3 Laboratory Testing

A series of shear tests were performed on soil-to-concrete interfaces using the Large Displacement Shear Box (LDSB). The objective of the tests was to compile experimental data for development of the new interface model described in Chapter 4.

Before interface testing, a soil box was designed and fabricated and the LDSB was modified to accommodate soil-to-concrete interfaces. A concrete specimen was prepared with a representative surface texture, according to the results of a survey of existing concrete walls.

Two types of sands were used for the interface tests performed for this investigation: Density Sand and Light Castle Sand. Laboratory testing of these soils included index property tests such as gradation, minimum/maximum density, and specific gravity, as well as triaxial compression and consolidation tests to determine their mechanical properties.

This chapter is divided into five sections:

1. *Soil properties*. Describes the gradation, mechanical properties, and hyperbolic parameter values of each of the soils used for interface testing.

2. *Concrete specimen*. Summarizes the preparation procedures and properties of the concrete specimen used for interface testing.

3. *Interface testing procedures*. Describes the equipment and procedures for testing the soil-to-concrete interfaces.

4. *Interface testing program*. Lists and describes the types of interface tests performed.

5. *Results of interface tests*. Summarizes the results obtained from the interface tests. These results formed the basis for development of the extended hyperbolic model.

3.1 Soil Properties

Two different sands were used for interface testing:

1) *Density Sand*. It is a fine-to-medium silica sand with subrounded to rounded grains, available commercially for in situ density determinations.

2) *Light Castle Sand*. It is a fine-to-medium sand with subangular to angular grains. This sand was used previously by Sehn and Duncan (1990) and Filz (1992) for testing at the IRW.

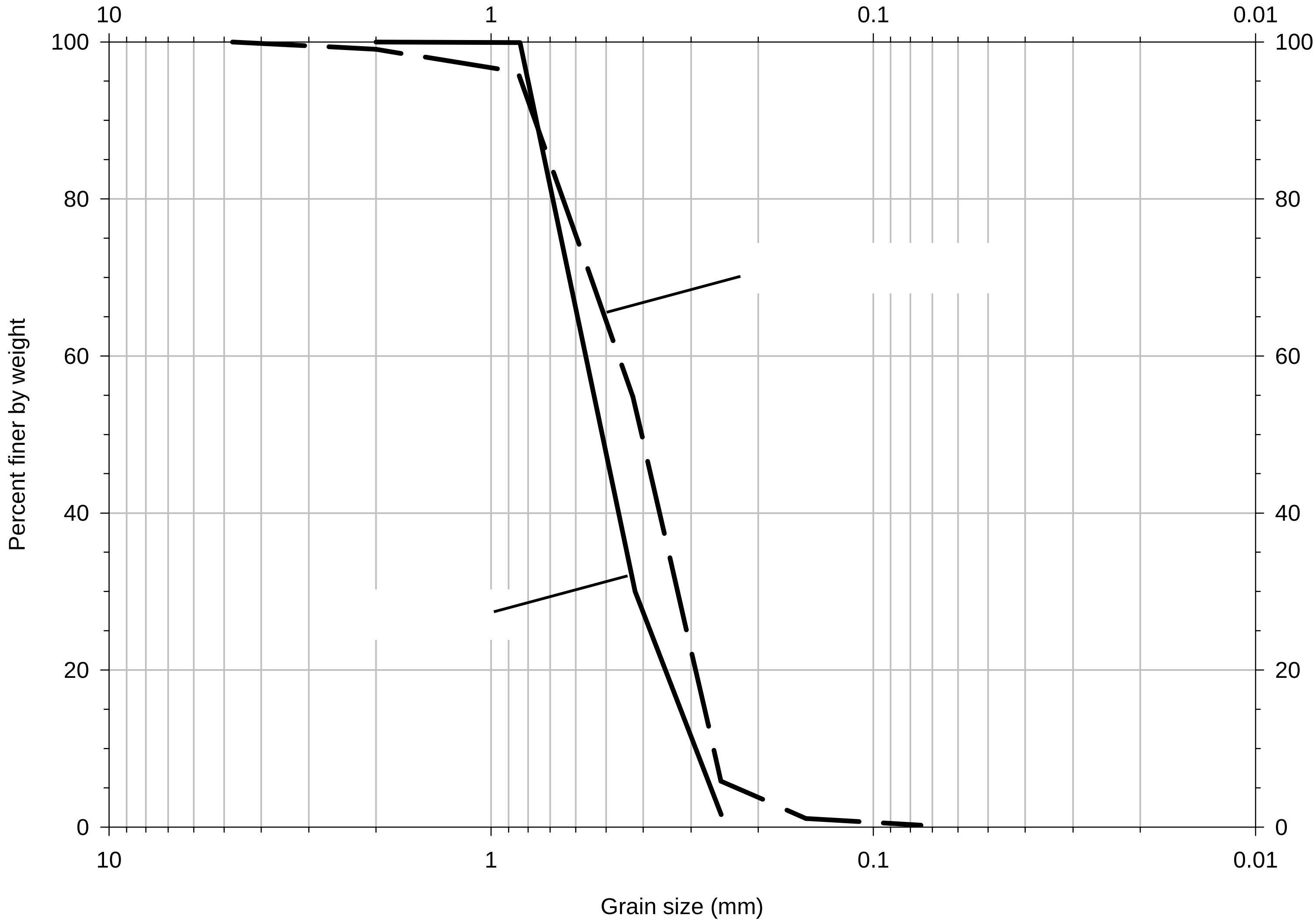
Table 3-1 summarizes the results of a series of laboratory tests performed to determine the gradation, maximum/minimum density, and specific gravity of the Density Sand and Light Castle Sand. The grain size distribution of these soils is presented in Figure 3-1. The Density Sand is uniform, with grain sizes ranging from 0.2 to 0.9 mm, and has no fines. The Light Castle Sand is also uniform, with grain sizes ranging from 0.1 to 2 mm, and has a negligible fraction of fines. Both soils classify as poorly graded sand SP (ASTM 1993a).

Examination of the Density Sand under an optical microscope revealed that the grains are subrounded to rounded (ASTM 1993b), with a length-to-width ratio typically ranging from 1 to 2, as shown in Figure 3-2a. The Light Castle Sand has angular to subangular grains with a length-to-width ratio ranging from 1.2 to 1.9, as shown in Figure 3-2b.

Table 3-1
Characteristics of the Soils used for Interface Testing

Parameter[1]	Value		Relevant Standard
	Density Sand	Light Castle Sand	
D_{10}, mm D_{30}, mm D_{60}, mm C_u C_c	0.3 0.42 0.55 1.8 1.1	0.25 0.32 0.45 1.8 0.9	D2487 (ASTM 1993a)
γ_{max}, kN/m^3	17.5	16.7	D4253 (ASTM 1993c)
γ_{min}, kN/m^3	15.1	13.7	D4254 (ASTM 1991)
G_s	2.65	2.66	D854 (ASTM 1992)

[1] Parameters are listed and defined in the Notation (Appendix F)


10
1
0.1
0.01
100
80
60
40
20
0
Percent finer by weight
Grain size (mm)

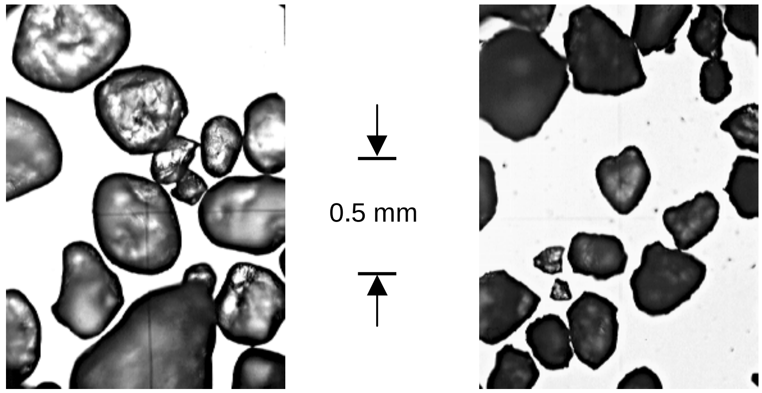


Figure 3-2. Microscopic view of the sands used for interface testing

3.1.1 Triaxial testing

Drained triaxial (CD) tests were performed to determine the internal friction angle and hyperbolic parameter values of the Density Sand and the Light Castle Sand for a range of relative densities. Sets of medium dense and dense specimens were prepared by pluviation for each type of sand. After preparation, each sample was subjected to an internal manometric pressure of -15 to -20 kPa, which was gradually removed during application of the cell pressure. The samples were de-aired using CO_2, inundated with de-aired distilled water, and backpressure saturated. The samples in each set were consolidated under effective confining pressures ranging from 45 to 280 kPa. These values are representative of the estimated values of confining pressure within the backfill of typical lock walls. Shearing was performed at a strain rate of 0.25 $^{\%}/_{min}$, which was found to be appropriate for pore pressure dissipation during previous trials.

The results of the tests are presented graphically in Figures A1 to A4 of Appendix A, and are summarized in Table 3-2. All the specimens exhibited dilation during shear and strain-softening after mobilization of the peak strength. The peak strength values measured during the tests defined curved strength envelopes. The peak friction angle value ϕ for a given confining pressure σ'_3 can be calculated from the following expression (Duncan et al., 1980):

$$\phi = \phi_o - \Delta\phi \cdot \log_{10}\left(\frac{\sigma'_3}{p_a}\right) \qquad (3\text{-}1)$$

where

ϕ_o = peak secant friction angle at a confining pressure of 101.4 kPa (1 atm)

$\Delta\phi$ = reduction in the peak friction angle value for a tenfold increase in σ'_3

Table 3-2
Summary of Results of CD Triaxial Tests

Parameter[1]	Density Sand		Light Castle Sand	
	Medium Dense (D_r = 41%)	Dense (D_r = 92%)	Medium Dense (D_r = 50%)	Dense (D_r = 80%)
ϕ_o, deg	35	43.8	37.2	42.4
$\Delta\phi$, deg	2.5	3.2	1.4	6
ϕ_{cv}, deg	32.4	34.7	33.2	36.5
Figure	A1	A2	A3	A4

[1] Parameters are listed and defined in the Notation (Appendix F)

The values of ϕ_o and $\Delta\phi$ for each of the soils tested are given in Table 3-2. The strength envelopes corresponding to the strength measured at a 15 percent strain are linear, and the corresponding friction angle ϕ_{cv} values are also given in Table 3-2.

3.1.2 Consolidation testing

A set of consolidation tests was performed on the Density Sand and Light Castle Sand to provide additional data on their mechanical properties and determine their susceptibility to hydrocompression. Two specimens of each type of sand were prepared at different relative densities in a dry condition. Each specimen was consolidated under a series of vertical stress increments. Once a predetermined stress was reached, the specimen was inundated. The ensuing hydrocompression, or vertical deformation induced by inundation, was measured. The results of these tests are presented in Figures A5 to A8 of Appendix A.

3.1.3 Hyperbolic parameters

Determination of the hyperbolic parameter values listed in Table 3-3 is described in detail in Appendix B. The triaxial and consolidation test data were used to obtain the parameter values for initial loading, according to the procedures described by Duncan et al. (1980). For the Light Castle Sand, the parameter values for unloading-reloading were determined based on the unload-reload data from the consolidation tests following the method outlined by Clough and Duncan (1969).

It is convenient to evaluate the accuracy of these parameter values by comparing them with values reported in the literature for soils of similar gradation. Table 3-3 shows ranges of values for the modulus exponent *n*, failure ratio R_f, bulk modulus exponent *m*, and the parameter $\Delta\phi$ reported by Duncan et al. (1980). The values of modulus number *K*, bulk modulus number K_b, and friction angle ϕ_o, presented in Table 3-3 for each of the soils, are compared in Figure 3-3 with typical values reported by Duncan et al. (1980). It can be observed that the hyperbolic parameter values of the soils tested during this investigation seem to be consistent with those reported in the literature.

Table 3-3
Hyperbolic Parameters Values of Soils used for Interface Testing

Hyperbolic Parameters[1]	Density Sand		Light Castle Sand		Reference Values for Soils of Similar Gradation (from Duncan et al., 1980)
	Medium Dense D_r = 41 %	Dense D_r = 92 %	Medium Dense D_r = 50 %	Dense D_r = 80 %	
K	780	1400	440	690	See Figure 3-3
n	0.62	0.63	0.48	0.79	0.26 - 0.79
R_f	0.852	0.850	0.880	0.813	0.77 - 0.97
K_b	530	915	290	660	See Figure 3-3
m	0.42	0.55	0.72	0.31	0.02 - 0.65
K_{ur}	-	-	880	1380	-
ϕ_o	35°	43.8°	37.2°	42.4°	See Figure 3-3
$\Delta\phi$	2.5°	3.2°	1.4°	6°	0 - 9°

[1] Hyperbolic parameters are listed and defined in the Notation (Appendix F) and Appendix B

Figure 3-3 allows the estimation of hyperbolic parameter values for relative densities different from those of the specimens tested. As discussed in Chapter 5, this figure was particularly useful for estimating the hyperbolic parameters of the Light Castle Sand backfill for the finite element analyses of the IRW test.

A comparison between the hyperbolic stress-strain relationships calculated using the parameters in Table 3-3 for each of the sands tested and the data from the triaxial tests is presented in Figures B4, B8, B12, and B16 of Appendix B. The hyperbolic model provides a good fit of the laboratory data, especially at the higher stress levels, but it does not model the post-peak strain-softening behavior or shear-induced dilation of the soil.

3.2 Concrete Specimen

A concrete slab was prepared for the soil-to-concrete interface tests, with dimensions 635 by 305 by 25.4 mm (25 by 16 by 1 in.). The main

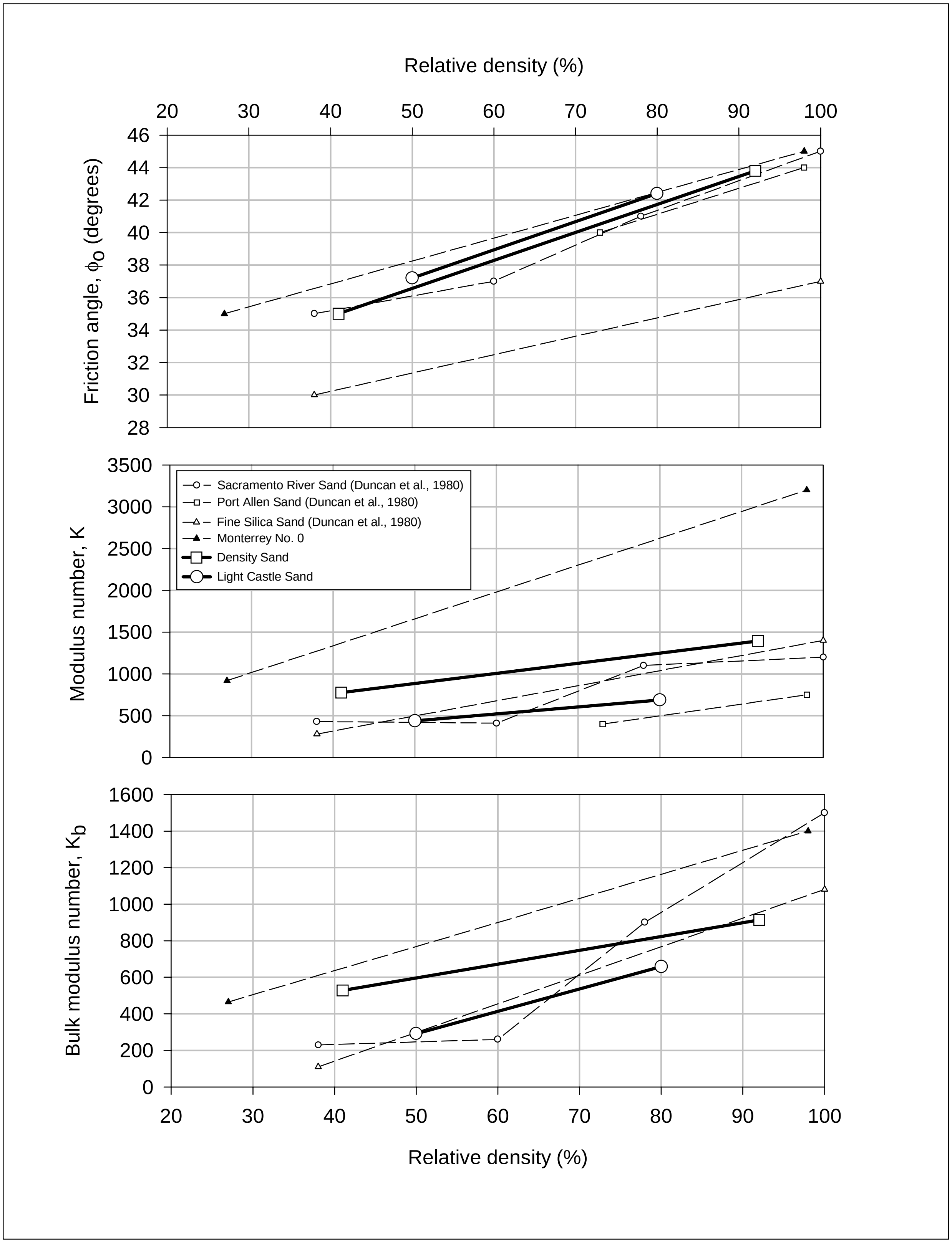


Figure 3-3. Hyperbolic parameter values for Density Sand and Light Castle Sand, and comparison to values reported by Duncan et al. (1980) for similar soils

considerations for the design of the specimen were: 1) to obtain a relatively high strength in order to minimize surface wear during shear, 2) create a surface texture representative of field conditions, and 3) minimize internal deformations of the concrete specimen during interface shear. To meet these requirements, several trials were required to develop the appropriate mixing and placement procedures for the concrete.

As illustrated in Figure 3-4, the concrete specimen was poured inside an aluminum frame specially fabricated for this testing program, and topped by a 3.2-mm (0.125-in.) steel plate. This system was designed to withstand compressive and tensile forces induced by the interface shear stresses with minimal deformations. The steel plate and aluminum sides of the frame act as an external reinforcement, minimizing tensile stresses in the concrete during the interface shear tests. A set of threaded, high-strength-steel studs work as shear connectors between the concrete and the steel plate.

3.2.1 Materials

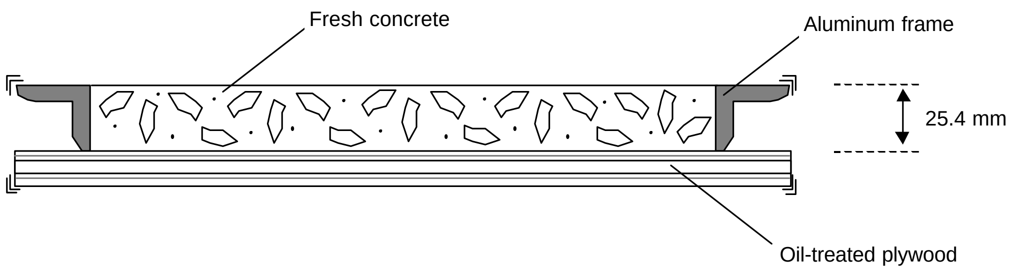
The fine aggregate for the specimen was a processed, well-graded sand, commercially available for the preparation of concrete. This sand will be referred to as Blacksburg Sand throughout this report. Examination under an optical microscope revealed that the sand grains smaller than 1 mm were predominantly subangular, with length-to-width ratios ranging from 1 to 3. The larger grains tended to be more angular and flat (ASTM 1993b).

The coarse aggregate was a crushed limestone with maximum grain size of 12.5 mm (1/2 in.), which is also commercially available for the preparation of concrete. The coarse aggregate was predominantly angular to subangular. The grain size distribution of the aggregates is presented in Figure 3-5.

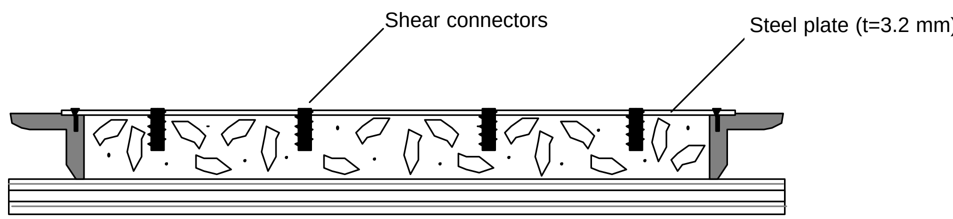
In order to obtain an adequate workability without compromising strength, an Air-Entraining Admixture (AEA) and a High-Range Water Reducer (HRWR) were included in the concrete mix. Additionally, a corrosion inhibitor admixture was added to prevent corrosion of the steel components of the concrete frame.

3.2.2 Preparation of the specimen

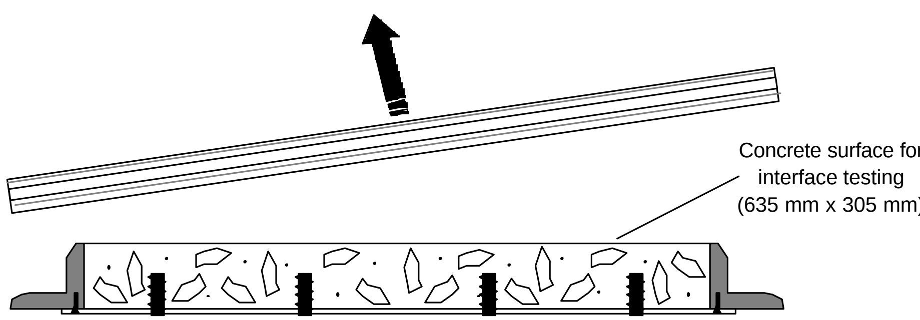
The concrete was prepared following a rigorous mixing sequence. First, the aggregates were impregnated with the corrosion inhibitor. Then, the AEA, cement, water, and HRWR were added in sequence and thoroughly mixed. The mixing proportions and some physical properties of the concrete are summarized in Table 3-4.



a) Placement and vibration of concrete



b) Placement of steel plate and introduction of shear connectors



c) Removal of plywood piece after 2 days curing

Figure 3-4. Preparation of the concrete specimen

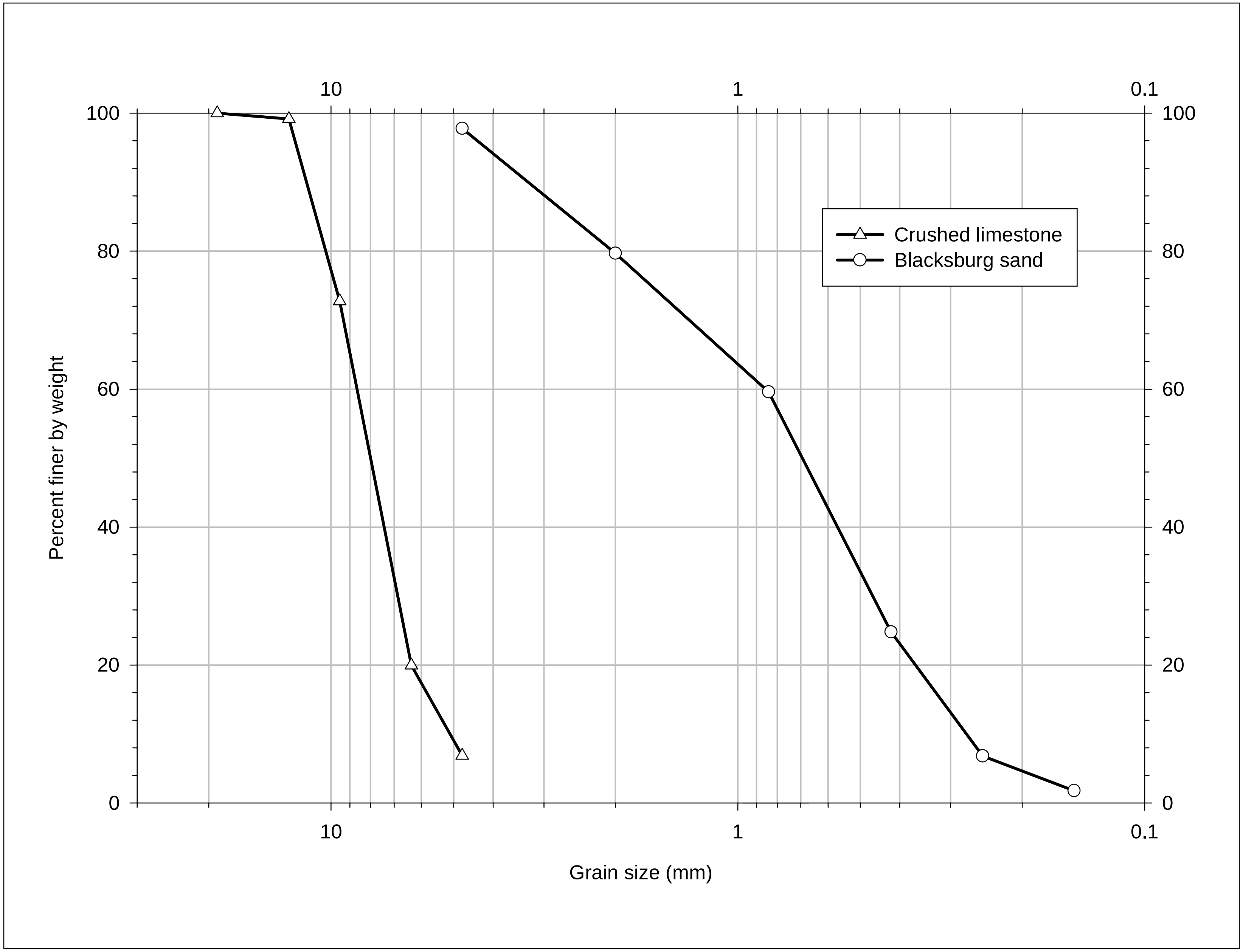


Figure 3-5. Grain size distribution of aggregates used for the preparation of the concrete specimen

Table 3-4 Mixing Proportions of Concrete	
Coarse aggregate	600 kg/m^3
Fine aggregate (Blacksburg Sand)	600 kg/m^3
Type I Portland cement	264 kg/m^3
Water	105 kg/m^3
HRWR (Daracem)	850 ml/m^3
AEA (Daravair)	530 ml/m^3
Corrosion inhibitor (DCI-S)	10,420 ml/m^3

Several trial batches were prepared until a mix with the appropriate physical properties was obtained. Tests were performed to determine the slump, air content, and compressive strength of the concrete, and the results are presented in Table 3-5.

Table 3-5 Physical Properties of the Concrete Mix	
Slump	180 mm (7 in.)
Air content	7%
Compressive strength 7 days 21 days 28 days	 24,700 kPa (3,580 psi) 27,450 kPa (3,980 psi) 33,100 kPa (4,800 psi)

The concrete was carefully placed onto the piece of plyform, inside the aluminum frame, as illustrated in Figure 3-4a. The concrete specimen was vibrated, trimmed, and covered with the steel plate. The plate was then attached to the aluminum frame as illustrated in Figure 3-4b. The threaded steel studs were screwed in place through the openings in the steel plate, and into the fresh concrete. The assembly was left in place for two days, after which the plywood piece was carefully removed, exposing the concrete surface for visual examination. At this point, an assessment was made of the surface texture of the specimen based on the results of the field survey of retaining walls, which is described in the following section. No surface treatment was applied to the specimen after its preparation. Once accepted as representative of field conditions, the specimen was placed in a wet room for 28 days, after which it was removed and used for the interface tests.

3.2.3 Surface texture

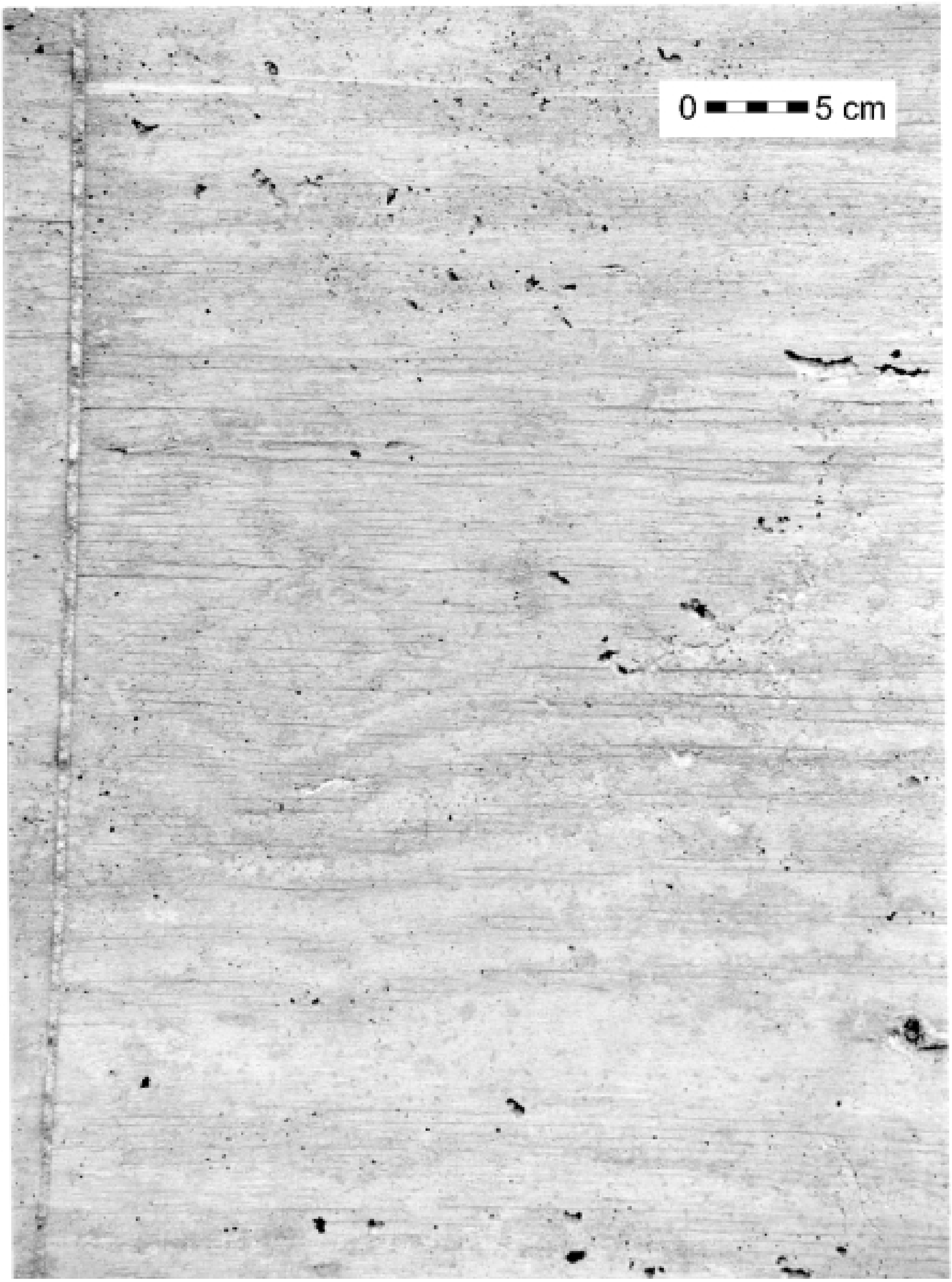
A field survey was performed in order to establish a range of surface textures representative of existing retaining walls. The survey, carried out throughout southwestern and northern Virginia, focused on mass and reinforced concrete retaining walls of height ranging from 3 to 7 m (10 to 23 ft), where plywood forms were used. For the purposes of this investigation, four main types of surface features were identified: 1) small-scale roughness controlled by fine aggregate of concrete, 2) waviness controlled by the formwork material, 3) pores, and 4) air pockets.

In total, ten retaining walls were surveyed. Two cases considered representative of all the walls surveyed are presented in Figure 3-6 at approximately the same scale. Wall A is a mass concrete retaining wall under construction, 5 m (16 ft) high and 60 m (200 ft) long, poured inside oil-treated plywood forms. The forms were reused up to six times throughout the length of the wall. The photography in Figure 3-6a shows the average texture of the back of the wall, where the forms had been reused two or three times. The imprint of the plywood pattern, or waviness, is evident and is a significant component of the concrete texture. It was also observed that the waviness increased with the number of times the forms were reused. Although the concrete was vibrated after placement, air pockets are frequent.

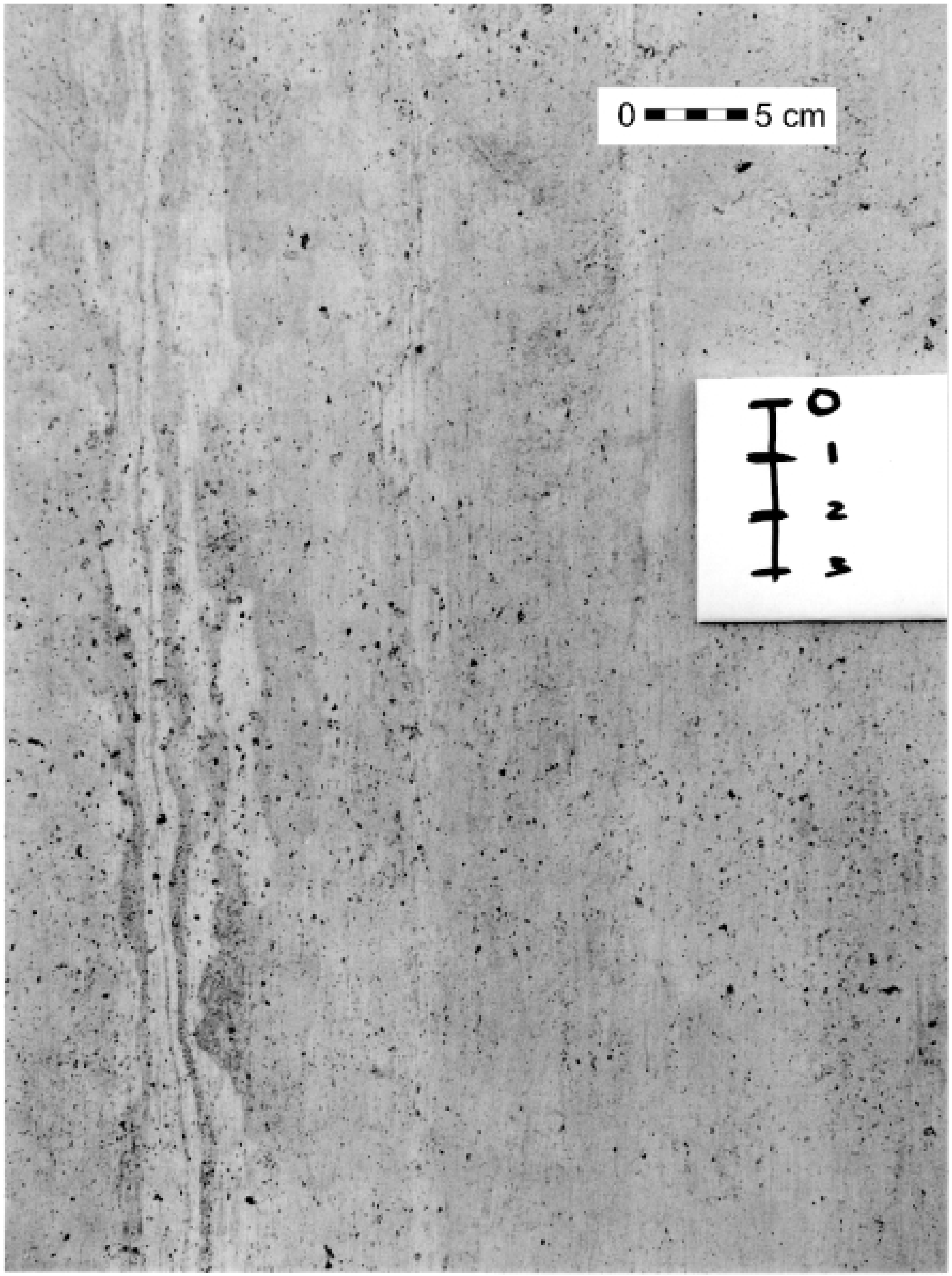
Wall B is a 7-m- (22-ft-) high reinforced concrete wall of recent construction. The photography in Figure 3-6b shows the average conditions on the exposed face of the wall before any surface treatment was applied. Here the pattern of the plywood is also evident, although not as marked as in wall A. There is a much higher frequency of air pockets than in wall A.

Figure 3-6c shows the surface of the finished concrete specimen. It has surface features similar to those of the walls surveyed. The imprint of the plywood pattern is clearly visible, suggesting a degree of waviness similar to that of the field cases. The frequency of air pockets was very difficult to control during the specimen preparation. The final specimen was obtained after several trials, and it presents a frequency of air pockets similar to that of wall B.

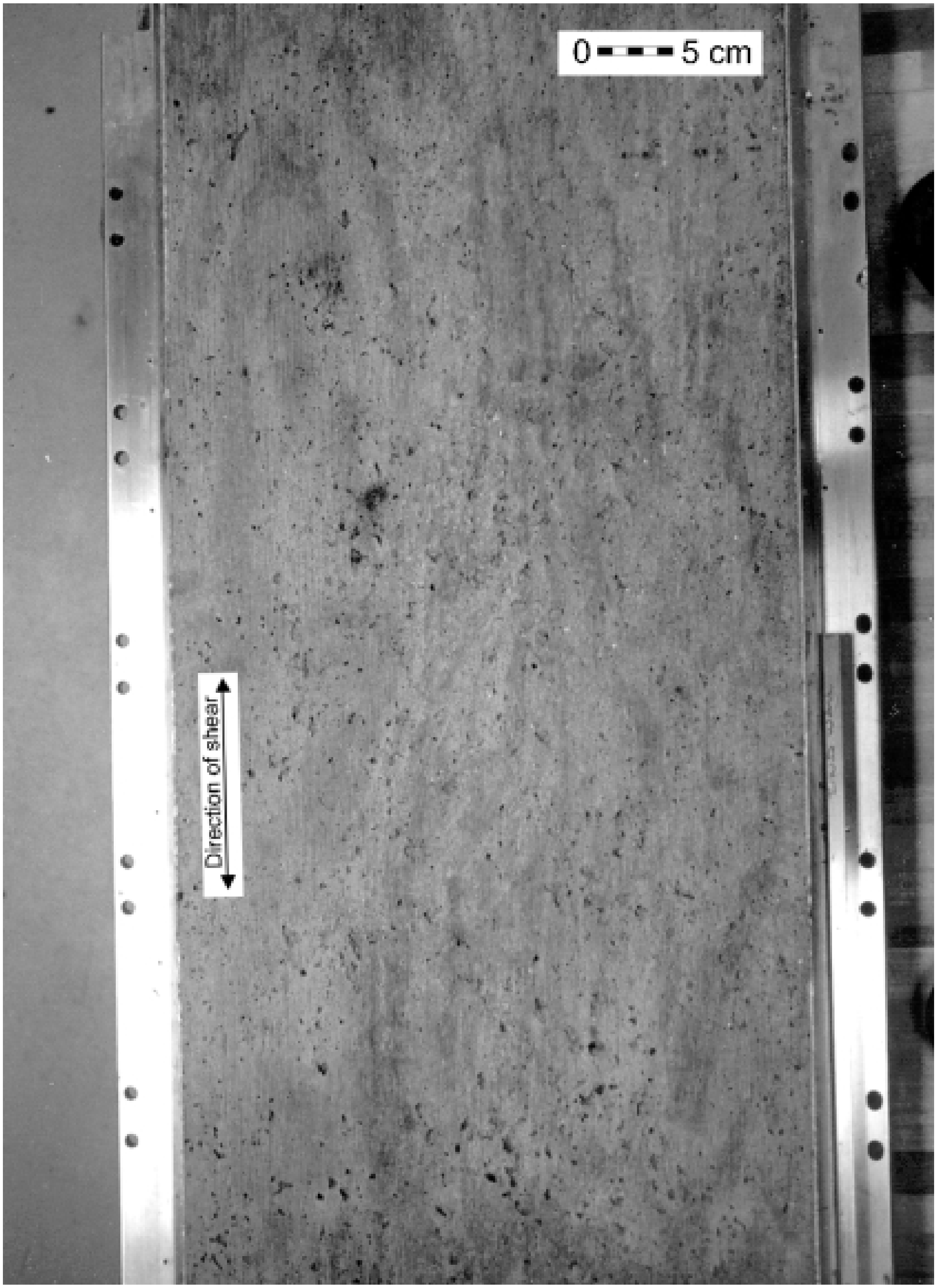
No controls were placed on surface features such as small-scale roughness and porosity, because they are very difficult to measure in the field. However, the concrete was prepared using materials and mixing procedures that are common in the industry and considered to be representative of actual construction practices.



a. Wall A
Figure 3-6. Surface texture of representative retaining walls (Sheet 1 of 3)



b. Wall B
Figure 3-6. (Sheet 2 of 3)



c. Concrete specimen
Figure 3-6. (Sheet 3 of 3)

3.3 Interface Testing Procedures

3.3.1 The soil box

For each test, the sand specimen is compacted inside a soil box that was designed and fabricated specially for sand-to-concrete interface testing. Its inner dimensions are 635 by 406 by 25.4 mm (25 by 16 by 1 in.), and it is composed of a bottom plate and sidewalls made entirely of aluminum. The face of the bottom plate in contact with the soil is coated with several layers of polyurethane paint mixed with medium to coarse sand. This prevents slippage of the soil during interface shear testing.

The sidewalls are connected to the bottom plate by a set of structural steel bolts, which also allow the creation of the gap at the interface as in Section 3.3.4. These bolts will be referred to as *set bolts* throughout this report.

A careful determination of the volume of the soil box was performed using several procedures. This was important to obtain reliable values of density of the sand sample.

3.3.2 Preparation of the interface

The sand-to-concrete interface is created by densification of the sand on top of the concrete specimen, as illustrated in Figure 3-7. The sidewalls of the soil box and an extension collar are placed on top of the slab. Four mounting brackets keep the sidewalls firmly in place throughout the entire process. In order to minimize the friction between the soil and the soil box, the inside of the walls is coated with vacuum grease and covered with a plastic sheeting 0.1 mm (4 mil) thick. Densification of the sand is achieved by either of two different procedures: vibration or pluviation.

In the vibration procedure, the system composed by the concrete slab, soil box, and trimming collar is attached to a 750- by 750-mm (30- by 30-in.) vibrating table. The soil is carefully placed inside the box to a height of approximately 5 mm above the trimming level, and vibrated under a 1.34kN (300-lbf) surcharge. The load and the extension collar are then removed, and the sample is trimmed as illustrated in Figure 3-7a. Theoretically, the final density of the sand can be adjusted by changing the frequency and amplitude of the vibrating table and the magnitude of the surcharge. However, it was found that the maximum relative density attainable with this setup was 70 to 80 percent. The scatter of the relative density values from one test to another was ± 5 percent. It is possible that higher densities can be obtained with heavier surcharges; however, sample preparation may become impractical.

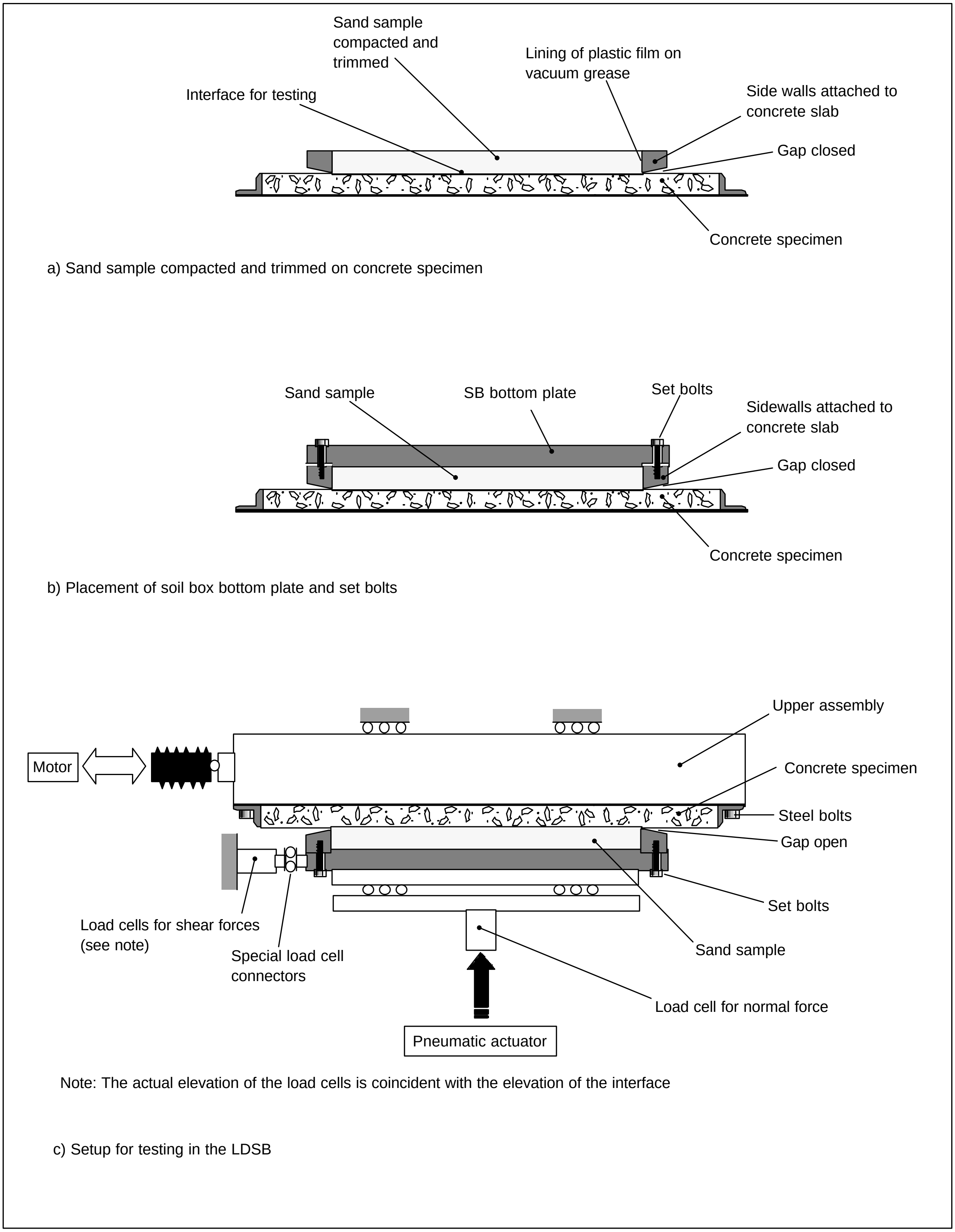


Figure 3-7. Preparation of sand-to-concrete interface and setup for testing

In the pluviation procedure, the sand is poured onto the surface of the concrete specimen. A simple *pluviation device* was constructed that allows a constant flow of sand dropping from a constant height on the concrete surface. The final density of the sand is determined by the drop height, the flow rate of sand, and the speed at which the device is translated over the concrete surface. The maximum density that can be attained by this procedure is 80 percent. The principal advantages of pluviation over vibration are a faster preparation of the specimen, and a greater repeatability with scatter in relative density values of ± 2 percent. After pluviation of the sand specimen, the extension collar is removed and the specimen is trimmed as illustrated in Figure 3-7a.

Once the soil specimen is prepared following either of these two procedures, the bottom plate is carefully placed on top of the sample, and bolted gently to the sidewalls, as illustrated in Figure 3-7b. The set bolts are tightened to generate a pressure of approximately 3.5 to 5 kPa between the bottom plate and the soil, and therefore between the soil and the concrete specimen as well. This low contact pressure maintains the rigidity of the assembly during handling, without disturbance to the interface. The assembly is then flipped and placed in position in the LDSB.

3.3.3 The Large Direct Shear Box (LDSB)

The LDSB was developed at Virginia Tech for testing of clay-to-HDPE interfaces (Shallenberger and Filz 1996). Some modifications to the LDSB were implemented to accommodate sand-to-concrete interface testing of the type described in this report. The LDSB is essentially a direct shear box type device with the capability to handle interfaces as large as 711 by 406 mm (28 by 16 in.), allowing a maximum interface displacement of 305 mm (12 in.). A view of the LDSB modified for soil-to-concrete interface testing is presented in Figure 3-8.

Shallenberger and Filz (1996) pointed out the advantages of the LDSB over conventional devices: 1) end effects are negligible, 2) the maximum displacement of 305 mm (12 in.) allows the determination of the interface residual shear strength, and 3) no eccentric normal loads are generated during shear. The large displacement capabilities of the LDSB are a particularly useful feature for the type of testing performed during this investigation.

Figure 3-7c illustrates the main components of the device. The concrete specimen is rigidly attached to a moveable upper assembly by a set structural steel bolts on each end of the slab. A screw jack transmits the action of a stepper motor to the upper assembly, which can be moved in both forward and reverse directions. The normal stress at the interface is provided by a pneumatic actuator capable of applying a force of up to 200 kN (44,000 lbf). During shear, the normal and tangential forces at the interface are measured by load cells as illustrated in the figure. The vertical and horizontal displacements at the interface are monitored by a system of four Linear Variable Displacement Transducers

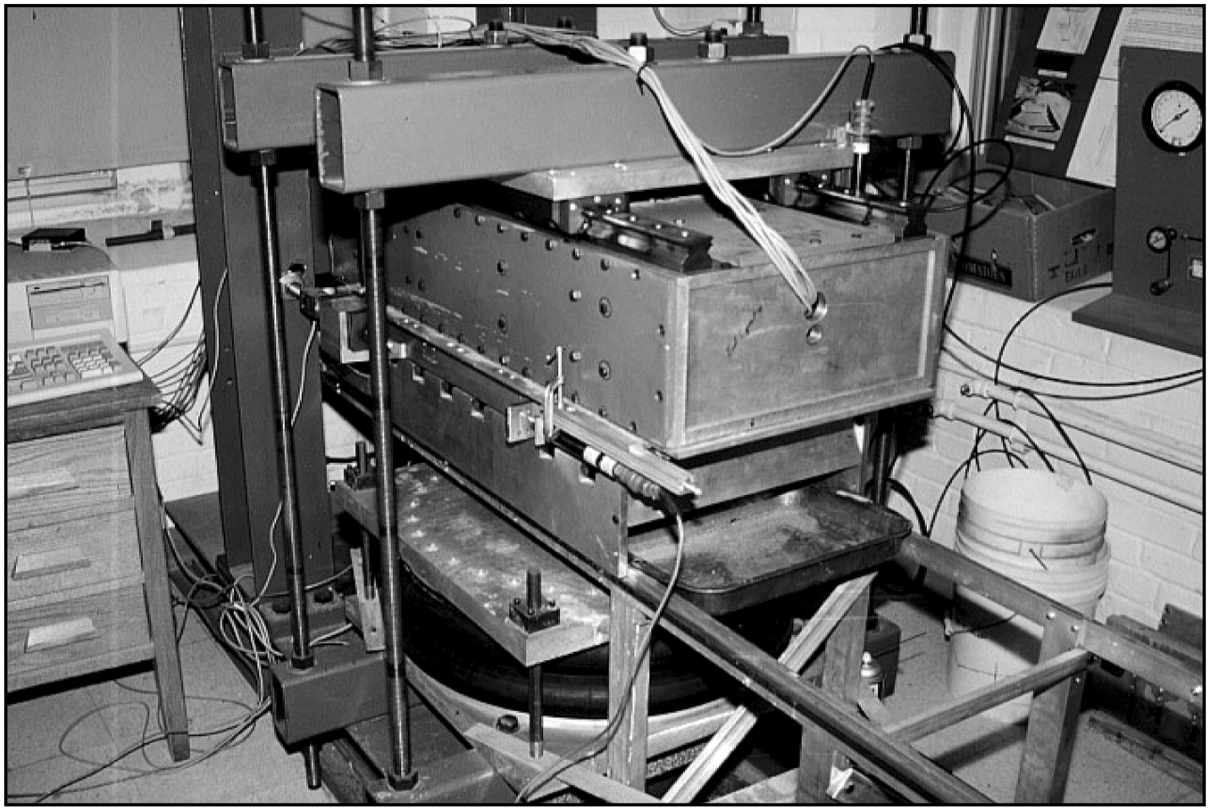


Figure 3-8. View of the LDSB ready for soil-to-concrete interface testing

(LVDTs), two in each direction, located on both sides of the sample. The LVDTs are capable of resolving displacements as small as 0.0025 mm (0.0001 in.).

A data acquisition system connected to a personal computer constantly monitors and records the readings from the load cells and transducers. The rate and direction of displacement along the interface are controlled by a program developed specially for this investigation. Testing features such as repeated unload-reload cycles between two predefined stress levels can be programmed or manually controlled. The program also facilitates loading of the interface along predefined stress paths. Conversion of the readings to units of force or displacement is done automatically using appropriate calibration factors.

3.3.4 Test setup

Once the sand-concrete sample is placed in the LDSB, it is lifted by the pneumatic actuator into its final position, and the concrete slab is bolted to the upper assembly. The normal pressure is increased until the initial pressure applied by the set bolts is produced. At this point, the set bolts become loose, and they can be turned by hand. The sidewalls are released from the mounting brackets and lowered by turning the set bolts in order to create a gap between the sidewalls and the concrete surface. The sidewalls slide gently into position due to their lubricated inner lining. The gap, illustrated in Figure 3-7c, is approximately 3.2 mm (0.125 in.) wide.

During the entire setup operation, the data acquisition system registers vertical and horizontal displacements and vertical loads. These data are checked to verify that undesired interface displacements and stresses have not occurred. After the normal pressure is increased to its final value, the soil box is connected to the horizontal load cells by special load cell connectors. These connectors restrain longitudinal displacement during shear in the forward or reverse direction. The sample is then ready for testing.

3.3.5 Data reduction

The load and displacement data recorded during the interface tests are processed using a data reduction program developed for the LDSB. The program creates a series of files containing the reduced data such as shear stress versus displacement and vertical versus horizontal displacement. These output files are copied onto a spreadsheet for processing and creation of graphic files.

3.4 Interface Testing Program

Interface tests were carried out between the concrete specimen and the soils described previously in this chapter. The purpose of the interface tests was to collect experimental data that allowed the development of a new interface model applicable to SSI analyses of lock walls. The results of these tests also add to the database on interface properties available in the literature.

3.4.1 Testing parameters

As indicated in Table 3-6, three types of interfaces were prepared: dense Density Sand against concrete, medium-dense Density Sand against concrete, and dense Light Castle Sand against concrete. For each of the dense Density Sand against concrete tests, the soil specimen was densified by vibration. For the medium-dense Density Sand and dense Light Castle Sand against concrete tests, the specimens were prepared by pluviation on the concrete surface. All the tests were performed at a displacement rate of 1 mm/min (0.04 in./min), under normal pressures ranging from 15 to 274 kPa (2 to 40 psi). This range of normal stresses is representative of field conditions in many lock walls.

Table 3-6
Types of Interfaces Tested

Interface Type	Average Relative Density	Procedure for Densification	Types of Tests Performed
Dense Density Sand against concrete	75 %	Vibration	Initial loading Staged shear Unload-reload Multi-directional stress path
Medium dense Density Sand against concrete	49 %	Pluviation	Initial loading Multi-directional stress path
Dense Light Castle Sand against concrete	80 %	Pluviation	Initial loading Multi-directional stress path

A set of preliminary tests was performed to study the influence of inundation of the interface on the test results. They showed that inundation does not induce any significant effect on the response of the interface. In average, the strength in inundated tests was 3.5 kPa higher than in dry tests at normal stresses ranging from 100 to 270 kPa. All subsequent tests were performed in a dry condition.

3.4.2 Types of interface tests

The types of interface tests performed for this investigation are summarized in Table 3-7, and illustrated in Figure 3-9. They can be grouped as: 1) initial loading, or virgin shear, tests; 2) staged shear tests; 3) unload-reload tests; and 4) multi-directional stress path tests.

3.4.2.1 Initial loading tests. The initial loading tests were performed under constant normal stress on the newly prepared interface. In all initial loading tests, shearing continued until the residual strength was mobilized. For each type of interface, a set of initial loading tests under different normal stresses was performed to determine the peak and residual friction angle values and hyperbolic parameter values of the interface.

3.4.2.2 Staged shear tests. These tests were performed to study the interface response under a finite change in the normal pressure during shear. As illustrated in Figure 3-9b, the interface is initially sheared under a constant normal stress to a point such as *S*. Then, shearing is paused and the normal stress increased to point *S'*. Finally, shearing is resumed under this increased normal stress. The tests modeled conditions in which the normal pressure increment occurs before mobilization of the peak strength as in Figure 3-9b, or after development of the residual condition. The results of these tests were used to determine the shape of the yield surface incorporated in the extended hyperbolic model, as discussed in Chapter 4.

3.4.2.3 Unload-reload tests. These tests modeled the interface response upon a change in the direction of shear. As shown in Figure 3-9c, the interface is sheared under constant normal stress to a point such as *U*, where the shear direction is reversed. Unloading progresses to a point such as *R*, where reloading is applied to the interface. These tests provided the basis for developing a formulation for interface response under unloading-reloading, as described in Chapter 4.

3.4.2.4 Multi-directional stress path tests. In this type of test, the interface was sheared following a stress path that involved simultaneous changes in normal and shear stresses, as illustrated in Figure 3-9d. They were intended to model loading conditions similar to those expected at the interface between a lock wall and the backfill, and to provide a basis for evaluation of the extended hyperbolic model described in Chapter 4.

Some staged shear and shear reversal tests were performed at the end of initial loading tests, after mobilization of the residual strength (Table 3-7). Although the large cyclical interface displacements required for this type of loading are not expected to take place in backfill-to-lock-wall interfaces, these results may be applicable to other Corps structures. These test results are reported in Appendix C.

Table 3-7
Summary of Interface Tests

Type of test	Type of Interface	Specimen	Test Number	Normal Stress (kPa)	Figures[1]	Observations
Initial loading (virgin shear)	Dense Density Sand against concrete	S101 S102 S103 S104	T101_2 T102_5 T103_15 T104_40	15 33 102 274	C1, C2	---
	Medium dense Density Sand against concrete	S302 S303 S304	T302_5 T303_15 T304_40	35 104 276	C3, C4	---
	Dense Light Castle Sand against concrete	S401 S402 S403 S404	T401_2 T402_5 T403_15 T404_40	15 35 104 276	C5, C6	---
Staged shear	Dense Density Sand against concrete	S105	T105_40	102 to 274	C7	Staged before peak
		S106	T106_15	33 to 102	C8	
		S101	T101_5 T101_15 T101_40	15 to 33 33 to 102 102 to 274	C9	Staged on residual condition
		S102	T102_15 T102_40	33 to 102 102 to 274	C10	
		S103	T103_40	102 to 274	C11	
Unload-reload	Dense Density Sand against concrete	S201	T201_5	33	C12	Unload-reload applied before peak
		S202	T202_5	33	C13	
		S203	T203_15	102	C14	
		S101	T101_2	15	C15	Reversals applied on residual condition. Two or three cycles were performed per test.
		S102	T102_5	33	C16	
		S103	T103_15	102	C17	
		S104	T104_40	274	C18	
	Medium dense Density Sand against concrete	S302	T302_5	35	C19	
		S303	T303_15	104	C20	
		S304	T304_40	276	C21	
	Dense Light Castle Sand against concrete	S401	T401_2	15	C22	
		S402	T402_5	35	C23	
		S403	T403_15	104	C24	
		S404	T404_40	276	C25	
Multi-directional stress path	Dense Density Sand against concrete	S204	T204_5	35 to 280	C26	---
		S205	T205_5	35 to 240	C27	
		S206	T206_5	35 to 275	C28	
	Medium dense Density Sand against concrete	S305	T305_10	70 to 250	C29	
	Dense Light Castle Sand against concrete	S405	T405_10	70 to 275	C30	

[1] Figures are included in Appendix C

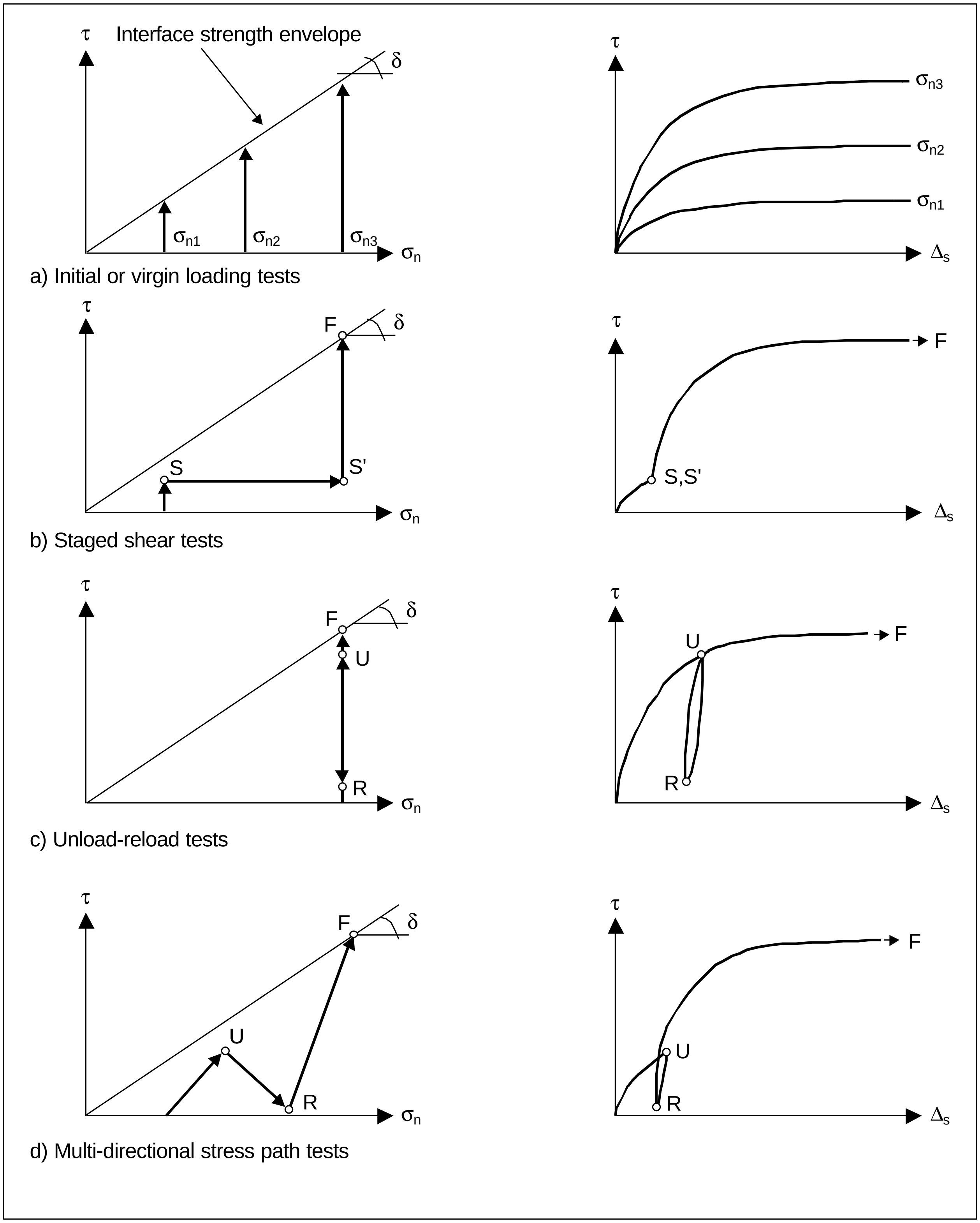


Figure 3-9. Types of laboratory interface shear tests performed

3.5 Results of Interface Tests

The results of the interface tests performed for this investigation are presented in the figures of Appendix C. Table 3-7 lists the figures in Appendix C that are relevant to each interface test. In this section, the results of the interface testing are summarized. The emphasis is on the issues that are the most important for interface modeling.

3.5.1 Interface response to initial loading

The results of the initial loading tests performed on interfaces between concrete and Density Sand or Light Castle Sand are presented in Figures C1 to C6, and summarized in Table 3-8. In the figures, the information regarding staged shear and shear reversal cycles, which were performed at the end of initial loading tests, is omitted for the sake of clarity and will be discussed in the following sections.

Table 3-8
Summary of Results of Initial Loading Interface Tests

Interface Type	Displacement [1], mm		Interface Friction Angle [1], deg		Interface/Soil Friction Angle Ratio	
	to Peak Δ_{sp}	to Residual Δ_{sr}	Peak δ	Residual δ_r	δ / ϕ_o	δ_r / ϕ_{cv}
Dense Density Sand against concrete	1-2	6-13	31	28	0.86	0.82
Medium dense Density Sand against concrete	2.5-3.5	12-20	29.5	28.5	0.71	0.86
Dense Light Castle Sand against concrete	1-2	6-11	33.7	29.5	0.79	0.81

[1] Parameters are listed and defined in the Notation (Appendix F)

The peak shear strength was mobilized at small displacements ranging from 1 to 3.5 mm. After mobilization of the initial peak strength, shear stresses remained practically constant before displacement softening took place. This plateau of relatively constant shear stress may exist up to displacements as large as 20 mm, as evidenced by Test T304_40 in Figure C3 of Appendix C, after which the residual condition was attained. In some cases, a secondary peak was observed in the shear stress-displacement curve. For the purposes of this investigation, the peak strength was assumed to correspond to the first peak that occurred during the test.

During shear, extension occurred normal to the interface in most of the tests. Once the residual condition was reached, no significant normal displacements took place. After removal of the concrete slab from the sand surface at the end of the tests, a layer of loose sand approximately 2 mm thick could be observed overlying the denser material immediately below. This is consistent with the observations by Uesugi, Kishida, and Tsubakihara (1988, 1989), Uesugi, Kishida, and Uchikawa (1990), and Hryciw and Irsyam (1993). They indicate that this dilation is produced mostly by normal deformations of a thin shear band developed in the soil adjacent to the concrete surface.

The peak and residual strength envelopes are also presented in Appendix C for each type of interface. Both peak and residual strength envelopes are linear for the range of normal stresses considered, and show no apparent cohesion intercept. The peak and residual friction angle δ and δ_r values are summarized in Table 3-8, and compared to the internal friction angle ϕ_o and 15 percent strain friction angle ϕ_{cv} for each of the soils. The values of ϕ_o and ϕ_{cv} were interpolated from Figure 3-3a for the relative density values of the soil specimens prepared for interface testing. It can be seen that the values of δ and δ_r correspond to approximately 80 percent of ϕ_o and ϕ_{cv} respectively.

The shear stress versus displacement plots reveal that interface displacements of less than 20 mm may not suffice for defining the value of the residual strength of sand-to-concrete interfaces. Consequently, shear box devices with displacement capabilities of less than 20 mm may not be adequate to collect data in situations where knowledge of the residual interface strength value is necessary.

The results of the initial loading tests were used to determined the hyperbolic parameter values of the three types of interface tested, as discussed in Chapter 4.

3.5.2 Interface response to staged shear

The results of the staged shear tests are presented in Figures C7 to C11 of Appendix C. These figures include the shear stress-displacement response of the interface together with the history of normal stress applied during the test and the record of displacements normal to the interface.

Two groups of staged shear tests were performed on the dense Density Sand against concrete interface (Table 3-7). In the first group (Figures C7 and C8), the normal stress was increased during shear and before mobilization of the peak strength. The specimens were sheared under constant normal stress to a stress level of approximately 0.65. The relative motion of the interface was then arrested, and the normal pressure increment was applied while maintaining approximately the same magnitude of shear stress. Shearing was resumed after the normal pressure increment was completed. A significant increase of the interface shear stiffness was observed upon application of the normal stress increment. The interface peak shear strength values determined from these staged shear tests are consistent with those determined from the initial loading tests described in the previous sections.

The test results presented in Figures C7 and C8 were used to determine the shape of the yield surface included in the new extended hyperbolic model, as discussed in Chapter 4.

In the second group of staged shear tests (Figures C9 to C11), several steps of normal stress increments were applied during shear in the residual condition. These tests were carried out at the end of the initial loading tests described previously. The normal pressure steps followed the sequence: 15 to 33 kPa, 33 to 102 kPa, and 102 to 274 kPa. In most of the tests shown, some shear reversal cycles were performed, which have been omitted for the sake of clarity.

Some conclusions may be drawn from Figures C9 to C11. The residual condition established at the interface after mobilization of the peak strength and at large displacements is maintained during further shearing of the interface and under subsequent increments of the normal stress. A peak strength greater than the residual strength occurs only in the initial loading stage; for subsequent stages, the shear stress increases to the residual value and then remains constant until the application of the next normal pressure increment.

It can also be observed that the residual strength value does not depend on the stress path followed and is the same in all tests performed under the same normal stress. Therefore, it is possible to determine a complete residual strength envelope from staged shear tests, as long as the displacement capability of the equipment is enough for the development of the residual condition.

3.5.3 Interface response to unloading-reloading

A group of three unload-reload interface tests was performed on the dense Density Sand against concrete interface. The results are presented in Figures C12 to C14 of Appendix C. These tests included one unload-reload cycle between two predetermined stress levels during initial loading. Tests T201 and T203 (Figures C12 and C14) were performed under constant normal stresses of 33 and 102 kPa, respectively. The unloading-reloading cycle was applied between stress levels of approximately 0.7 and 0.2. In Test T202, shown in Figure C13, the interface was unloaded from a stress level of approximately +0.75 to a stress level of -0.75, and then reloaded to failure.

A significant increase in the interface stiffness took place upon unloading and reloading of the interface. Hysteretic behavior was evident in all three tests. However, in Tests T201 and T203, in which shear stresses were always positive (shear in the first quadrant), the interface response was almost linear. In Test T202, in which unloading progressed into the fourth quadrant, the hysteretic loop was wider and the interface response was strongly nonlinear. The normal displacement plots included in the figures reveal that the interface undergoes compression during unloading and extension during reloading. These testes provided the basis for developing the unloading-reloading components of the interface model discussed in Chapter 4.

Some additional tests, performed in all three types of interfaces, consisted of one or more shear reversal cycles applied at the end of initial loading after mobilization of the residual strength. The results of such tests are presented in Figures C15 to C25 of Appendix C. It can be seen that the residual strength values are very similar or identical in both directions of shear.

3.5.4 Interface response to multi-directional stress paths

Five interface tests were performed following relatively complicated stress paths. Figures C26 to C30 in Appendix C show the stress path applied in each of these tests and the measured interface response. Test T204_5 (Figure C26) is a succession of seven staged shear tests applied on the dense Density Sand against concrete interface. The normal stress was increased from 35 to 275 kPa in 35-kPa steps. Although such type of loading is not expected to take place at a backfill-to-wall interface, the results of the test were very useful in the validation of the new extended hyperbolic model.

Test T205_5 (Figure C27) consisted of the application of a continuously increasing normal stress during shear of the interface between dense Density Sand and the concrete specimen. A similar type of loading may be expected at the interface between a wall and the backfill during placement and compaction of the backfill, as discussed in Chapter 1 (Figure 1-3a).

Tests T206_5 and T305_10 on the dense and medium-dense Density Sand against concrete interfaces and T405_10 on the dense Light Castle Sand against concrete interface (Figures C28 to C30) consisted of the application of more complicated stress paths. Initial loading, unloading, reloading, and simultaneous changes in shear and normal stresses were applied to the interface. The purpose of these tests was to provide a basis for a performance evaluation of the extended hyperbolic model under complicated loading paths. They also model certain aspects of the type of loading expected at the backfill-structure interface during backfill placement and operation of a lock wall.

3.6 Summary

The following laboratory and field activities were performed for this investigation:

a. Modifications to the LDSB.

b. Selection of sand specimens for interface testing.

c. Grain size distribution, minimum/maximum density, specific gravity, consolidation testing, and triaxial testing on the Density Sand and Light Castle Sand.

d. Field survey of existing concrete retaining walls to determine a range of representative surface textures for the concrete specimen.

e. Design and construction of a soil box and concrete slab.

f. Development of appropriate testing procedures.

g. Interface tests following a variety of laboratory stress paths to investigate the constitutive behavior of interfaces and to determine the interface response under field conditions for lock walls.

The LDSB was modified specifically to accommodate the soil-to-concrete interface testing for this investigation. A special aluminum soil box was designed and constructed that allows compaction of the sand sample directly onto the concrete specimen, and minimizes the disturbance of the interface during test setup operations.

A field survey of concrete walls was performed. Two representative cases were presented to convey the most common surface features of retaining walls cast against plywood. After a trial-and-error process, a concrete specimen was obtained with surface features similar to those observed in the field. The concrete specimen was contained in a frame, which was designed and constructed to act as an external reinforcement for the concrete and to minimize its deformations during interface shear.

A fine, rounded, silica sand (Density Sand), and a fine, angular sand (Light Castle Sand) were selected for interface testing. A series of basic laboratory tests, such as minimum/maximum density and grain size analyses, were performed on these sands. Consolidation and CD triaxial tests were also performed to determine sets of hyperbolic parameter values for these soils for a range of relative densities representative of the backfill in lock walls. These hyperbolic parameter values are consistent with values reported by Duncan et al. (1980) for soils of similar gradation.

An interface testing program was carried out that included initial loading tests, staged shear tests, unload-reload tests, and multi-directional stress path tests. Three types of interfaces were tested: dense Density Sand against concrete, medium-dense Density Sand against concrete, and dense Light Castle Sand against concrete. The results of these tests are presented in Appendix C.

The peak and residual interface friction angle values for the three types of interfaces tested are summarized in Table 3-8. The average ratio between the values of interface friction angle and internal friction angle of the soil was 0.8. Displacement softening was observed in all tests. The displacements required for the development of the residual condition were as large as 20 mm. Consequently, displacements of at least 20 mm are required for the determination of the residual strength of sand-to-concrete interfaces.

Staged shear tests were performed by increasing the normal pressure in steps during shear. The staged shear tests provided important information about the behavior of sand-to-concrete interfaces and were used to define the yield surfaces implemented in the extended hyperbolic model described in Chapter 4. It was found that it is possible to determine a complete residual strength envelope from staged shear tests, as long as the displacement capability of the equipment is enough for the development of the residual condition.

Several unload-reload tests were performed in which a complete loading cycle was applied between two predetermined stress levels. These tests followed stress paths similar to field stress paths in which the shear stresses may decrease as a consequence of a rise of the water table behind a lock wall. A substantial increase in the interface shear stiffness was observed upon unloading and reloading. It was observed that compression takes place during unloading, followed by dilation during subsequent reloading of the interface. In some tests, one or several cycles of shear were performed upon mobilization of the residual strength. Similar shear stress-displacement response and residual strength values were obtained for both directions of shear in all tests.

Multi-directional stress path tests were performed on all three types of interfaces. The purpose of these tests was to provide a basis for a performance evaluation of the extended hyperbolic model under complicated loading paths. They also modeled certain aspects of the type of loading expected at the backfill-structure interface during backfill placement and operation of a lock wall. The model discussed in Chapter 4 was validated against the results of these tests.