THE ROLE OF SOIL MECHANICS

IN ENVIRONMENTAL GEOTECHNICS

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by

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THE ROLE OF SOIL MECHANICS IN ENVIRONMENTAL GEOTECHNICS

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SYNOPSIS Geotechnical considerations are central to most problems of environmental control, restoration and enhancement. Principles of soil mechanics, as they relate to soil, contaminated groundwater, and waste materials and to the design, construction, operation and maintenance of geoenvironmental earth structures are among the most important of the several tools used in geoenvironmental engineering. In particular, considerations of strength and stability, stress-deformation behavior, seepage and contaminant transport, and seismicity and soil dynamics are required. Case history examples of waste landfills, tailings dams, and slurry walls are used to illustrate the important role of soil mechanics in environmental geotechnics.

1. INTRODUCTION

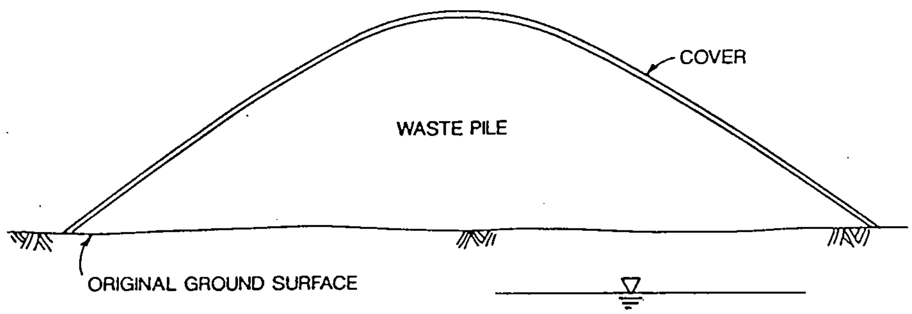
Worldwide concern for the cleanup, protection, and enhancement of the environment has resulted, over the past 15 to 20 years, in the development of a new specialization area or subdiscipline within the field of geotechnical engineering; namely, *environmental geotechnics,* which is concerned with the application of geotechnical engineering for environmental control. The broader term, *geoenvironmental engineering*, is reserved herein for the overall engineering of environmental projects that encompass the earth, groundwater, safe waste containment, site remediation, etc., and which require the expertise of scientists and engineers from several disciplines.

Soil mechanics, which served as the starting point and has become an essential building block of *geotechnical engineering*, has a very important role also in environmental geotechnics, and this role is the subject of this paper. In particular we focus on how soil mechanics, that is, the application of the laws and principles of mechanics and hydraulics to engineering problems dealing with soil as an engineering material, can be extended and applied to several different types of problems in environmental geotechnics. Examples taken both from the recent literature and our own experience are provided.

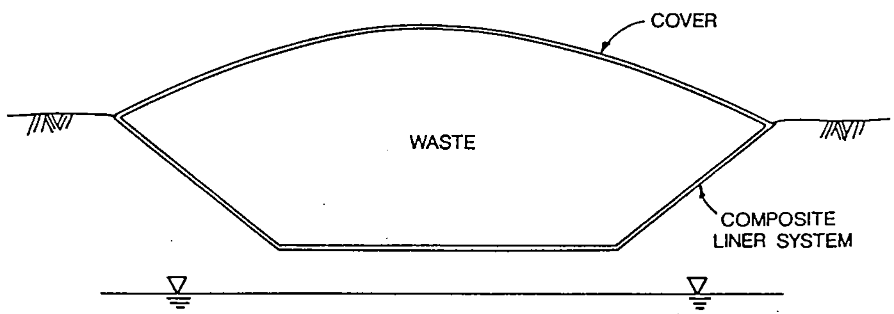
Soil mechanics is extended herein to include waste mechanics and geosynthetics mechanics and applied to problems involving strength and stability, settlement and stress-deformation behavior, seepage barriers and waste isolation, and waste containment facilities in seismic areas.

2. APPLICATIONS OF SOIL MECHANICS IN ENVIRONMENTAL GEOTECHNICS

Almost all geoenvironmental projects and problems requiring geotechnical expertise fall into three general categories; namely, (1) design, construction, operation, and maintenance of new waste disposal and containment facilities, (2) isolation of contaminated ground, and (3) remediation of contaminated sites. Soil mechanics plays a very important role in each of these, with perhaps the greatest need for the applications of mechanics in (1) and (2). It is in these two areas that we focus our coverage in this paper.



a. Waste Pile Over Existing Ground Surface



b. Landfill Within Excavated Area or Existing Valley

Fig. 1 Two General Types of Soild Waste Landfill

2.1 Waste disposal and Safe Containment of Wastes

Modern municipal solid waste and hazardous waste landfills are generally configured as shown schematically in Fig. 1. Primary concerns in the design and operation of such facilities are:

1. The liner system must restrict the escape of leachate to acceptable limits through a combination of an effective leachate collection and removal system and a suitably impervious seepage barrier. A typical modern composite double liner that is used for this purpose is a complex system as shown in Fig. 2. To assure proper performance over the long life of a waste landfill requires that there be chemical, biological, and mechanical compatibility between the several components. The latter involves consideration of the stress-deformation and strength behavior of each of the components alone and together under a range of static and dynamic loading conditions. The leachate collection and containment function requires application of hydraulic conductivity, seepage, and drainage principles.

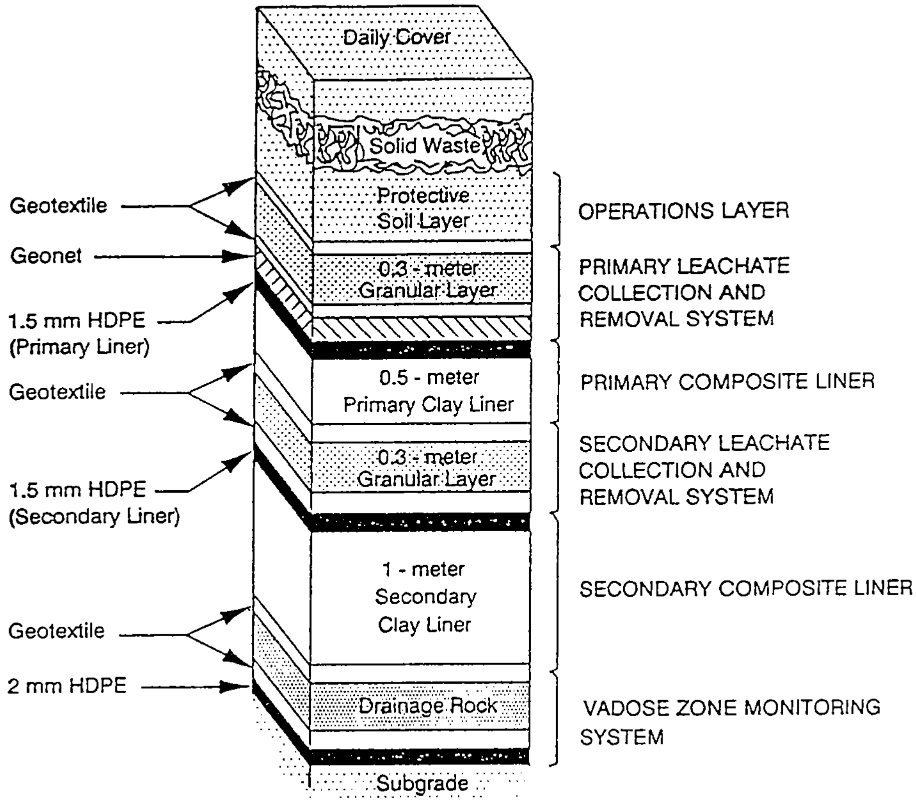


Fig. 2 Composite Double Liner System Used at the Kettleman Hills Waste Repository - An Example of a Liner for a Modern Hazardous Waste Storage Facility

2. The cover, also a complex system of several materials as shown schematically in Fig. 3, must restrict the infiltration of surface water, support vegetation or other ground cover suitable for post-closure landfill use, and be capable of withstanding large deformations to accommodate settlements within the waste below. In many cases the cover system will also be called upon to provide a barrier against uncontrolled escape of landfill gases and to support gas collection and venting systems.

3. The landfill must be safe against several possible types of stability failure, both during the construction and filling periods and after closure. Schematic diagrams of some potential failure modes are shown schematically in Fig. 4.

4. The landfill must be able to withstand earthquake shaking, without gross stability failure, rupture of the liner system, or failure of the leachate collection and removal system.

None of these issues can be addressed properly without correct application of soil mechanics. The principles, concepts, and analytical procedures that have been developed for stress-deformation behavior, shear strength, consolidation and settlement analysis, seepage, and dynamic response analysis of soils are essential.

Tailings dams are another major category of waste disposal and containment structure that require extensive and intensive application of soil mechanics. The schematic diagram of a typical tailings dam and impoundment structure, Fig. 5, illustrates some of the issues that must be addressed. The elements of soil mechanics that are needed to address them are just the same as for landfills:

1. The foundation soils must be capable of supporting the tailings embankment.

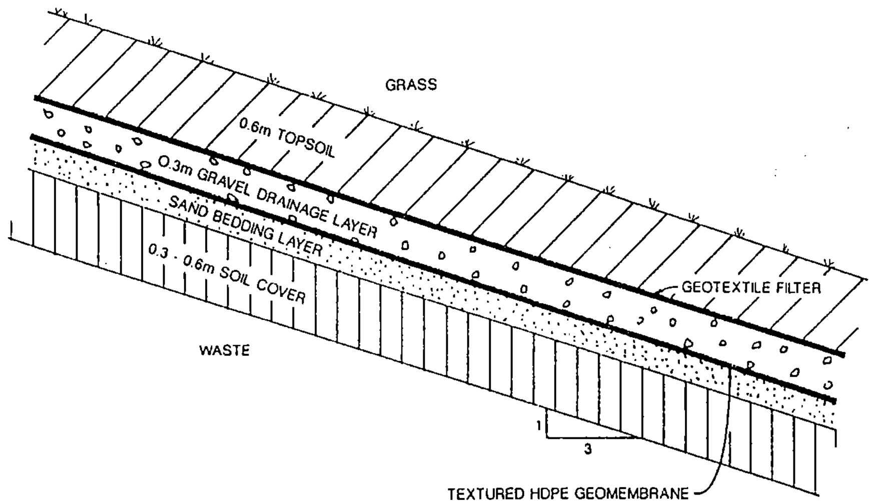


Fig. 3 Typical Solid Waste Landfill Cover Design

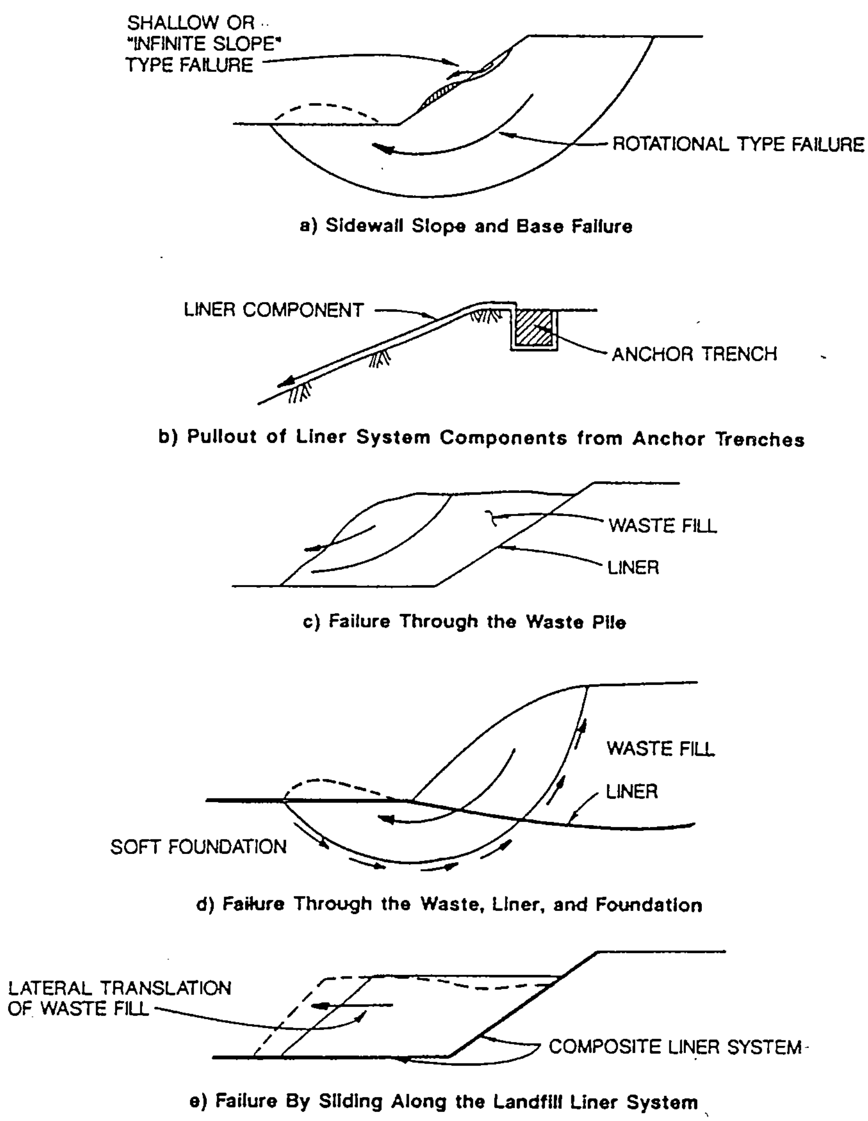


Fig. 4 Schematic Diagrams of Potential Failure Modes in Landfills

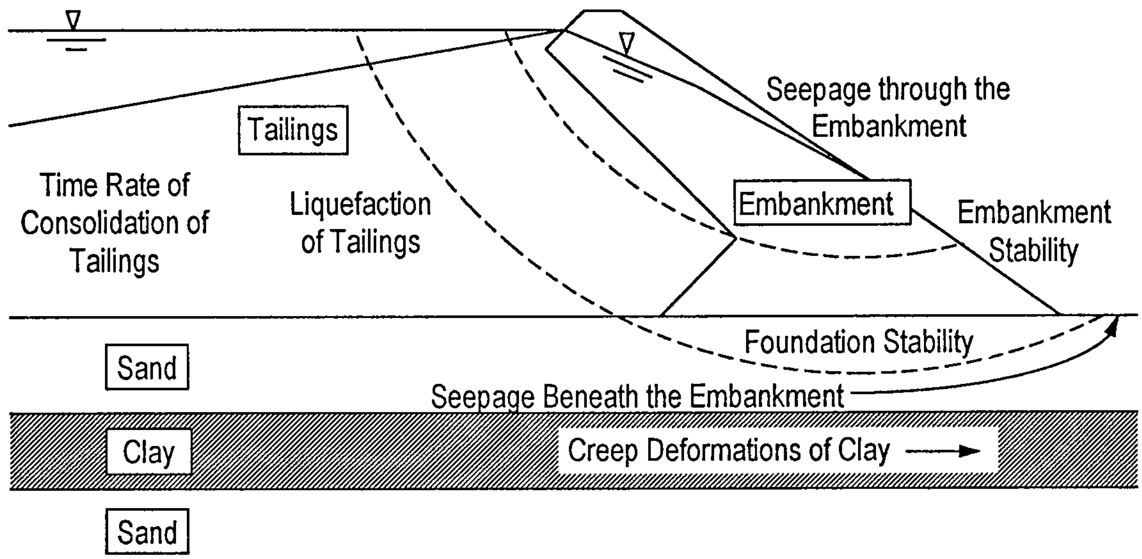


Fig. 5 Schematic Diagram of a Tailings Dam to Illustrate Soil Mechanics Issues

2. Patterns of seepage from the retained tailings and through and from the tailings dam must be determined in order to compute (1) the rates of escape of any environmentally unacceptable fluids and (2) the pore pressure distributions so that strengths may be reliably evaluated and stability analyses properly done, and so that consolidation rates and settlement estimates of the tailings can be made.

3. The dam must be stable under both static and seismic loading at different stages of construction and after completion to maximum height.

2.2 Isolation and Containment of Contaminated Ground

Isolation and containment of contaminated ground, for both temporary and permanent applications, have a very important role in environmental protection. In both cases the purpose is to effectively prevent contact between hazardous and toxic materials and uncontaminated groundwater, soil and air. A typical containment barrier system usually involves some type of cutoff wall, a bottom seal, and a cover, as shown schematically in Fig. 6. Comprehensive information about the types, characteristics, and construction procedures of the different in-ground waste systems that are currently used is given by Rumer and Ryan (1995). Among the issues that must be addressed that require consideration of soil mechanics are the following:

1. The stability of slurry trenches during excavation.

2. The state of stress within different types of trench backfills; e.g., soil-bentonite, cement-bentonite, and its influence on the hydraulic conductivity and stress-deformation properties of the backfill material.

3. Stresses and deformations of the ground adjacent to the slurry trench and their potential adverse effects on structures and facilities.

4. Seepage and contaminant transport through the barrier walls, floors, and covers. The susceptibility of barrier materials to attack by the contaminants they are intended to contain must also be considered.

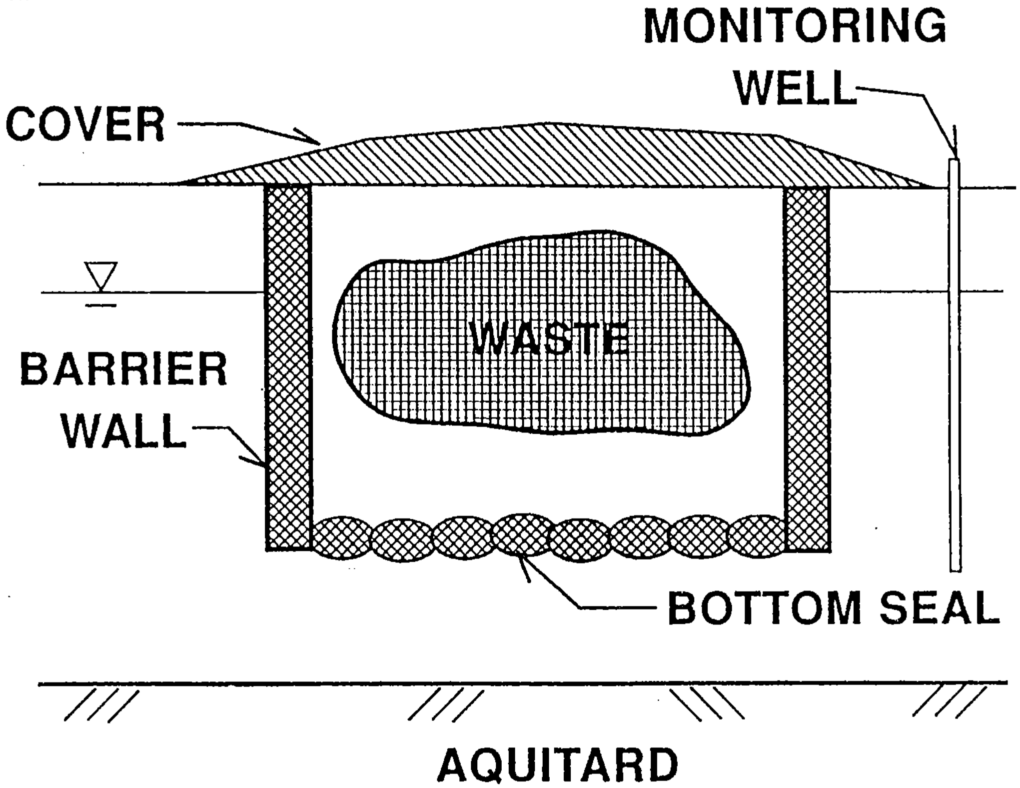


Fig. 6 Schematic Diagram of a Typical Waste Containment Barrier System

Thus, a specialist in environmental geotechnics must have a sound understanding of all aspects of soil mechanics, because the solution of problems of stability, stress-deformation analysis, settlement analysis, and seepage and groundwater flow requires it. We devote the remainder of this paper to illustrating this important aspect of geoenvironmental engineering by means of a series of examples. Before presenting the first example, however, a few words about materials and material properties are in order.

3. MATERIALS

One of the major challenges in geotechnical engineering is understanding the nearly limitless range of soil types that are encountered and quantifying their important hydraulic and mechanical properties. If this quantification is not done properly, no amount of soil mechanics analyses can provide useful results, and, in fact, analyses may generate misleading conclusions if the characterization of materials and their properties is improperly done. Accordingly, in our teaching and in our practice, as much time is devoted to identifying, classifying, and measuring soil properties as is used for the engineering analyses that guide our designs and decisions.

This phase of our work is even more important in environmental geotechnics, because we deal not only with naturally occurring soils and rocks, but also it is necessary to know about waste materials, the minutest details of groundwater chemistry, soil-waste interactions, the properties and behavior of non-aqueous phase liquids (NAPLs), the generation and effects of gases, and about the unique mechanical and durability properties of the vast range of geosynthetic and geosynthetic-clay composite materials that are now widely used in waste containment and environmental control systems. Further complicating the situation in most cases is that the properties of many of the materials may change with time as a result of the mechanical, chemical, and biological processes to which they are subjected.

A detailed review of the many important properties of waste materials of various types, of geosynthetics and clay-geosynthetic composites, and of the chemical and biological processes that may be important in any case is beyond the scope of this paper. Some useful recent references on these subjects include Bonaparte (1995), Mitchell et al. (1995), and Kavazanjian et al. (1995).

4. EXAMPLES OF STRENGTH AND STABILITY ANALYSES IN ENVIRONMENTAL GEOTECHNICS

Although conventional methods of stability analysis are often used, the analysis of the stability of geoenvironmental earth structures is often complicated by dissimilar strength and stress-deformation properties of the materials in barrier systems and by the presence of interfaces, which many times have low strength. The sliding failure of the Kettleman Hills hazardous waste landfill in 1988 provides an excellent example. Our second example, the failures of the Prestavel tailings dams illustrate how failure to incorporate proper analyses into the initial designs can lead to catastrophic consequences.

4.1 Stability Failure of the Kettleman Hills Landfill

With the sudden failure of the Kettleman Hills Hazardous Waste Landfill in Kettleman Hills, California in 1988, it was realized that attention must be directed at the strength of liner system components and interfaces, as well as at the leachate collection and containment capabilities of the liner for the successful design, construction, and filling of waste repositories.

The Kettleman Hills Landfill occupied the northern part of a lined basin having 2:1 and 3:1 (H:V) side slopes and a gently sloping base (for leachate drainage and collection purposes), as shown in Fig. 7. Filling operations had been in progress for about a year at the time of the failure. About 450,000 m^3 of waste had been placed, to a maximum height of about 27.5 m at the time of the failure. The failure involved a sliding displacement of the entire waste mass towards the southeast for a distance of about 11 m, with vertical settlements of somewhat over 4 m at the heel of the slide. The total movement developed over a period of only a few hours, and there were no triggering events such as an earthquake or heavy rains that could account for its initiation. The results of detailed failure investigations are given by Mitchell et al. (1990), Seed et al. (1990) and Byrne et al. (1992).

Preliminary investigations indicated that the sliding most probably occurred along one or more interfaces within the landfill liner system. A cross section of the base liner system was shown earlier as Fig. 2. Subsequent excavation of the waste pile confirmed the location of the surface of sliding along the base at the interface between the secondary clay and the secondary geomembrane of the base liner system, between the primary geomembrane and secondary geotextile in the upper portions of the northwest and southwest sideslopes, and on multiple surfaces in the regions near the intersections of the base and side slopes.

It was determined from the failure investigations and subsequent research involving model tests (Chang, 1991) and rigorous stability analyses representative of the actual conditions (Chang, 1991; Byrne et al., 1992) that the failure was triggered simply because the waste pile reached a height suf-

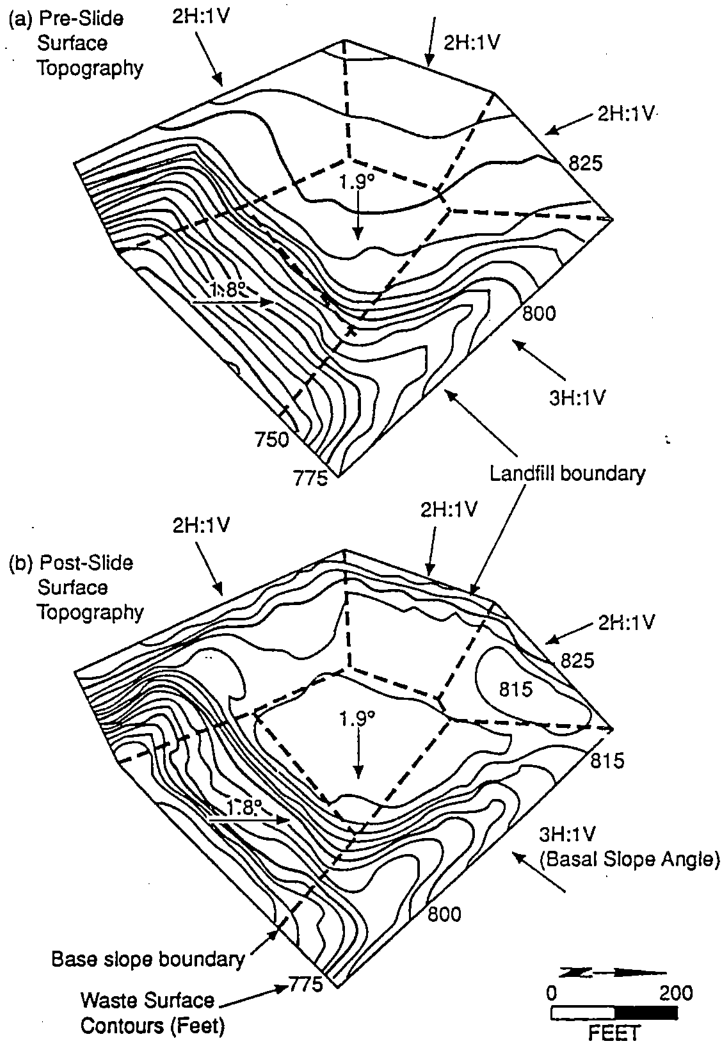


Fig. 7 Waste Surface Topography, Kettleman Hills Landfill (from Byrne et al., 1992)

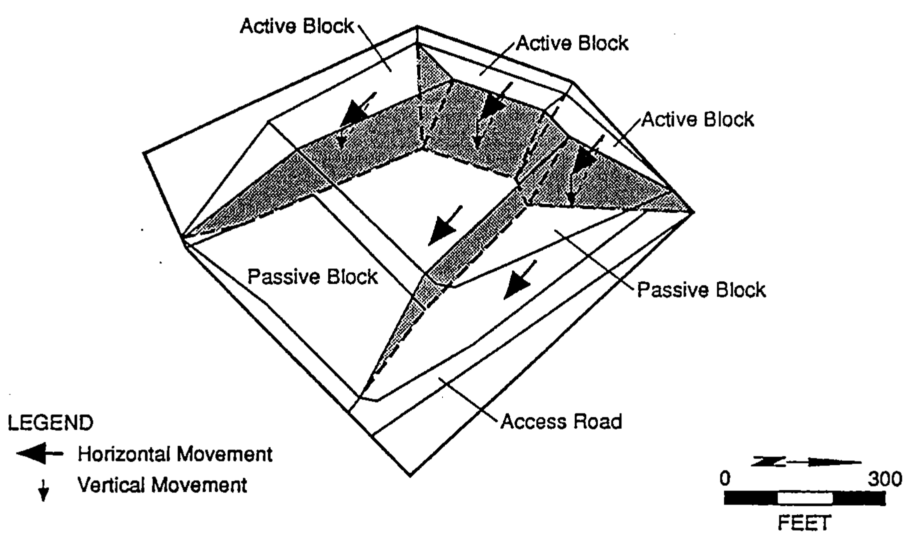


Fig. 8 Displacement Vectors for Waste Blocks During Failure of Kettleman Hills Landfill (from Byrne et al., 1992)

ficient to produce a driving force that exceeded the resistance that could be provided by the passive block, as shown schematically in Fig. 8. Although such a conclusion may appear obvious in retrospect, to confirm the mechanism and to demonstrate that the factor of safety had dropped below unity at the time the failure initiated and was equal to unity for the post-failure geometry required careful study of several things. These included (1) measurement of the stress-displacement and strength properties of different geosynthetic interfaces and compacted clay-geosynthetic interfaces (Mitchell et al., 1990; Byrne et al., 1992), (2) determination of the most probable in-situ stress and moisture conditions at the time of failure, (3) evaluation and selection of appropriate stability analysis procedures, and (4) accounting for three dimensional effects that impact the stability and the methods used to assess it. It is not possible to go into all aspects of these issues here; they can be found in the references already cited as well as in Chang et al. (1993). However, there are several conclusions that have emerged which have both important practical significance and point to the important role played by soil mechanics in the design and filling of waste landfills. They include:

1. Liner systems used for the containment and removal of landfill leachate may contain geosynthetics interfaces with very low strengths; e.g., friction angles of 8 degrees or less.
2. Both the compacted clay and the interface between a HDPE geomembrane and compacted clay may have a very low shearing resistance. This is because the best condition for placement of a compacted clay to achieve a low hydraulic conductivity is wet of optimum, and this corresponds to the region of low compacted clay strength. In addition, nearly saturated clay compacted wet of optimum will have a low coefficient of consolidation, which means that excess pore pressures that build up during waste placement above the liner will not dissipate rapidly, thus delaying the strength increase associated with the increase in normal effective stress.

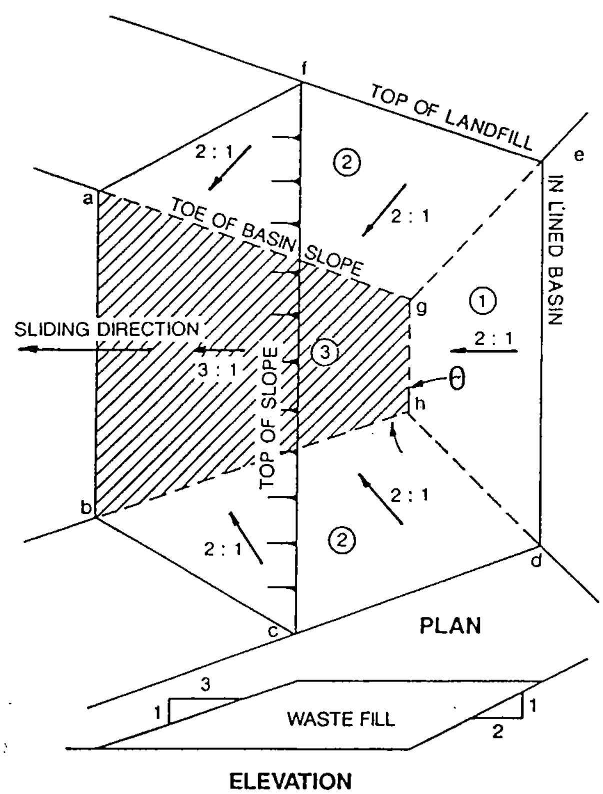


Fig. 9 Schematic Diagram of Lined Waste Repository with Divergent Side Slopes

3. The selection of the critical cross-section for a two-dimensional analysis of the stability of a landfill is not easily done in many cases. In fact, for situations where there are diverging side slopes such as shown schematically in Fig. 9, the stability in three dimensions can be more critical than what would be computed by a two-dimensional stability analysis. Fortunately, however, the difference in safety factors for force equilibrium analyses that use the same assumptions for two and three-dimensional analyses should be less than 15 to 20 percent (Chang et al., 1993). If the three-dimensional stability is critical, designs to assure stability during filling can be made by increasing the safety factors for simple two-dimensional analyses.

4. Landfill filling plans should be developed to maintain an adequate factor of safety for all fill heights and geometry's.

4.2 Stability Failure of the Prestavel Tailings Dams

On 19 July 1985 at 12:23 p.m., two tailings dams at the Prestavel Mine in the territory of Tesero, Trentino in northern Italy failed, and 190,000 m^3 of liquid mud flowed 4 km down the Porcellini and Stava river valleys in 5 to 6 minutes, "destroying everything" (Berti, et al., 1988). 268 people lost their lives in the worst tailings dam failure in European history.

This case history serves to illustrate, by omission, the importance of soil mechanics to an important area of geoenvironmental practice.

The Prestavel tailings dams, which are located in the valley of the Porcellini River, a tributary to the Stava River, are underlain by fluvioglacial deposits, and, at greater depth, by calcareous deposits with karst features that extend in some areas to the ground surface. Sinkholes exist beneath and adjacent to the tailings dams. The Porcellini River disappears before reaching the mine site and resurfaces as a spring a few hundred meters downstream from the tailings dams. Other springs also exist in the vicinity of the dams.

As shown in the plan view in Figure 10 and the cross-section in Figure 11, lower and upper tailings dams were constructed contiguously. The lower tailings dam was built during the period from 1961 to 1969. First, a 9 m high "starter dam" was constructed using on-site borrow material consisting of a mixture of silt, sand, and gravel. Sand from the mine tailings was deposited by cyclone to cover the starter dam and accomplish a dam raise of about 5 meters. From that point on, the "upstream method" of construction was generally used. The final height of the lower dam was 26 m.

The upper tailings dam was built and operated during the period from 1969 to 1985. Again, a starter dam was used. In this case, a combination of centerline, upstream, and mechanical placement methods was used to construct the impoundment. It can be seen in Figure 11 that a portion of the upstream embankment shell was placed on silt that had settled out from the pond in the lower impoundment. The final height of the upper dam was 29 m. The downstream slope of the upper dam was at an inclination of 1.5H:1V and included a 4 to 5 m wide bench.

During operation of the upper tailings dam, the effluent from the upper dam was routed to the pond behind the lower dam in order to achieve additional clarification of the tailings effluent. In this mode of operation, the flow was from line A in Figure 10 to the lower impoundment, and then out of the lower impoundment through line B.

The following events preceded the failure:

January 1985: A local slide occurred in the lower portion of the downstream slope of the upper dam. In plan view, the slide occurred near the location at which the decant lines passed beneath the upper dam (see Figure 10). Seepage from the area of this slide was evident during the period from January to March, 1985.

June 1985: The decant pipeline for the lower embankment (line B in Figure 10) broke, draining all the free water and liquid mud from the lower impoundment. Repairs included blocking the broken pipe and installing a new bypass line through the lower embankment (line C in Figure 10). During the shutdown period associated with these repairs, the upper embankment was raised by mechanical means using material from the upstream side of the upper embankment.

15 July 1985: Both the upper and lower impoundment were refilled with tailings fluid.

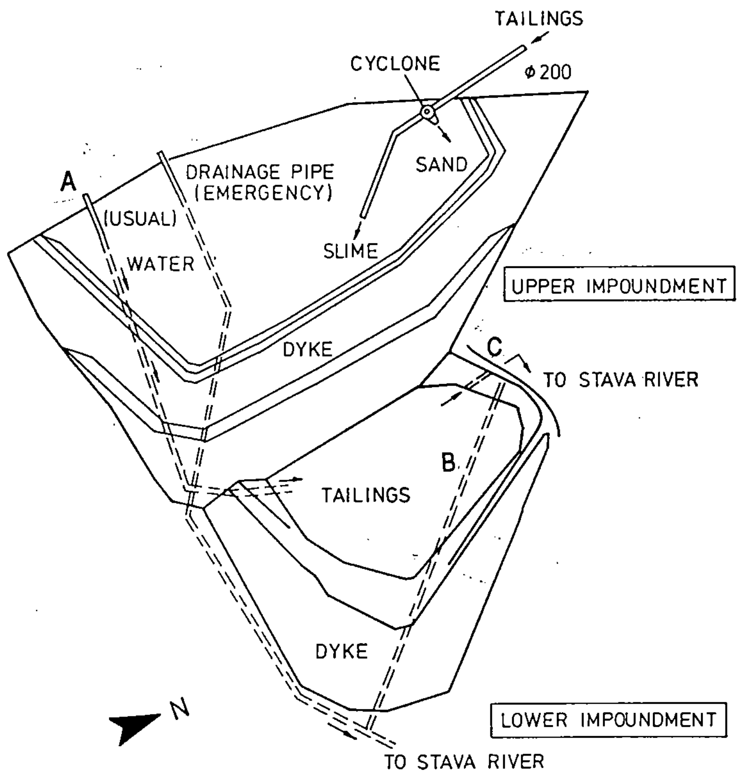


Fig. 10 Plan View of the Prestaval Tailings Dams (from Berti, et al., 1988)

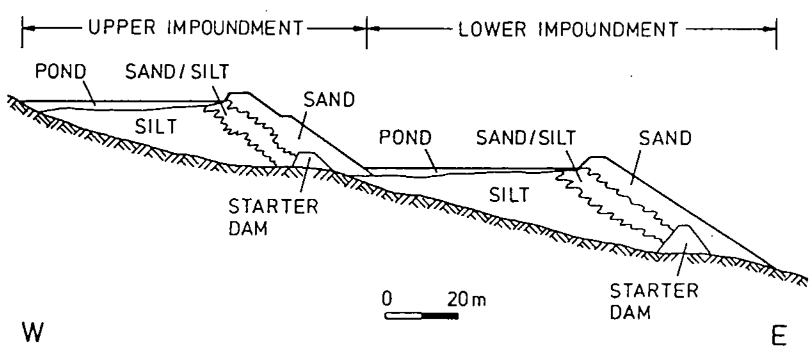


Fig. 11 Cross Section Through the Prestaval Tailings Dams (from Berti, et al., 1988)

The failure occurred four days after refilling. The sequence of the failure was that it "started with a slide of the upper dam pushing away nearly the entire body of the lower one."

Static slope stability analyses of the embankments were performed based on information obtained from an investigation conducted after the failure. Substantial uncertainty existed regarding the pre-failure geometry of the tailings dams, groundwater conditions, and material property values necessary for the analyses. These uncertainties resulted in ranges of calculated values of the factor of safety, FS, as follows: for the lower embankment, FS = 0.92 to 1.42, with a mean value of 1.21; for the upper embankment, FS = 0.71 to 1.30, with a mean value of 1.00.

The results of the analyses indicate that the embankments were only marginally stable. Consequently, even relatively small changes in embankment geometry or pore water pressure conditions resulting from the June 1995 repairs and the 15 July 1995 refilling could have triggered the catastrophic slide that occurred 4 days after refilling.

According to Berti, et al. (1988), shortcomings in the design, construction, and operation of the Prestavel tailings dams included the following:

- The dams were built without prior geological, hydrogeological, or geotechnical investigations.
- Systematic stability analyses were never made during the lifetime of the structures.
- No tests were made of any of the embankment or tailings materials.
- No instrumentation (e.g., piezometers) were installed in the embankments.
- Both impoundments were operated with high pond levels such that the upstream sandy beaches were submerged and the ponds were in contact with the embankment crests.

These tailings dam failures can be understood using principles of soil mechanics. One of the lessons learned is that a tailings dam should be designed, analyzed, constructed, and operated as an engineered structure.

5. EXAMPLES OF STRESS-DEFORMATION ANALYSES IN ENVIRONMENTAL GEOTECHNICS

Two examples are illustrative of how soil mechanics can be used: ground movements adjacent to a slurry trench seepage cutoff wall and predictions of landfill settlements

5.1 Ground Movements Adjacent to a Slurry Trench Cutoff Wall

A soil-bentonite cutoff wall was constructed at a contaminated site in the Silicon Valley area of California to prevent on-site chemicals from moving off-site in a down-gradient direction and to prevent up-gradient, off-site chemicals from moving onto the site. A building located near the trench experienced so much settlement during and after construction of the cutoff wall that manufacturing operations in the building were disrupted, and a lawsuit resulted.

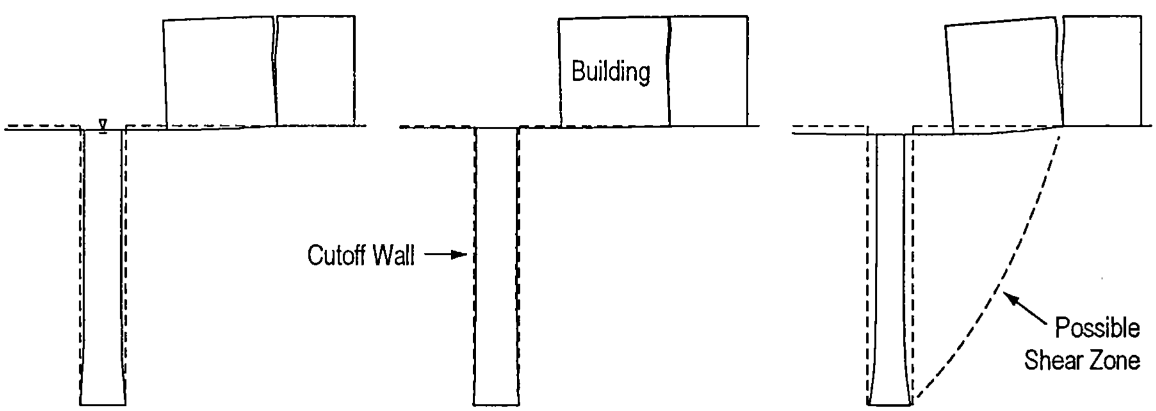
The project site is underlain by multiple sand and gravel lenses interbedded with silts and clays. The complex stratigraphy and groundwater conditions at the site are described in more detail in section 6.2, where this case history is also used to illustrate the importance of groundwater seepage in geoenvironmental projects. For our present purposes, it is sufficient to note that the 30 m deep cutoff wall was intended to isolate and contain the two uppermost aquifers at the site: the "A" aquifer, with an upper boundary at depth 4.5 to 6 m and a lower boundary at depth of 7.5 to 12.2 m; and the

"B1" aquifer, with an upper boundary at depth of 13.8 to 16.8 m and a lower boundary at depth 23 to 24.5 m. The piezometric level in the A and B1 aquifers is about 4.5 to 6 m below the ground surface.

The cutoff wall, which is 0.9 m wide, 30 m deep, and over 1000 m long, was excavated in 2 stages: first, excavation to depth 15 m was accomplished with a backhoe, and then excavation from 15 m to 30 m was by clam shell. Trench stability was maintained with a bentonite-water slurry. The trench excavation spoils were mixed with bentonite-water slurry and dry bentonite to produce a backfill mix with 2 percent bentonite and a 125 mm slump. At the beginning of backfill placement, the backfill was end-dumped on a starter slope until the backfill "daylighted" above the bentonite-water slurry. From that time on, the backfill was always placed on daylighted slurry so that it would slough down the approximately 10H:1V backfill slope in the trench.

Movements of the ground adjacent to the cutoff wall occurred during trench excavation, backfill placement, and backfill consolidation. Lateral movements were documented by inclinometers installed near the cutoff wall. One typical inclinometer moved about 15 mm towards the cutoff wall during trench excavation. During the six month period following backfill placement, the inclinometer moved an additional 50 mm toward the cutoff wall. The displaced shape shown by the inclinometer, suggests that a shear zone with about 50 mm of relative displacement may have developed at a depth of 25 m.

Settlement of the adjacent ground accompanied the lateral movement. Settlements of several cm were measured at monuments installed near the cutoff wall. At one location, the wall passes within about 6 m of an existing building. A rather sensitive manufacturing operation was underway in the building at the time the cutoff wall was being constructed, and the resulting settlement rendered the building unusable for that purpose. A schematic diagram of the deformations that occurred, to exaggerated scale, is shown in Fig. 12.



a) Trench excavation under bentonite-water slurry.

b) Possible rebound during soil-bentonite backfill placement.

c) Consolidation of the soil-bentonite backfill after placement.

Fig. 12 Schematic Diagram of Ground Movements Adjacent to a Slurry Wall

It is thought that the adjacent ground movements occurring during the period after construction were due to consolidation of the soil-bentonite backfill. Support for this hypothesis is provided by a series of cone penetrometer test (CPT) probes that were made in the soil-bentonite backfill. Pore water pressure measurements were taken during probing and during pauses in probing to measure the rate of excess pore water pressure dissipation. Static pore water pressures measured after dissipation of the excess pressures that result from probing can reveal the consolidation state of the backfill. Probes made two to three months after construction of the wall, disclosed static pore water pressure heads above the ground surface. Eighteen months later the static pore water pressure heads were about equal to adjacent groundwater levels. This reduction in pore water pressure was accompanied by consolidation of the soil-bentonite backfill and movements of the adjacent ground.

It should also be noted that groundwater extraction from within an area enclosed by a soil-bentonite cutoff wall can increase effective stresses in both the backfill and the contained area, thus causing additional increments of backfill consolidation and adjacent ground movement. Groundwater extraction that occurred as part of the remedial measures at this site probably contributed to the observed adjacent ground movements.

The observed movements substantially exceeded estimates made during design. Although established and verified methods to estimate such movements were not available at the time (and still are not available), it is the authors' opinion that ground movements adjacent to soil-bentonite cutoff walls can be understood by applying the principles of soil mechanics. The authors are currently involved in a research program to develop and verify appropriate methods for making such estimates.

In addition to predicting adjacent ground movements, the mechanics of soil-bentonite consolidation in cutoff walls can have important influences on the hydraulic conductivity of the backfill, resistance of the backfill to chemical attack, and, in the case of cutoff walls installed through and/or beneath dams, resistance to hydraulic fracturing.

5.2 Landfill Settlement Prediction

An existing 10 ha municipal waste landfill in Massachusetts is to be expanded vertically to accommodate the disposal of up to 27.5 m of wastewater treatment plant sludge. The waste in the existing landfill is up to 25 m deep, and the settlements to be expected in this material as a result of the vertical expansion must be estimated in order design the new liner, and the leachate and gas collection systems required for the expanded facility. This important case of landfill settlement analysis is described in some detail by Stulgis et al. (1995).

The waste in the old landfill was placed prior to the early 1980's and consists of wood, paper, leaves, plastic, cloth, metal, newspapers, and Styrofoam, and it is interbedded with granular fill which was used for the daily cover. Contours showing the thickness of this existing fill are presented in Fig. 13. Laboratory tests were done to establish the biodegradation state of the waste as well as relevant geotechnical properties, and the results are given by Stulgis, et al. (1995). In addition, a test fill was constructed at the location shown in Fig. 13 to obtain information about waste compression under large-scale loading. The settlements measured in the field test are summarized in Fig. 14. It may be seen that a significant proportion of the total settlement was contributed by compression of a "Chemfix"- stabilized sludge cap, up to about 5 m thick that was placed over a portion of the landfill in 1985.

Stulgis, et al. (1995) note that a "soil mechanics" approach for settlement analysis of municipal waste fills is generally used, and they described the Bjarngard and Edgers (1990) settlement model, which is shown in Fig. 15. In this model CR is the compression ratio, $C_{\alpha(1)}$ is the intermediate secondary compression index, $C_{\alpha(2)}$ is the long term secondary compression index, P_o is the initial average vertical effective stress, and Δp is the average induced vertical effective stress increment. They also used the Fassett, et al. (1995) model which is similar, except that the intermediate and long term secondary compression indices are replaced by a single one, C_α. The field test data, Fig. 14 agree well with both models, because the intermediate and long term secondary compression index values are about the same. A detailed discussion of the compression mechanisms is beyond the scope of this paper; however, the secondary compression phase of landfill settlement will usually include a component due to decomposition of biodegradable organic matter in addition to that due to creep.

The settlement models were used to predict the total settlement contours, for a time of 50 years, shown in Fig. 16. These contours and the differential settlements that are associated with them provide information necessary for design of liner, cover, and leachate and gas collection systems that will remain functional throughout the life of the project.

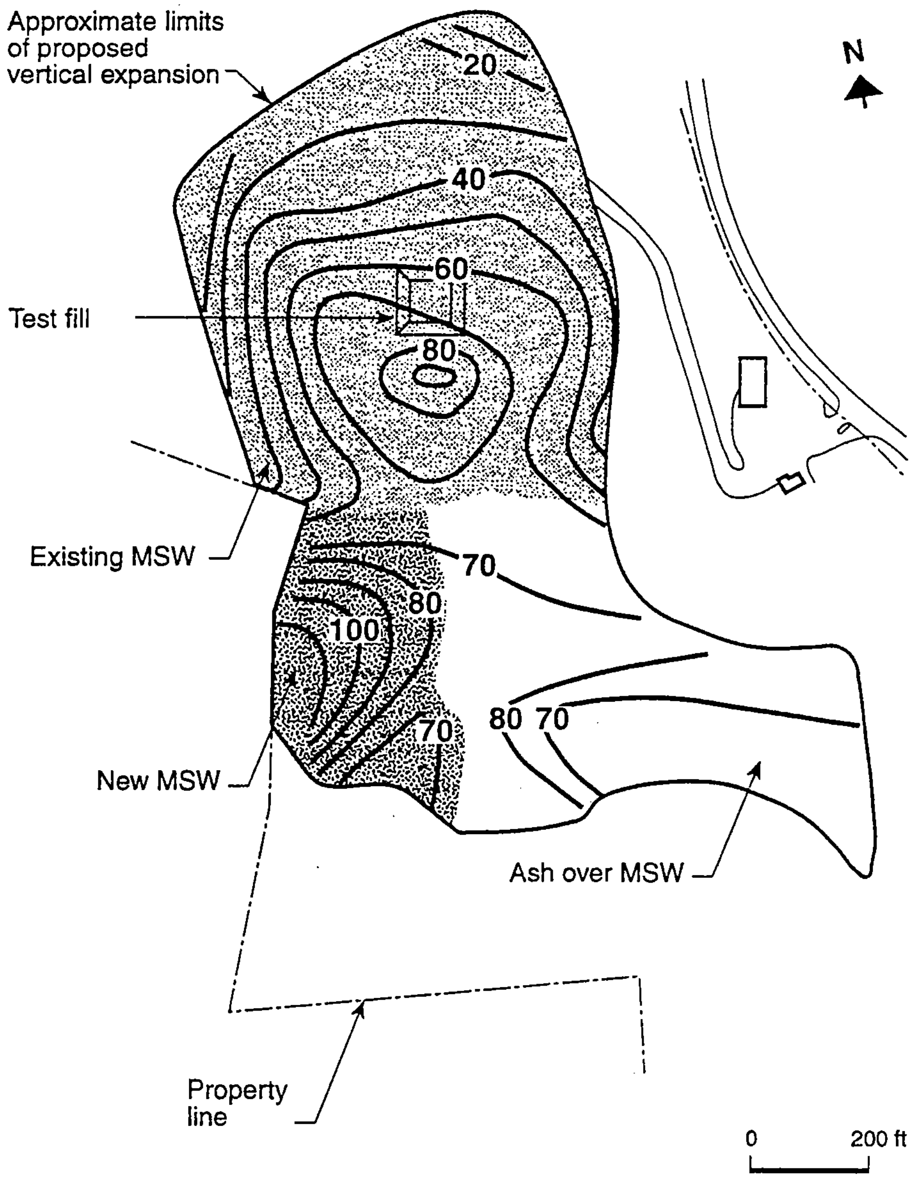


Fig. 13 Contours of Waste Thickness (in feet) at an Existing Landfill (from Stulgis, et al., 1995)

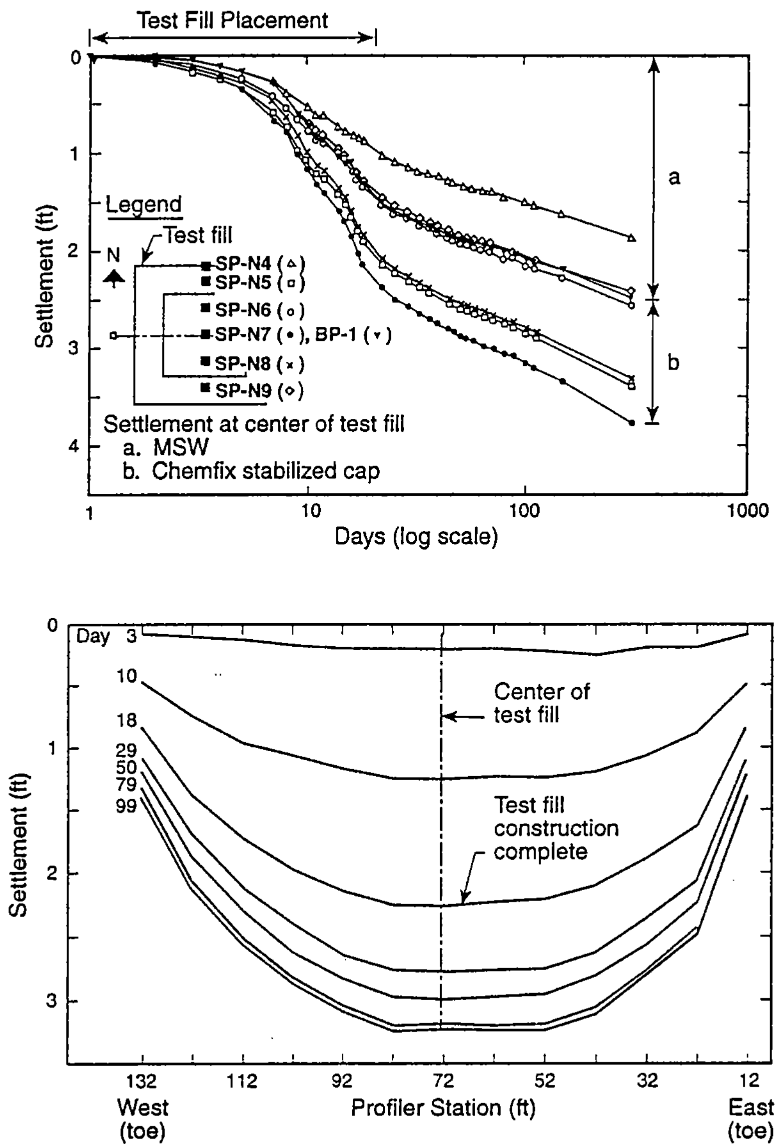


Fig. 14 Test Fill Settlements (from Stulgis, et al., 1995)

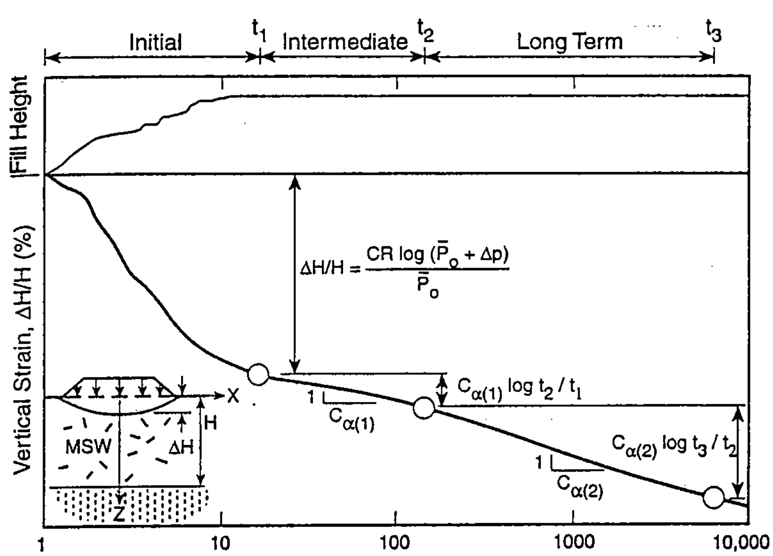


Fig. 15 Municipal Waste Settlement Model Proposed by Bjarngard and Edgers (1990) (from Stulgis,et al., 1995)

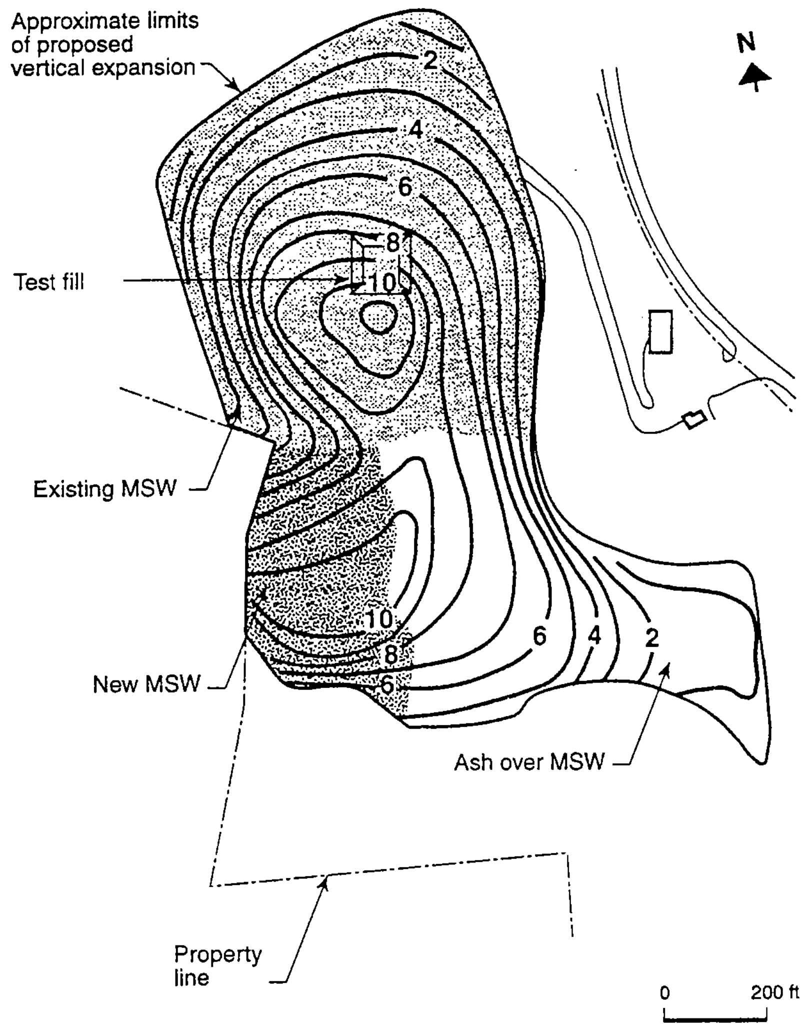


Fig. 16 Predicted Settlement Contours (in feet) for a Waste Landfill (from Stulgis, et al., 1995)

6. EXAMPLES OF SEEPAGE AND CONTAMINANT TRANSPORT PROBLEMS IN ENVIRONMENTAL GEOTECHNICS

Determinations of groundwater levels, water pressure distributions, and seepage and drainage quantities are routinely made as parts of most geotechnical studies. They are central to geoenvironmental projects as well. However, the focus and acceptable limits of variability in predictions of flow quantities are quite different in the two categories of studies. For purposes such as dewatering, estimation of seepage quantities through a dam, or a drainage design, it may be adequate to make flow quantity estimates within perhaps an order of magnitude. On the other hand, the most frequent need for seepage analyses in geoenvironmental engineering is for prediction and evaluation of contaminant transport in groundwater flow or through a liner or other type of waste containment barrier. In such problems it is sometimes necessary to deal with contaminant concentrations in parts per million or even parts per billion. Furthermore, flow rates through very low permeability systems such as compacted clay liners and soil-bentonite slurry walls are often of interest. Nonetheless, for both types of problems, the classical methods for seepage analysis are equally valid.

6.1 Seepage and Diffusion Through Waste Containment Barriers

When chemical gradients exist across very low permeability materials, chemical transport by diffusion is likely as well as by solution flow (advection). Theoretical analyses of combined diffusive and advective flows indicate that in soils with a hydraulic conductivity less than about 1 x 10^{-9} m/s, transport by diffusion may exceed that by advection for hydraulic gradients characteristic of those in most waste containment situations. A field case to illustrate this has been reported by Quigley and his colleagues at the University of Western Ontario, and it is summarized here.

Detailed study of chemical profiles within a massive gray clay beneath the Confederation Road municipal waste landfill in Sarnia, Ontario, Canada was reported by Quigley and Rowe (1986), Quigley et al. (1987), and Yanful et al. (1988a, b). The soil conditions are shown in Fig. 17. About 7.5 m of waste, including cover, are placed in a 5.5 m deep trench. A small regional gradient produces a downward pore fluid velocity that averages about 2-4 mm/yr. The soil is overconsolidated by about 90 kPa, with a small increase near the clay/waste interface. The clay is free of fissures throughout its full depth beneath the landfill.

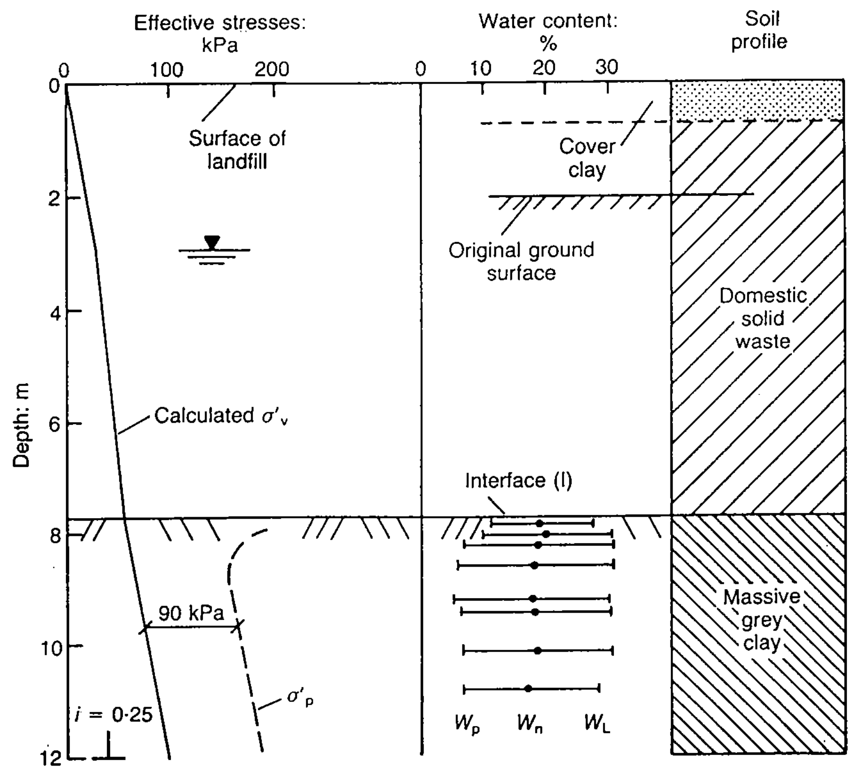


Fig. 17 Present Soil Conditions at Confederation Road Landfill Site, Sarnia, Ontario, Canada (from Quigley and Rowe, 1986): σ_v' is vertical effective stress, σ_p' is preconsolidation pressure, w_p is plastic limit, w_n is water content, w_L is liquid limit

Profiles of Na^+ and Cl^- after twelve years are shown in Fig. 18. At that time salt had migrated about 1.5 - 2.0 m below the waste/clay interface, whereas the advection distance was only 30 mm. The hydraulic conductivity of the clay is within the range of about 0.8 x 10^{-10} to 1.6 x 10^{-10} m/s. Diffusive flows would be expected to be much greater than advective flows for the conditions at the Confederation Road site, and these results confirm that expectation.

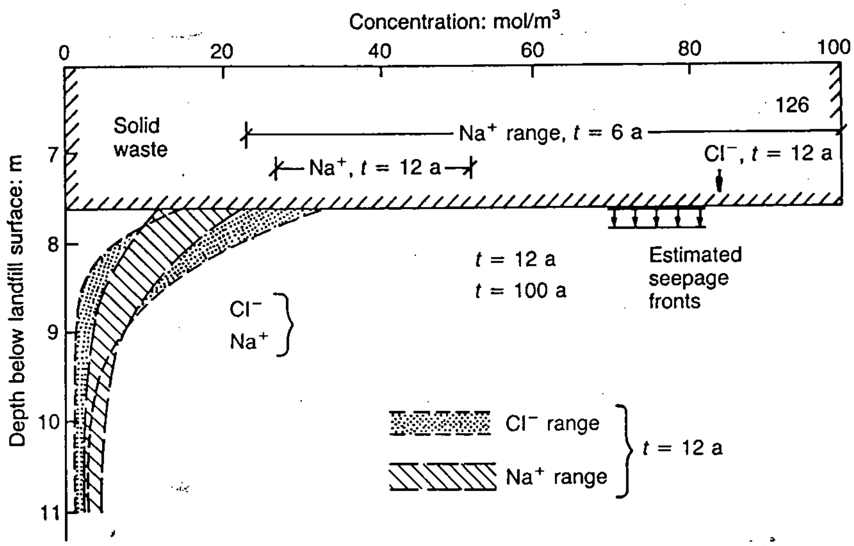


Fig. 18 Concentrations of Sodium and Chloride in Clay Beneath the Waste at the Confederation Road Landfill after Twelve Years (from Quigley and Rowe, 1986)

6.2 Groundwater Seepage Issues at the Silicon Valley Cutoff Wall Project

Groundwater seepage issues were an important consideration for the soil-bentonite cutoff wall project in the Silicon Valley area of California, that was described previously to illustrate stress-strain-deformation aspects of geoenvironmental projects. The complex stratigraphy at this site includes multiple sand and gravel lenses interbedded with silts and clays. A series of alternating aquifers and aquitards were identified, as indicated in Figure 19.

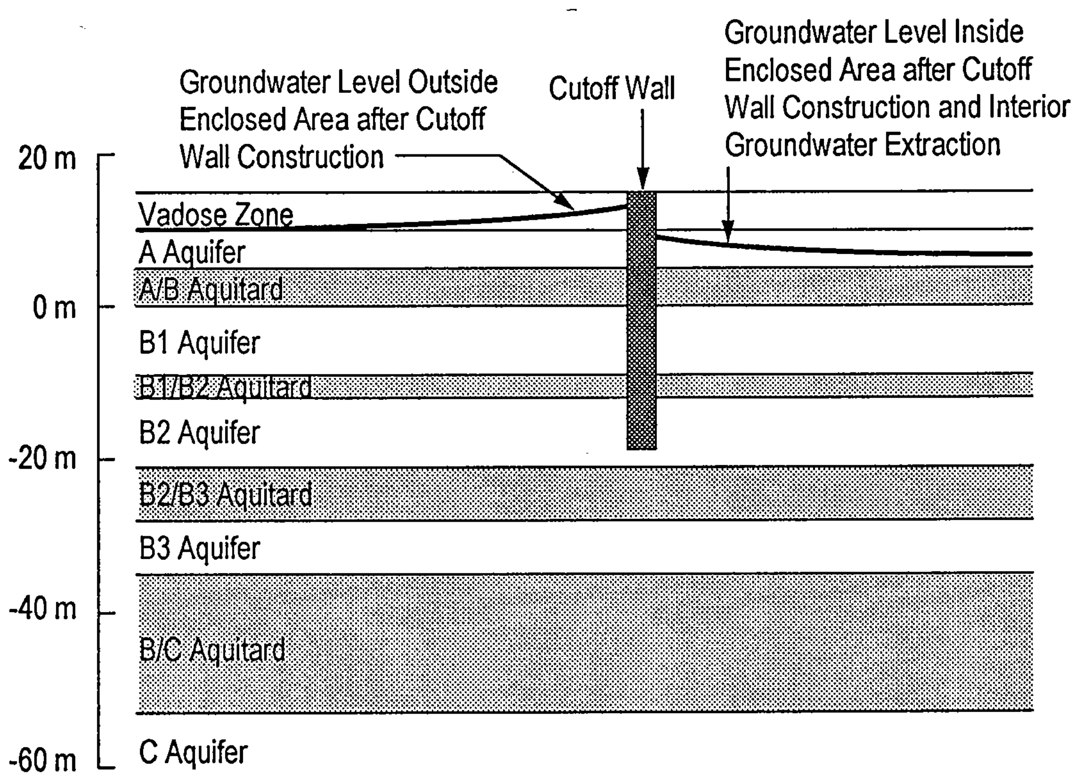


Fig. 19 Stratigraphy and Piezometric Levels at the Silicon Valley Cutoff Wall

Most of the aquifers at the site are thought to be interconnected to some degree by "leaky" aquitards that contain interbedded permeable layers. A possible exception is the B/C aquitard, which has a significantly lower hydraulic conductivity than the overlying aquitards and which may effectively isolate the B3 and C aquifers. Upward directed hydraulic gradients exist across the B aquifers. The existence of the aquitards and the upwards hydraulic gradients reduced the amount of downwards migration of contaminants that would have otherwise occurred at this site.

Potentiometric surface maps indicate that, within each aquifer, the lateral hydraulic gradient is directed to the north. In the A and B aquifers, the lateral hydraulic gradient is about 0.004, while in the C aquifer, it is only about 0.0006. It is thought that the lateral hydraulic gradient in the A and B aquifers served to carry on-site contaminants off-site in a down gradient direction, and also to possibly carry up-gradient contaminants onto the site.

One of the objectives of the soil-bentonite cutoff wall was to laterally isolate the A and B1 aquitards. Groundwater was extracted from within the area enclosed by the cutoff wall in order to create an inward hydraulic gradient and to maintain an upwards directed hydraulic gradient.

An interesting effect developed along the southern side of the cutoff wall due to the combination of the natural hydrologic regime and the artificial ground water extraction that was accomplished at the site. Measurements of groundwater levels in the A aquifer just inside and just outside the cutoff wall indicate that an inward hydraulic gradient developed across the wall, as desired. The data also suggest that the northward flowing groundwater outside the enclosed area may have risen, or "mounded," when it encountered the obstruction of the south cutoff wall, thus influencing the seepage rates across the wall and the effective stresses in the trench backfill.

This case history illustrates the importance of groundwater seepage and the impact that complexities in stratigraphy and geohydrologic conditions can have on contaminant transport. Remedial measures should be designed with these conditions in mind, and the consequential impacts of remedial measures on adjacent groundwater conditions should be considered. Principles of soil mechanics can be used to make such evaluations.

7. EXAMPLES OF SOIL MECHANICS ANALYSES FOR GEOENVIRONMENTAL PROJECTS IN SEISMIC AREAS

Applications of soil dynamics and geotechnical earthquake engineering have a major role in geoenvironmental engineering. In the U.S.A., municipal waste landfills in seismic areas must be designed to withstand specified levels of ground shaking. As many mining operations are located in active seismic regions, the design and construction of tailings dams and tailings disposal ponds are influenced, or even controlled, by seismic design considerations. Among the issues of primary concern are:

- Site seismic response analysis
- Stability and movements of landfills during earthquakes
- Liquefaction susceptibility of foundations and waste containment structures (e.g., tailings dams)
- Mitigation of liquefaction risk to foundations and waste containment structures.

Case histories to illustrate the applications of soil mechanics and soil dynamics to these types of problems are presented in this section and include projects involving the analysis of the performance of a municipal waste landfill in a seismic area and tailings dam liquefaction.

7.1 Analysis and Performance of Landfills in Seismic Areas

U.S. Code of Federal Regulation, Protection of the Environment, *Title 40, Part 258,* "Criteria for Municipal Solid Waste Landfills," became effective in October 1993. Provisions of these regulations require that all municipal waste landfills within a "seismic impact zone" be designed for earthquake generated ground accelerations that have only a 10 percent chance of being exceeded in 250 years. A seismic impact zone is a region within which the maximum horizontal ground acceleration for the stated probability level is greater than 0.1g. This means that landfills in most parts of the country will require seismic design. A series of recent papers published in "Geoenvironment 2000," *Geotechnical Special Publication No. 46*, ASCE, 1995, provides comprehensive information about the current state of seismic analyses for evaluation and design of waste landfills (Augello et al., 1995; Kavazanjian and Matasovic, 1995; Singh and Sun, 1995; Klimkiewicz, 1995; Yegian et al., 1995; Kavazanjian et al., 1995).

Studies related to the closed "superfund" OII Landfill in Monterey Park, California, an above ground waste pile that contains some 27.5 x 10^6 m^3 of municipal, industrial, and liquid waste and soil cover layers to a maximum depth of about 100 m, illustrate several of the analysis issues that must be addressed. As a part of these studies solid waste properties were back-calculated from the measured and observed seismic responses during the 17 January 1994 Northridge (M 6.7) and 28 June 1992 Landers (M 7.4) earthquakes (Kavazanjian and Matasovic, 1995). The results obtained in this way are especially important, because detailed knowledge of the static and dynamic properties of waste has been limited, yet specific values and their variations with time and position in a landfill may be expected to have important influences on behavior.

Kavazanjian and Matasovic (1995) analyzed a 1.2 m thick layer of compacted soil cover placed over a 75 m thick column of waste founded on bedrock as typical of the OII landfill. Unit weight and low strain shear wave velocity profiles from Kavazanjian et al (1995) and shown in Fig. 20 were used along with the shear modulus and damping degradation curves also shown in Fig. 20. The MKZ model referred to in the figure is a modified hyperbolic constitutive model that has been shown by Matasovic and Vucetic (1993) to represent well the behavior of liquefiable sand under undrained cyclic loading.

Both equivalent linear (SHAKE) and nonlinear (DMOD) dynamic response analyses were made, and comparisons between the computed and measured response spectra for the Northridge and Landers earthquakes are shown in Fig. 21. From comparisons of these spectra and the results of additional analyses, Kavazanjian and Matasovic (1995) were able to conclude several things about the seismic response of a large above ground landfill, including:

1. The assumption of constant unit weights and initial shear wave velocities with depth can yield significant errors in the response analysis. These errors may be greatest at the top of the landfill.

2. Rock outcrop peak accelerations may be substantially amplified as they pass upward through the landfill. For example, the relationship between peak ground surface acceleration and peak outcrop acceleration for landfills is compared with that showing amplification for soft ground sites (Idriss, 1990) in Fig. 22. Such amplifications should be considered in the design of cover systems and gas collection and removal systems.

3. Equivalent linear and non-linear response analyses agree reasonably well for low magnitude; e.g., 0.1 g, rock motions. However, for strong base motions the non-linearity of soil and waste stress-strain behavior become sufficiently great that the actual and equivalent linear response spectra differ over significant ranges of period.

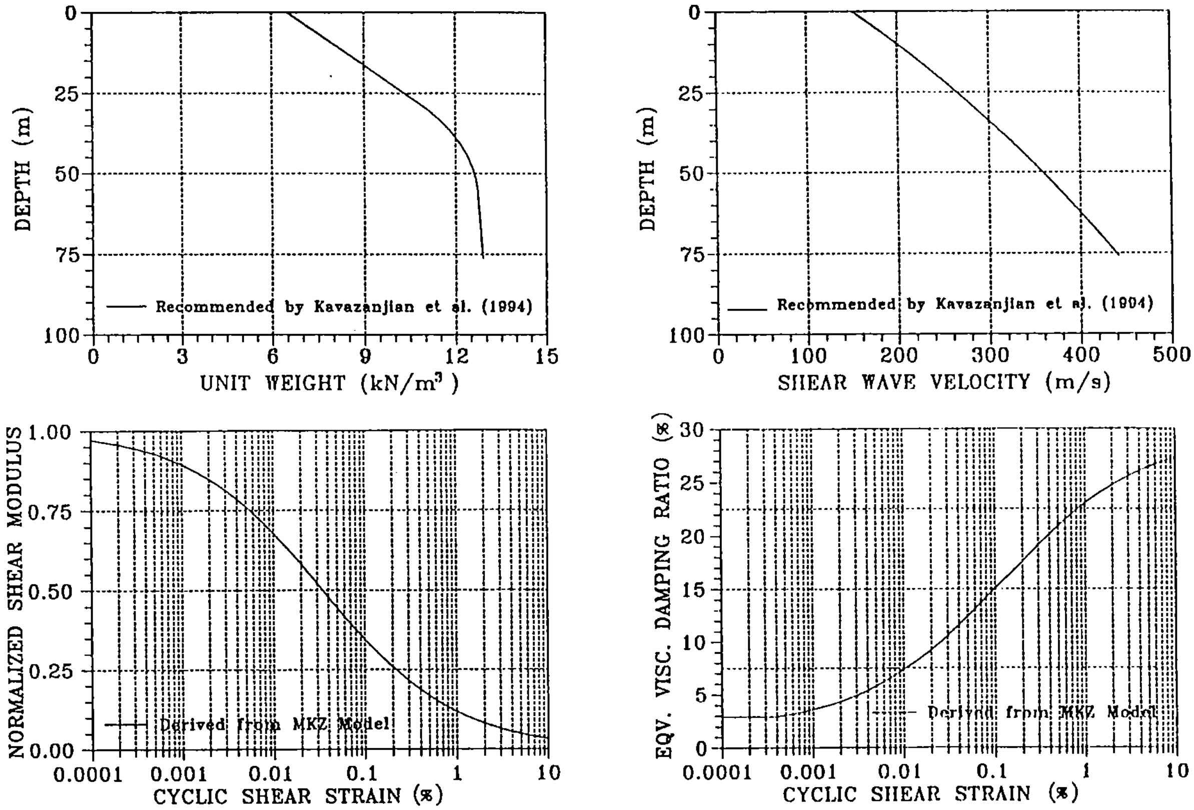


Fig. 20 Material Properties of Solid Waste Used in the Dynamic Response Analysis of the OII Landfill (from Kavazanjian and Matasovic, 1995)

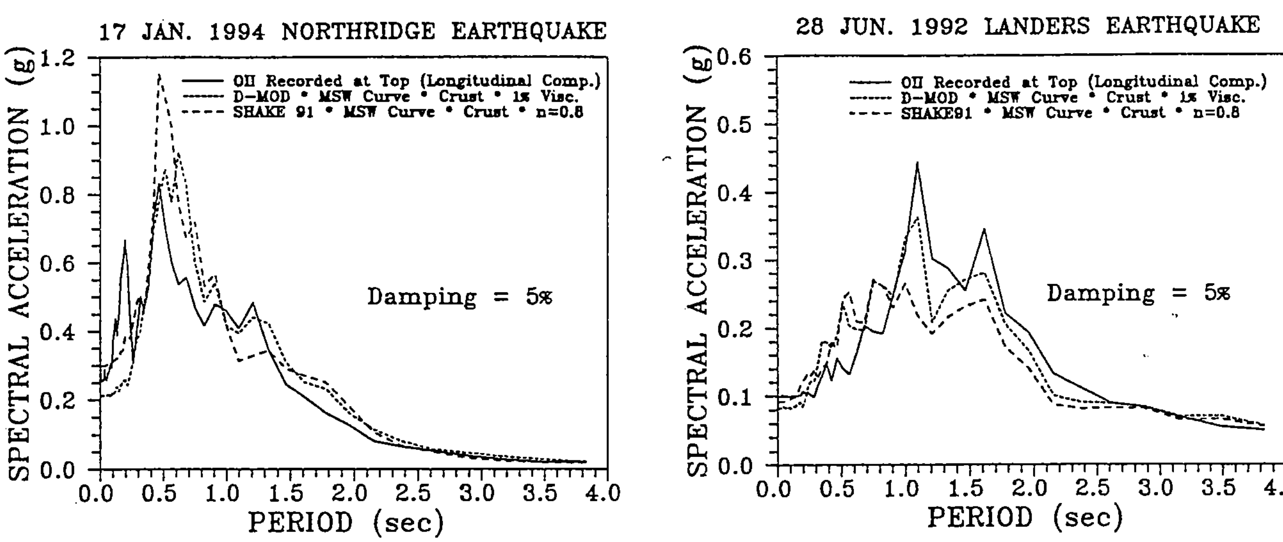


Fig. 21 Recorded and Measured Response of the OII Landfill in the Longitudinal Direction (from Kavazanjian and Matasovic, 1995)

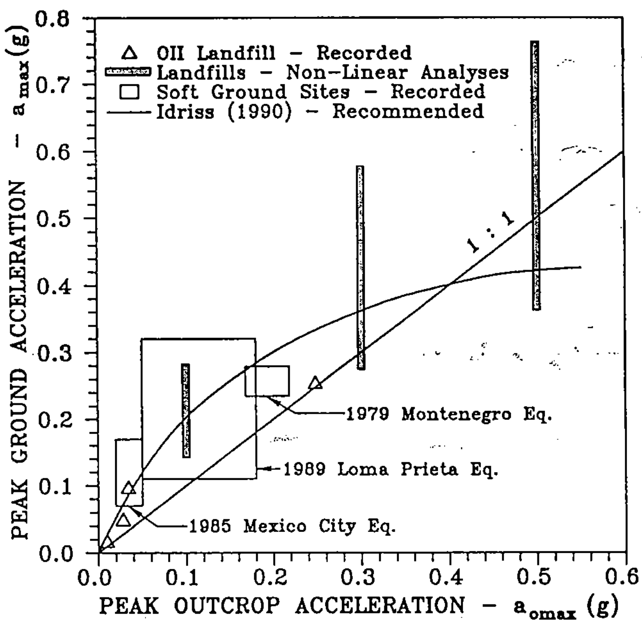


Fig. 22 Amplification of Peak Horizontal Accelerations for Soft Ground and Waste Landfills (from Kavazanjian and Matasovic, 1995)

7.2 Seismic Liquefaction of El Cobre Tailings Dams

On 28 March 1965, an earthquake shook the provinces of Aconcagua, Valparaiso, and Santiago in central Chile. Most of the tailings dams in this region failed during or soon after the earthquake (Dobry and Alvarez, 1967). The failure of two dams at El Cobre was especially devastating. Approximately 2,400,000 tons of tailings flowed out of the failed dams and traveled about 12 km in a matter of minutes, destroying part of the town of El Cobre and killing more than 200 people.

In 1965, seismic design of tailings dams was not a developed part of engineering practice. Since that time, important advances in the state-of-the-art permit design and construction of safe tailings dams in seismic regions. Some of the recent advances can be found in ASCE (1988) and ICOLD (1990) proceedings. The following description of the failure of the El Cobre tailings dams is offered as a example of what could easily happen again today if such structures were designed and built without due consideration of the principles of modern soil mechanics, including seismic response analysis and studies of liquefaction susceptibility.

Tailings had been deposited at El Cobre since the 1930s. The gravitational method, in which the tailings are deposited at the downstream crest of the dam without use of a cyclone and allowed to flow upstream, was used. The coarsest tailings settle out first to form the shell of the dam. At El Cobre, three tailings dams were constructed. Two of the dams, the Old Dam and the Small Dam, were filled during the period from 1930 to 1963, with a rapid acceleration of the rate of filling at the end of the period. The third dam was referred to as the New Dam, and it was constructed during the period between December 1963 and the time of the failure.

The epicenter of the 28 March 1965 earthquake was about 40 km north of El Cobre. At El Cobre, the Modified Mercalli intensity was in the range of VIII-IX. A maximum horizontal acceleration of 0.18g was measured at Santiago, about 150 km from the earthquake's epicenter.

The earthquake caused two of the three El Cobre tailings dams, the Old Dam and the New Dam, to fail. The fill at the New Dam had been placed within 1½ years before the earthquake and was just beginning to consolidate. The slimes at the New Dam liquefied to such an extent that eyewitnesses observed waves forming on the surface. A gap opened in the dam shell and the liquefied tailings flowed out. There are no eyewitness accounts of the failure of the Old Dam because dust from the desiccated crust of that impoundment temporarily reduced visibility.

To gain an understanding of the mechanisms responsible for collapse of the Old Dam, a post-failure investigation was performed. This involved surveying the site and making soil borings. The log of a boring from the central part of the impoundment of the Small Dam, which did not fail, is shown in Figure 23. It can be seen that a desiccated crust about 4 m thick overlies an "underconsolidated" zone about 7 m thick. The underconsolidated zone is underlain by about 2½ m of older, normally consolidated tailings. The histories of the filling operations at the Old Dam and the Small Dam are substantially the same; the principal difference is that the Old Dam is higher and larger. It was thought that the underconsolidated layer in the Old Dam was about 10 m thick.

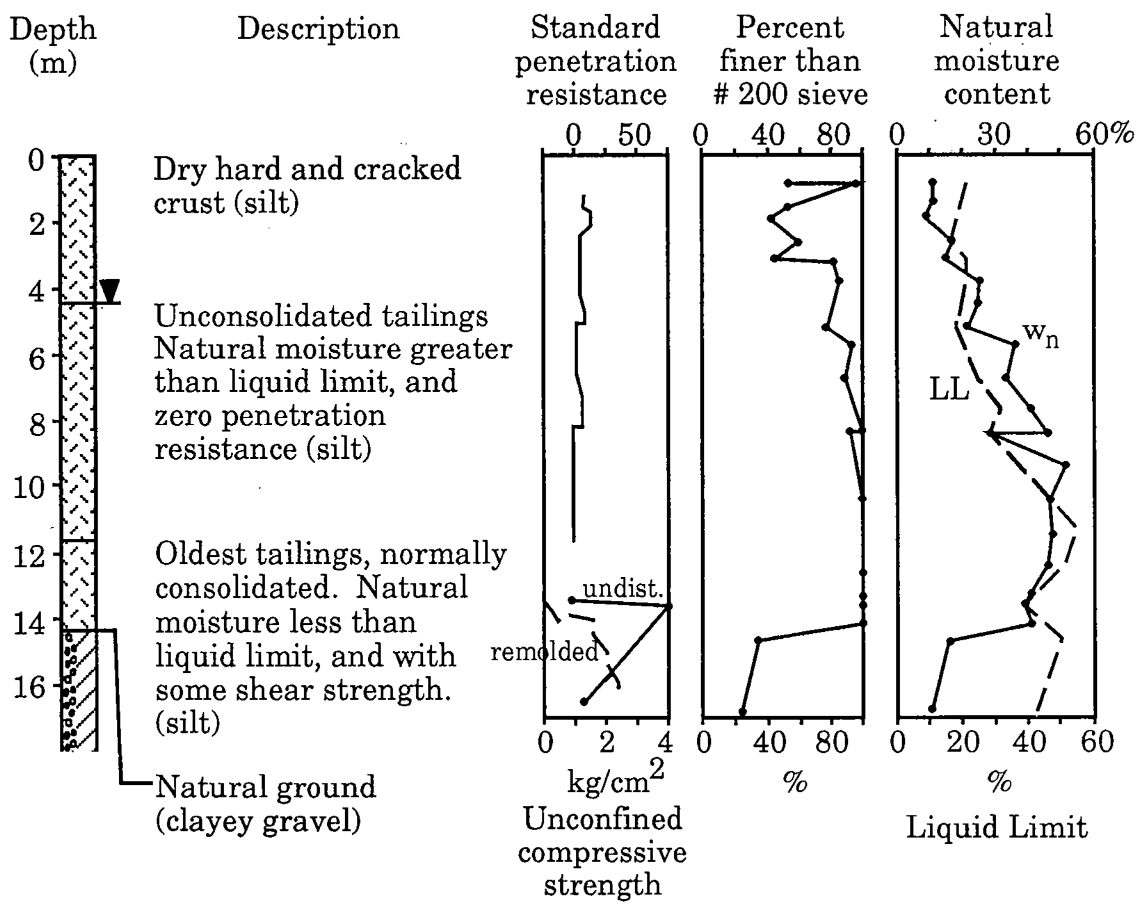


Fig. 23 Log of a Boring from the Central Part of the Small Dam at El Cobre (from Dobry and Alvarez, 1967)

The profile of the Old Dam after the failure consists of a series of terraces and scarps, as shown in Figure 24. It was surmised by Dobry and Alvarez (1967) that the underconsolidated tailings in the Old Dam liquefied as a result of the earthquake shaking and that the liquefied material applied pressure to the dam shell. A hypothetical failure mechanism taking into account a horizontal seismic acceleration and the increased pressure from the liquefied material is shown in Figure 25. In this scenario, intact blocks of the desiccated crust remained intact as the underlying liquefied tailings flowed out. The intact blocks created the final terraced profile shown in Figure 24. Silt boils were

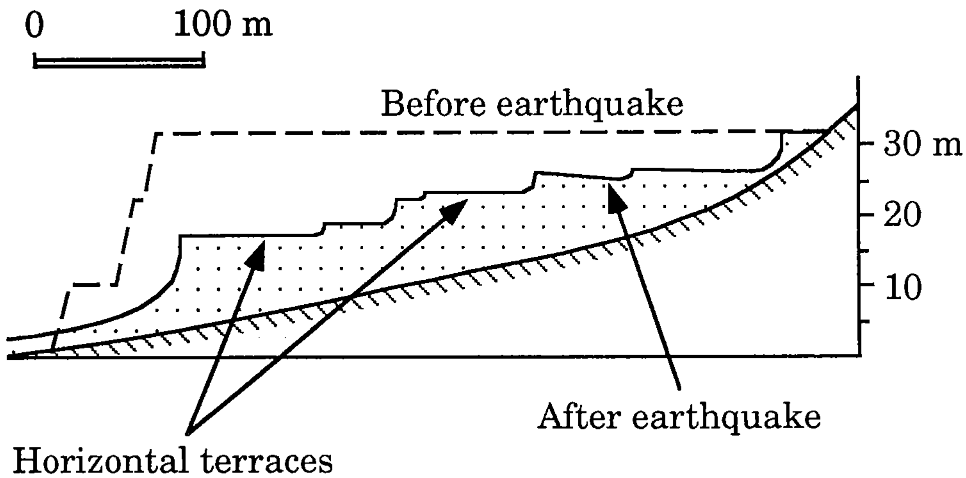


Fig. 24 Profile Before and After the Failure of the Old Dam at Al Cobre (from Dobry and Alvarez, 1967)

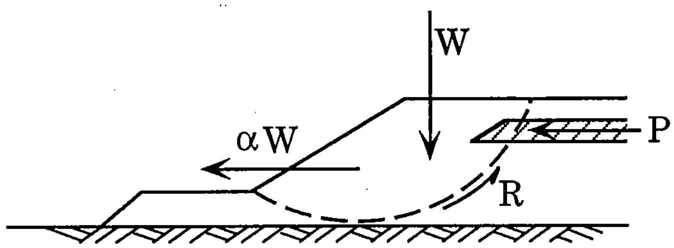


Fig. 25 Possible Failure Mechanism of the Old Dam at El Cobre (from Dobry and Alvarez, 1967)

found in the surface of the intact blocks, supporting the idea that the underlying tailings had liquefied. Similar post failure topographies and failure mechanisms have been observed for the Turnagain Heights landslides in the Great Alaska Earthquake of 1964 and the failure of the hydraulic fill Lower San Fernando Dam in the 1971 San Fernando earthquake.

8. CONCLUSION

Waste containment structures and in-ground flow barrier systems have demanding geotechnical requirements. The safe and economical design, construction, operation and maintenance of these environmental control systems require the application of the same principles and many of the same procedures that are needed for more traditional foundation engineering and earthwork construction projects. In this paper we have used waste landfills, tailings dams, and slurry wall examples to show the importance of proper application of soil mechanics in geoenvironmental engineering.

In virtually every case it is necessary to provide a proper idealization of the site conditions, to develop a reliable characterization of the soil and waste material properties, and to undertake analyses that provide insights into stability, deformation, settlement, and seepage behavior. The principles of soil mechanics - effective stress, seepage controlled by Darcy's law but with chemical transport often dependent on both advection and diffusion, compressibility, stress-dependent deformation and strength properties - are essential for proper execution of these analyses. In addition, success in the application of environmental geotechnics to the solution of geoenvironmental problems requires an understanding of the properties of dissimilar materials; e.g., soils, wastes of various types, and geosynthetics, appreciation of the importance of even very small quantities of pollutant transport, and that chemical and biological factors may have major impacts on the work that is done.

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