

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Introduction

A soil-bentonite cutoff wall is a type of subsurface vertical barrier constructed by back-filling a trench with a mixture of soil, bentonite, and water. It is typically constructed using the slurry trench method, in which the trench is stabilized with bentonite-water slurry. The purpose of a soil-bentonite cutoff wall is to create a low permeability structure in the ground to contain or direct groundwater flow. Although soil-bentonite cutoff walls are common, their mechanical behavior is not well understood. Current design procedures for soil-bentonite cutoff walls are based on experience, with objectives of producing constructable, stable, and low hydraulic conductivity cutoff walls. Current design procedures do not consider the final state of stress in the soil-bentonite backfill or deformations in adjacent ground. The final stress state in the completed wall is important because it influences the hydraulic conductivity of the cutoff (*Barrier* 1995), the susceptibility to hydraulic fracture, and the magnitude of deformations adjacent to the cutoff wall. Deformations adjacent to the cutoff wall can be significant and can cause damage to adjacent structures (Filz et al. 1999).

The objectives of this research are to provide information about the mechanical behavior of soil-bentonite cutoff walls. Specific objectives are to 1) add to the current body of knowledge of soil-bentonite properties, 2) evaluate constitutive models and select a model to represent soil-bentonite, 3) model a soil-bentonite cutoff wall using finite elements, and 4) investigate the influence of several factors on the deformations in adjacent ground.

Objective 1 was met by first summarizing information in the literature on soil-bentonite properties and then executing a laboratory testing program on soil-bentonite. Objective 2 was met by evaluating the degree to which different constitutive models match the data

from the laboratory testing program, and selecting a model that best represents the behavior of soil-bentonite. Objective 3 was met by developing a finite element model of a soil-bentonite cutoff wall using a well-documented case history. Objective 4 was met by performing parametric studies using the finite element model.

The rest of this chapter provides summaries and conclusions from the literature review, laboratory testing program, constitutive modeling of soil-bentonite, finite element modeling of a soil-bentonite cutoff wall, and parametric study. Also included in this chapter are a discussion of the practical implications of this research for the practicing engineer, and recommendations for future research.

8.2 Literature Review

Summary of Literature Review

A literature review was performed to gather information concerning the mechanical behavior of soil-bentonite cutoff walls. In addition, information on the construction process and design procedures for soil-bentonite cutoff walls was presented. The limited available information on mechanical properties and behavior of soil-bentonite cutoff walls can be categorized as follows: engineering properties of soil-bentonite, in situ stress-state of soil-bentonite backfill, and deformations of soil-bentonite cutoff walls and adjacent ground.

Conclusions from Literature Review

1. Current design procedures for soil-bentonite cutoff walls are based on experience, with the objective of achieving a soil-bentonite cutoff wall that is easily constructable, stable, and exhibits a low hydraulic conductivity. Current design procedures do not consider the final state of stress in the soil-bentonite backfill or deformations in adjacent ground.
2. Strength and compressibility data on soil-bentonite in the literature is limited; however, reported effective friction angles fall within a narrow range of 31 to 33 degrees.

3. Consolidation pressure is an important consideration for hydraulic conductivity testing. Increasing the consolidation pressure results in a decrease in hydraulic conductivity. In some cases, it appears that the consolidation pressures used in laboratory testing may have been too high, leading to under-estimates of hydraulic conductivity.
4. Two theories are available to predict the final stress-state in the soil-bentonite backfill, arching theory and lateral squeezing theory. According to both theories, the effective stresses in the backfill are smaller than geostatic stresses. To date, there is only limited field data to support the theories (Evans et al. 1995; Khoury et al. 1992).
5. The final stress state in the completed wall is important because it influences the hydraulic conductivity of the cutoff (*Barrier* 1995), the susceptibility to hydraulic fracture, and the magnitude of deformations adjacent to the cutoff wall.
6. Deformations due to construction of soil-bentonite cutoff walls and excavation of slurry filled trenches have caused damage to adjacent structures (Filz 1996). There is very limited field data on deformations in adjacent ground due to construction of soil-bentonite cutoff walls. One case history was found that was well instrumented, with inclinometers installed adjacent to a soil-bentonite cutoff wall. The inclinometer data indicate that the majority of measured lateral deformations occurred after construction of the cutoff wall, most likely due to consolidation of the soil-bentonite backfill.

8.3 Laboratory Testing Program

Summary of Laboratory Testing Program

A laboratory testing program was performed on three soil-bentonite mixtures (SB1, SB2, and SB3) to determine strength and compressibility properties. The soil-bentonite mixtures were fabricated in the laboratory by mixing various soils. SB1 and SB2 are identical except that SB1 was made with distilled water and SB2 was made with tap water. SB1 is classified as a clayey sand (SC) according to the Unified Soil Classification System (USCS), and SB3 is classified as a silty sand (SM) according to the USCS.

SB1 has 1.5% bentonite (by dry weight) and 35% plastic fines. SB3 has 3% bentonite and 16% non-plastic fines.

The most extensive testing was performed on SB1. Index testing and slump testing were performed. Consolidation properties were found from 1-D and isotropic consolidation tests. Hydraulic conductivity tests were performed in rigid wall permeameters. Stress-strain properties were determined from anisotropic and isotropic CU compression tests and isotropic CD compression tests.

Only hydraulic conductivity tests were performed on SB2. Tests were performed in flexible and rigid wall permeameters.

Extensive testing was also performed on SB3, but with a slightly smaller scope than the testing performed on SB1. Index tests and 1-D consolidation tests were performed. Hydraulic conductivity tests were performed in rigid wall permeameters. Stress strain properties were determined from isotropic CU and CD tests.

Conclusions from Laboratory Testing Program

1. The relationship between slump and water content was plotted for SB1, as well as for various soil-bentonite mixtures in the literature. The slump increases as water content increases, and all of the data falls within a narrow band regardless of the grain size distribution of the soil-bentonite.
2. The same hydraulic conductivity was measured for SB1 and SB2. The mixtures are identical except that SB1 was created and permeated with distilled water and SB2 was created and permeated with tap water. The results indicate that for this soil-bentonite mixture, the effect of using distilled water versus tap water for hydraulic conductivity testing is not significant.
3. Hydraulic conductivity testing using 4 different permeameter types gave similar results. The most consistent results were found for tests using consolidation per-

- meameters, flexible-wall permeameters, and API tests. Less consistent results were found for compaction mold permeameters.
4. For all of the soil-bentonite mixtures, the measured hydraulic conductivity decreases with increasing vertical consolidation pressure. The effect is more pronounced for SB1 and SB2 than SB3. This may be due to the fact that SB1 and SB2 have a higher percentage of fines than SB3 and were found to be more compressible than SB3.
 5. The hydraulic conductivity for SB1 and SB2 was higher than the hydraulic conductivity for SB3 at all consolidation pressures. The percentage of bentonite appears to have a larger influence on hydraulic conductivity than fines content since SB1 and SB2 has 1% bentonite and 36% plastic fines and SB3 has 3% bentonite and 16% non-plastic fines.
 6. For 1-D consolidation tests on SB1, the coefficient of consolidation exhibits a significant increase as the vertical effective stress increases.
 7. The void ratio for SB1 and SB3 has a unique relationship with the logarithmic of mean stress for both isotropic and one dimensional consolidation.
 8. It is difficult to form specimens of soil-bentonite for testing in triaxial cells due to the high slump of the soil-bentonite. Using new procedures and slightly modified triaxial cell equipment, specimens could be formed in the triaxial cell to isotropic consolidation pressures as low as 5 psi. Area and membrane corrections were applied to all triaxial data. Membrane corrections for the consolidation phase were found to be significant for soil-bentonite mixtures if consolidation is performed in the triaxial cell.
 9. Isotropic CU tests performed on SB1 resulted in stress paths indicative of an unusually high pore pressure response. Isotropic CU tests on SB3 exhibited similar stress paths to SB1, but with slight strain softening behavior. The effective friction angle for SB1 was found to be 32 degrees from both CU and CD strength envelopes. The cohesion intercept is zero. The effective friction angle for SB3 was found to be 32 degrees from the CU strength envelope and 33 degrees from the CD strength envelopes. The cohesion intercept is zero.

10. A K_o value of 0.57 was measured for SB1 in a K_o triaxial test. Triaxial compression on the K_o consolidated sample exhibited strain softening.
11. SB1 exhibited contractive behavior in undrained shear and drained shear for samples with OCR values below 3.1 and dilatant behavior for samples with OCR values above 7.8.

8.4 Constitutive Modeling of Soil-Bentonite

Summary of Constitutive Modeling of Soil-Bentonite

A review of the literature indicates that a constitutive model, for engineering purposes, should be as simple as possible (Wood 1990; Wroth and Houlsby 1985), and it should be possible to find the model parameters from conventional soil tests (Duncan 1994). These were the guiding principals used to select a constitutive model for soil-bentonite. The following three existing constitutive models were evaluated for their ability to describe the behavior of soil-bentonite SB1: a nonlinear hyperbolic model, Cam Clay model, and Modified Cam Clay model. These models are commonly used and are relatively simple models. Two Modified Cam Clay type models were then evaluated to provide models that more accurately describe the behavior of soil-bentonite. These models are referred to as the R model and the RS model. These models are simple cases of a more complex existing model. The extensive laboratory testing program performed on soil-bentonite mixture SB1 provided the stress-strain data to evaluate the constitutive models. The model parameter values for SB1 were obtained for each of the models. The models were then evaluated to determine how well they match the laboratory test data. In the course of evaluating the models, the shape of the yield surface and the plastic potential surface was estimated for SB1. Of the constitutive models studied, the RS model gives the best representation of soil-bentonite mixtures in conventional laboratory tests. The RS model is a non-associative Modified Cam Clay type model that has parameters to change the yield surface and plastic potential surface into ellipses of varying shapes.

Conclusions from Constitutive Modeling of Soil-Bentonite

1. If the hyperbolic model parameter values for SB1 are found from CD tests, the model can accurately predict CD tests but not 1-D consolidation tests. If the hyperbolic model parameter values for SB1 are found from 1-D consolidation tests, the model can accurately predict 1-D consolidation tests but not CD tests. The hyperbolic model cannot accurately predict pore pressure generation and dissipation during coupled consolidation analysis (Duncan 1994).
2. The Cam Clay model and the Modified Cam Clay model can be used to predict both drained and undrained behavior with one set of parameter values. The Cam Clay model gives a good prediction of CD tests but over predicts the undrained strength of SB1. The Cam Clay model predicts a physically unreasonable stress-state for K_o consolidation conditions. The Modified Cam Clay model gives a fair prediction of CD tests but greatly over predicts the undrained shear strength. The Modified Cam Clay model gives a good prediction of 1-D consolidation and predicts a reasonable value for K_o .
3. The shape of the yield surface for SB1 was found from a series of CD tests performed on overconsolidated samples. The shape of the yield surface is elliptical and has a different shape to the left and the right of the critical state line. It is a different shape than the elliptical yield surface used in the Modified Cam Model. The shape of the plastic potential surface for SB1 was also found from the same series of CD tests. The shape of the plastic potential surface is elliptical and has a different shape than the yield surface.
4. A method was developed to estimate the Poisson's ratio of soil-bentonite from the results of CU tests and consolidation tests. The method assumes that the Poisson's ratio is constant and that the shear modulus is a function of mean effective stress. In Chapter 5 it was shown that the method worked well for estimating Poisson's ratio for soil-bentonite, but not for heavily overconsolidated clays.

5. The R model is an associative, Modified Cam Clay type model with an elliptical yield surface whose shape can be varied with the R parameter. The R model either gives a good prediction of the CU tests and a poor prediction of the CD test, or a good prediction of the CD tests and a poor prediction of the CU test, depending on the value of R that is used.
6. The RS model is a non-associative, Modified Cam Clay type model that has parameters to change the yield surface and plastic potential surface into ellipses of varying shapes. The R parameter changes the shape of the yield surface, and the S parameter changes the shape of the plastic potential surface. The R and S values can be estimated from CU tests and CD tests respectively. By varying R and S, an optimum pair was found for SB1 that provided a good prediction of CU tests, CD tests, 1-D and isotropic consolidation tests, and predicts a reasonable K_o value.
7. The RS model also provided a good estimate of CU and CD tests performed on SB3. Although the compressibility of SB1 and SB3 is different, the R and S values were found to be the same.

8.5 Finite Element Modeling of a Soil-Bentonite Cutoff Wall

Summary of Finite Element Modeling of a Soil-Bentonite Cutoff Wall

Previous examples of finite element modeling of soil-bentonite cutoff walls were presented and evaluated. The examples are limited in number and much simpler than the model developed for this research.

A finite element model of a soil-bentonite cutoff wall was developed for this research to simulate all stages of construction including excavation of the trench under bentonite-water slurry, replacement of the bentonite-water slurry with soil-bentonite backfill, and consolidation of the soil-bentonite backfill. The Raytheon case history described in Chapter 5 was simulated. The finite element program SAGE (Bentler et al. 1998), developed at Virginia Tech, was used for the analyses. Fully coupled fluid flow and

deformation analyses were used. The RS constitutive model was implemented into SAGE and was used to represent soil-bentonite.

Half of the soil-bentonite trench and the area contained by the cutoff wall was modeled with the finite element mesh. Three columns of elements were used to model the half-width of the soil-bentonite trench. The depth of the trench was 100 ft deep, and the half-width was 1.5 ft.

To model the site conditions, geotechnical data were compiled from laboratory tests, field tests, and in situ tests performed by others. From this data, a soil profile and material parameter values were interpreted. A soil profile consisting of layers of clay and sand was developed. The Duncan and Chang (1970) hyperbolic model was used to model the sand, and the Modified Cam Clay model (Roscoe and Burland 1968) was used to model the clay.

The excavation phase was modeled by removing the elements in the trench and applying stress distributions to the trench wall and trench bottom to represent the bentonite-water slurry pressure. The backfilling phase was modeled by placing rows of soil-bentonite starting from the bottom of the trench and removing the appropriate stress distributions representing the bentonite-water slurry. Much difficulty was encountered to achieve a reasonable stress-state in recently placed soil-bentonite elements. A reasonable initial stress-state for recently placed soil-bentonite was estimated from laboratory and in situ data. This initial stress-state was revised during calibration of the model. In order to achieve the desired stress-state in recently placed soil-bentonite elements, it was necessary to assign excess pore pressures in the elements with a “heavy” unit weight of water, use a very low Young’s Modulus for the soil-bentonite during placement, and place only one row of soil-bentonite at a time. After backfilling, the initial excess pore pressures in the soil-bentonite were allowed to dissipate over time by modeling consolidation of the soil-bentonite

The model was calibrated using lateral movements from inclinometers and vertical movements from settlement points. Each construction phase was calibrated using the measured incremental movements. Before calibration, the model over predicted the deformations for each phase. During calibration some material properties were changed in order to match the predicted deformation to the measured deformations. The following changes were made during calibration: the assumed initial effective stress of the soil-bentonite was increased, the R_f value for the sand was increased, the K_{ur} value for the sand was increased, and the Poisson's ratio value of the clay was decreased.

Good agreement was achieved between predicted and measured deformations in adjacent ground after calibration. The predicted and measured lateral deformations were within 1/4 inch for all phases of construction. The predicted settlements were within 1/4 inch to 1/2 inch of the observed settlements during excavation and backfilling. The predicted settlements were within 1/2 inch of most of the settlement points.

Conclusions from Finite Element Modeling of a Soil-Bentonite Cutoff Wall

1. Previous examples of finite element modeling of soil-bentonite cutoff walls indicate that simply changing material properties does not model the excavation and backfilling phases accurately. The changes in stresses that result due to excavation and backfilling must be modeled by removing the elements and backfilling with new elements. It is important to assume a realistic initial stress condition for the soil-bentonite first placed in the trench. In addition, the sides of the trench must be allowed to move in order to accurately model the consolidation of the soil-bentonite.
2. The permeability of the filter cake was modeled by "smearing" the hydraulic conductivity of the filter cake into the element of soil adjacent to the trench wall. The hydraulic conductivity of the adjacent element was reduced to reflect the lower hydraulic conductivity of the filter cake. This procedure helped with convergence of the finite element model.

3. A limitation of using the computer program SAGE was that for coupled fluid flow and deformation analysis, saturation of the soils is necessary. To accurately model the consolidation of soil-bentonite using SAGE, it was necessary to assume that negative pore pressures existed in the native ground and the soil-bentonite above the water table. This assumption increases the effective stress above the water table and may unrealistically increase settlement predictions in some circumstances. For the Raytheon case history, the assumption may be reasonable due to capillary rise in the existing silty sand and clay soils.
4. A limitation of using the computer program SAGE was that interface elements did not work properly for excavation and placement of interface elements during coupled fluid flow and deformation analyses. Since interface elements were not available, thin 2-D elements of soil-bentonite were used next to the trench wall. These elements did not perform well after failure at some of the gauss points, as discussed in the next section.
5. The initial effective vertical stress in recently placed soil-bentonite was first estimated to be 40 psf. This was estimated based on the initial water content of the soil-bentonite and stress-strain relationships from 1-D consolidation tests. During calibration of the model, this stress was found to be too low, and produced movements during backfilling that were too large. An initial effective vertical stress of 100 psf was found to produce deformations during backfilling that matched measured deformations.
6. It was found that recently placed elements of soil-bentonite would have initial negative effective stresses using the assumptions used by SAGE for fill placement. It is thought that the Young's modulus assumed by SAGE for fill placement was too high. This resulted in shear stresses in the elements that were too high, and negative vertical and horizontal effective stresses. SAGE was modified to allow control of the material model and material properties during fill placement. When an elastic model with a very low Young's modulus was used for soil-bentonite elements during fill placement, the desired initial stress-state in the soil-bentonite was achieved.

7. In order to achieve the desired stress-state in recently placed soil-bentonite elements, it was necessary to assign excess pore pressures in the elements with a “heavy” unit weight of water. In SAGE, the initial stresses of recently placed elements are calculated from the finite element method using the unit weight of the fill and initial pore pressures. Initial pore pressures are specified with the use of a piezometric line. A height of the piezometric line was calculated to establish high pore pressures in recently placed elements that would give the desired initial effective stress at the center of the elements. A “heavy” unit weight of water equal to 117.5 pcf was assigned to the piezometric line. The unit weight of water was just slightly below the unit weight of the soil-bentonite (118 pcf). This was used to achieve essentially the same vertical effective stress throughout the depth of a recently placed element.
8. In order to achieve the desired stress-state in recently placed soil-bentonite elements, it was necessary to place only one row of soil-bentonite at a time. It was found that a maximum height to width ratio of 5:1 was necessary to achieve the desired stresses consistently across the row and for each subsequently placed row. If a larger height to width ratio was used, the effective stress in the soil-bentonite elements was too low or negative, and was not constant for each row.

The following trends were found during the model calibration process. These trends were not used in calibrating the model, but they are provided here for completeness. These trends may be specific to modeling the case history.

1. Reducing the K_o value of the existing sands reduced the amount of lateral deformations during excavation, but had little effect on deformations during backfilling and consolidation.
2. Assuming essentially undrained conditions for the excavation and backfilling phases resulted in the same deformations as assuming that drainage occurs during these phases. At the Raytheon site, the time between excavation and the end of backfilling was 42 days at the analyzed section. The undrained conditions were simulated by

using a very short time between excavation and backfilling of less than 0.3 days. There may have been no difference between these two cases because of the clayey nature of the Raytheon site and the low permeability of the native soils.

8.6 Parametric Study using Finite Elements

Summary of Parametric Study using Finite Elements

The finite element model described in the previous section was used to perform a parametric study. Various elements of the model were changed to evaluate the influence of existing soil conditions, soil-bentonite properties, and trench configurations on the predicted deformations. The same mesh and boundary conditions used for analysis of the Raytheon case history was used for the parametric study.

A base case analysis was run first assuming a normally consolidated, medium dense sand site. The results of the base case analysis were presented in detail and evaluated. The evaluation indicated that there is good confidence in the prediction of deformations in the adjacent ground, but there is lower confidence in the predicted stresses in the soil-bentonite and the settlement of the soil-bentonite in the trench. The predicted deformations in the adjacent ground compare well to the deformations predicted and measured for the Raytheon case history. As a result of failure at some of the gauss points in some soil-bentonite elements, the final stresses in the consolidated soil-bentonite may be too high, and the settlement of the soil-bentonite may be over predicted.

A parametric study was performed assuming uniform sand sites of varying density and OCR value. The R parameter, λ parameter, and κ parameter were varied for the soil-bentonite. The height of the water table and the depth of the trench was also varied. The effect of these changes on the deformations in adjacent ground was reported.

The factor of safety against trench stability was calculated for various cases. The maximum settlement after excavation and the maximum settlement after consolidation was

also reported. The factor of safety was plotted versus the normalized maximum settlements.

Conclusions from Parametric Study using Finite Elements

The following conclusions are from evaluation of the base case analysis:

1. As described in Chapter 6, it was necessary to assume that negative pore pressures existed above the water table to model the dissipation of pore pressures in the soil-bentonite. During evaluation of the base case analysis, it was found that the negative pore pressures resulted in negative horizontal stress increments due to excavation above the water table. The negative horizontal stress increments caused unreasonable movements at the trench wall after the excavation phase. To counteract this, a pressure was applied to the face of the excavation to remove the effect of the negative pore pressures on the total horizontal stress. This pressure was removed during backfill placement to restore equilibrium.
2. For analysis of the base case, which is a site with sandy existing soil, very little deformation occurred during the excavation phase, and most of the deformations occurred during backfilling and consolidation. For analysis of the Raytheon case history, which is a site with clayey existing soils, very little deformation occurred during the excavation and backfilling phases; most of the deformations occurred during the consolidation phase.
3. Approximately 3 ft of settlement was predicted at the centerline of the soil-bentonite trench, and approximately 1 inch was predicted at the native ground surface near the trench wall. The settlement of 3 ft is 3% of the trench depth, which is within the range of reported values in the literature. There are 3 columns of elements representing the half-width of the soil-bentonite trench. Mostly uniform settlement of the soil-bentonite was predicted in the trench and most of the differential settlement between the soil-bentonite and the existing sand occurred in the thin soil-bentonite element next to the trench wall.

4. The soil-bentonite elements were examined after consolidation for evidence of failure. Although the average stresses of the elements indicated that no elements were at failure, it was found that in the column of elements nearest the trench wall, typically 2 of the 4 gauss points in each element were at failure. The gauss points at failure were located closest to the trench wall. At these gauss points it was found that the shear stress was very low. The failure occurred during the backfilling phase, at low shear stresses. After failure, the mean stresses and shear stresses increase greatly at non-failed gauss points due to consolidation of the soil-bentonite; however, the mean stresses and shear stresses do not increase significantly at the failed gauss points. A constitutive model and numerical implementation that are capable of increasing shear stress with increased mean stresses after failure are required. It is thought that if interface elements had been available for use in the finite element model, the stresses could have been modeled more accurately. As described in Section 6.6, problems were encountered with interface elements, and their use had to be abandoned.
5. Due to failure at some gauss points in some of the soil-bentonite elements, it is thought that the shear stresses are too low in the soil-bentonite. Since the shear stresses were too low, they were not able to transmit enough load to the adjacent sand, and the vertical effective stresses predicted in the soil-bentonite are too high. In addition, predicted settlements in the trench may be too high due to excessive deformations near the failed gauss points.
6. The predicted vertical effective stress with depth in the first 2 columns of soil-bentonite was found to be similar. The predicted vertical effective stress with depth in the third column was found to be less than the other 2 columns. The predicted vertical effective stress increases with depth for all columns. The predicted vertical effective stress are less than geostatic stresses and are greater than stresses predicted from lateral squeezing theory and arching theory. As stated above, the predicted vertical effective stress is thought to be over predicted.

7. The predicted horizontal effective stress with depth in the first 2 columns of soil-bentonite are similar. The predicted horizontal effective stress with depth in the third column was less than the other 2 columns and is thought to be too low. Total horizontal force equilibrium does not seem to be met for the third column. The column is very thin and the shear stresses in the element are not great enough to account for the large difference in horizontal stress from the second to the third column. Horizontal force equilibrium does seem to be met for the column of sand next to the trench and the first two columns of soil-bentonite. For the first two columns, the predicted horizontal effective stress are less than geostatic stress, reasonably close to the stresses predicted from lateral squeezing theory, and greater than the stresses predicting from arching theory.

The following conclusions are from the parametric study:

1. Plots of predicted settlement and horizontal deformation in the adjacent ground were generated for each case of the parametric study. The figures indicate that the magnitudes for maximum settlement and lateral deformation are similar for each case analyzed. The maximum predicted settlement typically occurs at the trench wall and decreases with distance from the trench. The maximum predicted lateral deformation is approximately constant and has its greatest magnitude over the 50 ft closest to the trench. The lateral deformation decreases at greater distances from the trench. The graphs indicate that most of the predicted deformations occurs within 200 ft from the trench centerline, and this region has the greatest differential settlement. Significant deformation (greater than 1/4 inch) is predicted up to distances of 100 ft to 290 ft from the trench centerline for 100 ft deep trenches.
2. The parametric study indicates the following trends: decreasing the depth of the trench decreases the deformations. Decreasing the density of the sand, increases the deformations. Increasing the OCR of the sand increases the deformations. Increasing the R value of soil-bentonite increases the deformations. Increasing the λ value of

- soil-bentonite increases the deformations. Lowering the water table decreases the deformations.
3. Changing the density of the sand has the greatest effect on the deformations compared to the base case analysis. The next greatest effect is caused by varying the depth of the trench, followed by depth of water, soil-bentonite λ value, sand OCR, and soil-bentonite R value. Changing the R value of soil-bentonite has very little effect on deformations in adjacent ground. Changing the OCR from 1 to 2 has greatest effect on the magnitude of deformations for loose sands and the smallest effect on the magnitude of deformations for dense sands. The percent change in the deformations is about the same for changing the OCR from 1 to 2 regardless of soil density.
 4. A trend was found between increasing normalized maximum settlement after consolidation and decreasing factor of safety against trench stability during excavation. The maximum settlement for various cases was normalized by the trench depth. Trend lines were estimated for various soil-bentonite λ values.

8.7 Practical Implications of Research

The most important implication of this research for the practicing engineer is that the research provides the engineer with information which can be used to estimate deformations in adjacent ground due to construction of a soil-bentonite cutoff wall. The research provides plots of estimated ground deformations with distance from the trench due to construction of a 3 ft wide soil-bentonite wall at sand sites for various site conditions.

The plots of estimated ground deformations are the final result of this research on the mechanical behavior of soil-bentonite cutoff walls which includes laboratory testing on soil-bentonite, constitutive modeling of soil-bentonite, finite element modeling of construction of a soil-bentonite cutoff wall, and a parametric study using finite elements. Using Figures 7.12 through 7.19, the engineer can estimate deformations in adjacent ground at various distances from the trench. In addition, using Figure 7.20, the maximum

deformation of the adjacent ground, which can be assumed to occur at the trench wall, can be estimated from a correlation between the factor of safety of the trench during excavation, and the compressibility of the soil-bentonite. Again, these figures apply for construction of a 3 ft wide soil-bentonite cutoff wall. Construction of a wider cutoff wall will most likely induce greater ground deformations.

8.8 Recommendations for Further Research

Recommendations to increase the understanding of the mechanical behavior of soil-bentonite cutoff walls are suggested below:

1. Case history data is very limited. It is recommended that more soil-bentonite cutoff walls are instrumented for lateral deformations and settlement in adjacent ground as well as settlement of the soil-bentonite in the trench. It is recommended that soil-bentonite cutoff walls be instrumented for stresses in the soil-bentonite backfill.
2. The finite element model described in Chapter 6 can be modified to analyze a section including the native ground outside the area enclosed by the cutoff wall as well as the native ground inside the area enclosed by the cutoff wall. It is anticipated that a much larger mesh is required than the mesh described in Chapter 6. Analyses could be done using fully coupled fluid flow and deformation analyses to model draw down of the water table inside the area enclosed by the cutoff wall. In the model described in Chapter 6, the draw down was assumed to occur on both sides of the cutoff wall. The results could be compared to the analysis described in Chapter 6 to estimate the effect of seepage across the cutoff and the differential water pressure. If the effect is significant, the new model can be used to evaluate the effect of varying the draw down on adjacent ground deformations.
3. The results of the finite element analyses and parametric study were not able to provide accurate predictions of the final stress state in the soil-bentonite or the settlement of the soil-bentonite in the trench. It is thought that use of an interface element at the trench wall would produce more accurate results, provided that the constitutive

model for the interface element is capable increasing shear stresses with increased mean stresses after failure. It is recommended these numerical analyses be performed using fully coupled fluid flow and deformation analyses using a finite element program other than SAGE. The interface elements in SAGE did not perform well during coupled analysis during fill placement. The finite element program should be capable of modeling all phases of construction similar to the finite element model described in Chapter 6. The interface elements should be excavated and backfilled with the soil-bentonite. It is important to achieve a reasonable initial stress state in the interface elements that is consistent with the initial stresses in the soil-bentonite.

4. The parametric study described in Chapter 7 can be expanded to include the influence of unit weight of bentonite-water slurry, height of bentonite-water slurry in the trench, draw down of water table, and influence of surcharges and buildings adjacent to the trench on the deformations in adjacent ground.