

2 Literature Review

This chapter is divided into the following sections:

1. *Interface testing*. This section contains a review of previous work on shear testing of interfaces between soils and structural materials.
2. *Interface modeling*. The interface models that are most commonly used in SSI analyses are reviewed. A detailed description is presented of the hyperbolic model for interfaces by Clough and Duncan (1971).
3. *SSI analyses of retaining walls*. The development of SSI analysis techniques is outlined in this section. A simplified procedure for estimating the downdrag forces on retaining walls is described.
4. *Summary*.

2.1 Interface Testing

Several studies have been published regarding laboratory interface testing. Most often, interface tests were performed to determine the soil-to-structure friction angle for design of geotechnical structures, such as retaining walls, buried culverts, piles, etc., and, in some cases, for the determination of parameters for constitutive modeling of interface response.

Interface tests have been performed on many types of soil-to-structure, soil-to-rock, and rock-to-rock interfaces. In this section, previous studies of soil-to-concrete and soil-to-steel interfaces are emphasized. The results of tests performed on both types of interfaces provide valuable insights into fundamental aspects of interface behavior.

2.1.1 Direct Shear Box (DSB) devices

Early systematic efforts to obtain data on the behavior of soil-to-structure interfaces were carried out by Potyondy (1961), Clough and Duncan (1971), and Peterson et al. (1976), among others. Their tests were performed using a slightly modified Direct Shear Box (DSB) in which a concrete specimen occupied one of the halves of the shear box. In most cases, the soil sample was prepared against a concrete specimen situated at the bottom. The tests were typically performed by first increasing the normal pressure to a desired value, then shearing the interface under constant normal stress to a maximum displacement of about 12.5 mm (0.5 in.).

Peterson et al. (1976) studied the fundamental factors that influence interface behavior. They performed a large number of sand-to-concrete interface tests using a 102- by 102-mm (4- by 4 in.) DSB. In their tests, the interface was inundated and sheared, under drained conditions and constant normal load, to a maximum displacement of 12.5 mm (0.5 in.). They analyzed the influence of normal stress, interface roughness, and soil characteristics on interface behavior, and developed a database of sand-to-concrete interface friction angles.

Peterson et al. (1976) also demonstrated the usefulness of the Clough and Duncan (1971) hyperbolic formulation to model interface behavior. They developed a set of hyperbolic parameter values, which have been used as an important source of data for SSI analyses up to the present time (Ebeling et al. 1993; Ebeling and Mosher 1996; Ebeling, Pace, and Morrison 1997; Ebeling, Peters, and Mosher 1997; Ebeling and Wahl 1997).

The DSB presents two important advantages: wide availability and relatively simple test setup and sample preparation procedures. Consequently, it has been the common choice for interface testing in research and practice. Some of the most relevant studies performed using DSB-type devices are summarized in Table 2-1. Other applications of the DSB include testing interfaces such as soil to geomembrane, soil to geotextile, and geomembrane to geotextile. Commercial devices have been developed for larger interface areas of up to 305 by 305 mm (12 by 12 in.), and they can be used for soil-to-concrete testing.

Traditional DSB devices present several limitations. The maximum relative displacement that can be attained in a conventional DSB is limited; hence, the determination of the interface residual strength becomes difficult. In addition, end effects, induced by the presence of the rigid walls of the soil container, may introduce errors in the test results.

Kishida and Uesugi (1987), Fakharian and Evgin (1995), and Evgin and Fakharian (1996) have pointed out that the actual sliding displacement Δ_{actual} between the soil particles and the concrete cannot be directly measured in the DSB, as illustrated in Figure 2-1a. The displacement Δ_{meas} measured between the

Table 2-1 Previous Work on Direct Shear Testing of Sand-to-Concrete and Sand-to-Steel Interfaces			
Source	Type of Interface and Dimensions	Type of Loading	Summary
Potyondy (1961)	Sand to concrete Sand to steel	Monotonic under constant normal stress	<ul style="list-style-type: none"> Developed a database of interface friction parameter values for interfaces between sand and concrete of varying roughness
Clough and Duncan (1971)	Sand to concrete	Monotonic shear under constant normal stress	<ul style="list-style-type: none"> Developed a hyperbolic formulation for modeling interface response
Peterson et al. (1976) and Kulhawy and Peterson (1979)	Sand to concrete 102 mm x 102 mm	Monotonic shear and shear reversal under constant normal stress	<ul style="list-style-type: none"> Analyzed the relationship between the interface response and the interface roughness, soil type, and soil density and gradation Added important contributions to the database of parameters for the Clough and Duncan (1971) hyperbolic formulation
Acar, Durgunoglu, and Tumay (1982)	Sand to concrete Sand to steel	Monotonic shear under constant normal stress	<ul style="list-style-type: none"> Studied the relationship between void ratio of the sand and interface friction angle Presented a relationship between void ratio and hyperbolic parameter values for Clough and Duncan (1971) formulation for the materials used in their tests
Desai, Drumm, and Zaman (1985)	Sand to concrete 305 mm x 305 mm	Cyclic shear under constant normal stress	<ul style="list-style-type: none"> Developed the <i>Cyclic Multi-Degree-of-Freedom</i> (CYMDOF) device for interface testing Studied the influence on interface response of the following factors: displacement and shear stress amplitude, number of loading cycles, and initial density of the sand
Bosscher and Ortiz (1987)	Sand to concrete Sand to rock	Cyclic shear under constant normal stress	<ul style="list-style-type: none"> Studied the relationship between interface roughness and interface friction angle Assessed the effect of roughness on damping ratio of the interface
Lee et al. (1989)	Sand to concrete 100 mm x 100 mm	Monotonic shear under constant normal stress	<ul style="list-style-type: none"> Developed of a set of hyperbolic parameter values for the response of the interfaces used in their tests
Hryciw and Irsyam (1993)	Sand to ribbed steel 267 mm x 76 mm	Monotonic and cyclic shear under constant normal stress	<ul style="list-style-type: none"> Studied the mechanisms of dilation and shear band formation at the interface Studied the influence of rib geometry and spacing, and soil density on the interface response

soil box and the concrete specimen includes the sliding displacement at the interface, as well as the deformation Δ_{dis} of the sand mass due to distortion under the applied shear stresses.

2.1.2 Direct Simple Shear (DSS) devices

Direct Simple Shear (DSS) devices have been intensively employed for interface testing during the last two decades, primarily on sand-to-steel and clay-to-steel interfaces. Sand-to-steel tests have yielded interesting results regarding the general behavior of interfaces. Many of these results are applicable to sand-

**Table 2-2
Previous Work on Direct Simple Shear Testing of Sand-to-Concrete and Sand-to-Steel Interfaces**

Source	Type of Interface and Dimensions	Type of Loading	Summary
Uesugi and Kishida (1986a and 1986b)	Sand to steel 100 mm x 40 mm	Monotonic under constant normal stress	<ul style="list-style-type: none"> Concluded that distortion of the sand sample is an important component of the total displacement
Kishida and Uesugi (1987)	Sand to steel 400 mm x 100 mm	Monotonic shear under constant normal stress	<ul style="list-style-type: none"> Found direct relationship between steel roughness and interface friction coefficient was found
Uesugi, Kishida and Tsubakihara (1988)	Sand to steel 400 mm x 100 mm	Monotonic shear and shear reversal under constant normal stress	<ul style="list-style-type: none"> Found that slippage and rolling of sand particles occurs during shear, on rough steel surface Found that only slippage occurs on smooth steel surface Found that large volume changes of sand occur near the contact with the steel surface Reported shear band formation on rough surfaces, not on smooth surfaces
Uesugi, Kishida and Tsubakihara (1989)	Sand to steel 100 mm x 40 mm	Cyclic shear under constant normal stress	<ul style="list-style-type: none"> Confirmed observations on shear band formation by Uesugi, Kishida and Tsubakihara (1988)
Uesugi, Kishida and Uchikawa (1990)	Sand to concrete 100 mm x 40 mm	Cyclic shear under constant normal stress	<ul style="list-style-type: none"> Observed similar behavior as in sand-to-steel interfaces Found that large volume changes of the sand occur in the vicinity of the concrete surface Observed shear band formation Reported that actual sliding displacement between sand particles and concrete is small Found a direct relationship between concrete roughness and friction coefficient
Evgin and Fakharian (1996)	Sand to steel 100 mm x 100 mm	Monotonic shear under constant normal stiffness	<ul style="list-style-type: none"> Developed the <i>Cyclic 3-D Simple Shear Interface (C3DSSI)</i>, from a previous DSB version by Fakharian and Evgin (1996), capable of applying shear stresses in two orthogonal directions
Fakharian and Evgin (1997)	Sand to steel 100 mm x 100 mm	Cyclic shear with constant normal stiffness	<ul style="list-style-type: none"> Studied the interface shear strength degradation during cyclic shear under constant normal stiffness
Desai and Rigby (1997)	Clay to steel and clay to rock 165-mm diameter	Cyclic shear under constant normal stress	<ul style="list-style-type: none"> Presented the CYMDOF-P device, with pore pressure measurement capabilities, which is still under development.

cylindrical specimen of the structural material is embedded in sand. During testing, the specimen of structural material is pulled along its axis to failure under a confining pressure applied on the boundary of the sand sample. This device was created in an attempt to model the behavior of a pile shaft. The sample preparation for this type of test is complicated, and the normal stresses at the interface are difficult to control and depend on the relative stiffness between the structural specimen and the sand (Kishida and Uesugi 1987).

Ring shear devices have been used by Huck and Saxena (1981) and Yoshimi and Kishida (1981) for sand-to-concrete and sand-to-steel interface testing. According to Stark, Williamson, and Eid (1996), ring shear devices have the following advantages: 1) unlimited interface displacement, making possible the

determination of residual interface shear strengths, 2) shearing along the same interface throughout the test and, and 3) no eccentric loading during shear.

The principal disadvantages of the ring shear device are as follows: 1) complicated sample preparation procedures, especially for sand-to-concrete interfaces (Kishida and Uesugi, 1987), 2) relatively narrow soil samples, which may induce scale effects in some interface tests, 3) nonuniform radial distribution of shear stresses (Stark, Williamson and Eid 1996), and 4) unknown actual sliding displacement at the interface in the case of rigid ring shear devices.

2.1.4 Summary of previous findings on interface testing and interface behavior

There seems to be no universal agreement on procedures and data interpretation for interface testing. Furthermore, little progress on the understanding of the behavior of sand-to-concrete interfaces has occurred since the work of Clough and Duncan (1971) and Peterson et al. (1976). However, several observations, which have been substantiated in more recent investigations, may be considered of special interest for this research:

1) The Clough and Duncan (1971) hyperbolic formulation is an appropriate model for the behavior of interfaces under constant normal stress and monotonic shearing (Peterson et al. 1976; Acar, Durgunoglu, and Tumay 1982; Lee et al. 1989).

2) The main factors affecting interface behavior under monotonic loading are interface roughness, soil density, particle angularity, and normal stress (Peterson et al. 1976; Bosscher and Ortiz 1987; Hryciw and Irsyam 1993; Uesugi and Kishida 1986a; Kishida and Uesugi 1987; Uesugi, Kishida, and Tsubakihara 1988).

3) In all the studies reviewed, displacement softening behavior was reported in interface tests between dense sand and structural materials.

4) The interface peak friction angle increases steadily with increasing interface roughness until a maximum is reached (Peterson et al. 1976; Bosscher and Ortiz 1987; Uesugi and Kishida 1986a; Kishida and Uesugi 1987; Uesugi, Kishida, and Uchikawa 1990). This maximum is very close to or slightly lower than the internal peak friction angle of the sand. The roughness value at which this maximum value is reached is commonly referred to as the *critical roughness*. There are several criteria to quantify the interface roughness, of which none seems to have been adopted universally. Therefore, the critical roughness values are given in units that are not consistent among different investigators.

5) Dilation occurs during shear of a dense sand-to-concrete or dense sand-to-steel interface (Peterson et al. 1976). The dilative deformations of sand in contact with a rough surface are usually large, and take place in a thin zone within the soil adjacent to the interface (Hryciw and Irsyam 1993; Uesugi, Kishida, and Tsubakihara 1988). Dilation is followed by the development of large displacements along the interface. In loose sand samples, compression occurs during shear and is also followed by large displacements. This zone of large volumetric changes and interface displacements is commonly known as the *shear band*. Shear band formation has not been observed on smooth interfaces.

6) Interface behavior during cyclic shear is affected by interface roughness, soil density, particle angularity, normal stress, displacement and stress amplitude, and number of loading cycles (Desai, Drumm, and Zaman 1985; Uesugi, Kishida, and Tsubakihara 1989; Uesugi, Kishida, and Uchikawa 1990).

7) Distortions of the sand mass above the interface are significant; consequently, the actual displacement at the interface cannot be determined in DSB devices (Uesugi and Kishida 1986a, 1986b).

2.1.5 The Large Direct Shear Box (LDSB)

Shallenberger and Filz (1996) developed a Large Displacement Shear Box (LDSB) especially designed for interface testing. The LDSB is essentially a DSB-type device with the capability to handle interfaces as large as 711 by 406 mm (28 by 16 in.). The device is capable of attaining interface displacements as large as 305 mm (12 in.) and has been used extensively for clay-to-High Density Polyethylene (HDPE) geomembrane testing. The soil sample is prepared in a soil box and pressed against a moveable upper assembly containing the specimen of HDPE or structural material. An isolated test section 305 by 305 mm (12 by 12 in.) in the upper assembly, located at the center of the interface, allows the measurement of normal and shear stresses away from the edges.

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 principal disadvantage of this apparatus is that sample preparation is a time-consuming process due to the large size of the interface. Additionally, the distortion deformations of the soil sample cannot be measured; therefore, the actual interface displacements are not known.

The large displacement capabilities of the LDSB, which permit staged tests with several steps of normal pressure increments, and its reduced end effects are

the main reasons for its use in this investigation. Several modifications were implemented in the device to accommodate sand-to-concrete interfaces and perform tests that included unloading-reloading and shear reversals. A more detailed description of the LDSB will be provided later in this report.

2.2 Interface Modeling

In SSI analyses, the soil-structure interface is represented by interface elements. Several kinds of interface elements have been developed to model the behavior of the interface under certain loading conditions. When an interface element is developed for a particular problem, an appropriate constitutive relationship must be adopted. The constitutive relationship should be capable of modeling the interface response under the expected loading conditions.

A literature review of interface elements and interface constitutive models has been performed for this investigation. The most significant contributions pertinent to this research are outlined in the following sections.

2.2.1 Interface elements

Interface elements were first introduced by Goodman, Taylor, and Brekke (1968) for finite element analysis of jointed rock masses. They were soon extended to SSI analyses of retaining walls by Clough and Duncan (1971) and Duncan and Clough (1971). The adoption of interface elements represented a significant improvement over previous methods, which assumed either of two conditions: a perfectly rough interface with no slip between soil and structure, or a perfectly smooth interface with no shear stresses developed (Clough and Duncan 1971).

The element developed by Goodman, Taylor, and Brekke (1968) is commonly referred to as *joint element* or *zero-thickness interface element*. It is a four-node element of zero thickness as illustrated in Figure 2-2. In their derivation of the joint element stiffness matrix, they used a very simple constitutive law consisting of constant values for both the shear stiffness and the normal stiffness:

$$k_n \cdot \Delta_n = \sigma_n \quad (2-1)$$

$$k_s \cdot \Delta_s = \tau \quad (2-2)$$

where

k_n = normal interface stiffness

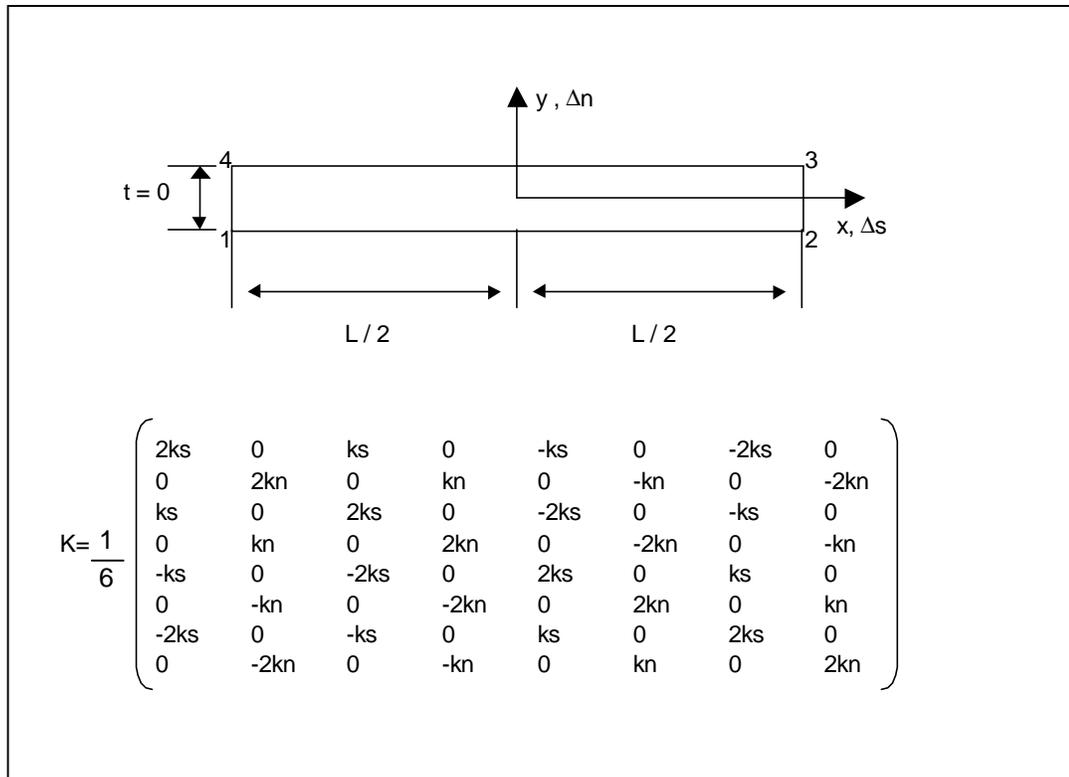


Figure 2-2. Goodman, Taylor, and Brekke (1968) zero thickness interface element and corresponding element stiffness matrix

Δ_n = displacement normal to the interface

σ_n = normal stress acting on the interface

k_s = interface shear stiffness

Δ_s = displacement along the interface

τ = interface shear stress

In this formulation, coupling effects between tangential and normal displacements along the interface are excluded, as evidenced by zero off-diagonal elements in the stiffness matrix.

Clough and Duncan (1971) observed that compressive stresses normal to the interface would induce overlapping among soil and structural elements adjacent to the interface. To minimize this effect, they proposed assigning a high value of normal stiffness to the joint element under compression. Similarly, for interface

elements under tension, they proposed assigning a small value of normal stiffness to minimize the development of tensile stresses at the interface.

A continuous development of improved joint elements has taken place since the original formulation by Goodman, Taylor, and Brekke (1968). Heuze and Barbour (1982) presented a review of the historical development of joint elements for analyses of jointed rock masses; most were linear and one-dimensional and did not include rotational degrees of freedom. A limited number of joint elements were developed to model displacement softening and dilation at the joints.

Morrison (1995) also presented a comprehensive review of previous investigations of interface elements. Many of the studies he reviewed described numerical integration problems arising from the use of interface elements of high normal stiffness adjacent to softer soil elements. Morrison (1995) studied an interface element formulation with relative degrees of freedom proposed by Wilson (1975) to minimize such numerical problems. Morrison (1995) showed that the Wilson (1975) formulation is not necessary if Newton-Raphson iteration is used to find the interface displacements resulting from each load increment.

Heuze and Barbour (1982) presented a zero-thickness axisymmetric joint element for finite element analyses of footings on rock, underground openings, and excavations, where dilation effects play an important role. Although no coupling terms are included in the formulation of the element, the dilation-induced normal stresses are determined explicitly based on the stiffness of the surrounding rock and the dilation angle. Yuan and Chua (1992) presented a more general formulation of the Heuze and Barbour (1982) axisymmetric element.

Matsui and San (1989) proposed an elastoplastic joint element to model interface behavior of rock joints. It accounts for the generation of normal stresses during shear, due to fully restrained dilation of the joint, in a way similar to that of Heuze and Barbour (1982).

Desai et al. (1984) and Zaman, Desai, and Drumm (1984) presented the *thin layer interface element*. It is based on the idea that interface behavior is controlled by a narrow band of soil adjacent to the interface with different properties from those of the surrounding materials. The thin layer element is treated mathematically as any other element of the finite element mesh and is assigned special constitutive relations. The Desai et al. (1984) thin layer element prevents overlapping between structural and geological materials due to its finite thickness. Desai, Muqtadir and Scheele (1986) implemented the thin layer element in interaction analyses of grouted anchors-soil systems.

Wong, Kulhawy, and Ingraffea (1989) implemented a three-dimensional version of the thin layer interface element for SSI analyses of drilled shafts under generalized loading.

2.2.2 Interface constitutive models

A number of interface constitutive models have been developed by different authors. Depending on the type of analysis performed, the interface behavior may be represented by a quasi-linear or a nonlinear model. Quasi-linear models consider a constant value of stiffness over a range of interface displacements, until yield is reached. After yield, a low constant value of stiffness is usually assigned to the interface. Quasi-linear models have been used by Goodman, Taylor, and Brekke (1968); Desai, Muqtadir, and Scheele (1986); Matsui and San (1989); and Wong, Kulhawy, and Ingraffea (1989).

In nonlinear models, the interface shear stress-displacement relationship is represented by a mathematical function of higher degree. The interface shear stiffness changes during shear, depending on the magnitude of the displacement and any other factor included in the model. Nonlinear models have been used by Clough and Duncan (1971); Zaman, Desai, and Drumm (1984); and Desai, Drumm, and Zaman (1985) among others.

Clough and Duncan (1971) developed the hyperbolic model for interfaces. This model has been used extensively in SSI analyses and design of geotechnical structures, including analyses of lock wall behavior (Ebeling et al. 1993; Ebeling and Mosher 1996; Ebeling, Peters, and Mosher 1997; Ebeling and Wahl 1997; and Ebeling, Pace, and Morrison 1997). The hyperbolic model, described in detail in the next section, often provides an accurate approximation to the interface response under monotonic loading at constant normal stress. It has not been extended to cyclic loading, or to staged shear.

Zaman, Desai, and Drumm (1984) developed a constitutive model for cyclic loading of interfaces. It is based on a polynomial formulation that includes the effects of the number of cycles, amplitude of shear displacements, and normal stress on interface response.

Desai, Drumm, and Zaman (1985) presented a *modified Ramberg-Osgood* model for interfaces under cyclic loading. The model accounts for shear stress reversals, hardening or degradation effects with number of load cycles, normal stress, relative density of the sand, and maximum displacement amplitude. Uesugi and Kishida (1985) observed that the modified Ramberg-Osgood model yields inconsistent results for shear stresses close to failure.

In all the interface models described previously, the interface yield stress is determined by the Mohr-Coulomb criterion (Goodman, Taylor, and Brekke 1968; Clough and Duncan 1971; Zaman, Desai, and Drumm 1984; Desai, Muqtadir, and Scheele 1986; and Wong, Kulhawy, and Ingraffea 1989).

Post-peak displacement softening has been included in the formulation of some quasi-linear models that are used in conjunction with iterative finite element procedures (Wong, Kulhawy, and Ingraffea, 1989). Esterhuizen (1997) presented a nonlinear constitutive formulation for clay-HDPE interfaces that accounts for work-softening behavior of the interface.

Coupling between normal and shear deformations is not included in any of the constitutive formulations found in the literature. Changes in normal stress during shear, due to restrained dilation of rock joints, are accounted for in the models proposed by Heuze and Barbour (1982) and Matsui and San (1989). This was accomplished by including an explicit formulation relating changes in normal stresses with shear displacement, dilation angle, and elastic properties of the adjacent rock mass.

2.2.3 The hyperbolic model

Duncan and Chang (1970) presented a hyperbolic model for soil behavior following previous work by Kondner (1963) and Kondner and Zelasko (1963). The hyperbolic model was extended to interfaces by Clough and Duncan (1969, 1971) and implemented into the Goodman, Taylor, and Brekke (1968) joint element formulation.

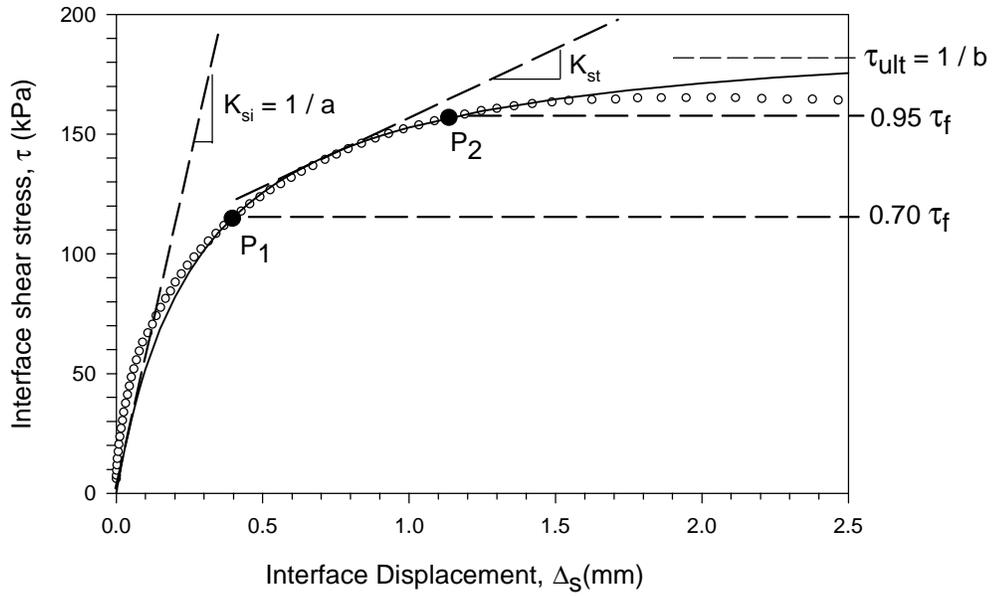
Figure 2-3 illustrates the basic aspects of the Clough and Duncan (1971) hyperbolic model for interfaces. A hyperbola, shown in Figure 2-3a, has been used to fit a set of data from an interface shear test. The equation of the hyperbola can be written as:

$$\tau = \frac{\Delta_s}{a + b \cdot \Delta_s} \quad (2-3)$$

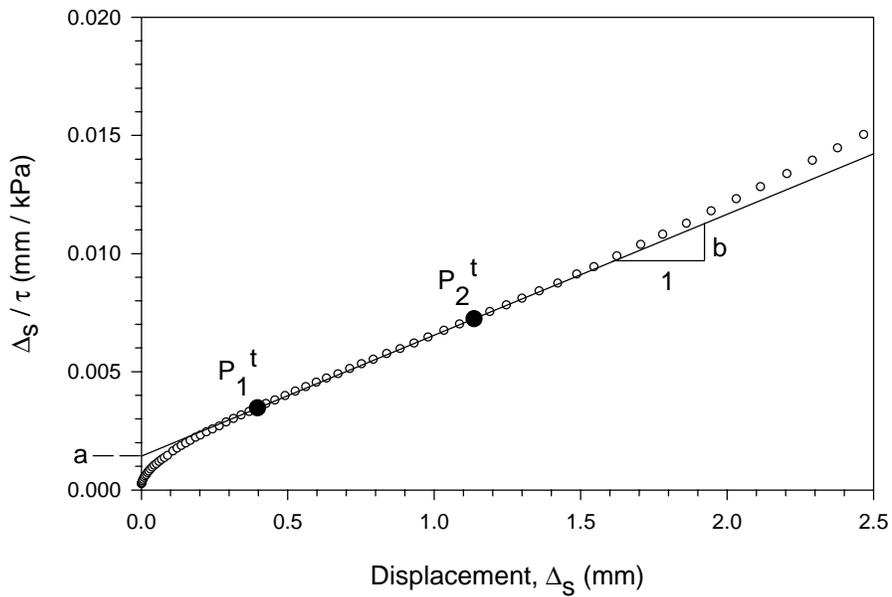
in which a and b are parameters evaluated to fit the hyperbola to the experimental data. Equation 2-3 can be re-written as:

$$\frac{\Delta_s}{\tau} = a + b \cdot \Delta_s \quad (2-4)$$

Figure 2-3b shows the same test data of Figure 2-3a, plotted in terms of Δ_s / τ and Δ_s . This is called the *transformed plot*. If the interface shear stress-displacement behavior follows a hyperbolic relationship, the transformed plot will be a straight line. The hyperbolic parameters a and b of Equation 2-4 will be the intercept and slope of this straight line, respectively. The actual interface test data do not exactly follow a hyperbolic relationship, and the transformed plot must then be fitted to a straight line to determine the hyperbolic parameters a and b .



a) Comparison between test data and the hyperbolic model



b) Transformed plot and determination of hyperbolic parameters a and b

Figure 2-3. Application of the Clough and Duncan (1971) interface hyperbolic model to a typical set of test data

Duncan and Chang (1970) observed that the transformed plot for strength test data in soils diverged from a straight line, both at low and high values of strain. They concluded that the best fit to the data was obtained when the hyperbola intersected the test data at 70 and 95 percent of the strength. Clough and Duncan (1971) adopted this same criterion for interface shear tests. Figure 2-3a shows points P_1 and P_2 corresponding to a mobilized strength of 70 and 95 percent, respectively. Points P_1' and P_2' are the corresponding representations of P_1 and P_2 in the transformed plot. A straight line is drawn between P_1' and P_2' , and the hyperbolic parameters a and b of Equation 2-4 are found as illustrated in the figure.

The hyperbolic shear stress-displacement relationship, calculated from equation 2-3, is presented in Figure 2-3a. It can be seen that the model intersects the test data at points P_1 and P_2 .

One of the advantages of the Clough and Duncan (1971) model is that the hyperbolic parameters a and b are physically meaningful. The value of a is the reciprocal of the initial shear stiffness K_{si} of the interface. The value of b is the reciprocal of the asymptotic shear stress value τ_{ult} of the hyperbola. The value of τ_{ult} is usually larger than the actual interface shear strength τ_f . To address this discrepancy, Clough and Duncan (1971) defined the failure ratio R_{ff} as:

$$\tau_f = R_{ff} \cdot \tau_{ult} \quad (2-5)$$

Clough and Duncan proposed that the value of initial interface stiffness be calculated using the following expression:

$$K_{si} = K_I \cdot \gamma_w \cdot \left(\frac{\sigma_n}{p_a} \right)^{n_j} \quad (2-6)$$

where

K_I = dimensionless stiffness number

γ_w = unit weight of water

p_a = atmospheric pressure

n_j = dimensionless stiffness exponent

From a series of interface tests under different normal stresses, the hyperbolic parameter a , and consequently K_{si} , can be evaluated for each normal stress. The stiffness number K_I and stiffness exponent n_j can then be calculated by fitting K_{si} and σ_n data to Equation 2-6.

For interfaces without an adhesion intercept, the shear strength of the interface τ_f can be expressed as:

$$\tau_f = \sigma_n \cdot \tan \delta \quad (2-7)$$

where δ is the angle of interface friction, which can be also determined from a series of interface shear tests at different values of normal stress σ_n . Finally, by substituting Equations 2-5, 2-6, and 2-7 into Equation 2-3, the following hyperbolic expression is obtained:

$$\tau = \frac{\Delta_s}{\frac{1}{K_I \cdot \gamma_w \cdot \left(\frac{\sigma_n}{p_a}\right)^{n_j} + \frac{R_{ff} \cdot \Delta_s}{\sigma_n \cdot \tan \delta}}} \quad (2-8)$$

For incremental analyses, it is necessary to determine the tangent stiffness value K_{st} at any point during shear. By differentiating Equation 2-8 with respect to Δ_s the following expression is obtained:

$$K_{st} = K_I \cdot \gamma_w \cdot \left(\frac{\sigma_n}{p_a}\right)^{n_j} \cdot \left(1 - \frac{R_{ff} \cdot \tau}{\sigma_n \cdot \tan \delta}\right)^2 \quad (2-9)$$

If the stress level SL is defined as:

$$SL = \frac{\tau}{\sigma_n \cdot \tan \delta} \quad (2-10)$$

the tangent stiffness K_{st} can be expressed as:

$$K_{st} = K_I \cdot \gamma_w \cdot \left(\frac{\sigma_n}{p_a}\right)^{n_j} \cdot (1 - R_{ff} \cdot SL)^2 \quad (2-11)$$

Important advantages of the Clough and Duncan (1971) hyperbolic model are as follows: 1) nonlinearity of the interface shear stress-displacement relationship is well represented by Equations 2-8 or 2-11, 2) the hyperbolic parameters have a clear physical meaning, and 3) the method is easy to implement in SSI analyses.

The Clough and Duncan (1971) hyperbolic model for interfaces has some important limitations regarding its use in SSI analyses of earth retaining structures, such as lock walls. The hyperbolic formulation does not model displacement softening of the interface and does not include any coupling effects between shear and normal displacements. It has not been extended to cases in

which the shear and normal stresses both change, and it has not been fully implemented for cases of cyclic loading and shear stress reversals.

2.3 SSI Analyses of Retaining Walls

2.3.1 Review of previous work

The first systematic SSI analyses of retaining wall behavior were presented by Clough and Duncan (1969, 1971), and Duncan and Clough (1971). They used the hyperbolic constitutive relationship by Duncan and Chang (1970) to model the behavior of the backfill and extended it to model the behavior of the wall-to-soil interfaces. Relative movement at the interfaces was achieved by the use of the joint element developed by Goodman, Taylor, and Brekke (1968).

In their analyses of Port Allen and Old River U-frame locks, Clough and Duncan (1969) and Duncan and Clough (1971) demonstrated the importance of close modeling of the construction stages of the lock and backfill placement. They demonstrated that a simple linear elastic model for the soil and gravity turn-on analyses are not adequate to model the behavior of the soil-lock system. They also proved that the downdrag or vertical shear force exerted by the backfill on the wall has an important influence on the behavior of U-frame locks. Their work provided fundamental understanding of previously unknown aspects of lock wall behavior.

Clough and Duncan (1971) presented a systematic approach to SSI analyses of retaining wall behavior. They observed the importance of modeling the different stages of construction of the wall and placement of the backfill in SSI analyses. They found that when the stages of placement of the backfill were closely modeled, the resulting horizontal and vertical loads acting on the wall were substantially larger than those obtained using classical earth pressure theories. The results of these analyses were consistent with some previous experimental work and field observations.

Ebeling, Duncan, and Clough (1990) performed a comparison between results from conventional equilibrium and finite element analyses of several hypothetical gravity walls founded on rock. Their analyses were performed with the option of backfill placement analysis incorporated in SOILSTRUCT (Clough and Duncan 1969). A range of possible values of shear stiffness was assumed at the interfaces between the wall and the backfill, and between the backfill and the rock. Ebeling, Duncan, and Clough (1990) concluded that the magnitude of downdrag force is significantly affected by the concrete-to-backfill and rock-to-backfill shear stiffness values. They also concluded that conventional equilibrium analyses neglect the true process of soil-structure interaction and tend to yield very conservative results.

Ebeling et al. (1992) performed analyses of several hypothetical gravity walls founded on rock. The hypothetical walls were based on several representative examples of existing lock walls. Ebeling et al. (1992) found that conventional equilibrium analyses are very conservative because they do not account for the stabilizing effect of the downdrag forces generated by settlement of the backfill. At the time of this study, it was not known whether these vertical shear forces persisted under field conditions, and if they could be relied upon for the stability of the structure. Ebeling et al. (1992) also indicated that the behavior of retaining structures founded on soil might differ substantially from that of structures founded on rock. In soil-founded structures, the concrete-to-foundation interface is not bonded as in the case of concrete-to-rock interfaces, and greater relative interface displacements may occur, inducing more redistribution of the earth pressures.

Ebeling et al. (1993), Ebeling and Mosher (1996), and Ebeling, Peters, and Mosher (1997) presented the results of extensive SSI analyses for the soil-founded Red River Lock and Dam No. 1. A reinforced soil berm was recommended, among other alternatives, as a solution to problems induced by siltation of the lock. The SSI analysis procedures were validated against instrumentation measurements from the lock taken at the end of construction and at several operational stages. Their analyses revealed that important changes in normal stresses may occur at the soil-to-structure interface during backfill placement and operation of the lock, and underscored the importance of selecting the appropriate interface stiffness values for these loading conditions. They also noted that conventional equilibrium analyses are inadequate for the design of this type of structure.

A simplified procedure was presented in Appendix F of EM 1110-2-2100 (HQUSACE, in preparation)¹ for evaluating the downdrag force on retaining walls founded on rock. This procedure is described in detail in the following section. It was observed that measurements in existing lock walls, as well as previous experimental data from the IRW facility at Virginia Tech (Filz, 1992), showed that downdrag forces were significant and tended to increase with time. For the case of U-frame locks and retaining structures founded on soil, the EM recommended performing complete SSI analyses. The simplified procedure has also been described in detail by Ebeling, Pace, and Morrison (1997).

Ebeling and Wahl (1997) presented the results of SSI analyses of the proposed North Lock wall at McAlpine Locks. They determined that the downdrag force was significant, and that it could be substantially affected by the response mode of the interface to unload-reload cycles. Further details were given in Chapter 1 of this report.

¹ Starting in 1997, the Headquarters, U.S. Army Corps of Engineers (HQUSACE) began to revise and consolidate their guidance on stability criteria for concrete gravity dams and other hydraulic structures. The Corps guidance contained in this technical report is based on the summer 1999 draft of this guidance (Engineer Manual (EM) 1110-2-2100). This Corps draft guidance has undergone peer review by District engineers as EC 1110-2-291. EM 1110-2-2100 is in the final stages of preparation at the time of publication of this report.

Filz and Duncan (1997) and Filz, Duncan, and Ebeling (1997) presented a theory for the quantification of the downdrag forces on the back of nonmoving retaining walls and described previous large-scale retaining wall tests and field measurements. They observed that post-construction settlement causes an increase in the downdrag on the back of the wall. They cited measurements at Eibach Lock in Germany, where large vertical shear forces were persistent for 10 years under repeated filling and emptying cycles and temperature fluctuations. The measured K_v remained at an approximately constant average value of 0.30. These vertical shear forces cause an important reduction in the lateral earth pressures acting on the wall.

2.3.2 Simplified procedure for calculating the downdrag force

There are some cases in which it is possible to estimate the downdrag force acting on the back of a retaining wall without performing sophisticated SSI analyses. In this section, a simplified procedure is presented for calculating the downdrag force as described by Ebeling, Pace, and Morrison (1997). It applies to retaining walls with nonyielding backfills. This is the case for rock-founded gravity retaining walls with engineered backfills that do not creep, such as soils classified as SW, SP, GW, and GP according to the Unified Soil Classification System (American Society for Testing and Materials (ASTM) 1990). It also applies to select SM backfills with nonplastic fines that do not creep. This method is based on the results of analyses performed on walls with geometrical configurations that are representative of many, but not all, Corps of Engineers rock-founded lock walls.

The simplified procedure was first reported in Engineer Technical Letter (ETL) 1110-2-352 (HQUSACE 1994). Ebeling, Pace, and Morrison (1997) presented an improved version of the original procedure, based on additional SSI analyses on rock-founded gravity walls (Filz, Duncan, and Ebeling 1997; Ebeling and Filz, in preparation). It is also described in Appendix F of EM 1110-2-2100 (HQUSACE, in preparation). This improved version is applicable to situations in which there is no water table behind the wall or when the groundwater level rises as the backfill is being placed.

The simplified procedure is based on the use of the vertical earth pressure coefficient K_v , defined in Chapter 1 of this report. Combining Equations 1-1 and 1-3, the vertical shear force F_v can be expressed as:

$$F_v = K_v \cdot \int_{heel}^{top} \sigma'_v dy \quad (2-12)$$

Figure 2-4 shows the vertical force F_v acting on a vertical plane, passing through the heel of a retaining wall with a hydrostatic water table. It also shows the diagram of vertical effective stress in the backfill. For this case, the vertical

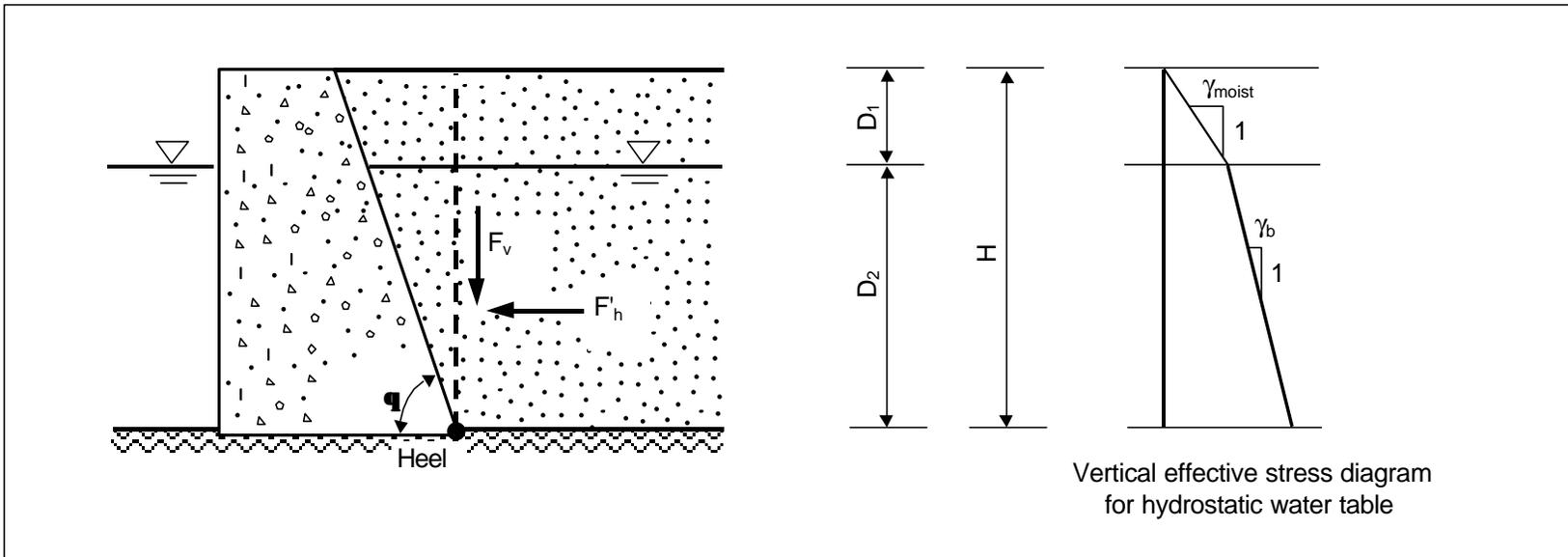


Figure 2-4. Vertical and effective horizontal earth pressure forces on vertical plane extending from through the backfill from the heel of the monolith. Adadaped from ETL 1110-2-352 (HQUSACE 1994)

shear force can be expressed using the equation presented in ETL 1110-2-352 (HQUSACE 1994):

$$F_v = K_v \cdot \left[\frac{1}{2} \gamma_{moist} (D_1)^2 + \gamma_{moist} (D_1 D_2) + \frac{1}{2} \gamma_b (D_2)^2 \right] \quad (2-13)$$

where

γ_{moist} = moist unit weight of the backfill above the water table

D_1 = thickness of the backfill above the hydrostatic water table

D_2 = thickness of the submerged backfill above the heel of the wall

γ_b = buoyant unit weight of the submerged backfill.

Equation 2-13 is valid for a horizontal backfill with no surcharge loads applied. Furthermore, Equation 2-13 applies only when the backfill is placed in a submerged condition or the groundwater level in the backfill rises concurrently with backfill placement. The case in which the groundwater level rises after construction is discussed later in this section. Filz, Duncan, and Ebeling (1997) and Ebeling and Filz (in preparation) expanded Equation 2-13 to include the effects of surcharge and sloping backfill. In the case of rock-founded gravity walls with the inclined backfill surface shown in Figure 2-5a, F_v is calculated using:

$$F_v = F_{v,soil} + F_{v,q} \quad (2-14)$$

where

$$F_{v,soil} = K_{v,soil} \cdot \left[\frac{1}{2} \gamma_{moist} (D_1)^2 + \gamma_{moist} (D_1 D_2) + \frac{1}{2} \gamma_b (D_2)^2 \right] \quad (2-15)$$

and

$$F_{v,q} = K_{v,q} \cdot q_s \cdot H \quad (2-16)$$

where

$K_{v,soil}$ = the vertical shear force coefficient for self-weight of the backfill

$K_{v,q}$ = the vertical shear force coefficient for sloping backfill and surcharge

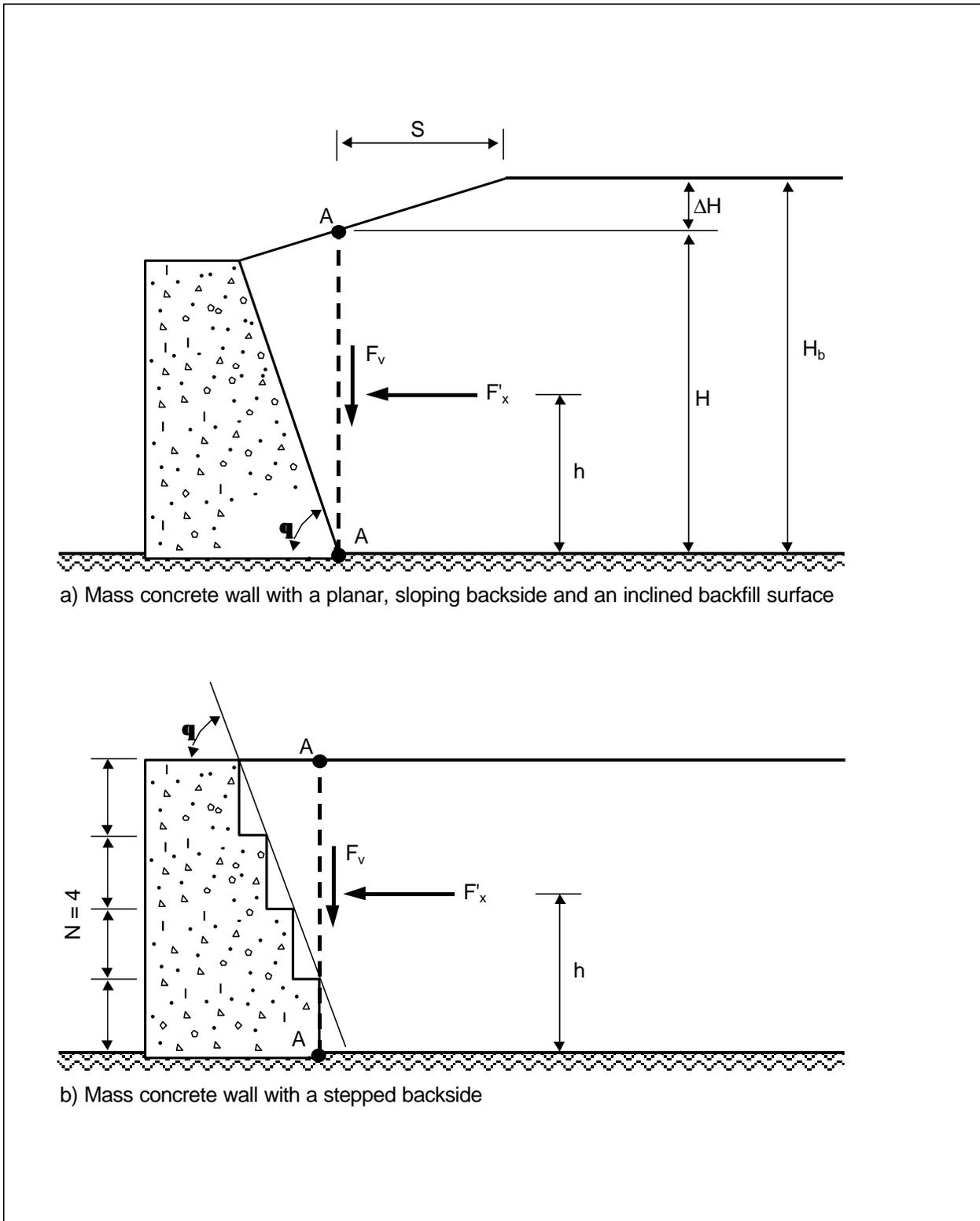


Figure 2-5. Rock-founded retaining wall definition sketches (adapted from Filz, Duncan, and Ebeling 1997)

q_s = applied surcharge pressure

H = height measured along the vertical plane A-A extending through the backfill, as indicated in Figure 2-5a.

The surcharge q_s can be determined from:

$$q_s = \Delta H \cdot \gamma_{moist} = [H_b - H] \cdot \gamma_{moist} \quad (2-17)$$

where H_b is the total backfill height measured as illustrated in the figure. The vertical shear force coefficient for self-weight of the backfill $K_{v,soil}$ is computed using:

$$K_{v,soil} = (1 - C_\theta \cdot C_N) \cdot K_{v,soil,ref} \quad (2-18)$$

where

C_θ = correction factor for inclination of the backside of a rock-founded gravity wall

C_N = correction factor for the number of steps in the backside of a rock-founded gravity wall

$K_{v,soil,ref}$ = reference value of $K_{v,soil}$ obtained for an inclination of the back of the wall, θ , of 90 degrees

Calculation of the value for the number of steps in the back of a stepped wall N is shown in Figure 2-5b.

The vertical shear force coefficient for sloping backfill and surcharge $K_{v,q}$ is given by:

$$K_{v,q} = C_S \cdot K_{v,q,ref} \quad (2-19)$$

where

C_S = correction factor for a rock-founded gravity retaining wall with an inclined backfill surface

$K_{v,q,ref}$ = reference value of $K_{v,q}$ obtained for a value of $S = 0$

S = horizontal distance from the vertical plane through the wall heel to the top of the backfill slope, as shown in Figure 2-5a

Given the density of the backfill and the height H as defined in Figure 2-5a, values for $K_{v,soil,ref}$ and $K_{v,q,ref}$ are obtained from Figures 2-6 and 2-7, respectively, using the curves designated as "design" curves. The data designated as "FEM" are based on the results of complete soil-structure interaction analyses using SOILSTRUCT-ALPHA (Ebeling and Filz, in preparation; or Filz, Duncan, and Ebeling 1997) and are for reference only. Correction factors C_θ , C_N , and C_S are given in Figure 2-8.

Filz, Duncan, and Ebeling (1997) presented a complete example calculation for F_v , using this simplified procedure for a 9-m- (30-ft-) high, step-tapered, rock-founded gravity wall. The wall was backfilled with dense sand and was subject to surcharge loading. No ground water table was present in the backfill. In this example, a 14 percent reduction in base width was obtained by including F_v in the analyses without compromising the design safety requirements. This illustrates the impact of including the downdrag force F_v in equilibrium calculations of a rock-founded gravity wall.

As pointed out by Ebeling, Pace, and Morrison (1997), a rebound of the backfill can occur during a post-construction rise in the groundwater level behind the wall. This may result in a reduction in the shear force F_v , as reported by Ebeling et al. (1993) and Ebeling and Mosher (1996) from their analyses of the Red River Lock No. 1. Ebeling, Pace, and Morrison (1997) indicated that, in this case, SOILSTRUCT-ALPHA (Ebeling, Duncan, and Clough 1990; Ebeling et al., 1992) can be used to perform the necessary SSI analyses to calculate F_v . These analyses must include the rise in water table and the corresponding "unloading" of the backfill (Ebeling et al. 1993; Ebeling and Mosher 1996). The work performed by Ebeling and Wahl (1997) for the new roller-compacted concrete lock at McAlpine Locks is a good example of this type of analysis.

The simplified procedure for the calculation of the downdrag force was extended to include a post-construction rise in the water table behind the wall as shown in Appendix F of EM 1110-2-2100 (HQUSACE, in preparation). As illustrated in Figure 1-2b, a post-construction rise in the water table may induce rebound of the backfill, with the consequent reduction in the magnitude of the vertical shear force. The vertical shear force coefficient for backfill inundation can be calculated from:

$$K_v = K_{v, soil} \cdot C_{wt} \quad (2-20)$$

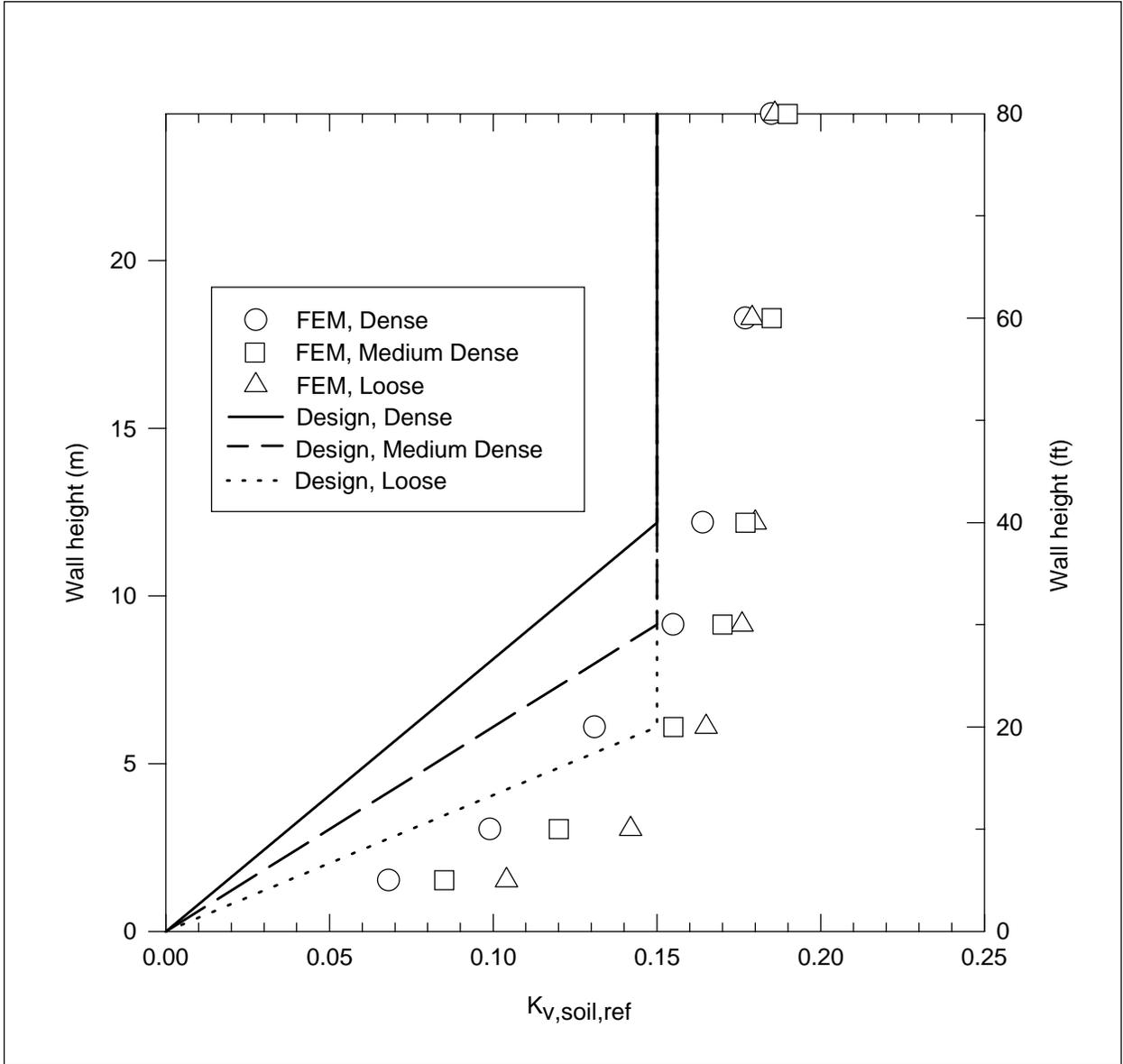


Figure 2-6. Values of $K_{v,soil,ref}$ recommended for design (adapted from Filz, Duncan, and Ebeling, 1997)

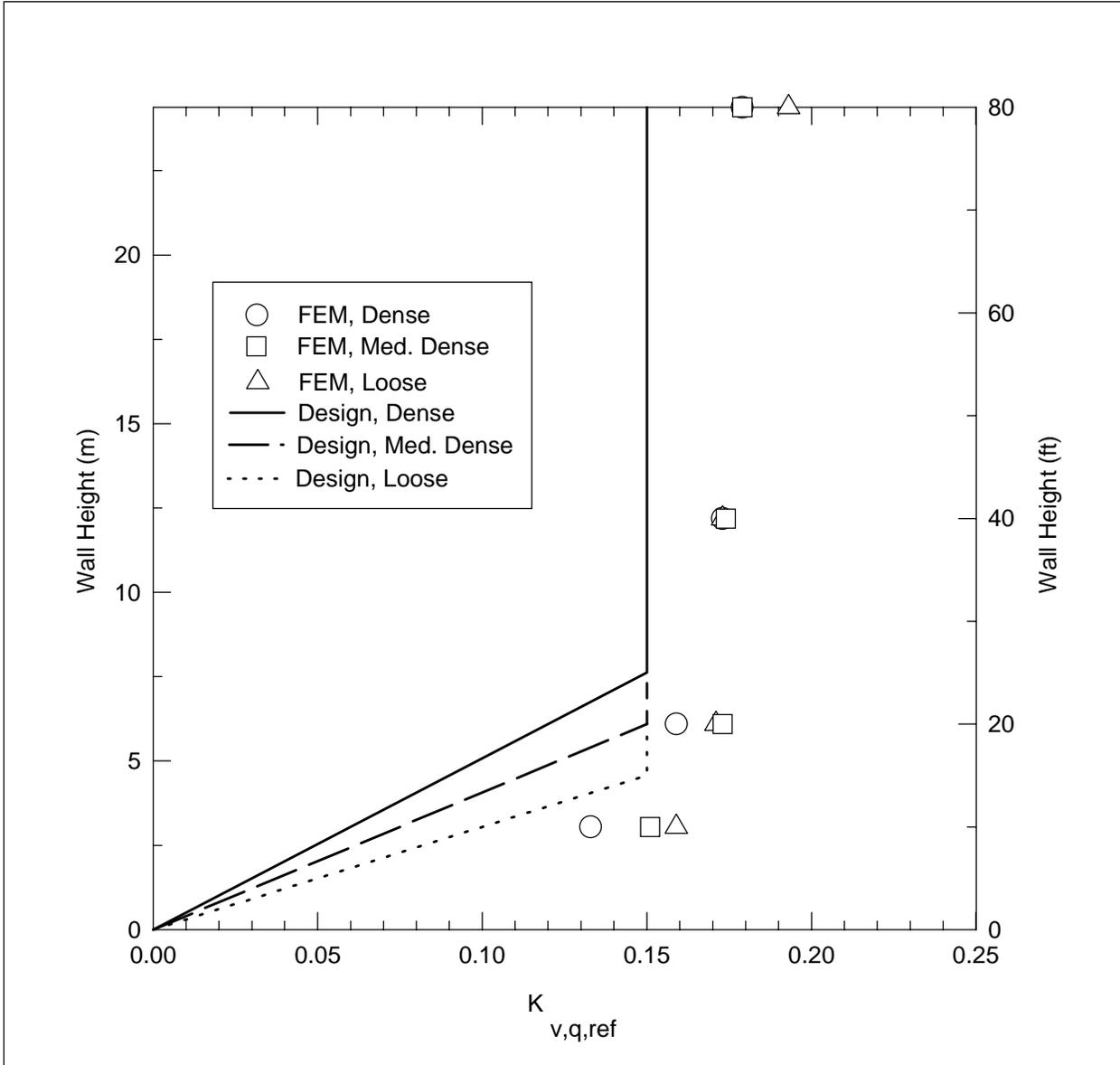


Figure 2-7. Values of $K_{v,q,ref}$ recommended for design (Adapted from Filz, Duncan, and Ebeling, 1997)

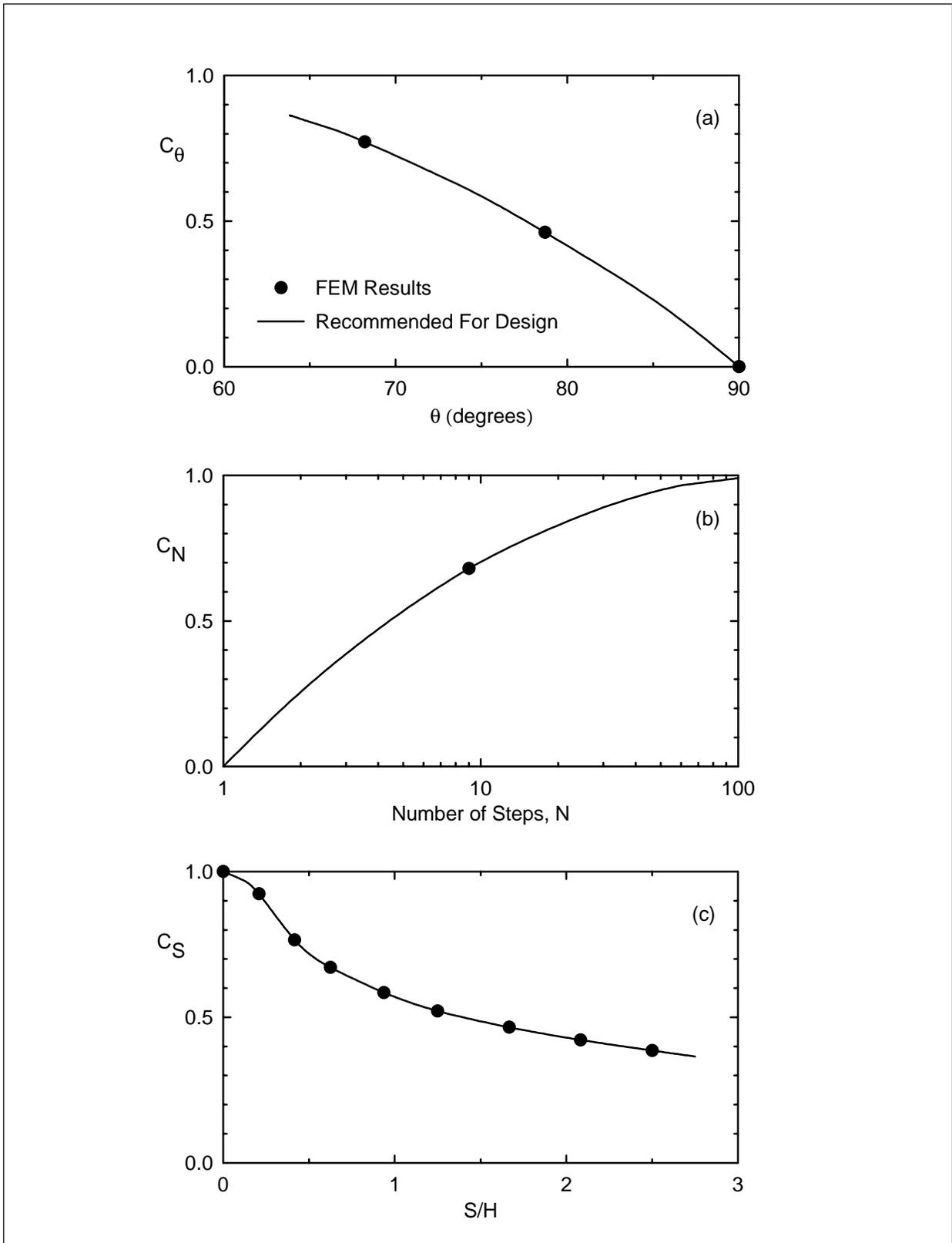


Figure 2-8. Values of the correction factors C_θ , C_N , and C_S (adapted from Filz, Duncan, and Ebeling, 1997)

where C_{wt} is the correction factor for submergence of the backfill, which can be estimated from the following expression:

$$C_{wt} = \left(1 - \frac{D_2}{H} \right) \quad (2-21)$$

For the application of this method, the value of $K_{v, soil}$ for a "dry" backfill is calculated from Equation 2-18, and corrected using Equations 2-20 and 2-21. Equations 2-20 and 2-21 underestimate K_v if a post-construction rise in the water table takes place in a backfill that was placed partially submerged, or if there was a concurrent rise in the water table during backfill placement.

Figure 2-9 is a graphical representation of Equation 2-21. It includes the results of a finite element analysis of the roller-compacted lock at McAlpine Locks (Ebeling and Wahl 1997). It can be seen that Equation 2-21 provides a conservative estimate of the value of the correction factor C_{wt} . This was confirmed by the results of the lock wall simulation performed for this investigation, as discussed in Chapter 5.

2.4 Summary

A literature review was carried out that included previous work on interface testing, interface modeling, and SSI analyses of retaining walls. The review focused on the most relevant issues for this investigation.

In the experimental work reviewed, the direct shear box (DSB) and the direct simple shear (DSS) are the devices most frequently used for testing of sand-to-concrete and sand-to-steel interfaces. Most of the previous work on interfaces investigated monotonic shear of the interface under constant normal stress. Some investigations have been published regarding cyclic shear of interfaces under conditions of constant normal stress or constant normal stiffness. No previous studies were found on interface response under staged shear.

All of the interface testing devices described in the literature present limitations. The interface sizes are limited and do not allow the determination of the residual interface strength in all cases. In addition, end effects may be present, inducing errors in the measurement of the pre-peak and peak interface response. The Large Direct Shear Box (LDSB) at Virginia Tech allows testing of interfaces as large as 711 by 406 mm under monotonic or cyclic shear. The size of the interface minimizes end effects and permits maximum interface displacements of 305 mm, allowing the determination of the residual interface strength. The large displacement capabilities of the LDSB also make possible shearing of the interface in several stages with changing normal stress.

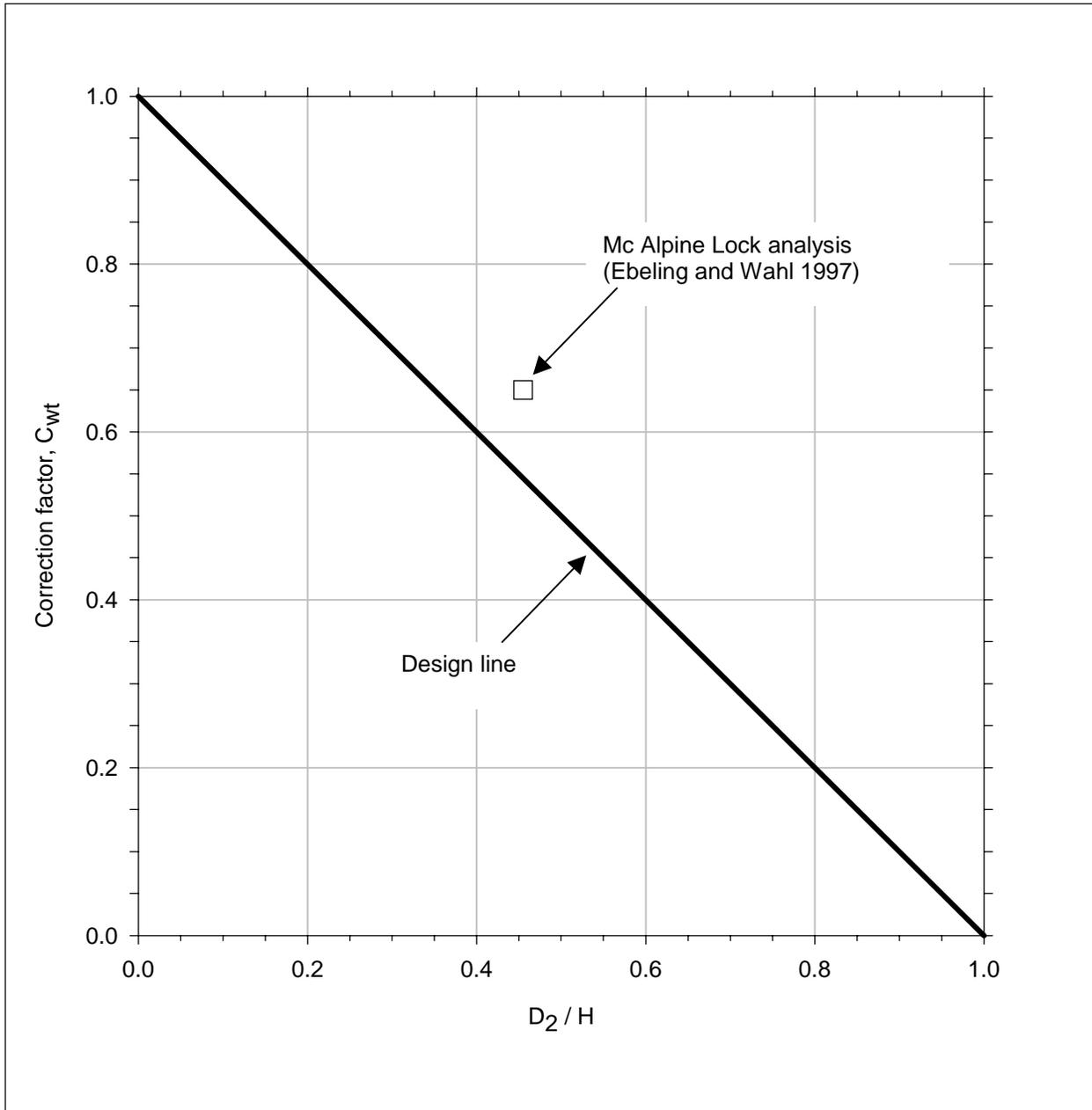


Figure 2-9. Values of the correction factor C_{wt} recommended for design. Adapted from EM 1110-2-2100 (HQUSACE, in preparation)

Two types of elements are commonly implemented for modeling interfaces: the joint element and the thin layer element. The joint element, developed by Goodman, Taylor, and Brekke (1968), appears to be used most frequently due to the simplicity of its formulation.

Several models of interface response under shearing have been described in the literature. The hyperbolic formulation by Clough and Duncan (1971) was described in detail in this chapter. It has been widely used for modeling the interface response to monotonic shear under constant normal stress. It is a simple model that incorporates the most important aspects of interface behavior using parameters that have physical meaning. However, the Clough and Duncan (1971) hyperbolic formulation was not developed to model the interface response under cyclic loading or staged shear. None of the other interface models found in this literature review accounts for simultaneous changes in shear and normal stresses.

Several studies have been published regarding SSI analyses of retaining structures. From these studies, it may be concluded that the downdrag force acting on the back of a retaining wall can contribute significantly to the stability of the structure. In typical lock walls, the downdrag develops during fill placement. During this stage, the shear and normal stresses acting on the backfill-to-structure interface are changing simultaneously. During submergence and operation of the lock, the shear stresses may be reduced or even reversed. Hence, it is important to model accurately the interface response under staged shear, unloading-reloading, and shear reversals.

A detailed description of a simplified method described in Appendix F of EM 1110-2-2100 (HQUSACE, in preparation) to estimate the downdrag force was presented in this chapter. It is based on a number of SSI analyses of typical lock structures. The simplified method is useful to illustrate the importance of an adequate estimation of the downdrag force in design.

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