

# 1 Introduction

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## 1.1 Background

Soil-Structure Interaction (SSI) analyses have proven to be powerful tools for analyzing, designing, and monitoring geotechnical structures. A substantial amount of the geotechnical literature of the last 30 years has dealt with the development or improvement of techniques for SSI analyses of retaining walls, piles, anchors, etc. These analyses may be performed to address key issues concerning the behavior of structures in the design stage, and often provide a means for evaluation of instrumentation data from completed structures.

SSI analyses are particularly useful in problems with complex geometry and loading conditions such as lock walls. In these cases, simple analyses are not adequate to characterize the behavior of the soil-structure system. Factors such as the placement of the backfill, filling of the lock with water, changes in the water table elevation behind the wall, temperature fluctuations, etc., play an important role in the behavior of the structure.

The first clear evidence of the importance of these factors was provided by the analyses of the Port Allen and Old River locks performed by Clough and Duncan (1969). Extensive instrumentation data suggested deformation patterns of the locks that seemed unreasonable and were thought to be the product of instrumentation errors. Clough and Duncan (1969, 1971) showed that close modeling of the construction stages of the lock and the use of a simple but adequate constitutive model for the soil and the soil-to-structure interface yielded results in close agreement with the measured data. For their analyses they used the hyperbolic model for soils proposed by Duncan and Chang (1970) following previous work by Kondner (1963) and Kondner and Zelasko (1963). The adequacy of this simple stress-strain model for use in SSI analyses is discussed in Section 4.1 of Ebeling, Peters, and Mosher (1997). This model has been extended to interface behavior, as described by Clough and Duncan (1971).

Several important contributions followed the pioneering work of Clough and Duncan. Studies of the Red River Lock and Dam No.1 (Ebeling et al. 1993; Ebeling and Mosher 1996; and Ebeling, Peters, and Mosher, 1997), the North

Lock Wall at McAlpine Locks (Ebeling and Wahl 1997), and Locks 27 (Ebeling, Pace, and Morrison, 1997) are good examples of state-of-the-art techniques available for SSI analyses. These studies showed that the behavior of the soil-structure interface has a significant influence on the magnitudes of the loads acting against the lock wall. They also illustrated that the pre- and post-construction stress paths followed by interface elements are complex, often involving simultaneous changes in normal and shear stresses, as well as shear stress reversals due to post-construction rise of the ground water level.

The hyperbolic interface model developed by Clough and Duncan (1971) is a very useful tool for SSI analyses. It is easy to implement, and the parameters involved in the hyperbolic fit to laboratory data have a physical meaning. Although it models interface behavior in the primary loading stage very closely, it has not been extended to accurately model simultaneous changes in shear and normal stresses, reduction of shear stress, reversals in the direction of shear, or unload-reload cycles at the interface.

The purpose of the research described in this report is the development of an improved numerical model for soil-structure interfaces that can handle a variety of stress paths. A series of interface tests were performed in the laboratory to collect data on interface response to different types of loading. An extended hyperbolic model was developed based on the test results and implemented in the finite element program SOILSTRUCT-ALPHA, which is commonly used by the U.S. Army Corps of Engineers for analysis of lock walls.

A pilot-scale lock wall simulation was carried out in the Instrumented Retaining Wall (IRW) facility, where vertical and horizontal forces exerted by the backfill on the wall were measured. Finite element calculations of the IRW simulation were performed using the updated version of SOILSTRUCT-ALPHA, which contains the formulation of the extended hyperbolic model. Comparisons between the finite element calculations and the IRW test data suggest that the extended hyperbolic model may provide accurate predictions of the magnitudes of vertical shear forces at lock wall-backfill interfaces.

This dissertation contains the results of all the interface tests performed for this investigation and the final version of the extended hyperbolic model. The procedure for the implementation of the model in SOILSTRUCT-ALPHA is also described. In addition, the results of the lock wall simulation are presented together with a critical evaluation of the accuracy of the model and suggestions for future work on interface modeling.

## 1.2 Interface Behavior in SSI Analyses

Based on SSI analyses of four hypothetical earth retaining structures, Ebeling, Duncan, and Clough (1990) concluded that the interface shear stiffness has a significant influence on the distribution of forces on the structure. They performed two different analyses of the same structure using the expected maximum and minimum values of shear stiffness of the backfill-to-structure interface. A difference of 12.5 percent was found between the values of friction angle mobilized at the base of the structure for the two analyses. Further evidence of the importance of interface behavior in SSI analyses has been provided in a study of the North Lock wall at McAlpine Locks.

### 1.2.1 The North Lock Wall at McAlpine Locks

The North Lock wall at McAlpine Locks (Ebeling and Wahl 1997) has been designed as a monolith of roller-compacted concrete (RCC) directly founded on rock as illustrated in Figure 1-1. The lock wall will be 22.25 m (73 ft) high with a dense granular backfill extending from the new RCC lock to the existing lock wall as shown in the figure. The analyses were performed using the finite element program SOILSTRUCT-ALPHA (Ebeling, Duncan, and Clough 1990), which is an updated version of SOILSTRUCT (Clough and Duncan 1969) that allows for separation between the base of the wall and the foundation.

The analyses performed for the North Lock wall were completed in three phases: 1) gravity turn-on analysis prior to construction of the new lock, 2) incremental construction of the new lock and concurrent backfill placement, and 3) post-construction submergence of the backfill and flooding of the lock chamber. The lock-to-backfill, lock-to-rock, and backfill-to-rock interfaces were modeled using Goodman, Taylor, and Brekke (1968) one-dimensional interface elements. The hyperbolic models proposed by Duncan and Chang (1970) and Clough and Duncan (1971) were used for the backfill and the interfaces, respectively.

Table 1-1 shows the results of the analyses in terms of the effective horizontal and vertical forces  $F_x'$  and  $F_v$  per unit length acting on Section A-A represented in Figure 1-1<sup>1</sup>. The vertical force  $F_v$  is caused by downdrag on Section A-A produced by settlement of the backfill.

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<sup>1</sup> For convenience, symbols are listed and defined in the Notation, Appendix F

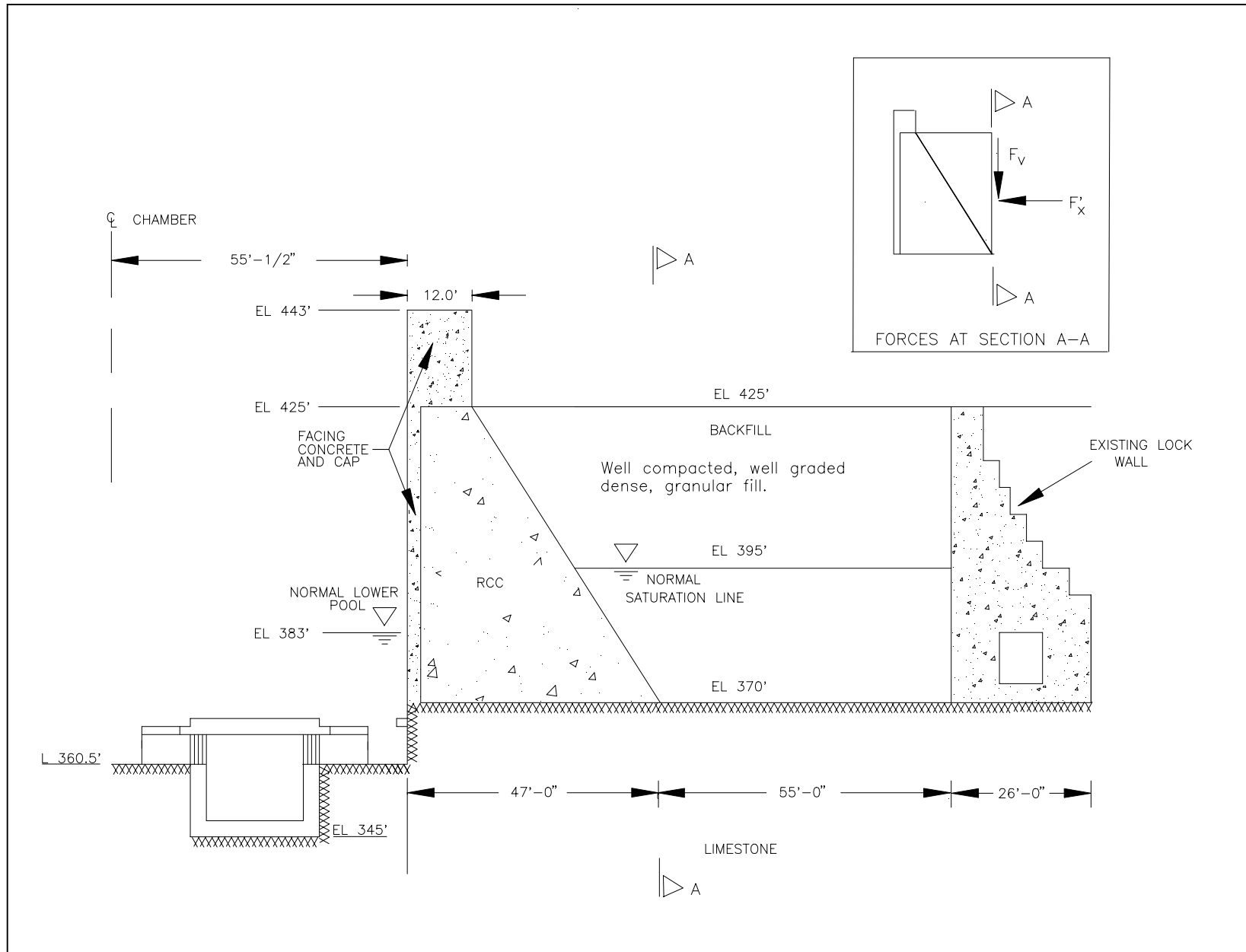


Figure 1-1. Typical section, north wall of new RCC McAlpine Lock (adapted from Ebeling and Wahl, 1997)  
(1ft = 0.305 m)

<b>Table 1-1 Summary of Results of SSI Analyses for the North Lock at McAlpine Locks (adapted from Ebeling and Wahl, 1997)</b>					
<b>Condition</b>	<b>Effective Overburden, KN per m Run of Wall</b>	<b>F<sub>x</sub>' , kN per m Run of Wall</b>	<b>F<sub>v</sub>, kN per m Run of Wall</b>	<b>K<sub>h</sub></b>	<b>K<sub>v</sub></b>
After backfilling	2,782	1,217	167	0.437	0.060
After submergence	2,515	1,155	99	0.459	0.039

The earth pressure coefficients  $K_h$  and  $K_v$ , also shown in Table 1-1, are a convenient way to quantify the effective horizontal and vertical forces acting on Section A-A. They represent the ratio of these forces to the *effective overburden* and are calculated as follows (Ebeling and Wahl 1997):

$$Effective\ Overburden = \int_{EI370}^{EI425} \sigma'_v dy \quad (1-1)$$

$$K_h = \frac{F'_x}{Effective\ Overburden} \quad (1-2)$$

$$K_v = \frac{F_v}{Effective\ Overburden} \quad (1-3)$$

where  $\sigma'_v$  is the effective vertical stress acting along section A-A.

The earth force coefficients  $K_h$  and  $K_v$  are useful for consistent comparison of results of analyses at different operational stages of the lock wall and even between retaining walls of different geometry and loading conditions. The vertical earth force coefficient  $K_v$  is also used in a simplified procedure to estimate the downdrag on retaining walls founded on rock. This simplified procedure (Appendix F in Engineering Circular (EC) 1110-2-291) is described in detail in Chapter 2.

The results in Table 1-1 show that after backfilling and prior to inundation the magnitude of the downdrag on the wall is substantial and amounts to 6 percent of the effective overburden. The mechanism for generation of this downdrag during backfilling is illustrated in Figure 1-2a. The initial horizontal position of the surface of a compacted lift is shown. As the backfill placement progresses, the weight of the newly placed and compacted lifts compresses the underlying backfill. The relative movement between the compressing backfill and the wall generates shear stresses at the backfill-to-structure interface. The final configuration of the lift illustrates the nonuniform compression of the backfill due to the restraint imposed by the interface shear stresses.

Filz (1992) and Filz and Duncan (1997) showed that the distribution of the backfill-to-structure interface shear stresses is not uniform along the height of

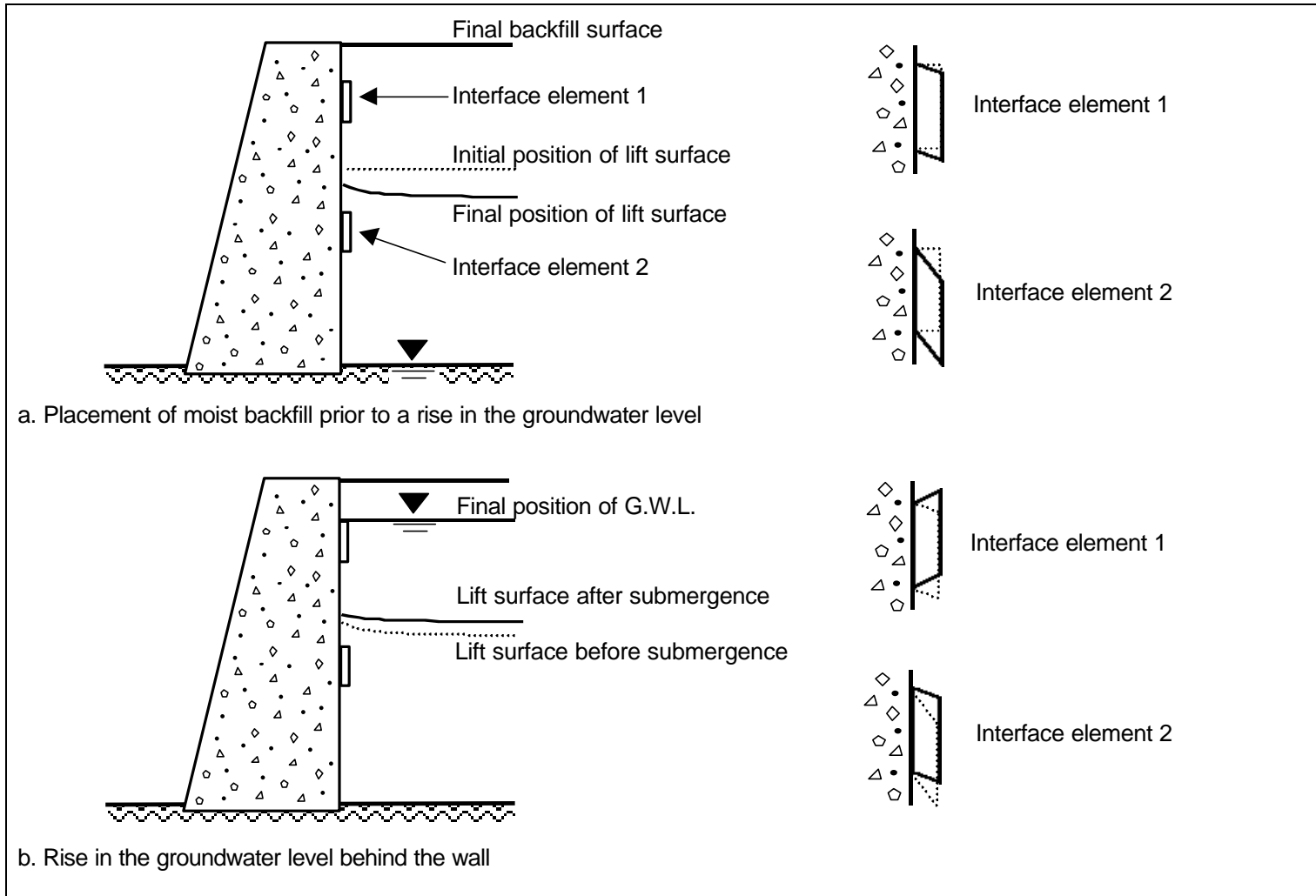


Figure 1-2. Simplified illustration of the mechanism of downdrag and shear reversal in a typical lock wall

the wall. For walls founded on relatively incompressible materials, such as the rock-founded North Lock wall, there is no settlement at the bottom of the backfill due to the absence of underlying compressible material. The top of the backfill does not settle if no further loads are applied after completion of backfilling with soils that do not creep. The maximum interface shear stress occurs at some intermediate point between the top and bottom of the backfill.

Table 1-1 also shows the results of the analyses considering a post construction submergence of the backfill. There is a substantial reduction of the vertical shear force and the vertical earth force coefficient  $K_v$ , with respect to the analysis before submergence. The mechanism for this reduction of shear stresses is illustrated in Figure 1-2b. As the groundwater level in the backfill rises, there is a decrease in the effective stresses and, in the absence of hydrocompression, an upward movement of the backfill. As the backfill in contact with the wall rises, the shear stresses at the interface decrease.

The magnitude of the vertical shear forces acting on the back of the wall may have a significant impact on the stability of the structure. If these stabilizing forces are accounted for in the design of the structure, a more economical construction can be achieved. Reliable calculation of these forces requires an adequate constitutive model for the interface response to conditions such as those represented in Figure 1-2.

### 1.2.2 Limitations of existing interface models

Two interface elements are represented in the simplified lock wall scheme of Figure 1-2. The normal and shear stresses acting on these elements change simultaneously during backfill placement and subsequent submergence of the backfill. Figure 1-3a shows the typical field stress paths followed by these elements during placement of the backfill. Element 1, which is located close to the top of the backfill, follows the stress path *A-B*. Element 2, which is located at midheight of the backfill, is subjected to larger normal and shear stresses at the end of the backfill placement and may follow a stress path such as *A-C*. It also undergoes larger interface displacements than Element 1.

Figure 1-3a also illustrates the type of stress paths typically applied during laboratory testing, in which the interface is sheared to failure under constant normal stress. Although the Clough and Duncan (1971) hyperbolic model for interfaces works well for typical laboratory stress paths, the response of the interface to the actual field stress path may differ substantially from that predicted by the model.

Figure 1-3b shows the stress paths that may be followed by the two interface elements in Figure 1-2 during inundation of the backfill. Element 2 may follow

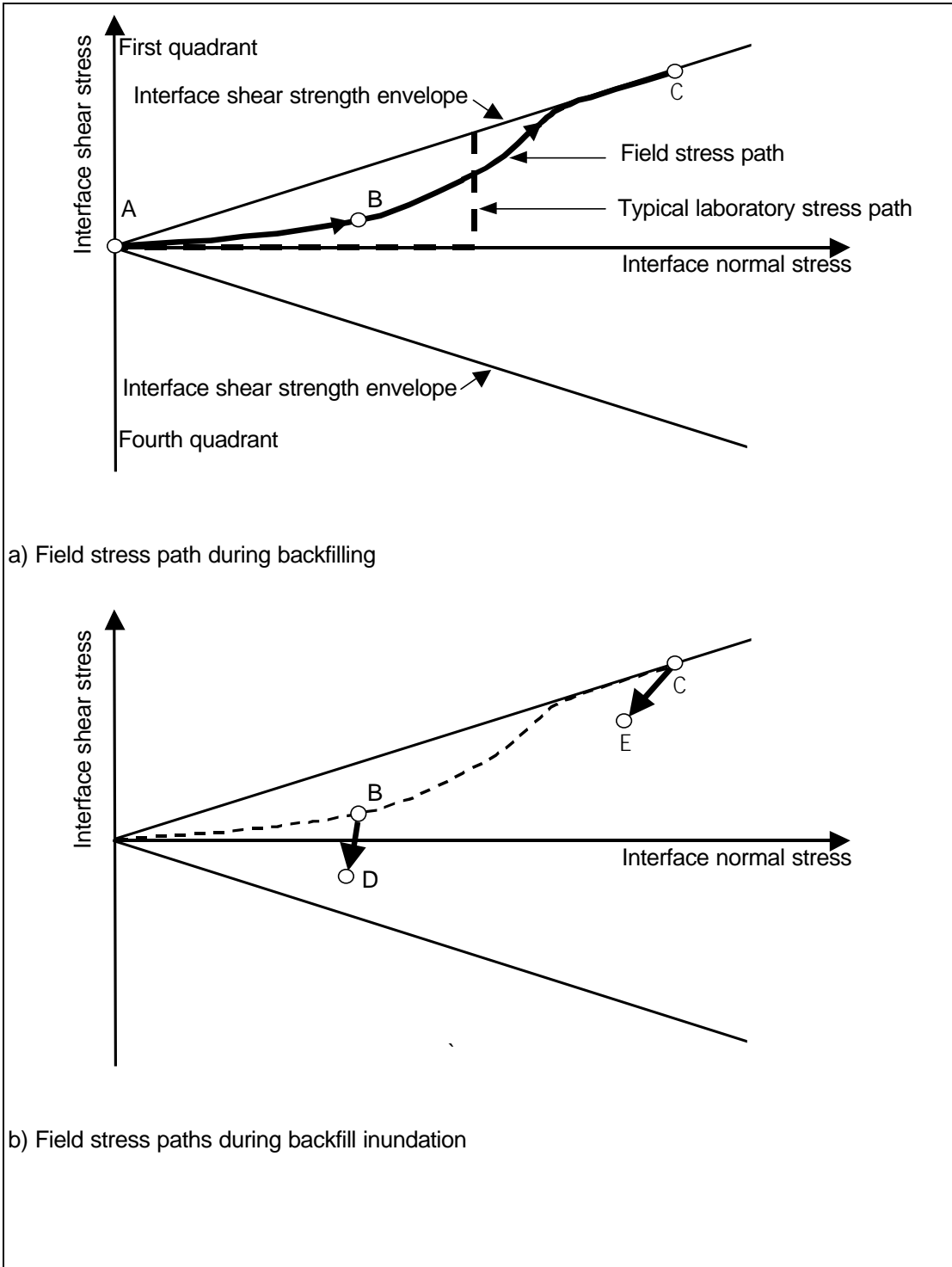


Figure 1-3. Types of loading expected on the interface between a lock wall and the backfill



the stress path *C-E*. The shear stresses acting on Element 2 decrease simultaneously with the normal stresses. The stress path for Element 1 is represented by line *B-D*. Element 1 undergoes a larger displacement along the interface because the thickness of the underlying backfill subject to rebound is larger. Due to this large displacement and the low initial shear stress acting on Element 1, inundation of the backfill may induce a reversal in the direction of shear. Further fluctuations in the water table behind the lock wall may induce unload-reload cycles on the interface.

Two different SSI analyses were performed for the North Lock wall at McAlpine Locks considering partial submergence of the backfill. Figure 1-4 shows the two different interface responses used for these analyses. In the stiff unload interface model, shown in Figure 1-4a for a constant effective normal stress, the shear stiffness during reversal is assumed equal to the initial shear stiffness. Changes in effective normal stress occurred in the interface elements as the water table rose in the backfill. This resulted in a change in the interface shear stiffness used during unloading of the interface elements. In the soft unload interface model, shown in Figure 1-4b for a constant effective normal stress, the primary loading curve is also used during unloading.

The results of the two SSI analyses are summarized in Table 1-2. These results show that the vertical shear forces acting on the wall after partial submergence of the backfill are significantly lower for the stiff interface response model during unloading than for the soft interface response model. The computed effective base pressure below the heel of the lock wall (results not shown) is lower for the SSI analysis using the stiffer interface model during post-construction, partial submergence of the backfill. This behavior is attributed to both the lower shear force and slightly larger horizontal earth pressure acting on the back of the lock wall for the SSI analysis with the stiff interface model.

<b>Table 1-2 Comparison of Results of SSI Analyses at Section A-A for Two Different Models of Interface Response to Unloading (adapted from Ebeling and Wahl, 1997)</b>					
<b>Condition</b>	<b>Effective Overburden, kN per m Run of Wall</b>	<b><math>F_x'</math>, kN per m Run of Wall</b>	<b><math>F_v</math>, kN per m Run of Wall</b>	<b><math>K_h</math></b>	<b><math>K_v</math></b>
"Stiff" model	2,515	1,155	99	0.459	0.039
"Soft" model	2,515	1,131	130	0.450	0.052

Only very limited published information is available concerning interface response to shear stress reversals, unload and reload cycles, or simultaneous change in shear and normal stresses. One of the most significant works on the response of concrete-to-soil interfaces was performed by Peterson et al. (1976). Their results, along with those reported in Clough and Duncan (1969), have been the main source of information on interface response for SSI analyses of lock

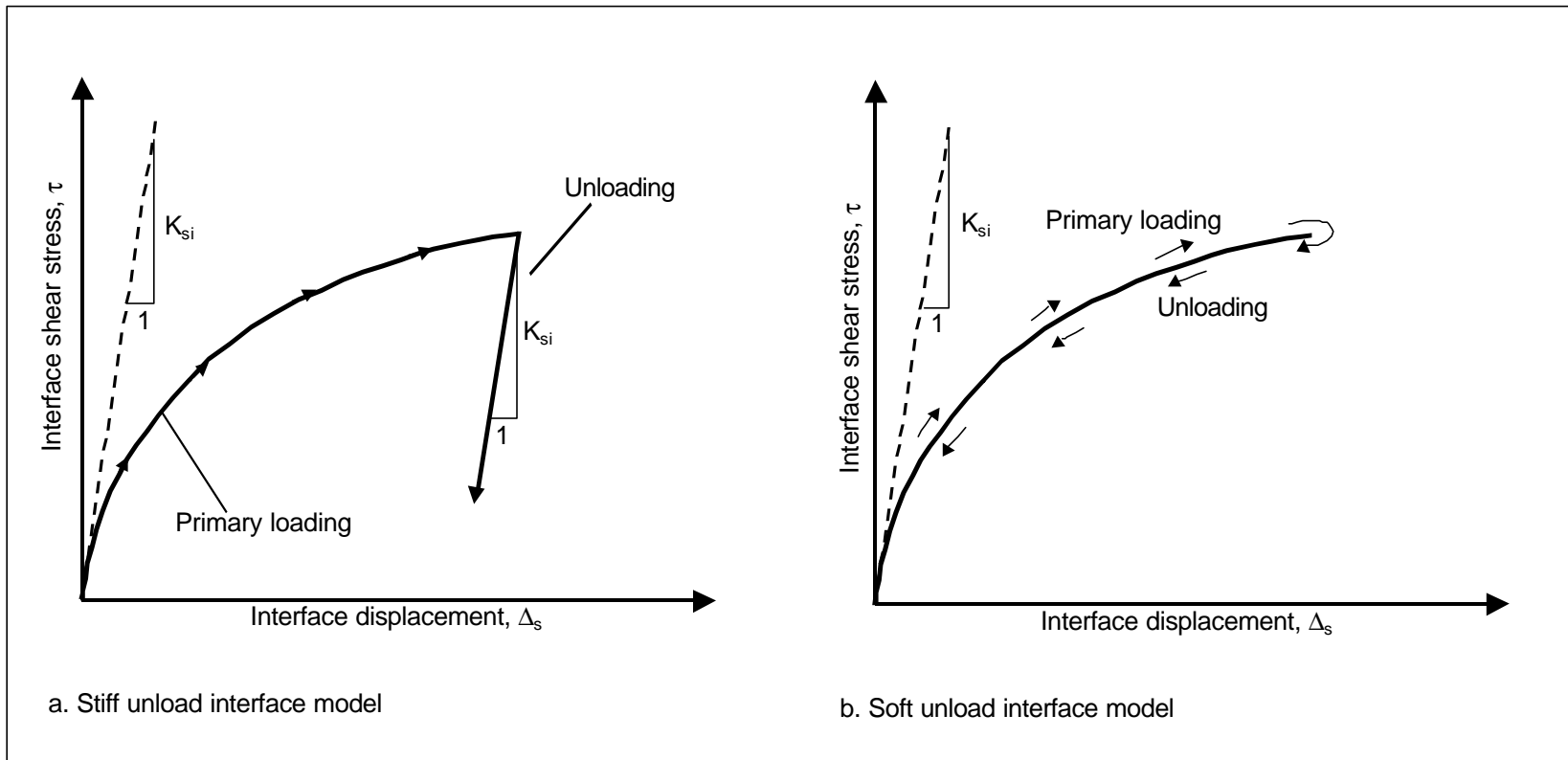


Figure 1-4. Two models for interface shear stress-displacement response under unloading (adapted from Ebeling and Wahl 1997)

walls. Peterson et al. (1976) used a 102- by 102-mm (4- by 4-in.) shear box to test combinations of different concrete surfaces and sands. The most important variables analyzed were roughness of the concrete surface and gradation and relative density of the sand. Their work has three main limitations: 1) the frequency of the reported data points collected during interface testing is in general not adequate to model the behavior during shear reversals, 2) no tests were performed to model simultaneous changes in normal and shear stresses, and 3) end effects due to the small dimensions of the shear box may have influenced the results, especially with respect to the initial stiffness during shear load reversals.

## 1.3 Project Scope

This investigation included laboratory testing of soil-to-concrete interfaces and development of an improved numerical model for interfaces. The model was implemented in the finite element program SOILSTRUCT-ALPHA and was validated against the results of a lock simulation test performed using the IRW at Virginia Tech.

### 1.3.1 Interface testing

In order to introduce improvements in existing interface models, data were collected from laboratory tests performed using stress paths such as those described previously. Figure 1-5a shows a stress path in which both shear and normal stresses change simultaneously. This stress path was designed to model the type of loading expected at the wall-backfill interface during backfill placement.

Figure 1-5b illustrates stress paths applied to study the interface response to unloading-reloading and reversal in the direction of shear. These stress paths model the type of loading expected at the wall-backfill interface during inundation of the backfill.

Figure 1-5c shows a stress path designed to probe the yield surface. The interface is sheared under constant normal stress to the state of stress represented by point  $S$ . Subsequently, the normal stress is increased to point  $S'$ , while a constant shear stress is maintained. Finally, the interface is sheared to failure. This type of loading is referred to as *staged shear* in this report.

The data collected from such tests allowed the development of the extended hyperbolic model for interfaces. A series of additional interface tests were performed that combined the types of loading illustrated in Figure 1-5 into

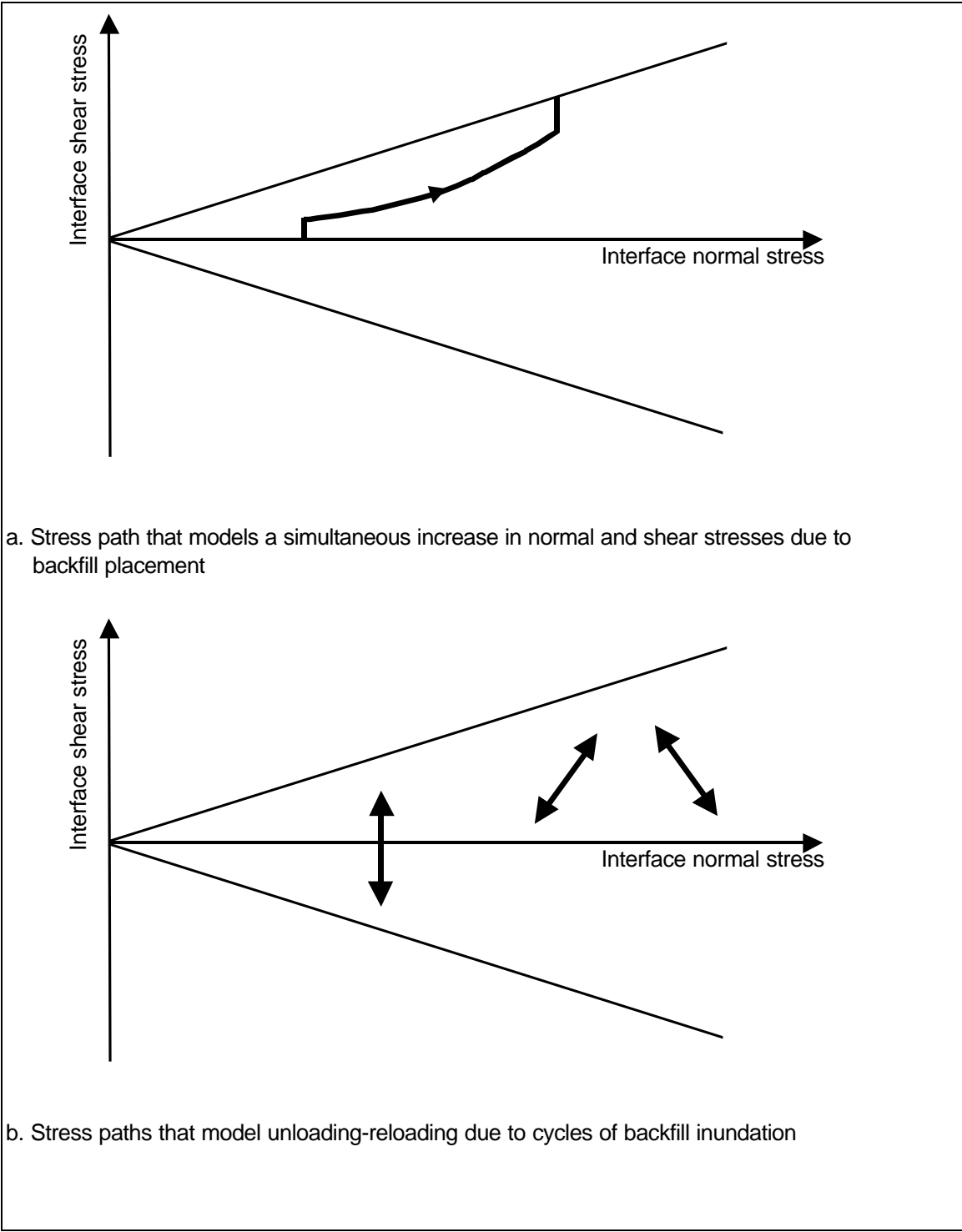


Figure 1-5. Laboratory stress paths to study the response of a wall-backfill interface (Continued)

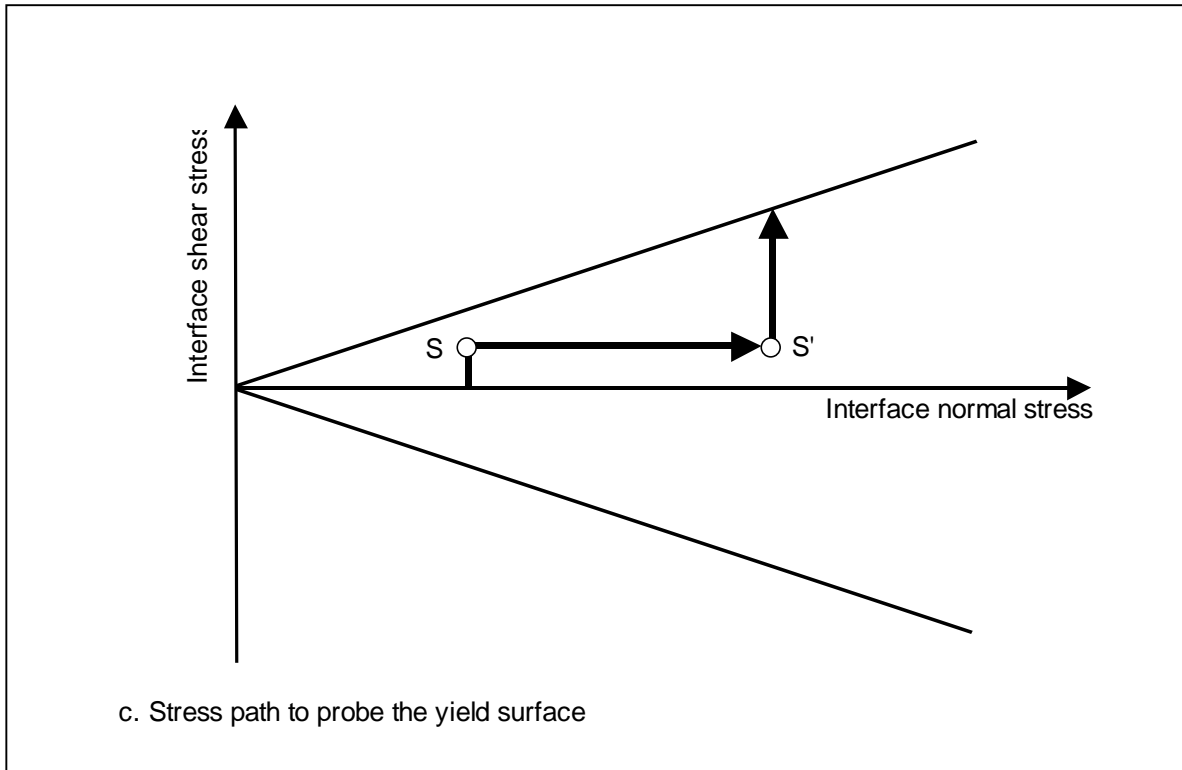


Figure 1-5. (Concluded)

complex stress paths. The results from these tests were the basis for the evaluation of the accuracy of the extended hyperbolic model.

Virginia Tech's Large Displacement Shear Box (LDSB) was used to perform the soil-to-concrete interface shear tests. The LDSB was used previously to test clay-geomembrane interfaces (Shallenberger and Filz 1996). Several modifications to the original configuration of the LDSB were necessary to permit the kind of soil-to-concrete interface testing necessary for this project. The tests were performed on three different sand-to-concrete interfaces. The details of specimen preparation, testing procedures, and results are described in this report.

### 1.3.2 Extended hyperbolic model for interfaces

A new, improved numerical model was developed using the results of the interface tests performed during this investigation. The new model is based on the hyperbolic formulation developed by Clough and Duncan (1971), which has been extended to model the type of loading illustrated in Figure 1-5.

The staged shear tests provided information about the evolution of the yield surface during interface shear. The model contains a formulation for interfaces undergoing yielding and a formulation for unloading-reloading and staged shear. For interfaces at yield, the extended hyperbolic model accounts for changes in the normal stress during shear, which can be applied to the type of loading illustrated in Figure 1-5a.

Three different versions of the model were developed for interfaces undergoing unloading-reloading. In Version I, a linear, normal stress-dependent interface response is assumed. Version II assumes a hyperbolic response that can model the hysteretic behavior of the interface. Version III provides a smooth transition between the reloading and yielding responses of the interface.

The model was evaluated against the results of the interface tests performed following complex stress paths. It was found that the model provides an accurate representation of interface response. Higher version numbers produce better response to complex stress paths.

### **1.3.3 Implementation of the model in SOILSTRUCT-ALPHA**

The finite element program SOILSTRUCT-ALPHA as described in Ebeling and Wahl (1997) and Ebeling, Pace, and Morrison (1997) is frequently used for SSI analyses of Corps of Engineers structures. The program is capable of modeling the different stages of construction and operation of a lock wall. The formulation for yield-inducing shear and Version II of the unload-reload formulation were implemented in the program. The updated version of SOILSTRUCT-ALPHA was used to model the lock wall simulation performed in the IRW.

### **1.3.4 Lock wall simulation**

A pilot-scale test was performed in Virginia Tech's IRW facility to simulate backfilling, surcharge application, and backfill inundation for a lock wall. The purpose of the test was to collect data on the behavior of the wall-backfill system during surcharge application and inundation, and to compare the vertical force measurements to calculations performed using the updated version of SOILSTRUCT-ALPHA. It was found that the vertical force, which develops at the wall-backfill interface during backfilling, may change substantially during surcharge application and backfill inundation.

Comparisons between the lock simulation data and SOILSTRUCT-ALPHA calculations suggest that the extended hyperbolic model may provide accurate predictions of interface response in SSI analyses of lock walls

## 1.4 Report Organization

The report is organized in four main sections. The literature review is presented in Chapter 2, which contains a description of previous work on interface testing, interface modeling, and SSI analyses.

Chapter 3 contains a complete description of the testing equipment, testing procedures, and results of laboratory tests performed on the soils and on the soil-to-concrete interfaces used for this investigation.

The formulation of the extended hyperbolic model is presented in Chapter 4. Relevant aspects of the interface response measured during the interface tests are discussed. The hypotheses leading to the formulation of the model are introduced. The derivation of the mathematical formulation of the extended hyperbolic model is described in detail, together with the recommended procedures for the determination and adjustment of the model parameters. The model is evaluated against the results of the interface tests described in Chapter 3. Finally, an outline of the implementation of the model in the program SOILSTRUCT-ALPHA is presented.

The lock wall simulation performed in the IRW is described in Chapter 5. Details are given regarding preparation of the facility, testing procedures, and results of the test. The finite element analyses of the IRW using SOILSTRUCT-ALPHA are presented, and the measured and calculated magnitudes of the forces acting on the wall are compared.

Chapter 6 summarizes the results and conclusions of this investigation, and includes recommendations for future work on interface modeling, and SSI analyses of lock walls.

The report also includes six appendices. Appendix A contains the results of tests performed on the soils used for interface testing. Appendix B describes the determination of the hyperbolic parameter values for the soils tested. The results of the interface tests are presented in Appendix C. The determination of the hyperbolic parameter values for the interfaces is described in Appendix D. Appendix E contains some example calculations that serve to illustrate the practical application of the extended hyperbolic model. Finally, all the symbols and abbreviations used in this report are defined in Appendix F.

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