \quad (22)$$

where f_R, the tensile pull reduction factor, is given by:

$$f_R = \sqrt{5.57 - (r+1.09)^2} - 1.09 \quad (23)$$

and

$$r = \frac{\sigma_i}{2(SPS)} \quad (24)$$

σ_i = maximum average axial tensile pull stress from Eq 19, psi (kPa), and
SPS = safe pull tensile stress, psi (kPa).

The allowable collapse pressure, P_{pba}, should equal or exceed the sum of the net effective pressure during pull-back and the hydrokinetic pressure:

$$P_{pba} \geq P_{eff} + \Delta P \quad (25)$$

where:
P_{eff} = net effective pressure acting on pipe during pull-back, psi (kPa), and
ΔP = hydrokinetic pressure, psi (kPa).

NOTE 14—The modulus value used in Eq 22 and in the deflection calculation for determining ovality for use in Eq 22 during pull-back should be selected to match the time-interval of the pull-back.

8.2.9.2 The net effective external pressure term, P_{eff}, in Eq corresponds to the external head of drilling fluid or slurry duced by the internal pressure due to any fluid used as ballast. For the case of an open-ended pulling grip allowing the drilling fluid to serve as ballast (see 8.2.6.5), the net effective external pressure, P_{eff}, including the hydrokinetic pressure, is negligible and the possibility of collapse due to external pressure during the installation stage is essentially eliminated.

8.2.9.3 *Thermal Effects*—Potential effects due to thermal expansion may be minimized by allowing the pipe to reach temperature equilibrium with the soil before cutting the pipe to length to complete the installation.

8.2.10 *Combined Loads During Operation*—In general, it is the responsibility of the owner or owner's contractor or engineer to ensure that the design will be compatible with the long term operation of the pipe line, including sections away from that being placed by the drilling operation, as well as sections in the vicinity of the crossing, both at the surface and passing beneath the obstacle.

8.2.10.1 *Thermal Stress*—Thermal stresses due to temperature differentials existing during the placement process may be considered small, as discussed in 8.2.10. However, possible thermal effects during long-term operation due to seasonal expansion or contraction at the surface, including at sections away from the drilled crossing, are not specific to the HDD process and should be considered by the owner as for non-drilled pipe lines, in combination with the other stress contributions.

Implementation

9.1 Due to the magnitude of the typical operation and complexity of the equipment and control systems, maxi-HDD requires a highly trained crew. See Mini-Horizontal Directional Drilling Manual. It is beyond the scope of this guide to provide operational procedures for the various equipment. Such training is generally provided by the manufacturer. Contractors should be required to demonstrate evidence of proper training for their crews, including classroom and field experience for the primary personnel. The following items represent some of the issues related to the implementation process for placement of pipe or conduit.

9.1.1 *Machine Size & Capability*—The size and capacity of the drilling equipment must be compatible with the thrust and torque required to perform the drilling, reaming, and pipe pullback operations. It is difficult to estimate the drill rig forces associated with the reaming operation, which may be significantly greater than that directly applied to the pipe itself during pullback (as estimated by the formulas in 8.2.4), particularly when both operations are performed simultaneously. The estimated forces applied to the pipe may be considered a minimum equipment requirement.

9.1.2 *Drill Unit Positioning*—The drill rig unit is positioned consistent with the discussion in Section 7 and the desired bore route and pipe depth. Proper anchoring is especially important for soft or sandy soils.

9.1.3 *Boring and Drill Rods*—HDD operations begin with the initial pilot bore. Different ground conditions will require different type drill heads for the pilot bore operation. The drill rods should be as least as strong as the equipment capability. The planned bore route should also be compatible with drill rod capabilities with respect to cumulative fatigue stresses (Section 7). Proper care and handling of the drill rods is important to avoid breakage during boring or backreaming. The rod threads must be cared for and properly coated (greased) when inserted into the drill string. Proper torque should initially be applied to the drill rods as added at the bore entry to avoid potential loosening of the rods and loss of connection in the ground.

9.1.4 *Washover Pipe*—For many maxi-HDD operations, a washover pipe is inserted over the drill string as the bore progresses to support the hole and reduce torque. This steel pipe may be removed during the backreaming operation. If reaming is not required, the washover pipe may be left in place and used as a casing into which a group of small plastic pipes may be placed by a later independent pulling operation.

9.1.5 *Drilling Fluid Usage*—Drilling fluids serve a critical role in maxi-HDD operations. The fluid powers the mud-motor at the front of the drill string that bores the pilot hole. The fluid also provides lubrication during the pilot boring, reaming, and pullback operations to reduce the required torque and thrust or pullback loads. In addition, the drilling fluid stabilizes the bore hole, cools the drill head (and internal circuitry), and removes cuttings and spoils. The crew must be trained in the proper use of drilling fluids and the appropriate types for various ground conditions. Note that excessive drilling fluid pressures or volumes may result in greater disposal problems or appearances at undesired surface locations as the fluid penetrates through fissures.

9.2 *Tracking and Locating*:

9.2.1 *Location Interval*—In order to maintain the actual bore along the planned path, the pilot bore must be carefully cked, and path confirmation established at least once each 30 ft (10 m) interval (for example, when adding drill rods). For paths with horizontal or vertical turns, or in critical areas including the vicinity of other obstacles, shorter intervals for example, 15 ft (5 m) are recommended. In areas with pockets of cobbles or other obstacles that may divert the drill head, measurements should be made whenever contact with such obstacles is suspected. A misdirected drill head must be corrected as soon as possible.

9.2.2 *As-Built Drawings*—A record of the actual as-built bore path, including plan and profile views and vertical and horizontal deviations, indicating the relation to the planned path, must be submitted to the owner. Any information obtained during the initial bore regarding soil characteristics, etc. should be added. The experiences gained during the initial bore may be used to provide guidance for the backreaming operating, as well as for subsequent operations in the project area. Additional information should also be included, such as steering or correction commands, drilling fluid usage, and the type of drill head being used. Regarding the reaming and pullback operations, the pipe insertion velocity, duration, type and size of reamers (cutters or compactors), final bore hole size, drilling fluid usage, and required pullback forces should be recorded.

9.3 *Reaming*—In some maxi-HDD applications, a backreaming operation to increase the hole size may not be required (for example, when a small pipe is to be pulled back into the initial bore hole or, possibly, a bundle of small pipes is to be pulled into the remaining washover pipe by a separate procedure after completion of the HDD operation). However, a backreaming operation is typically performed to produce a hole size sufficiently large to readily install the pipe(s) or conduit. Appropriate cutters and compactors compatible with the soil conditions are required, including proper usage of drilling fluid. In some cases, several reaming (that is, pre-reaming) operations may be required. In general, pre-reaming is not required for placing pipe 20 in. (500 mm) or less in diameter, and the reaming and pipe pullback may be performed simultaneously. The pre-reaming operations allow relatively large holes to be created in stages, reducing the required torque and thrust loads at the machine. For difficult installations for which a high pulling load is anticipated, a pre-reaming operation will help ensure that the capability of the machine is not exceeded due to the combined forces due to increasing the hole diameter and pulling the pipe. The pullback operation may also then be performed at a faster rate, reducing the time the pipe is under axial load. In addition, pre-reaming reduces the possibility of voids or surface heaving or settlement, including unanticipated drilling fluid appearances. Hole diameter increments should be restricted to approximately 10 in. (250 mm) or less during a single pass. The final hole diameter is typically 50 % greater than the outer diameter of the pipe (or pipe bundle) to provide clearance for pipe grips, allow spoils flow, and reduce the required loads during the pipe pullback operation. During pre-reaming, additional drill rods must be available at the pilot bore exit which are connected to a swivel at the rear of the reamer and pulled into the hole to maintain the path.

9.3.1 *Grouting*—If grouting has been specified to fill the annulus of the hole surrounding the pipe(s), it may be pumped during the pullback operation, serving as drilling fluid. However, if the pullback encounters any difficulty, the grout can set-up. Consideration should be given to placing grout through a tremie pipe pulled in during pullback. The requirement and formulation of the grouting shall have been established in advance by the owner and the owner's engineer following the preliminary surface and subsurface studies and route planning, for environmental considerations, or to increase the long-term collapse resistance of the pipe or provide additional strength or mechanical protection. The grouting requires proper formulation consistent with desired set-up time; appropriate fluid pumps are required to handle the thicker fluid mixture. In may cases it may only be required to plug the entry and exit penetration points, possibly using a cement-bentonite mixture (5).

9.4 *Gripping the Pipe*—If not supplied as a continuous length on a reel, it is assumed that the pipe(s) have been fused and tested prior to completion of the boring operation to avoid unnecessary delays in completing the installation. The bored and reamed hole may tend to close in or collapse after an extended period of time, significantly inhibiting or preventing the insertion of the pipe.

9.4.1 Due to the distance of the operation and the relatively high pullback loads generated, secure gripping procedures must be used. Basket-type or internal only grips are not recommended. The gripping method selected must allow essentially the full tensile rating of the pipe to be developed. Appropriate types may include an internal/external clamping or bolting device, or a fused PE pipe adapter with a built-in pulling eye. In the latter case, a smaller diameter section of the adapter may serve as a breakaway link protecting the main section of pipe (see 9.4.3). In general, the end of the pipe should be plugged or sealed to prevent contamination during the pull-back operation. However, if it is desired to allow the mud slurry to serve as ballast (see 8.2.5), a gripping method should be used that allows the fluid to enter the pipe. Several pipes may be pulled simultaneously, but the position of the grips should be staggered, if necessary, to avoid a single large bulge.

9.4.2 *Swivel*—A swivel is required between the reamer or compactor preceding the pipe to prevent the transmission of torsional loads to the pipe. The rating of the swivel should be somewhat larger than the lower of the pull force capability of the drill rig or the total strengths of the bundle of pipes to be installed, but not excessively greater. Inefficiencies in overly large swivels may result in relatively significant twist transmitted to small pipes.

9.4.3 *Breakaway Link*—In general, the recorded pulling forces as indicated at the drill rig will exceed the tensions experienced by the pipe or conduit throughout most of the pullback process. Limiting these loads to that of the allowable pipe strength will generally be overly conservative. It is recommended that individual breakaway links be provided between the main swivel and the grip(s) at the pipe(s), to ensure that the pipelines are installed within allowable load levels. Broken links will require removal of the pipe(s) from entry end, or possibly abandonment. Following a determination of the problem, and an appropriate solution, another attempt may be made, possibly requiring a new bore path.

9.4.3.1 Each breakaway link rating should be within the safe pull tensile load, also called the allowable tensile load of its corresponding pipe. See Table X1.1.

9.4.3.2 Although less desirable, a single breakaway link may be used for a bundle of pipes. The corresponding safe working loads for the individual pipes in the bundle are added to determine the total safe working load and the corresponding rating of the breakaway link. If a breakaway swivel is used as the breakaway link, and not specifically designed for direct exposure with soil, this item should be cleaned well after each application. The use of such a breakaway swivel does not eliminate the need for the main swivel described in 9.4.2.

9.5 *Handling the Pipe*—Extreme care must be exercised when handling the pipe to ensure that it is not subject to excessively sharp bends which may cause a kink or other damage to the pipe. Section 8 provides appropriate guidelines, including discussion of the combined effects of bending loads and tension in the pipe. Particular areas of concern typically include the pipe entry or exit points. It is important to minimize bending of the pipe as it enters the bore hole, consistent with 7.3, 7.4.4 and 8.2.7, and to ensure low friction on the portion of the pipe outside the hole. This may be accomplished by the use of appropriate lifting equipment and roller stands to reduce friction. Due to the potentially high tensile load at the pipe exit, it is especially important to avoid sharp bends at this point.

10. Inspection and Site Cleanup

10.1 *Completion and Inspection*—It is necessary to minimize any residual stresses or strains remaining in the pipe following the installation, due to the imposed pulling forces and potential thermal expansion or contraction. Thus, the pipe should be allowed to achieve mechanical and thermal equilibrium with its surroundings prior to cutting the pipe at either end. Premature cutting of the pipe may allow the ends to shrink back into the hole. The pipe may be cut after it has been verified that there has been insignificant movement at the pipe entry end and negligible residual tensile load at the drill rig end. If any fluid or slurry was allowed to enter the pipe to serve as ballast (see 8.2.6), the fluid must be purged and the pipe thoroughly flushed and cleaned.

10.1.1 *Integrity*—Some pipes, such as for gas or fluid transport, may be required to pass hydrostatic pressure or leakage tests, before or after pullback, or both, as specified by the owner. For pipes to be used as paths for cables, the integrity of the path should be verified by pulling a "pig" through the installed pipe prior to splicing or terminating.

10.1.2 *Visual Inspection*—The pipe exiting the borehole should not show signs of yielding or necking-down. The surface of the pipe should be inspected for gouges or scratches. Gouges or scratches in excess of 10 % of the minimum wall thickness should be assessed as to whether pipe is suitable or not for pressure service.

10.1.3 *Bore Path*—The as-built drawings shall be submitted to the owner's representative to indicate the pipe was placed at the proper location and depth, or within acceptable limits. Maintaining an appropriate minimum depth of cover beneath the river bottom is critical, including margin to account for scouring, to avoid subsequent exposure or damage. Recording of the exact location will help avoid damage during any future construction activities in the area. In addition, records of pullback forces at the drill rig, breakaway link ratings, installation rate, final hole diameter, grouting information, etc., should be recorded and provided.

10.2 *Cleanup*—After inspection and approval by the owner or representative, the surface area must be restored to its original condition. The site must be cleaned of equipment, tools, and spoils. All drilling fluid must be cleaned from the site or its vicinity and properly disposed of, consistent with Section 6.

APPENDIXES

(Nonmandatory Information)

X1. MATERIAL PROPERTIES OF POLYETHYLENE

X1.1 *Material Properties of Polyethylene*—Typical values for the apparent modulus of elasticity and tensile strength at 73°F (23°C) for medium density (PE 2406) and high density polyethylene (PE3408) resins are presented in Table X1.1.

TABLE X1.1 Apparent Modulus of Elasticity and Safe Pull Tensile Stress at 73°F

Typical Apparent Modulus of Elasticity			Typical Safe Pull Stress		
Duration	HDPE	MDPE	Duration	HDPE	MDPE
Short-term	110 000 psi (800 MPa)	87 000 psi (600 MPa)	30 min	1300 psi (9.0 MPa)	1000 psi (6.9 MPa)
10 h	57 500 psi (400 MPa)	43 500 psi (300 MPa)	60 min	1200 psi (8.3 MPa)	900 psi (6.2 MPa)
100 h	51 200 psi (350 MPa)	36 200 psi (250 MPa)	12 h	1150 psi (7.9 MPa)	850 psi (5.9 MPa)
50 years	28 200 psi (200 MPa)	21 700 psi (150 MPa)	24 h	1100 psi (7.6 MPa)	800 psi (5.5 MPa)

onsult the manufacturer for specific applications.

X2. POST–INSTALLATION LOADS AND DEFLECTION OF HORIZONTAL DIRECTIONAL DRILLED PIPES

X2.1 Allowable Tensile Load—The safe pull tensile load for a pipe is equal to its allowable tensile load ATL, which can be calculated from the safe pull tensile stress SPS, as follows:

$$ATL = (SPS)\,\pi\,D^2\left(\frac{1}{DR} - \frac{1}{DR^2}\right) \qquad \text{(X2.1)}$$

where:

D = pipe outer diameter, in. (mm),
SPS = safe pull stress, psi (kPa), and
DR = pipe dimension ratio (outer diameter/minimum wall thickness).

For gas pipes, see Practice F 1804 for determining ATL.

X2.2 *Earth Pressure Calculation*—The soil load on directional drilled pipe is essentially dependent on the depth of cover, borehole diameter, mud-slurry properties, and the in situ properties. Earth and live-load pressures are transferred to the pipe through the deformation of the soil around the borehole. As the deformation occurs, a cavity of loosened soil forms above the borehole. This cavity is filled by soil sloughing from above it. The process causes the soil to bulk, that is, the density of the sloughed soil is less than the density of the undisturbed soil. The sloughing process continues until an equilibrium is reached where the stiffness of the sloughed soil is sufficient to resist further sloughing from the soil above. This bulking state results in arching of load around the pipe (that is, the earth load applied to the pipe is less than the geostatic stress (or prism load).) There is a lack of published equations for calculating earth loads on directional pipes. However, equations have been published for calculating loads on jacked pipe. Although the applicability of these equations to directional drilling has not been confirmed, they are likely applicable where the PE is installed in a mud slurry. The normal jacking procedure like the directional drilled process overcuts the hole but the overcut is typically less than 10 % of the pipe diameter with jacked pipe, whereas with directional drilled pipes the overcut may be 50 %. Equations for calculating the loads occurring on jacked-pipe due to the bulking process are given by O'Rourke et al. Another interpretation of arching above jacked-pipe is given in **(10)**. Stein's method in Ref. **(10)** considers the process of arching to be similar to trench arching. Only Stein's method is given below as O'Rourke's method in Ref. **(9)** involves extensive calculations and typically results in lesser load than Stein's method. Credit for arching should only be considered where the depth of cover is sufficient to develop arching (typically exceeding five pipe diameters), dynamic loads such as traffic or rail loads are insignificant, the soil has sufficient internal friction to transmit arching, as confirmed by a geotechnical engineer.

X2.2.1 Use of Terzaghi's equation as given in Eq X2.2 for calculating earth loads on jacked pipe is suggested in Ref. **(10)**. Note that the friction angle, has been reduced in Terzaghi's equation by 50 %.

$$P_{EV} = \frac{\kappa \gamma H}{144\frac{in.^2}{ft^2}} \qquad \text{(X2.2)}$$

$$\kappa = \frac{1 - exp\left(-2\frac{KH}{B}\tan\left(\frac{\delta}{2}\right)\right)}{2\frac{KH}{B}\tan\left(\frac{\delta}{2}\right)} \qquad \text{(X2.3)}$$

[For metric units, the conversion factor of 144 in²/ft² should be dropped]

where:

P_E = external earth pressure, psi (kPa),
Y = soil weight, pcf (kN/m³),
H = depth of cover, ft (m),
κ = arching factor,
B = "silo" width, ft (m),
δ = angle of wall friction, degrees (for directional drilling, assume $\delta = \phi$, and ϕ = angle of internal friction, degrees.), and
K = earth pressure coefficient given by:

$$K = \tan^2\left(45 - \frac{\phi}{2}\right) \qquad \text{(X2.4)}$$

The silo width must be estimated based on the application. It varies between the pipe diameter and the borehole diameter. A conservative approach is to assume the silo width equals the borehole diameter. (If the effective soil weight is used the groundwater pressure must be added back into Eq X2.2 to get the total external pressure acting on the pipe. The effective soil weight is the dry unit weight of the soil for soil above the groundwater level; it is the saturated unit weight less the weight of water for soil below the groundwater level.)

X2.3 *Earth Load Deflection*—Earth load is generally applied at the pipe crown with a reaction at the invert. As slurry provides essentially no side-support, there is little pressure at the springline to restrain vertical deflection. The primary resistance to deflection is provided by the pipe's stiffness. Whereas, actual soil loads will occur over a good portion of the top and bottom halves of the pipe, Ref. **(11)** gives two ring deflection formulas for uniform loading on the top half of a pipe in the Appendix of the text. One formula assumes the pipe's invert is supported on a rigid, flat base while the other assumes the invert reaction load is uniform around the bottom half of the pipe. Neither case fits exactly what occurs with directional drilled pipe but the average of the two formulas may come close.

$$\frac{\Delta}{D} = \frac{0.0125 P_E}{\frac{E}{12\,(DR-1)^3}} \qquad \text{(X2.5)}$$

where:

= pipe diameter, in. (mm),
Δ = ring deformation, in. (mm),
P_E = earth pressure, psi (kPa),
DR = pipe dimension ratio, and
E = modulus of elasticity, psi (kPa).

X2.4 *Buoyant Deflection*—An external pressure difference between crown and invert occurs when pipe is submerged in grout due to the difference in grout head pressure across the pipe. The pressure difference applies a force which deflects the invert upward toward the crown, thus creating ovality. Deflection is given by Eq X2.6. This can be converted to percent deflection by multiplying it by 100.

$$\frac{\Delta}{D} = \frac{0.1169\gamma_w\left(\frac{D}{2}\right)^4}{EI} \quad \text{(X2.6)}$$

where:
Δ = ring deflection, in. (m),
D = pipe diameter, in. (m),
Y_w = weight of fluid in borehole, lbs/in.^{3} (to convert fluid weight from lbs/ft^3 to lbs/in^3 divide by 1728) (kN/m^3),
E = modulus of elasticity, psi (kPa), and
I = moment of inertia of pipe wall cross-section ($t^3/12$), in.^{4}/in. (m^4/m).

X2.5 *Reissner Effect*—Longitudinal bending of a pipe induces ovality. For entrenched pipes this ovality is usually ignored as it is oriented transverse to earth load deflection. In a directional drilled pipe ovality is additive to earth load deflection. For DR 21 or lower pipes, when the bending radius is greater than or equal to 40 pipe diameters, the ovality is negligible. Ovality in terms of percent deflection can be calculated from the Reissner equation:

$$\frac{\Delta y}{D} = \left(\frac{2}{3}\right)z + \left(\frac{71}{135}\right)z^2 \quad \text{(X2.7)}$$

$$z = \frac{\frac{3}{2}(1-\mu^2)(D-t)^4}{16t^2R^2} \quad \text{(X2.8)}$$

where:
μ = Poisson's ratio,
D = pipe OD, in. (mm),
t = pipe wall thickness, in. (mm),
R = radius of curvature, in. (mm), and
$\Delta y/D$ = deflection, in./in. (mm/mm) (convert to percent by multiplying by 100).

X2.6 *Deflection Limits*—The limiting deflection (in percent) is determined by the geometric stability of the deflected pipe, hydraulic capacity, and the strain occurring in the pipe wall. It has been observed that for PE, pressure-rated pipe, subjected to soil pressure only, no upper limit from a practical design point of view seems to exist for the bending strain **(12)**. Therefore, for non-pressure pipes or conduits the safe long-term deflection is 7.5 % of the diameter. When subjected to internal pressure in addition to soil pressure, the localized bending strain resulting from deflection combines with the hoop tensile strain caused by internal pressure to produce a higher, localized tensile fiber-stress. However, as the internal pressure is increased the pipe re-rounds and the bending strain is reduced. At high pressures, the bending strain is reduced and the ring tensile stress approaches that due to internal pressure alone. For calculation method, see Ref. **(13)**. This fact coupled with the ductility of PE permits the designer to ignore the combined effect of pressure and deflection. In lieu of an exact calculation based on allowable strain, the designer can use the safe long-term design deflection values for pressure pipe shown to Table X2.1.

X2.6.1 Design deflections are for use in selecting DR and for field quality control. Field measured deflections exceeding the design deflection do not necessarily indicate unstable or over-strained pipe. In this case, an engineering analysis of such pipe should be performed before acceptance.

TABLE X2.1 Safe Long-Term Design Deflection values for Buried Pressurized Polyethylene Pipe

DR or SDR	Deflection Limits as % of Diameter
21	7.5
17	6.0
15.5	6.0
13.5	6.0
11	5.0
9	4.0
7.3	3.0

X3. CRITICAL BUCKLING PRESSURE FOR HDPE PIPE

X3.1 *Critical Buckling Pressure*—Table X3.1 gives the critical collapse pressure for HDPE pipes. The values do not contain a safety factor nor any compensation for ovality or pulling force. See 9.2.3.1 for discussion.

TABLE X3.1 Critical Collapse Pressure for Unconstrained HDPE Pipe[A,B,C] at 73°F

NOTE—Table does not include ovality compensation or safety factor.

	Pipe SDR, psi, ft H_2O, in Hg						
Service Life	7.3	9	11	13.5	15.5	17	21
Short-term	1003, 2316, 2045	490, 1131, 999	251, 579, 512	128, 297, 262	82, 190, 168	61, 141, 125	31, 72, 64
100 h	488, 1126, 995	238, 550, 486	122, 282, 249	62, 144, 127	40, 92, 82	30, 69, 61	15, 35, 31
50 years	283, 653, 577	138, 319, 282	71, 163, 144	36, 84, 74	23, 54, 47	17, 40, 35	9, 20, 18

[A] Axial Tension during pull-back reduces collapse strength.
[B] Full vacuum is 14.7 psi, 34 ft water, 30 in Hg.
[C] Multipliers for temperature rerating:

60°F (16°C)	73.4°F (23°C)	100°F (38°C)	120°F (49°C)
1.08	1.00	0.78	0.63

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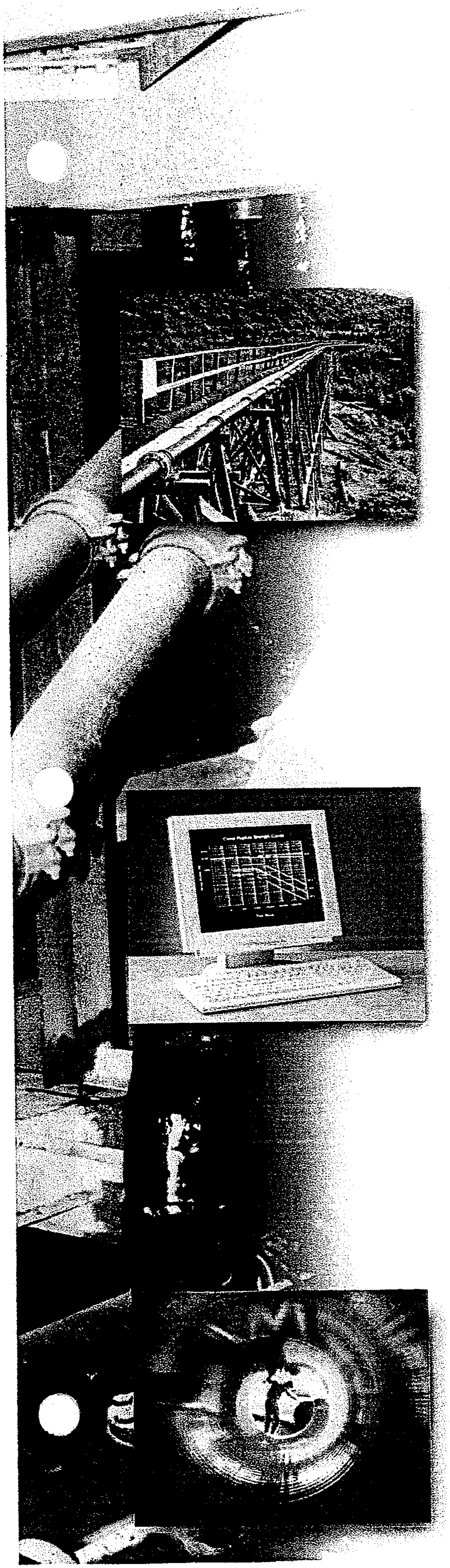

high density polyethylene pipe

Sclairpipe®

Systems Design

KWH PIPE

Earthloading - Design of Underground Piping Systems

INTRODUCTION

This section defines the performance limits for SCLAIRPIPE polyethylene pipe in the following three burial environments - in varying soils and soil compaction levels, in firm soils where the buried pipe is subjected to external hydrostatic pressure and in firm and loose soils with the buried pipe subjected to internal vacuum or net external hydrostatic pressure. In all cases, the pipe is considered to be empty with no resistance to deflection contributed by internal pressure.

Flexible conduits react to earthloads or external hydrostatic loads very differently than rigid pipes do. The natural ring stiffness of the flexible pipe contributes only a small portion of the total resistance to deflection; most of the resistance arises from the soil stiffness. When the buried pipe deflects slightly in the vertical axis, the accompanying outward movement of the pipe side walls mobilizes the support available due to the stiffness of the surrounding soil envelope. Figure 2 provides an illustration of this mobilization process. The pipe is supported against further movement and exhibits load-bearing capabilities far greater than unsupported pipe. The amount of support which is available in the embedment soil is a direct consequence of the installation procedure. The stiffer the embedment materials are; the less deflection occurs and the more stable the pipe-soil system is.

DESIGN CRITERIA

When selecting the most appropriate wall thickness or DR for Sclairpipe to resist anticipated burial conditions or when confirming the adequacy of a selection which was made based on pressure class requirements three design criterion are considered separately; vertical deflection, wall buckling and wall compression or crushing. The amount of deflection which can be expected under specific burial conditions may be estimated using the form of the Iowa pipe deflection formula presented below. The estimated vertical deflection as a percentage of the mean pipe diameter is then compared to the safe design limits presented in Table 1. In order to verify the adequacy of the pipe-soil system against wall buckling or collapse the *safe allowable buckling load (q_a)* is determined using the equation presented and compared to the anticipated applied loads. Compressive stress in the pipe walls may also be estimated and compared to the safe compressive strength of HDPE which is conservatively estimated as 800 psi.

DEFLECTION

$$\Delta y = \frac{(D_l W_c + W_L) K_x r^3}{EI + 0.061E'r^3} \qquad (1.0)$$

Where;
- Δy = predicted vertical pipe deflection in inches.
- D_l = the deflection lag factor to compensate for the time-consolidation rate of the soil, dimensionless. Normally estimated as 1.5.
- W_c = vertical soil load on the pipe per unit length, in pounds per linear inch. W_c is estimated by multiplying the appropriate value from Table 2 by the outside diameter (in inches) of the pipe.
- W_L = live load on the pipe per unit length, in pounds per linear inch. W_L is estimated by multiplying the appropriate value from Figure 3 by the outside diameter (in inches) of the pipe.
- K_x = deflection coefficient, dimensionless, Use 0.083 for most installations.
- r = mean pipe radius in inches.
- r = $(O.D. - t_{min})/2$
- t_{min} = minimum wall thickness of pipe in inches.
- E = Apparent modulus of elasticity of the pipe material in psi. A long-term apparent modulus of 30,000 psi may be used in most situations.
- I = the moment of inertia of the pipe wall for ring bending in inches^4/inch.
- I = $t_{min}^3/12$
- E' = modulus of soil reaction, in psi. The appropriate value for E' should be selection from Table 3.

Table 1
SAFE DESIGN LIMITS

Dimension Ratio	Allowable Vertical Ring Deflection as a % of Diameter
32.5	8.6
26	6.5
21	5.0
17	4.0
11	3.3
9	2.6

WALL BUCKLING

The safe allowable buckling load for the soil-pipe structure (q_a) is estimated as follows;

$$q_a = (DF)\,(32\,R_w\,B'\,E'\,EI/D_{avg}^3)^{0.5} \qquad (2.0)$$

Where:
- q_a = safe allowable buckling load in psi.
- DF = design factor, 0.40
- R_w = water buoyancy factor, calculated as follows;
- R_w = $1 - 0.33(h_w/h)$; $0 \leq h_w \leq h$

Where:
- h_w = height of ground water surface above top of pipe in inches.
- h = height of ground surface above top of pipe in inches.

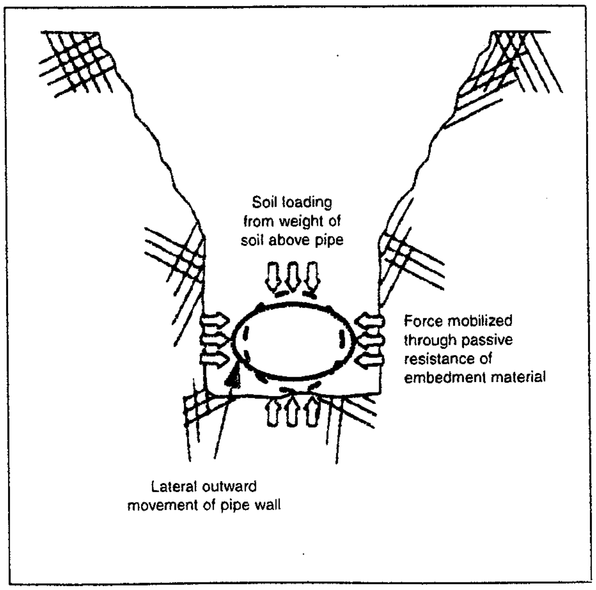

Figure 2: Mobilization of Enveloping Soil through Pipe Deformation

Table 2

Vertical Soil Load in lbs./in²				
Depth to Top of Pipe in ft.	Soil Density 90 lbs.ft³	Soil Density 100 lbs.ft³	Soil Density 110 lbs.ft³	Soil Density 120 lbs.ft³
1	0.6	0.7	0.8	.8
2	1.3	1.4	1.5	1.7
3	1.9	2.1	2.3	2.5
4	2.5	2.8	3.1	3.3
5	3.1	3.5	3.8	4.2
6	3.8	4.2	4.6	5.0
7	4.4	4.9	5.3	5.8
8	5.0	5.6	6.1	6.7
9	5.6	6.3	6.9	7.5
10	6.3	6.9	7.6	8.3
12	7.5	8.3	9.2	10.0
14	8.8	9.7	10.7	11.7
16	10.0	11.1	12.2	13.3
18	11.3	12.5	13.8	15.0
20	12.5	13.9	15.3	16.7
25	15.6	17.4	19.1	20.8
30	18.8	20.8	22.9	25.0

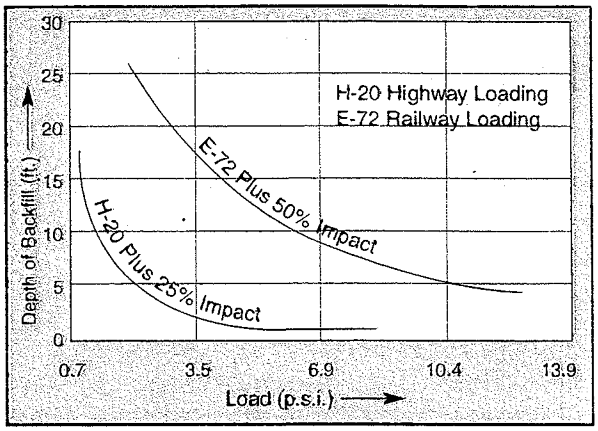

Figure 3: Live loading due to vehicle traffic

Table 3
Embedment Classes per ASTM D-2321

Class	Soil Description	Soil Group Symbol	Average Value of E′ — Degree of Compaction of Embedment Material[1] (Standard Proctor): Dumped	Slight 85%	Moderate 90%	Heavy > 95%
IA	Manufactured aggregate angular open-graded and clean. Includes crushed stone, crushed shells.	None	500	1000	3000	3000
IB	Processed aggregate, angular dense-graded and clean. Includes Class 1A material mixed with sand and gravel to minimize migration.	None	200	1000	2000	3000
II	Coarse-grained soils, clean. Includes gravels, gravel-sand mixtures, and well and poorly graded sands. Contains little to no fines (less than 5% passing #200).	GW, GP, SW, SP	200	1000	2000	3000
II	Coarse-grained soils, borderline clean to "with fines". Contains 5% to 12% fines (passing #200).	GW-GC, SP-SM	200	1000	2000	3000
III	Coarse-grained soils containing 12% to 50% fines. Includes clayey gravel, silty sands, and clayey sands.	GM, GC, SM, SC	100	200	1000	2000
IVa	Fine-grained soils (inorganic). Includes inorganic silts, rock flour, silty-fine sands, clays of low to medium plasticity, and silty or sandy clays.	ML, CL	50	200	400	1000
IVb	Fine-grained soils (inorganic). Includes diatomaceous soils, elastic silts, fat clays.	MH, CH	No date available; consult a competent soils engineer. Otherwise use E′ equals zero.			
V	Organic soils. Includes organic silts or clays and peat.	OL, OH, PT	No date available; consult a competent soils engineer. Otherwise use E′ equals zero.			

[1] E′ values taken from Bureau of Reclamation table of average values and modified slightly herein to make the values more conservative.

B′ = empirical coefficient of elastic support, dimensionless. Calculated as follows;

$B' = (1 + 4e^{-0.065H})^{-1}$

Where: H = burial depth to the top of the pipe in ft.

D_{avg} = mean pipe diameter (O.D. - t_{min})

For most pipe installations satisfaction of the wall buckling requirement is assured when the following equation is true;

$$\gamma_w h_w + R_w(W_c/D_{avg}) + P_v \leq q_a \qquad (2.1)$$

Where; γ_w = specified weight of water (that is, 0.0361 lbs./in.³) in pounds per cubic inch.

P_v = internal vacuum pressure (that is, atmospheric pressure less the absolute pressure inside of the pipe), in pounds per square inch.

In some situations, consideration of live loads in addition to dead loads may be appropriate. However, simultaneous application of the live-load and internal vacuum transients need not normally be considered. When live loads are being considered, the buckling requirement is assured when the following equation is true;

$$\gamma_w h_w + R_w(W_c/D_{avg}) + W_l/D_{avg} \leq q_a \qquad (2.2)$$

COMPRESSION

The compressive stress which will exist in the pipe wall due to anticipated burial loads (σ_c) can be estimated using the following equation;

$$\sigma_c = (W_c + W_l) / (2t_{min}) \qquad (3.0)$$

Satisfaction of the wall compression is assured when the following equation is true;

$$\sigma_c \leq 800 \text{ psi} \qquad (3.1)$$

CRITICAL PRESSURE FOR UNSUPPORTED PIPE:

In locations such as bogs, swamps or underwater, empty polyethylene pipelines can collapse if subjected to an excessive external/internal pressure differential. Such differential pressures may be caused by drawing a vacuum or by simply increasing the external hydraulic loading. Limiting critical pressures have been calculated from the modified Iowa Equation using a modulus (pipe stiffness) equivalent to 50 years of exposure to the critical pressure. Table 4 shows the critical pressure at 73.4°F for various pipe DRs or wall thicknesses. These critical pressures will cause full collapse of a pipe which has no initial deflection and is subjected to no stresses other than the net external pressure. However, damage can result to the pipe through excessive straining before full collapse, necessitating other safety factor considerations. For more information on the selection of pressure rating for unsupported pipe, see the section on Vacuum & External Hydraulic Overpressure.

Table 4
CRITICAL PRESSURES FOR PIPE WITHOUT SUPPORT

Dimension Ratio	Net External Critical Pressure (Pcr) (psi)
32.2	1.0
26	1.9
21	3.6
17	6.8
15.5	8.9
13.5	13.5
11	25.0
9	45.7

CRITICAL PRESSURE FOR SOIL-SUPPORTED PIPE:

Experimental work has shown that soil-supported pipe has a much greater capacity to withstand vacuum or net external pressures than pipe without support. This is particularly important when evaluating the effect of negative hydraulic transient pressures that may arise in pressure lines with sudden valve closures or pump failures. Treatment of this problem should be referred to your nearest KWH Pipe office.

Bedding Limitations:

- Always level the trench bottom, taking care to remove all sharp rocks and/or protrusions within 6 inches of the pipe.
- Ensure that the bedding material is worked into uniform contact with the pipe at the haunches.
- When bedding soil is non-compactible by its own weight, use mechanical compactions - DO NOT MECHANICALLY COMPACT DIRECTLY ON TOP OF THE PIPE - PLACE ONE FOOT OF BEDDING BEFORE COMPACTING DIRECTLY OVER THE PIPE.
- Do not allow rocks or frozen clods within a one foot bedding "envelope" around the pipe.
- See the Construction brochure for further details and burial information.

SAMPLE PROBLEM:

Problem

A 48" DR32.5 sewer pipe is to be buried with a depth of cover of 10 feet to the top of the pipe and must withstand H-20 truck traffic.

If the pipe is above the groundwater table and embedded in Type IB material (γ_s = 110 lbs/ft.) compacted to 85% Standard Proctor Density, is the pipe selection adequate?

Solution

Part 1 Deflection;

W_C = 7.6 x 48 = 364.8 lbs/in.
W_L = 1.4 x 48 = 67.2 lbs/in.
r = (48 - 1.453)/2 = 23.274 in.
I = 1.453^3 / 12 = 0.256 in.
E' = 1,000 psi

$$\therefore y = \frac{(1.5 \times 364.8 + 67.2)\ 0.083 \times 23.274^3}{30{,}000 \times 0.256 + 0.061 \times 1{,}000 \times 23.274^3} \quad (1.0)$$

= 0.828 in.
= 1.78 % of the mean pipe diameter

∴ Pipe selection is adequate for deflection criteria

Part 2 Wall Buckling;

h_w = 0.00 in.
R_w = 1.00
B' = $(1 + 4e^{-0.065 \times 10})^{-1}$
= 0.324
D_{avg} = 48 - 1.453
= 46.547 in.
$\therefore q_a = (1/2.5)(32 \times 0.324 \times 1{,}000 \times 30{,}000 \times 0.256/46.547^3)^{0.5}$ (2.0)
= 11.24 psi

Now check:

$0.0361 \times 0.00 + 1.00 \times 364.8/46.547 + 67.2/46.547 \le q_a$ (2.2)
9.281 psi ≤ 11.24 psi
∴ Pipe selection is adequate for buckling criteria

Part 2 Wall Compression; (3.0)

σ_c = (364.8 + 67.2) / (2x1.453)
= 148.658 psi
$\sigma_c \le$ 800 psi (3.1)
∴ Pipe selection is adequate for wall crushing criteria

Since the selected pipe meets the requirements of all three of the design criteria the pipe selection is structurally adequate.

HDPE LEACHATE COLLECTION PIPE DESIGN BY FUNDAMENTALS OF MECHANICS

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Abstract:

Two methods for design of HDPE leachate collection pipes (LCP) are discussed and contrasted. The Modified Iowa Formula is examined and its shortcomings as a tool for flexible pipe design are illustrated. A design approach widely used in the steel pipe industry and based primarily on the ring compression formula and the strength of the pipe walls is presented and shown to be more applicable to LCP design.

Introduction:

Modern buried pipe design started in 1913 with Anson Marston, Dean of Engineering at Iowa State College. Iowa's muddy roads were a problem. Marston reasoned, correctly, that any remedy must include buried drain pipes. For design of the pipes he proposed the Marston theory of earth loads on buried pipes. A pipe would have to support the weight of backfill soil in the trench above the pipe, reduced by friction on the trench walls. A student, M.G. Spangler, was assigned the task of testing Marston's load theory in the laboratory. He discovered that soil loads could be modeled by the Marston load only with modifications because the sidefill soil, next to the pipe, supports part of the load. Spangler's work to quantify this support later led to the use of the "prism" load which is manifested by the backfill material above the pipe only (not the full trench width). At the time when Spangler began his work however, drain pipes were rigid pipes of concrete and clay tile and for design the pipe had to be strong enough, in three edge bearing, to support the Marston load.

Then, however, Armco began marketing corrugated steel drain pipes. The flexibility of these pipes required a different theory. Spangler noted that a flexible pipe depends upon the side fill soil to support the pipe arch against spreading and to support much of the Marston load. He developed a more relevant pipe-soil interaction theory of analysis and focused on the new performance limit of ring deflection that flexible pipe brought with it. Spangler derived the Iowa formula to predict ring deflection and published it in 1941. The original formula was flawed, however, and Spangler, by now a professor at Iowa State, asked his student R.K. Watkins to check the derivation. Watkins found that the soil modulus did not have the correct units. The Modified Iowa Formula was published in 1958[1]. It has been widely, but not exclusively, used ever since. It should be noted here that Spangler

Presented at the Nineteenth International Madison Waste Conference, September 25-26, 1996, Department of Engineering Professional Development, University of Wisconsin-Madison.

never intended that his equation be used as a primary design tool. The Iowa formula only predicts ring deflection. Unfortunately, it is based on some difficult assumptions and judgments.

In 1960 Howard White[2] of Armco, published a design approach based on ring compression and the pipe's wall crushing strength. This approach has been in widespread use by Armco and others ever since. Watkins, meanwhile, attempted to measure the modulus of soil reaction in the laboratory. He discovered that the value of this variable varies with depth of cover and is not just a property of the soil. He also found that the Modified Iowa Formula overestimates ring deflection for flexible pipe in stiff embedment. Watkins concluded that the ring compression formula is the correct fundamental design approach and should be used while also checking for other possible performance limits.

Since the early 1960s, two approaches to the design of buried flexible pipe have been in common use. The landfill industry has focused on relatively flexible HDPE pipe and the Modified Iowa Formula. HDPE pipe design guides generally include the ring compression formula as a check but imply that ring ***deflection***, calculated using the Modified Iowa Formula, should govern. It is not entirely clear why this is so; perhaps because it appears more conservative. The following discussion of the formulae and issues involved explains why the Modified Iowa Formula should not be used to design HDPE leachate collection pipes.

The Modified Iowa Formula

The Modified Iowa Formula is as follows

$$\Delta X = \frac{D_L \; K \; W_C \; r^3}{EI + (0.061 \; E' r^3}$$

Where

ΔX is the *horizontal* deflection of the pipe wall, at the spring line, on one side of the pipe, in inches. Usually assumed equal to vertical deflection (ΔY).

D_L is the deflection lag factor. This was introduced by Marston and generally accepted by Spangler as part of their load calculations. Though Marston used trench width and Spangler used pipe width to define the potential load on the pipe, both assumed that the subject column or "prism" of backfill material would not immediately follow a deflected pipe downward because friction between this prism and the adjacent soils would resist such movement and partially support the prism. They started with a load less than the weight of the subject backfill and then, in order to account for the fact that this friction based support would partially deteriorate with time, Spangler applied this deflection lag factor.

He effectively put back some of the load he had taken out when accounting for friction on the prism side wall. If a designer simply uses the weight of the entire prism as the load in the first place then he may use a value of 1 for the deflection lag factor and consider his load estimate to be conservative (unless down-drag by the adjacent soils, on the prism, will occur - this should not be the case with LCPs).

K is the pipe bedding factor. It accounts for the completeness and quality of the pipe embedment in the haunches. The values assigned to this variable only vary from approximately 0.083 for a condition where the pipe is carefully bedded with good embedment material from springline to springline, to 0.11 if a pipe is to be placed on a hard flat surface with no effort made to fill the haunches properly. Its published values were arrived at by empirical methods. A value of 0.10 is usually assigned.

W_C is the prism load, or the weight of the soil prism minus the side wall friction support per unit length of pipe, in lbs/inch. If the deflection lag factor is set equal to one then the weight of the prism without any correction can be considered to be conservative.

r is the radius of the pipe, inches.

E is the modulus of elasticity of the pipe wall, lbs/in^2.

I is the moment of inertia of the pipe wall, in^4/in. E times I expresses the stiffness of the pipe.

E' is the modulus of soil reaction, lbs/in^2. It expresses the ability of the sidefill soil to resist horizontal movement of the pipe into the sidefill. Spangler first attempted to estimate this value for various soil types but could not measure it directly. He then used limited field data from relatively shallow pipe burials and relatively stiff pipes and back-calculated to find values for his original soil modulus. When Watkins replaced this modulus with the dimensionally correct E', Spangler's values were adjusted accordingly. The table of E' values, subsequently published in the late 1950's, are the values typically employed in the use of this formula whenever it is applied. It is not possible to have a soils lab find E' for a specific embedment material. There is no ASTM procedure for it. Confusion sometimes occurs in regard to this issue because there is another, similar, definition normally applied to the term E'. The normal E' is simply the modulus of elasticity which is the ratio of stress to strain . This normal E' reflects elastic, not passive, behavior. The E' in the Modified Iowa Formula is about passive soil resistance at soil slip. It was conceived in response to the belief that the pipe side walls would be motivated, by a concentrated load at the top of the pipe,

> to push a significant distance into the sidefill soil and that the sidefill soils would be under a lesser confining pressure. Under deep burial conditions no such concentrated forces exist and the modulus of soil reaction E' becomes inconsequential.

Spangler modeled the system as three moving parts: The soil prism load, sitting on top of the pipe; the pipe that deflects under the load; and the sidefill which he treated as a spring that resists horizontal deflection of the pipe. This model is not representative of conditions to which an LCP is exposed for the following reasons:

- The modulus of soil reaction, E', is employed as a spring constant whereas the soil is not an elastic material (not spring-like).
- After deep burial, such as the first lift of waste, the forces requiring a spring-like reaction from the sidefill soils no longer exist (see discussion of E' above).
- The model does not consider the behavior of the sidefill in the vertical dimension.

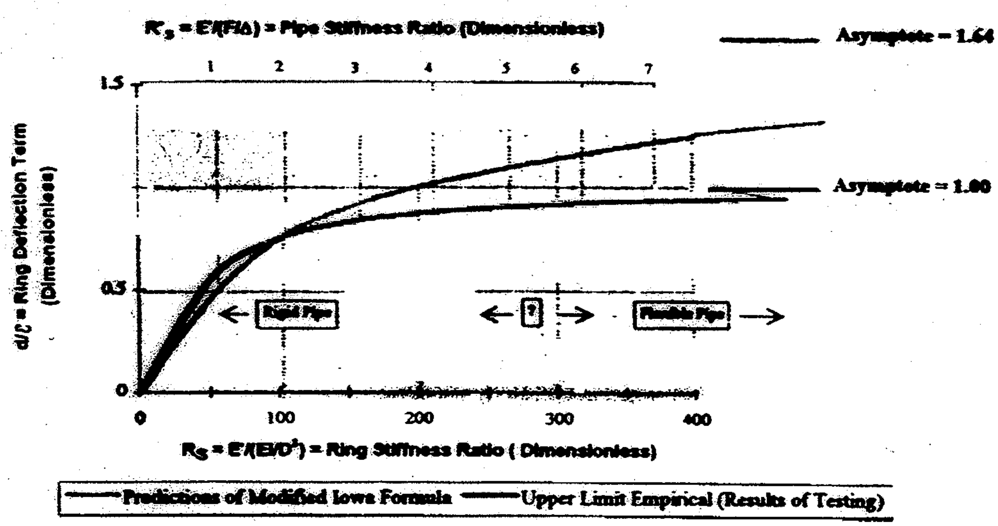
Extensive studies by Watkins at the Utah State University Buried Structures Lab and by others have shown that **the pipe will not deflect downward more than the sidefill soils.** It actually deflects slightly less than the sidefill soils due to the soil arch that forms over the pipe, but assuming that it will deflect equally is an easy and conservative approach. When one understands this relationship, one can predict the deflection of the pipe by measuring the deflection of the embedment material, under the design load, in the lab, or estimate it using the normal soil modulus.

The premise that the pipe will not deflect more than the sidefill soil implies that the prism of soil does not follow the pipe downward because it is supported by a soil arch. Arches, unlike trench wall friction, can be relied upon to provide permanent support. Of course, if the columns on which the arch is resting (the sidefill soils) compress then the arch will move downward. The flexible pipe will offer relatively little resistance and deflect under the arch about as much as the "columns" compress.

If however, a stiff pipe is installed in a more compressible embedment, it may deflect considerably less than the sidefill soils, punch through the soil arch from below and end up carrying the full prism load by itself. In this case, the sidefill soils do little more than support the sides of the pipe against outward movement, and the model on which the Iowa Formula is based is valid. Assigning a value to the modulus of soil reaction is still guesswork, but at least you are operating in the correct model *if* you are concerned with deflection (usually not the mode of failure for stiff pipes). Over-deflection of pipe is the primary mode of failure only when flexible pipe is installed within highly compressible embedment or when hydrostatic forces act on a partially deflected flexible pipe (not possible in perforated pipe systems- unless they somehow become plugged).

The following graph illustrates the inaccurate behavior of the Modified Iowa Formula. It behaves in this inaccurate manner for reasons just discussed. The ratio of the percent ring deflection (ΔX/pipe diameter) to sidefill soil strain is entered on the vertical axis of the graph. The ratio of soil stiffness (E') to pipe stiffness (EI/D^3) is entered on the horizontal axis of the graph. Two lines are plotted. One line represents the results of many large scale "lab" tests conducted at Utah State. The other line is constructed by using all the same values, except, instead of using the measured percent ring deflection, it uses the percent ring deflection predicted by the Modified Iowa Formula.

Comparison of the Iowa Formula and an Upper Limit Empirical plot for predicting ring deflection of flexible buried pipes.



As shown by the graph and discussed above, the Modified Iowa Formula is reasonably accurate when applied to the situation of a semi-rigid pipe in a compressible soil embedment but very inaccurate when applied to a flexible pipe in a stiff embedment. Properly embedded HDPE leachate collection pipes fall into this second category.

HDPE leachate collection pipes are flexible, perforated and not exposed to concentrated loads (like the loads applied by wheels of a truck over a shallow culvert - this type of loading can potentially "punch through" the soil arch from above). LCPs are under high fill conditions and the same circumstances that maintain a tunnel through a mountain help prevent properly bedded LCPs from flattening or otherwise failing. The embedment, preferably crushed rock, is strong and granular. The pipe is weak but continuous. Together they form a system with an open conduit at its center. The embedment resists crushing while the pipe holds the embedment in place and helps stabilize the limits of the arching material against spalling. The pipe also supports the rubble load between the soil arch and the top of the pipe, but this is a small load. The HDPE LCP's hardest days are immediately following the application of new loads. The details of why this is and how to properly design these pipes are presented below. There is, however, still one more important thing to say about design by pipe deflection analysis.

Whether a designer estimates pipe deflection by the Modified Iowa Formula or by compression of the sidefill soils, he/she must still decide what the allowable limit of deflection will be. The limit for HDPE pipe is not standardized. The following criteria have all been suggested for use in selecting allowable deflection for HDPE pipes (roughly in order of magnitude).

- The yield deflection, which is that degree of deflection where the load need not increase in order to result in continued deformation (magnitude varies).
- The field evidence of case histories which show that deflections up to at least 20% do not lead to failure. (Case histories on file at ASTM).
- Ease of cleaning considerations which suggest that deflection should be kept below 5 to 7%.
- Calculations of allowable tangential strain in the outer surface of the pipe wall. This approach results in different values of allowable deflection for different SDRs but it does not seem to account for the creep and subsequent stress relaxation behavior of HDPE. It also considers surficial cracking as failure whereas such cracks may not affect the performance of properly bedded, perforated leachate collection pipe. (See ASTM F 714-94).

Fortunately for landfill designers, deflection should not usually be an issue.

Design by fundamentals of mechanics:

This approach focuses on wall crushing strength and checks deflection and longitudinal effects. It aims to design the pipe and embedment as a system. The embedment material is chosen to limit deflection and the pipe is selected based on wall crushing strength as determined using the ring compression formula. The approach is conservative in that, while it recognizes that the arch over the pipe prevents the soil prism from following the pipe downward, it does not quantify the transfer of load from the pipe to the arch. The pipe is still selected such that the compressive strength of the pipe walls is adequate to support the entire prism load. This approach is applicable to situations where the embedment material is stiffer than the (flexible) pipe and concentrated loads are not expected, such as when HDPE LCPs are embedded in crushed rock or other stiff aggregates for service at the bottom of a waste pile.

A designer may enter this design process at any one of several points. Typically, he/she will have a preliminary design configuration in mind and need to check and/or refine it. At this point the designer may check the structural adequacy of the pipe using the ring compression formula.

The Ring Compression Formula:

This formula may be written as follows:

$$\sigma_C = \frac{PD}{2T}$$

Where:

σ_C is the magnitude of compressive force, or pressure, experienced by the pipe wall (psi). If this pressure is greater than the compressive strength of the wall then the wall will fail by crushing. The unit compressive strength of the material from which the pipe wall is made may be substituted here. The equation may then be solved for wall thickness.

P is the maximum unit pressure expected to act on the top of the pipe. This is equal to the density of the material over the pipe (the density of the waste pile) times the thickness, or depth, of burial.

D is the diameter of the pipe. P times D is Spangler's prism load without corrections for sidewall friction (conservative).

T is the thickness of the pipe wall. The use of 2T as the denominator expresses the fact that the load (PD) is supported by both the right wall of the pipe and by the left wall (at the 3 and 9 o'clock positions).

The equation can also be written $\sigma_C = P\,(dr)/2$ where "dr" is the dimensional ratio expressing the wall thickness of the pipe (also called "SDR").

If the pipe is made of HDPE then the *initial* compressive strength of the material should be used (approximately 3500psi). This is because, in the long run, the pipe will be exposed to a condition of constant strain, not constant stress, and its compressive strength will not regress with time. Under conditions of constant strain the rate of *relaxation* of the HDPE matrix is greater than the rate of *strength regression*. This means that a newly buried pipe will react, relatively quickly, to pressure within its walls, by relaxing. This, in turn, transfers more of the load from the pipe to the soil arch and soon the pipe is no longer experiencing the subject pressure. Without the constant pressure, or stress, the strength of the HDPE will not regress. If additional load is applied which compresses the embedment further and re-loads the pipe, then the pipe will be able to resist this new pressure with all of its original compressive strength as it begins, again, to relax.

Relaxation is not creep. Creep implies a long chain molecule in an irregular coil-like shape experiencing a change to the overall length of the coil. Relaxation is the subsequent behavior of the chain to establish its new, stable, configuration as its equilibrium (or "relaxed") shape. Relaxation cannot occur while the applied stress, and the associated creep, are happening. It does occur, under conditions of constant strain, after the limit of deformation is reached.

The use of the ring compression formula described above is conservative primarily because it does not account for the pipe's ability to relax and reset for additional loading. The use of the ring compression formula described above assumes that the entire maximum prism load is applied in one lift and that the pipe must bear all of it. In normal landfill operations the load is applied in several lifts of waste, usually months or years apart. By the time the second lift is applied the stresses induced in the pipe walls by the application of the first lift are gone and the pipe need only be strong enough to tolerate the pressure created by the second lift. Because of relaxation and the formation of the soil arch, the LCP walls never experience the internal pressure calculated by the ring compression formula using the full depth of burial. Assuming that they do is conservative.

The two remaining steps in this design approach are to check the deflection and for longitudinal effects.

Pipe deflection is assumed equal to the deflection of the sidefill soils. Selecting the sidefill soils, and the embedment material in general, is critical to a successful LCP design. This material should be durable and as incompressible as possible. Durable crushed and rounded rock are both good materials for this application. Sand may also be adequate if it can be installed in a dense condition which limits its subsequent settlement or compressibility. Under extreme loads rounded rock may be best since the point loads that develop within the matrix are less than such loads within a crushed rock installation. The greater friction angle of crushed rock adds some benefit during construction, when concentrated loads are possible, but little or no benefit during service.

The expected deflection of the sidefill soil material is determined in the lab or by using published estimates of the modulus of elasticity for the selected material type. Other soil characteristics may also have to be considered if material other than free draining rock is selected. In any case the pressure used is the pressure expected on the material at the design burial depth. The length, or height, of the column of material being compressed is set equal to the original outside diameter of the pipe. The deflection of the pipe is assumed equal to the deflection of the side fill soils only. Deflection of embedment materials above and below the pipe does not effect the deflection of the pipe.

Once the deflection of the pipe has been estimated in this way it is compared to whatever deflection criteria the designer has selected.

Longitudinal effects are deformations of the pipe due to inconsistent support by the pipe bedding and/or material below the bedding. For instance, if a pipeline is installed over a soft spot in otherwise firm foundation soils, it will be well supported before and after but sag within the soft area. This sagging will cause compression of the pipe walls at the top of the pipe and tension at the bottom. A designer may determine the magnitude of these compressive and tensile forces and check them against the strength of the proposed pipe. This situation should not occur in LCP installations and so is not discussed further here.

Embedment geometry considerations include the width of sidefill soils and cover over the pipe. Experiments at Utah State have shown that half a pipe diameter is usually sufficient to manifest the arching action but the authors recommend a minimum of one full pipe diameter. For LCPs set at a simple break in grade the embedment material must be installed to sufficient width to provide adequate cover while considering the angle of repose of the material. This usually dictates more than enough sidefill embedment. It is also best to taper the embedment material from its high point above the pipe down gradually to meet the drainage material of the leachate collection layer (or construction layer) such that there is no abrupt protrusion of the embedment material into the waste. Such an intrusion could tend to increase the load on the soil arch by causing down-drag on the prism of waste directly above the pipe. Every effort should be made to densify all embedment materials. Pea gravel bedding and other clean aggregate embedment material should be installed in lifts and vibrated in place.

Summary

Perforated HDPE leachate collection pipes installed within stiff and durable embedment material for service under a waste pile represent one of the most straightforward and fail- safe of all pipe installation scenarios in regard to structural stability. Their deflection is not predicted by the Modified Iowa Formula and should not be a limiting design criteria. The mode of failure for such pipes is wall crushing, which is avoided by selecting good embedment and pipes with adequate wall strength as expressed by the ring compression formula. Using this formula with the full prism load is conservative.

Watkins, R.K. and M.G. Spangler: "Some Characteristics of the Modulus of Passive Resistance of Soil - A Study in Similitude", Highway Research Board Proceedings, Vol. 37, 1958, pp. 576-583.

2. White, H.L. and J.P. Layer: "The Corrugated Metal Conduit as a Compressive Ring", Highway Research Board Proceedings, 1960, Vol. 39, pp. 389 - 397.

APPENDIX I.9

STRUCTURAL CAPACITY OF LEACHATE COLLECTION PIPE

Shaw®

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Calculated By: PCT Date: 10/3/07

Checked By: JPV Date: 10/4/07

TITLE: STRUCTURAL CAPACITY OF THE LEACHATE COLLECTION PIPE

Problem Statement

Determine if the proposed leachate collection pipe possesses sufficient strength to support the overlying landfill materials, in accordance with 35 Ill. Admin. Code Section 811.308 (e), considering the following failure modes:

1. Wall buckling
2. Wall crushing

Given

1. Calculation *Earthloads on the Leachate Collection System* contained in this application (Appendix I).
2. Calculation *Ring Deflection of Leachate Collection Pipe* contained in this application (Appendix I).
3. KWH Sclairpipe® product information (refer to attached pages).
4. Formula used to calculate the safe allowable buckling load for the pipe (reference KWH Sclairpipe®):

$$q_a = (DF)\left(\frac{32 R_w B' E' EI}{D_{avg}^3}\right)^{0.5}$$

Where,

q_a = Safe allowable buckling load (psi)
DF = Design Factor
R_w = Water Buoyancy factor = $1-0.33(h_w/h)$
B' = Coefficient of elastic support = $(1+4e^{-0.065H})^{-1}$
D_{avg} = Mean pipe diameter = O.D. - t_{min}
E = Long-term modulus of elasticity of the pipe material (psi)
I = Moment of inertia of the pipe wall for ring bending (in^4/inch) = $t_{min}^3/12$

e = 2.718
H = Height of fill above pipe (ft)
h = Height of fill above pipe (in)
h_w = Height of water above pipe (in)
E' = Modulus of soil reaction (psi)

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An equivalent form of the equation is derived as follows:

$$\frac{32I}{D_{avg}^3} = \frac{32\left(\frac{t_{\min}^3}{12}\right)}{(O.D.-t_{\min})^3} = 2.67\left(\frac{t_{\min}}{(O.D.-t_{\min})}\right)^3 = \frac{2.67}{\left(\frac{O.D.}{t_{\min}}-1\right)^3} = \frac{2.67}{(SDR-1)^3}$$

$$\therefore q_a = (DF)\left(\frac{2.67R_w B' E' E}{(SDR-1)^3}\right)^{0.5}$$

Where,

SDR = (Standard) Dimension Ratio = (pipe outer diameter)/(pipe wall thickness)

5. Formula used to calculate compressive stress which will exist in the leachate collection pipe due to overlying loads (reference KWH Sclairpipe®):

$$\sigma_c = \frac{W}{2t_{min}}$$

Where,

σ_c = compressive stress acting on leachate collection pipe (psi)
W = Maximum earthloads acting on leachate collection pipe (lb/in)
t_{min} = thickness of leachate collection pipe wall (inch)

6. The worst case scenario is the leachate collection piping within the Chemical Waste Unit which specifies a 6-inch SDR 11 HDPE pipe.

7. DF = 0.4 (reference KWH Sclairpipe®)

8. h_w = 1.0 inch (conservative estimate)

9. H = 172.7 ft (reference "Earthloads" calculation)

10. h = 2,072.4 inches

11. E = 30,000 psi (reference KWH Sclairpipe®)

12. E' = 3,000 psi (reference KWH Sclairpipe®)

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13. O.D. = 6.63 inches for a 6-inch SDR 11 Pipe (reference KWH Sclairpipe®)

14. t_{min} = 0.602 inches for a 6-inch SDR 11 Pipe (reference KWH Sclairpipe®)

15. W = 680.3 lb/in for a 6-inch SDR 11 Pipe (reference "Earthloads" calculation)

16. σ_c (max allowable) = 800 psi (reference KWH Sclairpipe®)

Calculations

Wall Buckling

Calculate q_a for a 6-inch SDR 11 Pipe:

$$R_w = 1 - 0.33\left(\frac{h_w}{h}\right) = 1 - 0.33\left(\frac{1}{2{,}072.4}\right) = 0.99984$$

$$B' = \left(1 + 4e^{-0.065H}\right)^{-1} = \left(1 + 4(2.718)^{-0.065(172.7)}\right)^{-1} = 1.0$$

$$q_a = (DF)\left(\frac{2.67R_w B'E'E}{(SDR - 1)^3}\right) = (0.4)\left(\frac{(2.67)(0.99984)(1)(3{,}000)(30{,}000)}{(11-1)^3}\right)^{0.5} = 196.1$$

Verify that the landfill load is less than the safe allowable buckling load for a 6-inch SDR 11 Pipe:

$$\text{Verify :} \qquad R_w\left(\frac{W}{D_{avg}}\right) \le q_a$$

$$R_w\left(\frac{W}{D_{avg}}\right) = 0.99984\left(\frac{680.3}{(6.63 - 0.602)}\right) = 112.8\ \text{psi}$$

$$\text{Factor of Safety} = \frac{196.1}{112.8} = 1.74$$

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TITLE: STRUCTURAL CAPACITY OF THE LEACHATE COLLECTION PIPE

Wall Crushing

Verify that the compressive stress, σ_c, is less than 800 psi for a 6-inch SDR 11 Pipe:

$$\sigma_c = \frac{W}{2t_{min}} = \frac{680.3}{2(0.602)} = 565\text{ psi}$$

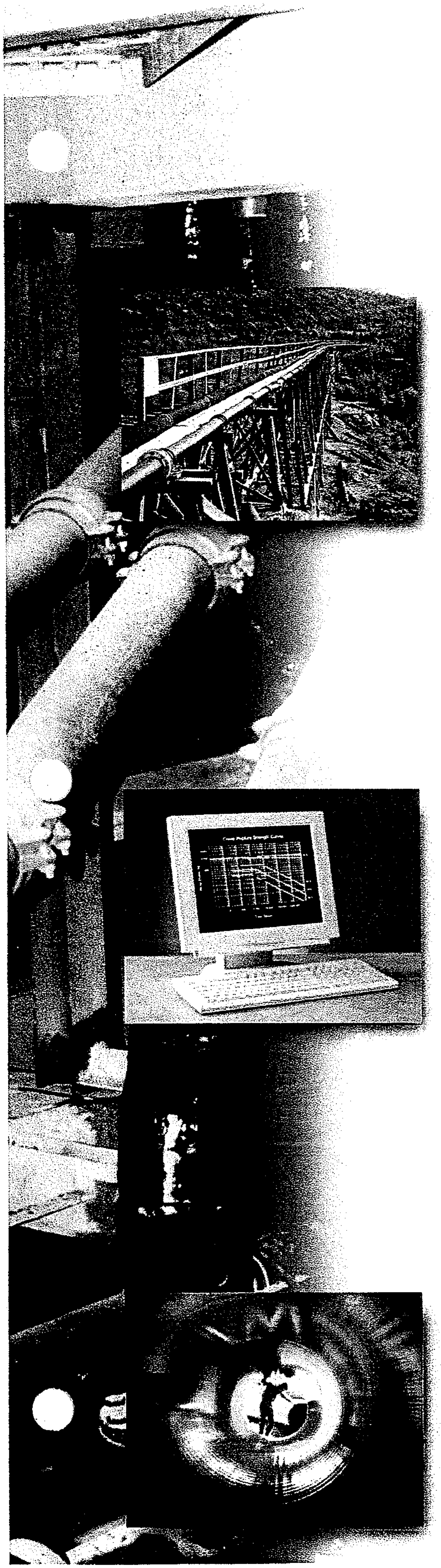
$$\text{Factor of Safety} = \frac{800}{565} = 1.42$$

The above calculation for Wall Crushing utilized very conservative assumptions when calculating the Factor of Safety. The leachate collection pipe is surrounded by a gravel envelope that serves as a additional level of protection if the leachate collection pipe would be crushed. If the pipe was crushed or became clogged, leachate would still flow to the designated collection point through the gravel envelope.

Results

In accordance with 35 Ill. Admin. Code Section 811.308 (e), the proposed leachate collection pipes will possess sufficient strength to support the overlying landfill, as shown by the calculated factors of safety against pipe wall buckling and pipe wall crushing for a 6-inch SDR 11 Pipe.

6-inch SDR 11	
Pipe Failure Mode	**Factor of Safety**
Wall Buckling	1.74
Wall Crushing	1.42



high density polyethylene pipe

Sclairpipe®

Systems Design

KWH PIPE

Rules for Choice of Pipe Weight

PRESSURE CLASS DESIGNATION

SCLAIRPIPE® high density polyethylene (HDPE) pipe pressure class ratings are designated by a Dimension Ratio (DR) number. This is a common "rating" system specified by ASTM, AWWA and CSA for polyethylene pipes. The DR number is also used for pressure classification of other non-metallic piping materials such as PVC, ABS and polypropylene.

A dimension ratio is defined as the ratio of outside pipe diameter to minimum allowable wall thickness. The relationship of a pipe's dimension ratio to a pipe's standard pressure rating is described in the modified ISO formula as detailed below:

$$P = \frac{(2)(HDS)(t)}{D_O - t}$$

where:
- P = maximum operating pressure at 73.4°F under steady state conditions
- HDS = Hydrostatic Design Stress at 73.4°F
- t = minimum pipe wall thickness
- D_O = pipe outside diameter

and: $DR = D_O/t$

By substituting the above relationship into the modified ISO formula, it reduces to:

$$P = \frac{2\,(HDS)}{DR - 1}$$

This simplified relationship shows the pipe pressure rating, P, as a function of the pipe DR number and the hydrostatic design stress of the resin used to extrude the pipe.

The HDS is derived from the extrapolation of a series of hydrostatic pressure tests used to define the pipe's time-to-failure envelope. Circumferential wall stress (hoop stress) is developed by pressurizing a number of pipe samples and recording the time to failure. This data is analyzed according to the method described in ASTM D2837 to extrapolate and pinpoint the pipe compounds Long-Term Hydrostatic Strength (LTHS). The LTHS is then used to categorize the pipe's Hydrostatic Design Basis (HDB) based on the respective range that it fits into. Once the HDB is assigned to the pipe's hydrostatic capabilities, it is reduced by a design factor of 0.50 to determine the Hydrostatic Design Stress (HDS). This allows an appropriate safety margin and permits operation with the reasonable expectation that the pipe will have indefinite life (i.e. 50 years or more).

DESIGN CRITERIA

For each pipe DR number, there is a corresponding maximum allowable continuous operating pressure at 73.4°F when used in water service. This pressure rating varies when different pipe design hoop stress values (HDS) are substituted into the pipe design equation. Typically, HDPE pipes are made from materials qualified as PE 3408 which means the compound has a HDS of 800 psi.

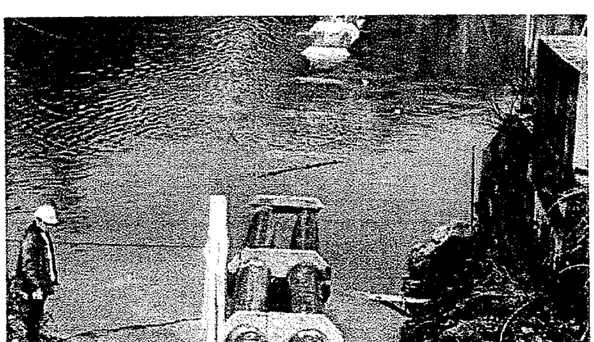
This pipe design methodology has been checked against long term pipe strain. Strain in polyethylene pipe has been found to govern the life of the pipe system. Operation at the design stress level should induce no greater than 3% strain over 50 years of continuous service at 73.4°F. This is consistent with other investigations where the long term strain design limit of 3% to 4%, incorporating a 0.5 design factor, has been designated.

SUMMARY OF RULES FOR PIPE SELECTION

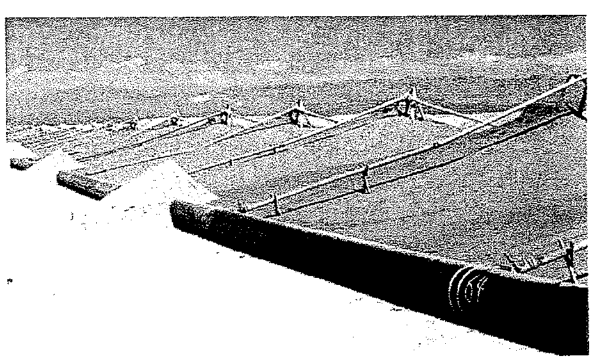
As described previously, a specific DR and material hydrostatic design stress, HDS, produces the same continuous standard maximum operating pressure for 50 years life at 73.4°F incorporating a 2:1 safety factor, regardless of the nominal pipe size (NPS).

In design, it is this "pressure" rating which can be factored to provide a "service" rating depending on the conditions of service. Service factors can vary from 1.0 (or more) to 0.25 (or less) and will depend on the relationship between the pipes' operating conditions, the pipes' intended use and expected lifetime.

Certain operating conditions may not necessarily utilize a design service factor such a buckling and pipe deflection in buried pipe applications. Here, design performance limits have been defined for each pipe DR rating. How service factors and design performance limits are defined, are discussed in the appropriate sections of this manual.



Installation of 24 inch SCLAIRPIPE for a twin sewage siphon line in Victoria, B.C. The pipe is completely resistant to seawater and its smooth surface discourages the adherence of algae and other marine growths.



SCLAIRPIPE used in a tailing applications at a molybdenum mine in Arizona. Inclusion of 2% finely dispersed carbon black ensures that the pipe is resistant to ultraviolet light degradation enabling it to be installed at grade. Anchoring of the pipeline is achieved simply by dumping a load of tails on the pipe at regular intervals.

Internal Pipe Pressure

Pressure performance requirements for SCLAIRPIPE at 73.4°F are as follows:

ASTM F714 The pipe shall not fail, when tested by the methods detailed in ASTM D1599, in 60 to 70 seconds at a pressure less than 3.63 times the standard pressure rating.

ASTM F714 The pipe shall not fail, when tested in accordance with ASTM D1598, in 1,000 hours at a pressure equal to 2 times the pipe standard pressure rating.

ASTM D2837 The pipe shall withstand a pressure equivalent to 1.97 times the pipe's standard pressure rating for a period of 11.4 years (100,000 hours).

It should be noted that the above requirements are test requirements under laboratory conditions and therefore must be adjusted by a design factor to be used for pipe pressure rating purposes. Although a basic design factor of 0.5 is used for determining long-term (50 years) operating limits, shorter term phenomena may be related to the "safe strain limit" which laboratory investigations demonstrate to be approximately 3% to 4%. The following maximum stress levels are therefore recommended for protection against varying terms of pressure exposure for Sclairpipe produced from a PE3408 material:

Duration of Surge or Pressure Phenomena	Maximum Allowable Hoop Stress at 73.4°F
Instantaneous (up to 60 sec.)	1600 psi
Up to 1 hour	1465 psi
1 hour to 1,000 hours	1070 psi
Sustained pressure, 50 years	800 psi

The above recommendations are based on the assumption that the pipe will not be subjected to other imposed stresses. They refer to phenomena which cease within the time limits given and the pipe then returns to a "normal" operating pressure. These phenomena may be repeated with reasonable expectation that the service life expectancy of the piping will not be significantly affected.

Regular pressure cycling, outside of hydraulic transient situations, should not be accommodated in this way. When such cycling is expected as a regular condition of operation, the highest pressure anticipated for the majority of the operating time should be considered as the operating pressure and treated as though it would persist continuously for the design life of the system.

SHOCK LOADS

Hydraulic shock loads (sometimes called "water hammer") can be difficult to calculate in complex systems, however, their presence and cause can be predicted. For further information reference should be made to the "Waterhammer and Hydraulic Transients" section of this manual.

It is often more economical to eliminate the cause rather than attempt to accommodate stresses by increasing the standard pressure rating of the selected pipe. It is known that under some conditions, a lighter weight pipe will be more resistant to damage under these conditions than a heavier weight pipe and that rigid materials and structures will increase the magnitude of the stresses. Overpressure is not likely to be a limiting factor in design. Negative pressures, resulting from column separation and pressure shocks resulting from the collapse of the separation, are more likely to be limiting factors in design.

EXTERNAL LOADS

Performance limits with regard to earthloading design and external hydraulic loading follow the recommendations given in "Earthloading - Design of Underground Piping Systems" and "Vacuum and External Hydraulic Overpressure" sections of this manual. Strength requirements under these conditions are functions of the cube of the Dimension Ratio (DR).

ENVIRONMENTAL CONDITIONS

Temperature is the most important environmental consideration. For operation at temperatures in excess of 73.4°F, a thermal service factor should be applied to the pressure rating as described in "Design Considerations Related to Environment" section of this manual.

Corrosive conditions are normally not a consideration with SCLAIRPIPE, but they do occur in industrial processing uses associated with strong oxidizing chemicals (see section on "Chemical Resistance and Permeability"). Oxidation, which results from exposure to certain aggressive chemicals is usually manifested by embrittlement of the surface and a significant reduction in the long-term stress resistance of the material.

The polyethylene material used in the manufacture of SCLAIRPIPE has a high resistance to environmental stress cracking. However, when the pipe is stressed in the presence of certain surface active chemicals, e.g. wetting agents, environmental stress cracking can take place with a detrimental effect to the products projected long-term life.

When chemical resistance is in doubt, exposure tests are recommended. Generally these tests follow the procedures described in ASTM D543. Changes in tensile properties can be measured on ring tensile specimens in accordance with the procedures described in ASTM D2513, paragraph 8.6. Significant variation between control specimens and those exposed to the chemical is generally accepted as evidence of corrosive degradation and decisions as to use of SCLAIRPIPE in this application shall be made accordingly.

Direct assistance of KWH technical personnel is recommended where further explanation and assistance is required.

Earthloading - Design of Underground Piping Systems

INTRODUCTION

This section defines the performance limits for SCLAIRPIPE polyethylene pipe in the following three burial environments - in varying soils and soil compaction levels, in firm soils where the buried pipe is subjected to external hydrostatic pressure and in firm and loose soils with the buried pipe subjected to internal vacuum or net external hydrostatic pressure. In all cases, the pipe is considered to be empty with no resistance to deflection contributed by internal pressure.

Flexible conduits react to earthloads or external hydrostatic loads very differently than rigid pipes do. The natural ring stiffness of the flexible pipe contributes only a small portion of the total resistance to deflection; most of the resistance arises from the soil stiffness. When the buried pipe deflects slightly in the vertical axis, the accompanying outward movement of the pipe side walls mobilizes the support available due to the stiffness of the surrounding soil envelope. Figure 2 provides an illustration of this mobilization process. The pipe is supported against further movement and exhibits load-bearing capabilities far greater than unsupported pipe. The amount of support which is available in the embedment soil is a direct consequence of the installation procedure. The stiffer the embedment materials are; the less deflection occurs and the more stable the pipe-soil system is.

DESIGN CRITERIA

When selecting the most appropriate wall thickness or DR for Sclairpipe to resist anticipated burial conditions or when confirming the adequacy of a selection which was made based on pressure class requirements three design criterion are considered separately; vertical deflection, wall buckling and wall compression or crushing. The amount of deflection which can be expected under specific burial conditions may be estimated using the form of the Iowa pipe deflection formula presented below. The estimated vertical deflection as a percentage of the mean pipe diameter is then compared to the safe design limits presented in Table 1. In order to verify the adequacy of the pipe-soil system against wall buckling or collapse the *safe allowable buckling load (q_a)* is determined using the equation presented and compared to the anticipated applied loads. Compressive stress in the pipe walls may also be estimated and compared to the safe compressive strength of HDPE which is conservatively estimated as 800 psi.

DEFLECTION

$$\Delta y = \frac{(D_l W_c + W_L)\, K_x\, r^3}{EI + 0.061E'r^3} \qquad (1.0)$$

Where; Δy = predicted vertical pipe deflection in inches.

D_l = the deflection lag factor to compensate for the time-consolidation rate of the soil, dimensionless. Normally estimated as 1.5.

W_c = vertical soil load on the pipe per unit length, in pounds per linear inch. W_c is estimated by multiplying the appropriate value from Table 2 by the outside diameter (in inches) of the pipe.

W_L = live load on the pipe per unit length, in pounds per linear inch. W_L is estimated by multiplying the appropriate value from Figure 3 by the outside diameter (in inches) of the pipe.

K_x = deflection coefficient, dimensionless, Use 0.083 for most installations.

r = mean pipe radius in inches.

r = (O.D. $-t_{min}$)/2

t_{min} = minimum wall thickness of pipe in inches.

E = Apparent modulus of elasticity of the pipe material in psi. A long-term apparent modulus of 30,000 psi may be used in most situations.

I = the moment of inertia of the pipe wall for ring bending in inches^4/inch.

I = $t_{min}^3/12$

E' = modulus of soil reaction, in psi. The appropriate value for E' should be selection from Table 3.

Table 1
SAFE DESIGN LIMITS

Dimension Ratio	Allowable Vertical Ring Deflection as a % of Diameter
32.5	8.6
26	6.5
21	5.0
17	4.0
11	3.3
9	2.6

WALL BUCKLING

The safe allowable buckling load for the soil-pipe structure (q_a) is estimated as follows;

$$q_a = (DF)\,(32\, R_w\, B'\, E'\, EI/D_{avg}^3)^{0.5} \qquad (2.0)$$

Where: q_a = safe allowable buckling load in psi.

DF = design factor, 0.40

R_w = water buoyancy factor, calculated as follows;

R_w = $1 - 0.33(h_w/h)$; $0 \leq h_w \leq h$

Where: h_w = height of ground water surface above top of pipe in inches.

h = height of ground surface above top of pipe in inches.

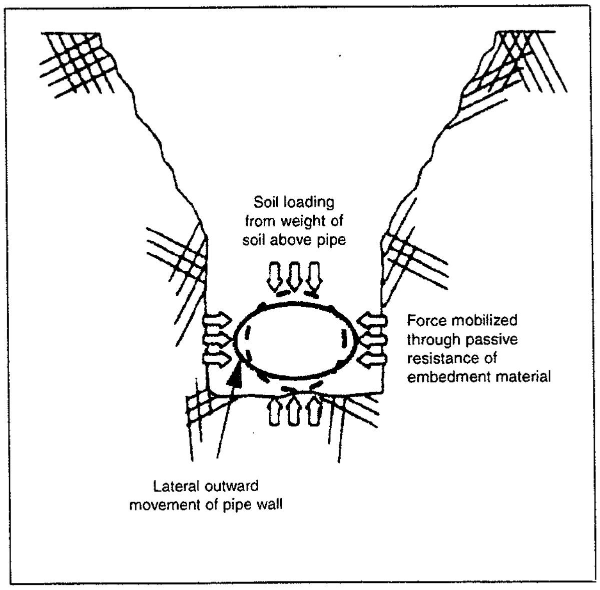


Figure 2: Mobilization of Enveloping Soil through Pipe Deformation

Table 2

Vertical Soil Load in lbs./in²				
Depth to Top of Pipe in ft.	Soil Density 90 lbs.ft³	Soil Density 100 lbs.ft³	Soil Density 110 lbs.ft³	Soil Density 120 lbs.ft³
1	0.6	0.7	0.8	.8
2	1.3	1.4	1.5	1.7
3	1.9	2.1	2.3	2.5
4	2.5	2.8	3.1	3.3
5	3.1	3.5	3.8	4.2
6	3.8	4.2	4.6	5.0
7	4.4	4.9	5.3	5.8
8	5.0	5.6	6.1	6.7
9	5.6	6.3	6.9	7.5
10	6.3	6.9	7.6	8.3
12	7.5	8.3	9.2	10.0
14	8.8	9.7	10.7	11.7
16	10.0	11.1	12.2	13.3
18	11.3	12.5	13.8	15.0
20	12.5	13.9	15.3	16.7
25	15.6	17.4	19.1	20.8
30	18.8	20.8	22.9	25.0

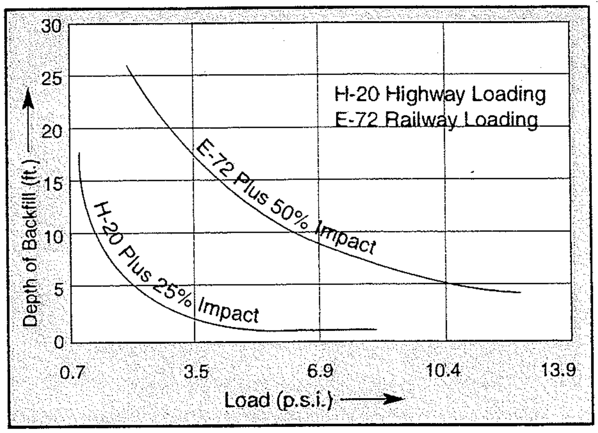


Figure 3: Live loading due to vehicle traffic

Table 3
Embedment Classes per ASTM D-2321

			Average Value of E'			
			Degree of Compaction of Embedment Material [1] (Standard Proctor)			
Class	Soil Description	Soil Group Symbol	Dumped	Slight 85%	Moderate 90%	Heavy > 95%
IA	Manufactured aggregate angular open-graded and clean. Includes crushed stone, crushed shells.	None	500	1000	3000	3000
IB	Processed aggregate, angular dense-graded and clean. Includes Class 1A material mixed with sand and gravel to minimize migration.	None	200	1000	2000	3000
II	Coarse-grained soils, clean. Includes gravels, gravel-sand mixtures, and well and poorly graded sands. Contains little to no fines (less than 5% passing #200).	GW, GP, SW, SP	200	1000	2000	3000
II	Coarse-grained soils, borderline clean to "with fines". Contains 5% to 12% fines (passing #200).	GW-GC, SP-SM	200	1000	2000	3000
III	Coarse-grained soils containing 12% to 50% fines. Includes clayey gravel, silty sands, and clayey sands.	GM, GC, SM, SC	100	200	1000	2000
IVa	Fine-grained soils (inorganic). Includes inorganic silts, rock flour, silty-fine sands, clays of low to medium plasticity, and silty or sandy clays.	ML, CL	50	200	400	1000
IVb	Fine-grained soils (inorganic). Includes diatomaceous silts, elastic silts, fat clays.	MH, CH	No date available; consult a competent soils engineer. Otherwise use E' equals zero.			
V	Organic soils. Includes organic silts or clays and peat.	OL, OH, PT	No date available; consult a competent soils engineer. Otherwise use E' equals zero.			

[1] E' values taken from Bureau of Reclamation table of average values and modified slightly herein to make the values more conservative.

B' = empirical coefficient of elastic support, dimensionless. Calculated as follows;

$$B' = (1 + 4e^{-0.065H})^{-1}$$

Where: H = burial depth to the top of the pipe in ft.

D_{avg} = mean pipe diameter (O.D. - t_{min})

For most pipe installations satisfaction of the wall buckling requirement is assured when the following equation is true;

$$\gamma_w h_w + R_w(W_c/D_{avg}) + P_v \le q_a \qquad (2.1)$$

Where; γ_w = specified weight of water (that is, 0.0361 lbs./in.³) in pounds per cubic inch.

P_v = internal vacuum pressure (that is, atmospheric pressure less the absolute pressure inside of the pipe), in pounds per square inch.

In some situations, consideration of live loads in addition to dead loads may be appropriate. However, simultaneous application of the live-load and internal vacuum transients need not normally be considered. When live loads are being considered, the buckling requirement is assured when the following equation is true;

$$\gamma_w h_w + R_w(W_c/D_{avg}) + W_L/D_{avg} \le q_a \qquad (2.2)$$

COMPRESSION

The compressive stress which will exist in the pipe wall due to anticipated burial loads (σ_c) can be estimated using the following equation;

$$\sigma_c = (W_c + W_L) / (2t_{min}) \qquad (3.0)$$

Satisfaction of the wall compression is assured when the following equation is true;

$$\sigma_c \le 800 \text{ psi} \qquad (3.1)$$

CRITICAL PRESSURE FOR UNSUPPORTED PIPE:

In locations such as bogs, swamps or underwater, empty polyethylene pipelines can collapse if subjected to an excessive external/internal pressure differential. Such differential pressures may be caused by drawing a vacuum or by simply increasing the external hydraulic loading. Limiting critical pressures have been calculated from the modified Iowa Equation using a modulus (pipe stiffness) equivalent to 50 years of exposure to the critical pressure. Table 4 shows the critical pressure at 73.4°F for various pipe DRs or wall thicknesses. These critical pressures will cause full collapse of a pipe which has no initial deflection and is subjected to no stresses other than the net external pressure. However, damage can result to the pipe through excessive straining before full collapse, necessitating other safety factor considerations. For more information on the selection of pressure rating for unsupported pipe, see the section on Vacuum & External Hydraulic Overpressure.

Table 4
CRITICAL PRESSURES FOR PIPE WITHOUT SUPPORT

Dimension Ratio	Net External Critical Pressure (Pcr) (psi)
32.2	1.0
26	1.9
21	3.6
17	6.8
15.5	8.9
13.5	13.5
11	25.0
9	45.7

CRITICAL PRESSURE FOR SOIL-SUPPORTED PIPE:

Experimental work has shown that soil-supported pipe has a much greater capacity to withstand vacuum or net external pressures than pipe without support. This is particularly important when evaluating the effect of negative hydraulic transient pressures that may arise in pressure lines with sudden valve closures or pump failures. Treatment of this problem should be referred to your nearest KWH Pipe office.

Bedding Limitations:

- Always level the trench bottom, taking care to remove all sharp rocks and/or protrusions within 6 inches of the pipe.
- Ensure that the bedding material is worked into uniform contact with the pipe at the haunches.
- When bedding soil is non-compactible by its own weight, use mechanical compactions - DO NOT MECHANICALLY COMPACT DIRECTLY ON TOP OF THE PIPE - PLACE ONE FOOT OF BEDDING BEFORE COMPACTING DIRECTLY OVER THE PIPE.
- Do not allow rocks or frozen clods within a one foot bedding "envelope" around the pipe.
- See the Construction brochure for further details and burial information.

SAMPLE PROBLEM:

Problem

A 48" DR32.5 sewer pipe is to be buried with a depth of cover of 10 feet to the top of the pipe and must withstand H-20 truck traffic.

If the pipe is above the groundwater table and embedded in Type IB material (γ_s = 110 lbs/ft.) compacted to 85% Standard Proctor Density, is the pipe selection adequate?

Solution

Part 1 Deflection;

$W_C = 7.6 \times 48 = 364.8$ lbs/in.

$W_L = 1.4 \times 48 = 67.2$ lbs/in.

$r = (48 - 1.453)/2 = 23.274$ in.

$I = 1.453^3 / 12 = 0.256$ in.

$E' = 1{,}000$ psi

$$\therefore y = \frac{(1.5 \times 364.8 + 67.2)\ 0.083 \times 23.274^3}{30{,}000 \times 0.256 + 0.061 \times 1{,}000 \times 23.274^3} \quad (1.0)$$

$= 0.828$ in.

$= 1.78$ % of the mean pipe diameter

$\therefore$ Pipe selection is adequate for deflection criteria

Part 2 Wall Buckling;

$h_w = 0.00$ in.

$R_w = 1.00$

$B' = (1 + 4e^{-0.065 \times 10})^{-1}$

$= 0.324$

$D_{avg} = 48 - 1.453$

$= 46.547$ in.

$\therefore q_a = (1/2.5)(32 \times 0.324 \times 1{,}000 \times 30{,}000 \times 0.256/46.547^3)^{0.5}$ (2.0)

$= 11.24$ psi

Now check:

$0.0361 \times 0.00 + 1.00 \times 364.8/46.547 + 67.2/46.547 \leq q_a$ (2.2)

$9.281 \text{ psi} \leq 11.24 \text{ psi}$

$\therefore$ Pipe selection is adequate for buckling criteria

Part 2 Wall Compression; (3.0)

$\sigma_c = (364.8 + 67.2) / (2 \times 1.453)$

$= 148.658$ psi

$\sigma_c \leq 800$ psi (3.1)

$\therefore$ Pipe selection is adequate for wall crushing criteria

Since the selected pipe meets the requirements of all three of the design criteria the pipe selection is structurally adequate.

APPENDIX I.10

LEACHATE CLEANOUT

Shaw®

Photo 1 - Equipment Used for Hydraulic Jetting of the Leachate Collection Pipe

Photo 2 - Hydraulic Jetting Hose entering HDPE Leachate Collection Pipe

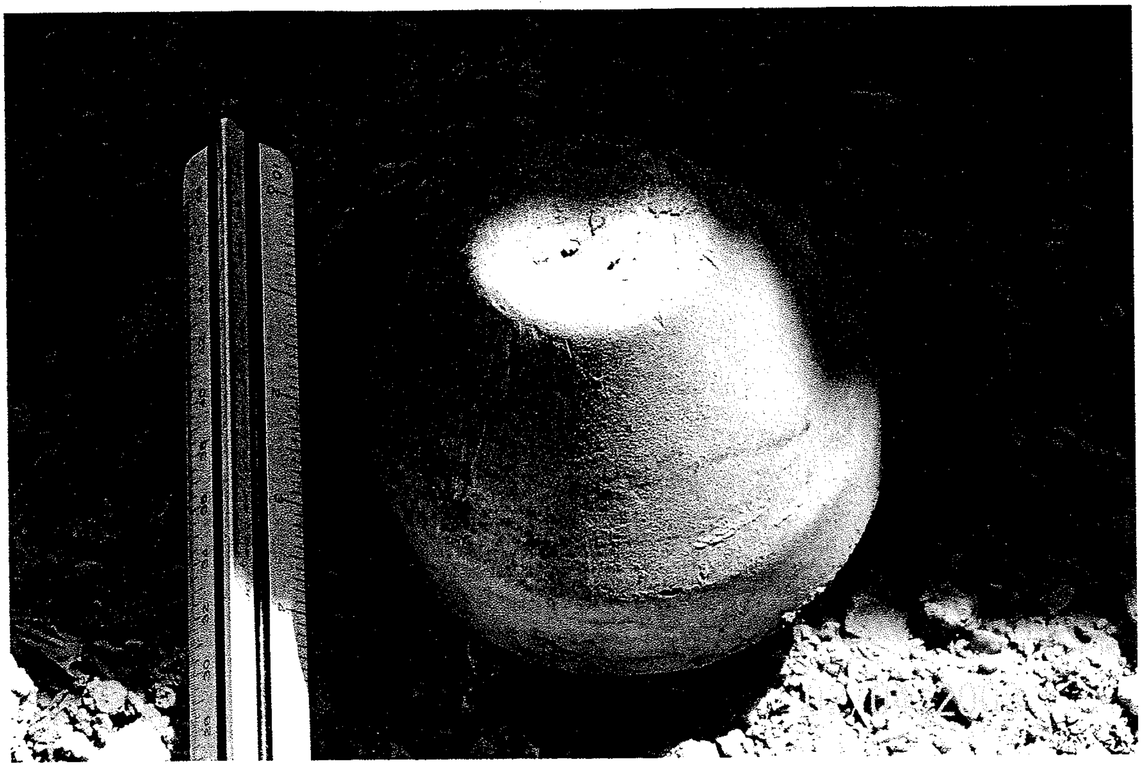


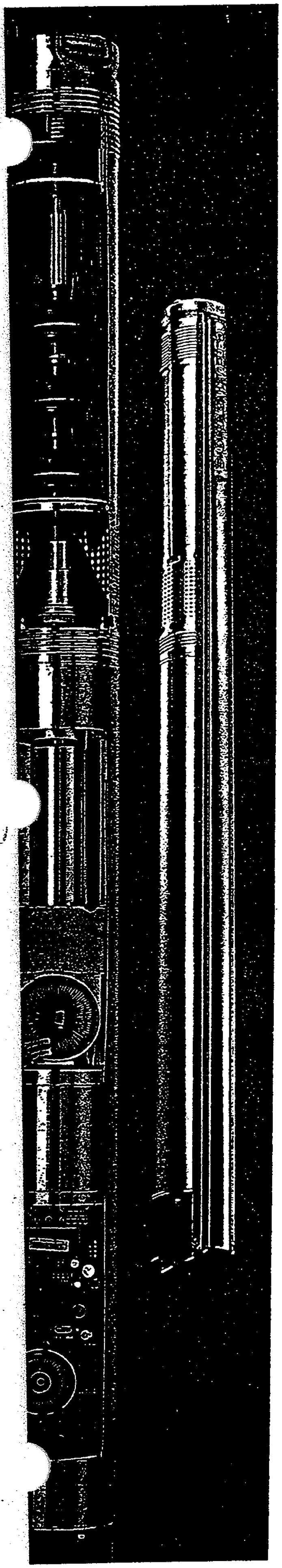
Photo 3 - Top View of 3" Diameter Cleaning Plug

Photo 4 - Bottom View of 3" Diameter Cleaning Plug showing Hydraulic Jets

Shaw®

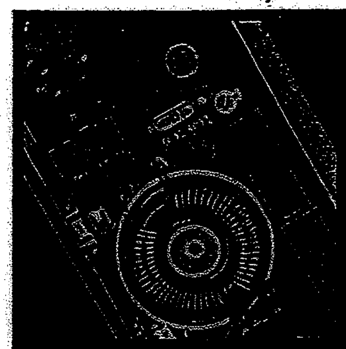
APPENDIX I.11

TYPICAL LEACHATE PUMPS



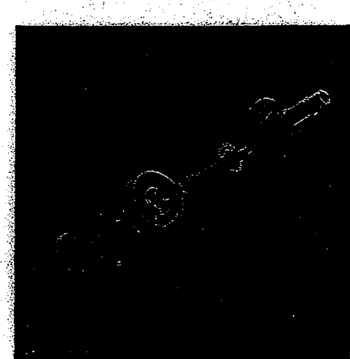
Redi-Flo3

Grundfos brings environmental pumping systems into the 21st century with the new Redi-Flo3 submersible pump.



Advanced Electronics

By combining advanced electronics, permanent-magnet motors, and Grundfos' own micro-frequency converter, we are now able to control and communicate with pumps in ways never before possible. A few of the features that come out of this combination are Fluid Level Control, Soft-Start and integrated Dry-Run Protection.



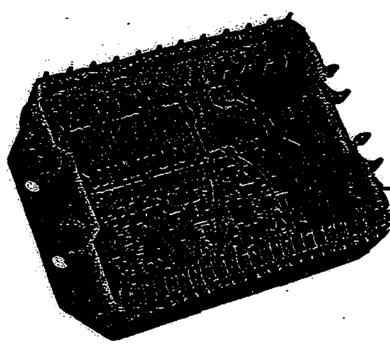
Permanent-Magnet Motor

The Redi-Flo3 features a newly developed permanent-magnet motor, controlled by advanced electronics, featuring Grundfos' micro-frequency converter.

As a result of the high and flat performance curve of the motor, a wider performance ratio can be covered by fewer models as compared to pumps with conventional motors.

The motor has a soft-start system which allows the pump to start with gradually increasing speed and with the highest possible starting torque.

The starting torque is 1.5 times greater than a conventional 3-wire motor.

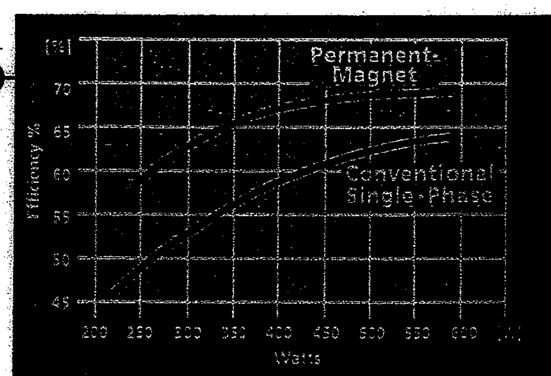


Micro-Frequency Converter

The Grundfos' designed micro-frequency converter controls the permanent-magnet motor.

Motor Efficiency Curve

Permanent-magnet motors produce a high efficiency over a wide load range as compared to conventional single-phase motors.

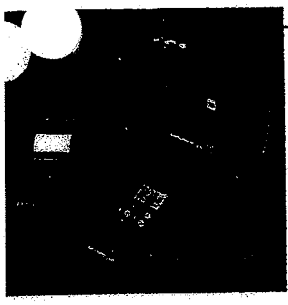


GRUNDFOS®

Leaders in Pump Technology

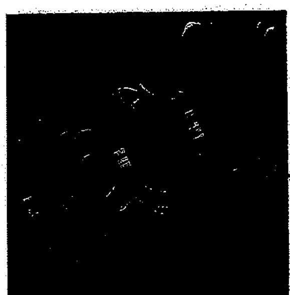
Redi-Flo3

Technical Data



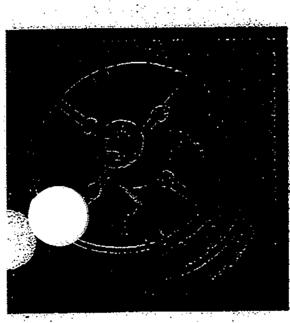
Status Box/R100 Infrared Remote

The optional Redi-Flo3 status box and R100 at the surface allows you to communicate with the pumps integrated electronics through the standard power leads. No additional wires are required! This feature provides the direct use of multiple sensors, digital input and relays without adding any extra electronics and cost. Pump status readout and parameter changes can easily be performed at the surface with the R100 or the Redi-Flo3 PC Tool.



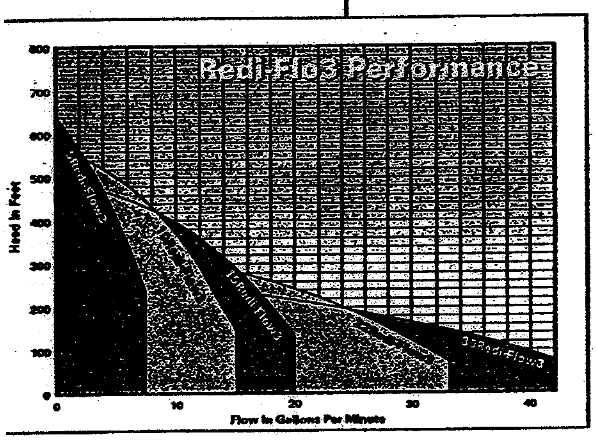
Rugged Design

Redi-Flo3 pump design uses "floating" impellers. Each impeller has its own tungsten carbide/ceramic bearing. This design and the environmentally tough 316 stainless steel and PVDF construction provide excellent wear resistance and solids handling capability.



Reliable Check Valves

Reliable built-in spring loaded check valves let you operate the pump in any position from vertical to horizontal.



ELECTRIC

Supply Voltage [illegible]

Operation on Generator [illegible]

Starting Current [illegible]

[illegible]

Available from:

Performance curves and technical information listed as a range only and subject to change without notice. Consult Grundfos product data for <u>exact</u> pump specifications.

LRFSL-005	4/00
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Leaders in Pump Technology

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ELECTRONICS

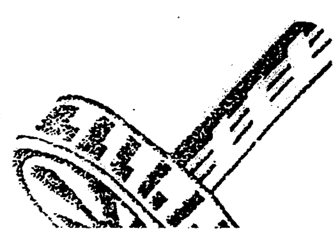
Redi-Flo3

DATA BOOK

MUNICATIONS

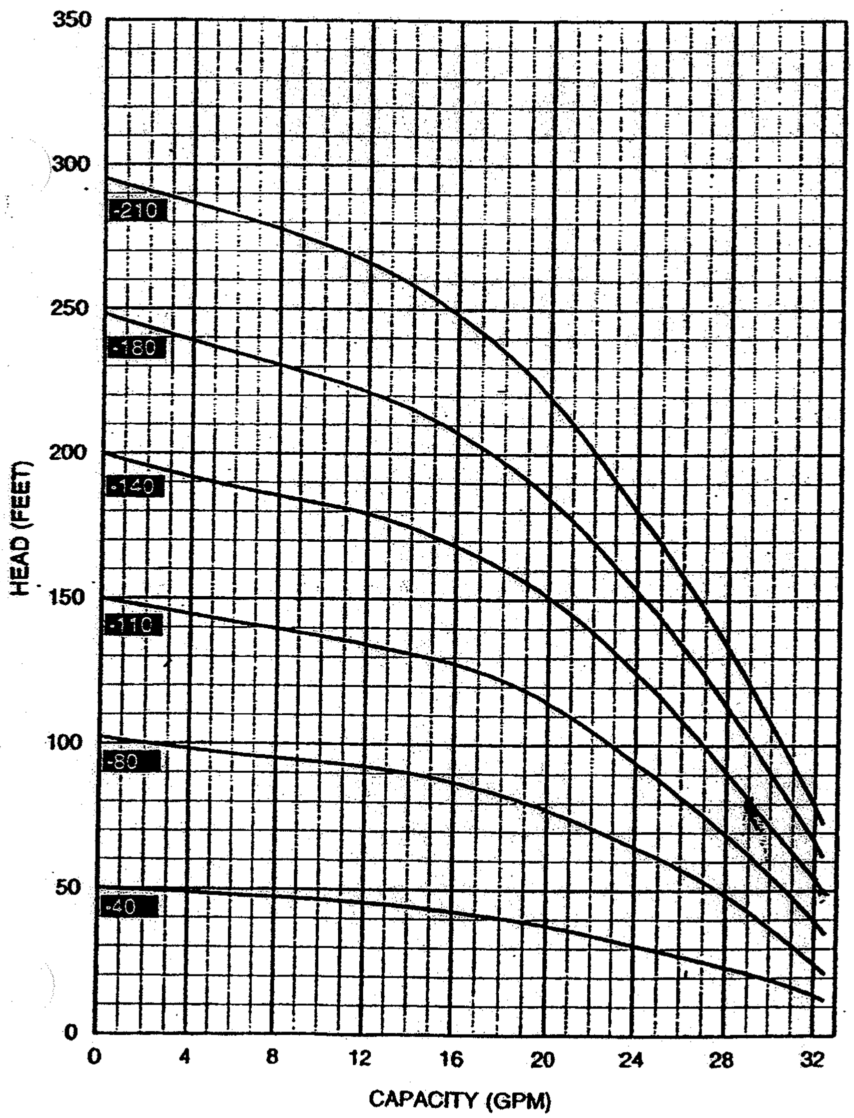
PERFORMANCE

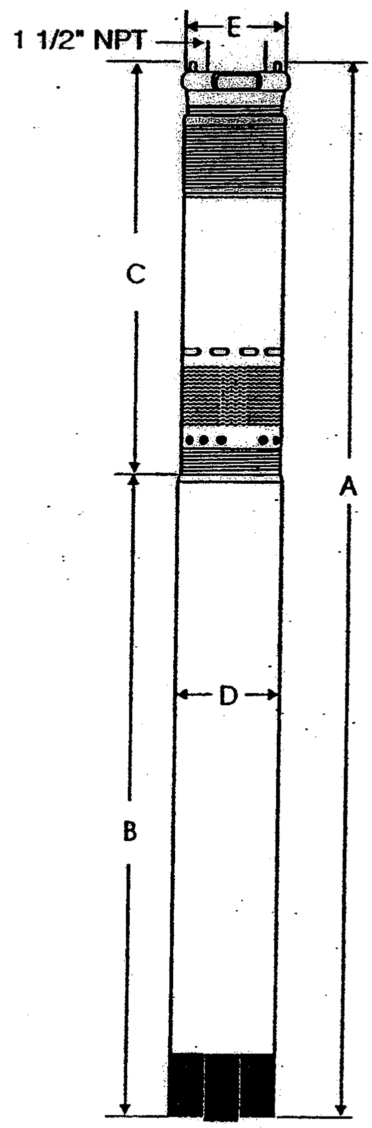
Grundfos Environmental Products Division



Model #	HP	Size	Disch. Size	Dimensions in Inches					Approx. Ship Wt.
				A	B	C	D	E	(pounds)
22Redi-Flo3-40	1/3A	3"	1 1/2" NPT	30.4	19.8	10.6	2.6	2.9	12
22Redi-Flo3-80	1/2A	3"	1 1/2" NPT	30.4	19.8	10.6	2.6	2.9	12
22Redi-Flo3-110	1/2B	3"	1 1/2" NPT	31.5	19.8	11.6	2.6	2.9	13
22Redi-Flo3-140	3/4B	3"	1 1/2" NPT	33.6	19.8	13.7	2.6	2.9	13
22Redi-Flo3-180	1C	3"	1 1/2" NPT	38.2	21.3	16.9	2.6	2.9	16
22Redi-Flo3-210	1 1/2C	3"	1 1/2" NPT	38.2	21.3	16.9	2.6	2.9	16

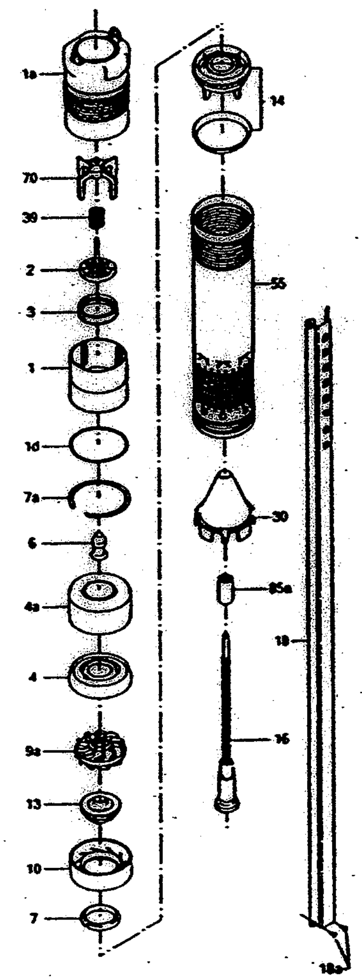
Note: Weights include pump ends with motors





ATERIAL SPECIFICATION – REDI-FLO3 PUMP END

Pos.	Component	Material	DIN W. Nr.	AISI
1	Valve Casing	PVDF		
1a	Discharge Chamber	Stainless Steel	1.4401	316
1d	O-Ring	FPM Rubber		
2	Valve Cone	PVDF		
3	Valve Seat	FPM Rubber		
4	Top Chamber	PVDF		
4a	Empty Chamber	PVDF		
6	Top Bearing	FPM Rubber		
7	Neck Ring	PVDF		
7a	Lock Ring	Stainless Steel	1.4401	316
9a	Guide Vanes	PVDF		
10	Bottom Chamber	PVDF		
13	Impeller w/ tungsten carbide bearing	PVDF		
14	Suction Interconnector	PVDF		
14a	Ring	Stainless Steel	1.4401	316
16	Shaft w/ coupling	Stainless Steel	1.4401	316
		Sintered Steel		
18	Cable Guard	Stainless Steel	1.4401	316
18a	Cable Guard Screws	Stainless Steel	1.4401	316
30	Pressure Equalization Cone	PVDF		
39	Valve Spring	Stainless Steel	1.4406	316LN
55	Pump Sleeve	Stainless Steel	1.4401	316
70	Valve Guide	PVDF		
85a	Spacer	Stainless Steel	1.4401	316

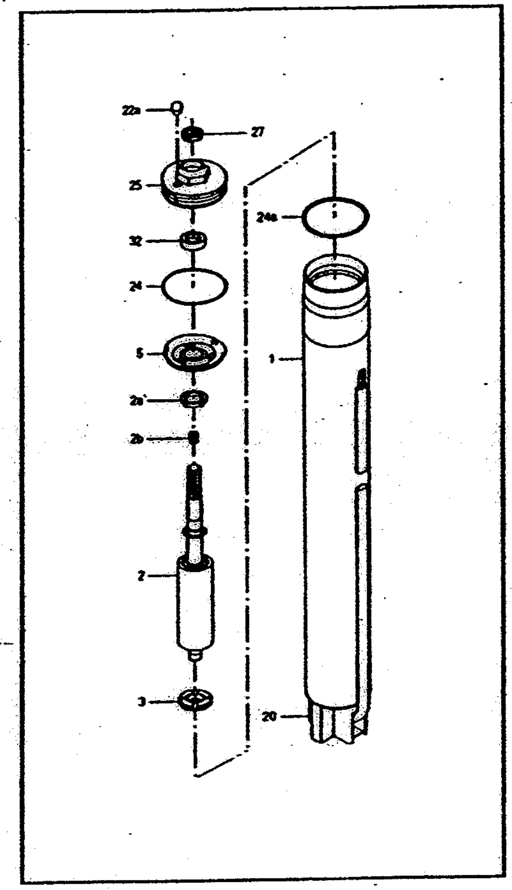


GRUNDFOS

Redi-Flo3 Technical Specifications

-RIAL SPECIFICATION – REDI-FLO3 PUMP MOTOR

Pos.	Component	Material	DIN W. Nr.	AISI
1	Stator	Stainless Steel	1.4401	316
2	Rotor	Stainless Steel	1.4401	316
2a	Stop Ring	PP		
2b	Filter	Polyester		
3	Thrust Bearing	Carbon		
5	Radial Bearing	Ceramic/ tungsten carbide		
20	Motor Cable w/ plug	Tefzel		
		PVDF		
22a	Filling Plug	FPM Rubber		
24	O-Ring	FPM Rubber		
24a	O-Ring	FPM Rubber		
25	Top Cover	PPS		
7	Filter	Polyester		
2	Shaft Seal	FPM Rubber		
	Motor Liquid	SML-2		



ELECTRIC	
Supply Voltage:	1x200-240V +6%/-10%, 50/60 Hz, PE 1x100-115V
Operation via Generator:	As a minimum, the generator output must be equal to the motor P1[KW] +10%
Starting Current:	The motor starting current is equal to the highest value stated on the motor nameplate
Starting:	Soft-start
Run-up Time:	Maximum: 2 seconds
Motor Protection:	The motor is protected against: Dry running, overvoltage, undervoltage, overload, overtemperature
Power Factor:	PF=1
Service Factor:	0.33-0.50A[Hp]-1.75 at 115V/230V 0.50-0.75B[Hp]-1.4 at 230V 1.0-1.5C[Hp]-1.15 at 230V
Motor Cable:	3 Wire, Tefzel Cable Kit
Motor Liquid:	Type SML 2
pH Values:	4-9
Liquid Temperature:	The temperature of the pumped liquid should not exceed 104°F.
Note: If liquids with a viscosity higher than that of water are to be pumped, please contact GRUNDFOS	
Minimum Ambient Temperature:	-4°F
Maximum Ambient Temperature:	+140°F
Frost Protection:	If the pump is to be stored after use, it must be stored in a frost-free location or it must be ensured that the motor liquid is frost-proof. Otherwise motor must be stored without being filled with motor liquid.
OPERATING CONDITIONS	
Minimum Ambient Fluid Temperature:	-4°F
Maximum Ambient Fluid Temperature:	+104°F
APPROXIMATE DIMENSIONS AND WEIGHT	
Motor Dimensions (MSE - NE 3):	
0.33-0.50A[Hp]	20.9" length x 2.68" diameter
0.50-0.75B[Hp]	20.9" length x 2.68" diameter
1.0-1.5C[Hp]	22.3" length x 2.68" diameter
Pump Diameter, incl. cable guard:	2.91"
Motor Weights (MSE - NE 3):	
0.33-0.50A[Hp]	6.0 lbs
0.50-0.75B[Hp]	7.1 lbs
1.0-1.5C[Hp]	8.2 lb
Pump End Dimensions:	
Pump Diameter:	2.68"
Pump Diameter, incl. cable guard:	2.91"
Pump End Dimensions (min. and max.):	
5 Redi-Flo3	10.6" to 18.0"
10 Redi-Flo3	10.6" to 16.9"
15 Redi-Flo3	10.6" to 16.9"
22 Redi-Flo3	10.6" to 16.9"
30 Redi-Flo3	10.6" to 13.7"
Pump End Weights (min. and max.):	
All	2.2 lbs to 3.5 lbs
Well Diameter (minimum):	3"
Installation Depth (maximum):	500 feet, below static water level

APPENDIX I.12

HELP MODEL

HYDROLOGICAL EVALUATION OF LANDFILL PERFORMANCE MODELING

Shaw® Shaw Environmental, Inc.

Client:	**Clinton Landfill, Inc.**
Project:	**Clinton Landfill No. 3 Chemical Waste Unit**
Proj. #:	**128017**
Calculated By:	**PCT** — **Date: 10/4/07**
Checked By:	**JPV** — **Date: 10/8/07**

TITLE: ESTIMATION OF LEACHATE HEAD ACCUMULATION

Problem Statement:

Estimate the leachate head accumulation on the bottom liner for the proposed landfill design using Hydrogeologic Evaluation of Landfill Performance (HELP) modeling based on the following landfill scenarios:

1. Daily Cover Operations (1 Year-Simulation)
2. Intermediate Cover (10 Year-Simulation)
3. Post Closure, Final Cover (30 Year-Simulation) with Leachate Collection and Removal
4. Post Closure, Final Cover (100 Year-Simulation) with No Leachate Collection and Removal

Given:

1. Landfill layers as shown in Exhibit 1.
2. HELP Model Version 3.07.
3. Soil Types Presented in Exhibit 2.

Assumptions:

1. Temperature and precipitation data synthetically generated by HELP using the monthly average values obtained from a nearby National Weather Station (Lincoln, Illinois) as reported by the National Oceanographic and Atmospheric Administration (NOAA). (Please refer to Exhibit 3).
2. Solar Radiation Data synthetically generated by HELP using the data for Chicago, Illinois and adjusted for the site latitude (please refer to Exhibit 3).
3. Length of growing season data was obtained from a nearby National Weather Station (Lincoln, Illinois) as reported by NOAA (please refer to Exhibit 3).
4. Wind speed data was obtained from a nearby National Weather Station (Lincoln, Illinois) as reported by NOAA (please refer to Exhibit 3).
5. Relative humidity was obtained from from a nearby National Weather Station (Lincoln, Illinois) as reported by NOAA (please refer to Exhibit 3).

6. The 'HELP' modeling results are independent of the landfill area. One (1) acre area was considered for the analysis.

7. The HELP Model default Texture No. 35 was used to represent the High Density Polyethylene (HDPE) liners in the bottom liner system and the for the liner used in the final cover system (please refer to Exhibit 2).

8. HDPE Geomembrane characteristics: Bottom and Final Cover Liners
Pinhole density = 1 hole per acre;
Installation defects = 10 holes per acre;
Placement Quality = 4 (Poor).

Note: A Construction Quality Assurance (CQA) program has been developed for the facility which addresses proper installation of the geomembrane liner and cover. The assumed placement quality for the final cover geomembrane is unlikely with the proper implementation of the facility's CQA plan, and represents a conservative condition.

10. The hydraulic conductivity (k) of the Final Cover Soil Barrier layer was conservatively assumed to be 1×10^{-5} cm/sec in order to account for possible dessication cracks and settlement that may occur over the long term period. Note however, the Design Report (Section 3) and the Construction Quality Assurance Plan (Section 5) require that the Final Cover Soil Barrier layer be constructed to have a maximum hydraulic conductivity of 1×10^{-7} cm/sec.

11. Compacted Earth Liner layer assumed to be saturated.

12. The HELP Model default Texture No. 18 was used to represent the Municipal Solid Waste, and default Texture No. 9 was used to represent the Chemical Waste (please refer to Exhibit 2).

13. The HELP Model defines field capacity as the soil water storage/volumetric content after a prolonged period of gravity drainage from saturation corresponding to the soil water storage when a soil exerts a soil suction of 1/3 bar.

14. The initial water content of the chemical waste for Scenario 1 - Daily Cover, was assumed equal to 71%, and was based on an average of laboratory moisture content test values (please refer to Appendix I).

15. Groundwater seepage was assumed through the bottom compacted earth liner (Layer 9 in the Operational Period Scenarios - 1 and 2, and Layer 16 in the Post Closure Period - Scenarios 3 and 4). The groundwater seepage rate was calculated to be

0.2476 inches/year (please refer to Exhibit 4).

16. The HELP Model gives results in inches/time period but all results were converted to gal/acre/day using conversion factors listed below:

$$\frac{\#\text{ inches}}{\text{time period}} \times \frac{1\text{ foot}}{12\text{ inches}} \times \frac{43{,}560\text{ ft}^2}{\text{acre}} \times \frac{7.48\text{ gallons}}{1\text{ ft}^3} \times \frac{\text{time period}}{\#\text{ days}}$$

HELP Model Scenarios:

Scenario 1 - Daily Cover Operations

The daily cover operations were modeled as a 10-foot thick layer of waste covered by 6 inches of daily cover material, as shown in Exhibit 1. This scenario was modeled for one year which is overly conservative, because an intermediate cover must be placed down on all surfaces of the landfill where no additional waste will be deposited within 60 days. Other assumptions used in running the 'HELP' model are listed below.

1. Bare ground condition was used for the operation runs with daily cover. The corresponding Maximum Leaf Index is 0.0 (refer to Exhibit 3).

2. The 'HELP' default (Chicago, Illinois) Evaporative Zone Depth is 6 inches corresponding to bare ground condition.

3. No runoff was allowed during the operational period with daily cover. The SCS curve number input for the operational simulations with daily cover was interpolated to be 93.1 as shown in Exhibit 4.

4. The initial water content of the leachate collection system and daily cover material were set at the field capacity of the respective materials.

Scenario 1 - Daily Cover Results

Exhibit 6 includes the Scenario 1 Daily Cover 'HELP' simulations, the table below summarizes the results.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 10/4/07
Checked By: JPV Date: 10/8/07

TITLE: ESTIMATION OF LEACHATE HEAD ACCUMULATION

Scenario 1 - Daily Cover Initial & Final Water Contents			
Layer No.	Layer	Initial Water Content (%)	Final Water Content (%)
2	Chemical Waste	71.00	72.93
3	Leachate Drainage Layer	4.50	6.73
9	Compacted Earth Liner	42.70	42.70

Scenario 1 - Daily Cover Leachate Generation Results			
Peak Daily Leachate Generation (gal/acre/day)	Average High Monthly Leachate Generation (gal/acre/day)	Average Annual Leachate Generation (gal/acre/day)	Peak Daily Leachate Head (inches)
1,385.59	675.66	474.60	3.319

Scenario 2 - Intermediate Cover

Exhibit 1 shows the cross section of the landfill during the intermediate cover period. The maximum thickness of the chemical waste was estimated to be 140 feet with an overlying 12-inch layer of intermediate cover. The initial waste water content was set at 72.93% which was the final waste water content at the end of the daily cover scenario. This value is conservative because the daily cover scenario was run for 1 year instead of 60 days. By running the HELP Model for a full year more water is allowed to enter the waste. The intermediate cover simulation was run for a period of 10 years. Other assumptions used in running the 'HELP' Model are listed below.

1. Poor vegetation conditions were assumed for the intermediate cover simulation. The Maximum Leaf Area Index corresponding to poor vegetation condition is 1.0 (please refer to Exhibit 3).

2. The 'HELP' default (Chicago, Illinois) Evaporative Zone Depth is 14 inches corresponding to poor vegetation condition.

3. 75% of the runoff was allowed during the intermediate period. This is a conservative approach. A more likely scenario would be to use 100% for intermediate cover scenario. The SCS number input for the operational simulation with intermediate cover was interpolated to be 89.1 as shown in Exhibit 5.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 10/4/07
Checked By: JPV Date: 10/8/07

TITLE: ESTIMATION OF LEACHATE HEAD ACCUMULATION

4. The initial water contents for the chemical waste, leachate drainage layer, and compacted earth liner were set equal to the final water contents (Water Content = %) of the respective layers from the daily cover scenario.

Scenario 2 - Intermediate Cover Results

Exhibit 7 includes the Scenario 2 Intermediate Cover 'HELP' simulations, the tables below summarizes the results.

Scenario 2 - Intermediate Cover Initial & Final Water Contents			
Layer No.	Layer	Initial Water Content (%)	Final Water Content (%)
2	Chemical Waste	72.93	70.83
3	Leachate Drainage Layer	6.73	6.42
9	Compacted Earth Liner	42.70	42.70

Scenario 2 - Intermediate Cover Leachate Generation Results			
Peak Daily Leachate Generation (gal/acre/day)	Average High Monthly Leachate Generation (gal/acre/day)	Average Annual Leachate Generation (gal/acre/day)	Peak Daily Leachate Head (inches)
1,670.96	605.50	577.66	3.929

Scenario 3 - Post Closure Period: Yrs. 1- 30 (with Leachate Collection & Removal)

This simulation was run for post closure period years 1 through 30 with leachate collection and removal. Leachate collection and removal for years 1-30 was simulated by modeling the sand layer underneath the waste as a lateral drainage layer. Other assumptions used in running the 'HELP' model are listed below.

1. Good vegetation condition was assumed for the post closure period. The Maximum Leaf Area Index corresponding to good vegetation condition is 4.0 (refer to Exhibit 3).

2. The HELP default (Chicago, Illinois) Evaporative Zone Depth is 20 inches corresponding to good vegetation condition.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 10/4/07
Checked By: JPV Date: 10/8/07

TITLE: ESTIMATION OF LEACHATE HEAD ACCUMULATION

3. The initial water contents for the chemical waste, leachate drainage layer, and compacted earth liner were set equal to the final water contents (Water Content = %) of the respective layers from the intermediate cover scenario. The initial water content for the MSW was set equal to the field capacity value of 29.20%.

2. The Final Cover Soil Barrier layer was modeled with a hydraulic conductivity (k) of 1 x 10^{-5} cm/sec in order to be conservative and to account for possible dessication cracks and settlement that may occur over the long term period. Note however, the Design and the Construction Quality Assurance Plan require that the Final Cover Soil Barrier layer be constructed to have a hydraulic conductivity of 1 x 10^{-7} cm/sec.

3. The final land form simulation utilized a maximum thickness of 140 feet for the chemical waste, with an overlying 12-inch separation layer, and an 11.1-foot layer of municipal solid waste above (please refer to Exhibit 1). These thicknesses correspond to the location within the Chemical Waste Unit at which the chemical waste is at a peak or maximum thickness (140 feet).

4. 100% of the runoff was allowed during the post closure operations.

5. The SCS number was computed by 'HELP' based on surface slope, slope length, soil texture, and quantity of vegetative cover (assumed good stand of grass and simulation value of 4 for vegetative cover).

Scenario 3 - Post Closure Period: Yrs. 1- 30 Results

Exhibit 8 includes the post closure period 'HELP' simulations. The table below summarizes the results.

Scenario 3 - Post Closure Period: Years 1-30 Initial & Final Water Contents			
Layer No.	**Layer**	**Initial Water Content (%)**	**Final Water Content (%)**
7	MSW	29.20	29.20
9	Chemical Waste	70.83	61.26
10	Leachate Drainage Layer	6.42	5.88
16	Compacted Earth Liner	42.70	42.70

Client:	Clinton Landfill, Inc.
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
Calculated By: PCT	Date: 10/4/07
Checked By: JPV	Date: 10/8/07

Shaw® Shaw Environmental, Inc.

TITLE: ESTIMATION OF LEACHATE HEAD ACCUMULATION

Scenario 3 - Post Closure Period: Years 1 - 30 Leachate Generation Results			
Peak Daily Leachate Generation (gal/acre/day)	Average High Monthly Leachate Generation (gal/acre/day)	Average Annual Leachate Generation (gal/acre/day)	Peak Daily Leachate Head (inches)
1,421.70	405.10	398.94	3.396

The leachate collection system has been designed to maintain a maximum leachate head of less than 12 inches under final cover conditions.

Scenario 4 - Post Closure Period: Yrs. 31-131 (with No Leachate Collection & Removal)

This simulation was run for post closure period years 31 through 131 with no leachate collection and removal. No leachate collection and removal was simulated for years 31-131 by modeling the sand layer underneath the waste as a vertical percolation layer. Other assumptions used in running the 'HELP' model are listed below.

1. Good vegetation condition was assumed for the post closure period. The Maximum Leaf Area Index corresponding to good vegetation condition is 4.0 (refer to Exhibit 3).

2. The HELP default (Chicago, Illinois) Evaporative Zone Depth is 20 inches corresponding to good vegetation condition.

3. The initial water contents for the waste and soil layers, as presented in the table below, were set equal to the final water contents (Water Content = %) of the respective layers from the post closure period, years 1-30 scenario.

2. The Final Cover Soil Barrier layer was modeled with a hydraulic conductivity (k) of 1 x 10^{-5} cm/sec in order to be conservative and to account for possible dessication cracks and settlement that may occur over the long term period. Note however, the Design and the Construction Quality Assurance Plan require that the Final Cover Soil Barrier layer be constructed to have a hydraulic conductivity of 1 x 10^{-7} cm/sec.

3. The final land form simulation utilized a maximum thickness of 140 feet for the chemical waste, with an overlying 12-inch intermediate cover layer, and an 11.1-foot layer of municipal solid waste above (please refer to Exhibit 1). These thicknesses correspond to the location within the Chemical Waste Unit at which the chemical waste is at a peak or maximum thickness (140 feet).

4. 100% of the runoff was allowed during the post closure operations.

Client:	Clinton Landfill, Inc.			
Project:	Clinton Landfill No. 3 Chemical Waste Unit			
Proj. #:	128017			
Calculated By:		PCT	Date:	10/4/07
Checked By:		JPV	Date:	10/8/07

Shaw® Shaw Environmental, Inc.

TITLE: ESTIMATION OF LEACHATE HEAD ACCUMULATION

5. The SCS number was computed by 'HELP' based on surface slope, slope length, soil texture, and quantity of vegetative cover (assumed good stand of grass and simulation value of 4 for vegetative cover).

Scenario 4 - Post Closure Period Yrs. 31-131 (Steady State Conditions with <u>No</u> Leachate Collection & Removal) Results

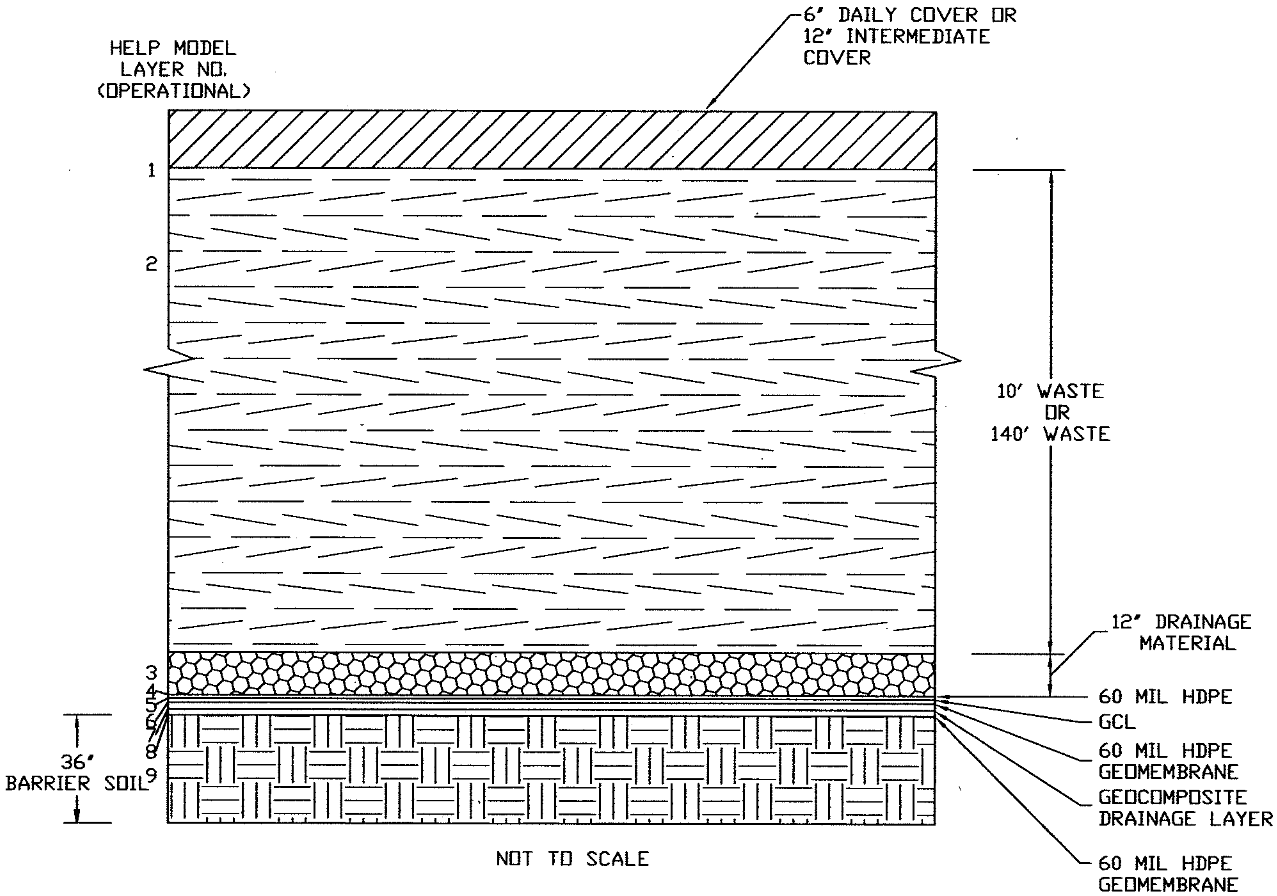
Exhibit 9 includes the post closure period 'HELP' simulations. The table below summarizes the results.

Scenario 4 - Post Closure Period: Years 31-131 Initial & Final Water Contents			
Layer No.	Layer	Initial Water Content (%)	Final Water Content (%)
1	Vegetative Layer	15.20	42.12
2	Protective Layer	23.83	29.25
5	Compacted Cohesive Soil Liner	42.70	42.70
6	Foundation Layer	31.00	31.00
7	MSW	29.20	29.20
8	Separation Layer	31.00	31.00
9	Chemical Waste	61.26	61.26
10	Leachate Drainage Layer	5.88	5.60
16	Compacted Earth Liner	42.70	42.70

Scenario 4 - Post Closure Period: Years 31 - 131 Leachate Generation Results	Peak Daily Average Head (inches)
	0.445

The leachate collection system has been designed to maintain a maximum leachate head of less than 12 inches under final cover conditions. In addition, 130 year after closure, the primary geomembrane will have less than 1 inch of leachate head.

EXHIBIT 1
LANDFILL CROSS SECTION SCHEMATICS



NOT TO SCALE

Shaw® Shaw Environmental, Inc.

CLINTON LF. NO. 3 CHEMICAL WASTE UNIT
DEWITT COUNTY, ILLINOIS

LANDFILL CROSS SECTION - SCHEMATIC
OPERATIONAL PERIODS
DAILY & INTERMEDIATE COVER

APPROVED BY:	DAM	PROJ. NO.:	128017	DATE:	OCT. 2007

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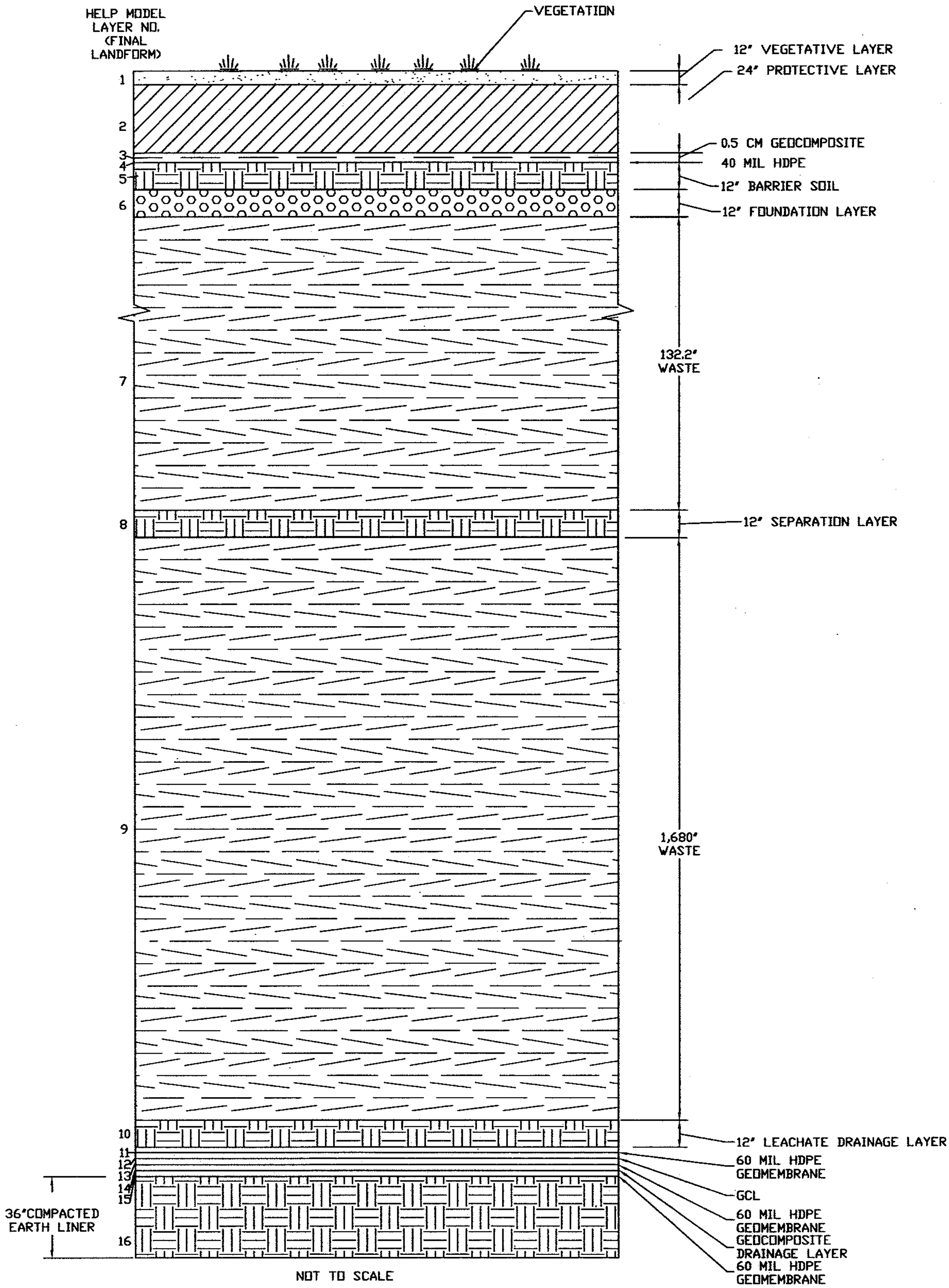

Shaw® Shaw Environmental, Inc.

CLINTON LF. NO. 3 CHEMICAL WASTE UNIT
DEWITT COUNTY, ILLINOIS

LANDFILL CROSS SECTION - SCHEMATIC
POST-CLOSURE PERIODS

APPROVED BY: DAM	PROJ. NO.: 128017	DATE: OCT. 2007

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EXHIBIT 2
SOIL TYPES - 'HELP' MODELING

Client:	Clinton Landfill, Inc.
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
Calculated By: PCT	Date: 10/5/07
Checked By: JPV	Date: 10/9/07

Shaw® Shaw Environmental, Inc.

TITLE: LAYER TYPES AND DEFAULT TEXTURES FOR HELP MODELING

Purpose:

Select the appropriate HELP odel default textures / soil properties for model input.

Operational Period Layers:

1. The previous Exhibit (Exhibit 1) in this Appendix shows the landfill layers for the operational simulation.

2. This Exhibit shows the HELP default classification characteristics. The following table summarizes the appropriate HELP default textures for the various layers, the layer types, and their respective thicknesses:

HELP Model Operational Period Material Layers					
				HELP Model Material Classification / Texture No.	
Layer No.	Layer Type	Layer Thickness (in.)	Layer Description	Classification	Texture No.
Waste					
1	1	6 or 12	Daily Cover or Intermediate Cover	CL	11
2	1	120 or 1,680	Chemical Waste	ML	9
Leachate Drainage & Bottom Liner System					
3	2	12	Leachate Drainage Layer ($k \geq 3.0 \times 10^{-2}$ cm/sec)	SP	1
4	4	0.06	60-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
5	1	0.2362	Geosynthetic Clay Liner (GCL) ($k \geq 3.0 \times 10^{-9}$ cm/sec)	Bentonite Mat (0.6 cm)	17
6	4	0.06	60-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
7	2	0.200	Geocomposite Drainage Layer	Drainage Net (0.5 cm)	20
8	4	0.06	60-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
9	3	36	Compacted Earth (Cohesive Soil) Liner ($k \leq 1 \times 10^{-7}$ cm/sec)	Barrier Soil	16

Post Closure Period Layers:

1. The previous Exhibit (Exhibit 1) in this Appendix shows the landfill layers for the post closure simulation.

2. This Exhibit shows the HELP default classification characteristics. The following

Client:	Clinton Landfill, Inc.
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
Calculated By:	PCT Date: 10/5/07
Checked By:	JPV Date: 10/9/07

Shaw® Shaw Environmental, Inc.

TITLE: LAYER TYPES AND DEFAULT TEXTURES FOR HELP MODELING

table summarizes the appropriate HELP default textures for the various layers, the layer types, and their respective thicknesses:

HELP Model Post Closure Period Material Layers					
				HELP Model Material Classification / Texture No.	
Layer No.	Layer Type	Layer Thickness (in.)	Layer Description	Classification	Texture No.
Final Cover System					
1	1	12	Vegetative Layer	ML	8
2	1	24	Protective Layer	ML	9
3	2	0.200	Geocomposite Drainage Layer	Drainage Net (0.5 cm)	20
4	4	0.04	40-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
5	3	12	Compacted Cohesive Soil Liner (see note below)	Barrier Soil	16
6	1	12	Foundation Layer	CL	11
Waste					
7	1	133.2	Municipal Solid Waste	Municipal Waste	18
8	1	12	Separation Layer	CL	11
9	1	1,680	Chemical Waste	ML	9
Leachate Drainage & Bottom Liner System					
10	1 or 2	12	Leachate Drainage Layer ($k \geq 3.0 \times 10^{-2}$ cm/sec)	SP	1
11	4	0.06	60-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
12	1	0.2362	Geosynthetic Clay Liner (GCL) ($k \geq 3.0 \times 10^{-9}$ cm/sec)	Bentonite Mat (0.6 cm)	17
13	4	0.06	60-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
14	1 or 2	0.200	Geocomposite Drainage Layer	Drainage Net (0.5 cm)	20
15	4	0.06	60-mil HDPE Geomembrane ($k \geq 2.0 \times 10^{-13}$ cm/sec)	High Density Polyethylene (HDPE)	35
16	3	36	Compacted Earth (Cohesive Soil) Liner ($k \leq 1 \times 10^{-7}$ cm/sec)	Barrier Soil	16

Note: The Final Cover Soil Barrier layer was modeled with a hydraulic conductivity (k) of 1×10^{-5} cm/sec in order to be conservative and to account for possible dessication cracks and settlement that may occur over the long term period. Note however, the Design Report (Section 3) and the Construction Quality Assurance Plan (Section 5) require that the Final Cover Soil Barrier layer be constructed to have a hydraulic conductivity of 1×10^{-7} cm/sec.

United States
Environmental Protection
Agency

Office of Research and
Development
Washington DC 20460

EPA/600/R-94/168a
September 1994

EPA

The Hydrologic Evaluation of Landfill Performance (HELP) Model

User's Guide for Version 3

TABLE 4. DEFAULT SOIL, WASTE, AND GEOSYNTHETIC CHARACTERISTICS

Classification			Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity
HELP	USDA	USCS	vol/vol	vol/vol	vol/vol	cm/sec
1	CoS	SP	0.417	0.045	0.018	1.0×10^{-2}
2	S	SW	0.437	0.062	0.024	5.8×10^{-3}
3	FS	SW	0.457	0.083	0.033	3.1×10^{-3}
4	LS	SM	0.437	0.105	0.047	1.7×10^{-3}
5	LFS	SM	0.457	0.131	0.058	1.0×10^{-3}
6	SL	SM	0.453	0.190	0.085	7.2×10^{-4}
7	FSL	SM	0.473	0.222	0.104	5.2×10^{-4}
8	L	ML	0.463	0.232	0.116	3.7×10^{-4}
9	SiL	ML	0.501	0.284	0.135	1.9×10^{-4}
10	SCL	SC	0.398	0.244	0.136	1.2×10^{-4}
11	CL	CL	0.464	0.310	0.187	6.4×10^{-5}
12	SiCL	CL	0.471	0.342	0.210	4.2×10^{-5}
13	SC	SC	0.430	0.321	0.221	3.3×10^{-5}
14	SiC	CH	0.479	0.371	0.251	2.5×10^{-5}
15	C	CH	0.475	0.378	0.265	1.7×10^{-5}
16	Barrier Soil		0.427	0.418	0.367	1.0×10^{-7}
17	Bentonite Mat (0.6 cm)		0.750	0.747	0.400	3.0×10^{-9}
18	Municipal Waste (900 lb/yd^3 or 312 kg/m^3)		0.671	0.292	0.077	1.0×10^{-3}
19	Municipal Waste (channeling and dead zones)		0.168	0.073	0.019	1.0×10^{-3}
20	Drainage Net (0.5 cm)		0.850	0.010	0.005	$1.0 \times 10^{+1}$
21	Gravel		0.397	0.032	0.013	3.0×10^{-1}
22	L*	ML	0.419	0.307	0.180	1.9×10^{-5}
23	SiL*	ML	0.461	0.360	0.203	9.0×10^{-6}
24	SCL*	SC	0.365	0.305	0.202	2.7×10^{-6}
25	CL*	CL	0.437	0.373	0.266	3.6×10^{-6}
26	SiCL*	CL	0.445	0.393	0.277	1.9×10^{-6}
27	SC*	SC	0.400	0.366	0.288	7.8×10^{-7}
28	SiC*	CH	0.452	0.411	0.311	1.2×10^{-6}
29	C*	CH	0.451	0.419	0.332	6.8×10^{-7}
30	Coal-Burning Electric Plant Fly Ash*		0.541	0.187	0.047	5.0×10^{-5}
31	Coal-Burning Electric Plant Bottom Ash*		0.578	0.076	0.025	4.1×10^{-3}
32	Municipal Incinerator Fly Ash*		0.450	0.116	0.049	1.0×10^{-2}
33	Fine Copper Slag*		0.375	0.055	0.020	4.1×10^{-2}
34	Drainage Net (0.6 cm)		0.850	0.010	0.005	$3.3 \times 10^{+1}$

* Moderately Compacted

(Continued)

TABLE 4 (continued). DEFAULT SOIL, WASTE, AND GEOSYNTHETIC CHARACTERISTICS

Classification		Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity
HELP	Geomembrane Material	vol/vol	vol/vol	vol/vol	cm/sec
35	High Density Polyethylene (HDPE)				2.0×10^{-13}
36	Low Density Polyethylene (LDPE)				4.0×10^{-13}
37	Polyvinyl Chloride (PVC)				2.0×10^{-11}
38	Butyl Rubber				1.0×10^{-12}
39	Chlorinated Polyethylene (CPE)				4.0×10^{-12}
40	Hypalon or Chlorosulfonated Polyethylene (CSPE)				3.0×10^{-12}
41	Ethylene-Propylene Diene Monomer (EPDM)				2.0×10^{-12}
42	Neoprene				3.0×10^{-12}

(concluded)

user-defined soil option accepts non-default soil characteristics for layers assigned soil type numbers greater than 42. This is especially convenient for specifying characteristics of waste layers. User-specified soil characteristics can be assigned any soil type number greater than 42.

When a default soil type is used to describe the top soil layer, the program adjusts the saturated hydraulic conductivities of the soils in the top half of the evaporative zone for the effects of root channels. The saturated hydraulic conductivity value is multiplied by an empirical factor that is computed as a function of the user-specified maximum leaf area index. Example values of this factor are 1.0 for a maximum LAI of 0 (bare ground), 1.8 for a maximum LAI of 1 (poor stand of grass), 3.0 for a maximum LAI of 2 (fair stand of grass), 4.2 for a maximum LAI of 3.3 (good stand of grass) and 5.0 for a maximum LAI of 5 (excellent stand of grass).

The manual option requires values for porosity, field capacity, wilting point, and saturated hydraulic conductivity. These and related soil properties are defined below.

Soil Water Storage (Volumetric Content): the ratio of the volume of water in a soil to the total volume occupied by the soil, water and voids.

Total Porosity: the soil water storage/volumetric content at saturation (fraction of total volume).

EXHIBIT 3
WEATHER INPUT DATA

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.
Project: Clinton LF No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: JPV Date: 10/4/07
Checked By: PCT Date: 10/4/07

TITLE: WEATHER INPUT PARAMETERS FOR HELP MODELING

Purpose:

Select appropriate weather input parameters for use in HELP modeling.

Solution:

1. Temperature and precipitation data were based on mean monthly precipitation and temperature data from a nearby National Weather Service station (Lincoln, Illinois) as reported by the National Oceanographic and Atmospheric Administration (NOAA).

2. Normal Mean Monthly Temperature and Precipitation Values for the period of 1950 - 2000.

Month	Normal Mean Temperature (^0F)	Normal Mean Precipitation (inches)
January	24.80	1.86
February	30.00	1.65
March	39.90	2.93
April	52.50	3.99
May	63.50	4.09
June	72.50	4.24
July	75.80	4.14
August	73.60	3.71
September	66.70	3.04
October	55.10	2.74
November	41.60	2.96
December	30.00	2.40

3. Normal Average Annual Wind Speed = 10.30 mph.

4. Normal Average Monthly and Quarterly Relative Humidity Values.

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.
Project: Clinton LF No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: JPV Date: 10/4/07
Checked By: PCT Date: 10/4/07

TITLE: WEATHER INPUT PARAMETERS FOR HELP MODELING

Quarter	Normal Average Quarterly Relative Humidity %
1st	71
2nd	65
3rd	70
4th	72

5. The Growing Season Start and End Dates and is represented by the Julian Dates (day of the year):

   - Growing Season Start Date = 117
   - Growing Season End Date = 290

6. The Site Latitude was taken from USGS topographic map and is approximately 40.10°.

7. Maximum Leaf Area index is 0.0 for bare ground condition; 1.0 for poor vegetation condition; and 4.0 for good vegetation condition (refer to HELP documentation included in this Exhibit).

EXHIBIT 4
GROUNDWATER SEEPAGE CALCULATION

Shaw® Shaw Environmental, Inc.

Client:	Clinton Landfill, Inc.
Project:	Clinton LF. No. 3 Chemical Waste Unit
Calculated By: PCT	Date: 10/02/07
Checked By: JPV	Date: 10/08/07

Title: GROUNDWATER SEEPAGE

Problem Statement:

Calculate inward leakage through the composite liner (CSL) design based on Giroud et al. (1989).

Given:

1) Inward gradient with 27 feet of head on the liner during operational and post-closure periods (assumed maximum potentiometric elevation of 691.4 ft.MSL (measured in Well EX-4 in Nov. 2004) and top of leachate drainage layer at lowest elevation equal to 664 ft.MSL (assumes drainage layer completely saturated, see Drawing D6):
   $\rightarrow \therefore$ 691.4 ft. - 664 ft. = 27.4 ft.
2) Poor contact between the geomembrane and the compacted earth liner is considered.
3) The rate of leakage was calculated using the equation: where $Q = (0.0008) * (X^{0.9176})$; source for the equation *is Giroud and Bonaparte (1989): Leakage Through Liner Constructed With Geomembrane-Part II (VI, pp. 71-111).*

Solution:

The rate of leakage through geomembrane defect (Q) expressed in $m^3/s/$ acre =

$Q = (0.0008) \bullet (X^{0.9176})$ (Source: IEPA based on Giroud and Bonaparte (1989))

Where,

X = Leachate depth on top of top of the geomembrane (m)

Q = Flow rate or leakage rate (m/yr) = $(0.0008) \bullet (X^{0.9176})$

Calculation:

X = Leachate depth on top of top of the geomembrane = 27.4 ft. = 8.3515 m

Q = Rate of leakage = $(0.0008) \bullet (X^{0.9176})$ = 6.29E-3 m/yr
= 0.0206 ft/yr
= 0.2476 in/yr

EXHIBIT 5
SCS CURVE NUMBER

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: PCT **Date:** 10/4/07
Checked By: JPV **Date:** 10/8/07

TITLE: SCS CURVE NUMBER FOR HELP MODELING - OPERATIONAL PERIODS

Purpose

The uppermost layers for the operational 'HELP' simulations are daily cover and intermediate cover. Estimate the runoff curve number for HELP model input.

Solution

1. The HELP default Texture No. 11 is assumed to represent the daily cover soil material and bare ground conditions were considered for the daily cover simulations. Using the chart presented on page 36 of the HELP v.3 User's Guide (see attached), an SCS Runoff Curve Number of 93.1 was interpolated for Texture No. 11 soils and bare ground conditions.

2. The HELP default Texture No. 11 is assumed to represent the intermediate cover soil material and poor vegetative conditions were considered for the intermediate cover simulations. Using the same chart as above (see attached), an SCS Runoff Curve Number of 89.1 was interpolated for Texture No. 11 soils and poor vegetative conditions.

Results

1. For the daily cover with bare ground conditions, SCS Runoff Curve Number Interpolated = 93.1.

2. For the intermediate cover with poor vegetative conditions, SCS Runoff Curve Number Interpolated = 89.1.

United States
Environmental Protection
Agency

Office of Research and
Development
Washington DC 20460

EPA/600/R-94/168a
September 1994

EPA

The Hydrologic Evaluation of Landfill Performance (HELP) Model

User's Guide for Version 3

2. A curve number defined by the user and modified according to the surface slope and slope length of the landfill

3. A curve number is computed by the HELP model based on landfill surface slope, slope length, soil texture of the top layer, and the vegetative cover. Some general guidance for selection of runoff curve numbers is provided in Figure 2 (USDA, Soil Conservation Service, 1985).

Two of the options account for surface slope. The correlation between surface slope conditions and curve number were developed for slopes ranging from 1 percent to as high as 50 percent and for slope lengths ranging from 50 feet to 2000 feet.

3.8 OVERVIEW OF MODELING PROCEDURE

The hydrologic processes modeled by the program can be divided into two categories: surface processes and subsurface processes. The surface processes modeled are snowmelt, interception of rainfall by vegetation, surface runoff, and surface evaporation. The subsurface processes modeled are evaporation from soil profile, plant transpiration, unsaturated vertical drainage, barrier soil liner percolation, geomembrane leakage and saturated lateral drainage.

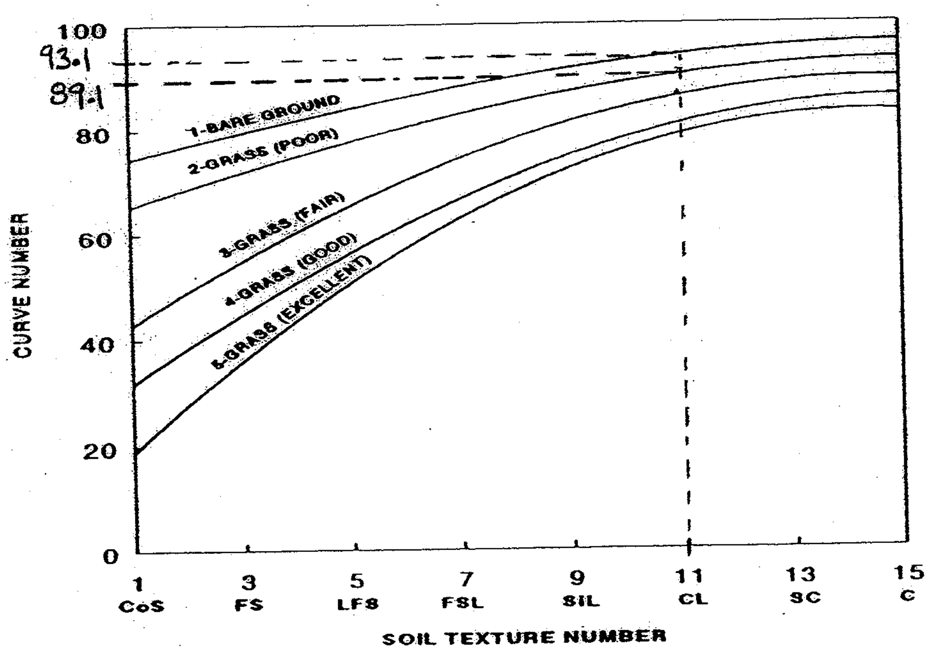

Figure 2. Relation between SCS Curve Number and Default Soil Texture Number for Various Levels of Vegetation

EXHIBIT 6
'HELP' MODEL RESULTS
DAILY COVER OPERATIONS

```
******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     c:\HELP\PRECIP.D4
TEMPERATURE DATA FILE:       C:\HELP\TEMP.D7
SOLAR RADIATION DATA FILE:   C:\HELP\SOLAR.D13
EVAPOTRANSPIRATION DATA:     C:\HELP\EVAP_DC.D11
SOIL AND DESIGN DATA FILE:   C:\HELP\SOIL_DC.D10
OUTPUT DATA FILE:            C:\HELP\CLIN_DC.OUT


TIME:  12:23      DATE:  10/ 8/2007


******************************************************************************

     TITLE:  CLINTON LF. NO.3 CHEMICAL WASTE UNIT (DAILY COVER 1 YR.)

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.


                                  LAYER  1
                                  --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER  11
          THICKNESS                        =       6.00    INCHES
          POROSITY                         =       0.4640 VOL/VOL
          FIELD CAPACITY                   =       0.3100 VOL/VOL
          WILTING POINT                    =       0.1870 VOL/VOL
          INITIAL SOIL WATER CONTENT       =       0.3100 VOL/VOL
          EFFECTIVE SAT. HYD. COND.        =   0.639999998000E-04 CM/SEC


                                  LAYER  2
```

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	120.00 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.8000 VOL/VOL
WILTING POINT	=	0.1350 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7100 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.190000006000E-03 CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4170 VOL/VOL
FIELD CAPACITY	=	0.0450 VOL/VOL
WILTING POINT	=	0.0180 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0450 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.299999993000E-01 CM/SEC
SLOPE	=	2.66 PERCENT
DRAINAGE LENGTH	=	170.0 FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7470 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

LAYER 6

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 7

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC
SLOPE	=	2.66 PERCENT
DRAINAGE LENGTH	=	170.0 FEET

LAYER 8

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 9

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 16

THICKNESS	=	36.00	INCHES
POROSITY	=	0.4270	VOL/VOL
FIELD CAPACITY	=	0.4180	VOL/VOL
WILTING POINT	=	0.3670	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4270	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000001000E-06	CM/SEC
SUBSURFACE INFLOW	=	0.25	INCHES/YR

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	93.10	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	1.860	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.784	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.122	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	103.150	INCHES
TOTAL INITIAL WATER	=	103.150	INCHES
TOTAL SUBSURFACE INFLOW	=	0.25	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
Lincoln Illinois

STATION LATITUDE	=	40.10	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	117	
END OF GROWING SEASON (JULIAN DATE)	=	290	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	10.30	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	71.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	65.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	70.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	72.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC

1.60	1.31	2.59	3.66	3.15	4.08
3.63	3.53	3.35	2.28	2.06	2.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
21.40	26.00	36.00	48.80	59.10	68.60
73.00	71.90	64.70	53.50	39.80	27.70

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS
AND STATION LATITUDE = 40.10 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.45	110533.531	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	20.809	75537.781	68.34
DRAINAGE COLLECTED FROM LAYER 3	6.3799	23158.877	20.95
PERC./LEAKAGE THROUGH LAYER 4	0.000807	2.930	0.00
AVG. HEAD ON TOP OF LAYER 4	0.6565		
PERC./LEAKAGE THROUGH LAYER 6	0.000368	1.336	0.00
AVG. HEAD ON TOP OF LAYER 6	0.0815		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.81
DRAINAGE COLLECTED FROM LAYER 7	0.2480	900.124	0.81
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	3.260	11835.497	10.71
SOIL WATER AT START OF YEAR	103.503	375716.562	
SOIL WATER AT END OF YEAR	106.764	387552.062	

SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.035	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 1

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.66	0.58	2.21	3.11	1.87	4.54
	3.84	3.56	3.15	2.02	2.10	2.81
STD. DEVIATIONS	0.00	0.00	0.00	0.00	0.00	0.00
	0.00	0.00	0.00	0.00	0.00	0.00
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.364	0.300	0.670	3.008	1.564	4.079
	2.441	3.051	1.803	2.092	0.800	0.639
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.7714	0.4424	0.4972	0.4992	0.5128	0.4917
	0.5183	0.5257	0.5149	0.5488	0.5073	0.5502
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

SUBSURFACE INFLOW INTO LAYER 9

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LATERAL DRAINAGE COLLECTED FROM LAYER 7

TOTALS	0.0211	0.0190	0.0211	0.0204	0.0211	0.0204
	0.0211	0.0211	0.0204	0.0211	0.0204	0.0211
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 9

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.9357	0.5942	0.6031	0.6258	0.6220	0.6163
	0.6287	0.6377	0.6454	0.6657	0.6359	0.6674
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

DAILY AVERAGE HEAD ON TOP OF LAYER 6

AVERAGES	0.0177	0.0317	0.0427	0.0543	0.0659	0.0770
	0.0878	0.0988	0.1094	0.1207	0.1310	0.1409
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

DAILY AVERAGE HEAD ON TOP OF LAYER 8

AVERAGES	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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

	INCHES		CU. FEET	PERCENT
PRECIPITATION	30.45	(0.000)	110533.5	100.00
RUNOFF	0.000	(0.0000)	0.00	0.000
EVAPOTRANSPIRATION	20.809	(0.0000)	75537.78	68.339
LATERAL DRAINAGE COLLECTED FROM LAYER 3	6.37986	(0.00000)	23158.877	20.95190
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00081	(0.00000)	2.930	0.00265
AVERAGE HEAD ON TOP OF LAYER 4	0.656	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00037	(0.00000)	1.336	0.00121
AVERAGE HEAD ON TOP OF LAYER 6	0.081	(0.000)		
SUBSURFACE INFLOW INTO LAYER 9	0.24760		898.788	0.81314
LATERAL DRAINAGE COLLECTED FROM LAYER 7	0.24797	(0.00000)	900.124	0.81434
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00000	(0.00000)	0.000	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000	(0.000)		
CHANGE IN WATER STORAGE	3.260	(0.0000)	11835.50	10.708

****

****

PEAK DAILY VALUES FOR YEARS 1 THROUGH 1

	(INCHES)	(CU. FT.)
PRECIPITATION	1.81	6570.300
RUNOFF	0.000	0.0000
DRAINAGE COLLECTED FROM LAYER 3	$\boxed{0.05103}$	185.25325
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000008	0.02777
AVERAGE HEAD ON TOP OF LAYER 4	1.919	
MAXIMUM HEAD ON TOP OF LAYER 4	$\boxed{3.319}$	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	22.9 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000001	0.00443
AVERAGE HEAD ON TOP OF LAYER 6	0.146	
DRAINAGE COLLECTED FROM LAYER 7	0.00068	2.46686
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000000	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000	
MAXIMUM HEAD ON TOP OF LAYER 8	0.004	
LOCATION OF MAXIMUM HEAD IN LAYER 7 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	0.84	3034.7012

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4640
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1870

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 1

LAYER	(INCHES)	(VOL/VOL)
1	2.5364	0.4227
2	87.5161	0.7293
3	0.8074	0.0673
4	0.0000	0.0000
5	0.1769	0.7489
6	0.0000	0.0000
7	0.0020	0.0100
8	0.0000	0.0000
9	15.3720	0.4270
SNOW WATER	0.000	

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EXHIBIT 7
'HELP' MODEL RESULTS
INTERMEDIATE COVER

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


TIME: 12:29     DATE: 10/ 8/2007


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     TITLE:  CLINTON LF. NO.3 CHEMICAL WASTE UNIT (INTERMED.COVER 10YRS.)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.


                                    LAYER  1
                                    --------

                       TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER  11
           THICKNESS                      =     12.00    INCHES
           POROSITY                       =      0.4640 VOL/VOL
           FIELD CAPACITY                 =      0.3100 VOL/VOL
           WILTING POINT                  =      0.1870 VOL/VOL
           INITIAL SOIL WATER CONTENT     =      0.3100 VOL/VOL
           EFFECTIVE SAT. HYD. COND.      =   0.639999998000E-04 CM/SEC
        NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  1.80
                 FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
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LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	1680.00 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.8000 VOL/VOL
WILTING POINT	=	0.1350 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7293 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.190000006000E-03 CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4170 VOL/VOL
FIELD CAPACITY	=	0.0450 VOL/VOL
WILTING POINT	=	0.0180 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0673 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.299999993000E-01 CM/SEC
SLOPE	=	2.66 PERCENT
DRAINAGE LENGTH	=	170.0 FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 5

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL

INITIAL SOIL WATER CONTENT	=	0.7489 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

LAYER 6

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 7

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC
SLOPE	=	2.66 PERCENT
DRAINAGE LENGTH	=	170.0 FEET

LAYER 8

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 9

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 16

THICKNESS	=	36.00	INCHES
POROSITY	=	0.4270	VOL/VOL
FIELD CAPACITY	=	0.4180	VOL/VOL
WILTING POINT	=	0.3670	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4270	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000001000E-06	CM/SEC
SUBSURFACE INFLOW	=	0.25	INCHES/YR

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS USER-SPECIFIED.

SCS RUNOFF CURVE NUMBER	=	89.10	
FRACTION OF AREA ALLOWING RUNOFF	=	75.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	5.179	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	7.268	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.514	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	1245.302	INCHES
TOTAL INITIAL WATER	=	1245.302	INCHES
TOTAL SUBSURFACE INFLOW	=	0.25	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
Lincoln Illinois

STATION LATITUDE	=	40.10	DEGREES
MAXIMUM LEAF AREA INDEX	=	1.00	
START OF GROWING SEASON (JULIAN DATE)	=	117	
END OF GROWING SEASON (JULIAN DATE)	=	290	
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	10.30	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	71.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	65.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	70.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	72.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
1.60	1.31	2.59	3.66	3.15	4.08
3.63	3.53	3.35	2.28	2.06	2.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
21.40	26.00	36.00	48.80	59.10	68.60
73.00	71.90	64.70	53.50	39.80	27.70

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS
AND STATION LATITUDE = 40.10 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.45	110533.531	100.00
RUNOFF	1.603	5817.457	5.26
EVAPOTRANSPIRATION	25.521	92640.648	83.81
DRAINAGE COLLECTED FROM LAYER 3	8.3672	30373.102	27.48
PERC./LEAKAGE THROUGH LAYER 4	0.001102	4.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.8596		
PERC./LEAKAGE THROUGH LAYER 6	0.000534	1.937	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2183		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.81
DRAINAGE COLLECTED FROM LAYER 7	0.2481	900.727	0.81
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-5.041	-18299.330	-16.56
SOIL WATER AT START OF YEAR	1245.656	4521731.500	

SOIL WATER AT END OF YEAR	1240.615	4503432.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.285	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	35.90	130317.016	100.00
RUNOFF	4.032	14636.987	11.23
EVAPOTRANSPIRATION	23.805	86413.664	66.31
DRAINAGE COLLECTED FROM LAYER 3	8.2342	29890.074	22.94
PERC./LEAKAGE THROUGH LAYER 4	0.001081	3.925	0.00
AVG. HEAD ON TOP OF LAYER 4	0.8483		
PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.69
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.69
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-0.173	-626.264	-0.48
SOIL WATER AT START OF YEAR	1240.615	4503432.000	
SOIL WATER AT END OF YEAR	1237.349	4491576.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	3.093	11229.284	8.62
ANNUAL WATER BUDGET BALANCE	0.0001	0.535	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.40	161171.953	100.00
RUNOFF	9.018	32733.705	20.31
EVAPOTRANSPIRATION	31.652	114895.859	71.29
DRAINAGE COLLECTED FROM LAYER 3	7.9950	29021.994	18.01
PERC./LEAKAGE THROUGH LAYER 4	0.001046	3.795	0.00
AVG. HEAD ON TOP OF LAYER 4	0.8242		
PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.56
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.56
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-4.265	-15481.678	-9.61
SOIL WATER AT START OF YEAR	1237.349	4491576.500	
SOIL WATER AT END OF YEAR	1235.923	4486401.500	
SNOW WATER AT START OF YEAR	3.093	11229.284	6.97
SNOW WATER AT END OF YEAR	0.254	922.749	0.57
ANNUAL WATER BUDGET BALANCE	0.0000	0.049	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.12	109335.602	100.00
RUNOFF	4.597	16688.738	15.26
EVAPOTRANSPIRATION	24.248	88018.984	80.50
DRAINAGE COLLECTED FROM LAYER 3	8.1584	29615.119	27.09

PERC./LEAKAGE THROUGH LAYER 4	0.001069	3.880	0.00
AVG. HEAD ON TOP OF LAYER 4	0.8387		
PERC./LEAKAGE THROUGH LAYER 6	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.248278	901.250	0.82
DRAINAGE COLLECTED FROM LAYER 7	0.2488	903.274	0.83
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-6.884	-24988.781	-22.86
SOIL WATER AT START OF YEAR	1235.923	4486401.500	
SOIL WATER AT END OF YEAR	1229.293	4462335.000	
SNOW WATER AT START OF YEAR	0.254	922.749	0.84
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.481	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	32.29	117212.703	100.00
RUNOFF	3.951	14343.528	12.24
EVAPOTRANSPIRATION	25.434	92324.047	78.77
DRAINAGE COLLECTED FROM LAYER 3	7.7188	28019.305	23.90
PERC./LEAKAGE THROUGH LAYER 4	0.001002	3.637	0.00
AVG. HEAD ON TOP OF LAYER 4	0.7949		
PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.77
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.77

PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-4.814	-17476.465	-14.91
SOIL WATER AT START OF YEAR	1229.293	4462335.000	
SOIL WATER AT END OF YEAR	1224.479	4444859.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.267	0.00

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ANNUAL TOTALS FOR YEAR 6

	INCHES	CU. FEET	PERCENT
PRECIPITATION	34.58	125525.437	100.00
RUNOFF	2.853	10357.521	8.25
EVAPOTRANSPIRATION	28.068	101888.203	81.17
DRAINAGE COLLECTED FROM LAYER 3	7.2453	26300.416	20.95
PERC./LEAKAGE THROUGH LAYER 4	0.000929	3.372	0.00
AVG. HEAD ON TOP OF LAYER 4	0.7475		
PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.72
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.72
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-3.587	-13022.612	-10.37
SOIL WATER AT START OF YEAR	1224.479	4444859.000	
SOIL WATER AT END OF YEAR	1220.859	4431717.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.033	119.300	0.10

ANNUAL WATER BUDGET BALANCE	0.0000	-0.117	0.00

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ANNUAL TOTALS FOR YEAR 7

	INCHES	CU. FEET	PERCENT
PRECIPITATION	35.24	127921.203	100.00
RUNOFF	4.069	14770.168	11.55
EVAPOTRANSPIRATION	26.447	96002.539	75.05
DRAINAGE COLLECTED FROM LAYER 3	7.5099	27261.014	21.31
PERC./LEAKAGE THROUGH LAYER 4	0.000972	3.529	0.00
AVG. HEAD ON TOP OF LAYER 4	0.7740		
PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.70
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.70
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-2.787	-10115.093	-7.91
SOIL WATER AT START OF YEAR	1220.859	4431717.000	
SOIL WATER AT END OF YEAR	1218.105	4421721.000	
SNOW WATER AT START OF YEAR	0.033	119.300	0.09
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0002	0.557	0.00

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ANNUAL TOTALS FOR YEAR 8

	INCHES	CU. FEET	PERCENT

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PRECIPITATION	35.13	127521.930	100.00
RUNOFF	4.520	16407.953	12.87
EVAPOTRANSPIRATION	23.920	86828.648	68.09
DRAINAGE COLLECTED FROM LAYER 3	7.4594	27077.582	21.23
PERC./LEAKAGE THROUGH LAYER 4	0.000961	3.489	0.00
AVG. HEAD ON TOP OF LAYER 4	0.7663		
PERC./LEAKAGE THROUGH LAYER 6	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.248278	901.250	0.71
DRAINAGE COLLECTED FROM LAYER 7	0.2488	903.274	0.71
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-0.770	-2793.762	-2.19
SOIL WATER AT START OF YEAR	1218.105	4421721.000	
SOIL WATER AT END OF YEAR	1215.794	4413332.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	1.541	5595.296	4.39
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.518	0.00

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ANNUAL TOTALS FOR YEAR 9

	INCHES	CU. FEET	PERCENT
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PRECIPITATION	38.61	140154.297	100.00
RUNOFF	7.429	26968.176	19.24
EVAPOTRANSPIRATION	26.539	96335.375	68.74
DRAINAGE COLLECTED FROM LAYER 3	7.3743	26768.697	19.10
PERC./LEAKAGE THROUGH LAYER 4	0.000949	3.446	0.00
AVG. HEAD ON TOP OF LAYER 4	0.7587		

PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.64
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.64
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-2.733	-9920.101	-7.08
SOIL WATER AT START OF YEAR	1215.794	4413332.000	
SOIL WATER AT END OF YEAR	1214.603	4409007.000	
SNOW WATER AT START OF YEAR	1.541	5595.296	3.99
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.133	0.00

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ANNUAL TOTALS FOR YEAR 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.75	111622.523	100.00
RUNOFF	3.685	13375.109	11.98
EVAPOTRANSPIRATION	23.471	85199.219	76.33
DRAINAGE COLLECTED FROM LAYER 3	7.5899	27551.475	24.68
PERC./LEAKAGE THROUGH LAYER 4	0.000986	3.578	0.00
AVG. HEAD ON TOP OF LAYER 4	0.7819		
PERC./LEAKAGE THROUGH LAYER 6	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 6	0.2362		
SUBSURFACE INFLOW INTO LAYER 9	0.247600	898.788	0.81
DRAINAGE COLLECTED FROM LAYER 7	0.2482	900.806	0.81
PERC./LEAKAGE THROUGH LAYER 9	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 8	0.0001		
CHANGE IN WATER STORAGE	-3.996	-14504.979	-12.99

SOIL WATER AT START OF YEAR	1214.603	4409007.000	
SOIL WATER AT END OF YEAR	1210.164	4392894.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.443	1608.462	1.44
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.323	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 10

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	1.51	1.60	2.69	3.26	4.17	4.46
	4.10	3.31	4.00	1.92	1.71	2.01
STD. DEVIATIONS	0.75	0.64	1.50	2.13	2.13	2.62
	1.41	1.35	1.41	1.28	0.79	0.98
RUNOFF						
TOTALS	0.342	0.811	1.610	0.352	0.127	0.305
	0.231	0.242	0.305	0.036	0.050	0.165
STD. DEVIATIONS	0.415	0.784	1.505	0.491	0.186	0.547
	0.239	0.284	0.305	0.084	0.087	0.277
EVAPOTRANSPIRATION						
TOTALS	0.486	0.443	0.811	3.070	3.666	4.059
	3.993	3.011	2.845	1.840	1.091	0.597
STD. DEVIATIONS	0.116	0.083	0.457	0.830	1.353	1.115
	1.692	1.386	0.600	0.349	0.224	0.136
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.6590	0.5935	0.6780	0.6069	0.6703	0.6343
	0.6316	0.6471	0.6527	0.6483	0.6522	0.6913
STD. DEVIATIONS	0.0955	0.1041	0.0576	0.0640	0.0892	0.0672
	0.0909	0.0918	0.0902	0.0895	0.1111	0.0640
PERCOLATION/LEAKAGE THROUGH LAYER 4						
TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001

	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

SUBSURFACE INFLOW INTO LAYER 9

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LATERAL DRAINAGE COLLECTED FROM LAYER 7

TOTALS	0.0211	0.0192	0.0211	0.0204	0.0211	0.0204
	0.0211	0.0211	0.0204	0.0211	0.0204	0.0211
STD. DEVIATIONS	0.0000	0.0003	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 9

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.7994	0.7905	0.8224	0.7607	0.8131	0.7951
	0.7661	0.7850	0.8182	0.7864	0.8174	0.8385
STD. DEVIATIONS	0.1159	0.1304	0.0698	0.0802	0.1082	0.0842
	0.1103	0.1114	0.1131	0.1086	0.1392	0.0776

DAILY AVERAGE HEAD ON TOP OF LAYER 6

AVERAGES	0.2293	0.2307	0.2319	0.2333	0.2347	0.2358
	0.2362	0.2362	0.2362	0.2362	0.2362	0.2362
STD. DEVIATIONS	0.0218	0.0173	0.0135	0.0092	0.0048	0.0012
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

DAILY AVERAGE HEAD ON TOP OF LAYER 8

AVERAGES	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001

STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 10

	INCHES		CU. FEET	PERCENT
PRECIPITATION	34.75	(4.368)	126131.6	100.00
RUNOFF	4.576	(2.1452)	16609.94	13.169
EVAPOTRANSPIRATION	25.910	(2.4898)	94054.72	74.569
LATERAL DRAINAGE COLLECTED FROM LAYER 3	$\boxed{7.76525}$	(0.39507)	28187.875	22.34799
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.00101	(0.00006)	3.665	0.00291
AVERAGE HEAD ON TOP OF LAYER 4	0.799	(0.040)		
ERCOLATION/LEAKAGE THROUGH LAYER 6	0.00055	(0.00001)	2.010	0.00159
AVERAGE HEAD ON TOP OF LAYER 6	0.234	(0.006)		
SUBSURFACE INFLOW INTO LAYER 9	0.24760		898.788	0.71258
LATERAL DRAINAGE COLLECTED FROM LAYER 7	0.24829	(0.00030)	901.292	0.71456
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00000	(0.00000)	0.000	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000	(0.000)		
CHANGE IN WATER STORAGE	-3.505	(2.0005)	-12722.91	-10.087

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 10

	(INCHES)	(CU. FT.)
PRECIPITATION	4.09	14846.700
RUNOFF	1.538	5583.8047
DRAINAGE COLLECTED FROM LAYER 3	$\boxed{0.06154}$	223.38950
PERCOLATION/LEAKAGE THROUGH LAYER 4	0.000010	0.03530
AVERAGE HEAD ON TOP OF LAYER 4	2.314	
MAXIMUM HEAD ON TOP OF LAYER 4	$\boxed{3.929}$	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	25.6 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000002	0.00552
AVERAGE HEAD ON TOP OF LAYER 6	0.236	
DRAINAGE COLLECTED FROM LAYER 7	0.00068	2.46796
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000000	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000	
MAXIMUM HEAD ON TOP OF LAYER 8	0.004	
LOCATION OF MAXIMUM HEAD IN LAYER 7 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	3.78	13732.6572

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.5136
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1796

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR   10

LAYER	(INCHES)	(VOL/VOL)
1	3.5724	0.2977
2	1189.9114	$\boxed{0.7083}$
3	0.7708	$\boxed{0.0642}$
4	0.0000	0.0000
5	0.1815	0.7682
6	0.0000	0.0000
7	0.0020	0.0100
8	0.0000	0.0000
9	15.3720	$\boxed{0.4270}$
SNOW WATER	0.443	

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EXHIBIT 8
'HELP' MODEL RESULTS
POST CLOSURE PERIOD YEARS 1 - 30
WITH LEACHATE COLLECTION AND REMOVAL

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**             HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                   **
**              DEVELOPED BY ENVIRONMENTAL LABORATORY                       **
**                USAE WATERWAYS EXPERIMENT STATION                         **
**            FOR USEPA RISK REDUCTION ENGINEERING LABORATORY               **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:      D:\tsca\PRECIP.D4
TEMPERATURE DATA FILE:        D:\tsca\TEMP.D7
SOLAR RADIATION DATA FILE:    D:\tsca\SOLAR.D13
EVAPOTRANSPIRATION DATA:      D:\tsca\EVAP_C.D11
SOIL AND DESIGN DATA FILE:    D:\tsca\SOIL_C30.D10
OUTPUT DATA FILE:             D:\tsca\soil_c30.OUT


TIME: 11: 2      DATE: 10/ 9/2007


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     TITLE:  CLINTON LF. NO.3 CHEMICAL WASTE UNIT (CLOSURE 30 YRS/SS/LCS)

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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.


                                 LAYER  1
                                 --------

                       TYPE 1 - VERTICAL PERCOLATION LAYER
                           MATERIAL TEXTURE NUMBER   8
          THICKNESS                    =       12.00   INCHES
          POROSITY                     =       0.4630 VOL/VOL
          FIELD CAPACITY               =       0.2320 VOL/VOL
          WILTING POINT                =       0.1160 VOL/VOL
          INITIAL SOIL WATER CONTENT   =       0.2320 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =   0.369999994000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  4.90
                  FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
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                    LAYER  2
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          TYPE 1 - VERTICAL PERCOLATION LAYER
            MATERIAL TEXTURE NUMBER   9
THICKNESS                   =     24.00   INCHES
POROSITY                    =      0.5010 VOL/VOL
FIELD CAPACITY              =      0.2840 VOL/VOL
WILTING POINT               =      0.1350 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.2840 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =   0.190000006000E-03 CM/SEC


                    LAYER  3
                    --------

           TYPE 2 - LATERAL DRAINAGE LAYER
            MATERIAL TEXTURE NUMBER  20
THICKNESS                   =      0.20   INCHES
POROSITY                    =      0.8500 VOL/VOL
FIELD CAPACITY              =      0.0100 VOL/VOL
WILTING POINT               =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =    10.0000000000     CM/SEC
SLOPE                       =     25.00   PERCENT
DRAINAGE LENGTH             =    875.0    FEET


                    LAYER  4
                    --------

           TYPE 4 - FLEXIBLE MEMBRANE LINER
            MATERIAL TEXTURE NUMBER  35
THICKNESS                   =      0.04   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =   0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =     10.00   HOLES/ACRE
FML PLACEMENT QUALITY       =   4 - POOR


                    LAYER  5
                    --------

             TYPE 3 - BARRIER SOIL LINER
            MATERIAL TEXTURE NUMBER   0
THICKNESS                   =     12.00   INCHES
POROSITY                    =      0.4270 VOL/VOL
FIELD CAPACITY              =      0.4180 VOL/VOL
WILTING POINT               =      0.3670 VOL/VOL
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INITIAL SOIL WATER CONTENT  =       0.4270 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.999999975000E-05 CM/SEC


                       LAYER  6
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          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  11
THICKNESS                   =      12.00   INCHES
POROSITY                    =       0.4640 VOL/VOL
FIELD CAPACITY              =       0.3100 VOL/VOL
WILTING POINT               =       0.1870 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.3100 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.639999998000E-04 CM/SEC


                       LAYER  7
                       --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  18
THICKNESS                   =     133.20   INCHES
POROSITY                    =       0.6710 VOL/VOL
FIELD CAPACITY              =       0.2920 VOL/VOL
WILTING POINT               =       0.0770 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.2920 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.100000005000E-02 CM/SEC


                       LAYER  8
                       --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  11
THICKNESS                   =      12.00   INCHES
POROSITY                    =       0.4640 VOL/VOL
FIELD CAPACITY              =       0.3100 VOL/VOL
WILTING POINT               =       0.1870 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.3100 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.639999998000E-04 CM/SEC


                       LAYER  9
                       --------

          TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                   =    1680.00   INCHES
POROSITY                    =       0.8500 VOL/VOL
FIELD CAPACITY              =       0.8000 VOL/VOL
WILTING POINT               =       0.1350 VOL/VOL
```

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INITIAL SOIL WATER CONTENT  =       0.7083 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.190000006000E-03 CM/SEC


                         LAYER 10
                         --------

             TYPE 2 - LATERAL DRAINAGE LAYER
               MATERIAL TEXTURE NUMBER   0
THICKNESS                   =      12.00    INCHES
POROSITY                    =       0.4170 VOL/VOL
FIELD CAPACITY              =       0.0450 VOL/VOL
WILTING POINT               =       0.0180 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.0642 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.299999993000E-01 CM/SEC
SLOPE                       =       2.66    PERCENT
DRAINAGE LENGTH             =     170.0     FEET


                         LAYER 11
                         --------

             TYPE 4 - FLEXIBLE MEMBRANE LINER
               MATERIAL TEXTURE NUMBER  35
THICKNESS                   =       0.06    INCHES
POROSITY                    =       0.0000 VOL/VOL
FIELD CAPACITY              =       0.0000 VOL/VOL
WILTING POINT               =       0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =       1.00    HOLES/ACRE
FML INSTALLATION DEFECTS    =      10.00    HOLES/ACRE
FML PLACEMENT QUALITY       =  4 - POOR


                         LAYER 12
                         --------

           TYPE 1 - VERTICAL PERCOLATION LAYER
               MATERIAL TEXTURE NUMBER  17
THICKNESS                   =       0.24    INCHES
POROSITY                    =       0.7500 VOL/VOL
FIELD CAPACITY              =       0.7470 VOL/VOL
WILTING POINT               =       0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT  =       0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC


                         LAYER 13
                         --------

             TYPE 4 - FLEXIBLE MEMBRANE LINER
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MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 14

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC
SLOPE	=	2.66 PERCENT
DRAINAGE LENGTH	=	170.0 FEET

LAYER 15

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 16

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 16

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4270 VOL/VOL
FIELD CAPACITY	=	0.4180 VOL/VOL
WILTING POINT	=	0.3670 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4270 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000001000E-06 CM/SEC

SUBSURFACE INFLOW = 0.25 INCHES/YR

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 8 WITH A GOOD STAND OF GRASS, A SURFACE SLOPE OF 25.% AND A SLOPE LENGTH OF 875. FEET.

SCS RUNOFF CURVE NUMBER	=	72.60	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	20.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	5.056	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	9.564	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.472	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	1267.326	INCHES
TOTAL INITIAL WATER	=	1267.326	INCHES
TOTAL SUBSURFACE INFLOW	=	0.25	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM Lincoln Illinois

STATION LATITUDE	=	40.10	DEGREES
MAXIMUM LEAF AREA INDEX	=	4.00	
START OF GROWING SEASON (JULIAN DATE)	=	117	
END OF GROWING SEASON (JULIAN DATE)	=	290	
EVAPORATIVE ZONE DEPTH	=	20.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	10.30	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	71.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	65.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	70.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	72.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
1.60	1.31	2.59	3.66	3.15	4.08
3.63	3.53	3.35	2.28	2.06	2.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
21.40	26.00	36.00	48.80	59.10	68.60
73.00	71.90	64.70	53.50	39.80	27.70

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS
AND STATION LATITUDE = 40.10 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.45	110533.531	100.00
RUNOFF	0.403	1463.866	1.32
VAPOTRANSPIRATION	25.794	93631.258	84.71
DRAINAGE COLLECTED FROM LAYER 3	2.5135	9123.876	8.25
PERC./LEAKAGE THROUGH LAYER 5	0.000299	1.085	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0005		
DRAINAGE COLLECTED FROM LAYER 10	7.3147	26552.240	24.02
PERC./LEAKAGE THROUGH LAYER 11	0.000943	3.424	0.00
AVG. HEAD ON TOP OF LAYER 11	0.7549		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.81
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.81
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
HANGE IN WATER STORAGE	-5.576	-20239.289	-18.31
SOIL WATER AT START OF YEAR	1267.680	4601678.500	
SOIL WATER AT END OF YEAR	1262.104	4581439.500	

SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.442	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	35.90	130317.016	100.00
RUNOFF	2.407	8736.625	6.70
EVAPOTRANSPIRATION	25.199	91473.375	70.19
DRAINAGE COLLECTED FROM LAYER 3	8.0221	29120.248	22.35
PERC./LEAKAGE THROUGH LAYER 5	0.000882	3.201	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0014		
DRAINAGE COLLECTED FROM LAYER 10	7.1049	25790.635	19.79
PERC./LEAKAGE THROUGH LAYER 11	0.000910	3.305	0.00
AVG. HEAD ON TOP OF LAYER 11	0.7324		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.69
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.69
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.834	-24806.619	-19.04
SOIL WATER AT START OF YEAR	1262.104	4581439.500	
SOIL WATER AT END OF YEAR	1252.177	4545403.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	3.093	11229.284	8.62
ANNUAL WATER BUDGET BALANCE	0.0002	0.733	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.40	161171.953	100.00
RUNOFF	5.073	18414.098	11.43
EVAPOTRANSPIRATION	33.949	123235.414	76.46
DRAINAGE COLLECTED FROM LAYER 3	7.1436	25931.330	16.09
PERC./LEAKAGE THROUGH LAYER 5	0.000780	2.831	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0013		
DRAINAGE COLLECTED FROM LAYER 10	6.6756	24232.283	15.04
PERC./LEAKAGE THROUGH LAYER 11	0.000848	3.079	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6884		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.56
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.56
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-8.442	-30643.309	-19.01
SOIL WATER AT START OF YEAR	1252.177	4545403.500	
SOIL WATER AT END OF YEAR	1246.575	4525066.500	
SNOW WATER AT START OF YEAR	3.093	11229.284	6.97
SNOW WATER AT END OF YEAR	0.254	922.749	0.57
ANNUAL WATER BUDGET BALANCE	0.0000	0.115	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.12	109335.602	100.00
RUNOFF	3.635	13195.945	12.07
EVAPOTRANSPIRATION	24.154	87677.727	80.19
DRAINAGE COLLECTED FROM LAYER 3	2.2583	8197.716	7.50
PERC./LEAKAGE THROUGH LAYER 5	0.000259	0.942	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0004		
DRAINAGE COLLECTED FROM LAYER 10	6.6807	24250.984	22.18
PERC./LEAKAGE THROUGH LAYER 11	0.000851	3.089	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6869		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.82
DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.83
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.608	-23988.670	-21.94
SOIL WATER AT START OF YEAR	1246.575	4525066.500	
SOIL WATER AT END OF YEAR	1240.221	4502000.500	
SNOW WATER AT START OF YEAR	0.254	922.749	0.84
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.126	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	32.29	117212.703	100.00
RUNOFF	2.793	10140.076	8.65

EVAPOTRANSPIRATION	25.787	93608.164	79.86
DRAINAGE COLLECTED FROM LAYER 3	3.7144	13483.307	11.50
PERC./LEAKAGE THROUGH LAYER 5	0.000409	1.486	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 10	6.3151	22923.844	19.56
PERC./LEAKAGE THROUGH LAYER 11	0.000796	2.890	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6506		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.77
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.77
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.321	-22944.508	-19.58
SOIL WATER AT START OF YEAR	1240.221	4502000.500	
SOIL WATER AT END OF YEAR	1233.900	4479056.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.197	0.00

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ANNUAL TOTALS FOR YEAR 6

	INCHES	CU. FEET	PERCENT
PRECIPITATION	34.58	125525.437	100.00
RUNOFF	1.187	4308.437	3.43
EVAPOTRANSPIRATION	27.797	100902.641	80.38
DRAINAGE COLLECTED FROM LAYER 3	5.5717	20225.309	16.11
PERC./LEAKAGE THROUGH LAYER 5	0.000631	2.292	0.00

AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 10	6.4209	23307.707	18.57
ERC./LEAKAGE THROUGH LAYER 11	0.000814	2.953	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6618		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.72
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.72
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.397	-23220.908	-18.50
SOIL WATER AT START OF YEAR	1233.900	4479056.000	
SOIL WATER AT END OF YEAR	1227.470	4455716.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.033	119.300	0.10
ANNUAL WATER BUDGET BALANCE	0.0001	0.228	0.00

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ANNUAL TOTALS FOR YEAR 7

	INCHES	CU. FEET	PERCENT
PRECIPITATION	35.24	127921.203	100.00
RUNOFF	1.367	4962.718	3.88
EVAPOTRANSPIRATION	26.931	97760.992	76.42
DRAINAGE COLLECTED FROM LAYER 3	7.1442	25933.486	20.27
PERC./LEAKAGE THROUGH LAYER 5	0.000812	2.947	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0013		
RAINAGE COLLECTED FROM LAYER 10	6.3156	22925.650	17.92
PERC./LEAKAGE THROUGH LAYER 11	0.000800	2.904	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6505		

PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.70
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.70
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.519	-23663.342	-18.50
SOIL WATER AT START OF YEAR	1227.470	4455716.000	
SOIL WATER AT END OF YEAR	1220.984	4432172.000	
SNOW WATER AT START OF YEAR	0.033	119.300	0.09
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.316	0.00

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ANNUAL TOTALS FOR YEAR 8

	INCHES	CU. FEET	PERCENT
PRECIPITATION	35.13	127521.930	100.00
RUNOFF	2.423	8795.479	6.90
EVAPOTRANSPIRATION	25.113	91161.617	71.49
DRAINAGE COLLECTED FROM LAYER 3	6.3221	22949.160	18.00
PERC./LEAKAGE THROUGH LAYER 5	0.000698	2.533	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0011		
DRAINAGE COLLECTED FROM LAYER 10	5.9945	21760.092	17.06
PERC./LEAKAGE THROUGH LAYER 11	0.000753	2.733	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6157		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.71

DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.71
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.724	-17146.707	-13.45
SOIL WATER AT START OF YEAR	1220.984	4432172.000	
SOIL WATER AT END OF YEAR	1214.719	4409430.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	1.541	5595.296	4.39
ANNUAL WATER BUDGET BALANCE	0.0001	0.264	0.00

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ANNUAL TOTALS FOR YEAR 9

	INCHES	CU. FEET	PERCENT
PRECIPITATION	38.61	140154.297	100.00
RUNOFF	5.591	20294.348	14.48
EVAPOTRANSPIRATION	27.724	100636.656	71.80
DRAINAGE COLLECTED FROM LAYER 3	5.6713	20586.979	14.69
PERC./LEAKAGE THROUGH LAYER 5	0.000621	2.254	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 10	5.7895	21015.920	14.99
PERC./LEAKAGE THROUGH LAYER 11	0.000725	2.630	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5966		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.64
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.64
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.166	-22381.387	-15.97

SOIL WATER AT START OF YEAR	1214.719	4409430.000	
SOIL WATER AT END OF YEAR	1210.095	4392644.000	
SNOW WATER AT START OF YEAR	1.541	5595.296	3.99
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.230	0.00

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ANNUAL TOTALS FOR YEAR 10

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.75	111622.523	100.00
RUNOFF	1.311	4759.134	4.26
EVAPOTRANSPIRATION	23.978	87039.320	77.98
DRAINAGE COLLECTED FROM LAYER 3	6.1383	22281.852	19.96
PERC./LEAKAGE THROUGH LAYER 5	0.000680	2.468	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0011		
DRAINAGE COLLECTED FROM LAYER 10	5.8636	21284.984	19.07
PERC./LEAKAGE THROUGH LAYER 11	0.000736	2.673	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6040		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.81
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.81
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.541	-23744.820	-21.27
SOIL WATER AT START OF YEAR	1210.095	4392644.000	
SOIL WATER AT END OF YEAR	1203.110	4367290.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00

SNOW WATER AT END OF YEAR	0.443	1608.462	1.44
ANNUAL WATER BUDGET BALANCE	0.0000	0.032	0.00

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ANNUAL TOTALS FOR YEAR 11

	INCHES	CU. FEET	PERCENT
PRECIPITATION	25.85	93835.500	100.00
RUNOFF	1.894	6874.820	7.33
EVAPOTRANSPIRATION	21.274	77226.398	82.30
DRAINAGE COLLECTED FROM LAYER 3	2.7363	9932.829	10.59
PERC./LEAKAGE THROUGH LAYER 5	0.000306	1.112	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0005		
DRAINAGE COLLECTED FROM LAYER 10	5.8400	21199.154	22.59
ERC./LEAKAGE THROUGH LAYER 11	0.000733	2.662	0.00
AVG. HEAD ON TOP OF LAYER 11	0.6013		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.96
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.96
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-5.895	-21399.939	-22.81
SOIL WATER AT START OF YEAR	1203.110	4367290.500	
SOIL WATER AT END OF YEAR	1197.297	4346187.000	
SNOW WATER AT START OF YEAR	0.443	1608.462	1.71
SNOW WATER AT END OF YEAR	0.362	1312.330	1.40
NNUAL WATER BUDGET BALANCE	0.0001	0.219	0.00

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ANNUAL TOTALS FOR YEAR 12

	INCHES	CU. FEET	PERCENT
PRECIPITATION	28.81	104580.297	100.00
RUNOFF	0.782	2840.143	2.72
EVAPOTRANSPIRATION	22.585	81984.656	78.39
DRAINAGE COLLECTED FROM LAYER 3	5.7854	21000.941	20.08
PERC./LEAKAGE THROUGH LAYER 5	0.000662	2.403	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0010		
DRAINAGE COLLECTED FROM LAYER 10	5.6415	20478.584	19.58
PERC./LEAKAGE THROUGH LAYER 11	0.000706	2.561	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5800		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.86
DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.86
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-5.985	-21725.762	-20.77
SOIL WATER AT START OF YEAR	1197.297	4346187.000	
SOIL WATER AT END OF YEAR	1191.673	4325773.500	
SNOW WATER AT START OF YEAR	0.362	1312.330	1.25
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.286	0.00

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ANNUAL TOTALS FOR YEAR 13

	INCHES	CU. FEET	PERCENT

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PRECIPITATION	31.56	114562.812	100.00
RUNOFF	1.581	5740.238	5.01
EVAPOTRANSPIRATION	24.074	87389.937	76.28
DRAINAGE COLLECTED FROM LAYER 3	4.7439	17220.344	15.03
PERC./LEAKAGE THROUGH LAYER 5	0.000548	1.990	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 10	5.5846	20272.184	17.70
PERC./LEAKAGE THROUGH LAYER 11	0.000699	2.536	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5752		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.78
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.79
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.425	-16061.893	-14.02
SOIL WATER AT START OF YEAR	1191.673	4325773.500	
SOIL WATER AT END OF YEAR	1186.843	4308239.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.406	1472.621	1.29
ANNUAL WATER BUDGET BALANCE	0.0000	-0.022	0.00

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ANNUAL TOTALS FOR YEAR 14

	INCHES	CU. FEET	PERCENT
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PRECIPITATION	31.36	113836.836	100.00
RUNOFF	1.048	3803.416	3.34
EVAPOTRANSPIRATION	21.393	77654.914	68.22

DRAINAGE COLLECTED FROM LAYER 3	6.4878	23550.621	20.69
PERC./LEAKAGE THROUGH LAYER 5	0.000719	2.609	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0012		
DRAINAGE COLLECTED FROM LAYER 10	5.3647	19473.799	17.11
PERC./LEAKAGE THROUGH LAYER 11	0.000668	2.425	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5533		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.79
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.79
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-2.933	-10647.914	-9.35
SOIL WATER AT START OF YEAR	1186.843	4308239.000	
SOIL WATER AT END OF YEAR	1180.689	4285901.500	
SNOW WATER AT START OF YEAR	0.406	1472.621	1.29
SNOW WATER AT END OF YEAR	3.626	13162.146	11.56
ANNUAL WATER BUDGET BALANCE	0.0000	-0.016	0.00

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ANNUAL TOTALS FOR YEAR 15

	INCHES	CU. FEET	PERCENT
PRECIPITATION	24.36	88426.828	100.00
RUNOFF	3.005	10908.397	12.34
EVAPOTRANSPIRATION	20.537	74548.703	84.31
DRAINAGE COLLECTED FROM LAYER 3	3.4986	12699.758	14.36
PERC./LEAKAGE THROUGH LAYER 5	0.000391	1.419	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0006		
DRAINAGE COLLECTED FROM LAYER 10	5.2751	19148.545	21.65

PERC./LEAKAGE THROUGH LAYER 11	0.000657	2.383	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5431		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	1.02
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	1.02
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-7.956	-28881.217	-32.66
SOIL WATER AT START OF YEAR	1180.689	4285901.500	
SOIL WATER AT END OF YEAR	1176.359	4270182.500	
SNOW WATER AT START OF YEAR	3.626	13162.146	14.88
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0002	0.624	0.00

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ANNUAL TOTALS FOR YEAR 16

	INCHES	CU. FEET	PERCENT
PRECIPITATION	30.70	111440.992	100.00
RUNOFF	0.531	1927.310	1.73
EVAPOTRANSPIRATION	25.800	93653.836	84.04
DRAINAGE COLLECTED FROM LAYER 3	4.0199	14592.108	13.09
PERC./LEAKAGE THROUGH LAYER 5	0.000485	1.760	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0007		
DRAINAGE COLLECTED FROM LAYER 10	5.1359	18643.232	16.73
PERC./LEAKAGE THROUGH LAYER 11	0.000638	2.314	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5278		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00

AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.81
DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.81
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.787	-17377.207	-15.59
SOIL WATER AT START OF YEAR	1176.359	4270182.500	
SOIL WATER AT END OF YEAR	1171.572	4252805.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.310	0.00

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ANNUAL TOTALS FOR YEAR 17

	INCHES	CU. FEET	PERCENT
PRECIPITATION	38.44	139537.187	100.00
RUNOFF	1.932	7014.852	5.03
EVAPOTRANSPIRATION	31.159	113108.539	81.06
DRAINAGE COLLECTED FROM LAYER 3	4.8249	17514.289	12.55
PERC./LEAKAGE THROUGH LAYER 5	0.000547	1.987	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 10	5.1040	18527.396	13.28
PERC./LEAKAGE THROUGH LAYER 11	0.000634	2.301	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5259		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.64
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.65
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00

AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.581	-16630.037	-11.92
SOIL WATER AT START OF YEAR	1171.572	4252805.000	
SOIL WATER AT END OF YEAR	1166.940	4235994.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.050	181.312	0.13
ANNUAL WATER BUDGET BALANCE	0.0000	0.140	0.00

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ANNUAL TOTALS FOR YEAR 18

	INCHES	CU. FEET	PERCENT
PRECIPITATION	24.91	90423.328	100.00
RUNOFF	1.906	6917.433	7.65
EVAPOTRANSPIRATION	21.144	76753.516	84.88
DRAINAGE COLLECTED FROM LAYER 3	3.1782	11536.842	12.76
PERC./LEAKAGE THROUGH LAYER 5	0.000342	1.240	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0006		
DRAINAGE COLLECTED FROM LAYER 10	4.9700	18040.998	19.95
PERC./LEAKAGE THROUGH LAYER 11	0.000616	2.237	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5121		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.99
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	1.00
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.289	-22827.373	-25.25
SOIL WATER AT START OF YEAR	1166.940	4235994.000	

SOIL WATER AT END OF YEAR	1159.948	4210611.000	
SNOW WATER AT START OF YEAR	0.050	181.312	0.20
SNOW WATER AT END OF YEAR	0.754	2736.910	3.03
ANNUAL WATER BUDGET BALANCE	0.0000	-0.103	0.00

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ANNUAL TOTALS FOR YEAR 19

	INCHES	CU. FEET	PERCENT
PRECIPITATION	39.41	143058.250	100.00
RUNOFF	4.942	17938.773	12.54
EVAPOTRANSPIRATION	23.319	84649.195	59.17
DRAINAGE COLLECTED FROM LAYER 3	8.9154	32362.855	22.62
PERC./LEAKAGE THROUGH LAYER 5	0.000965	3.503	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0016		
DRAINAGE COLLECTED FROM LAYER 10	4.9765	18064.545	12.63
PERC./LEAKAGE THROUGH LAYER 11	0.000618	2.242	0.00
AVG. HEAD ON TOP OF LAYER 11	0.5128		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.63
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.63
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-2.744	-9959.294	-6.96
SOIL WATER AT START OF YEAR	1159.948	4210611.000	
SOIL WATER AT END OF YEAR	1156.875	4199455.500	
SNOW WATER AT START OF YEAR	0.754	2736.910	1.91
SNOW WATER AT END OF YEAR	1.083	3933.042	2.75
ANNUAL WATER BUDGET BALANCE	0.0000	0.151	0.00

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ANNUAL TOTALS FOR YEAR 20

	INCHES	CU. FEET	PERCENT
PRECIPITATION	34.99	127013.680	100.00
RUNOFF	5.073	18415.906	14.50
EVAPOTRANSPIRATION	24.060	87336.250	68.76
DRAINAGE COLLECTED FROM LAYER 3	6.9694	25298.793	19.92
PERC./LEAKAGE THROUGH LAYER 5	0.000759	2.756	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0013		
DRAINAGE COLLECTED FROM LAYER 10	4.8562	17627.891	13.88
PERC./LEAKAGE THROUGH LAYER 11	0.000602	2.184	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4987		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.71
DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.71
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-5.969	-21667.400	-17.06
SOIL WATER AT START OF YEAR	1156.875	4199455.500	
SOIL WATER AT END OF YEAR	1151.989	4181721.000	
SNOW WATER AT START OF YEAR	1.083	3933.042	3.10
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.217	0.00

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ANNUAL TOTALS FOR YEAR 21

	INCHES	CU. FEET	PERCENT
PRECIPITATION	38.73	140589.906	100.00
RUNOFF	1.232	4473.567	3.18
EVAPOTRANSPIRATION	32.180	116813.930	83.09
DRAINAGE COLLECTED FROM LAYER 3	4.9480	17961.123	12.78
PERC./LEAKAGE THROUGH LAYER 5	0.000577	2.094	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 10	4.6580	16908.629	12.03
PERC./LEAKAGE THROUGH LAYER 11	0.000575	2.089	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4800		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.64
RAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.64
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.289	-15568.694	-11.07
SOIL WATER AT START OF YEAR	1151.989	4181721.000	
SOIL WATER AT END OF YEAR	1146.873	4163148.750	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.827	3003.594	2.14
ANNUAL WATER BUDGET BALANCE	-0.0002	-0.657	0.00

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ANNUAL TOTALS FOR YEAR 22

	INCHES	CU. FEET	PERCENT
PRECIPITATION	39.61	143784.297	100.00

RUNOFF	0.840	3048.268	2.12
EVAPOTRANSPIRATION	31.143	113049.203	78.62
DRAINAGE COLLECTED FROM LAYER 3	6.4312	23345.104	16.24
PERC./LEAKAGE THROUGH LAYER 5	0.000743	2.696	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0011		
DRAINAGE COLLECTED FROM LAYER 10	4.5605	16554.445	11.51
PERC./LEAKAGE THROUGH LAYER 11	0.000562	2.041	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4699		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.63
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.63
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-3.365	-12215.503	-8.50
SOIL WATER AT START OF YEAR	1146.873	4163148.750	
SOIL WATER AT END OF YEAR	1142.276	4146460.000	
SNOW WATER AT START OF YEAR	0.827	3003.594	2.09
SNOW WATER AT END OF YEAR	2.060	7476.697	5.20
ANNUAL WATER BUDGET BALANCE	0.0002	0.770	0.00

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ANNUAL TOTALS FOR YEAR 23

	INCHES	CU. FEET	PERCENT
PRECIPITATION	43.18	156743.328	100.00
RUNOFF	2.261	8205.882	5.24
EVAPOTRANSPIRATION	34.371	124768.484	79.60
DRAINAGE COLLECTED FROM LAYER 3	8.0879	29359.102	18.73
PERC./LEAKAGE THROUGH LAYER 5	0.000904	3.281	0.00

AVG. HEAD ON TOP OF LAYER 4	0.0014		
DRAINAGE COLLECTED FROM LAYER 10	4.5832	16636.975	10.61
PERC./LEAKAGE THROUGH LAYER 11	0.000566	2.055	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4723		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.57
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.57
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-6.124	-22228.389	-14.18
SOIL WATER AT START OF YEAR	1142.276	4146460.000	
SOIL WATER AT END OF YEAR	1136.595	4125839.750	
SNOW WATER AT START OF YEAR	2.060	7476.697	4.77
SNOW WATER AT END OF YEAR	1.617	5868.675	3.74
ANNUAL WATER BUDGET BALANCE	-0.0002	-0.738	0.00

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ANNUAL TOTALS FOR YEAR 24

	INCHES	CU. FEET	PERCENT
PRECIPITATION	33.56	121822.836	100.00
RUNOFF	4.339	15751.293	12.93
EVAPOTRANSPIRATION	23.177	84132.805	69.06
DRAINAGE COLLECTED FROM LAYER 3	6.2773	22786.742	18.70
PERC./LEAKAGE THROUGH LAYER 5	0.000700	2.542	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0011		
DRAINAGE COLLECTED FROM LAYER 10	4.4705	16227.842	13.32
PERC./LEAKAGE THROUGH LAYER 11	0.000551	2.001	0.00

AVG. HEAD ON TOP OF LAYER 11	0.4592		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.74
DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.74
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.705	-17078.162	-14.02
SOIL WATER AT START OF YEAR	1136.595	4125839.750	
SOIL WATER AT END OF YEAR	1133.507	4114630.250	
SNOW WATER AT START OF YEAR	1.617	5868.675	4.82
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0001	0.285	0.00

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ANNUAL TOTALS FOR YEAR 25

	INCHES	CU. FEET	PERCENT
PRECIPITATION	27.19	98699.695	100.00
RUNOFF	0.896	3251.861	3.29
EVAPOTRANSPIRATION	23.327	84676.242	85.79
DRAINAGE COLLECTED FROM LAYER 3	3.2689	11866.257	12.02
PERC./LEAKAGE THROUGH LAYER 5	0.000370	1.342	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0006		
DRAINAGE COLLECTED FROM LAYER 10	4.3849	15917.333	16.13
PERC./LEAKAGE THROUGH LAYER 11	0.000540	1.961	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4517		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.91

DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.91
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.687	-17013.473	-17.24
SOIL WATER AT START OF YEAR	1133.507	4114630.250	
SOIL WATER AT END OF YEAR	1127.015	4091065.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	1.805	6551.396	6.64
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.542	0.00

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ANNUAL TOTALS FOR YEAR 26

	INCHES	CU. FEET	PERCENT
PRECIPITATION	32.03	116268.937	100.00
RUNOFF	3.339	12118.940	10.42
EVAPOTRANSPIRATION	26.003	94391.898	81.18
DRAINAGE COLLECTED FROM LAYER 3	2.8860	10476.013	9.01
PERC./LEAKAGE THROUGH LAYER 5	0.000323	1.171	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0005		
DRAINAGE COLLECTED FROM LAYER 10	4.3345	15734.094	13.53
PERC./LEAKAGE THROUGH LAYER 11	0.000534	1.938	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4467		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.77
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.77
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		

CHANGE IN WATER STORAGE	-4.533	-16454.578	-14.15
SOIL WATER AT START OF YEAR	1127.015	4091065.500	
SOIL WATER AT END OF YEAR	1124.287	4081162.250	
SNOW WATER AT START OF YEAR	1.805	6551.396	5.63
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0002	0.555	0.00

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ANNUAL TOTALS FOR YEAR 27

	INCHES	CU. FEET	PERCENT
PRECIPITATION	26.31	95505.328	100.00
RUNOFF	2.777	10078.783	10.55
EVAPOTRANSPIRATION	19.146	69499.000	72.77
DRAINAGE COLLECTED FROM LAYER 3	4.6279	16799.256	17.59
PERC./LEAKAGE THROUGH LAYER 5	0.000523	1.898	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 10	4.2821	15543.880	16.28
PERC./LEAKAGE THROUGH LAYER 11	0.000527	1.915	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4409		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.94
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.94
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.523	-16417.420	-17.19
SOIL WATER AT START OF YEAR	1124.287	4081162.250	
SOIL WATER AT END OF YEAR	1119.764	4064744.750	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00

SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.195	0.00

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ANNUAL TOTALS FOR YEAR 28

	INCHES	CU. FEET	PERCENT
PRECIPITATION	27.74	100696.219	100.00
RUNOFF	2.774	10070.005	10.00
EVAPOTRANSPIRATION	20.133	73083.227	72.58
DRAINAGE COLLECTED FROM LAYER 3	5.0442	18310.590	18.18
PERC./LEAKAGE THROUGH LAYER 5	0.000536	1.945	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0009		
DRAINAGE COLLECTED FROM LAYER 10	4.1786	15168.454	15.06
PERC./LEAKAGE THROUGH LAYER 11	0.000514	1.865	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4293		
PERC./LEAKAGE THROUGH LAYER 13	0.000557	2.022	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.248278	901.250	0.90
DRAINAGE COLLECTED FROM LAYER 14	0.2488	903.274	0.90
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-4.391	-15937.588	-15.83
SOIL WATER AT START OF YEAR	1119.764	4064744.750	
SOIL WATER AT END OF YEAR	1114.047	4043989.250	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	1.327	4817.929	4.78
ANNUAL WATER BUDGET BALANCE	-0.0001	-0.488	0.00

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ANNUAL TOTALS FOR YEAR 29

	INCHES	CU. FEET	PERCENT
PRECIPITATION	28.79	104507.719	100.00
RUNOFF	1.771	6428.971	6.15
EVAPOTRANSPIRATION	20.773	75405.172	72.15
DRAINAGE COLLECTED FROM LAYER 3	4.2962	15595.091	14.92
PERC./LEAKAGE THROUGH LAYER 5	0.000487	1.768	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0008		
DRAINAGE COLLECTED FROM LAYER 10	4.1327	15001.867	14.35
PERC./LEAKAGE THROUGH LAYER 11	0.000508	1.845	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4256		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
VG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	0.86
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	0.86
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-2.183	-7925.903	-7.58
SOIL WATER AT START OF YEAR	1114.047	4043989.250	
SOIL WATER AT END OF YEAR	1111.920	4036271.000	
SNOW WATER AT START OF YEAR	1.327	4817.929	4.61
SNOW WATER AT END OF YEAR	1.270	4610.207	4.41
ANNUAL WATER BUDGET BALANCE	0.0001	0.498	0.00

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ANNUAL TOTALS FOR YEAR 30

	INCHES	CU. FEET	PERCENT
PRECIPITATION	22.90	83126.992	100.00
RUNOFF	2.339	8490.116	10.21
EVAPOTRANSPIRATION	21.789	79093.164	95.15
DRAINAGE COLLECTED FROM LAYER 3	2.8308	10275.635	12.36
PERC./LEAKAGE THROUGH LAYER 5	0.000315	1.144	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0005		
DRAINAGE COLLECTED FROM LAYER 10	4.0766	14798.027	17.80
PERC./LEAKAGE THROUGH LAYER 11	0.000501	1.819	0.00
AVG. HEAD ON TOP OF LAYER 11	0.4200		
PERC./LEAKAGE THROUGH LAYER 13	0.000555	2.016	0.00
AVG. HEAD ON TOP OF LAYER 13	0.2362		
SUBSURFACE INFLOW INTO LAYER 16	0.247600	898.788	1.08
DRAINAGE COLLECTED FROM LAYER 14	0.2482	900.806	1.08
PERC./LEAKAGE THROUGH LAYER 16	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 15	0.0001		
CHANGE IN WATER STORAGE	-8.136	-29532.232	-35.53
SOIL WATER AT START OF YEAR	1111.920	4036271.000	
SOIL WATER AT END OF YEAR	1104.738	4010200.500	
SNOW WATER AT START OF YEAR	1.270	4610.207	5.55
SNOW WATER AT END OF YEAR	0.316	1148.660	1.38
ANNUAL WATER BUDGET BALANCE	0.0001	0.264	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 30

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC

PRECIPITATION

TOTALS	1.47	1.46	2.39	3.26	3.34	4.22
	3.43	3.42	3.11	2.19	2.10	2.22
STD. DEVIATIONS	0.68	0.71	1.18	1.52	1.65	2.14
	1.83	1.76	1.76	1.22	1.06	0.97
RUNOFF						
TOTALS	0.126	0.593	1.347	0.190	0.004	0.018
	0.014	0.009	0.003	0.000	0.000	0.077
STD. DEVIATIONS	0.279	0.550	1.246	0.405	0.020	0.074
	0.051	0.027	0.017	0.001	0.001	0.163
EVAPOTRANSPIRATION						
TOTALS	0.544	0.466	0.830	2.962	3.902	4.493
	3.485	3.228	2.403	1.278	0.937	0.598
STD. DEVIATIONS	0.111	0.081	0.434	0.684	0.984	1.478
	1.706	1.610	1.090	0.310	0.215	0.157
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0934	0.0020	0.7395	1.9245	0.7208	0.1486
	0.1946	0.0300	0.0566	0.1693	0.4760	0.5897
STD. DEVIATIONS	0.2156	0.0109	0.8094	0.9997	0.7793	0.2544
	0.6243	0.0816	0.1590	0.4077	0.8422	0.5806
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0000	0.0000	0.0001	0.0002	0.0001	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0001	0.0001	0.0001	0.0000
	0.0001	0.0000	0.0000	0.0000	0.0001	0.0001
LATERAL DRAINAGE COLLECTED FROM LAYER 10						
TOTALS	0.4525	0.4246	0.4581	0.4374	0.4625	0.4400
	0.4462	0.4458	0.4438	0.4515	0.4407	0.4599
STD. DEVIATIONS	0.0838	0.0966	0.0815	0.0825	0.0871	0.0753
	0.0750	0.0831	0.0858	0.0760	0.0872	0.0858
PERCOLATION/LEAKAGE THROUGH LAYER 11						
TOTALS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 13						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

SUBSURFACE INFLOW INTO LAYER 16

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

LATERAL DRAINAGE COLLECTED FROM LAYER 14

TOTALS	0.0211	0.0192	0.0211	0.0204	0.0211	0.0204
	0.0211	0.0211	0.0204	0.0211	0.0204	0.0211
STD. DEVIATIONS	0.0000	0.0003	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 16

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0002	0.0000	0.0016	0.0042	0.0015	0.0003
	0.0004	0.0001	0.0001	0.0004	0.0010	0.0012
STD. DEVIATIONS	0.0005	0.0000	0.0017	0.0022	0.0016	0.0006
	0.0013	0.0002	0.0003	0.0009	0.0018	0.0012

DAILY AVERAGE HEAD ON TOP OF LAYER 11

AVERAGES	0.5489	0.5656	0.5557	0.5482	0.5610	0.5515
	0.5412	0.5407	0.5562	0.5476	0.5524	0.5578
STD. DEVIATIONS	0.1017	0.1289	0.0988	0.1034	0.1057	0.0943
	0.0909	0.1008	0.1076	0.0921	0.1093	0.1041

DAILY AVERAGE HEAD ON TOP OF LAYER 13

AVERAGES	0.2362	0.2362	0.2362	0.2362	0.2362	0.2362
	0.2362	0.2362	0.2362	0.2362	0.2362	0.2362
STD. DEVIATIONS	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002

DAILY AVERAGE HEAD ON TOP OF LAYER 15

AVERAGES	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	32.60	(5.565)	118325.9	100.00
RUNOFF	2.382	(1.4552)	8645.66	7.307
EVAPOTRANSPIRATION	25.127	(4.0797)	91211.53	77.085
LATERAL DRAINAGE COLLECTED FROM LAYER 3	5.14525	(1.82591)	18677.254	15.78458
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00058	(0.00020)	2.090	0.00177
AVERAGE HEAD ON TOP OF LAYER 4	0.001	(0.000)		
LATERAL DRAINAGE COLLECTED FROM LAYER 10	$\boxed{5.36283}$	(0.93602)	19467.072	16.45208
RCOLATION/LEAKAGE THROUGH LAYER 11	0.00067	(0.00013)	2.435	0.00206
AVERAGE HEAD ON TOP OF LAYER 11	0.552	(0.097)		
PERCOLATION/LEAKAGE THROUGH LAYER 13	0.00056	(0.00000)	2.018	0.00171
AVERAGE HEAD ON TOP OF LAYER 13	0.236	(0.000)		
SUBSURFACE INFLOW INTO LAYER 16	0.24760		898.788	0.75959
LATERAL DRAINAGE COLLECTED FROM LAYER 14	0.24831	(0.00024)	901.382	0.76178
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.00000	(0.00000)	0.000	0.00000
AVERAGE HEAD ON TOP OF LAYER 15	0.000	(0.000)		
CHANGE IN WATER STORAGE	-5.421	(1.5409)	-19677.65	-16.630

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	4.09	14846.700
RUNOFF	1.479	5368.5151
DRAINAGE COLLECTED FROM LAYER 3	0.64158	2328.93848
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000058	0.21054
AVERAGE HEAD ON TOP OF LAYER 4	0.042	
MAXIMUM HEAD ON TOP OF LAYER 4	0.134	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 10	$\boxed{0.05236}$	190.04968
PERCOLATION/LEAKAGE THROUGH LAYER 11	0.000008	0.02869
AVERAGE HEAD ON TOP OF LAYER 11	1.969	
MAXIMUM HEAD ON TOP OF LAYER 11	$\boxed{3.396}$	
LOCATION OF MAXIMUM HEAD IN LAYER 10 (DISTANCE FROM DRAIN)	23.3 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 13	0.000002	0.00552
AVERAGE HEAD ON TOP OF LAYER 13	0.236	
DRAINAGE COLLECTED FROM LAYER 14	0.00068	2.46796
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.000000	0.00000
AVERAGE HEAD ON TOP OF LAYER 15	0.000	
MAXIMUM HEAD ON TOP OF LAYER 15	0.004	
LOCATION OF MAXIMUM HEAD IN LAYER 14 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	4.86	17645.5430
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.4284
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.1236

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas

ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 30

LAYER	(INCHES)	(VOL/VOL)
1	1.8237	0.1520
2	5.7190	0.2383
3	0.0020	0.0100
4	0.0000	0.0000
5	5.1240	0.4270
6	3.7200	0.3100
7	38.8944	0.2920
8	3.7200	0.3100
9	1029.1211	0.6126
10	0.7056	0.0588
11	0.0000	0.0000
12	0.1806	0.7646
13	0.0000	0.0000
14	0.0020	0.0100
15	0.0000	0.0000
16	15.3720	0.4270
SNOW WATER	0.316	

EXHIBIT 9
'HELP' MODEL RESULTS
POST CLOSURE PERIOD YEARS 31 - 131
WITH NO LEACHATE COLLECTION AND REMOVAL

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******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:      D:\tsca\PRECIP.D4
TEMPERATURE DATA FILE:        D:\tsca\TEMP.D7
SOLAR RADIATION DATA FILE:    D:\tsca\SOLAR.D13
EVAPOTRANSPIRATION DATA:      D:\tsca\EVAP_C.D11
SOIL AND DESIGN DATA FILE:    D:\tsca\SOILC100.D10
OUTPUT DATA FILE:             D:\tsca\CLINC100.OUT


TIME:  11:14     DATE:  10/ 9/2007


******************************************************************************

     TITLE:  CLINTON LF. NO.3 CHEMICAL WASTE UNIT (CLOSURE 31-131 YRS)

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.


                                  LAYER  1
                                  --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER   8
           THICKNESS                    =     12.00    INCHES
           POROSITY                     =      0.4630 VOL/VOL
           FIELD CAPACITY               =      0.2320 VOL/VOL
           WILTING POINT                =      0.1160 VOL/VOL
           INITIAL SOIL WATER CONTENT   =      0.1520 VOL/VOL
           EFFECTIVE SAT. HYD. COND.    =  0.369999994000E-03 CM/SEC
       NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  4.90
                FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 9

THICKNESS	=	24.00	INCHES
POROSITY	=	0.5010	VOL/VOL
FIELD CAPACITY	=	0.2840	VOL/VOL
WILTING POINT	=	0.1350	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.2383	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.190000006000E-03	CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	25.00	PERCENT
DRAINAGE LENGTH	=	875.0	FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.04	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR	

LAYER 5

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4270	VOL/VOL
FIELD CAPACITY	=	0.4180	VOL/VOL
WILTING POINT	=	0.3670	VOL/VOL

```
INITIAL SOIL WATER CONTENT   =       0.4270 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.999999975000E-05 CM/SEC


                      LAYER  6
                      --------

           TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  11
THICKNESS                    =      12.00   INCHES
POROSITY                     =       0.4640 VOL/VOL
FIELD CAPACITY               =       0.3100 VOL/VOL
WILTING POINT                =       0.1870 VOL/VOL
INITIAL SOIL WATER CONTENT   =       0.3100 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.639999998000E-04 CM/SEC


                      LAYER  7
                      --------

           TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  18
THICKNESS                    =     133.20   INCHES
POROSITY                     =       0.6710 VOL/VOL
FIELD CAPACITY               =       0.2920 VOL/VOL
WILTING POINT                =       0.0770 VOL/VOL
INITIAL SOIL WATER CONTENT   =       0.2920 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.100000005000E-02 CM/SEC


                      LAYER  8
                      --------

           TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER  11
THICKNESS                    =      12.00   INCHES
POROSITY                     =       0.4640 VOL/VOL
FIELD CAPACITY               =       0.3100 VOL/VOL
WILTING POINT                =       0.1870 VOL/VOL
INITIAL SOIL WATER CONTENT   =       0.3100 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.639999998000E-04 CM/SEC


                      LAYER  9
                      --------

           TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                    =    1680.00   INCHES
POROSITY                     =       0.8500 VOL/VOL
FIELD CAPACITY               =       0.8000 VOL/VOL
WILTING POINT                =       0.1350 VOL/VOL
```

INITIAL SOIL WATER CONTENT = 0.6126 VOL/VOL
EFFECTIVE SAT. HYD. COND. = 0.190000006000E-03 CM/SEC

LAYER 10

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4170	VOL/VOL
FIELD CAPACITY	=	0.0450	VOL/VOL
WILTING POINT	=	0.0180	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0588	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.299999993000E-01	CM/SEC

LAYER 11

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR	

LAYER 12

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24	INCHES
POROSITY	=	0.7500	VOL/VOL
FIELD CAPACITY	=	0.7470	VOL/VOL
WILTING POINT	=	0.4000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08	CM/SEC

LAYER 13

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES

POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 14

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC

LAYER 15

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	10.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	4 - POOR

LAYER 16

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 16

THICKNESS	=	36.00 INCHES
POROSITY	=	0.4270 VOL/VOL
FIELD CAPACITY	=	0.4180 VOL/VOL
WILTING POINT	=	0.3670 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4270 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000001000E-06 CM/SEC
SUBSURFACE INFLOW	=	0.25 INCHES/YR

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 8 WITH A GOOD STAND OF GRASS, A SURFACE SLOPE OF 25.% AND A SLOPE LENGTH OF 875. FEET.

SCS RUNOFF CURVE NUMBER	=	72.60	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	20.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	3.730	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	9.564	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	2.472	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	1104.428	INCHES
TOTAL INITIAL WATER	=	1104.428	INCHES
TOTAL SUBSURFACE INFLOW	=	0.25	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM Lincoln Illinois

STATION LATITUDE	=	40.10	DEGREES
MAXIMUM LEAF AREA INDEX	=	4.00	
START OF GROWING SEASON (JULIAN DATE)	=	117	
END OF GROWING SEASON (JULIAN DATE)	=	290	
EVAPORATIVE ZONE DEPTH	=	20.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	10.30	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	71.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	65.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	70.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	72.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
1.60	1.31	2.59	3.66	3.15	4.08
3.63	3.53	3.35	2.28	2.06	2.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR CHICAGO ILLINOIS

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
21.40	26.00	36.00	48.80	59.10	68.60
73.00	71.90	64.70	53.50	39.80	27.70

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR CHICAGO ILLINOIS
AND STATION LATITUDE = 40.10 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	1.51	1.36	2.62	3.62	3.12	4.39
	4.02	3.61	3.26	2.35	2.23	2.07
STD. DEVIATIONS	0.65	0.69	1.12	1.58	1.43	2.05
	1.99	1.85	1.76	1.34	1.21	1.03
RUNOFF						
TOTALS	0.136	0.507	1.484	0.460	0.001	0.017
	0.019	0.018	0.003	0.002	0.001	0.135
STD. DEVIATIONS	0.338	0.515	1.308	0.874	0.011	0.061
	0.065	0.083	0.013	0.010	0.005	0.364
EVAPOTRANSPIRATION						
TOTALS	0.538	0.450	0.763	2.908	3.754	4.674
	3.971	3.515	2.359	1.287	0.898	0.583
STD. DEVIATIONS	0.116	0.103	0.452	0.764	1.016	1.383
	1.617	1.544	0.984	0.256	0.197	0.162
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.1179	0.0006	0.6174	2.0621	0.7699	0.1975
	0.1551	0.1038	0.1246	0.2859	0.5208	0.6603
STD. DEVIATIONS	0.2168	0.0060	0.8607	1.0588	0.6937	0.3784
	0.4433	0.3553	0.4560	0.7458	0.7742	0.7173
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0000	0.0000	0.0001	0.0002	0.0001	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0001	0.0001

STD. DEVIATIONS	0.0000	0.0000	0.0001	0.0001	0.0001	0.0000
	0.0000	0.0000	0.0000	0.0001	0.0001	0.0001

PERCOLATION/LEAKAGE THROUGH LAYER 11

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 13

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

SUBSURFACE INFLOW INTO LAYER 16

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

PERCOLATION/LEAKAGE THROUGH LAYER 16

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0002	0.0000	0.0013	0.0045	0.0016	0.0004
	0.0003	0.0002	0.0003	0.0006	0.0011	0.0014
STD. DEVIATIONS	0.0005	0.0000	0.0018	0.0023	0.0015	0.0008
	0.0009	0.0008	0.0010	0.0016	0.0017	0.0015

DAILY AVERAGE HEAD ON TOP OF LAYER 11

AVERAGES	0.3859	0.3858	0.3857	0.3857	0.3856	0.3855
	0.3855	0.3854	0.3853	0.3852	0.3851	0.3850
STD. DEVIATIONS	0.0300	0.0300	0.0299	0.0299	0.0298	0.0298
	0.0297	0.0297	0.0297	0.0296	0.0296	0.0296

DAILY AVERAGE HEAD ON TOP OF LAYER 13

AVERAGES	0.1605	0.1604	0.1603	0.1602	0.1602	0.1601
	0.1600	0.1599	0.1598	0.1597	0.1596	0.1595

STD. DEVIATIONS	0.0318	0.0318	0.0317	0.0316	0.0316	0.0315
	0.0314	0.0314	0.0313	0.0313	0.0312	0.0311

DAILY AVERAGE HEAD ON TOP OF LAYER 15

AVERAGES	0.1981	0.1984	0.1986	0.1988	0.1991	0.1993
	0.1996	0.1998	0.2000	0.2000	0.2000	0.2000
STD. DEVIATIONS	0.0188	0.0164	0.0140	0.0115	0.0090	0.0066
	0.0041	0.0016	0.0000	0.0000	0.0000	0.0000

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

	INCHES		CU. FEET	PERCENT
PRECIPITATION	34.15	(5.545)	123948.9	100.00
RUNOFF	2.784	(1.6894)	10105.48	8.153
EVAPOTRANSPIRATION	25.700	(3.5726)	93290.58	75.265
LATERAL DRAINAGE COLLECTED FROM LAYER 3	5.61599	(2.38234)	20386.039	16.44713
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00063	(0.00026)	2.270	0.00183
AVERAGE HEAD ON TOP OF LAYER 4	0.001	(0.000)		
PERCOLATION/LEAKAGE THROUGH LAYER 11	0.00046	(0.00004)	1.666	0.00134
AVERAGE HEAD ON TOP OF LAYER 11	0.385	(0.030)		
PERCOLATION/LEAKAGE THROUGH LAYER 13	0.00046	(0.00004)	1.678	0.00135
AVERAGE HEAD ON TOP OF LAYER 13	0.160	(0.031)		
SUBSURFACE INFLOW INTO LAYER 16	0.00000		0.000	0.00000
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.00000	(0.00000)	0.000	0.00000
AVERAGE HEAD ON TOP OF LAYER 15	0.199	(0.007)		
CHANGE IN WATER STORAGE	0.294	(1.7761)	1066.61	0.861

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 100

	(INCHES)	(CU. FT.)
PRECIPITATION	4.64	16843.199
RUNOFF	2.044	7419.4551
DRAINAGE COLLECTED FROM LAYER 3	0.72682	2638.33862
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000063	0.23039
AVERAGE HEAD ON TOP OF LAYER 4	0.048	
MAXIMUM HEAD ON TOP OF LAYER 4	0.176	
LOCATION OF MAXIMUM HEAD IN LAYER 3 (DISTANCE FROM DRAIN)	0.0 FEET	
PERCOLATION/LEAKAGE THROUGH LAYER 11	0.000001	0.00529
AVERAGE HEAD ON TOP OF LAYER 11	$\boxed{0.445}$	
PERCOLATION/LEAKAGE THROUGH LAYER 13	0.000002	0.00552
AVERAGE HEAD ON TOP OF LAYER 13	0.236	
PERCOLATION/LEAKAGE THROUGH LAYER 16	0.000000	0.00000
AVERAGE HEAD ON TOP OF LAYER 15	0.200	
SNOW WATER	7.00	25420.0430

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4350
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1236

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 100

LAYER	(INCHES)	(VOL/VOL)
1	5.0548	0.4212
2	7.0196	0.2925
3	0.0020	0.0100
4	0.0000	0.0000
5	5.1240	0.4270
6	3.7200	0.3100
7	38.8944	0.2920
8	3.7200	0.3100
9	1029.2184	0.6126
10	0.6719	0.0560
11	0.0000	0.0000
12	0.1768	0.7486
13	0.0000	0.0000
14	24.8375	124.1876
15	0.0000	0.0000
16	15.3720	0.4270
SNOW WATER	0.000	

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