

Designing with Geosynthetics

Fifth Edition

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Contents

PREFACE xvii

1 OVERVIEW OF GEOSYNTHETICS 1

1.0 Introduction 2

1.1 Basic Description of Geosynthetics 3

1.1.1 Types of Geosynthetics 5

1.1.2 Organization by Function 8

1.1.3 Market Activity 8

1.2 Polymeric Materials 9

1.2.1 Brief Overview 10

1.2.2 Polymer Identification 15

1.2.3 Polymer Formulations 27

1.3 Overview of Geotextiles 29

1.3.1 History 29

1.3.2 Manufacture 30

1.3.3 Current Uses 38

1.3.4 Sales 40

v

1.4	Overview of Geogrids	41
1.4.1	History	41
1.4.2	Manufacture	42
1.4.3	Current Uses	43
1.4.4	Sales	44
1.5	Overview of Geonets	44
1.5.1	History	44
1.5.2	Manufacture	45
1.5.3	Current Uses	47
1.5.4	Sales	47
1.6	Overview of Geomembranes	48
1.6.1	History	48
1.6.2	Manufacture	49
1.6.3	Current Uses	56
1.6.4	Sales	58
1.7	Overview of Geosynthetic Clay Liners	59
1.7.1	History	59
1.7.2	Manufacture	59
1.7.3	Current Uses	61
1.7.4	Sales	62
1.8	Overview of Geopipe (aka Plastic Pipe)	62
1.8.1	History	62
1.8.2	Manufacture	63
1.8.3	Current Uses	64
1.8.4	Sales	65
1.9	Overview of Geofoam	66
1.9.1	History	66
1.9.2	Manufacture	66
1.9.3	Current Uses	68
1.9.4	Sales	68

1.10 Overview of Geocomposites 69

- 1.10.1 Geotextile-Geonet Composites 69
- 1.10.2 Geotextile-Geomembrane Composites 69
- 1.10.3 Geomembrane-Geogrid Composites 69
- 1.10.4 Geotextile-Geogrid Composites 70
- 1.10.5 Geotextile-Polymer Core Composites 70
- 1.10.6 Geosynthetic-Soil Composites 70
- 1.10.7 Other Geocomposites 72

1.11 Outline of Book 72

References 73

Problems 74

2 DESIGNING WITH GEOTEXTILES 79

2.0 Introduction 81

2.1 Design Methods 81

- 2.1.1 Design by Cost and Availability 82
- 2.1.2 Design by Specification 82
- 2.1.3 Design by Function 92

2.2 Geotextile Functions and Mechanisms 93

- 2.2.1 Separation 93
- 2.2.2 Reinforcement 94
- 2.2.3 Filtration 98
- 2.2.4 Drainage 103
- 2.2.5 Containment 105
- 2.2.6 Combined Functions 105

2.3 Geotextile Properties and Test Methods 106

- 2.3.1 General Comments 106
- 2.3.2 Physical Properties 107
- 2.3.3 Mechanical Properties 108
- 2.3.4 Hydraulic Properties 128
- 2.3.5 Endurance Properties 140
- 2.3.6 Degradation Considerations 152
- 2.3.7 Summary 161

- 2.4 Allowable Versus Ultimate Geotextile Properties 162**
 - 2.4.1 Strength-Related Problems 162
 - 2.4.2 Flow-Related Problems 165
- 2.5 Designing for Separation 166**
 - 2.5.1 Overview of Applications 166
 - 2.5.2 Burst Resistance 166
 - 2.5.3 Tensile Strength 168
 - 2.5.4 Puncture Resistance 171
 - 2.5.5 Impact (Tear) Resistance 173
 - 2.5.6 Summary 176
- 2.6 Designing for Roadway Reinforcement 177**
 - 2.6.1 Unpaved Roads 177
 - 2.6.2 Membrane-Encapsulated Soils 188
 - 2.6.3 Paved Roads 196
- 2.7 Designing for Soil Reinforcement 197**
 - 2.7.1 Geotextile Reinforced Walls 197
 - 2.7.2 Geotextile Reinforced Embankments 216
 - 2.7.3 Geotextile Reinforced Foundation Soils 226
 - 2.7.4 Geotextiles for Improved Bearing Capacity and Basal Reinforcement 235
 - 2.7.5 Geotextiles for In Situ Slope Stabilization 239
- 2.8 Designing for Filtration 246**
 - 2.8.1 Overview of Applications 246
 - 2.8.2 General Behavior 246
 - 2.8.3 Geotextiles Behind Retaining Walls 247
 - 2.8.4 Geotextiles Around Underdrains 251
 - 2.8.5 Geotextiles Beneath Erosion-Control Structures 254
 - 2.8.6 Geotextiles Silt Fences 257
 - 2.8.7 Summary 263
- 2.9 Designing for Drainage 263**
 - 2.9.1 Overview of Applications 263
 - 2.9.2 General Behavior 264
 - 2.9.3 Gravity Drainage Design 265

2.9.4	Pressure Drainage Design	269
2.9.5	Capillary Migration Breaks	270
2.9.6	Summary	272
2.10	Designing for Multiple Functions	273
2.10.1	Logic for Chapter	273
2.10.2	Reflection Crack Prevention in Pavement Overlays	273
2.10.3	Railroad Applications	285
2.10.4	Flexible Forming Systems	288
2.11	Construction Methods and Techniques Using Geotextiles	304
2.11.1	Introduction	304
2.11.2	Geotextile Installation Survivability	304
2.11.3	Cost and Availability Considerations	306
2.11.4	Summary	306
	References	307
	Problems	314
3	DESIGNING WITH GEOGRIDS	328
3.0	Introduction	328
3.1	Geogrid Properties and Test Methods	331
3.1.1	Physical Properties	331
3.1.2	Mechanical Properties	332
3.1.3	Endurance Properties	343
3.1.4	Degradation Issues	344
3.1.5	Allowable Strength Considerations	347
3.2	Designing for Geogrid Reinforcement	349
3.2.1	Paved Roads—Base Courses	349
3.2.2	Paved Roads—Pavements	351
3.2.3	Unpaved Roads	354
3.2.4	Embankments and Slopes	356
3.2.5	Reinforced Walls	362
3.2.6	Foundation and Basal Reinforcement	374
3.2.7	Veneer Cover Soils	380

x

- 3.3 Design Critique 387
- 3.4 Construction Methods 388
- References 389
- Problems 392

4 DESIGNING WITH GEONETS 396

- 4.0 Introduction 396
- 4.1 Geonet Properties and Test Methods 397
 - 4.1.1 Physical Properties 397
 - 4.1.2 Mechanical Properties 400
 - 4.1.3 Hydraulic Properties 403
 - 4.1.4 Endurance Properties 408
 - 4.1.5 Environmental Properties 411
 - 4.1.6 Allowable Flow Rate 412
- 4.2 Designing for Geonet Drainage 415
 - 4.2.1 Theoretical Concepts 415
 - 4.2.2 Environmental-Related Applications 416
 - 4.2.3 Transportation-Related Applications 420
- 4.3 Design Critique 423
- 4.4 Construction Methods 424
- References 425
- Problems 426

5 DESIGNING WITH GEOMEMBRANES 428

- 5.0 Introduction 430
- 5.1 Geomembrane Properties and Test Methods 431
 - 5.1.1 Overview 431
 - 5.1.2 Physical Properties 432
 - 5.1.3 Mechanical Properties 439
 - 5.1.4 Endurance Properties 458
 - 5.1.5 Lifetime Prediction 467
 - 5.1.6 Summary 474

- 5.2 Survivability Requirements 474**
- 5.3 Liquid Containment (Pond) Liners 476**
 - 5.3.1 Geometric Considerations 476
 - 5.3.2 Typical Cross Sections 478
 - 5.3.3 Geomembrane Material Selection 482
 - 5.3.4 Thickness Considerations 483
 - 5.3.5 Side-Slope Considerations 487
 - 5.3.6 Runout and Anchor Trench Design 500
 - 5.3.7 Summary 506
- 5.4 Covers for Reservoirs and Quasi-Solids 506**
 - 5.4.1 Overview 507
 - 5.4.2 Fixed Covers 507
 - 5.4.3 Floating Covers 509
 - 5.4.4 Quasi-Solid Covers 515
 - 5.4.5 Complete Encapsulation 515
- 5.5 Water Conveyance (Canal) Liners 517**
 - 5.5.1 Overview 517
 - 5.5.2 Basic Considerations 517
 - 5.5.3 Unique Features 521
 - 5.5.4 Summary 525
- 5.6 Solid-Material (Landfill) Liners 525**
 - 5.6.1 Overview 527
 - 5.6.2 Siting Considerations and Geometry 531
 - 5.6.3 Typical Cross Sections 532
 - 5.6.4 Grading and Leachate Removal 539
 - 5.6.5 Material Selection 545
 - 5.6.6 Thickness 546
 - 5.6.7 Puncture Protection 547
 - 5.6.8 Runout and Anchor Trenches 549
 - 5.6.9 Side Slope Subgrade Soil Stability 550
 - 5.6.10 Multilined Side Slope Cover Soil Stability 550
 - 5.6.11 Access Ramps 554
 - 5.6.12 Stability of Solid-Waste Masses 554
 - 5.6.13 Vertical Expansion (Piggyback) Landfills 558
 - 5.6.14 Heap Leach Pads 559
 - 5.6.15 Solar Ponds 559
 - 5.6.16 Summary 560

5.7	Landfill Covers and Closures	563
5.7.1	Overview	563
5.7.2	Various Cross Sections	564
5.7.3	Gas Collection Layer	566
5.7.4	Barrier Layer	568
5.7.5	Infiltrating Water Drainage Layer	570
5.7.6	Protection (Cover Soil) Layer	571
5.7.7	Surface (Top Soil) Layer	571
5.7.8	Post-Closure Beneficial Uses and Aesthetics	572
5.8	Wet (Bioreactor) Landfills	573
5.8.1	Background	574
5.8.2	Base Liner Systems	575
5.8.3	Leachate Collection System	575
5.8.4	Leachate Removal System	577
5.8.5	Filter and/or Operations Layer	577
5.8.6	Daily Cover Materials	577
5.8.7	Final Cover Issues	577
5.8.8	Waste Stability Concerns	578
5.8.9	Summary	579
5.9	Underground Storage Tanks	579
5.9.1	Overview	579
5.9.2	Low-Volume Systems	579
5.9.3	High-Volume Systems	581
5.9.4	Tank Farms	581
5.10	Hydraulic and Geotechnical Applications	581
5.10.1	Earth and Earth/Rock Dams	581
5.10.2	Concrete and Masonry Dams	583
5.10.3	Roller-Compacted Concrete Dams	583
5.10.4	Geomembrane Dams	585
5.10.5	Tunnels	585
5.10.6	Vertical Cutoff Walls	585
5.11	Geomembrane Seams	587
5.11.1	Seaming Methods	589
5.11.2	Destructive Seam Tests	593
5.11.3	Nondestructive Seam Tests	596
5.11.4	Summary	599

5.12 Details and Miscellaneous Items 602

- 5.12.1 Connections 602
- 5.12.2 Appurtenances 602
- 5.12.3 Leak Location (After Waste Placement) Techniques 606
- 5.12.4 Wind Uplift 607
- 5.12.5 Quality Control and Quality Assurance 608

5.13 Concluding Remarks 611**References 612****Problems 618****6 GEOSYNTHETIC CLAY LINERS 630****6.0 Introduction 630****6.1 GCL Properties and Test Methods 634**

- 6.1.1 Physical Properties 634
- 6.1.2 Hydraulic Properties 636
- 6.1.3 Mechanical Properties 642
- 6.1.4 Endurance Properties 647

6.2 Equivalency Issues 649**6.3 Designing with GCLs 652**

- 6.3.1 GCLs as Single Liners 652
- 6.3.2 GCLs as Composite Liners 654
- 6.3.3 GCLs as Composite Covers 657
- 6.3.4 GCLs on Slopes 659

6.4 Design Critique 661**6.5 Construction Methods 663****References 665****Problems 667****7 DESIGNING WITH GEOPIPES 669****7.0 Introduction 670****7.1 Geopipe Properties and Test Methods 672**

- 7.1.1 Physical Properties 672
- 7.1.2 Mechanical Properties 675

7.1.3	Chemical Properties	684
7.1.4	Biological Properties	685
7.1.5	Thermal Properties	686
7.1.6	Geopipe Specifications	686
7.2	Theoretical Concepts	689
7.2.1	Hydraulic Issues	689
7.2.2	Deflection Issues	694
7.3	Design Applications	698
7.3.1	Pavement Underdrains—Perforated Profiled Collection Pipes	699
7.3.2	Primary Leachate Collection Systems—Perforated Profiled Collection Pipes	702
7.3.3	Liquid Transmission—Solid-Wall Nonperforated Pipe with Deflection Calculations	704
7.4	Design Critique	705
7.5	Construction Methods	706
7.5.1	Subgrade Preparation	707
7.5.2	Connections	709
7.5.3	Placement	711
7.5.4	Backfilling Operations	711
	References	712
	Problems	713
8	DESIGNING WITH GEOFOAM	715
8.0	Introduction	715
8.1	Geofoam Properties and Test Methods	716
8.1.1	Physical Properties	716
8.1.2	Mechanical Properties	719
8.1.3	Thermal Properties	722
8.1.4	Endurance Properties	723
8.2	Design Applications	724
8.2.1	Lightweight Fill	724
8.2.2	Compressible Inclusion	726

- 8.2.3 Thermal Insulation 729
- 8.2.4 Drainage Applications 731

- 8.3 Design Critique 732**
- 8.4 Construction Methods 733**
- References 734**
- Problems 735**

9 DESIGNING WITH GEOCOMPOSITES 736

9.0 Introduction 737

9.1 Geocomposites in Separation 737

- 9.1.1 Temporary Erosion and Revegetation Materials 740
- 9.1.2 Permanent Erosion and Revegetation Materials—
Biotechnical-Related 741
- 9.1.3 Permanent Erosion and Revegetation Materials—
Hard Armor-Related 741
- 9.1.4 Design Considerations 742
- 9.1.5 Summary 747

9.2 Geocomposites in Reinforcement 747

- 9.2.1 Reinforced Geotextile Composites 747
- 9.2.2 Reinforced Geomembrane Composites 749
- 9.2.3 Reinforced Soil Composites 749
- 9.2.4 Reinforced Concrete Composites 755
- 9.2.5 Reinforced Bitumen Composites 755

9.3 Geocomposites in Filtration 756

9.4 Geocomposites in Drainage 757

- 9.4.1 Wick (Prefabricated Vertical) Drains 758
- 9.4.2 Sheet Drains 769
- 9.4.3 Highway Edge Drains 776

9.5 Geocomposites in Containment (Liquid/Vapor Barriers) 779

9.6 Conclusion 782

References 782

Problems 784

INDEX 788

If fines (silts and/or clays) are allowed for the reinforced zone backfill soil, any possible water in front, behind, and beneath the reinforced zone must be carefully collected, transmitted, and discharged. Proper drainage control is absolutely critical in this regard. Furthermore, the top of the zone should be waterproofed—for example, by a geomembrane or a geosynthetic clay liner—to prevent water from entering the backfill zone from the surface. Surface water drainage as well as drainage from the retained earth zone is obviously of concern with respect to potential buildup of pore water pressures behind or within the reinforced soil zone. (See Koerner and Soong [46] for wall drainage system designs in this regard.)

In closing this section on geogrid reinforced walls, the current tendency to create live (or evergreen) walls with open facing should be mentioned. As we saw earlier in Figure 3.14, the sequence is a steel wire mesh (alternatively a gabion), backed by a bidirectional geogrid and then by a geosynthetic erosion control material. The reinforcing geogrids (always unidirectional types) are either attached to the steel wire mesh facing, or they are frictionally connected by sufficient overlap length. Such walls avoid masonry block durability concerns and offer a considerably less expensive wall system. Of course, the durability of the steel wire and bidirectional geogrid backup must be considered and this is a viable research topic when considering 100-year permanent wall lifetimes.

3.2.6 Foundation and Basal Reinforcement

Geogrids have been used to increase bearing capacity of poor foundation soils in different ways: as a continuous layer, as multiple closely spaced continuous layers with granular soil between layers, and as mattresses consisting of three-dimensional interconnected cells. The technical database for the single-layer continuous sheets has been reported by Jarrett [47] and by Milligan and Love [48]; in both cases large-scale laboratory tests are used. Figure 3.19 presents some of Milligan and Love's work graphed in the conventional nondimensionalized q/c_u versus ρ/B manner and also as $q/\sqrt{c_u}$ versus ρ/B where q is the bearing capacity and ρ is the settlement. The latter graph is not conventional but does sort out the data nicely. Clearly shown in both instances is the marked improvement in load-carrying capacity using geogrids at high deformation and only a nominal beneficial effect at low deformation. Beyond these observations, a precise design formulation is not currently available.

Instead of focusing on a global increase in bearing capacity, it is quite likely that single or multiple layers of geogrid (or geotextile) will aid in minimizing or eliminating differential settlement. Here localized settlements due to abruptly settling or subsiding weak zones can be spanned by the layer of reinforcement. This is known as *foundation improvement* (rather than bearing capacity via base reinforcement). Notable in this regard is a technique called *piggybacking*—the construction of new landfills above existing landfills. The approach is to use arching theory in the calculation of the vertical stress arising from localized subsidence (i.e., differential settlement) and to provide suitably strong reinforcement.

It should be recognized that arching in natural soils overlying a locally yielding foundation is well established. In the 1930s, both Karl Terzaghi in Austria (calculating

$$\begin{aligned}
 FS &= \frac{-b + \sqrt{b^2 - 4ac}}{2a} \\
 &= \frac{13.8 + \sqrt{(-13.8)^2 - 4(6.1)(2.22)}}{2(6.1)} \\
 FS &= 2.10
 \end{aligned}$$

While the value appears to be acceptable, it is nevertheless disconcerting that the liner system per se is being used as the veneer reinforcement mechanism. Had higher reduction factors been used, the resulting FS value would be proportionately decreased. That said, when the solid waste is placed against the leachate collection soil, a resisting berm is created, bringing stability to the situation at that time.

5.6.11 Access Ramps

For below-grade landfills it is necessary to grade the subgrade to accommodate the necessary access ramp(s), line the entire facility, and then construct a road above the liner cross section. A typical geometry is shown in Figure 5.46a. A particularly troublesome aspect of this design is that the road must be built above the completed liner system. A variety of problems have occurred in the past:

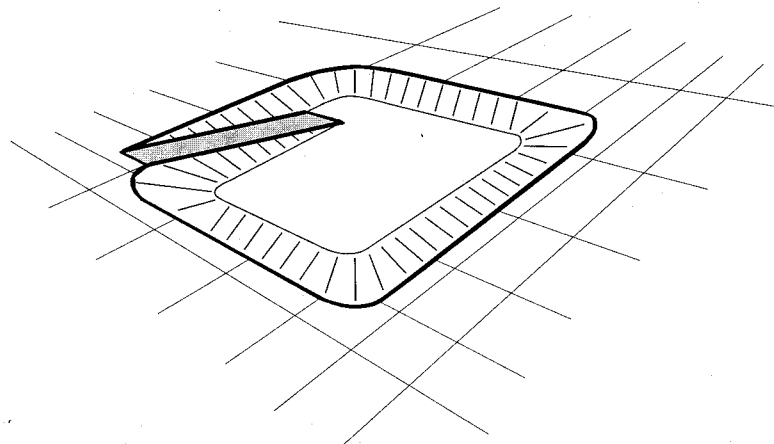
- Inadequate drainage where the ramp meets the upper slope, with subsequent erosion and scour of the roadway itself.
- Inadequate roadway material above the liner system, with ramp soil sliding off the upper geomembrane due to truck traffic.
- Inadequate roadway thickness above the liner system, with the upper geomembrane failing in tension along the slope due to truck traffic.
- Inadequate roadway thickness above the liner system, with an underlying hydrated GCL creating slippage of the overlying geomembrane and entire roadway.

Clearly, a conservative design is required; Figure 5.46b presents some recommendations. While a 600 to 900 mm thickness might seem excessive, the dynamic stresses caused by braking trucks are high, and furthermore, the ramp soil can be removed in whole or in part as the waste elevation rises during filling operations.

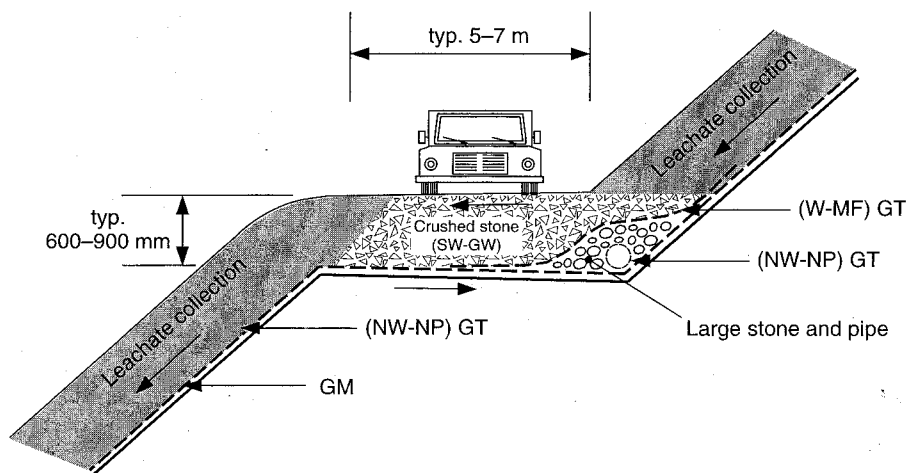
5.6.12 Stability of Solid-Waste Masses

Upon first consideration, the stability of solid waste failing within itself should present no particular concern since its shear strength characteristics should be quite high. Singh and Murphy [83] present shear strength parameters of solid waste transitioning from high in friction (24 to 36°) to being high in cohesion (80 to 120 kPa). Obviously, the aging of the waste is an issue, but at all times the shear strength is quite high. A widely used MSW shear strength envelope assembled by Kavazanjian [84] indicates a bilinear response of 33° friction transitioning at less than 30 kPa normal stress to a cohesion of 24 kPa.

Paradoxically, there have been some massive failures of solid waste. Koerner and Soong [85] report on ten such failures of which half were unlined or soil-lined sites, and



(a) Geometry of typical ramp-grades from gentle to 25% (14.0°)



(b) Suggested cross section

Figure 5.46 Typical geometry and cross section of a below-grade landfill access ramp.

half were at sites that contained geomembranes. Table 5.19a presents some details which were evaluated on the basis of both two-dimensional and three-dimensional analyses. On average the 3-D analyses were 16% higher than the comparable 2-D analyses. The 2-D representations of the individual failures are shown in Figure 5.47. Figure 5.48 shows the enormity of the problem at one of these sites. All of the failures were most dramatic and many involved litigation and fines, to say nothing of the deaths at one site and the environmental damage that ensued at all of the sites. The failure surfaces were either rotational or translational, the latter always occurring at the geomembrane-lined sites. Commercially available slope stability computer codes are

TABLE 5.19 SUMMARY OF LARGE LANDFILL FAILURES AND RELATED TRIGGERING MECHANISMS INVOLVED

(a) Site Listings and Related Information				
Identification	Year	Location	Type	Quantity of Waste Involved (m ³)
Unlined or soil-lined sites				
U-1	1984	North America	Single rotational	110,000
U-2	1989	North America	Multiple rotational	500,000
U-3	1993	Europe	Translational	470,000 ¹
U-4	1996	North America	Translational	1,100,000
U-5	1997	North America	Single rotational	100,000
Geomembrane-lined sites				
L-1	1988	North America	Translational	490,000
L-2	1994	Europe	Translational	60,000
L-3	1997	North America	Translational	100,000
L-4	1997	Africa	Translational	300,000
L-5	1997	South America	Translational	1,200,000

¹Included 27 deaths!**(b) Contributing Cause (Trigger) of Failures**

Case History	Reason for Low Initial FS Value	Triggering Mechanism
U-3	Leachate buildup within waste mass	Excessive buildup of leachate level due to ponding
U-4		Excessive buildup of leachate level due to ice formation
L-4		Excessive buildup of leachate level due to liquid waste injection
L-5		Excessive buildup of leachate level due to leachate injection
L-1	Wet clay beneath GM (i.e., GM/CCL or GM/GCL)	Excessive wetness of the GM/CCL interface
L-2		Excessive wetness of the GM/CCL interface
L-3		Excessive wetness of the bentonite in an unreinforced GCL
U-1	Wet foundation or soft backfill soil	Rapid rise in leachate level within the waste mass
U-2		Foundation soil excavation exposing soft clay
U-5		Excessive buildup of perched leachate level on clay liner

Source: After Koerner and Soong [85].

readily configured to handle these failures provided that accurate values of shear strength of the material and surfaces involved are known. The importance of direct shear testing (as described in Section 5.1.3) cannot be overstated.

While the stability factors of safety of all of the sites were relatively low prior to failure, each had a unique aspect that Koerner and Soong [85] call a *triggering mechanism*. It was found that all ten failures had triggering mechanisms that involved liquids. Table 5.19b groups the failures according to triggering mechanisms where the excessive liquids are either (1) in the waste mass itself above the liner system, (2) within

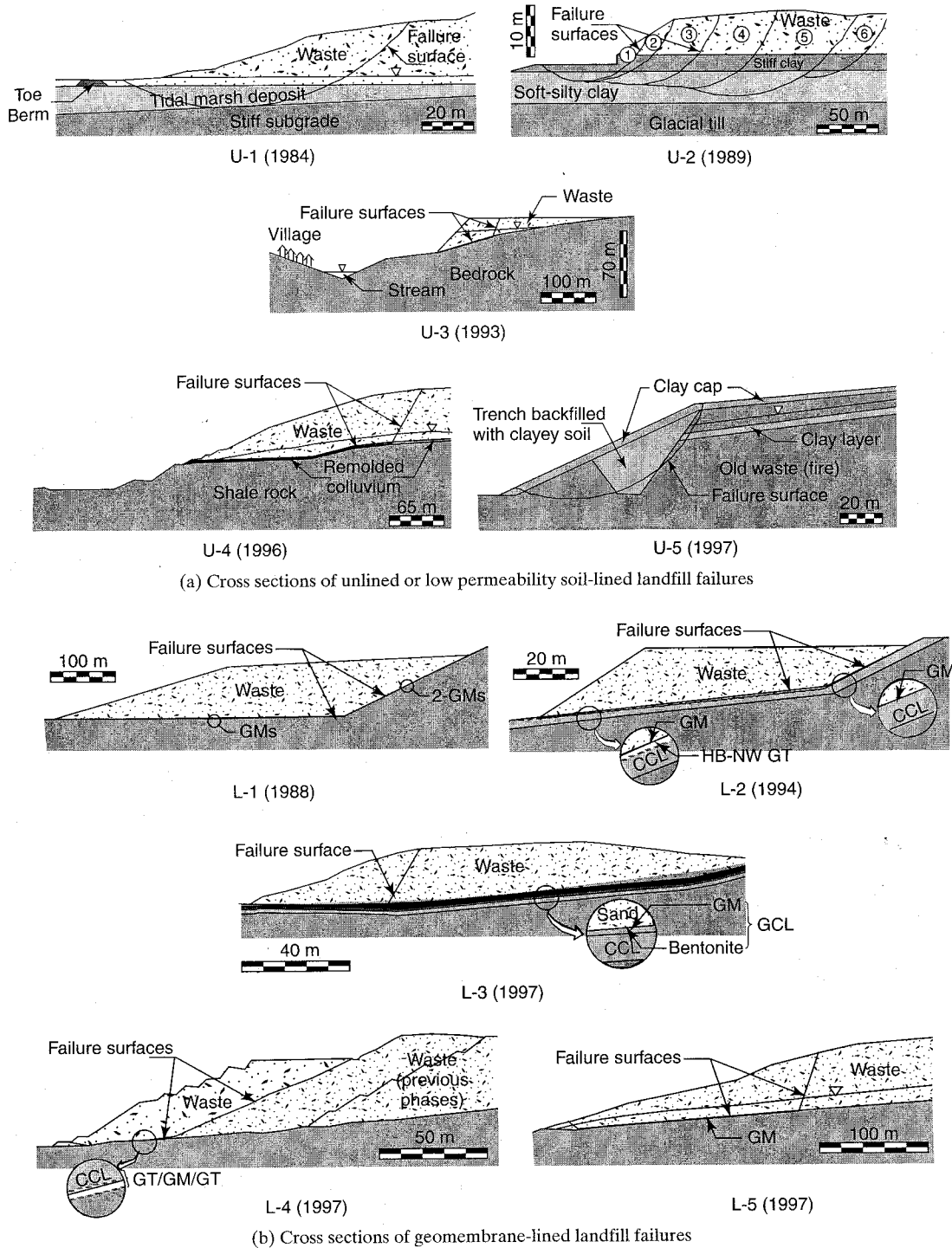
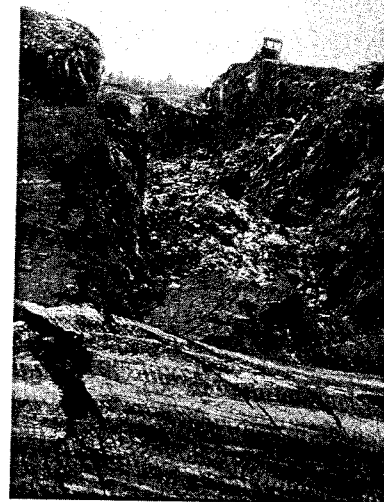


Figure 5.47 Two-dimensional cross sections of ten landfill failures. (After Koerner and Soong [85])



(a) Six individual failures which occurred sequentially within minutes of one another



(b) Solid waste within one of the failures

Figure 5.48 Failure of a municipal solid-waste landfill within the waste mass itself.

components of the liner system in the form of excessively wet CCLs or GCLs, or (3) in the foundation soil beneath the waste and/or liner system. This recognition of the negative influence of liquids on waste mass stability cannot be overemphasized. Of all of the problems mentioned in this book, this class of failures is the most serious and must be avoided at all costs.

5.6.13 Vertical Expansion (Piggyback) Landfills

In closing this section on geosynthetic systems related to solid waste, the concept of vertical expansions—*piggybacking* a new landfill on an existing one—should be mentioned. When many existing landfills are filled, there is nowhere else to go but up. Thus a new landfilling operation above an existing one sometimes becomes necessary. As noted in Qian et al. [86], certain precautions regarding this type of vertical expansion must be followed:

- Total settlement of the existing landfill must be anticipated and estimated accordingly. Thus, the slopes of the leachate collection system must reflect this requirement and will probably be quite high, as much as 10 to 15%.
- Estimation of differential settlements within the existing landfill may require a high-strength geogrid or geotextile network to be placed over all or a portion of the site (recall Section 3.2.6 and Example 3.11).
- Waste placement in the new landfill must be carefully sequenced to balance stress on the existing landfill [86]. The stability of the waste situation just discussed

is exacerbated greatly by the addition of a large surcharge stress, which is what the piggybacked landfill represents to the underlying waste.

- Methane gas (if generated) migrating from the existing landfill must be carried laterally under the new landfill liner to side-slope venting and/or collection locations. Active gas collection systems may be required.
- Leachate collection from the existing landfill should be considered. If required, directionally drilled withdrawal wells at the perimeter of the facility may be a consideration.
- Access to the site via haul roads must be carefully considered so that there will be no damage to, or instability of, the underlying liner system.

5.6.14 Heap Leach Pads

Heap leach pads consist of a geomembrane with an overlying drainage system, and then a precious metal (gold, silver, or copper) bearing ore heaped above. A cyanide or sulfuric acid solution is sprayed on top of the ore, leaches through it reacting with the metals, and carries the solution to the drainage system where it is collected. Beneath the drainage system is a geomembrane barrier, hence the topic is included at this location. Separation of the ore from the leachate occurs in an on-site processing plant. The leaching solution is renewed and the process is repeated until it is no longer economical. Figure 5.49a illustrates the general configuration.

The heap itself is often enormous in its proportions (see Figure 5.49b). Ores of 22 kN/m³ unit weight at heights up to 150 m produce enormous stresses on the drainage system and geomembrane. The concept is used widely in the western United States and Canada and in many South American countries (see Smith and Welkner [88]).

Regarding the design of the geomembrane, its thickness and type is very subjective and all resin types have been used to varying degrees. The drainage system is coarse gravel along with an embedded pipe system allowing for rapid and efficient removal of the ore-bearing solution from beneath the heap. This situation requires consideration of a sand cushion layer or a very thick protection geotextile between the geomembrane and drainage/collection gravel. The design method presented in Section 5.6.7 should be considered, with the reminder that it is developed on the basis that different geomembrane thicknesses and types will behave differently. Thiel and Smith [89] have summarized the key geotechnical concerns with respect to heap leach pads and related issues (see Table 5.20).

5.6.15 Solar Ponds

There are a number of solid material liner systems that have not yet been mentioned. A small but growing segment of these systems is solar ponds [90]. Here the geomembrane is placed in an excavation and then it is filled with salt. Solar energy is collected and stored as heat. A salt gradient effect is created, whereby zones are set up constantly replenishing new heat as it is gradually withdrawn from the lower storage zone for useful purposes. The main consideration insofar as the geomembrane is concerned is

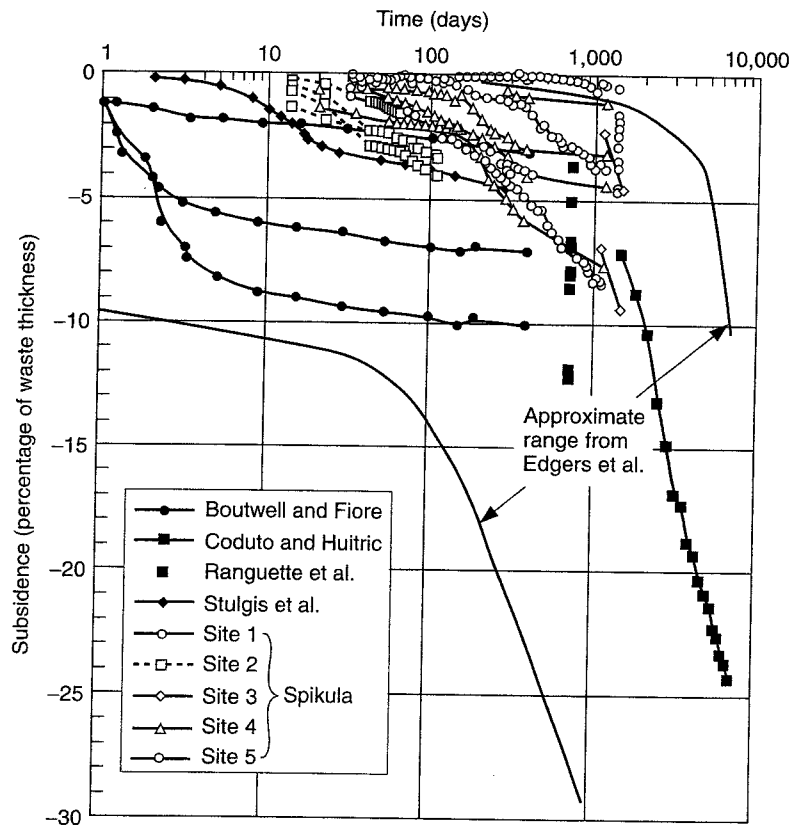


Figure 5.51 Municipal solid-waste landfill subsidence. (After Spikula [92])

- The geomembrane barrier above the compacted clay should have a minimum thickness of 0.75 mm.
- There should be adequate bedding above and below the geomembrane.
- The drainage layer above the geomembrane should have a minimum hydraulic conductivity of 0.01 cm/s and a final slope of 2% or greater after settlement and subsidence (thus necessitating subsidence predictions).
- The topsoil and protection soil above the drainage layer must have a minimum thickness of 600 mm.

As seen in Figure 5.50, there are many geosynthetic alternatives to the above-mentioned natural soils, for example:

- The CCL should be replaced by a GCL. (CCLs simply do not belong above a subsiding waste mass resulting in total and differential settlement.)
- The drainage layer could be replaced by a geocomposite or geonet drain.