CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

This chapter presents background information about soil-bentonite cutoff walls and information gathered from the literature concerning the mechanical behavior of soil-bentonite cutoff walls. Background information includes the construction process and current design procedures for soil-bentonite cutoff walls. Current design procedures are based primarily on experience and not engineering principles. Design procedures address permeability of the soil-bentonite and stability of the trench during excavation. Most research to date has been focused on these two areas.

What is not addressed in current design procedures is how the stress state in the ground is changed by construction of the soil-bentonite cutoff wall and what deformations will result in the adjacent ground. Information on this mechanical behavior of soil-bentonite cutoff walls is limited. In order to better understand the mechanical behavior of soil-bentonite cutoff walls, there is a need for greater knowledge of engineering properties of soil-bentonite, the in situ stress state of soil-bentonite backfill, and deformations of soil-bentonite cutoff walls and adjacent ground. The limited current state of knowledge concerning each of these topics is presented in this section.

2.2 Construction Process

Soil-bentonite cutoff walls are constructed using the slurry trench method. Using this method, a narrow trench (typically 3 to 5 feet wide) is excavated under a slurry that is used to support the trench. For a soil-bentonite cutoff wall, the trench is filled with a bentonite-water slurry, which is typically 4% to 6% bentonite by weight (*Barrier* 1995). The slurry is kept at an elevation higher than the water table in the adjacent soil. This causes the slurry to flow into the adjacent soil, forming a thin layer of bentonite at the trench wall, which is referred to as a “filter cake” (Filz et al. 1997). The lateral pressure from the slurry in the trench acts against the filter cake and provides a stabilizing force.

Excavation of the trench is typically performed with a backhoe with a modified boom to depths of 60 feet, and with clamshells for deeper depths (*Barrier* 1995). As excavation proceeds along one end of the trench, the trench is backfilled with soil-bentonite at the other end. Initially, the soil-bentonite must be placed at the bottom of the trench with a clamshell until the backfill reaches the ground surface and creates a ramp as shown in Figure 2.1. Subsequently, soil-bentonite can be pushed into the trench and be allowed to slide down the slope of the existing backfill. The soil-bentonite displaces the bentonite-water slurry, since it has a higher density, and becomes the final cutoff wall backfill.

The soil-bentonite is a mixture of the soils excavated from the trench and bentonite-water slurry. Off-site soils may also be used if necessary. The soil-bentonite is typically mixed next to the trench with a bulldozer, although a mixing pit or a pugmill may also be used. The soil-bentonite generally has a hydraulic conductivity of $1x10^{-7}$ to $1x10^{-8}$ cm/s (*Barrier* 1995).

2.3 Current Design Procedures

Design of a soil-bentonite cutoff wall first involves establishing the alignment and the depth of the wall. This is determined based on the purpose of the cutoff wall and the site specific geology and hydrology. Soil-bentonite cutoff walls are typically keyed into an impervious layer to prevent seepage under the wall. If an upward gradient exists or can be created, or if contaminants are less dense than water, it may not be necessary to key the wall into an impervious layer; these types of cutoffs are referred to as hanging walls.

The thickness of the wall is typically 2 to 5 ft, which corresponds to typical widths of a backhoe bucket (D’Appolonia 1980). Evans (1993) recommends that if walls will be exposed to high hydraulic head conditions, such as beneath a dam, they should be analyzed for hydraulic fracture. If hydraulic fracture is a concern, a thicker wall is recommended (Millet et al. 1992). Although detailed design procedures are not available for analysis of hydraulic fracture of soil-bentonite cutoff walls, some rule-of-thumb approaches do exist, such as the U.S. Army Corps of Engineers' recommendation that soil-bentonite cutoff walls be at least 0.1 ft wide for every foot of head difference (USACE 1986).

Procedures are fairly well established for analyzing a slurry supported trench (Xanthakos 1994; Adams et al. 1997). The factor of safety against collapse of the trench can be calculated based on the stabilizing effect of the hydrostatic slurry pressure at the trench wall and a Coulomb force equilibrium method (Adams et al. 1997). For the full slurry pressure to act at the trench wall, an impermeable bentonite filter cake must form on the trench wall. If the D_{15} grain size of the adjacent soil is 0.4 mm or less, a filter cake is likely to form (Filz et al. 1997). It is often assumed that a filter cake will form, and that the trench will be stable provided the slurry in the trench is kept higher than the water table in the adjacent ground and the slurry density is high enough. However, there are cases where failure of the trench has occurred (Morgenstern and Amir-Tahmasseb 1965; Duguid et al. 1971; Puller 1974; Davidson et al. 1992). These failures have been attributed to one or more of the following factors: a sudden rise in groundwater level, a reduction in slurry density due to settling of suspended particles in the slurry, fissures in adjacent clay, and excessive slurry loss through very porous adjacent soils.

In developing specifications for construction of soil-bentonite cutoff walls, emphasis is placed on proper construction quality control. The following items are typically specified: contractor qualifications, bentonite material properties, water properties, bentonite-water slurry properties, soil-bentonite properties, trench excavation procedures, and soil-bentonite backfill mixing and placement procedures (USACE 1996). Properties of soil-bentonite are typically specified in order to achieve a low hydraulic conductivity cutoff wall. Properties of bentonite-water slurry are typically specified in order to maintain a stable trench during excavation. Recommended ranges of property values can be found in many references (D’Appolonia 1980; Evans 1991; Millet et al. 1992) and are not reported here. For the most part, the recommended values are based on past experience.

Review of the literature indicates that current design procedures are based on experience in order to achieve a soil-bentonite cutoff wall that is easily constructable, stable, and exhibits a low hydraulic conductivity. Current design procedures do not include consideration of the final state of stress in the soil-bentonite or deformations in adjacent ground.

2.4 Engineering Properties of Soil-Bentonite

This section presents engineering properties of soil-bentonite found in the literature with a focus on deformation and strength properties. There is limited published information on deformation and strength properties. It can be difficult to characterize soil-bentonite in general because soil-bentonite mixtures can vary greatly. One reason for the variation is that soil-bentonite is typically made by mixing material excavated from the trench with bentonite-water slurry, and the excavated material can vary greatly from site to site or even across a particular site.

General Properties

A primary goal in designing a soil-bentonite mixture is to provide a cost-effective, low permeability material. In addition, a relatively low compressibility soil-bentonite mixture is desirable in order to prevent excessive settlement in the trench and reduce adjacent ground deformations. There are several recommendations on grain size distributions of the soil-bentonite in order to achieve these goals. D’Appolonia (1980) states that a soil-bentonite will have low compressibility if there are enough granular particles to have grain to grain contact. For both low compressibility and low permeability, a well graded material with gravel through clay sized particles is recommended (D'Appolonia 1980; Evans 1991; Millet et al. 1992). D'Appolonia (1980) recommends a granular matrix with 20% to 40% plastic fines and a minimum of 1% bentonite. Evans (1991) recommends a well graded matrix with sand and gravel, 20% to 50% fines, and 1% bentonite. Millet et al. (1992) recommend a well graded material similar to a glacial till with 10% to 20% fines and 2% to 4% bentonite. They also state that other gradations such as fine sands and clays have also been used successfully.

For best placement consistency, the recommended slump is 4-6 inches (Evans 1991, Millet et al. 1992) or 2-6 inches (D'Appolonia 1980). The slump is measured with a standard concrete slump cone apparatus.

Compressibility Properties of Soil-Bentonite

D'Appolonia (1980) plots the compression ratio versus fines content for various soil-bentonite mixtures as shown in Figure 2.2. The compression ratio is defined as

$$\text{Compression Ratio} = \frac{C_c}{1 + e_o} \tag{2.1}$$

where: C_c = Compression index

e_o = initial void ratio

The compression ratio corresponds to the stress range from 1000 to 4000 psf. Data from both one dimensional compression and isotropic compression is included in the figure. It can be seen that the compressibility increases with fines content. Also, soil-bentonites with plastic fines are more compressible than soil-bentonites with non-plastic fines. In general, a soil-bentonite with 20% to 40% fines has a compression ratio between 0.02 and 0.07 for the stated stress range. It can also be seen that soil-bentonite in one-dimensional compression has a higher compression ratio than in isotropic compression.

Khoury et al. (1992) present data from a soil-bentonite cutoff wall constructed in an earth dam. Several different soil-bentonite mixtures were tested. The mixtures were prepared with various grain size distributions to represent the range of onsite backfill material. The information is summarized in Table 2.1. The compressibility increases with fines content, following the same trend as D'Appolonia's data (1980). The void ratios or stress increment associated with the compression and swell ratios were not given. It appears that the mixtures tested by Khoury et al. (1992) are slightly more compressible than those reported by D'Appolonia (1980).

Table 2.1 Consolidation Data on Various Soil-Bentonite Mixtures (after Khoury et al. 1992)

Soil-Bentonite Property	Mix 3	Mix 4	Mix 5
Percent Bentonite	1.05	1.18	1.65
Percent Passing No. 200 Sieve	30.5	37.6	72.5
Water Content (%)	31.0	39.0	62.1
Compression Ratio	0.077	0.091	0.137
Swell Ratio	0.005	0.006	0.010
Coefficient of Consolidation for 2000-4000psf (ft^2/yr)	292	215	70

Evans and Cooley (1993) present consolidation data from undisturbed samples taken from a 4 year old and a 10 year old soil-bentonite wall. The consolidation information is presented in Table 2.2. The compression ratios are more similar to those reported by Khoury et al. (1992) than to those reported by D'Appolonia (1980). Standard penetration tests performed on the walls showed that resistance was weight of hammer at all depths (Evans et al. 1995).

Table 2.2 Consolidation Data from Undisturbed Samples (Evans and Cooley 1993)

Age of Wall (Years)	Sample Depth (Feet)	Compression Ratio	Swell Ratio
4	7	0.088	0.006
4	9	0.108	0.009
4	14.5	0.147	0.006
10	9.5	0.110	0.015
10	13	0.097	0.007

Strength Properties of Soil-Bentonite

Published information on strength properties of soil-bentonite is limited. Normalized triaxial test data from CD and CU tests by D'Appolonia (1980) on three different soil-bentonite mixtures is shown in Figure 2.3. For the CU tests, failure was taken as the point of maximum principal stress ratio (σ_1'/σ_3'). The s_u/p ratios for samples B and C are 0.40 and 0.32 respectively. The s_u/p ratio for sample A is 0.70. It can be seen from the stress path, that sample A shows a tendency for dilatant behavior in the CU test. In an undrained test, the dilatant tendency causes a decrease in pore pressure. The decrease in pore pressures results in a much higher deviator stress for sample A than for the other two samples which do not exhibit dilatant tendencies. The strain to failure ranges from approximately 5% to 10% in the undrained tests. All samples show similar behavior in the CD tests. The effective friction angle ranges from 31 to 33 degrees. The effective cohesion is zero for all samples. The strain to failure for the CD tests ranges from approximately 10% to 20%. All tests are normalized to the effective consolidation stress, which was not provided.

One additional value of effective friction angle is reported in the literature. Filz (1995) reports an effective friction angle of 32 degrees from CU tests for a soil-bentonite wall at a site in Silicon Valley, California. Another value can be interpreted from data in Evans et al. (1995). The authors use a K_o value for soil-bentonite based on the Brooker and Ireland correlation and triaxial testing. Using their value of 0.37 for K_o and the relationship, K_o=0.95-sinϕ' (Brooker and Ireland 1965), the effective friction angle would be 35 degrees.

All of the reported values of effective friction angle for soil-bentonite mixtures are between 31 and 33 degrees. Though not directly reported, an effective stress friction angle of 35 degrees can be interpreted for a soil-bentonite mixture described by Evans et al. (1995).

Hydraulic Conductivity of Soil-Bentonite

There is a significant amount of data reported on hydraulic conductivity of soil-bentonite. The effects of grain size distribution, percent bentonite, consolidation pressure, field stress conditions, permeameter type, water table position, gradient, and permeant on the hydraulic conductivity of soil-bentonite have been reported (Evans 1994; Evans and Fang 1988; Barvenik and Ayers 1987; D'Appolonia 1980). A summary of the literature to date on hydraulic conductivity of soil-bentonite is presented in Adams et al. (1997). Although hydraulic conductivity of soil-bentonite cutoff walls is not the focus of this research, low hydraulic conductivity is a primary function of the wall, and this research has implications for hydraulic conductivity measurements of soil-bentonite.

An issue that is pertinent to this research is the effect of confining pressure on the measured hydraulic conductivity. It is well established that testing conditions should simulate field conditions and that field stress is an important consideration (Mitchell and Madsen 1987; Rad et al. 1995). It is also well established that an increase in confining pressure will cause a reduction in hydraulic conductivity, and that the effect is more pronounced with more compressible soils, such as soil-bentonite (Evans 1994; Heslin et al. 1997) than less compressible soil, such as compacted sand-bentonite liners (Kallur et al. 1995). If values of confining stress that are used in testing soil-bentonite are greater than those in the field, the hydraulic conductivity may be significantly underestimated, which would be unconservative for a cutoff wall.

Figure 2.4 shows the effect of confining pressure on hydraulic conductivity (*Barrier* 1995) for several different soil-bentonite mixtures. The figure shows that an increase in effective confining stress of 2090 psf (100 kPa) can decrease the measured hydraulic conductivity by an order of magnitude. It should be noted that, based on a review of one of the data sources, some of the data appears to be misinterpreted as plotted in Figure 2.4. Data that is plotted as open circles with a dotted line is from McCandless and Bodocsi

(1988). These results are from small-scale models of soil-bentonite cutoff walls. In the small scale tests, a vertical surcharge was applied to the soil-bentonite wall with a horizontal hydraulic gradient to induce horizontal flow through the wall. To try and account for the effect of both the horizontal seepage forces and the vertical surcharge, the resulting hydraulic conductivity was reported versus the sum of the vertical effective stress and the horizontal hydraulic pressure. The summation of the horizontal pressure and the vertical surcharge should not have been interpreted to represent the confining pressure on the soil-bentonite. Disregarding this data as plotted in Figure 2.4, the data still illustrates that confining pressure has a significant effect on the measured hydraulic conductivity of soil-bentonite.

The selection and use of an appropriate confining pressure for hydraulic conductivity tests on soil-bentonite remains an unresolved issue. In the current state of the practice, confining stresses are not always considered or specified. Confining stresses as large as 10,000 psf have been used (Zamojski et al. 1995) in one case history. Such high pressures produce low measurements of hydraulic conductivity, but they are unrepresentative of field conditions. Some authors recommend or cite use of confining stresses corresponding to the middle of the wall assuming geostatic conditions (Rad et al. 1995; Burke and Achhorner 1988). Some authors recommend or cite use of confining stresses corresponding to "top of wall" stresses (Barvenik and Ayers 1987; Grube 1992). The term, "top of wall" is not always strictly defined, although Barvenik and Ayers (1987) define it as the upper 5 to 10 feet of the wall.

The issue of using an appropriate confining pressure is further complicated by the fact that estimating the stresses in a soil-bentonite cutoff wall is also an unresolved issue. It has been shown that stresses in soil-bentonite cutoff walls are less than geostatic (Evans et al. 1995). Since higher consolidation pressures significantly decrease the hydraulic conductivity of soil-bentonite, using geostatic stresses for hydraulic conductivity testing will lead to unconservative estimates of hydraulic conductivity for soil-bentonite. Evans

(1994) states that "unless the state of stress in the field is known, the hydraulic conductivity remains uncertain." Estimating the stresses in a soil-bentonite cutoff wall is discussed in further detail in the next section.

2.5 In Situ State of Stress in Soil-Bentonite Backfill

The final in situ state of stress in soil-bentonite cutoff walls is generally not known, yet it is important for many reasons. As previously discussed, the stress state of soil-bentonite significantly influences the measured hydraulic conductivity. Also, higher in situ stresses increase the resistance of the cutoff to hydraulic fracture and chemical attack (Filz 1995). The final stress state also influences the deformations that occur in the wall and adjacent to the wall. Many authors agree on the need for greater understanding of the state of stress in soil-bentonite cutoffs (Khoury et al. 1992; Evans 1994; Filz 1995)

When the soil-bentonite is initially placed into the trench, the water content is very high and the strength of the soil-bentonite is very low; it flows into the trench. As the trench is filled from the bottom up with soil-bentonite, it takes time for the soil-bentonite to consolidate and feel the effective stresses produced by the soil-bentonite above and the stresses in the adjacent ground. It is generally agreed that the final stress state in the soil-bentonite is less than geostatic, and there are currently two simplified theories to predict the stresses. These two theories, arching theory and lateral squeezing theory, will be presented in this section along with the few reported cases of field data on the in situ stress state of soil-bentonite.

Arching Theory

Arching is the conventional theory that is cited to explain vertical stresses in soil-bentonite trenches that are less than geostatic (Evans et al.1995; Rad et al. 1995). Terzaghi (1943) states that arching is the "transfer of pressure from a yielding mass of soil onto adjoining stationary parts" and that arching is one of the most common phenomena of soil behavior. Arching has been used to describe the pressures induced by grain in silos, vertical loads on pipes in trenches, pressures on retaining walls, and stresses in narrow clay cores in dams (Terzaghi 1943; Blight 1973; Handy 1985). According to Terzaghi (1943), arching predicts that the pressure in the yielding backfill of a vertical trench will increase at a rate that is less than geostatic and will approach a limiting value.

Evans et al. (1995) applied arching theory to soil-bentonite cutoff walls. Settlement of the soil-bentonite causes transfer of loads to the vertical sides of the trenches, which are assumed to be rigid and unmoving. The weight of the soil-bentonite is partially supported by the upward frictional resistance along the sides of the trench. Evans et al. (1995) present a closed form solution to the problem. The vertical stress in the soil-bentonite wall is given as a function of trench width, unit weight of the soil-bentonite, lateral earth pressure coefficient of the soil-bentonite, and interface friction between the soil-bentonite and the trench wall. As an example, the stresses in a 3 foot wide trench are shown in Figure 2.5, as predicted by arching theory. Geostatic conditions are also plotted on the figure. Using reasonable soil properties, it can be seen from the figure that arching theory predicts that vertical stresses in a soil-bentonite wall can be lower than geostatic stresses by an order of magnitude.

Lateral Squeezing Theory

Lateral squeezing theory (Filz 1996) is an alternative to arching for predicting in situ stresses in soil-bentonite walls. In lateral squeezing, it is assumed that the trench walls can deform and that the amount and direction of movement influence the stresses in the soil-bentonite. The lateral squeezing theory is derived in detail by Filz (1996) and is only presented here in abbreviated form. After placement of soil-bentonite in the trench, the long and narrow walls of the trench are assumed to move inwards due to consolidation of the soil-bentonite. The column of soil-bentonite is treated like a conventional 1-D consolidation test turned 90 degrees. It is assumed that the shear forces along the trench walls are sufficient to support the weight of the overlying soil-bentonite. The constrained modulus, M, is used to relate the horizontal stress in the soil-bentonite to the horizontal strain. As the trench walls move in, the horizontal stress in the adjacent ground can be estimated from Clough and Duncan's (1991) relationship between the lateral earth pressure coefficient, K_h, and lateral movement. The horizontal stresses in the soil-bentonite and in the adjacent ground are equated to solve for the horizontal stress as a function of the trench width and constrained modulus. The vertical stress in the soil-bentonite can be calculated using the K_o conditions that are assumed for 1-D lateral consolidation in the trench. According to Filz (1996), the larger of the consolidation stresses produced by arching theory and lateral squeezing should be assumed to control. Typically, arching controls in the upper several feet of the trench and lateral squeezing controls at greater depths.

In lateral squeezing, the vertical effective stress is the minor principal stress and the horizontal stress is the major principal stress. The major principal effective stress is plotted with depth in Figure 2.5 for the same example used for the arching theory. The lateral squeezing theory predicts lower stresses than geostatic conditions, but higher stresses than arching theory over most of the depth of the trench. The figure demonstrates the wide range of effective stresses that can be assumed. These ranges in effective stress have significant implications concerning the hydraulic conductivity of the cutoff and its resistance to hydraulic fracture and chemical attack.

<u>Field Data of In Situ Stress State of Soil-Bentonite Backfill</u>

Evans et al. (1995) performed in situ tests and laboratory tests on a 4 year old wall, a 10 year wall, and a newly constructed soil-bentonite wall. Their results generally indicate that in situ stresses in the trench are low and are less than geostatic:

- SPT tests indicate that the sampler was advanced by "weight of hammer" throughout the entire depth of each trench.
- Vertical effective stresses, estimated from dilatometer tests, increase with depth but are less than geostatic.
- Vane shear testing indicates that shear strength is approximately constant with depth and lower than would be produced by geostatic stresses.

- Consolidation tests on undisturbed samples show a lot of scatter, but preconsolidation pressures are less than geostatic.

Engemoen and Hensley (1986) present CPT testing done on a recently completed soil-bentonite cutoff wall constructed in Calamus dam. One portion of the wall was 5 ft wide and 110 ft deep and another portion was 3 ft wide and 45 ft deep. The measured tip resistance in the soil-bentonite was generally less than 10 tsf and did not increase with depth. In several tests, large horizontal deviations of the tip of the cone (as much as 17 feet) resulted in higher tip resistances, which were attributed to the high probability of the tip hitting the side of the trench. Because the CPT does not have good resolution below 10 tsf, it is difficult to draw definitive conclusions regarding consolidation pressures in the soil-bentonite from this data.

Khoury et al. (1992) present in situ stress data from a soil-bentonite cutoff wall constructed in Manasquan dam. The cutoff wall had a maximum depth of 75 feet, with some sections of the wall 3 feet wide and other sections 5 feet wide. Piezometers and total stress cells were pushed into the cutoff wall after construction. The total stress cells measured the total horizontal pressure in directions parallel, $\sigma_{h,par}$, and perpendicular, $\sigma_{h,per}$, to the dam axis. Using the measured pore pressures, u, the total horizontal pressure measured in the soil-bentonite, and estimated values of the lateral earth pressure coefficient, K_o, the authors estimated the total vertical pressure, σ_v, at 3 depths using equation 2.2.

$$\sigma_{h,par} = K_o(\sigma_v - u) + u \tag{2.2}$$

The estimated total vertical pressures were 66%, 78%, and 82% lower than the overburden pressure at depths of 28 ft, 43 ft, and 53 ft respectively. The corresponding effective vertical pressures in the soil-bentonite would be even smaller percentages of the effective vertical pressure based on geostatic conditions. The measurements taken at the 28 ft depth were taken in the 3 foot width wall, and the measurements taken at 43 ft and 53 ft were taken in the 5 foot width wall.

2.6 Deformations of Soil-Bentonite Cutoff Walls and Adjacent Ground

Deformations can be important design considerations for soil-bentonite cutoff walls for various reasons. For a cutoff wall in a large dam, it is important for the cutoff to be able to withstand significant deformations without cracking. At sites with adjacent structures, it is important to limit ground deformations to avoid damage to the adjacent structures. Damages to adjacent buildings have been reported due to construction of soil-bentonite cutoff walls (Filz 1996).

If the trench walls are deformable, as assumed in the lateral squeezing theory, deformations can occur during each construction phase (Filz 1996). As shown in Figure 2.6, the trench walls may move inward during excavation under bentonite-water slurry. This would induces settlement of the ground surface, which could cause damage to adjacent structures. During the backfilling process, the bentonite-water slurry is replaced with soil-bentonite, which has a higher unit weight. This could cause the trench walls to move outward and reduce surface settlements. During consolidation of the soil-bentonite, the trench walls may again move inward, resulting in more surface settlements.

This section presents information from the literature on deformations due to construction of soil-bentonite cutoff walls. There is some information in the literature on deformations of adjacent ground due to excavations of slurry filled trenches. Most of this information is reported for structural slurry walls. There is very little information on deformations of adjacent ground after backfilling with soil-bentonite and deformations due to consolidation of the soil-bentonite.

<u>Deformations Due to Excavation of Slurry Filled Trenches</u>

Information on deformations due to excavation of a trench filled with slurry have been reported for excavations of structural slurry walls. However, there are important differences between structural slurry walls and soil-bentonite cutoff walls that affect deformations.

Structural walls are typically constructed in short panels and not continuous trenches like soil-bentonite cutoff walls. Finite element analyses of short panels has shown that the lateral pressures acting perpendicular to the panel are transferred to the soil at the ends of the trench (Wong 1984). Thus, the use of short panels for structural slurry walls limits the lateral pressure that is exerted on the trench wall and most likely limits the amount of resulting deformations.

Design procedures for structural slurry walls typically consider stability of the trench filled with slurry, stability of the braced or tied back excavation within the completed diaphragm walls, and estimation of ground deformations as a result of the excavation within the completed diaphragm walls (Cowland and Thorley 1985; Poh and Wong 1998). Although the wall is typically “wished-in-place” during design and no deformations are assumed to be associated with wall construction (Poh and Wong 1998), these deformations have been shown to be significant (Poh and Wong 1998; Cowland and Thorley 1985).

Cowland and Thorley (1985) present settlement data for buildings adjacent to slurry diaphragm walls from 7 sites in Hong Kong. The walls varied from 2.6 ft to 3.9 ft in width and 65 ft to 164 ft in depth. All of the walls were constructed in reclaimed land close to the coast with similar geologic profiles. All of the walls were in high density urban areas with immediately adjacent structures. Structures ranged from 3 to 4 story buildings with shallow foundations to 30 story buildings with deep foundations. They state that it is typically assumed that deformations will be small if there is an adequate factor of safety against overall instability of the trench but that this was not found to be the case in Hong Kong. Building settlements of more than 2 inches were recorded.

The data from ground settlement and building settlements were plotted as a function of distance from the trench as shown in Figures 2.7 and Figure 2.8. The settlements and distance to the trench are normalized by the trench depth. The figures show that settlement decreases with distance from the trench, but that significant settlement can occur as far away as 1.5 times the depth of the trench. Figure 2.7 also shows the effect of panel construction on the deformations. The trend-line labeled E corresponds to data measured at the ends of a panel, and the trend-line labeled M corresponds to data measured at the middle of a panel. Movement near the ends of a panel is less than movement near the middle of a panel.

The authors also plot building settlement as various functions of foundation depth, trench depth, and depth of completely decomposed granite. They found a relationship between building settlement and the ratio of the foundation depth to trench depth. They postulate that the results may be specific to Hong Kong due to the following factors: 1) presence of decomposed granite which has high swelling capacity, high permeability, and boulders (the presence of boulders required chiseling and blasting in the slurry trench at many of the sites), and 2) high ground water table (typically 3 to 10 feet depth) and close proximity to sea-water.

Poh and Wong (1998) present deformations from a field test consisting of construction of one panel of a structural slurry wall. The panel was 3.9 ft thick, 19.7 ft long, and 182.1 ft deep. Inclinometers and settlement markers were installed to measure lateral and vertical movements at various distances from the panel. Excavation of the panel caused the ground adjacent to the panel to move laterally toward the panel and settle. Backfilling of the panel with concrete caused the ground adjacent to the panel to move laterally away from the panel and heave. The effects of standing time and fluctuations in slurry level were also investigated.

Typical results can be seen by looking at the maximum lateral movement in the closest inclinometer, which was 6.0 feet from the panel centerline. Maximum lateral movement occurred at a depth of approximately 36 ft, which is located in the middle of a very soft to soft clay layer. During excavation, the maximum lateral movement was 0.85 inch. After excavation, the panel was left standing for 24 hours, after which the level of the slurry was varied. The stand time increased the maximum lateral movement by 0.11 inch. Raising, lowering, and raising the slurry level back to the original level resulted in increasing the maximum lateral movement by an additional 0.46 inch. Backfilling with concrete caused the maximum lateral movement to decrease by 0.92 inches. In the 3 days after concreteing, the maximum lateral movement increased slightly by 0.13 inch. A trend of decreasing maximum lateral movement with distance from the trench was observed. Less movement was observed in layers of stiffer soils.

A maximum settlement of 0.9 inch was measured during excavation. The maximum settlement occurred next to the trench at the guide wall. Maximum settlement decreased with distance from the trench. Neither the 24 hours of stand time, nor raising the slurry level caused significant changes in settlement. Lowing the slurry caused an increase in maximum settlement of 0.43 inch at the guide wall. Backfill with concrete, caused a uniform reduction in settlements of about 0.2 inch.

Deformations Due to Consolidation of Soil-Bentonite

Vertical deformations of soil-bentonite walls due to consolidation are reported for several case histories. Khoury et al. (1992) present settlement versus time data from a soil-bentonite cutoff wall built in Manasquan dam. Some portions of the wall were 3 feet wide and other portions were 5 feet wide. The soil-bentonite wall was constructed in 2 stages. The lower stage was constructed when the dam reached 45 feet in height. The upper stage was constructed when the dam reached 55 feet in height. The upper stage was keyed into the lower stage by at least 3 feet. Vertical deformations with time were measured in the soil-bentonite trench using settlement plates. The lower stage was an average of 56 feet deep and underwent most of its settlement in 1-2 months. The upper stage was an average of 18 feet deep and experienced most of its settlement in about 2 weeks. The 3 foot section experienced a total of 3-4% vertical strain. The 5 foot section experienced a total of 7-9% vertical strain.

Engemoen and Hensley (1986) report that a soil-bentonite cutoff wall at Calamus dam underwent 0.1% vertical strain, which occurred in one month. The cutoff wall was up to 110 feet deep with widths from 3 to 5 feet.

Project files from Woodward-Clyde Consultants were reviewed and data on 4 soil-bentonite case histories were compiled and documented by Tiffany Adams (Adams et al. 1997). One site, owned by the Raytheon Company, is particularly well instrumented and documented. At this site, there were no stability problems; however, deformations of the trench caused structural damage to an adjacent building. Lateral deformation data were recorded for all phases of construction including excavation, backfill, and consolidation. This case history shows that most of the lateral deformation that was measured in ground adjacent to the trench occurred after backfilling of the trench with soil-bentonite. This case history was used to calibrate the finite element model developed for this research and is described in detail in Chapter 5.

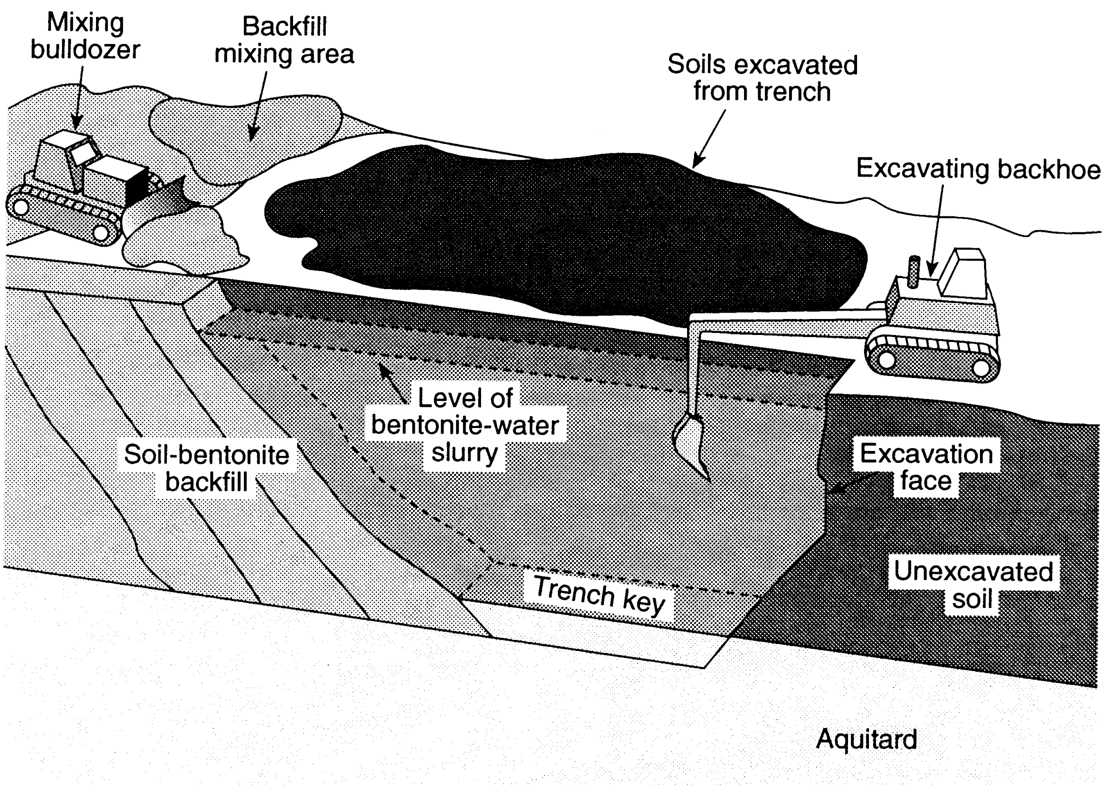


Figure 2.1 Soil-Bentonite Cutoff Wall Construction Process (*Barrier* 1995)

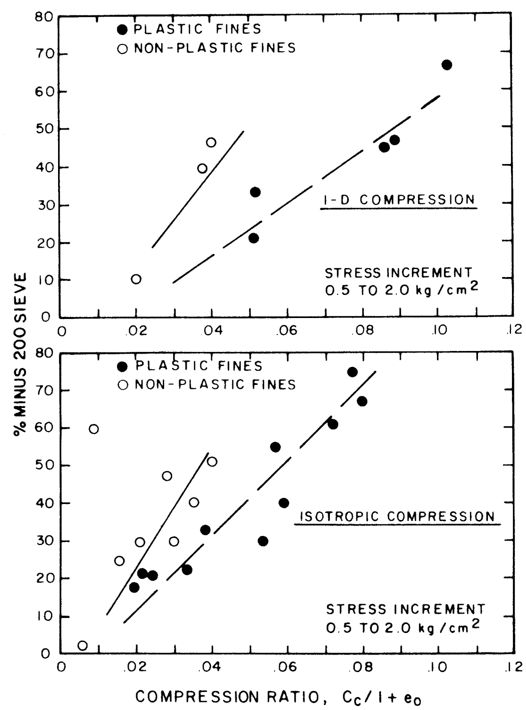


Figure 2.2 Compression Ratio Versus Fines Content for Various Soil-Bentonite Mixtures (D'Appolonia 1980)

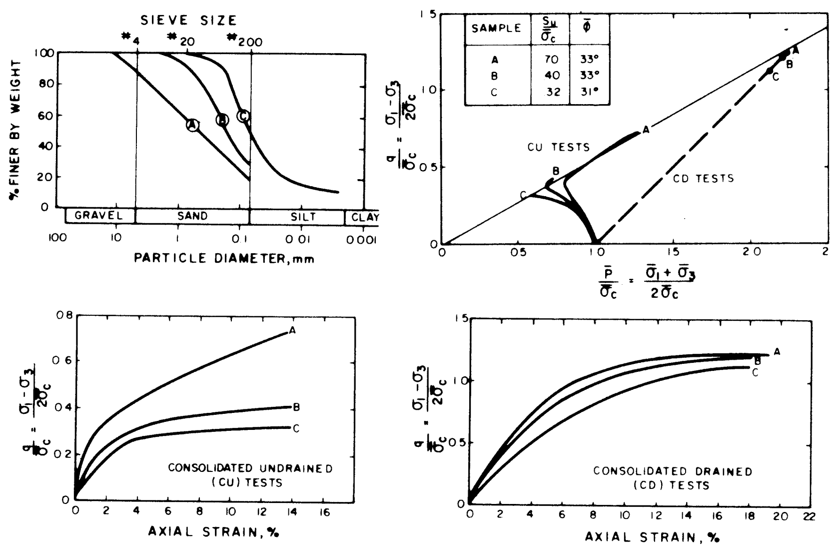


Figure 2.3 Triaxial Test Data on Various Soil-Bentonite Mixtures (D'Appolonia 1980)

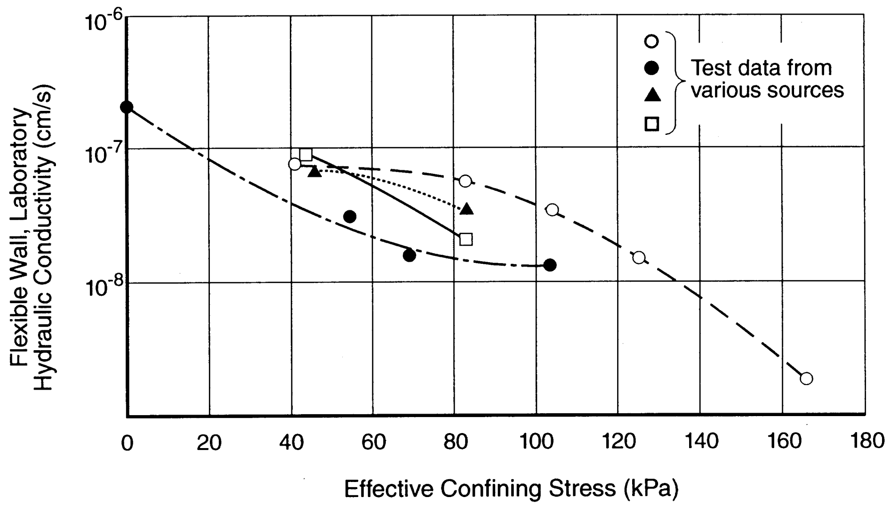
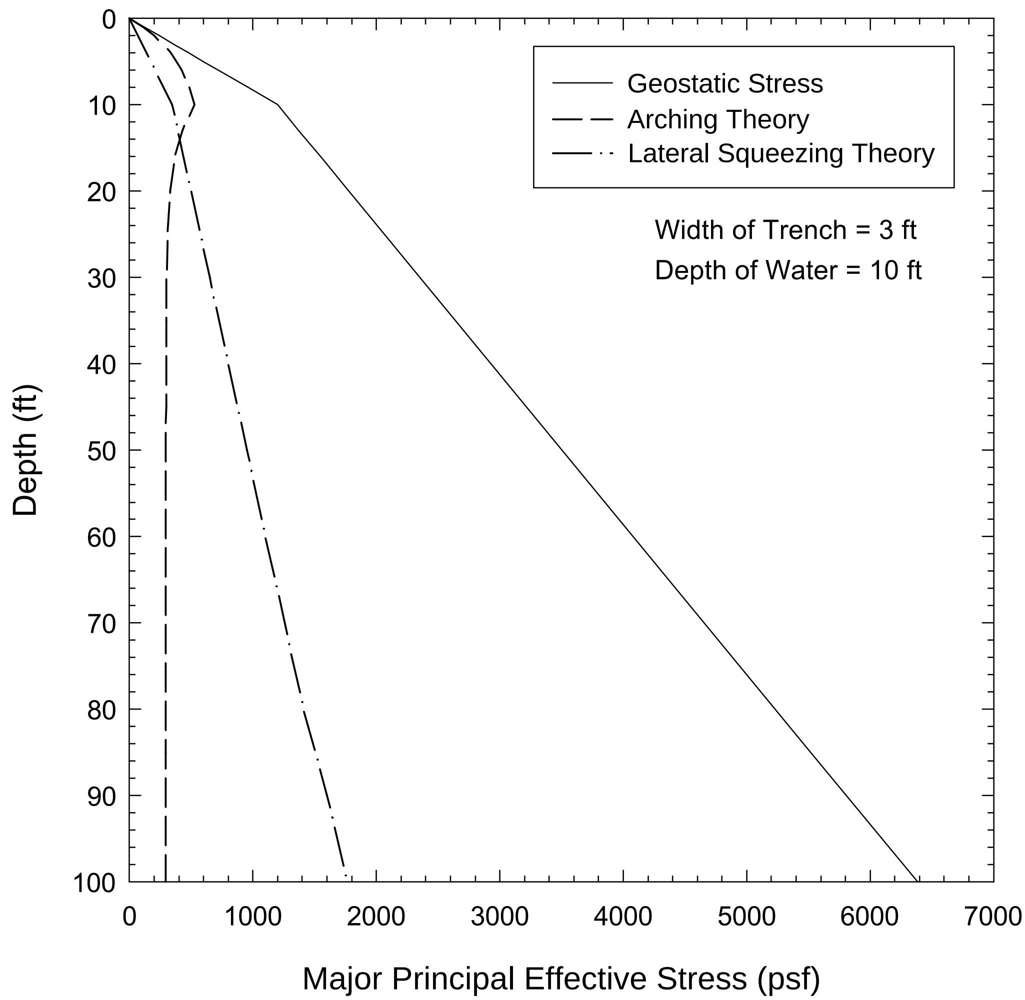


Figure 2.4 Effect of Consolidation Pressure on Hydraulic Conductivity of Soil-Bentonite Mixtures (*Barrier* 1995)



Assumptions:
Unit weight of soil adjacent to trench = 125 pcf
Unit weight of soil-bentonite = 120 pcf
Constrained modulus of soil-bentonite = 8000 psf
K_o of soil-bentonite = 0.47
Cohesion of interface = 0 psf
Effective stress friction angle of interface = 32 degrees

Figure 2.5 Theoretical Predictions of Major Principal Effective Stress in a Soil-Bentonite Cutoff Wall

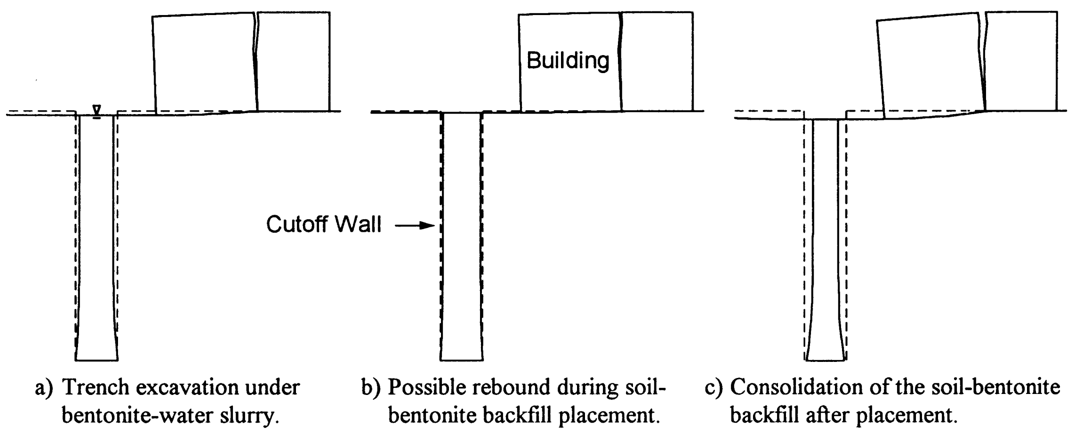


Figure 2.6 Ground Deformations Due to Construction of a Soil-Bentonite Cutoff Wall (Filz 1996)

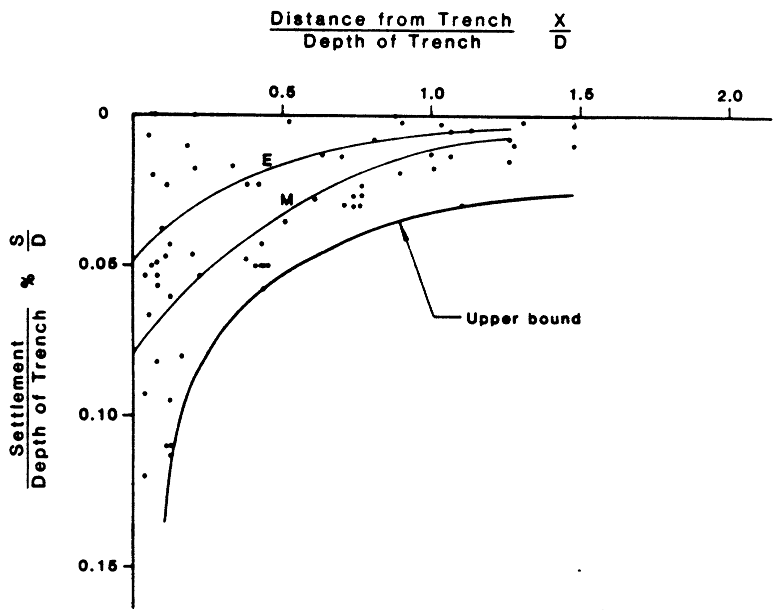


Figure 2.7 Ground Settlement Due to Slurry Trench Excavation (Cowland and Thorley 1985)

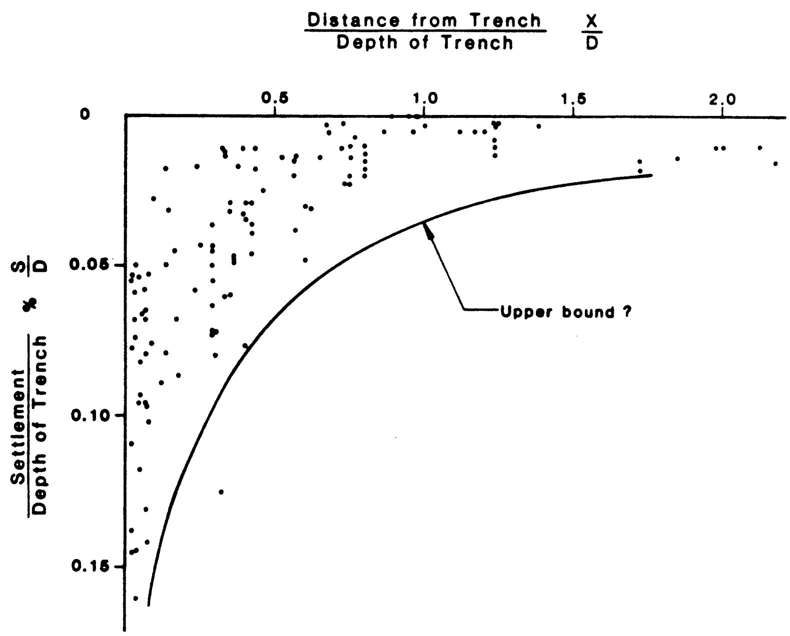


Figure 2.8 Building Settlements Due to Slurry Trench Excavation (Cowland and Thorley 1985)