

CHARACTERIZATION OF FRICTIONAL INTERFERENCE IN CLOSELY-SPACED REINFORCEMENTS IN MSE WALLS

FINAL PROJECT REPORT

by

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Executive Summary

The use of reinforced earth in the United States began in 1972; since then, Mechanically Stabilized Earth (MSE) walls have grown in popularity, and can be found along nearly every state and interstate highway corridor. Due to their inherent flexibility, MSE walls are being constructed to greater heights, in non-linear geometries, with multiple tiers and with very tight reinforcement spacing. For example, the four-tier West MSE wall at Sea-Tac International Airport was recently constructed to 46 m height, and is now the tallest wall in the Western Hemisphere (Stuedlein et al. 2010a). Tall walls (i.e., greater than 15 m in height) will proliferate due to increasing urbanization, right-of-way issues, and wetland mitigation; in other words, they offer a sustainable alternative to conventional grade separation, due to reduced mining and hauling of earth materials and reduced footprint. However, our understanding of the working stress behavior, including reinforcement strains and displacements, of tall, single and multi-tier walls is unsatisfactory. The research proposed herein aims to address one of several knowledge gaps in the understanding of tall MSE wall behavior: prediction of reinforcement loads impacted by frictional interference of closely-spaced reinforcements associated with tall walls and/or walls in seismically active regions.

The goal of this research is to characterize frictional performance and possible interference in closely-spaced steel strip reinforcements used in MSE wall construction. The research performed investigated the effect of soil-reinforcement interface behavior on the working stress behavior of tall, single and multi-tiered MSE walls. In order to substantiate the hypothesis of frictional interference, high-quality full-scale experimental data was required. The investigation required the following interrelated research tasks:

1. Characterization of the constitutive behavior a sandy gravel backfill soil and ribbed steel strip reinforcement;
2. Characterization of the soil-reinforcement interface behavior of single steel strip reinforcements using instrumented pull-out tests; and,
3. Characterization of the effect of spacing on the shearing resistance of reinforcements using instrumented multi-strip pull-out tests.

Key findings of this research include:

- New backfill-specific and gravelly-soil models for the prediction of the apparent friction coefficient with normal effective stress were developed for use with ribbed steel strip reinforcements.
- The performance of the proposed pullout resistance design models were compared to the current AASHTO design model and found to produce significantly more accurate predictions of the apparent friction coefficient.
- Preliminary test results indicate that frictional interference appear to exist in the vertical direction at vertical pressures of 100 kPa and may increase the peak pullout loads of reinforcement strips as much as 40% at a spacing of 152 mm.

Chapter 1 Introduction

1.1 Statement of Problem

As densely populated metropolitan areas continue to grow, right-of-way available for public infrastructure development becomes constricted. These pressures from urban expansion have resulted in a dramatic increase in the number of retaining walls being constructed around the world. Of the various types of retaining wall technologies currently used, Mechanically Stabilized Earth (MSE) walls are often the most cost-effective and reliable option, with thousands of walls being built since their introduction over 40 years ago (Christopher et al. 1990a). Surprisingly, few well-documented case histories with detailed material testing, instrumentation programs, and construction observation of performance are available in the literature. Despite the small number of case histories, empirical design methods are used in place of more theoretically-based methods. As a result, current design methods for MSE walls result in a large amount of inaccuracy, especially when their empirical calibration limits are exceeded.

1.2 Purpose and Scope

This study focuses on the internal stability of ribbed steel strip MSE walls used with gravels and sand-gravel mixtures by assessing the stiffness, strength, and volumetric response of a sandy gravel fill, and the reinforcement-backfill interaction developed during reinforcement strip pullout for single, isolated reinforcement strips and closely-spaced reinforcements.

An extensive laboratory testing program was performed on sandy gravel backfill and steel reinforcement strips used in the construction of two very tall MSE Walls. The purpose of the laboratory testing program was to understand the constitutive behavior of the soil backfill and its role in soil-reinforcement interaction.

The soil-reinforcement interaction of single reinforcement strips was evaluated using a newly constructed pullout test apparatus. Twenty pullout tests were performed on ribbed steel reinforcement strips embedded within sandy gravel to characterize the soil-reinforcement interaction behavior. The effect of the relative compaction and vertical effective stress acting on the reinforcement strip was assessed using multiple linear regression modeling. A backfill-specific pullout resistance design model was generated for use in predicting pullout capacities of ribbed steel reinforcement strips in the backfill evaluated. The pullout test results were then combined with data from other pullout tests results on gravels and sand-gravel mixtures found in the literature in order to create a global gravel pullout resistance model. The backfill-specific, global gravel, and standard pullout resistance models were used to assess the effect of model on pullout length calculations.

In order to evaluate the soil-reinforcement interaction of closely-spaced reinforcement strips, a new multi-strip pullout test apparatus was designed and constructed. The interaction between closely-spaced reinforcement strips was evaluated over a large range of normal stresses and reinforcement spacing, and point to potentially significant frictional interaction for closely spaced reinforcement strips.

1.3 Organization of this Report

Chapter 2 presents a literature review on granular soil mechanics and the internal stability of MSE walls, with emphasis on the development of design methodologies and the assumptions required for their use and implementation. Chapter 3 describes the research objectives and research program developed to achieve the objectives. Chapter 4 presents the comprehensive results of the characterization of the index properties and constitutive behavior of the steel reinforcement and granular backfill selected for evaluation. Chapter 5 describes the new single-strip pullout apparatus and the results of pullout tests conducted to form the baseline response of soil-reinforcement interaction for single strips. Chapter 6 describes the new 4 m³ multi-strip reinforcement pullout test apparatus and the preliminary results for closely-spaced reinforcements. This report concludes with a summary of work performed and the findings (Chapter 7).

Chapter 2 Literature Review

The concept of modern Mechanically Stabilized Earth (MSE) walls with was first developed by Vidal (1969) and used steel reinforcements. Since Vidal's initial work, which began over 50 years ago, the understanding of the behavior and design of MSE walls has grown extensively. As with many geotechnical design procedures, MSE wall design is based on empirical performance observations in consideration of soil mechanics principles.

An understanding of the history and development of design procedures for MSE walls is necessary to identify those areas where improvements can be made. However, to gain a deeper understanding of the basic mechanics governing the behavior of MSE walls, a firm understanding of the soil mechanics principles at work is needed. MSE walls built with steel strip reinforcements rely significantly on frictional resistance and therefore are almost always designed using granular reinforced backfill. Therefore soil mechanics principles relevant to granular soil-reinforcement interaction will be reviewed in Section 2.1, and general MSE wall design procedures will be covered in Section 2.2.

2.1 Behavior of Drained Granular Materials

The response of drained granular soils varies based on factors that include relative density, confining stress, gradation, angularity, surface roughness and boundary conditions, among others (Rowe 1962; Lee and Seed 1967; Cornforth 1973; Verdugo and Hoz 2007; Bareither et al. 2008; Hashash et al. 2009). The effects of relative density and confining stress are particularly important in the development of reinforcement loads and resistance, and therefore will be the focus of this Section.

2.1.1 Mohr-Coulomb Failure Criterion

The stresses on any plane within an element can be represented by a Mohr circle in stress-stress space as shown in Figure 0.1. The points at which the Mohr circle crosses the normal stress axis represent planes of zero shear stress and are termed principal stresses. For the purpose of discussion it is convenient, and common convention, to refer to these stresses as the major principal stress, σ_1 , the intermediate principal stress, σ_2 , and the minor principal stress, σ_3 , where $\sigma_1 \geq \sigma_2 \geq \sigma_3$. MSE walls are most typically constructed in long, linear geometries such that plane strain conditions exist and where $\sigma_2 > \sigma_3$, as shown in Figure 2.1.

The shear strength, s , of a soil mass can be defined by the Mohr-Coulomb failure criterion (Rowe 1962):

$$s = c + \sigma \tan \phi \quad (2.1)$$

where σ_f is the normal stress on the failure plane at failure, ϕ is the friction angle, and c is the apparent cohesion intercept associated with capillarity, and is typically neglected for granular materials because strength associated with seasonal moisture variations cannot be relied upon.

Assuming equal shear resistance on any plane within a soil mass, the Mohr-Coulomb failure criterion can be defined as the shear stress where the Mohr circle becomes tangent to the failure envelope defined by Equation 2.1. Figure 0.2 depicts the failure criterion for the triaxial condition where $\sigma_2 = \sigma_3$.

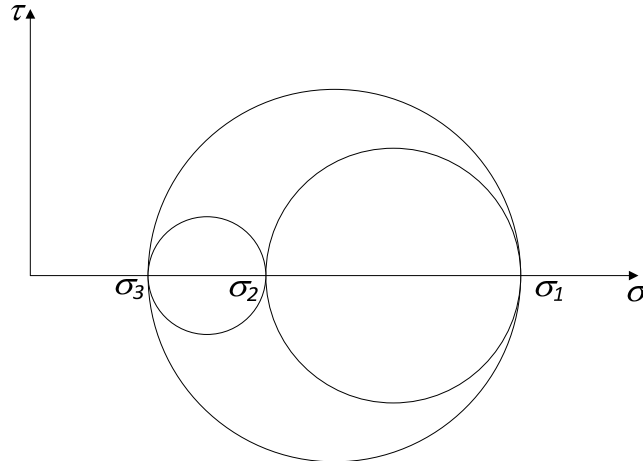


Figure 0.1. Mohr circle diagram in three dimensions.

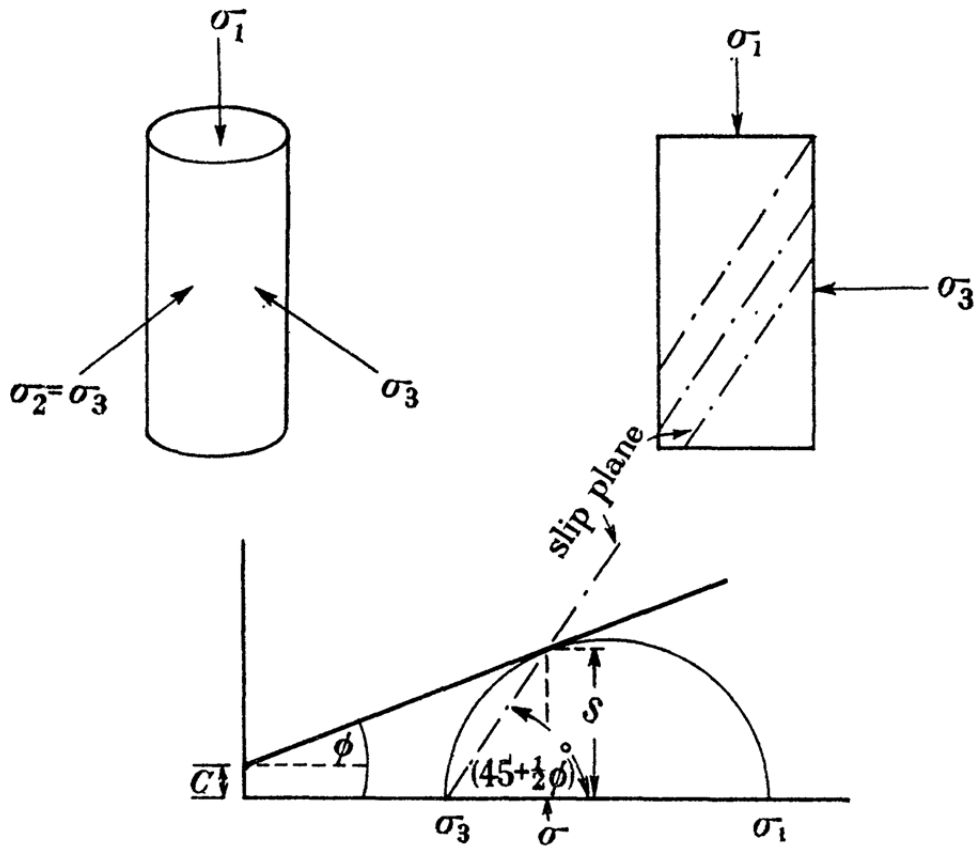


Figure 0.2. Mohr-Coulomb failure criterion for the triaxial condition (after Rowe 1962).

Since granular soils are assumed to have no apparent cohesion the strength is strictly defined by the friction angle, ϕ . The consequences of this definition will be explored further in the proceeding sections.

2.1.2 Effect of Relative Density on Response of Granular Materials

Early studies of the shear behavior of granular soils indicated that denser sands tended to increase in volume and looser sands tended to decrease in volume during shearing (Reynolds 1885; Casagrande 1938, 1940). Casagrande (1938) postulated the idea of a critical state void ratio, developed further by Casagrande (1940), where the influence of relative density on the peak friction angle was demonstrated. Casagrande (1940) described the observed response of the sand (Figure 0.3) as follows:

- Samples with initial void ratios less than the critical state void ratio tended to dilate during shearing, a peak friction angle, ϕ_d , was reached, then the shear strength reduced to a residual value where a critical state friction angle, ϕ_r , was observed.
- Samples with initial void ratios larger than the critical state void ratio (looser specimens) tended to contract, and the shear strength would slowly increase, without reaching a peak, to a residual or critical state value of ϕ_r , similar to dense specimens.
- Samples with initial void ratios equal to the critical state void ratio did not change in volume during shearing.

Taylor (1948) and Bishop (1954) attempted to explain the observations of Casagrande (1940) by suggesting that the observed shearing resistance was not entirely due to inter-granular sliding friction, but included a volume change component. The volume change component, as described by Bishop (1954), was quantified as the energy needed to overcome the confining pressure during expansion. Bishop proposed that the portion of the deviator stress required to produce the energy for sample expansion, σ_e , could be calculated as:

$$\sigma_e = \sigma_3 \frac{d\varepsilon_v}{d\varepsilon_1} \quad (2.2)$$

where $d\varepsilon_v$ is the incremental change in volumetric strain, and $d\varepsilon_1$ is the incremental increase in axial strain. Bishop (1954) derived the following expression for the friction angle with the dilation-induced portion removed, ϕ_B :

$$\phi_B = \sin^{-1} \left[\frac{(\sigma_1 - \sigma_e) - \sigma_3}{(\sigma_1 - \sigma_e) + \sigma_3} \right] \quad (2.3)$$

which implies that the difference between ϕ_B and the peak friction angle, ϕ_d , is equivalent to equivalent to the dilation angle.

Rowe (1962) performed direct shear tests, shown in Figure 0.4, on quartz soil particles with the bottom half of the shear box replaced with a quartz block. This enabled Rowe (1962) to

approximate the sliding friction between two particles, ϕ_{μ} . Rowe observed that the friction angle calculated using Bishop's expression to correct for dilation effects (Equation 2.3) was greater than the sliding friction, ϕ_{μ} , measured. This led Rowe to the conclusion that the peak friction angle, ϕ_a , is comprised of three components; the particle-to-particle sliding friction, a dilatant component, and a component related to the work performed by the soil particles as they rearrange and reorient. Rowe (1962) developed the following expression to determine the friction angle with the dilation effects removed, ϕ_f :

$$\phi_f = 2 \left(\tan^{-1} \frac{\sigma_1}{\sigma_3 \left(1 + \frac{d\varepsilon_v}{d\varepsilon_1} \right)} - 45 \right) \quad (2.4)$$

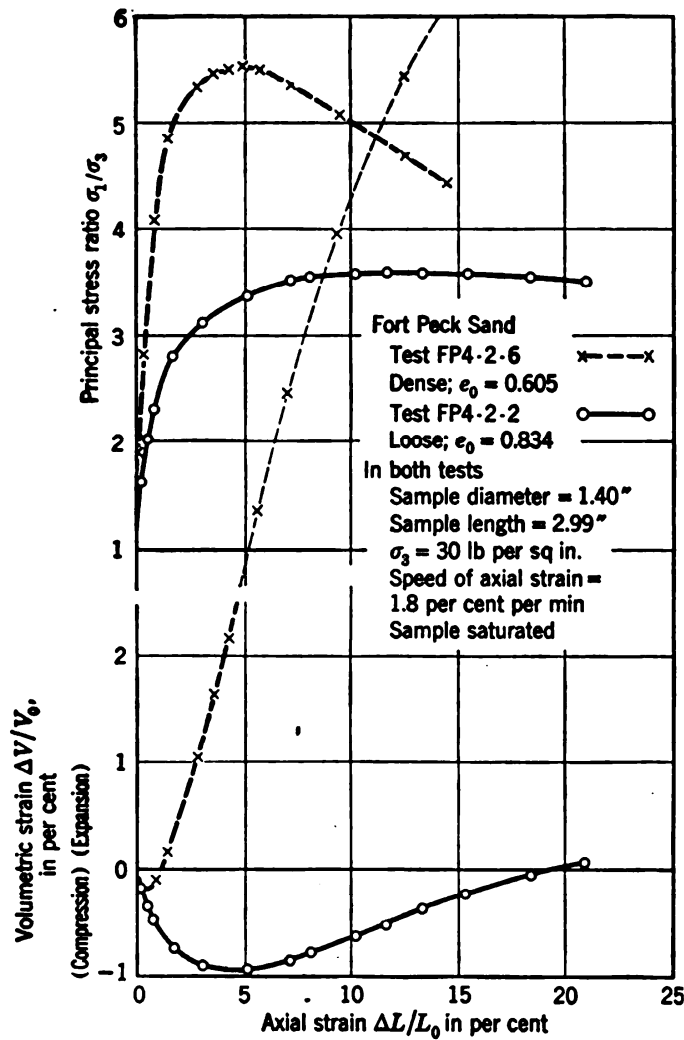


Figure 0.3. Stress-strain and volumetric response for both loose and dense sand in triaxial compression (after Taylor 1948).

As implied by Rowe (1962), the difference between ϕ_d and ϕ_f is the dilation angle, ψ , as illustrated in Figure 0.5. The difference between ϕ_f and ϕ_μ is attributed to reorientation effects, and is described graphically in Figure 0.6. At high void ratios (i.e., high initial porosities), the peak friction angle, ϕ_d , consists of sliding and particle re-orientation effects, as the void ratio decreases, the dilation component increases at a rate greater than the decrease in re-orientation, resulting in a net increase in total or peak friction angle.

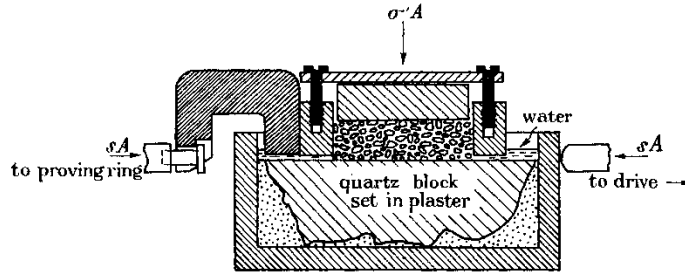


Figure 0.4. Standard shear box modified to record particle friction (after Rowe 1962).

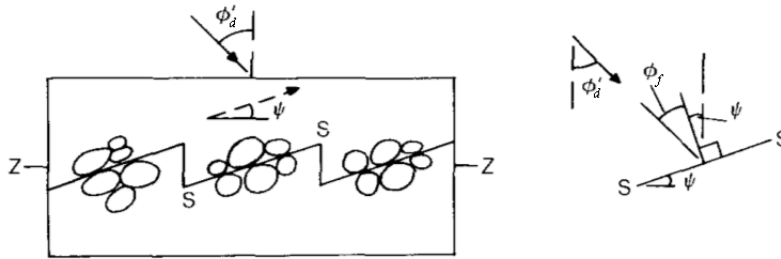


Figure 0.5. Saw tooth model for dilatancy (adapted from Bolton 1986).

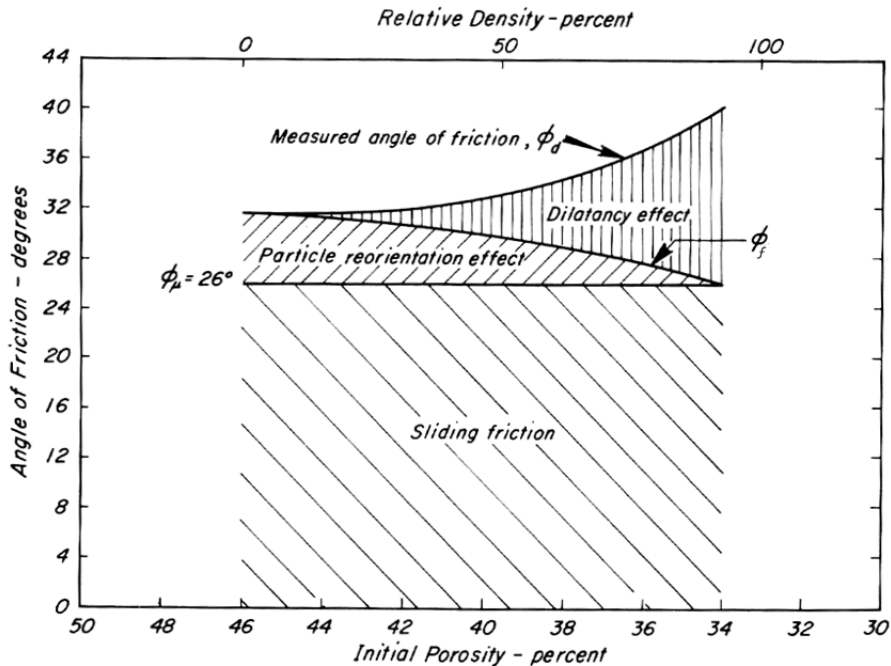


Figure 0.6. Components of friction angle at different relative densities as hypothesized by Rowe (1962) (adapted from (Lee and Seed 1967)).

2.1.3 Effects of Confining Pressure on Shear Strength

The work discussed to this point was performed at relatively low confining pressures. In the mid-1950's through the 1960's there was increased interest in the measurement of the response of granular soils at high confining pressures (Golder and Akroyd 1954; Hall and Gordon 1963; Hirschfeld and Poulos 1963; Vesić and Barksdale 1963; Lee and Seed 1967; Vesić and Clough 1968). These studies produced similar findings: at low confining pressures the Mohr-Coulomb failure envelope was steepest, then, with relatively small increases in confining pressure, peak friction angles reduced to a relatively constant value. Figure 0.7 presents strength test results by Hirschfeld and Poulos (1963) that shows the failure envelope curved at confining pressures exceeding approximately 20 kg/cm² (1.96 MPa); note that this figure does not show tests conducted at lower confining pressures and therefore the initial curvature of the Mohr-Coulomb envelope cannot be seen.

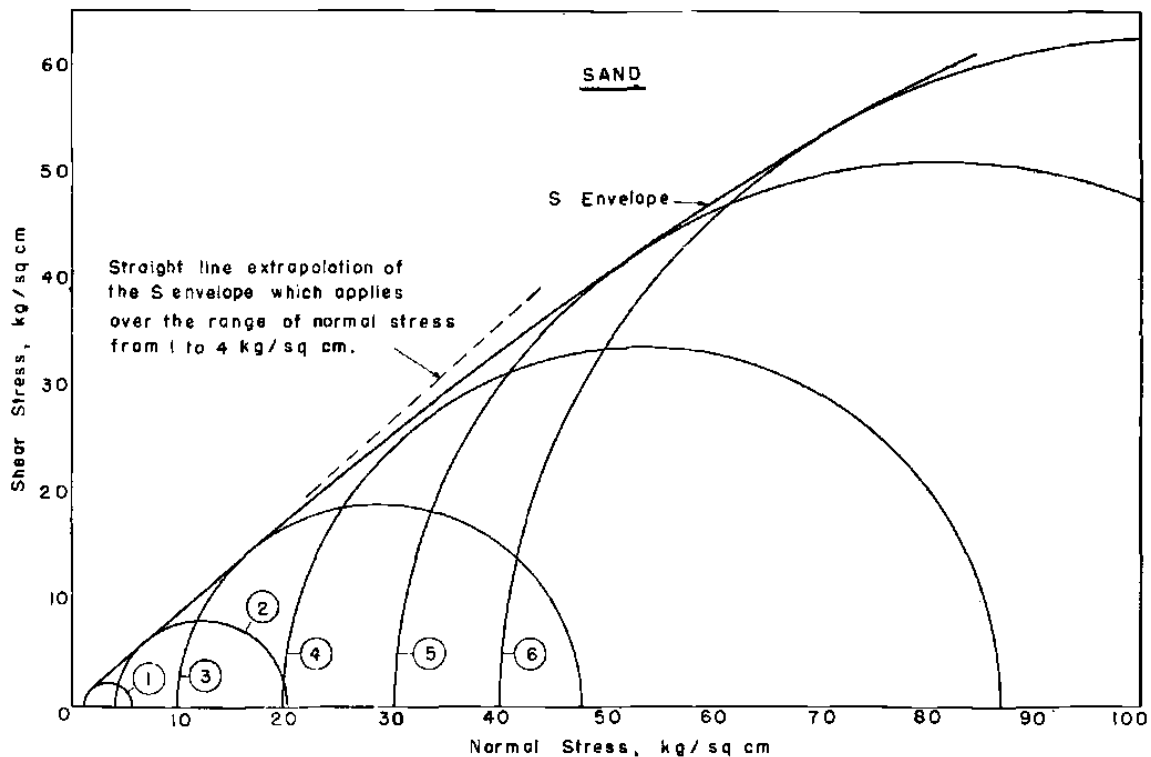


Figure 0.7. Mohr circles at failure showing the curvature of the failure envelope at high pressures (Hirschfeld and Poulos 1963). Note: 1 kg / sq cm = 98 kPa.

The reduction in friction angle at moderate confining pressures was mainly attributed to the suppression of dilation; as the confining pressure increased, soil particles tended to reorient and contract rather than dilate. At extremely large confining pressures, the stresses induced in the soil particles exceed their compressive strength, and particles tended to break apart and fill voids. This crushing behavior resulted in densification and changes in particle characteristics such as surface roughness and angularity which can have complex effects on the Mohr-Coulomb failure envelope. These crushing effects are not discussed herein, because, even at the base of the tallest MSE walls, the compressive strengths of most reinforced backfill particles are not reached.

Figure 0.8, exaggerated for clarity, presents the theoretical influence of dilation, sliding friction, and particle crushing, and particle rearrangement on the drained shear strength of sand (Lee and Seed 1967). The consequence of extrapolation of test data from low confining pressures is apparent; using low confining stresses to estimate ϕ_d for high confining stresses results in non-conservative estimates of ϕ .

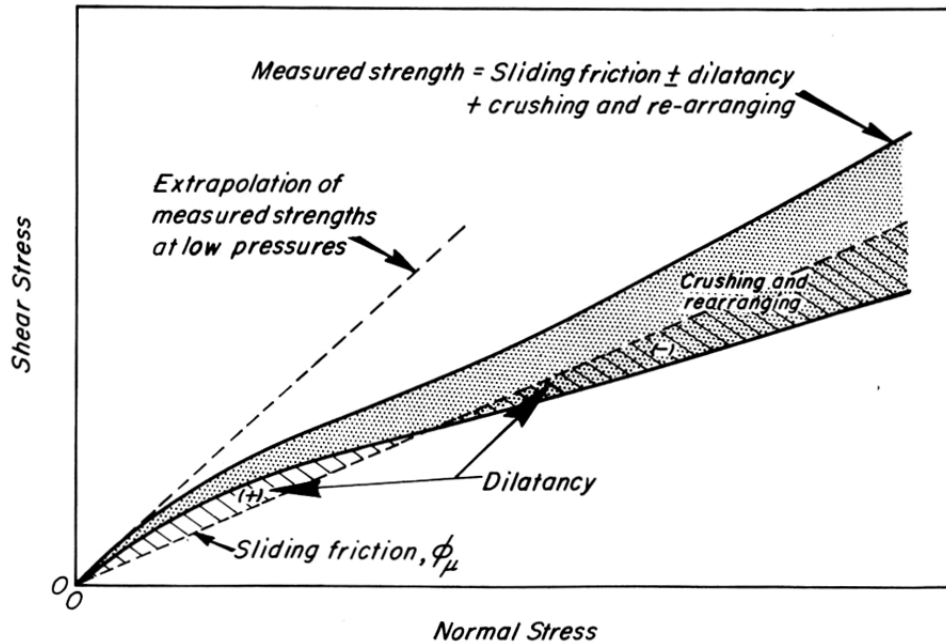


Figure 0.8. Effects of sliding friction, dilatancy and particle crushing on the measured failure envelope for drained sands as theorized by Lee and Seed (1967).

2.1.4 Stiffness of Granular Materials

For accurate predictions of deformation in geotechnical engineering applications, it is necessary to consider the nonlinear and stress-dependent response exhibited by soils. For simplicity, a soil is often assumed to follow a linear elastic-perfectly plastic stress-strain curve with a single elastic modulus and Poisson's ratio. Figure 0.3 shows that granular materials are neither linear, nor elastic; in other words, none of the above assumptions hold for a given soil over any significant range of strains.

For soils, the equivalent Young's modulus changes continuously with strain. The modulus at any strain can be determined using incremental stress analysis to determine the tangent modulus, E_t , as (Duncan and Chang 1970):

$$E_t = \frac{\partial(\sigma_1 - \sigma_3)}{\partial \varepsilon_1} \quad (2.5)$$

A typical measure of soil stiffness is the initial tangent modulus, E_i . As shown in Figure 0.9 E_i increases both with relative density, and confining pressure.

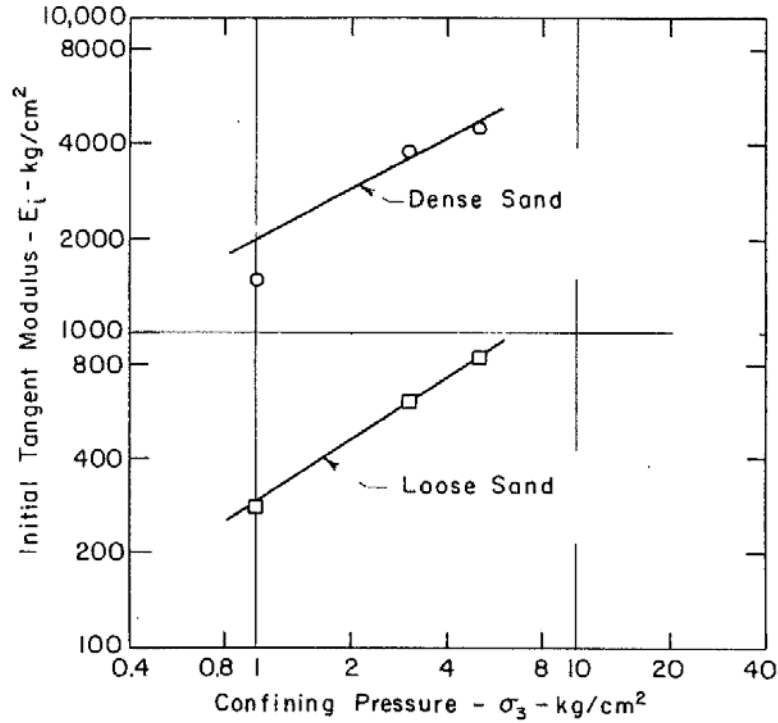


Figure 0.9. Variations of initial tangent modulus with confining pressure and initial relative density for drained triaxial tests on silica sand (after Duncan and Chang 1970).

2.1.5 Constitutive Modeling of Stress–Strain and Volumetric Response of Granular Materials

One of the most widely used nonlinear constitutive models for soils was developed by Duncan and Chang (1970). Some of the advantages of the Duncan and Chang (1970) model over other nonlinear models includes the ability to measure parameters using the ordinary triaxial test, the ability to model the contractive response of soils incorporated by Wong and Duncan (1974), and the availability of the database of parameters for over 150 types of soils including index properties provided in Duncan et al. (1980). The following is a summary of the Duncan et al. (1980) calibration procedure:

For a given drained triaxial test at a confining pressure of σ_3 , the volumetric strain at any point, $\varepsilon_{v,B}$, is determined, as well as the coinciding principal stress difference, $(\sigma_1 - \sigma_3)_B$ (i.e., at the same axial strain). From the principal stress difference-axial strain data, the following are determined: the principal stress difference at failure, $(\sigma_1 - \sigma_3)_f$ and the axial strains corresponding to 70 and 95 percent of the principal stress difference at failure, $\varepsilon_{1,70}$, and $\varepsilon_{1,95}$, respectively. The initial tangent modulus, E_i , is determined by:

$$E_i = \frac{2}{\frac{\varepsilon_{1,70}}{(\sigma_1 - \sigma_3)_{70}} + \frac{\varepsilon_{1,95}}{(\sigma_1 - \sigma_3)_{95}} - \left(\frac{1}{(\sigma_1 - \sigma_3)_{ult}} \right) (\varepsilon_{1,70} + \varepsilon_{1,95})} \quad (2.6)$$

where $(\sigma_1 - \sigma_3)_{70}$ and $(\sigma_1 - \sigma_3)_{95}$ are 70 and 95 percent of the principal stress difference at failure, and $(\sigma_1 - \sigma_3)_{ult}$ is the ultimate principal stress difference defined by:

$$(\sigma_1 - \sigma_3)_{ult} = \frac{\varepsilon_{1,95} - \varepsilon_{1,70}}{\frac{\varepsilon_{1,95}}{(\sigma_1 - \sigma_3)_{95}} - \frac{\varepsilon_{1,70}}{(\sigma_1 - \sigma_3)_{70}}} \quad (2.7)$$

The stiffness of soil when considering only volumetric strains (i.e., bulk modulus), B , is determined by:

$$B = \frac{(\sigma_1 - \sigma_3)_B}{3\varepsilon_{v,B}} \quad (2.8)$$

Once the initial tangent and bulk moduli are determined for several confining pressures, the modulus number, K , and the modulus exponent, n , can be determined by fitting a curve to the initial tangent modulus-confining pressure data using the functional form of a power law:

$$E_i = K \cdot \sigma_3^n \quad (2.9)$$

The bulk modulus number K_B , and the bulk modulus exponent, m , can also be determined by fitting a power law curve to the bulk modulus-confining pressure data using:

$$B = K_B \cdot \sigma_3^m \quad (2.10)$$

Refer to Duncan et al. (1980) for a more comprehensive discussion of the model calibration procedure.

The main disadvantage of the Duncan and Chang (1970) model is that the volume change behavior of dilative soils cannot be adequately simulated. Figure 0.10 shows that the fitted hyperbolic model can simulate the initial contractive behavior of Monterey Sand, but does not model the reversal in volumetric response and subsequent dilation. Additionally, the Duncan and Chang model cannot capture the strain softening behavior of dense sands at low to medium confining pressures. However, in many design situations this strain softening behavior occurs well outside the expected working stress range (i.e., at large stresses) and therefore, may not need to be modeled.

More advanced nonlinear models exist such as the cap-yield (C-y) soil model developed for use in the computer software FLAC. Models such the Cysoil model address some of the problems with using the Duncan-Chang (1970) model. For example, the Cysoil model allows for strain softening, strain hardening, and dilation to be modeled. More sophisticated constitutive models will not be described herein as numerical models are not directly employed within this thesis.

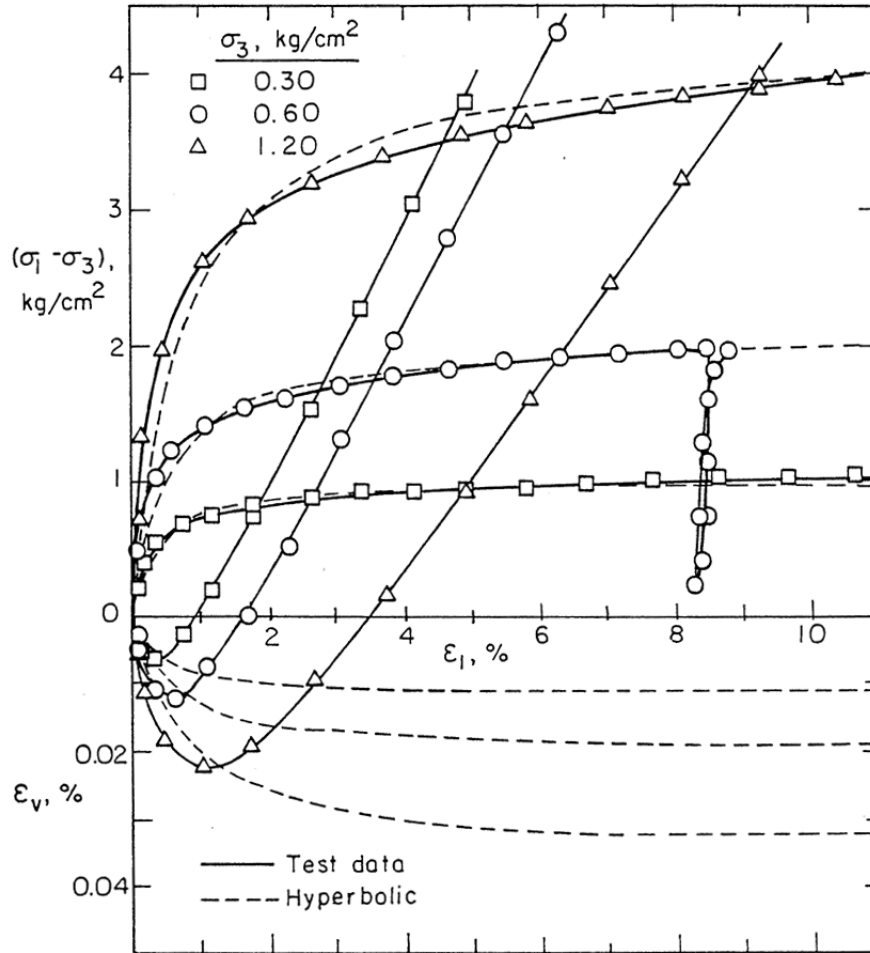


Figure 0.10. Comparison of hyperbolic fit for stress- strain and volume change data for Monterey No. 0 sand (Duncan et al. 1980).

2.2 Design of Mechanically Stabilized Earth Walls

In general, the reinforced soil mass within a MSE wall is subdivided into an active zone and resistant zone. The active zone is the zone closest to the wall facing and tends to move outward and downward away from the resistive zone. However, this movement is resisted by shear forces developed between the reinforcement and soil in the resistive zone as shown in Figure 0.11.

The main design goals for MSE walls include checks for external and internal stability. This section mainly focuses on internal stability as it is the main focus of this study. A complete overview of MSE wall design may be found in Christopher et al. (1990a), Elias et al. (2001), and Berg et al. (2009).

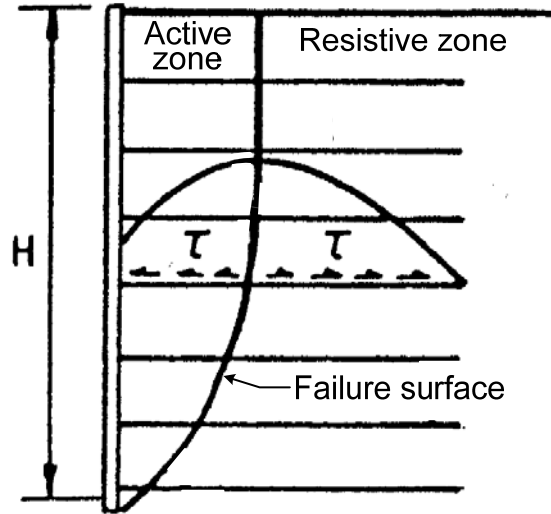


Figure 0.11. Development of shear forces between the backfill and reinforcement within an MSE wall (adapted from Schlosser 1990).

2.2.1 Design Methodology for Internal Stability

Internal instabilities may arise from three possible mechanisms: the shear resistance of the panel-reinforcement connection may be locally exceeded, the tensile stress in the reinforcement strip can exceed the yield stress, resulting in large plastic strains (and associated wall movement) and leading to the eventual rupture of the strip, or the tensile reinforcement forces will exceed the frictional pullout resistance provided to the reinforcement by the soil, resulting in movement relative to the surrounding soil. The three mechanisms of internal instability can result in large deformations and/or collapse of the wall depending on the degree of wall redundancy.

Stability is determined by comparing the maximum predicted tensile load, T_{max} , to the allowable reinforcement tension, T_a , and the maximum pullout resistance of the reinforcement, P_r , at each reinforcement level. The following criteria must be met for the wall to be considered stable:

$$T_{max} = \sigma_h A_t \quad (2.11)$$

$$T_{max} \leq T_a \quad (2.12)$$

$$T_{max} \leq P_r \quad (2.13)$$

where σ_h is the horizontal earth pressure, and A_t is the wall facing tributary area.

Many design methods have been developed for predicting maximum reinforcement loads. The main differences between methods are the way in which the lateral earth pressure coefficient for the reinforced soil mass, K_r , is calculated, and the assumption on where the maximum tension is located behind the retaining wall. These methods include the Coherent Gravity method (Schlosser 1978), the Tieback Wedge method (Lee et al. 1973; Bell et al. 1975, 1983), the

FHWA Structure Stiffness method (Christopher et al. 1990a; Christopher 1993), the working stress method by Ehrlich and Mitchell (1994), the American Association of State Highway Transportation Office (AASHTO) Simplified Method (Allen et al. 2001), and the K-Stiffness method (Allen et al. 2004). Stuedlein et al. (2012) provides a detailed discussion on the differences between many of these methods. Each of the design methods are summarized below.

Coherent Gravity Method (Schlosser 1978)

The maximum tensile reinforcement load is typically used to define the failure surface in the reinforced soil mass, though these are not usually coincident. Early methods of determining this failure surface, such as that by Lee et al. (1973), neglected the influence of the reinforcements on the failure surface and assumed a wedge failure surface that started at the toe of the wall that extends upward at an angle of $45 + \phi/2$ from the horizontal plane. Juran (1977) outlines the theoretical shortcoming of this Rankine-type failure plane. Using laboratory and full-scale testing, Juran and Schlosser (1978) concluded that the presence of inextensible reinforcements in an earth mass affects the failure surface, and that a log spiral-shaped failure surface starting at the toe of the wall and becoming vertical at ground surface would be more appropriate. Schlosser and Elias (1978) modified the work by Juran and Schlosser (1978) by approximating the log spiral surface as a bi-linear curve as shown in Figure 0.12.

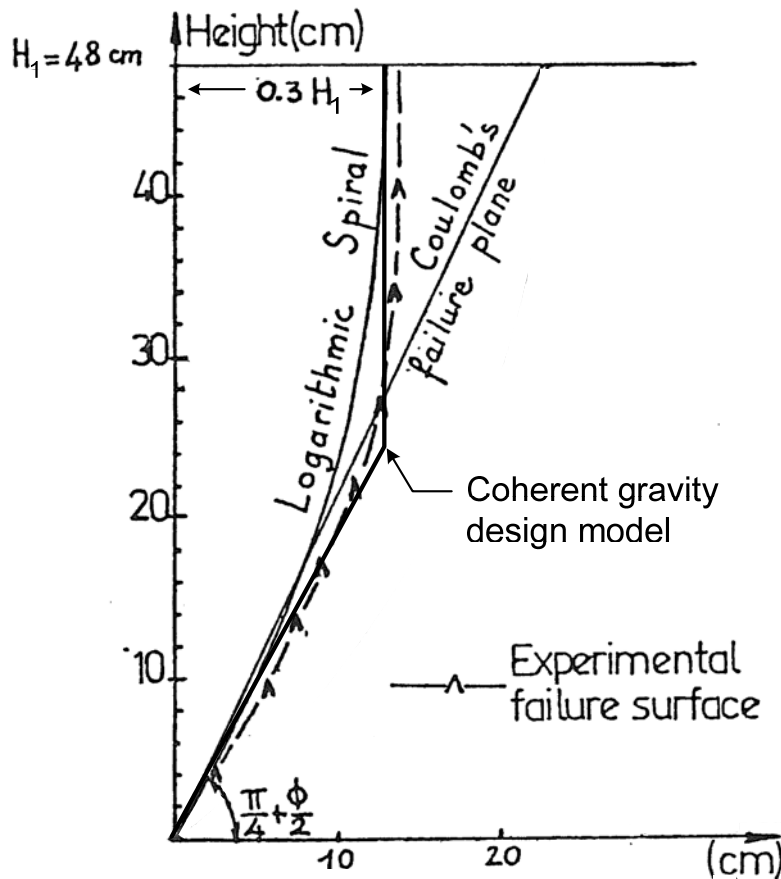


Figure 0.12. Results from scale model tests with theoretical logarithmic spiral failure surface, Coulomb's failure plane, and the coherent gravity design model (adapted from Juran and Schlosser 1978).

Loading in MSE walls is controlled by the earth pressures from the retained soil mass acting on the back of the wall and the influence of wall stiffness, described below. Baquelin (1978) suggested that the at-rest lateral earth pressure would dominate near the top of the wall, and then decrease with depth until the active state was reached at 6 meters below the top of the wall; this was referred to as the coherent gravity distribution. Figure 0.13 shows the design distribution for reinforced fill material with $\phi = 45$ deg. The lateral earth pressure coefficient decreased from an at-rest value, K_o , as defined by Jaky (1948), at the top of the wall:

$$K_o = 1 - \sin \phi \quad (2.14)$$

then linearly decreased to an active value, K_a , as defined by Rankine (1857) at a depth of 6 ft:

$$K_a = \tan^2 \left(45 - \frac{\phi}{2} \right) \quad (2.15)$$

Note that as the friction angle increases, the value of K_o/K_a increases.

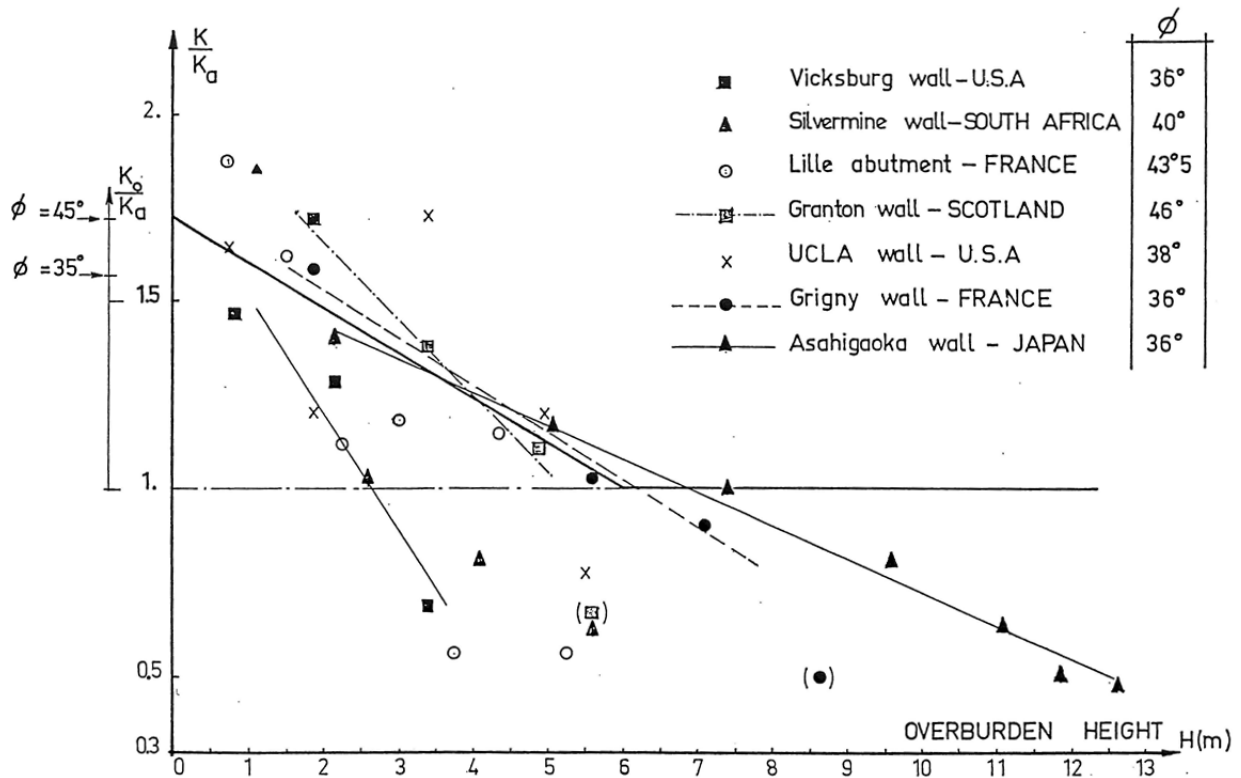


Figure 0.13. Depth below top of wall versus normalized lateral stress coefficient for the seven case studies used to calibrate the coherent gravity method and design distribution for $\phi = 45^\circ$ (Baquelin 1978).

The use of the earth pressures calculated from Baquelin (1978) and the failure surface proposed by Schlosser and Elias (1978), is generally referred to as the Coherent Gravity method. Though Juran and Schlosser (1978) used limit equilibrium analysis to argue its validity, the Coherent Gravity method was originally calibrated empirically with the use of seven wall case histories as shown above in Figure 0.13. These case histories included both smooth and ribbed steel strip reinforcements but did not include extensible reinforcements (e.g. geo-synthetic), or bar mat type in-extensible reinforcements (Schlosser 1978).

Schlosser and Segrestin (1979) summarized the Coherent Gravity method with assumed location of maximum tension (i.e., the bilinear failure surface) and the distribution of earth pressure coefficients. The maximum reinforcement load for a given reinforcement layer, T_{max} , was determined as:

$$T_{max} = K_r \sigma_v S_v R_c \quad (2.16)$$

where σ_v is the vertical effective stress at the level of the reinforcement, S_v is the vertical reinforcement spacing and R_c is the reinforcement coverage ratio defined as:

$$R_c = \frac{b}{S_h} \quad (2.17)$$

where b is the width of the strip, and S_h is the horizontal reinforcement spacing as shown in Figure 0.14.

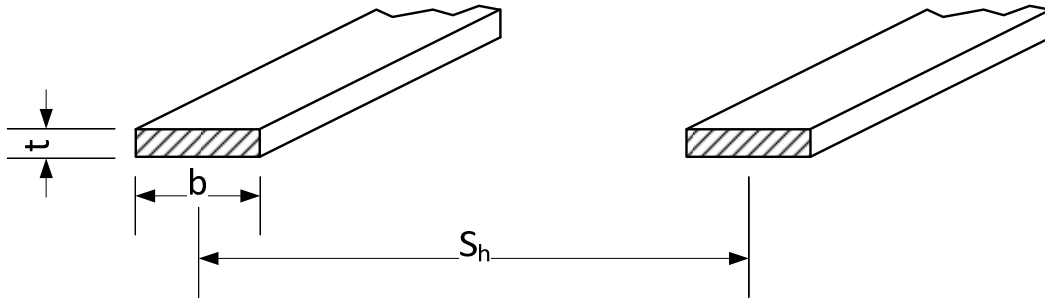


Figure 0.14. Parameters for coverage ratio, R_c , calculation with steel strip reinforcements.

The vertical effective stress at the level of the reinforcement of interest, σ_v , was determined by assuming the reinforced fill to be a rigid block (Schlosser and Segrestin 1979); then, the equations of static equilibrium were used to calculate vertical stresses. For the case of level backfill (i.e. a slope angle, β , equal to zero), the vertical effective stress is calculated as:

$$\sigma_v = \gamma_r Z \quad (2.18)$$

where γ_r is the unit weight of the reinforced fill, and Z is the vertical distance from the top of the wall to the reinforcement layer of interest. However, when the back slope angle is greater than zero, as shown in Figure 0.15, the resultant of the reaction stress distribution (i.e., R in Figure 0.15) is assumed to become eccentrically located a distance e from the center of the reinforcement towards the wall. This assumed eccentricity can be determined algebraically by setting the sum of the moments about the center of bottom of the “rigid block” (i.e. point c in Figure 0.15) equal to zero. Solving for the moment arm of the reaction force, e , produces the following expression:

$$e = \frac{F_T(\cos \beta)z / 3 - F_T(\sin \beta)L_r / 2 - V_2(L_r / 6)}{V_1 + V_2 + F_T \sin(\beta)} \quad (2.19)$$

where F_T is the resultant force of the lateral earth pressure acting on the back of the reinforced soil mass, z is the vertical distance from the reinforcement layer of interest to the intersection of the back slope and the plane defined by the back of the reinforced soil mass, L_r is the total length of the reinforcement, V_1 is the resultant of the pressure from the reinforced soil mass acting on the reinforcement layer of interest, V_2 is the resultant force produced by the weight of the slope mass acting vertically at the top of the reinforced soil mass. The vertical stress acting on the reinforcement is then calculated as the sum of the vertical forces acting over the reinforcement length less two times the eccentricity:

$$\sigma_v = \frac{V_1 + V_2 + F_T \sin(\beta)}{L_r - 2e} \quad (2.20)$$

where e is the eccentricity of the reaction force, and K_r is the lateral earth pressure coefficient of the reinforced soil at the depth of the reinforcement of interest as described in Figure 0.16.

The Coherent Gravity method assumes a bi-linear failure surface, as shown in Figure 0.16. The failure surface, and thus the active zone, was defined by the vertical distance from the toe of the wall to the point where the failure surface exists the back slope, H' , calculated as:

$$H' = H + \frac{0.3H \tan \beta}{1 - 0.3 \tan \beta} \quad (2.21)$$

where H is the vertical distance from the toe of the wall to the top of the wall face. The horizontal distance from the wall face to the point where the failure surface exists the back slope is assumed to be $0.3 H'$. For the top half of the length H' the failure surface is vertical, then, as shown in Figure 0.16, the failure surface exits the reinforced soil mass at the toe of the wall.

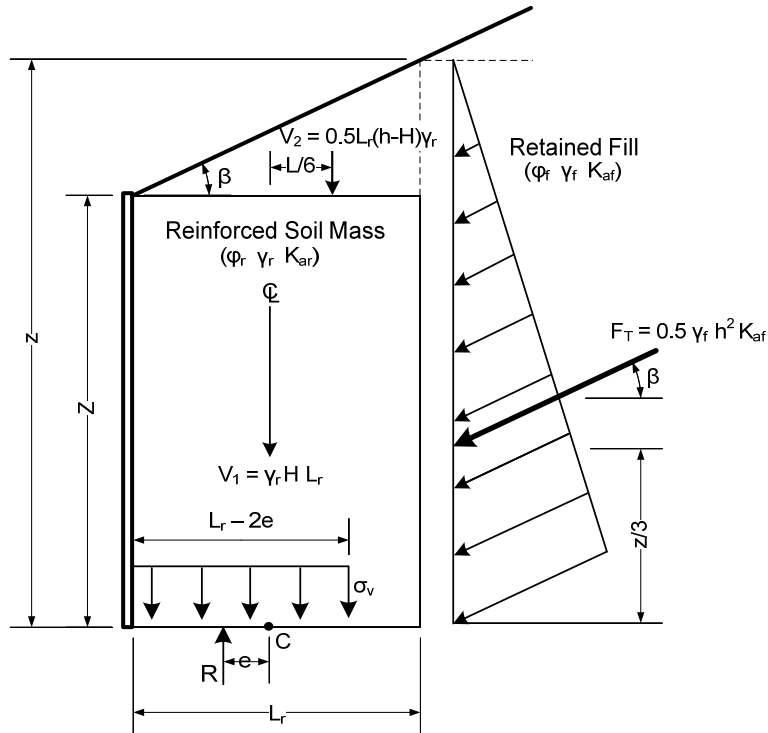


Figure 0.15. Meyerhof vertical stress distribution in MSE walls (adapted from Allen et al. 2001 and AASHTO 1999).

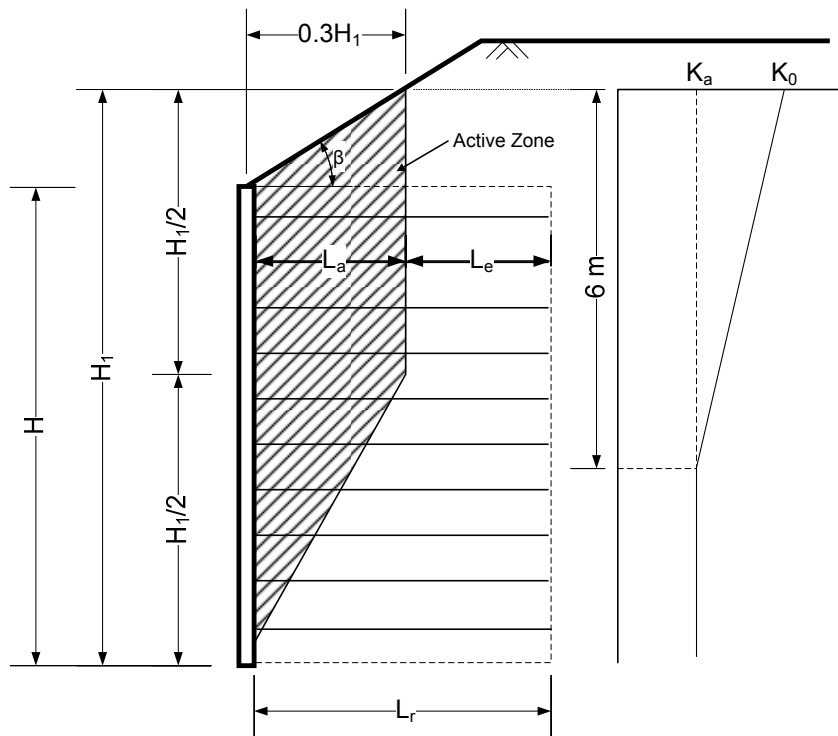


Figure 0.16. Graphic summary of the geometry assumed for use with the Coherent Gravity Method (adapted from Allen et al. 2001 and AASHTO 1996).

Bathurst et al. (2008) evaluated the accuracy of the Coherent Gravity method by comparing the ratio of measured to predicted maximum reinforcement load for walls with different backfill friction angles. For backfill soils with friction angles larger than 45 degrees, the Coherent Gravity method consistently under predicted the maximum reinforcement load as shown in Figure 0.17. This raises concern as it is common practice in design to reduce the friction angle in order to increase the reliability of the structure. However, reducing the friction angle can have non-conservative effect due to “locking-in” of compaction stresses near the top of the wall tending to be larger for soils with higher friction angles (Bathurst et al. 2008).

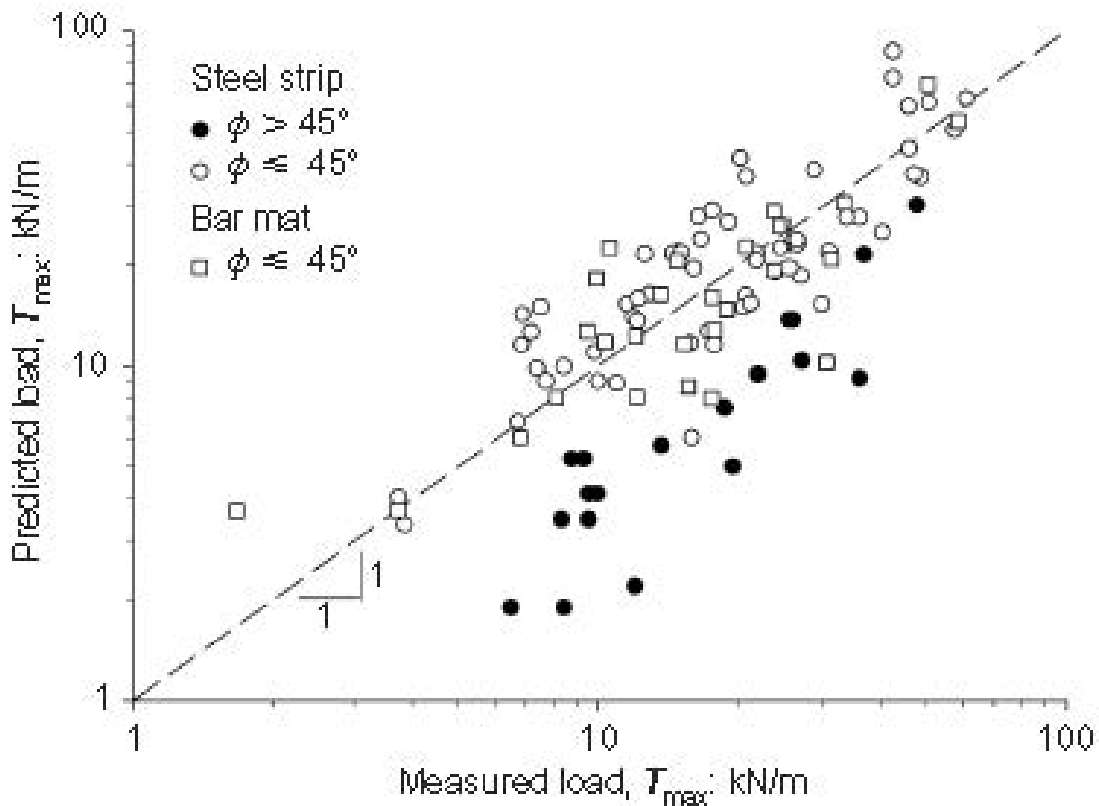


Figure 0.17. Predicted versus measured maximum reinforcement loads for the Coherent Gravity method and steel reinforced MSE walls showing the influence of friction angle (after Bathurst et al. 2008).

Since the Coherent Gravity method is empirically based, it is limited by the database of case histories to which it has been calibrated. The maximum wall height in the database used to calibrate the Coherent Gravity method was 13 m; therefore the use of the method should be limited to wall heights of 13 m or less (Stuedlein et al. 2012).

Tieback Wedge Method (Bell et al. 1983)

Originally developed by Bell et al. (1975) and Steward et al. (1977), the Tieback Wedge method was intended for use with geotextile and other extensible type reinforcement inclusions. The method was later adapted for use with welded wire reinforcements (Allen et al. 2001).

The primary difference between the Tieback and the Coherent Gravity methods is the assumed distribution of lateral earth pressure. For the Tieback Wedge method, the use of K_a was permitted along the entire back face of the wall due to the flexibility of the reinforced soil mass; this flexibility allowed minimum active pressure to be mobilized and a failure plane to develop. Walls reinforced with inextensible reinforcements are significantly stiffer, and it was proven that the displacements were minimized and the full development of an active wedge was not realized (Bell et al. 1983). The maximum reinforcement tensile load for the Tieback Wedge method is determined using Equation 2.16, where $K_r = K_a$ for all values of Z , and the vertical overburden stress calculated as:

$$\sigma_v = (\gamma_r Z + S + q) \quad (2.22)$$

where S is the average surcharge from the soil above the wall (if any), q is the vertical traffic surcharge stress, and all other variables have been defined (Allen et al. 2001).

FHWA Structure Stiffness Method (Christopher et al. 1990a; Christopher 1993)

The Federal Highway Administration (FHWA) Structure Stiffness method was developed from an extensive effort by the FHWA to research and standardize the design of reinforced earth. The Structure Stiffness method was the first method to directly consider the combined effect of reinforcement spacing (i.e., reinforcement density) and the reinforcement stiffness referred to as the global reinforcement stiffness, S_r , and its effect on reinforcement stresses:

$$S_r = \frac{E_r A_r}{(H / n_r)} \quad (2.23)$$

where E_r and A_r is the reinforcement modulus and the reinforcement area per unit width of wall, respectively, and n_r is the number of reinforcement layers. Christopher et al. (1990a) studied the behavior of eight 6 m tall instrumented walls with different facings, reinforcements, fill material, and geometries. The study of wall behavior lead to a design method that is applicable for both extensible and inextensible reinforcements.

The location of the failure plane is determined using Figure 0.16, similar to the Coherent Gravity method. The maximum reinforcement tension, T_{max} , is determined using Equation 2.16, with the vertical overburden stress, σ_v , calculated using Equation 2.22, and K_r calculated as:

$$K_r = \begin{cases} K_a \left(\Omega_1 \left(1 + 0.4 \frac{S_r}{47880} \right) \left(1 - \frac{Z}{6} \right) + \Omega_2 \frac{Z}{6} \right) & \text{if } Z \leq 6 \text{ m} \\ K_a \Omega_2 & \text{if } Z > 6 \text{ m} \end{cases} \quad (2.24)$$

where S_r is in units of kPa and Ω_1 is an empirical fitting coefficient related to the reinforcement type defined as:

$$\Omega_1 = \begin{cases} 1.0 & \text{for strip and sheet reinforcements} \\ 1.5 & \text{for grids and welded wire mats} \end{cases} \quad (2.25)$$

and Ω_2 is an empirical fitting coefficient related to the global reinforcement stiffness and defined as:

$$\Omega_2 = \begin{cases} 1.0 & \text{if } S_r \leq 47.88 \text{ MPa} \\ \Omega_1 & \text{if } S_r > 47.88 \text{ MPa} \end{cases} \quad (2.26)$$

Ehrlich and Mitchell (1994)

Compaction-induced stresses should always be considered in fill-type walls. Compaction-induced stresses are especially important for the design of MSE walls as facing panels and reinforcement connections are extremely stiff, and develop stress concentrations at the connections. The method developed by Ehrlich and Mitchell (1994) is the only method developed to-date that addressed both compaction-induced reinforcement stresses and global reinforcement stiffness directly. The maximum reinforcement load for the Ehrlich and Mitchell (1994) method was calculated using plane strain soil properties as:

$$T_{\max} = S_v S_h K_r \sigma'_z \quad (2.27)$$

where σ'_z is the overburden stress corrected for eccentric loading and for case of no surcharge loading is defined as:

$$\sigma'_z = \frac{\gamma'Z}{1 - (K_a/3)(Z/L_r)^2} \quad (2.28)$$

where L_r is the length of the reinforcement. When comparing the overburden stresses for level ground calculated using Equation 2.28 to those using the Coherent Gravity method, the overburden stresses calculated using the Coherent Gravity method are slightly larger (7 percent greater at $Z = 25$ m), but appears suitable for substitution due to the relatively small difference in stresses. Ehrlich and Mitchell (1994) do not provide guidance on overburden stress calculations for the case of a back slope or surcharge. However, Equation 2.20 from the Coherent Gravity method can be used in calculating the overburden stress for these cases as it also considers overturning moment.

The lateral earth pressure coefficient, K_r , is determined for each reinforcement layer iteratively using:

$$\frac{1}{S_i} \left(\frac{\sigma'_z}{P_{atm}} \right)^n = \frac{(1 - \nu_{un}^2) [(K_r - K_{\Delta 2}) - (K_c - K_{\Delta 2}) OCR]}{(k_{ur}/K)(K_c OCR - K_r) K_r^n} \quad (2.29)$$

where P_{atm} is atmospheric pressure, OCR = the overconsolidation ratio, k_{ur} is the modulus number for unloading, K is the modulus number for loading, and n is the modulus exponent, described previously in regard to the Duncan Chang (1970) hyperbolic model. The variables k_{ur} , K , and n can be found in Duncan et al. (1980) for various soils if soil-specific test information is unavailable. The relative reinforcement-soil stiffness index, S_i , was defined as:

$$S_i = \frac{(E_r A_r)}{k P_{atm} S_v S_h} \quad (2.30)$$

where the peak vertical stress during compaction, σ'_{zC} , was defined as:

$$\sigma'_{zC} = \frac{\nu_o (1 - K_a) \sqrt{\frac{\gamma' Q N_\gamma}{2L}}}{K_o} \quad (2.31)$$

where Q is the maximum compaction force produced during compaction, L is the length of the roller used to compact the back fill, and N_γ is the bearing capacity factor for Rankine wedge theory. The Poisson's ratio corresponding to K_o loading, ν_o , was defined as:

$$\nu_o = \frac{K_o}{1 + K_o} \quad (2.32)$$

Poisson's ratio for unloading, ν_{un} , was given as:

$$\nu_{un} = \frac{K_{\Delta 2}}{1 + K_{\Delta 2}} \quad (2.33)$$

where $K_{\Delta 2}$ is the "decremental lateral earth pressure coefficient" and defined as:

$$K_{\Delta 2} = \frac{K_o (OCR - OCR^\alpha)}{OCR - 1} \quad (2.34)$$

and the unloading coefficient, $\alpha = 0.7 \sin(\phi'_{PS})$.

The lateral earth pressure coefficient considering compaction, K_c , must be determined iteratively using:

$$\frac{1}{S_i} \left(\frac{\sigma'_{zc}}{P_{atm}} \right)^n = \frac{(1 - \nu_o^2)(1 - K_{aa})^2 (K_o - K_c) K_o}{(K_c - K_{aa})(K_o - K_{aa}) K_c^n} \quad (2.35)$$

where the "equivalent active earth pressure coefficient", K_{aa} was defined as:

$$K_{aa} = \frac{K_a}{(1-K_a) \left(\frac{c'}{(\sigma'_{zc} K_c \tan \phi') + 1} \right) + K_a} R_f \quad (2.36)$$

where c' is the effective cohesion intercept, and R_f is the failure ratio as defined by Duncan et al. (1980).

The main advantage of the Ehrlich and Mitchell (1994) method is its ability to predict maximum reinforcement tension during construction. The Coherent Gravity method typical does fairly well estimating the reinforcement tension for the case of end of construction; however, for intermediate construction stages the measured tension loads are typically larger than those at the end of construction (Stuedlein et al. 2012). Figure 0.18 compares measured maximum tension to those predicted using the Coherent Gravity, and Ehrlich and Mitchell (1994) methods.

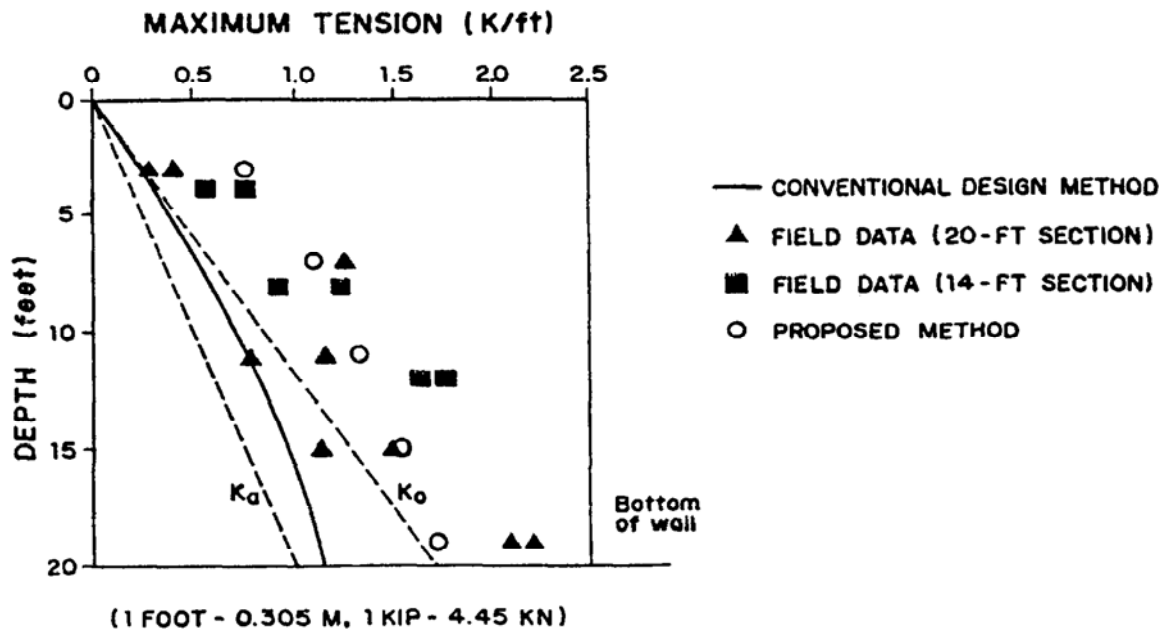


Figure 0.18. Comparison of the Ehrlich and Mitchell (1994) method (proposed method) to the Coherent Gravity method (conventional design method) and field measurements at the VSL wall at Hayward (after Neely 1995).

Because the Ehrlich and Mitchell (1994) method requires that the compaction equipment to be used during construction be known or estimated with some degree of certitude, it may be impractical to use this method in typical design situations. However, the Ehrlich and Mitchell

(1994) method it is an extremely valuable tool for the analysis of wall behavior due its theoretical robustness and accuracy (Stuedlein et al. 2012).

AASHTO Simplified method(Allen et al. 2001)

The most common method for estimating reinforcement loads in the U.S. is the AASHTO Simplified method (Bathurst et al. 2009). The Simplified method was developed with the goal of replacing the Coherent Gravity method, the Tieback Wedge method, and the FHWA Structure Stiffness method; however, the Coherent Gravity method is still widely used for design of walls with steel strip reinforcements and has recently been admitted back into AASHTO specifications. The three main goals for developing the Simplified method were (Allen et al. 2001):

- to develop a K_r/K_a curve for each general type of reinforcement as shown in Figure 0.19,
- to simplify calculations by excluding the rigid body assumption and associated overturning-induced stresses for internal stability calculations, and
- To calibrate the method using a large dataset of full scale wall case histories.

The use of K_r/K_a curves unique to the reinforcement type indirectly allows for the consideration of the stiffness of the reinforced soil mass and some compaction stress history effects. However, the density of reinforcements is not taken into consideration, that is, as S_v and S_h change, the K_r/K_a curves remain constant, despite the potential for an increase or decrease in the “locking-in” of compaction stresses due to the increase or decrease in spacing, respectively. Research performed by Stuedlein et al. (2010) showed that the local reinforcement stiffness, S_{local} , accounted for up to 81 percent of the measured reinforcement strains in two tall MSE walls. The rigid body assumption results in the application of an overturning moment to be assumed to be transmitted through the reinforced soil mass thus increasing vertical stresses. Removing this assumption greatly reduces computational effort and is arguably more accurate (Allen et al. 2001). Neither the Tieback Wedge method, FHWA Structure Stiffness method, nor the K-Stiffness method (discussed subsequently) assumes a rigid body. On the other hand, the Coherent Gravity and the Ehrlich and Mitchell (1994) methods do assume a rigid reinforced mass (Stuedlein et al. 2012).

Prior to the development of the Simplified method there was confusion in industry regarding which design method (mainly Coherent Gravity or Tieback method) was appropriate for different types of reinforcement material (e.g. inextensible or extensible). Typically, overly conservative designs resulted from the use of the Coherent Gravity method for extensible reinforcements.

Similar to the Coherent Gravity method the Simplified method is empirically based and is therefore limited by the database to which it has been calibrated. The data base used to calibrate the Simplified method is limited to wall heights less than 18 m (Allen et al. 2001).

The maximum reinforcement tension, T_{max} , for the AASHTO Simplified Method is determined using Equation 2.16, with σ_v determined using Equation 2.22, K_r is determined

depending on reinforcement type using Figure 0.19. The active lateral earth pressure coefficient for use with Figure 0.19 is determined using a friction angle as measured in triaxial compression or direct shear test, but limited to 40 degrees.

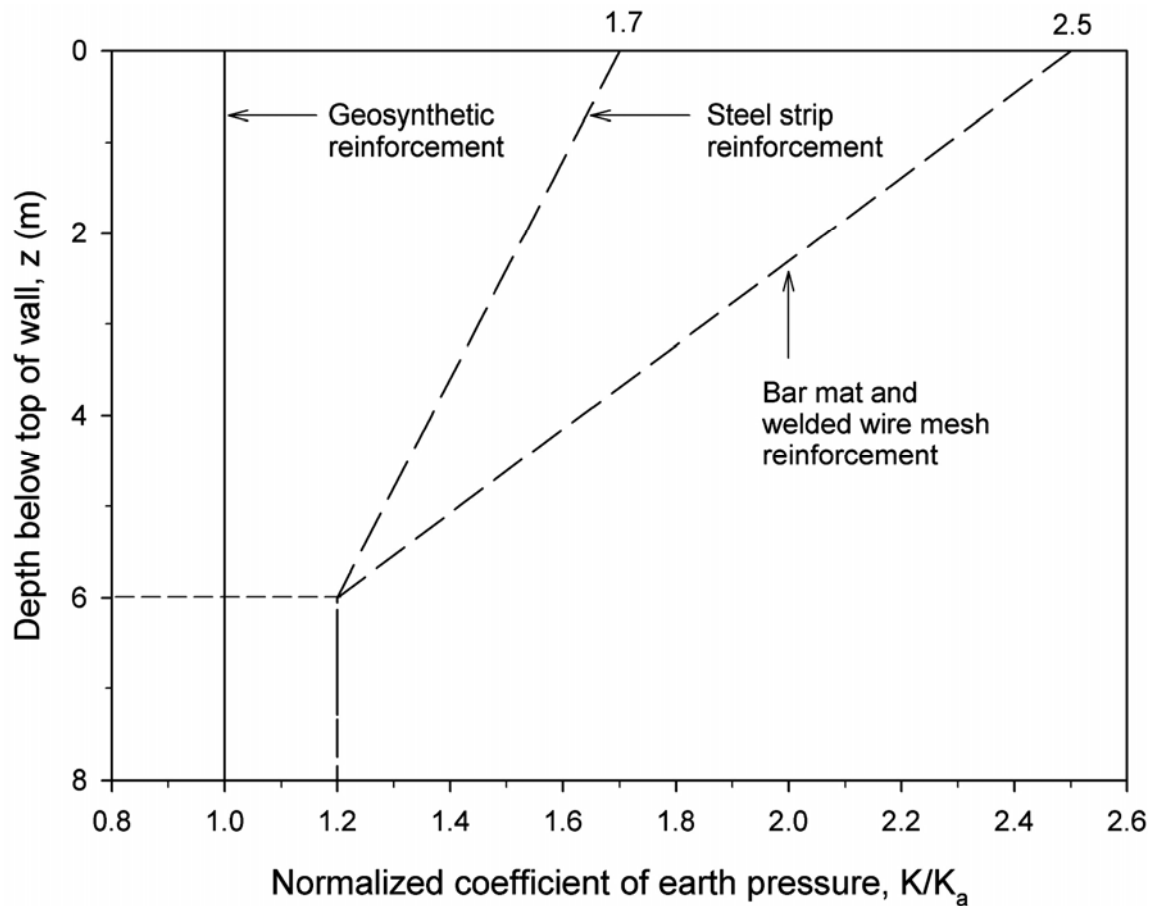


Figure 0.19. Normalized lateral earth-pressure distributions for use with the AASHTO simplified method (after Bathurst et al. 2009).

Bathurst et al. (2009) evaluated the accuracy of the Simplified method by comparing the ratio of measured to predicted maximum reinforcement load for walls with different backfill friction angles. For backfill soils with friction angles larger than 45 degrees, the Simplified method consistently under predicts the maximum reinforcement load as shown in Figure 0.20 and is similar to the Coherent Gravity method.

K-Stiffness Method (Allen et al. 2003, 2004)

The K-Stiffness method for steel reinforced walls developed by Allen et al. (2004) was intended to update the FHWA Structure Stiffness method by considering facing stiffness, wall batter, and local reinforcement stiffness. The method was also calibrated using an extensive dataset which included 19 full scale instrumented steel reinforced walls. Similar to the Coherent Gravity and Simplified methods, the K-Stiffness method is empirically based and is therefore

limited by the database to which it has been calibrated. The same data base used to calibrate the Simplified method was used to calibrate the K-Stiffness method; therefore the use of the method is also limited to wall heights of 18 m or less. Additionally, the K-Stiffness method utilizes plane strain soil properties; this is in contrast to the Coherent Gravity and Simplified methods which use shear strength parameters determined by triaxial or direct shear tests.

The maximum reinforcement tension for the K-Stiffness Method is calculated as:

$$T_{\max} = \frac{1}{2} K_r \gamma_r (H + S) S_v D_{t\max} \Phi \quad (2.37)$$

where $D_{t\max}$ is a factor that adjusts the load depending on the location of the reinforcement in relation to the top of the wall facing which varies from 0.1 at the top of the wall to one at 60 percent of the wall depth, as shown in Figure 0.21.

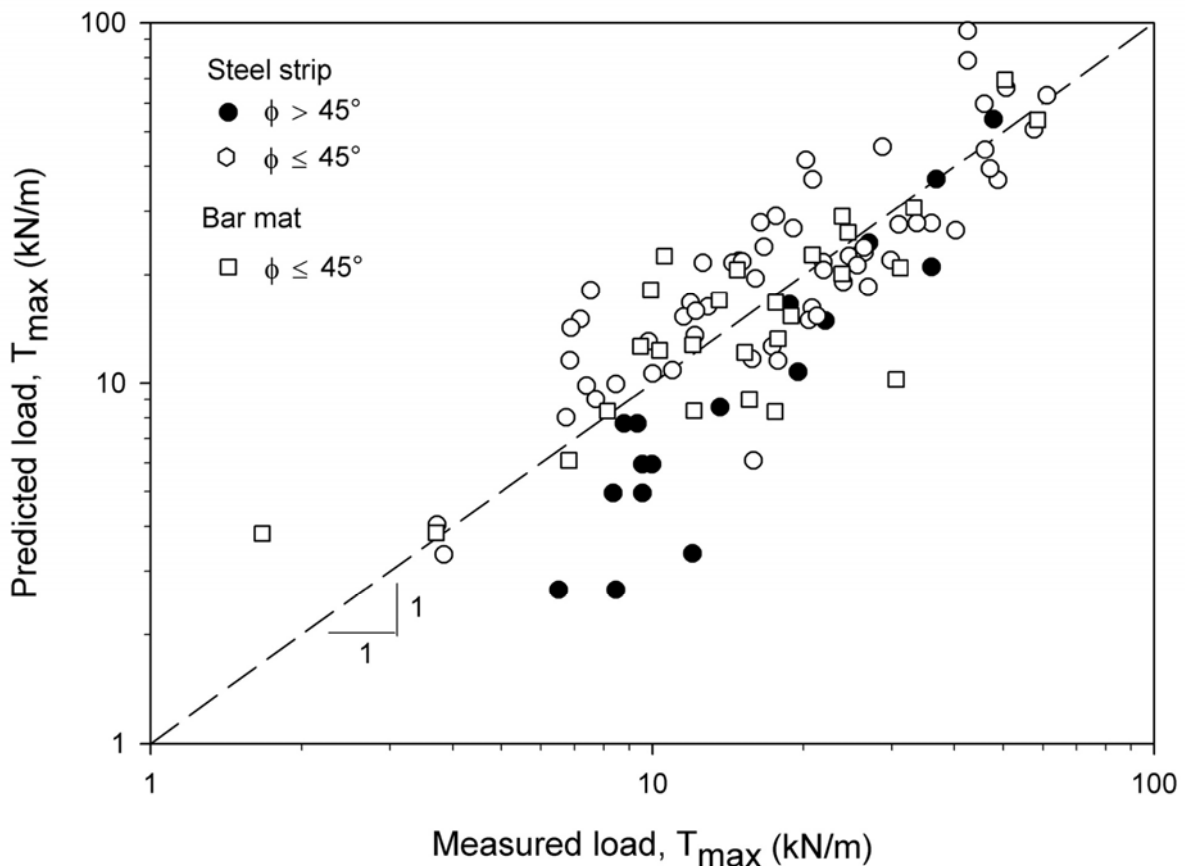


Figure 0.20. Predicted versus measured maximum reinforcement loads for the AASHTO Simplified method and steel reinforced MSE walls showing the influence of friction angle (after Bathurst et al. 2009).

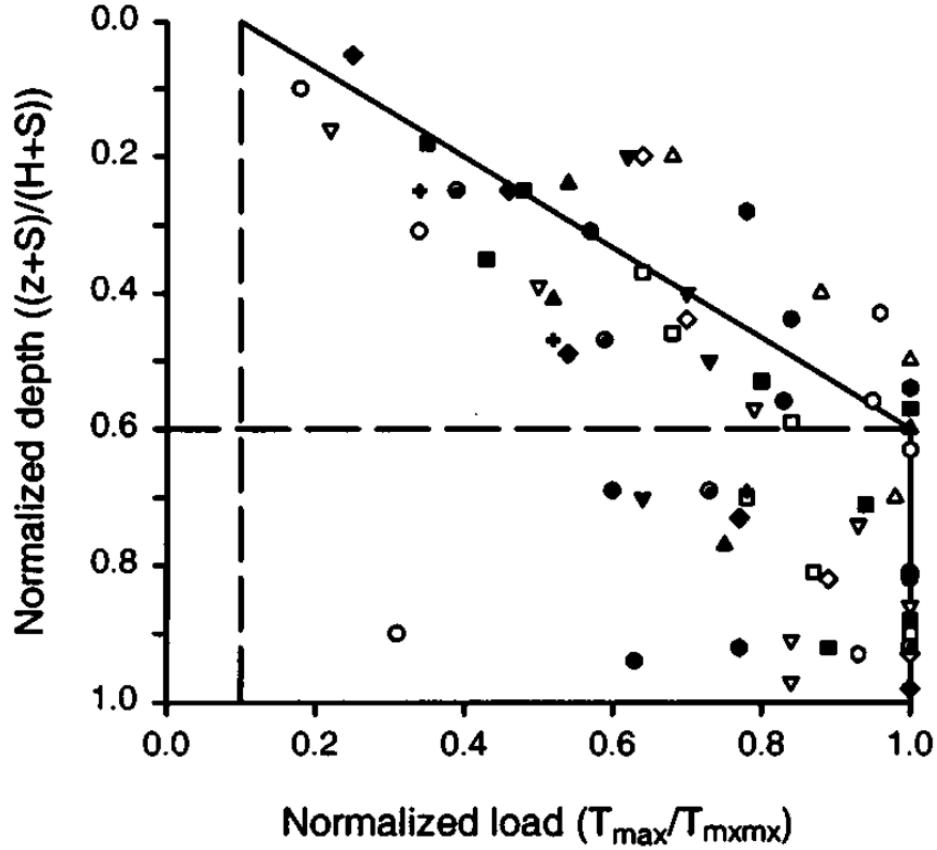


Figure 0.21. Distribution of $D_{l,max}$ recommended by Allen et al. (2004) for steel strip reinforced walls (after Alen et al. 2004).

The factor Φ attempts to capture the influence of different wall components on the magnitude of reinforcement load and is defined as follows:

$$\Phi = \Phi_{local} \times \Phi_{fs} \times \Phi_g \times \Phi_{fb} \quad (2.38)$$

where Φ_{local} and Φ_{fs} are the local reinforcement stiffness and facing stiffness factors, respectively, both taken as 1 for steel reinforcements ($\Phi_{local} = \Phi_{fs} = 1$). The global stiffness factor, Φ_g , is calculated as:

$$\Phi_g = \alpha_{ks} \left(\frac{S_r}{P_a} \right)^{\beta_{ks}} \quad (2.39)$$

where α_{ks} and β_{ks} are dimensionless fitting parameters. The value of $\alpha_{ks} = \beta_{ks} = 0.25$ was used by Allen et al. (2004), though no further guidance was provided. The facing batter factor, Φ_{fb} , is calculated as:

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}} \right)^d \quad (2.40)$$

where K_{abh} is the active lateral earth pressure coefficient considering the facing batter, K_{avh} is the active lateral earth pressure coefficient assuming the face batter is vertical, and d is a dimensionless fitting parameter. A value of $d = 0.25$ is suggested by Allen et al. (2004).

The improved accuracy of the K-Stiffness method, as shown in Figure 0.22, is not surprising since the method was developed to incorporate both primary and secondary factors (e.g., batter and facing stiffness) that influence the development of reinforcement load to improve its predictive ability. Because the K-Stiffness method is highly empirical in nature it should only be applied to those types of walls for which it was calibrated.

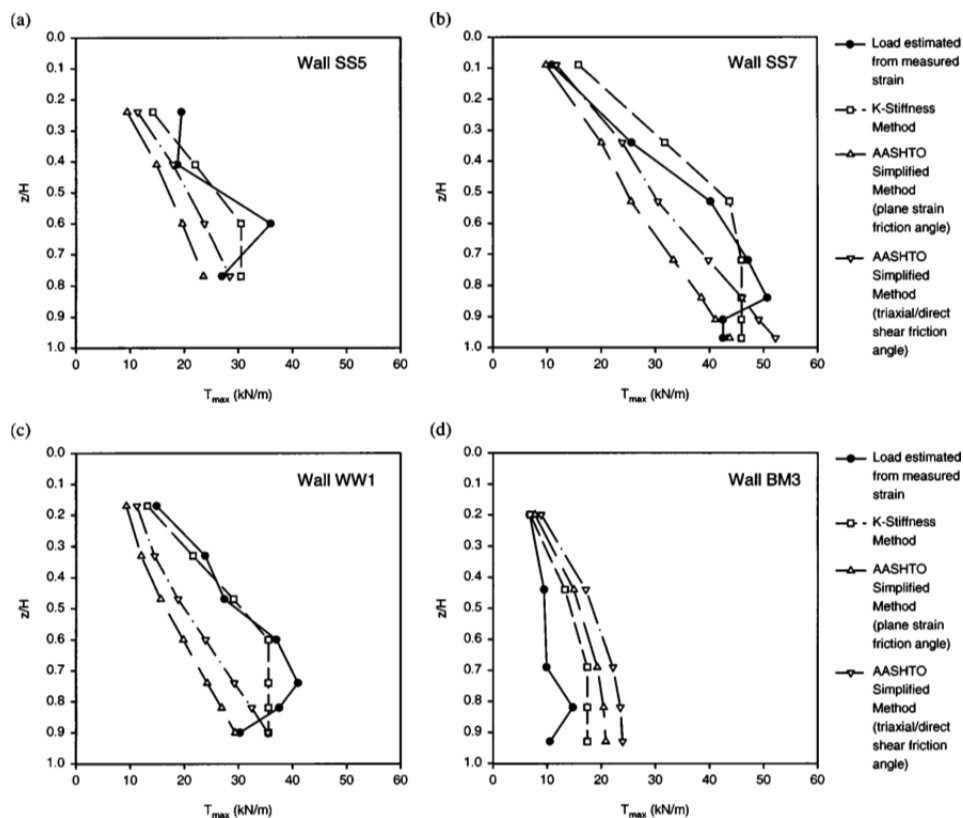


Figure 0.22. Comparison of maximum reinforcement tension as estimated by the AASHTO Simplified and K-Stiffness methods to those measured in the field: (a) steel strip reinforced wall, (b) steel strip reinforced wall, (c) welded wire reinforced wall, and (d) bar mat reinforced wall (after Allen et al. 2004).

Comparison of Design Methods for Use with Tall MSE Walls

Stuedlein et al. (2012) compared the reinforcement loads as predicted by the Coherent Gravity, Simplified, K-Stiffness, and the Ehrlich and Mitchell (1994) methods to those measured during and after the construction of two very tall MSE walls at Seattle-Tacoma International Airport (SeaTac). The soil properties used in the study were developed from direct shear test measurements required for contractor submittals. Since the K-Stiffness and Ehrlich and Mitchell (1994) methods require plane strain friction angles, the friction angle measured in direct shear of 43 degrees was converted to plane strain friction angles, ϕ'_{PS} , of 48 degrees considering the empirical correlations presented by Bolton (1986) and Jewell and Wroth (1987):

$$\phi'_{PS} = \tan^{-1}(1.2 \tan \phi_{DS}) \quad (2.41)$$

However, a triaxial and direct shear friction angle, $\phi'_{DS, TX} = 40$ degrees and a plane strain friction angle $\phi'_{PS} = 44$ degrees were used in load calculations to conform to AASHTO specifications.

Bias values, defined as the ratio of measured to predicted reinforcement loads, were determined for each of the reinforcement load prediction methods. It was observed that both the Simplified and Coherent Gravity methods tended to under-predict reinforcement loads during construction. The K-Stiffness method conservatively provided the best prediction of peak reinforcement stresses at intermediate construction stages where the backfill height was less than 20 m. The Ehrlich and Mitchell (1994) method provided the best overall prediction of reinforcement loads for both the North and West MSE wall as depicted in Table 0.1 by the bias values closest to one and low coefficients of variation (COV). Figure 0.23 compares the predicted distribution of reinforcement loads to those measured approximately 4 years after construction of the west wall was complete.

Pullout Resistance and Apparent Friction Coefficient

The earliest wall design methods involved smooth steel strips. When smooth strips are used, resistance is developed strictly through interface friction between the soil and reinforcement surface (Lee 1978). Examples of the application of this method can be found in Chang and Forsyth (1977), Lee et al. (1973), and Vidal (1969). The interface friction angle, ρ , used in early applications were those largely measured by Potyondy (1961) with the direct shear test. Potyondy (1961) conducted direct shear interface tests by replacing the bottom half of a standard direct shear box with different construction materials (e.g. wood, concrete, and steel). Direct shear tests were popular due to the availability of testing equipment, and produced fairly reasonable yet conservative results for smooth planar reinforcements where interface friction was responsible for a majority of the resistance (Schlosser 1990). Figure 0.24(a) describes the stress transfer mechanism for smooth planer reinforcements.

Table 0.1. Summary statistics of the bias in reinforcement loads for the SeaTac North and West MSE walls taken in August 2009 with capped constant friction angles ($\phi'_{DS, TX} = 40$ deg., $\phi'_{PS} = 44$ deg.) (Stuedlein et al. 2012).

	Mean bias	Range in bias		COV (%)
North MSE wall (n = 6)				
Coherent Gravity method	1.47	1.19	2.04	23
AASHTO Simplified method	1.27	1.03	1.72	21
Ehrlich & Mitchell (1994)	0.93	0.80	1.05	11
K-Stiffness method	0.73	0.58	0.85	15
West MSE wall (n = 14)				
Coherent Gravity method	1.40	1.04	2.06	21
AASHTO Simplified method	1.30	0.95	1.96	23
Ehrlich & Mitchell (1994)	0.87	0.69	1.19	17
K-Stiffness method	0.65	0.42	1.27	39

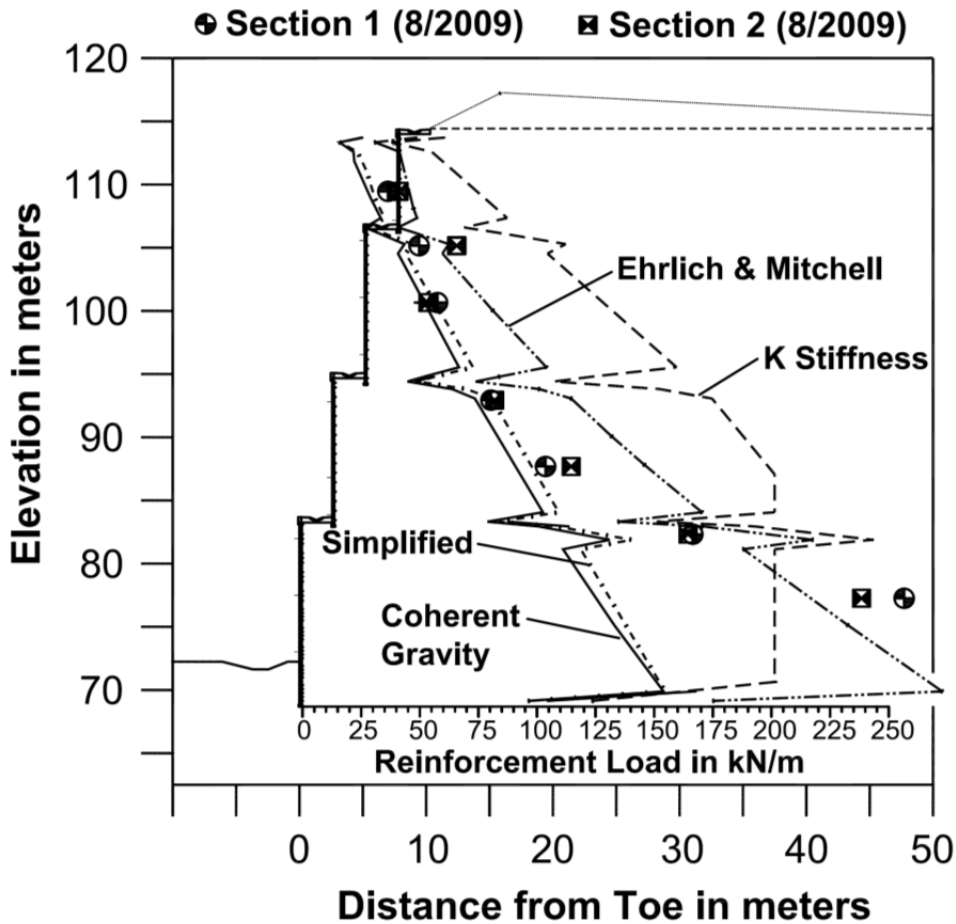


Figure 0.23. Comparison between measured and predicted peak reinforcement loads for the SeaTac West MSE wall after construction (after Stuedlein et al. 2012).

Ribbed reinforcements were developed by the Reinforced Earth Company to increase the amount of pullout resistance of steel reinforcements. In addition to generating friction along the reinforcement, ribbed reinforcements generate passive failure zones due to bearing in front of the ribs (Christopher et al. 1990b) as shown in Figure 0.24(b). Since performing direct shear interface friction tests on ribbed strips could not capture the complex soil reinforcement interaction, measuring the pullout resistance directly with pullout tests became the standard of practice (Schlosser 1990).

Schlosser and Elias (1978) conducted a parametric study on soil-reinforcement interaction and load transfer. Figure 0.25 presents the load- displacement data for pullout tests on a smooth and a ribbed strip at the same overburden pressure, each with the same plan dimensions. The following was concluded in light of the experimental data:

- At most working stress levels, displacement sufficient to mobilize passive resistance was not developed and resulted in similar behavior between smooth and ribbed strips.
- Once the ribbed strip displaced relative to the soil, passive resistance began to dominate the load displacement response, with significant gain in resistance as compared to the force-displacement curves for smooth strips.

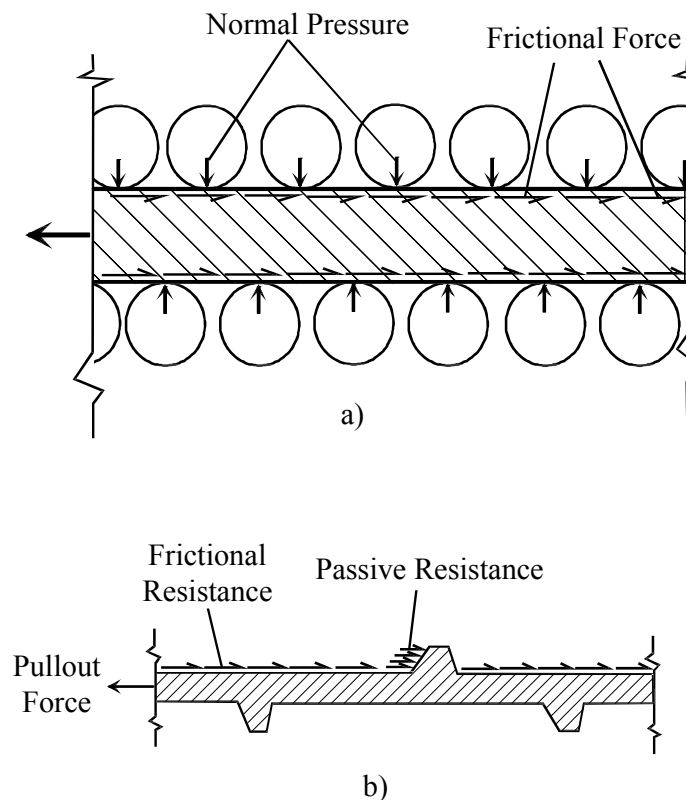


Figure 0.24. Stress transfer mechanisms for soil reinforcement: (a) frictional stress transfer between soil and reinforcement; and (b) soil passive resistance on reinforcement surface, reproduced from (Mitchell and Christopher 1990).

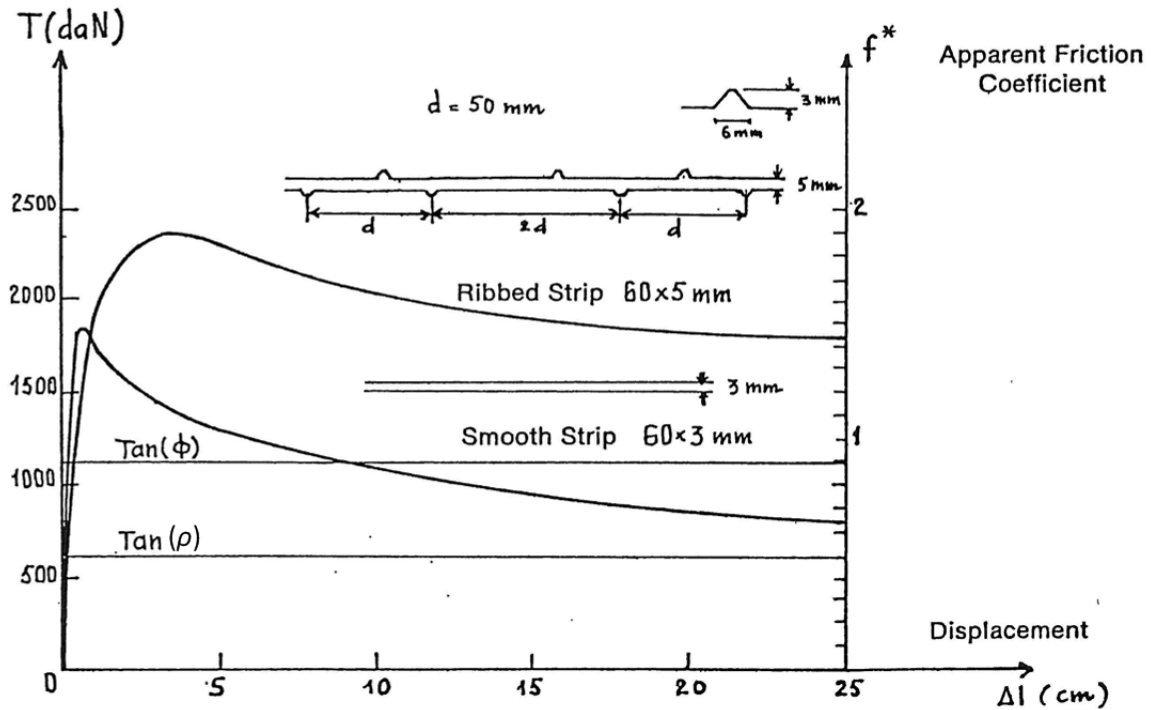


Figure 0.25. Smooth and ribbed steel strip reinforcement pullout load versus displacement curves (adapted from Schlosser and Elias 1978).

The soil-reinforcement interaction shown by the ribbed strips prompted Alimi et al. (1978) and Schlosser and Elias (1978) to develop an empirical design parameter referred to as the apparent friction coefficient (or friction bearing interaction factor), f^* . The apparent friction coefficient is determined using load-displacement pullout test data by:

$$f^* = \frac{P_{\max}}{\sigma_v A_s} \quad (2.42)$$

where P_{\max} is the maximum load observed during the pullout test.

Schlosser and Elias (1978) tested several factors hypothesized to affect pullout resistance, which included:

- strip width and height,
- surface characteristics (ribbed or smooth),
- relative density,
- embedment length, and
- effective overburden pressure.

Of these factors, overburden stress, σ'_v , relative density, and reinforcement surface characteristics had the largest effects on the apparent friction coefficient. It was observed that f^* decreases as σ'_v increases, this was mainly attributed to dilation effects; that is, as the overburden pressure increases, dilation is suppressed and thus the excess normal pressures provided by the dilation decreases. This accounts for the increased apparent friction coefficients typically observed at lower confining pressures as shown in Figure 0.26. Unfortunately, researchers rarely provide basic classification information (e.g., gradation and angularity) with f^* data and therefore the effects of these variables usually cannot be aggregated and quantified. Additionally, the development and peak magnitude of f^* can also be influenced by the testing conditions, as described subsequently.

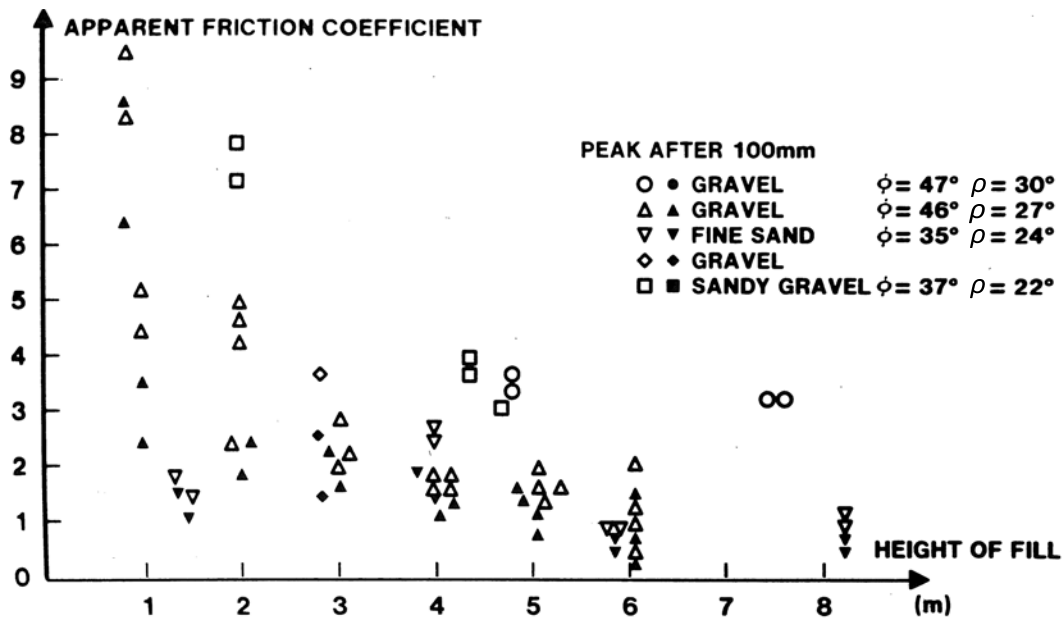


Figure 0.26. Apparent friction coefficient versus height of fill data used to originally calibrate the pullout design procedure (after McKittrick 1978).

Pullout Test Design Considerations

Pullout tests are typically performed in the laboratory using a large box with the overburden stress applied through an air or water bladder or full-scale tests where a sacrificial strip is placed during construction and pulled out using a hydraulic jack. The overburden pressure is not typically measured; rather, it is assumed that the full vertical overburden stress (e.g. the bladder pressure for the laboratory or γz for the full-scale method) acts on the surface of the reinforcement. For these tests, it is required that the reinforcement length or the overburden pressure be limited to prevent yielding at the connection to the reinforcement.

Palmeira and Milligan (1989) describe the design of different pullout tests and the effects of boundary conditions on the resulting behavior. The three aspects considered by Palmeira and Milligan (1989) were:

- the uniformity of load application or top boundary condition;
- the manner in which the strip exits the soil mass during the test; and,
- the reinforcement embedment length.

The top boundary condition was found to have some effect on the load-displacement response for steel grid reinforcements as shown in Figure 0.27. For the rigid boundary condition, a rough rigid plate attached to a hydraulic actuator was used to apply the overburden pressure. For the flexible condition, a water filled bladder was used to apply the overburden pressure. As shown in Figure 0.27, the top loading boundary condition didn't affect the initial response, but a rigid boundary condition, which deviates significantly from the field condition, resulted in approximately 10 percent larger peak pullout resistance.

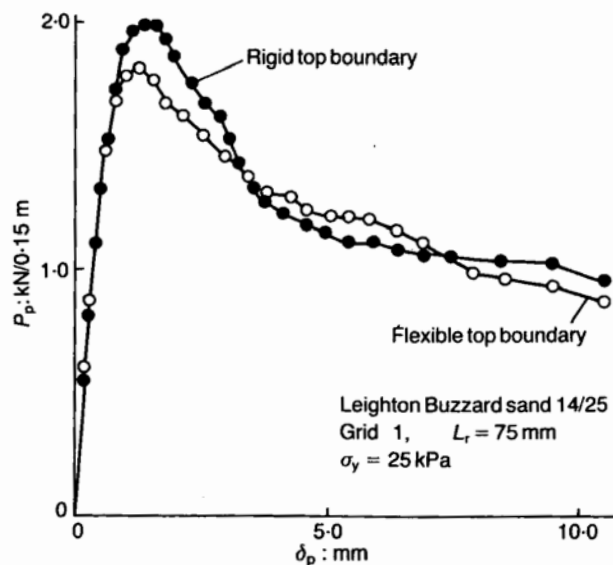


Figure 0.27. Effect of top boundary condition on pullout test results with steel grid reinforcements (after Palmeira and Milligan 1989).

Palmeira and Milligan (1989) concluded that neither the strip length nor the load application (rough rigid plate or pressure bladder) had a significant effect on the measured pullout force, but the boundary condition where the strip exits the soil mass did have a large effect on measured pullout resistance. It was found that large passive-type resisting stresses built up at the front of steel grid reinforcements, and locally increased the normal stress on the reinforcements, resulting in artificially high pullout resistance. These stresses were reduced when the front wall was lubricated. Figure 0.28 shows a large increase in peak bond strength for grid reinforcements when the front wall was not lubricated. However, the use of longer strips causes the front wall friction effects to be reduced since the ratio of the front wall friction load contribution to the load contribution of the soil-reinforcement interaction is smaller. No comparative study of the effect of front wall friction on the pullout load measured using ribbed steel strip reinforcements has been conducted. Some studies have chosen to have the strip exit the soil away from the cell wall using an intruded slot or pipe (e.g. Schlosser and Elias, 1978) as

shown in Figure 0.29, however, it is unclear as to what the effect of the pipe interacting with the strip and soil would have on the magnitude of measured pullout resistance.

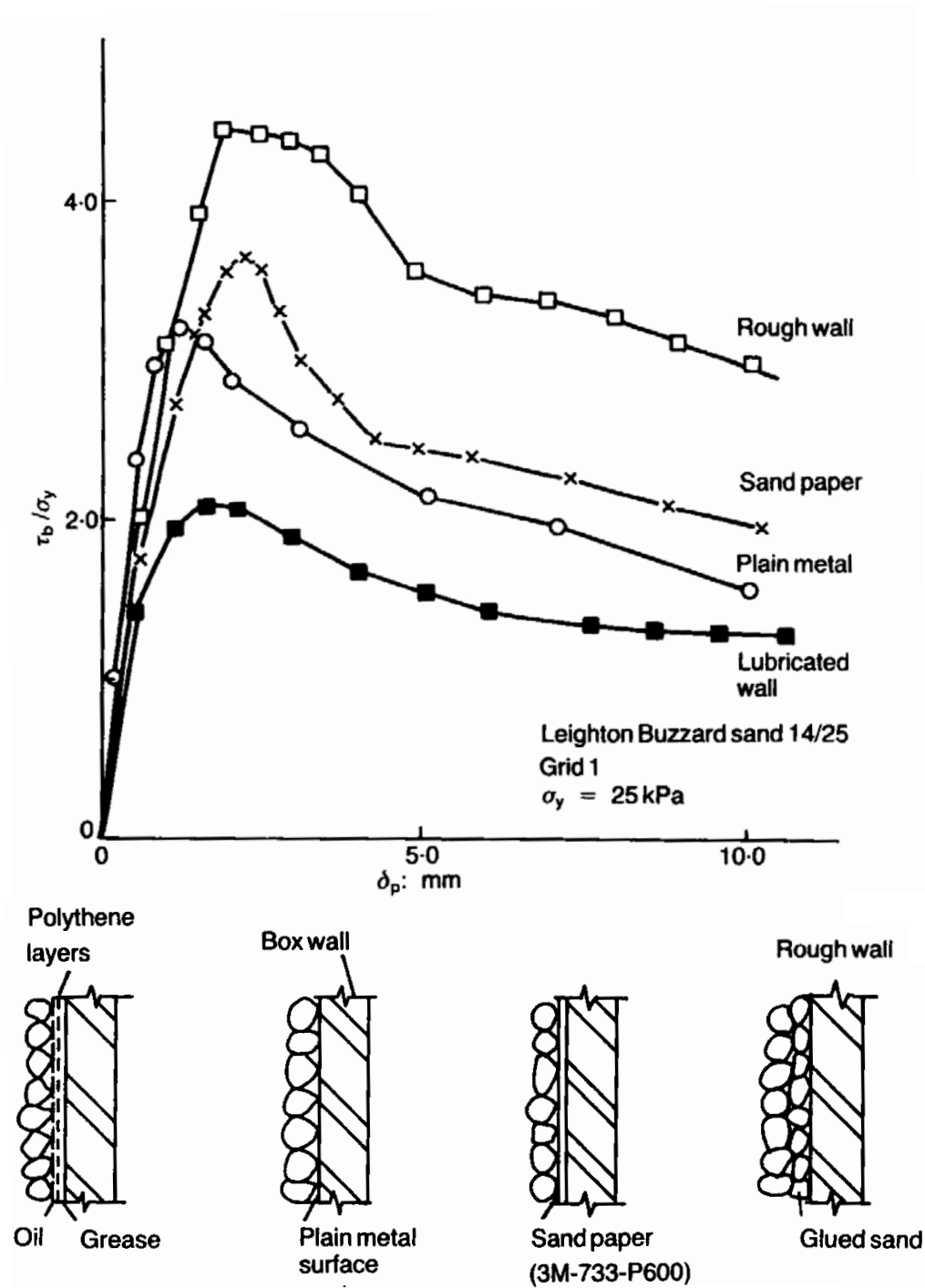


Figure 0.28. Bond strength versus displacement of a steel grid for different front wall friction conditions (after Palmeira and Milligan 1989).

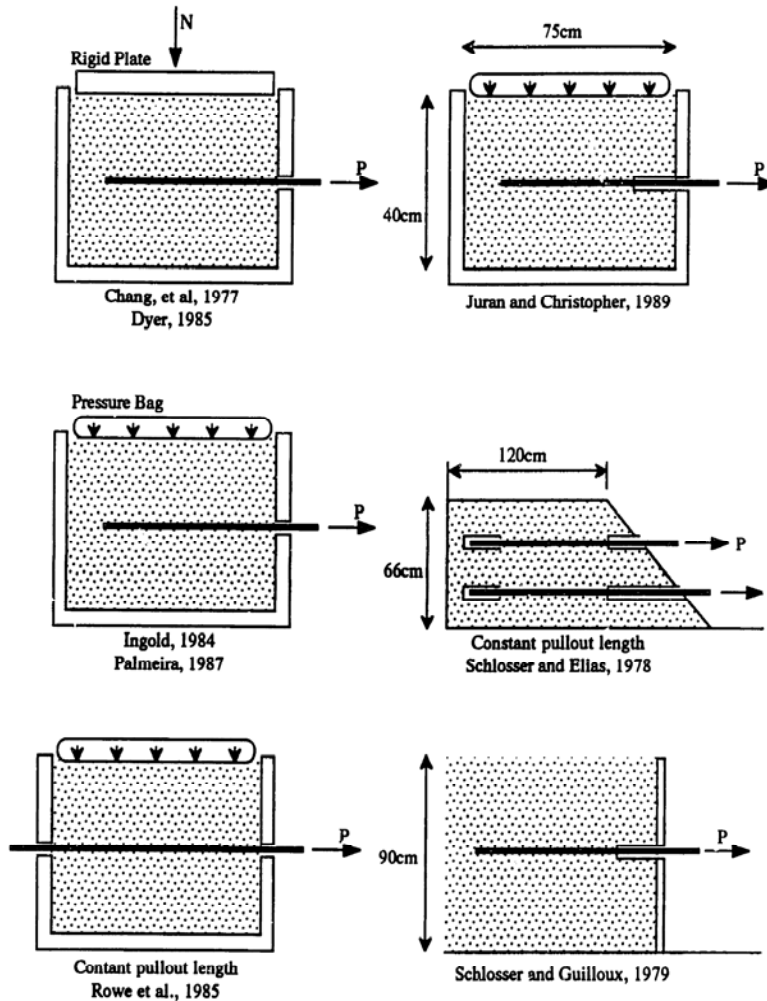


Figure 0.29. Boundary conditions for different pullout test presented in literature (after Larson 1992).

Development of Design Procedures for Pullout Resistance

For a given soil-reinforcement combination, most pullout studies observed that a constant value of f^* is obtained approximately 6 m below the top of the wall. This is the basis of the Allowable Stress Design (ASD) method outlined in Christopher et al. (1990) and which remains the standard of practice for steel ribbed strips (Berg et al. 2009). The pullout data used in calibrating this design procedure is presented in Figure 0.26.

As stated in Christopher et al. (1990a), the distribution of f^* can be obtained in two ways: either by performing pullout tests with the backfill and reinforcement to be used in construction, or by calculating f^* at the top of the wall using the coefficient of uniformity, C_u , of the retained fill as:

$$f^* = 1.2 + \log C_u \quad (2.43)$$

and reducing linearly to:

$$f^* = \tan \phi \quad (2.44)$$

at a depth of 6 m with the qualifier that $f^* \leq 2$ for all depths. This procedure results in a bilinear or tri-linear design curve, depending on the backfill characteristics, intended to follow a 95% confidence limit. However, this design model was calibrated using data from a variety of soil types and thus the model tends to underestimate f^* when dilative backfill soils such as dense gravels are used.

Other design procedures that have been proposed include those by Huang et al. (2012) and Miyata and Bathurst (2012). Using the same database as used to calibrate the current design model Huang et al. (2012) reported a Load and Resistance Factors Design (LRFD)-calibrated method for the internal stability of steel strip reinforced MSE walls. The intent of LRFD is to separate the uncertainty in the load from that of the resistance and use partial reduction or increasing factors to accommodate differences in uncertainty. LRFD-based procedures require that a model that accurately represents the statistical distribution of a sample population, and thus the design model presented in Figure 0.30 is not appropriate for use with current ASD design procedures.

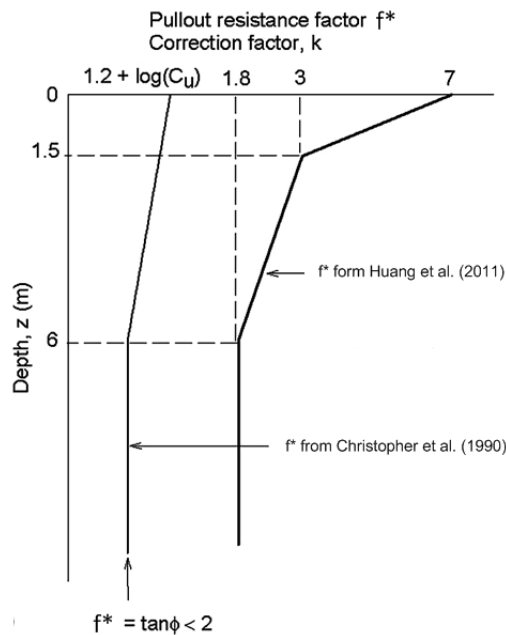


Figure 0.30. LRFD calibration model for apparent friction coefficient and example design distribution (adapted from Huang et al. 2012).

Miyata and Bathurst (2012) developed a new database of Japanese pullout test results and developed a three parameter exponential model. The previous Japanese design model was overly conservative at low confining pressures and exhibited a large bias when used with LRFD procedures (Miyata and Bathurst 2012). As shown in Figure 0.31, two models are proposed for use with steel ribbed reinforcements: the “fit” model which is intended for use with LRFD, and the “lower bound” model intended to replace the Japanese bilinear model currently for use with allowable stress design. Note that because LRFD procedures directly accounts for the uncertainty in an empirical model, a best-fit line (i.e., an unbiased model) is typically preferred.

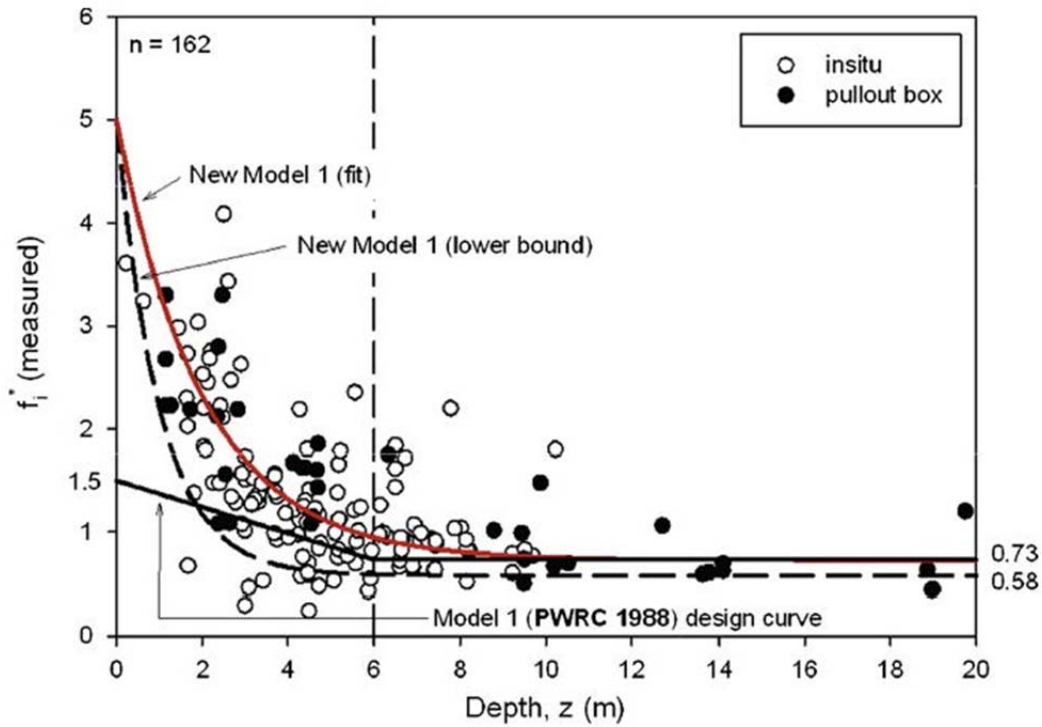


Figure 0.31. Apparent friction coefficient versus depth for Japanese pullout test database for ribbed steel strips with current Japanese design model and proposed model by Miyata and Bathurst (2012).

Factor of Safety and Resistance Factors with Respect to Pullout

For linear ribbed reinforcements, the Factor of Safety with respect to reinforcement pullout, FS_{PO} , at a given reinforcement layer, can be calculated as:

$$FS_{PO} = \frac{P_r R_c}{T_{\max}} \quad (2.45)$$

where P_r , the pullout resistance per reinforcement, is determined by:

$$P_r = \sigma'_v \cdot 2L_e b f^* \quad (2.46)$$

where L_e is the embedded length of reinforcement as depicted in Figure 0.16. Typically a factor of safety of 1.5 is used for ASD (Elias et al. 2001).

The FHWA has mandated the use of LRFD for bridges designed by state transportation departments. Because MSE walls are commonly used for bridge abutments and approaches, MSE wall design procedures are transitioning to LRFD. In order to be consistent with current design standards, the symbol ϕ_R was taken to represent the resistance factor; however, this should not be confused with friction angle. For LRFD pullout design, the following must be satisfied (Berg et al. 2009):

$$\phi_R L_e \geq \frac{T_{\max}}{f^* \sigma'_v 2R_c} \quad (2.47)$$

The resistance factors of for the static and seismic conditions are typically $\phi = 0.40$ to 0.75 and $\phi = 1.0$, respectively (Bathurst et al. 2008a; AASHTO 2010)

2.2.2 Design Methodology for External Stability

The reinforced soil mass is treated similar to a gravity wall for purposes of determining the external stability of MSE walls. As with gravity walls, four external, or global, failure mechanisms are typically considered (Christopher et al. 1990a). These mechanisms include sliding, limiting eccentricity (overturning), bearing failure, and deep seated instability and are summarized in Figure 0.32. Christopher et al. (1990a) provides an in-depth discussion on design procedures for each mechanism listed above; the most current design standards incorporate LRFD procedures and can be found in Berg et al. (2009). Because the focus of this research is on internal stability of MSE walls, interested readers are referred to Christopher et al. (1990a) and Berg et al. (2009).

2.3 Summary of the Literature Review

The state of practice for determining internal stability of MSE walls uses semi-empirical and empirical design procedures, some of which were developed over 30 years ago. This chapter reviewed the soil mechanics principles necessary to understand soil-reinforcement interaction with particular focus on the behavior of dilative soils, such as the response anticipated for compacted granular fill which is the focus of this study. The development of design procedures for internal stability has been discussed with regard to the development of reinforcement loads as well as the resistance. Six methods for determining reinforcement loads have been historically

used in North America. Of these, two are commonly used in the design of ribbed steel strip reinforcements: the Coherent Gravity and Simplified methods. A disadvantage of these two methods is that they do not directly account for global reinforcement stiffness in determining reinforcement loads or compaction induced loads during construction. Additionally, both of these design methods are empirically based and therefore limited in their use by the conditions in the database of case histories by which they were calibrated, such as their height or reinforcement density. An alternative method for determining reinforcement loads is the method developed by Ehrlich and Mitchell (1994). The Ehrlich and Mitchell (1994) method is theory-based and is therefore not bound by a case history database; also, this method explicitly accounts for compaction-induced stresses as well global reinforcement stiffness. Nonetheless, all of the models suffer from the requirement that a single friction angle be used in the computation of internal stresses; this presents a limitation that may be responsible for some inaccuracy in reinforcement load predictions, particularly for tall MSE walls that exhibit large stress gradients

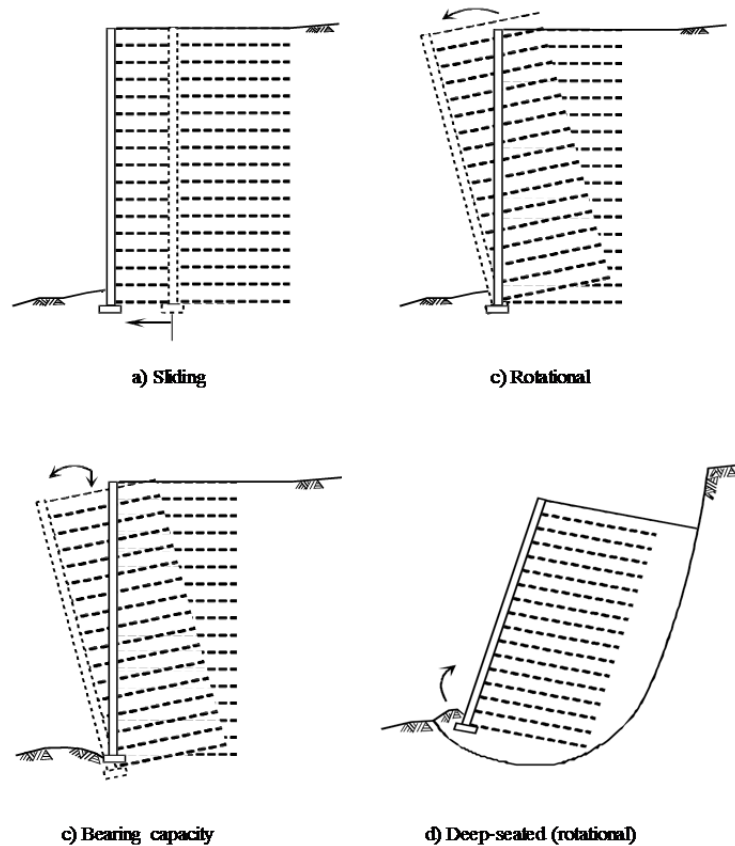


Figure 0.32. Global stability failure mechanisms for MSE walls (after Liang 2004).

In contrast to load prediction methods, resistance prediction methods, particularly pullout resistance, have remained relatively unchanged over the history of modern MSE wall design. The pullout resistance is calculated using a frictional model, however, due to dilation effects and accumulation of passive resistance, a constant friction coefficient cannot be used with varying

vertical effective stresses. This forces the designer to have to choose between using empirically based models or performing pullout tests on the specific reinforcement-backfill combination to be used in construction. It is not usually practical to perform pullout tests since the specific backfill to be used is not typically known prior to construction. However, the empirical model currently in use does not take into account soil type and consequently the apparent friction coefficient is often under-predicted when high strength-dilative soils are used. This apparent conservatism has not been evaluated for tall walls, and therefore this gap in the literature should be investigated.

Chapter 3 Research Objectives and Program

3.1 Research Objectives

The main objective of this study is to develop an improved understanding of the interaction between soil and ribbed steel reinforcements in MSE walls constructed using well-graded sandy gravel backfill and its role in providing internal stability to the reinforced soil system. Much of this research is performed using materials and data from two very tall MSE walls built during the SeaTac 3rd Runway Expansion Project. However, the intent of this study is not to focus on any specific wall, but add to the general knowledge of MSE wall design and performance. The specific objectives of this study include:

1. Characterization of the stress-strain behavior of the reinforcement material;
2. Characterization of the stress-strain response of the reinforced soil (i.e., backfill) material;
3. Determination of the soil-reinforcement interaction and pullout resistance of the reinforcement at varying normal stress magnitudes;
4. Development of a pullout resistance design model for widely-spaced reinforcement strips in well graded sand-gravel mixtures; and
5. Determination of the soil-reinforcement interaction of closely-spaced reinforcements and the effect on pullout resistance.

3.2 Research Program

The research program undertaken to achieve the objectives includes:

1. Perform tension tests on coupons cut from steel ribbed strips in order to measure its constitutive properties (Chapter 4);
2. Perform consolidated-drained axisymmetric triaxial strength tests at various effective confining pressures and relative densities on the reinforced backfill (Chapter 4);
3. Determine the curvature of the Mohr Failure Envelope and dilation characteristics of the backfill soil (Chapter 4);
4. Design and construct a pullout test apparatus for single reinforcement strips (Chapter 5);
5. Perform a series laboratory pullout tests at various normal effective stresses on the reinforced backfill and steel ribbed strip reinforcements (Chapter 5);
6. Develop a load-displacement model to estimate the pullout behavior of the reinforcements (Chapter 5);

7. Develop a new backfill-specific and global ultimate pullout resistance model for predicting the variation in apparent friction coefficient with normal effective stress for gravel backfills and steel ribbed strip reinforcements (Chapter 5);
8. Design and construct a pullout test apparatus for multiple reinforcement strips that allows the variation of the reinforcement spacing (Chapter 6); and,
9. Perform a series of laboratory pullout tests on closely- and widely-spaced reinforcement strips for various normal effective stresses (Chapter 6).

Chapter 4 Reinforcement and Reinforced Fill Soil Characterization

The characterization of soil-reinforcement interaction requires the measurement of index properties and strength of the soil and the reinforcement materials. This chapter presents the results of strength and index tests conducted on the ribbed steel strip reinforcements and the reinforced backfill materials, and discusses their role in the context of MSE walls.

4.1 Reinforcement Material Property Characterization

Generally speaking, the reinforcement strips manufactured by the Reinforced Earth Co. are 50 mm wide by 4 mm thick (Berg et al. 2009). However, the MSE wall reinforcements used in this study were galvanized ribbed steel strips 50 mm (2 in.) wide by 6 mm (0.24 in.) thick, because these were used in the construction of the SeaTac 3rd Runway Expansion Project (Stuedlein et al. 2010). The dimensions of the ribbed steel strip reinforcements are presented in Figure 0.1. In order to properly evaluate tensile stress-strain behavior of these reinforcement strips, tension testing was performed on the actual steel strips used in the 3rd Runway Project.

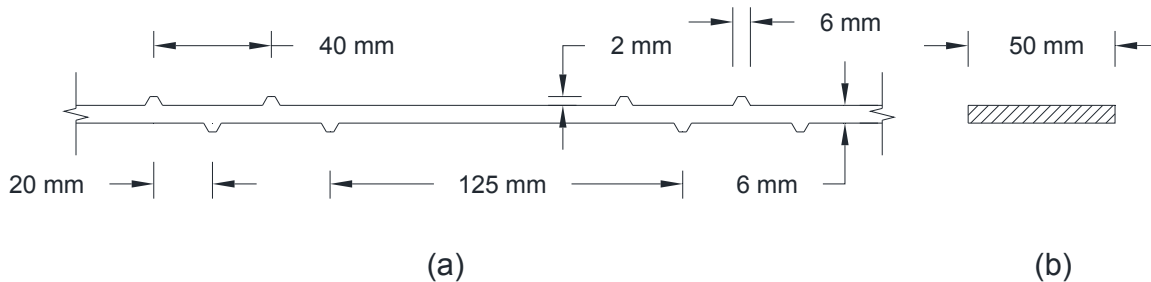


Figure 0.1 Steel ribbed reinforcement dimensions (a) elevation view, and (b) cross-section.

The constitutive parameters of the steel reinforcements such as the Young's modulus, yield strength, and ultimate strength, were needed to assess the internal stability of the MSE wall (presented in Chapter 6). Another motivation for studying the stress-strain behavior of the reinforcement material is the calibration of numerical models for use in analyzing the behavior of tall MSE walls. However, detailed numerical model calibration is beyond the scope of this thesis and therefore will not be discussed herein.

To measure the constitutive properties mentioned above, tension tests were conducted on steel coupons. Six dog-bone-shaped steel coupons, with dimensions shown in Figure 0.2, were cut from a MSE wall reinforcement strip. All tests were performed on an Instron universal testing machine in accordance with ASTM standards E8-08 and A370-12a. An extensometer with a gauge length of 50 mm (2 in.) and a 6 mm (2.5 in.) range was used to measure strain through yielding and strain hardening. Due to the limited range of the extensometer the tests were paused at 10 percent strain and the extensometer removed. A computer-controlled camera was used to measure strain for the remainder of the test. The specimens were designated S1 through S6. Samples S1 and S2 are not considered in this study because of software malfunctions that led to an inability to make extensometer measurements.

The cross head (Figure 0.2) speed was set to 3.05 mm per minute for all specimens. Approximately two and a half minutes was required to rupture each specimen. The information

obtained from the coupon testing program includes the Young’s modulus, the upper and lower yield strength, the yield point elongation, the ultimate tensile strength, the maximum elongation, and the area reduction. The yield strength was determined using the Autographic Diagram Method (ASTM 2008), and the elongation and area reduction were measured after rupture.

The stress-strain behavior of each specimen was similar to one another and produced an average modulus of elasticity of 211 GPa, a mean yield stress of approximately 530 MPa, a average yield point elongation of 0.92 percent, and a mean ultimate tensile strength of 700 MPa, summarized in Table 0.1 and Figure 0.3. The “representative” values listed in Table 0.1 were chosen using judgment in order to reduce the effect of possible outliers and were selected to model the strips in subsequent chapters.

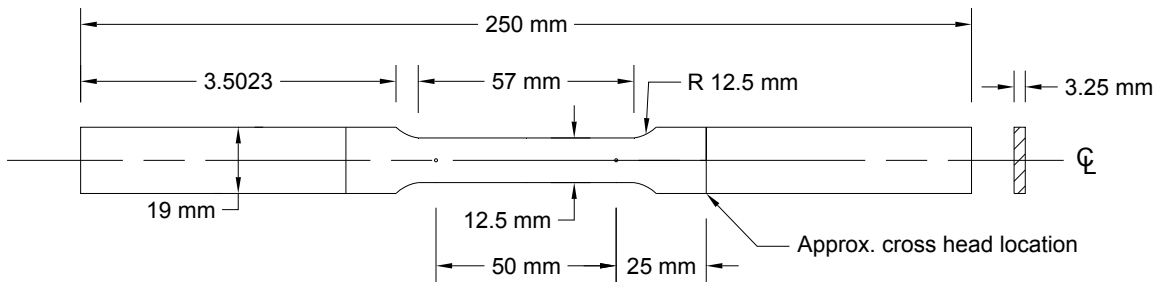


Figure 0.2. Typical coupon dimensions.

Table 0.1. Summary of MSE wall reinforcement material testing.

Specimen ID	Modulus of Elasticity (GPa)	UYS ^a (MPa)	LYS ^b (MPa)	YPE ^c (percent)	Ult. Tensile Strength (MPa)	Max. Elongation (percent)	Area Reduction ^d (percent)
S3	208.1	531	527	0.97	701	23	53
S4	207.2	529	526	0.88	690	22	50
S5	220.4	528	526	0.89	701	21	50
S6	209.0	534	530	0.94	707	23	54
Average	211.2	531	527	0.92	700	22	52
Representative	208.0	530	526	-	700	-	-

^aUpper Yield Strength, ^bLower Yield Strength, ^cYield Point Elongation, ^darea reduction was measured post rupture.

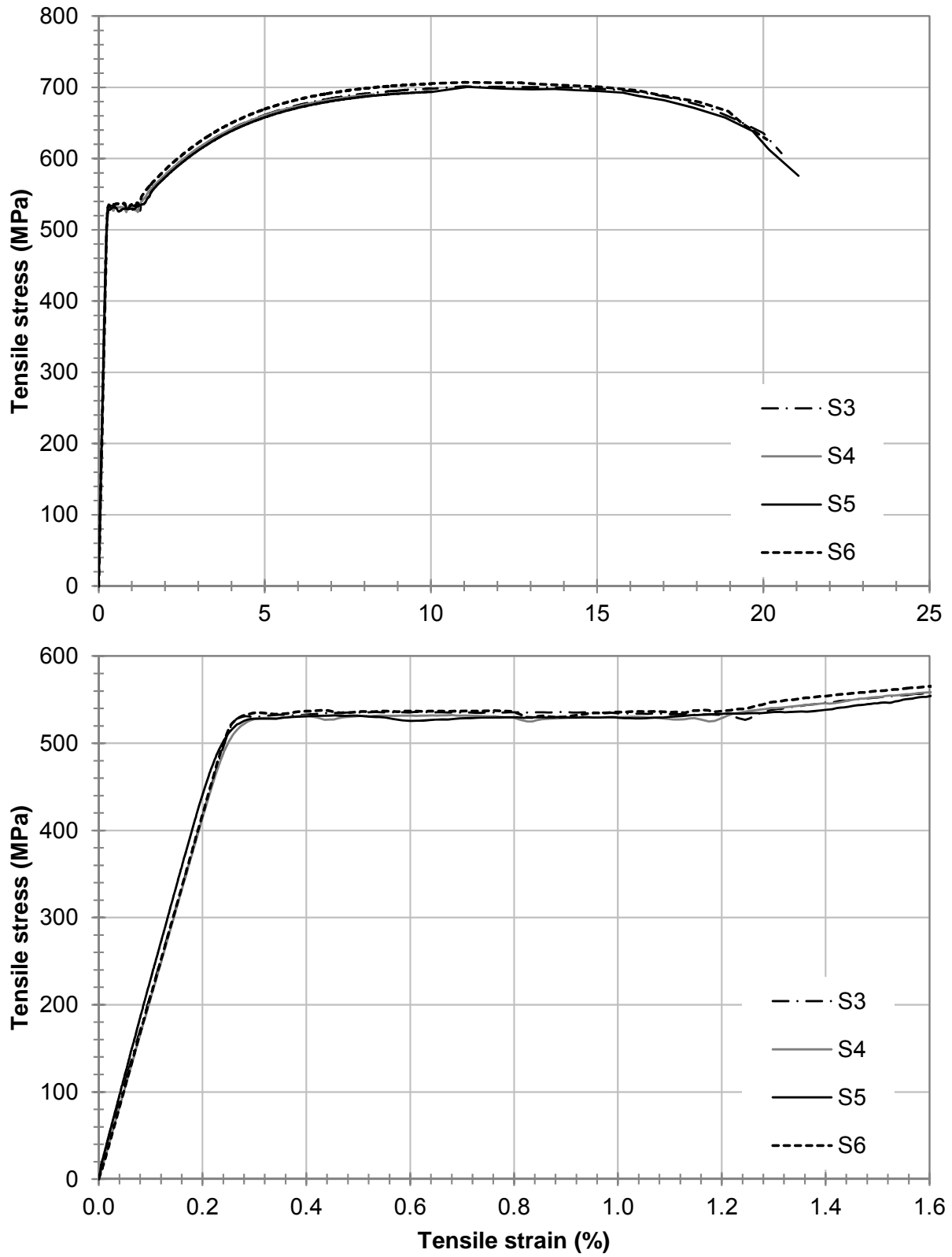


Figure 0.3 Stress - strain behavior of steel coupons.

4.2 Index and Shear Strength Testing of Reinforced Backfill

The reinforced backfill material used in the SeaTac MSE walls was quarried from a glacial outwash deposit located near Kanaskat, Washington at approximately N47°19'09", W121°52'57". The gravel particles ranged from sub-angular to well-rounded, and the sand particles ranged from angular to sub-round as determined by visual inspection. Gradation analysis, modified Proctor, maximum void ratio, and consolidated drained triaxial compression tests were performed on the backfill material. The results from these tests are discussed below.

4.2.1 Gradation Analysis

Approximately five cubic meters of the reinforced backfill material was delivered to the OSU geotechnical research laboratory. Six 32 kg samples were randomly extracted from the backfill material. Gradation analyses were performed on each sample and presented in Figure 0.4. The Quality Assurance (QA) limits in Figure 0.4 and Figure 0.5 plotted as bold solid black lines represent the rejection criteria for the reinforced backfill used in the SeaTac 3rd Runway Project, and indicate that the gradation of the soil used as reinforced backfill could not fall outside these lines.

The maximum and minimum grain sizes for each sample were similar, however, the average grain sizes, D_{50} ranged from 2 mm to 8 mm. The grain size distributions of all samples showed a “gap” with significantly small portions of material having grain sizes between 7 and 1 mm. Due to this “gap” the coefficient of curvature, C_c , was calculated to be less than 1, where C_c is determined using:

$$C_c = \frac{(D_{30})^2}{D_{10} \cdot D_{60}} \quad (3.1)$$

where D_{10} , D_{30} , and D_{60} are the grain sizes at which 10, 30, and 60 percent of material (by mass) are smaller, respectively. According to the Unified Soil Classification System, gravels and sands with coefficients of curvature less than one or greater than three classify as poorly-graded. However, if the ratio of D_{60} to D_{10} , the coefficient of uniformity, C_u , is greater than four, a gravel is considered well-graded. Though the coefficient of uniformity for the average sample gradation was 42, none of the samples classified as well-graded due to the C_c criterion.

The target gradation for the axisymmetric CD triaxial strength tests shown in Figure 0.5 was established in order provide a standard test gradation. The target gradation was developed by limiting the maximum grain size to 25.4 mm (1 in) and averaging the results of the six gradations. The target gradation had a $D_{50} = 6$ mm, a $C_u = 42$, and a $C_c = 0.46$. In order to reproduce the target gradation for subsequent CD triaxial testing, the backfill material was separated into 12 grain size ranges and then carefully recombined to produce the required 17 kg samples as shown in Figure 0.6.

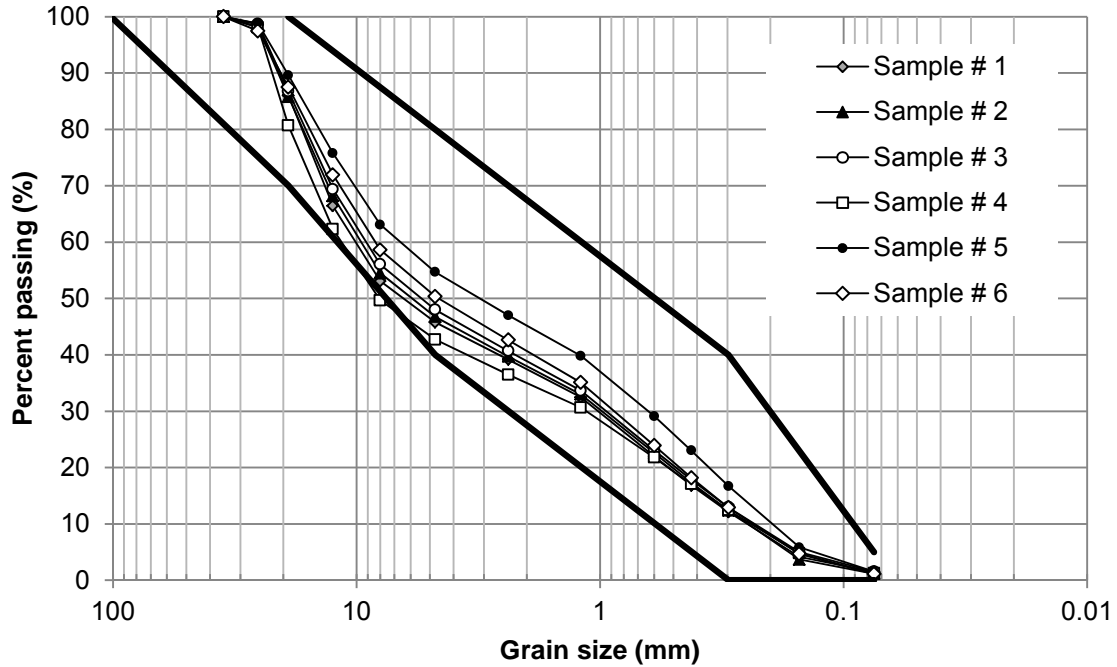


Figure 0.4 Individual gradations of six random samples of reinforced backfill material and the upper and lower bound gradation limits used for quality assurance (QA) during the SeaTac 3rd runway expansion.

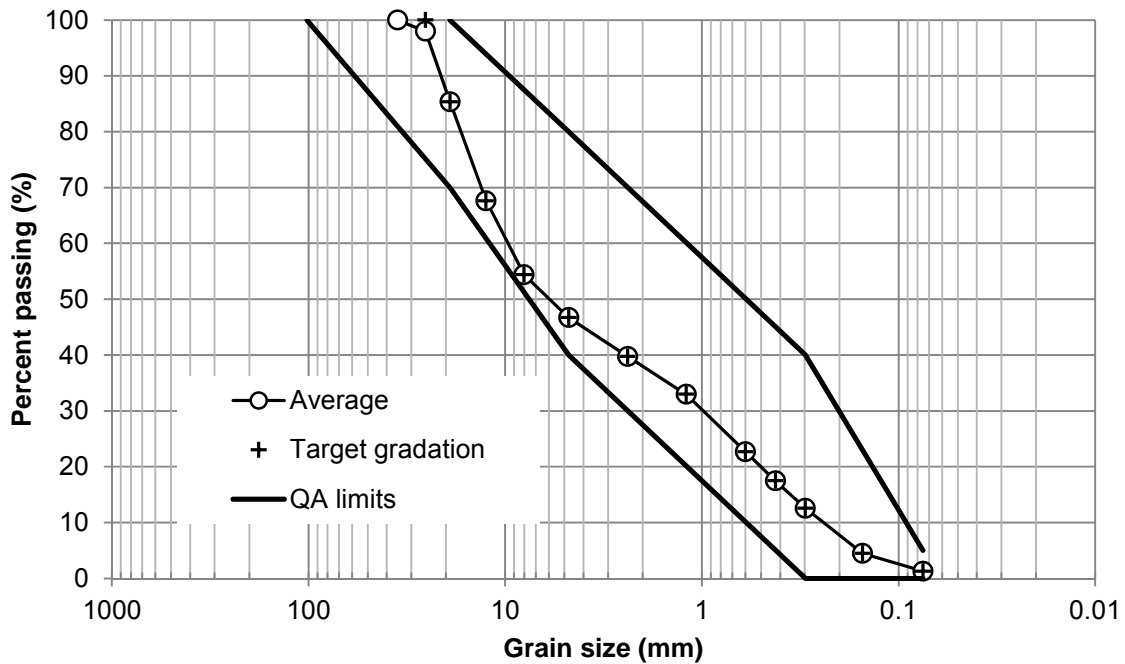


Figure 0.5. Average gradation of the reinforced back fill as measured at OSU and the target gradation for CD triaxial testing.

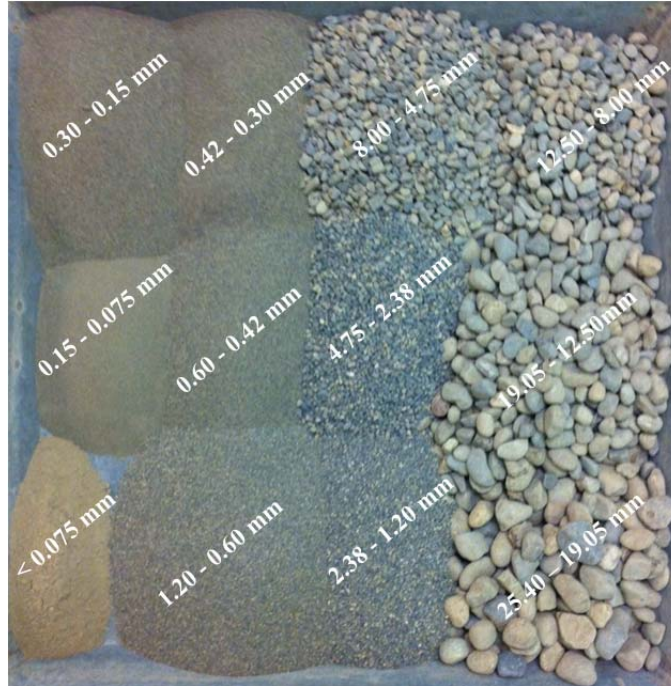


Figure 0.6. Photo of a 17 kg triaxial sample prior to mixing (maximum grain size = 25.4 mm).

4.2.2 Modified Proctor Tests

The specifications for the SeaTac MSE wall required that the reinforced backfill be compacted at ± 2 percent of optimum water content, and to a minimum of 92 percent of the maximum modified Proctor dry density (Stuedlein et al. 2010b). In order to determine the maximum modified Proctor dry density, and thus be able to select a target relative densities for the CD triaxial test specimens, modified Proctor tests were carried out on the target gradation. The modified Proctor tests were performed by FEI Testing & Inspection of Corvallis, OR (FEI 2011), and verified by the OSU geotechnical lab. Method C as defined in ASTM D1557 (ASTM 2009) was used. Method C required oversized particles to be removed and a correction to be applied to the resulting compaction curve. Both the uncorrected and corrected proctor curves, as reported by FEI (2011), are presented in Figure 0.7. The maximum dry unit weight and optimum moisture content was determined equal to 22.4 kN/m^3 and 6.5 percent, respectively.

4.2.3 Maximum and Minimum Void Ratio and Relative Density

The state of compaction of soils can be described using many metrics such as the dry unit weight, dry density, relative compaction, porosity, and void ratio. However, the state of compaction for granular materials is typically referred to in terms of relative density, D_r , given by:

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \quad (3.2)$$

where e is the void ratio, e_{\max} is the maximum void ratio or the void ratio at the loosest possible state of the soil, and e_{\min} is the minimum void ratio or the void ratio at a soil's densest possible state prior to particle crushing. Relative density, when compared to void ratio alone, typically provides the best correlation for strength (Cornforth 1973), and will therefore be the basis of discussion for this study.

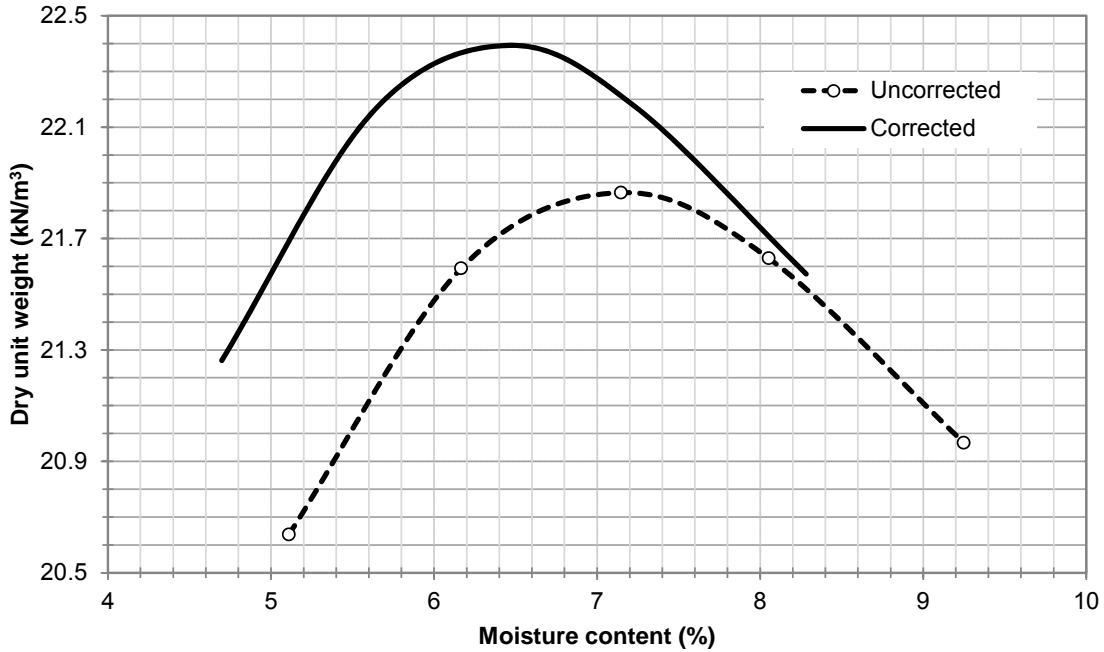


Figure 0.7 Corrected and uncorrected Modified – C Proctor curves as reported by FEI (2011).

To determine the maximum void ratio of the backfill material, Method A, described in ASTM D4254 (ASTM 2006a) was performed with the following exception: a Modified-C Proctor mold with a volume of 2124 cm^3 (0.075 ft^3) was used instead of the specified 2830 cm^3 (0.10 ft^3) mold due to availability of testing equipment. Seven tests were performed on the backfill design gradation and a mean maximum void ratio, e_{\max} , of 0.365 and a standard deviation of 0.003 was determined as shown in Table 0.2.

An effort was made to determine the minimum void ratio, e_{\min} , using the procedure outlined in ASTM D4253 (ASTM 2006b) in which a surcharge is placed on top of a cylinder filled with dry soil and then placed on a vibrating table. Densities obtained using method D4253 were significantly less than those obtained during the modified Proctor tests and thus did not represent 100 percent relative density. Following the recommendation of Duncan et al. (2007), the maximum dry unit weight was designated as the corrected maximum modified proctor dry unit weight of 22.4 kN/m^3 , corresponding to a minimum void ratio of $e_{\min} = 0.182$.

Table 0.2 Results from maximum void ratio tests on backfill material.

Test No.	Measured Void Ratio, e
1	0.365
2	0.363
3	0.368
4	0.368
5	0.360
6	0.368
7	0.364
Mean	0.365
St. Dev.	0.003

4.2.4 Consolidated Drained Triaxial Compression Tests

An extensive series of consolidated drained triaxial compression tests was performed on the target gradation at three different relative densities. A triaxial testing system was developed at the OSU Geotechnical Laboratory in order to perform these tests and in doing so a triaxial testing manual, that complies with ASTM (2011b), was developed for future research. The results from the CD triaxial tests are presented below.

Specimen Compaction Protocol and Test Program

In order to measure the effect of relative density on the constitutive behavior of the reinforced backfill, three target relative densities were considered for the CD triaxial testing series. These target relative densities were, 35, 55, and 65 percent, with corresponding target unit weights of 20.3, 20.9, and 21.3 kN/m³, respectively. Each cylindrical specimen was compacted in eight 3.81 cm (1.5 in.) thick lifts using a Modified Proctor hammer and a 152.4 mm (6 in.) diameter by 304.8 mm (12 in.) tall split compaction mold. The number of blows per lift was adjusted depending on the target relative density. Only specimens with initial relative densities that achieved +/- 2.5 percent of the target relative density were sheared in the test program.

The saturation of each specimen was determined by measuring the B-value as prescribed by ASTM (2011b). Black and Lee (1973) found that for very stiff soil structures the B-value corresponding to 100 percent saturation can be as low as 0.91. Therefore, a minimum B-value of 0.90 was required for all triaxial shear strength tests. Each specimen was sheared using the same axial strain rate of $d\varepsilon_1/dt = 0.25$ percent per minute. This axial strain rate is in accordance ASTM D7181 if a time to 90 percent consolidation, t_{90} , of 1.5 minutes is conservatively assumed. Each specimen was given a name indicating the effective confining pressure and the overconsolidation ratio (OCR) as well as the “test number” (e.g., σ'_3 in kPa – OCR – test number). For example the third specimen sheared at 100 kPa with and OCR = 1 would be labeled as specimen 100-1-3. Eight tests with varying confining pressures were performed at each target relative density for a total of 24 tests. The target confining pressures included 10, 20, 50, 100, 200, 300, 500, and 1,000 kPa. Table 0.3 summarizes the results from each CD triaxial test.

CD Triaxial Results

In this study, the two controlled variables of interest were relative density and confining pressure. Originally the effect of OCR was considered, however after initial testing and further review of the literature, the effect of OCR was found to be insignificant as compared to relative density. Since relative density affects the behavior of granular materials it is important to note that for a given specimen the relative densities before and after consolidation were typically not the same. It was observed that the magnitude of change in relative density depended on the consolidation confining pressure and the initial relative density. Table 0.3 and Figure 0.8 summarize the change in relative density for all of the strength tests. The increase in relative density due to consolidation is larger for tests with higher confining pressures and lower initial relative densities. In order to replicate field conditions, the initial (i.e., before consolidation) relative density is used as the basis for comparison.

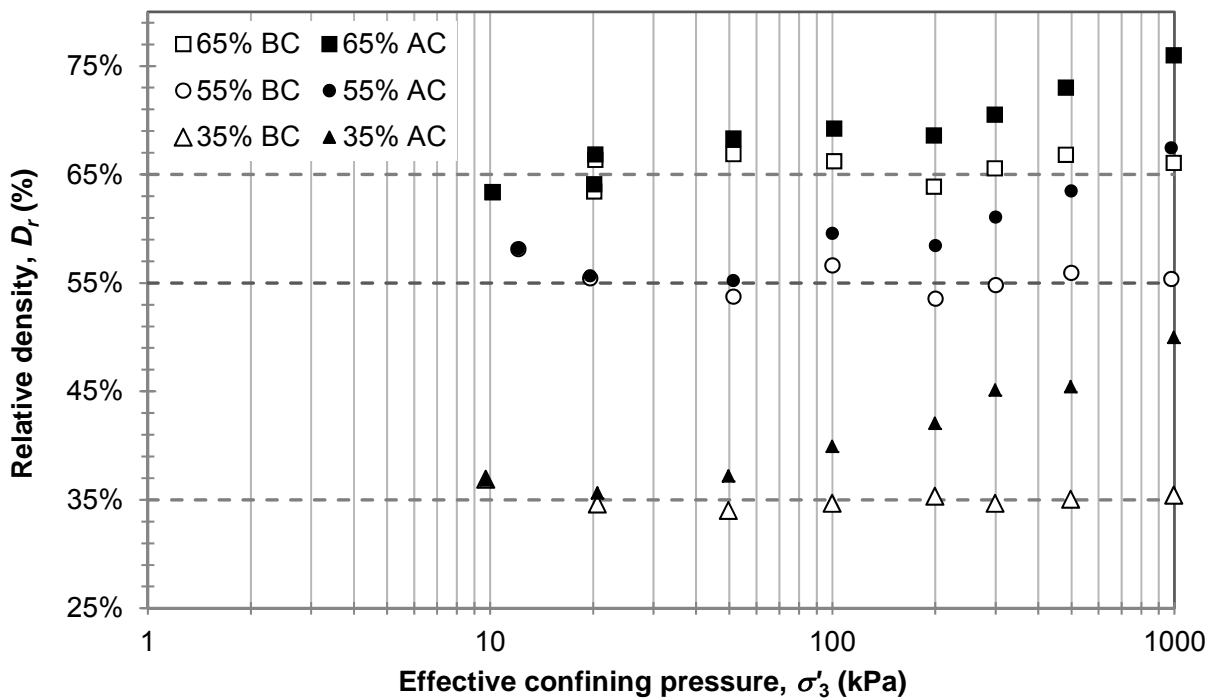


Figure 0.8. Relative density versus confining pressure for all test (BC = before consolidation, AC = after consolidation).

Table 0.3. Summary table of CD triaxial results.

Test designation	Initial void ratio, e_{BC} ^a	Initial relative density, $D_{r,BC}$ ^a	Void ratio after consolidation e_{AC} ^b	Relative density after consolidation $D_{r,AC}$ ^b	Peak friction angle (deg.) ϕ_{peak}
10-1-9	0.301	0.369	0.301	0.368	43.7
20-1-5	0.305	0.346	0.303	0.356	41.3
50-1-6	0.306	0.340	0.300	0.372	39.3
100-1-5	0.305	0.347	0.295	0.399	38.7
200-1-6	0.304	0.353	0.291	0.421	40.0
300-1-4	0.305	0.347	0.285	0.451	38.3
500-1-4	0.304	0.350	0.285	0.454	39.1
1000-1-3	0.303	0.354	0.276	0.500	38.8
10-1-11	0.261	0.581 ^c	0.261	0.581	50.1
20-2-3	0.266	0.554	0.265	0.556	48.4
50-1-4	0.269	0.537	0.266	0.552	45.6
100-1-2	0.264	0.566	0.258	0.595	45.9
200-1-1	0.269	0.535	0.260	0.584	44.2
300-1-3	0.267	0.548	0.255	0.610	43.2
500-1-1	0.265	0.559	0.251	0.634	41.6
1000-1-2	0.266	0.553	0.243	0.674	40.9
10-1-5	0.251	0.633	0.251	0.633	54.3
20-1-3	0.251	0.634	0.250	0.641	51.6
20-1-4	0.245	0.663	0.244	0.668	54.1
50-1-3	0.244	0.669	0.242	0.683	51.1
100-1-3	0.246	0.662	0.240	0.692	47.1
200-1-2	0.250	0.639	0.241	0.686	45.2
300-1-1	0.247	0.655	0.237	0.705	44.6
500-1-3	0.244	0.668	0.233	0.730	43.3
1000-1-1	0.246	0.660	0.227	0.760	41.6

^a Before Consolidation (BC), ^b After Consolidation (AC), ^c Test 10 - 1 - 11 did not meet density criteria by 0.6 percent.

Figure 0.9, Figure 0.10, and Figure 0.11 show the effective principal stress difference-axial strain behavior for tests at 35, 55, and 65 percent relative density, respectively. As shown in Figure 0.11, strain softening occurred at confining pressures up to and including 10 atmospheres (i.e., 1000 kPa) for specimens at a relative density of 65 percent. The amount of strain softening and the initial slopes of the stress-strain curves at 55 percent relative density, shown in Figure 0.10, were smaller than that observed at 65 percent relative density, and was stronger for data at 35 percent relative density, shown in Figure 0.9, as expected from well-known soil mechanics described in Chapter 2. See Appendix A for individual stress-strain plots.

An important constitutive parameter for soil is the Young's modulus defined as the ratio of the change in principal stress difference to the change in axial strain. Due to the non-linear stress-strain response of soil, Young's modulus is typically reported as an initial tangent modulus, E_i . Figure 0.12(a) presents the variation in initial tangent modulus with effective confining pressure for the 24 specimens evaluated. The initial tangent modulus was computed by dividing the principal stress difference at approximately 0.1 percent axial strain by the corresponding axial strain. In general E_i was found to vary with σ'_3 according to Equation 2.9 (Duncan et al. 1980):

$$\frac{E_i}{P_{atm}} = K \left(\frac{\sigma'_3}{P_{atm}} \right)^n \quad (2.9)$$

where K is the modulus number, n is the modulus exponent, and P_{atm} is atmospheric pressure.

The initial tangent modulus tended to be larger for specimens sheared at larger effective confining pressures and greater relative densities. Computed values of initial tangent modulus ranged from 18 to 165 MPa. Due to fluctuations in pressure supply, larger variability was observed in E_i for specimens sheared at lower confining pressures, consistent with other mechanical properties as described subsequently. Due to this variability the initial tangent modulus versus confining pressure was plotted again with only those specimens sheared at $\sigma'_3 > 20$ kPa and presented in Figure 0.12(b). With the specimens at lower confining pressure removed two general patterns became apparent: increasing the relative density caused K to increase and the modulus exponent n to decrease.

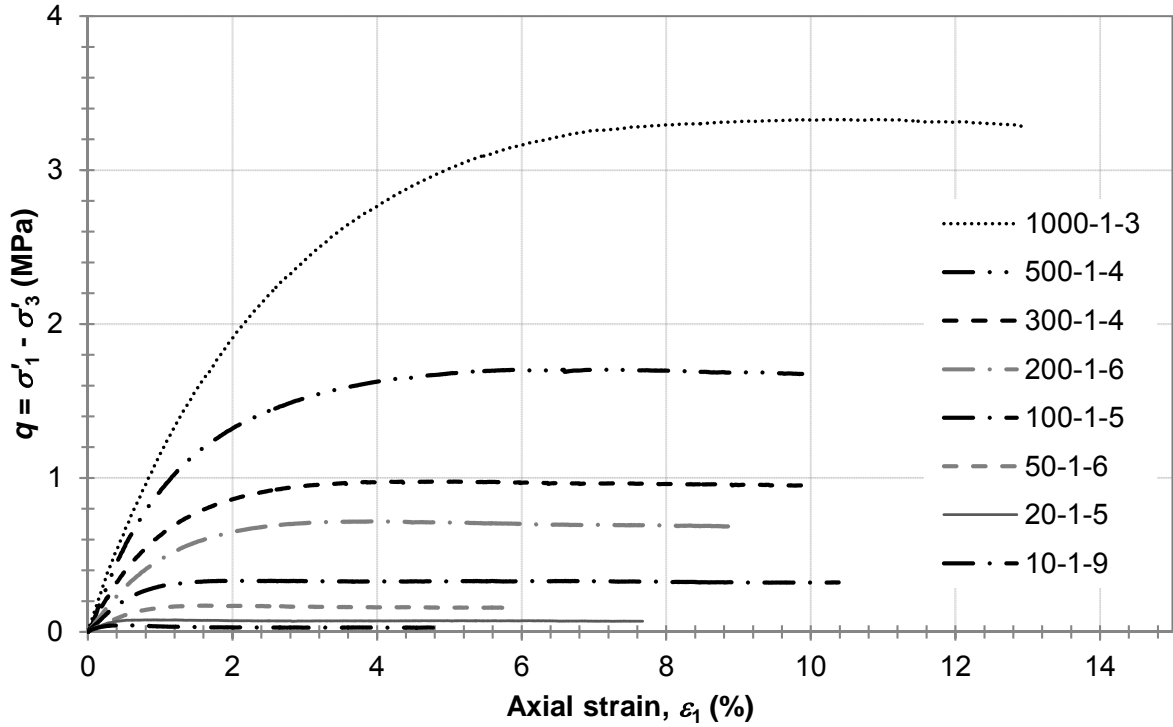


Figure 0.9. Effective principal stress difference versus axial strain for tests at 35 percent relative density.

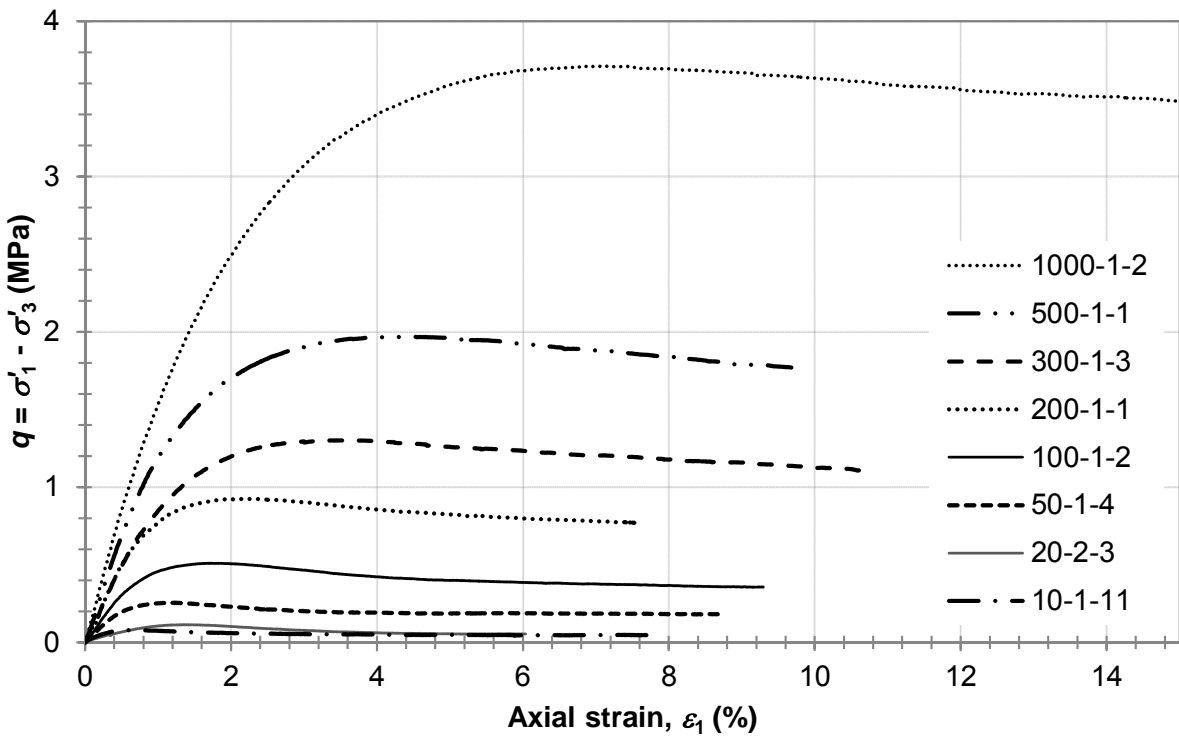


Figure 0.10 Effective principal stress difference versus axial strain for tests at 55 percent relative density.

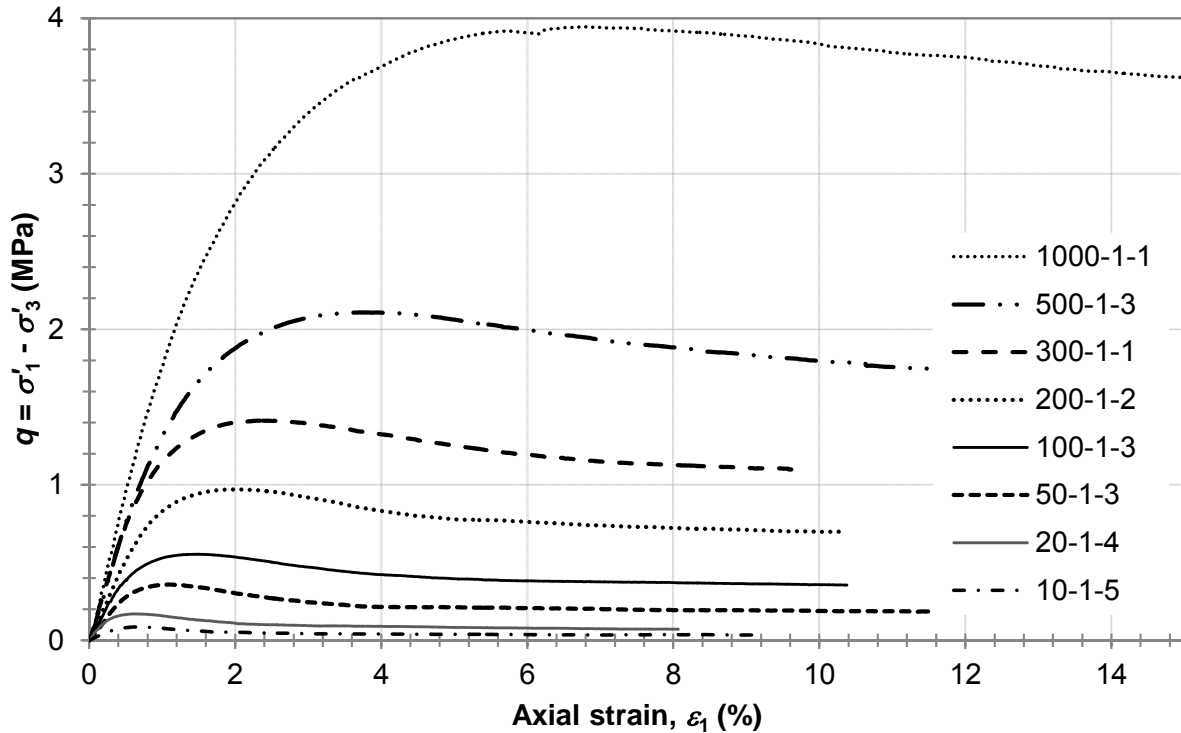


Figure 0.11 Effective principal stress difference versus axial strain for tests at 65 percent relative density.

For CD triaxial strength tests, failure is typically defined by the maximum principal stress difference $(\sigma'_1 - \sigma'_3)_{\max}$, the maximum principal stress ratio $(\sigma'_1 / \sigma'_3)_{\max}$, or at a predetermined amount of shear strain. For the present study, failure was defined as the maximum principal stress ratio as it provides a convenient way to visualize trends in the stress-strain response over a large range in confining pressures. Principal stress ratio versus axial strain for tests at 35, 55, and 65 percent relative density are presented in Figure 0.13, Figure 0.14, and Figure 0.15, respectively. The principal stress ratio increases gradually for tests sheared at $\sigma'_3 = 1000$ kPa to a maximum value at axial strains greater than 5 percent. For specimens sheared at $\sigma'_3 < 1000$ kPa, the principal stress ratio increases more rapidly, reaching a distinct peak before decreasing to a residual value approximately equal to that of the specimen sheared at $\sigma'_3 = 1000$ kPa. The rate of post-peak decay was greater for specimens sheared at lower confining pressures and higher relative densities. This behavior indicates that granular soil exhibits more dilatancy at lower confining pressures and higher relative densities, constant with values reported by Taylor (1948), Bishop (1954), Rowe (1962), Lee and Seed (1967), Rowe (1969), and Bolton (1986). Generally, the peak principal stress ratio tended to be larger at higher relative densities. Furthermore, the peak principal stress occurs at larger axial strains as the effective confining pressure increases, indicating that “failure” occurs at larger strains for specimens sheared at higher effective confining pressures. Figure 0.17 presents the principal stress ratio and axial strain at failure for all test specimens. The previous observation that the peak principal stress ratio tends to be larger for specimens with higher relative densities is reinforced by presenting Figure 0.18 and Figure 0.19.

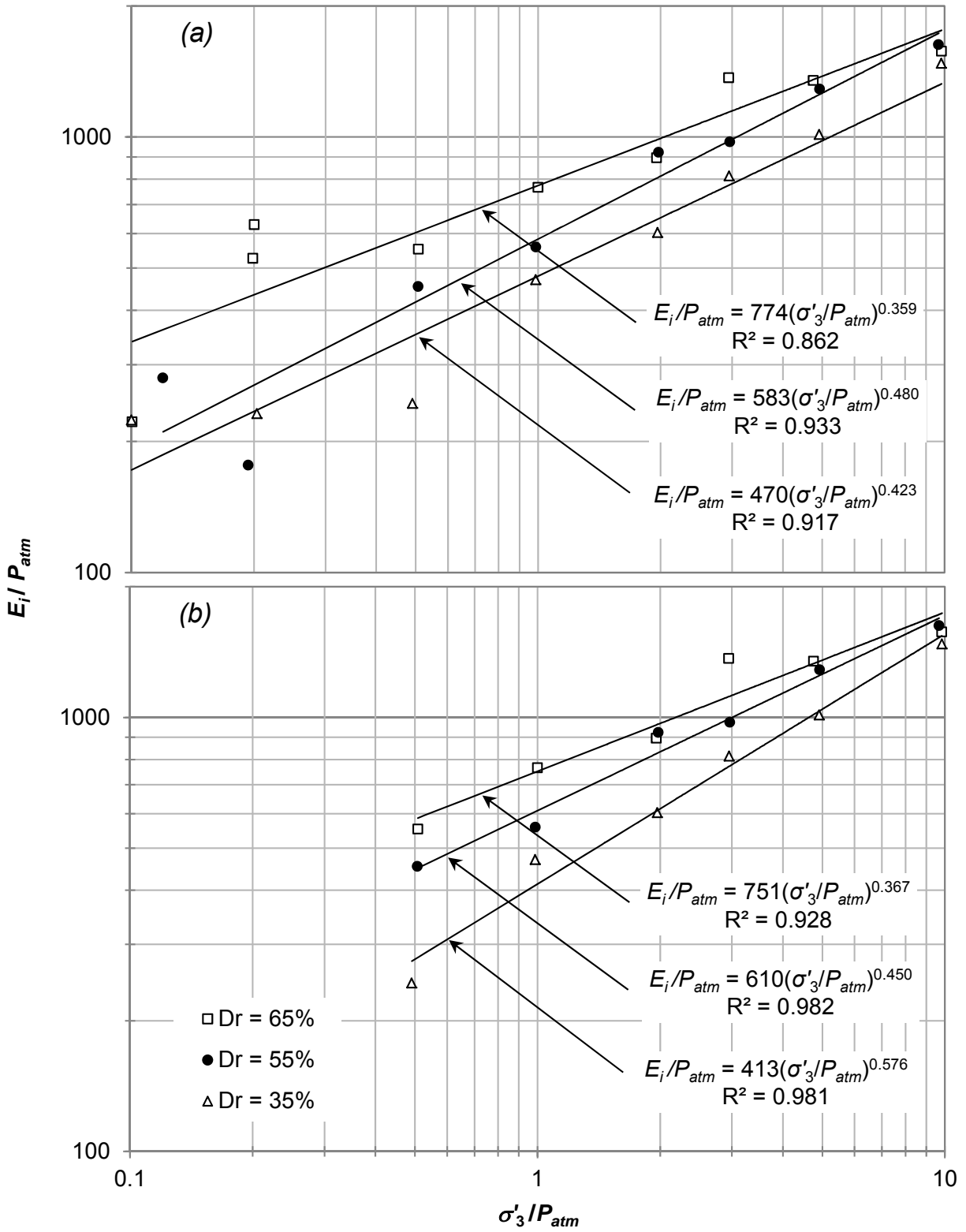


Figure 0.12. Initial tangent modules versus effective confining pressure normalized to atmospheric pressure for (a) all specimens and (b) excluding specimens sheared at $\sigma'_3 \leq 20$ kPa.

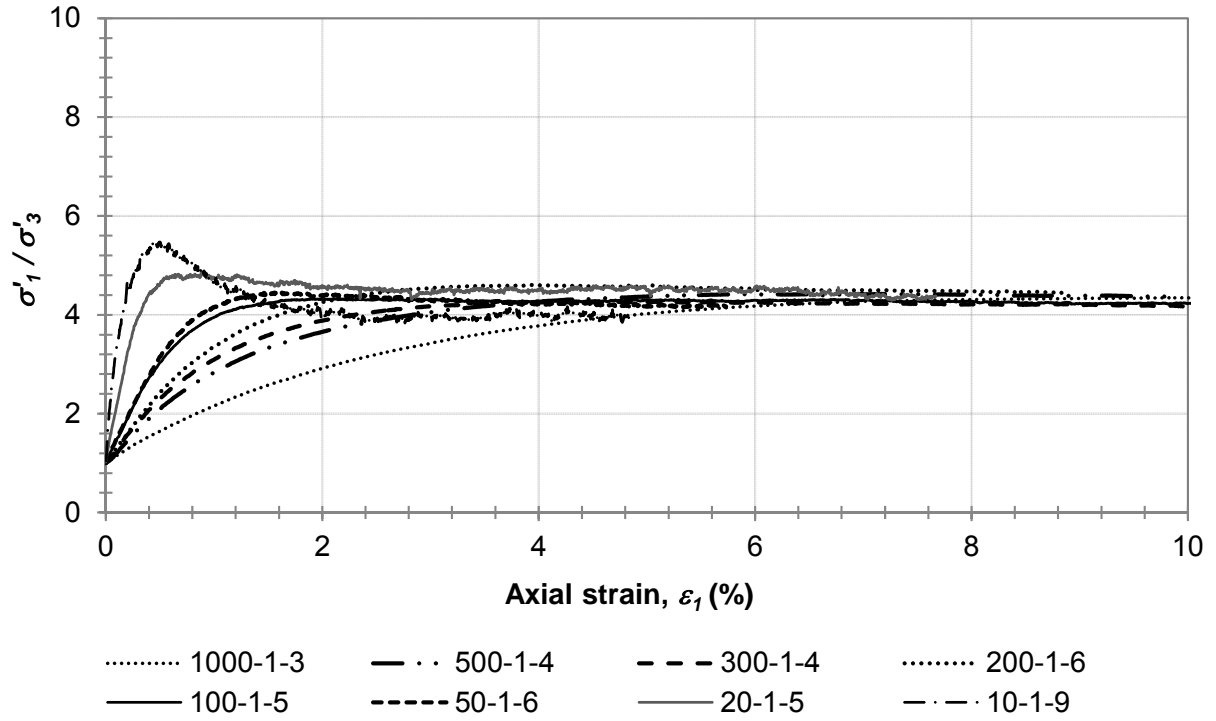


Figure 0.13. Principal stress ratio versus axial strain for tests at 35 percent relative density.

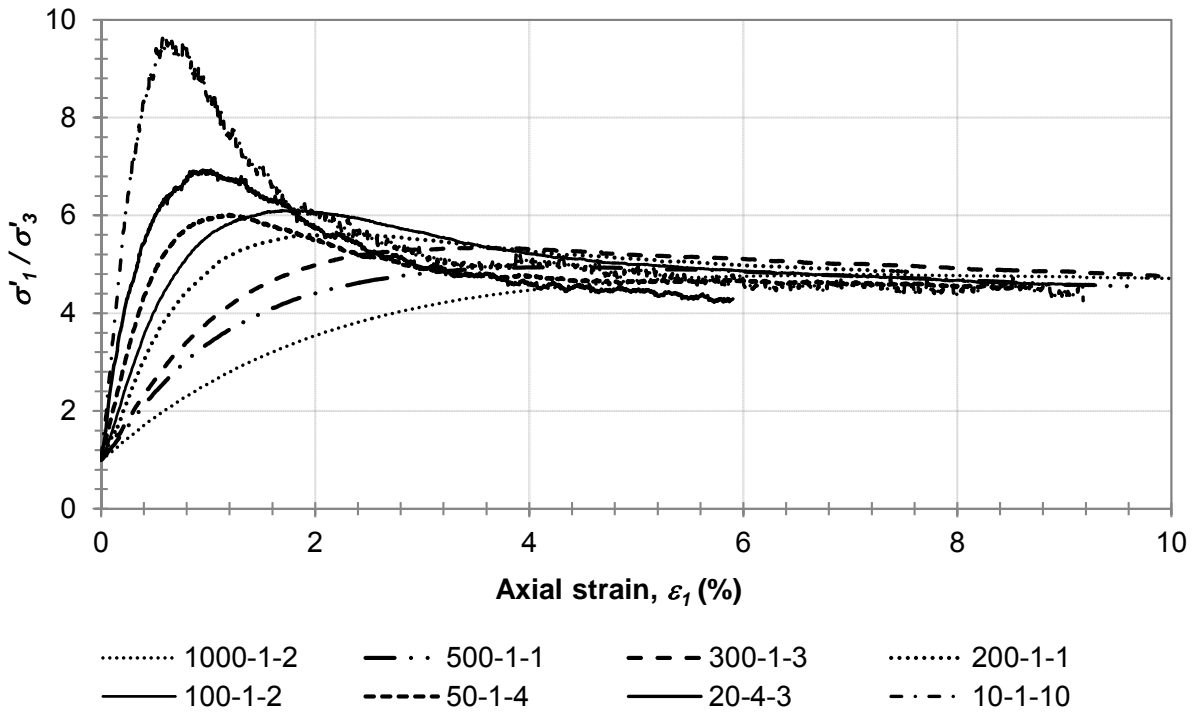


Figure 0.14. Principal stress ratio versus axial strain for tests at 55 percent relative density.

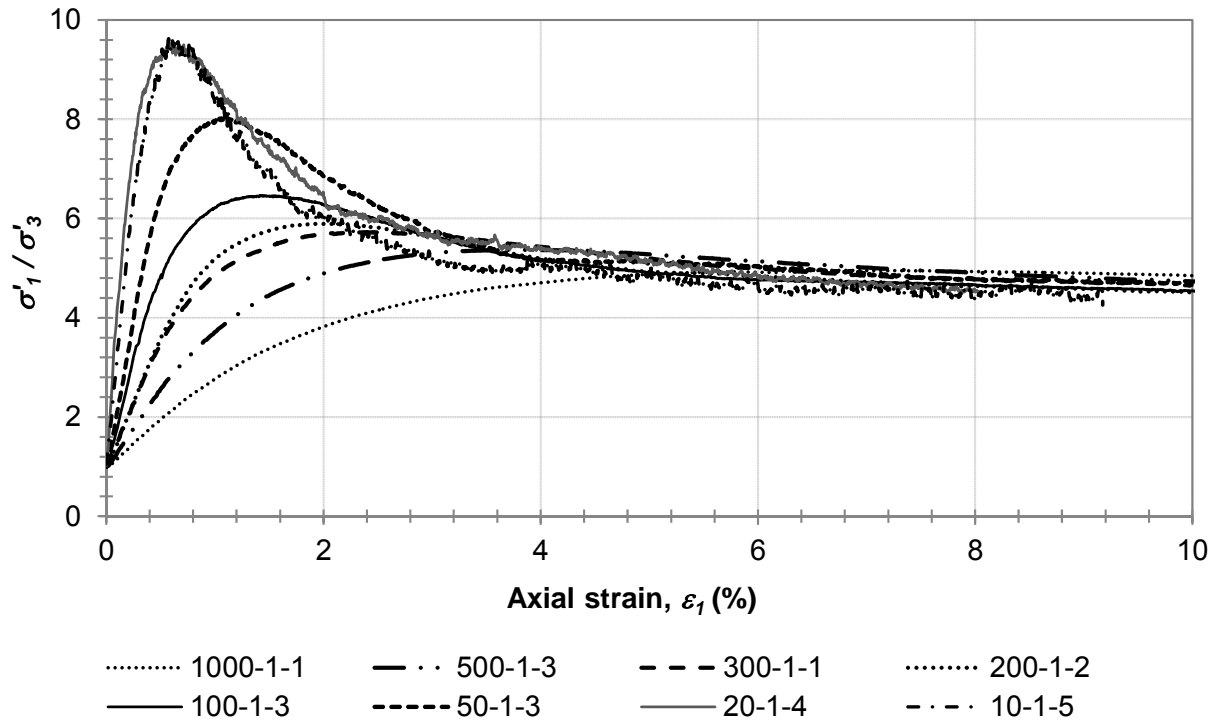


Figure 0.15. Principal stress ratio versus axial strain for tests at 65 percent relative density.

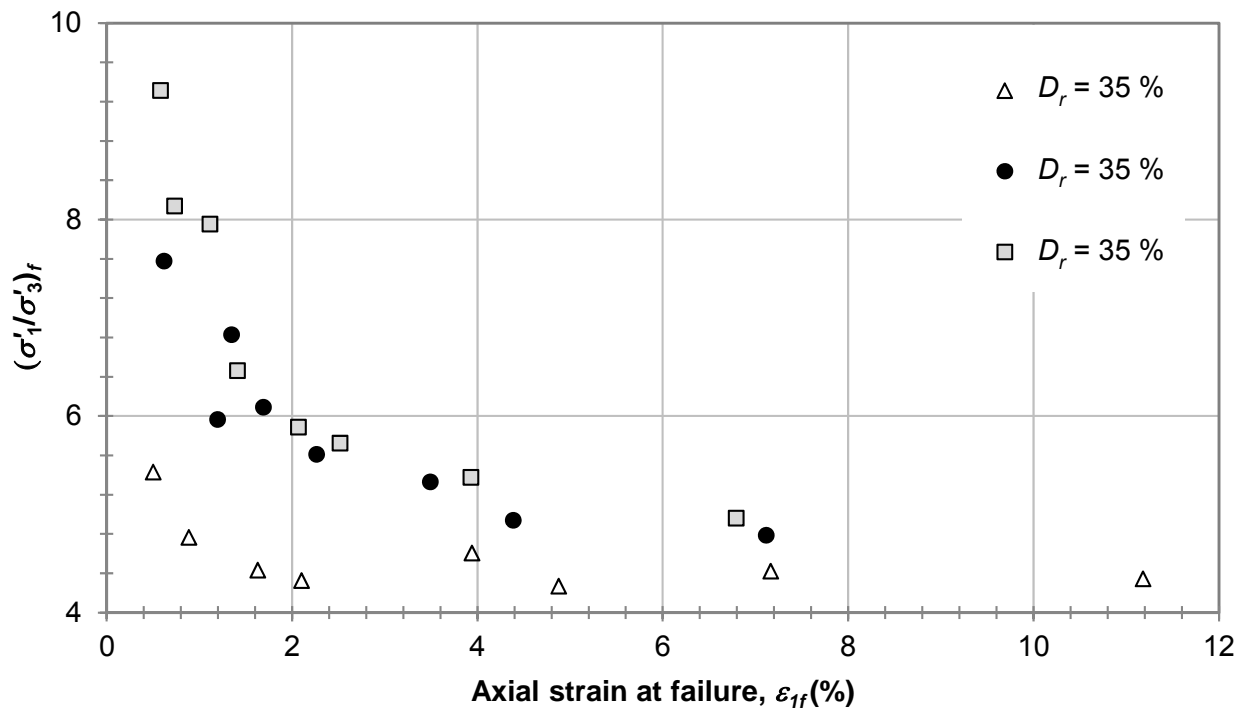


Figure 0.16. Principal stress ratio at failure versus axial strain at failure.

The volumetric behavior of dense granular materials is unique as it tends to increase in volume as it shears. Figure 0.17, Figure 0.18 and Figure 0.19 show the volumetric strain versus axial strain for specimens at 35, 55, and 65 percent relative density, respectively. Initially all specimens displayed contractive behavior (positive volumetric strain; note: expansion is considered negative in this thesis). For denser specimens (i.e. specimens with $D_r = 55$ or 65 percent) and for loose specimens sheared at lower confining pressures, the slope of the volumetric strain versus axial strain reversed, becoming progressively more negative until failure occurred corresponding to an inflection point on the volumetric strain-axial strain curve. For loose specimens sheared at higher confining pressures (i.e., tests 500-1-4 and 1000-1-3) the volumetric strain increased near-monotonically throughout the range in axial strains experienced. The volumetric strains recorded at large axial strains for relative densities of 55 and 65 percent shows continued linear dilative behavior. This behavior does not agree with evidence that suggests soil in a shear zone will reach a critical state where shearing can occur without volume change (Roscoe 1970). This observed linear dilation may be explained by considering that the deformed shape of the specimen progressively deviates from the assumed deformed shape due to membrane and end restraints (Bishop and Henkel 1962). The difference between the assumed and observed triaxial deformation is shown conceptually in Figure 0.20. Since the volumetric strains recorded at larger axial strains may be inaccurate, the volumetric behavior at these strains was not considered.

As discussed in Chapter 2, the maximum rate of dilation is defined as the ratio of change in volumetric strain and change in axial strain during the dilative portion of shearing, $(\Delta\varepsilon_v/\Delta\varepsilon_1)_{\max}$. As shown in Figure 0.17, Figure 0.18, and Figure 0.19 the maximum dilation rate depended on the confining pressure and relative density. Specimens sheared at low confining pressures and higher relative densities tended to have larger dilation rates. Dilation rates ranged from -2.83 for the specimen sheared at 10 kPa with a relative density of 65 percent, to 0.14 for the specimen sheared at 1000 kPa with a relative density of 35 percent. See Appendix B for individual volumetric strain-axial strain plots.

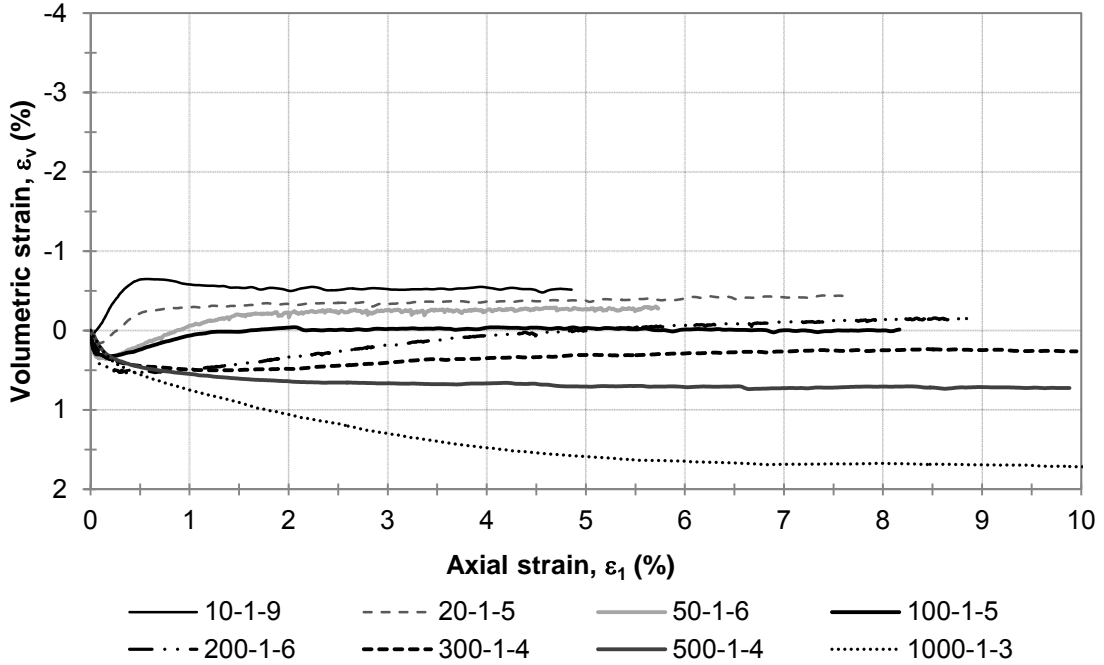


Figure 0.17. Volumetric strain versus axial strain for tests at 35 percent relative density.

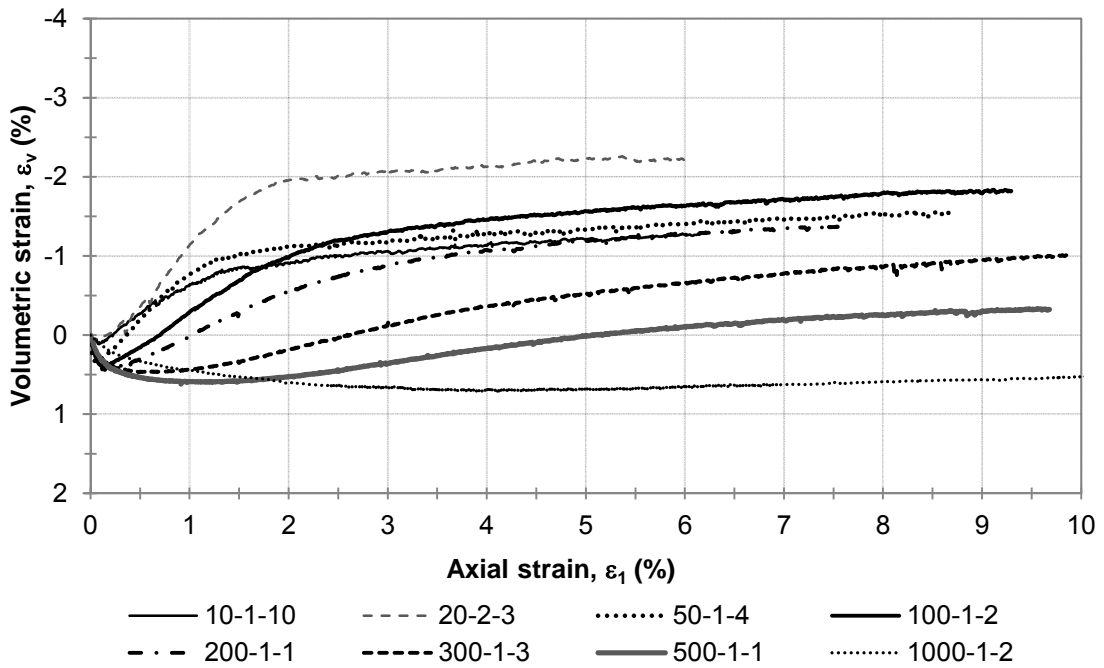


Figure 0.18. Volumetric strain versus axial strain for tests at 55 percent relative density.

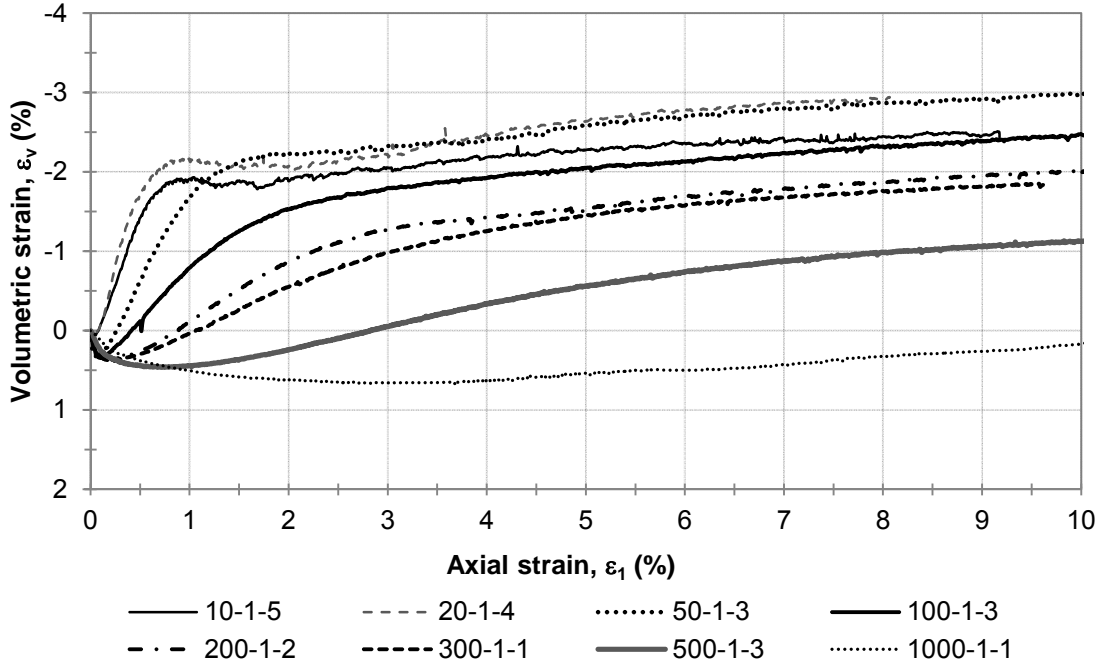


Figure 0.19. Volumetric strain versus axial strain for tests at 65 percent relative density.

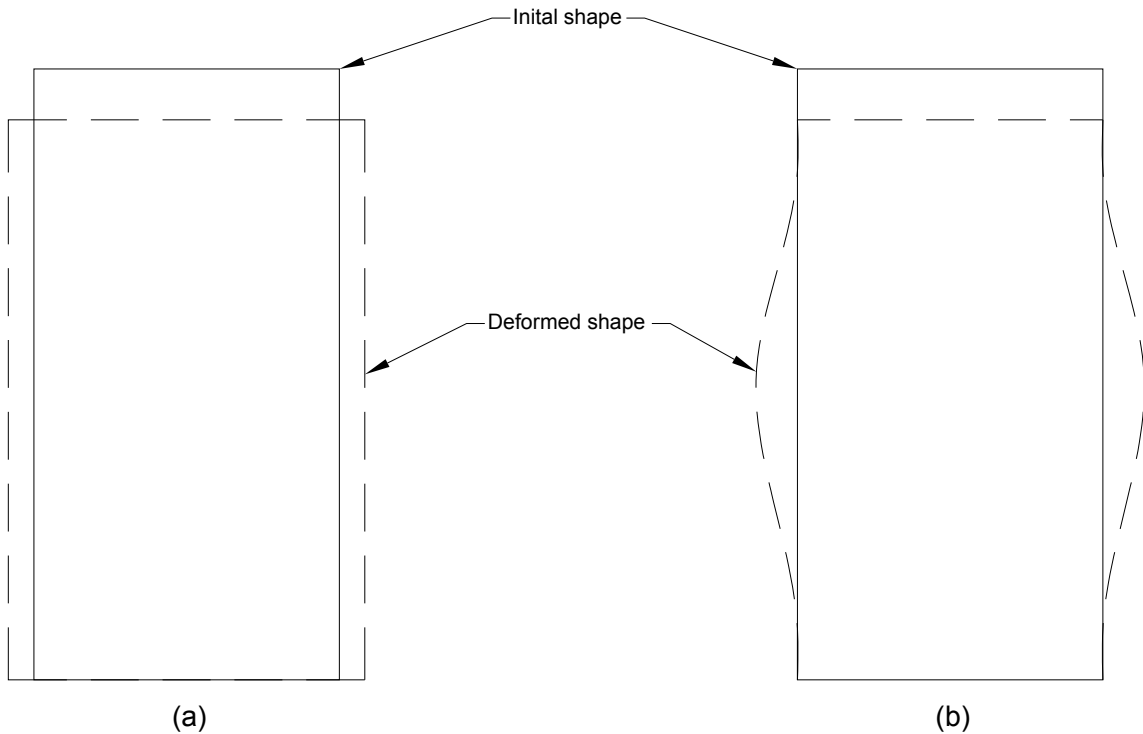


Figure 0.20. Difference between the (a) assumed and (b) observed deformed shape of triaxial specimens.

Friction Angles

Chapter 2 describes the basic behavior of granular soils and some typical strength parameters used to calculate the shear strength for different conditions. For each specimen, the friction angle was calculated twice using:

$$\phi' = \sin^{-1} \left(\frac{\sigma'_1 - \sigma'_3}{\sigma'_1 + \sigma'_3} \right) \quad (3.3)$$

First, the friction angle was calculated using the principal stresses at failure (i.e., at the maximum principal stress ratio), defined as the effective peak friction angle, ϕ'_d , and then using the residual principal stresses to compute what is termed the effective residual friction angle, ϕ'_r .

Figure 0.21 presents the variation in peak friction angle with effective confining pressure and the effective confining pressure normalized to atmospheric pressure ($P_{atm} = 101.3 \text{ kPa} = 14.7 \text{ psi}$). Generally, the peak friction angle decreases linearly with an increase in natural logarithm of effective confining stress as shown by coefficients of determination, R^2 , near unity. Peak friction angles ranged from 54° for the specimen sheared at 10 kPa with a relative density of 65 percent, to 39° for the specimen sheared at 1000 kPa with a relative density of 35 percent.

Similar to the dilation rate, it was observed that the peak friction angle was dependent on relative density. The log-linear rate of decrease of peak friction angle was greater for specimens sheared at higher relative densities. For specimens sheared at 65 percent relative density, a reduction of 7° per log cycle was observed, whereas a 2° reduction per log cycle was observed for samples sheared at 35 percent relative density. The smaller reduction in friction angles for specimens sheared at $D_r = 35$ percent may be due to the significant densification that occurred in specimens consolidated to $\sigma'_3 > 100 \text{ kPa}$. The consolidation-based increase in relative density would tend to result in an increase in friction angle.

Effective residual friction angles were computed for each test and are presented in Figure 0.22. The “residual” principal stresses were taken at 10 percent axial strain, or at the last recorded principal stresses prior to termination of the tests .

Residual friction angles were fairly constant over the range of confining pressures tested. This behavior may indicate that the sheared region of the specimens reached a pseudo-critical state. When considering all 24 tests, the average residual friction angle was 39.5 degrees. However, based on an analysis of variance (ANOVA) F-test, the mean residual friction angle for specimens sheared at 35 percent relative density was found to be significantly different from those sheared at 55 and 65 percent with a 5 percent significance level (p-value = 0.0003).

The average residual friction angles for specimens tested at relative densities of 35, 55, and 65 percent are presented in Table 0.4. Specimens tested at low confining pressures produced considerably more scatter than those tested at higher pressures. This behavior may be due to

small specimen imperfections being amplified by the lack of confinement, as well as the sensitivity to fluctuations in supply pressure as described previously. Two specimens in particular, 10-1-5 65 ($D_r = 65$ percent) and 20-2-3 ($D_r = 55$ percent), were suspected of being outliers with residual friction angles differing by more than 2 standard deviations from their mean.

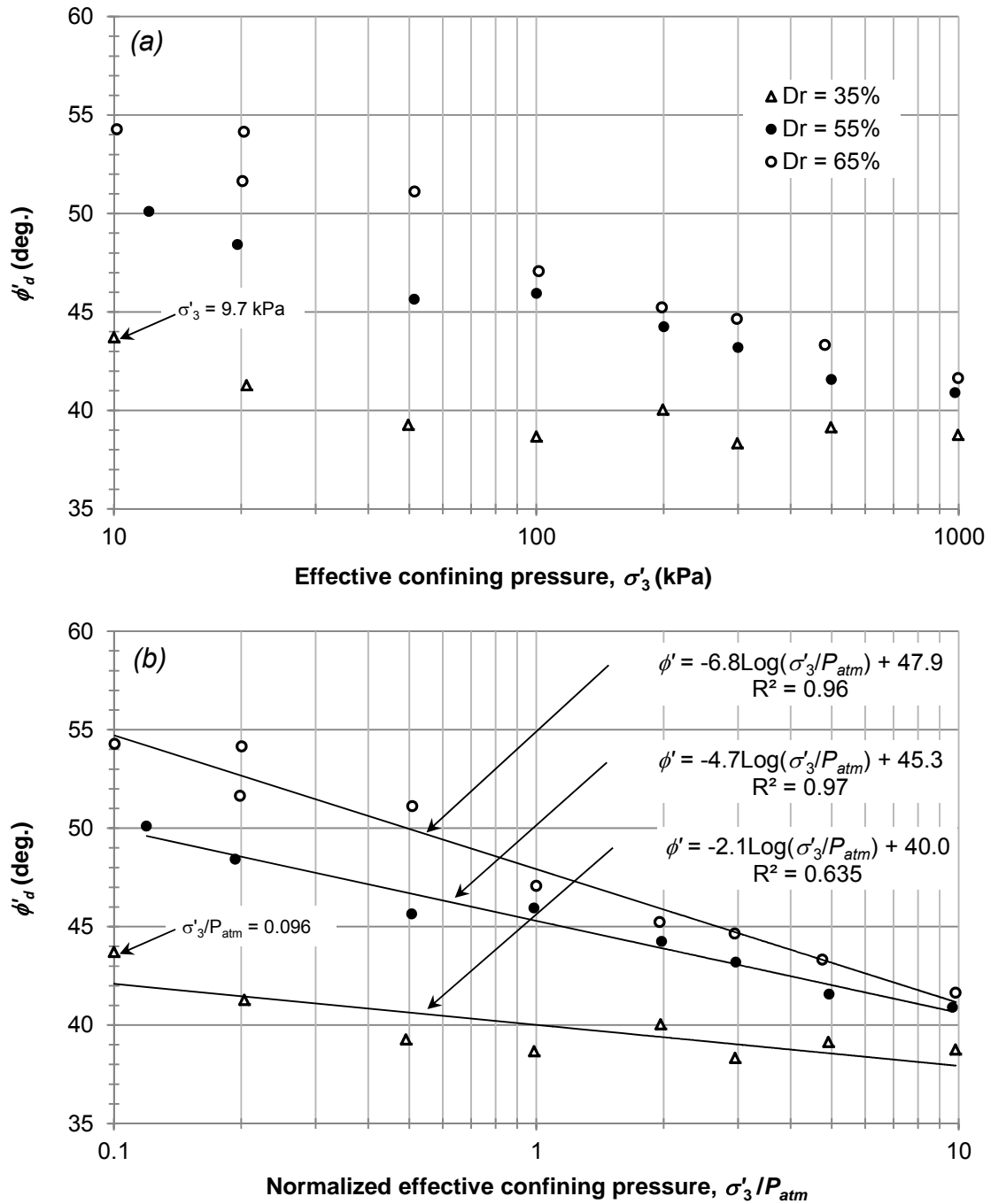


Figure 0.21. Peak friction angles as measured in triaxial compression versus (a) σ'_3 and (b) σ'_3/P_{atm} .

Table 0.4. Average and standard deviation of residual friction angles for specimens sheared at 35, 55, and 65 percent relative densities.

D_r (%)	$\phi'_{r,avg}$ (deg)	Standard Deviation (deg)
35	38.2	0.89
55	39.6	2.06
65	40.4	1.64
All	39.5	1.79

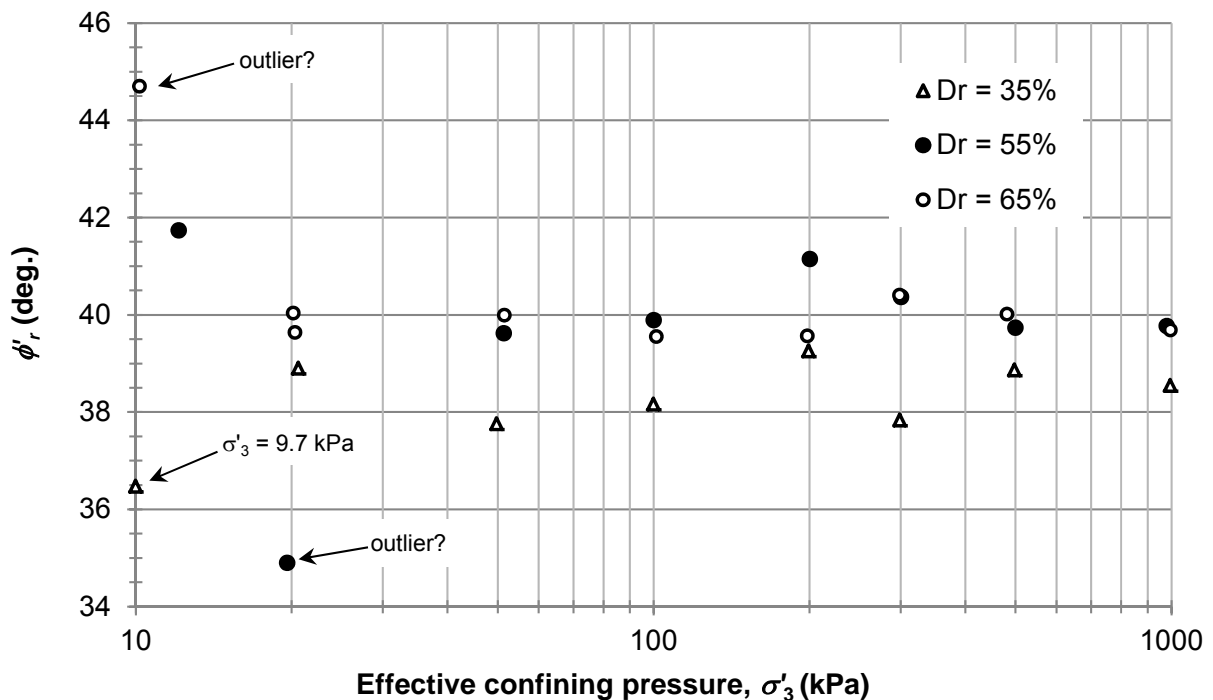


Figure 0.22. Residual friction angles as measured in triaxial compression.

Dilation Angles

The dilation angle is an important constitutive parameter used to model volumetric deformations during shearing as discussed in Chapter 2. Dilation angles were determined by measuring the maximum negative slope (maximum dilation rate) of the volumetric strain, ε_v , versus major principal strain, ε_1 , curves $(d\varepsilon_v/d\varepsilon_1)_{\max}$. This was done by fitting a third order polynomial to the dilative portion of the volumetric strain versus axial strain curve and then determining the maximum calculated incremental change in volumetric strain.

Two definitions of the dilation angle were compared. The friction angle with the contribution of dilation removed, ϕ_f , was first calculated using Equation 2.4 proposed by Rowe (1962). To determine the dilation angle, ϕ_f was subtracted from the peak friction angle. Secondly, the definition of dilation proposed by Tatsuoka (1987) was computed using:

$$\psi_{\max} = \sin^{-1} \left(-\frac{\frac{d\varepsilon_v}{d\varepsilon_1}}{2 - \frac{d\varepsilon_v}{d\varepsilon_1}} \right)_{\max} \quad (3.4)$$

Figure 0.23 presents the dilation angles calculated using the Rowe (1962) approach and the method by Tatsuoka (1987) for all tests with $D_r = 65$ percent; Table 0.5 presents dilation angles for each of the 24 specimens. The ratio of dilation angles calculated using the approach by Rowe (1962) to those computed using the method by Tatsuoka (1987) varies from 0.75 to 0.92 with mean and median of 0.83 and 0.84, respectively.

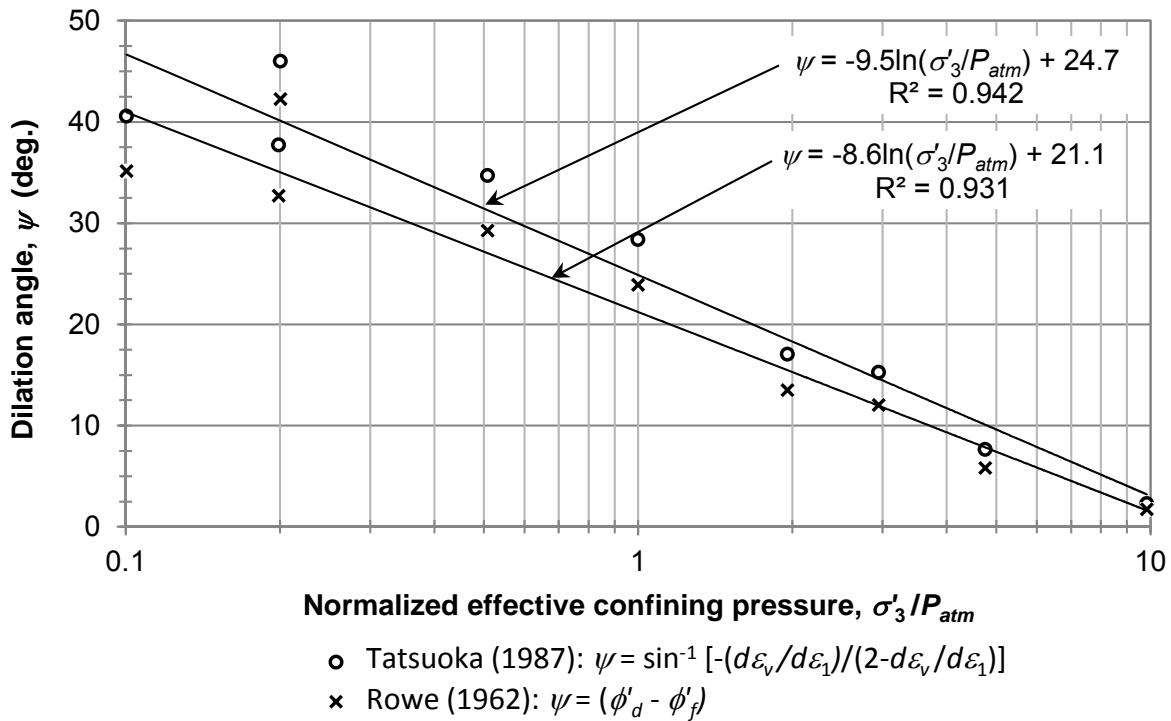


Figure 0.23. Comparison of dilation angles calculated using Tatsuoka (1987) and Rowe (1962) for $D_r = 65$ percent.

Figure 0.24 presents the variation in peak dilation angle following Tatsuoka (1987) with effective confining pressure normalized to atmospheric pressure. Generally, the peak dilation angle decreases linearly with an increase in the natural logarithm of effective confining stress as shown by coefficients of determination, R^2 , near unity. Peak dilation angles ranged from 40° for

the specimen sheared at 10 kPa with a relative density of 65 percent, to -0.2° for the specimen sheared at 1000 kPa with a relative density of 35 percent.

Similar to the peak friction angle, it was observed that the peak dilation angle was dependent on relative density and confining stress. The log-linear rate of decrease of peak dilation angle was greater for specimens sheared at higher relative densities. For specimens sheared at 65 percent relative density, a reduction of 9 degrees per log cycle was observed, whereas only a 6 degrees reduction was observed for samples sheared at 35 percent relative density.

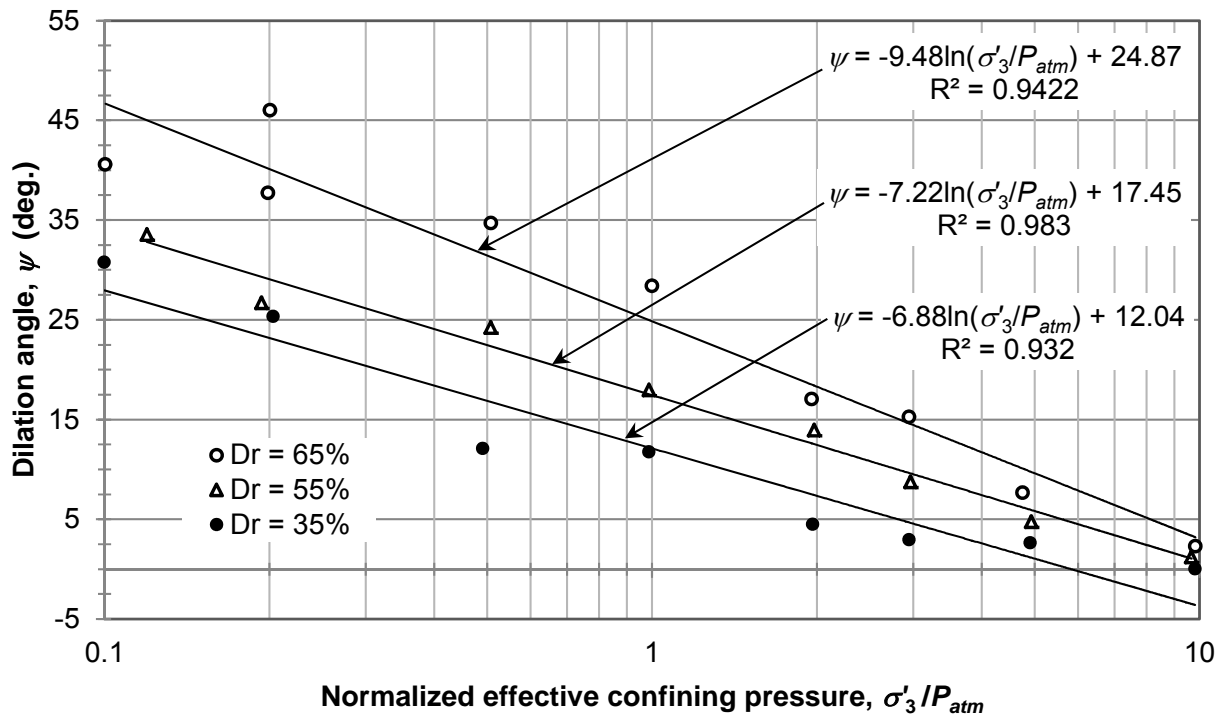


Figure 0.24. Dilation angles calculated using Tatsuoka (1987) versus normalized confining pressure.

To better visualize the combined effect of relative density and confining pressure on the volumetric response of the backfill soil, the three variables were plotted together to create what is typically referred to as a Peacock diagram (Holtz and Kovacs 1981). In Figure 0.25, a three dimensional surface presented as contour lines was created by plotting the relative density after consolidation on the y-axis, the normalized confining pressure on the x-axis, and the volumetric strains at failure on the z-axis. Because the relative density after consolidation is referenced in Figure 0.25, this diagram may best represent the global shearing response of the granular material investigated from a fundamental standpoint. In general, the most positive volumetric strains, indicating contractive behavior at failure, occur at high effective confining pressures and low relative densities. The most negative volumetric strains, indicating dilative behavior at failure, occur at lower confining pressures and higher relative densities. Additionally, the contour lines become more closely spaced at lower confining pressure indicating that the effect of

dilation increases nonlinearly as the confining pressure decreases. This behavior reinforces the trends observed with the dilation angles in Figure 0.24. Using the Peacock diagram, the volumetric strain at failure may be easily estimated for any combination of effective confining pressure and post-consolidation relative density.

Table 0.5. Summary table of dilation angles calculated using Tatsuoka (1987) and Rowe (1962).

Test ID (σ'_3 - OCR - Test #)	D_r BC ^a	σ'_3/P_{atm}	ψ (Tatsuoka 1987) (deg.)	ψ (Rowe 1962) (deg.)
10-1-9	0.369	0.10	30.8	27.0
20-1-5	0.346	0.20	25.3	23.1
50-1-6	0.340	0.49	12.1	10.1
100-1-5	0.347	0.99	11.7	9.8
200-1-6	0.353	1.97	4.5	3.5
300-1-4	0.347	2.95	2.9	2.3
500-1-4	0.350	4.91	2.6	2.1
1000-1-3	0.354	9.82	-0.3	-0.2
10-1-11	0.581 ^b	0.12	33.5	28.2
20-2-3	0.554	0.19	26.7	22.1
50-1-4	0.537	0.51	24.2	20.3
100-1-2	0.566	0.99	18.0	14.2
200-1-1	0.535	1.98	14.0	11.0
300-1-3	0.548	2.97	8.7	6.8
500-1-1	0.559	4.93	4.8	3.7
1000-1-2	0.553	9.67	1.2	1.0
10-1-5	0.633	0.10	40.6	35.1
20-1-3	0.634	0.20	37.7	32.7
20-1-4	0.663	0.20	46.0	42.2
50-1-3	0.669	0.51	34.7	29.3
100-1-3	0.662	1.00	28.4	23.9
200-1-2	0.639	1.96	17.0	13.5
300-1-1	0.655	2.95	15.3	12.0
500-1-3	0.668	4.76	7.6	5.8
1000-1-1	0.660	9.84	2.3	1.7

^a Before Consolidation (BC), ^b Test 10 - 1 - 11 did not meet density criteria by 0.6 percent.

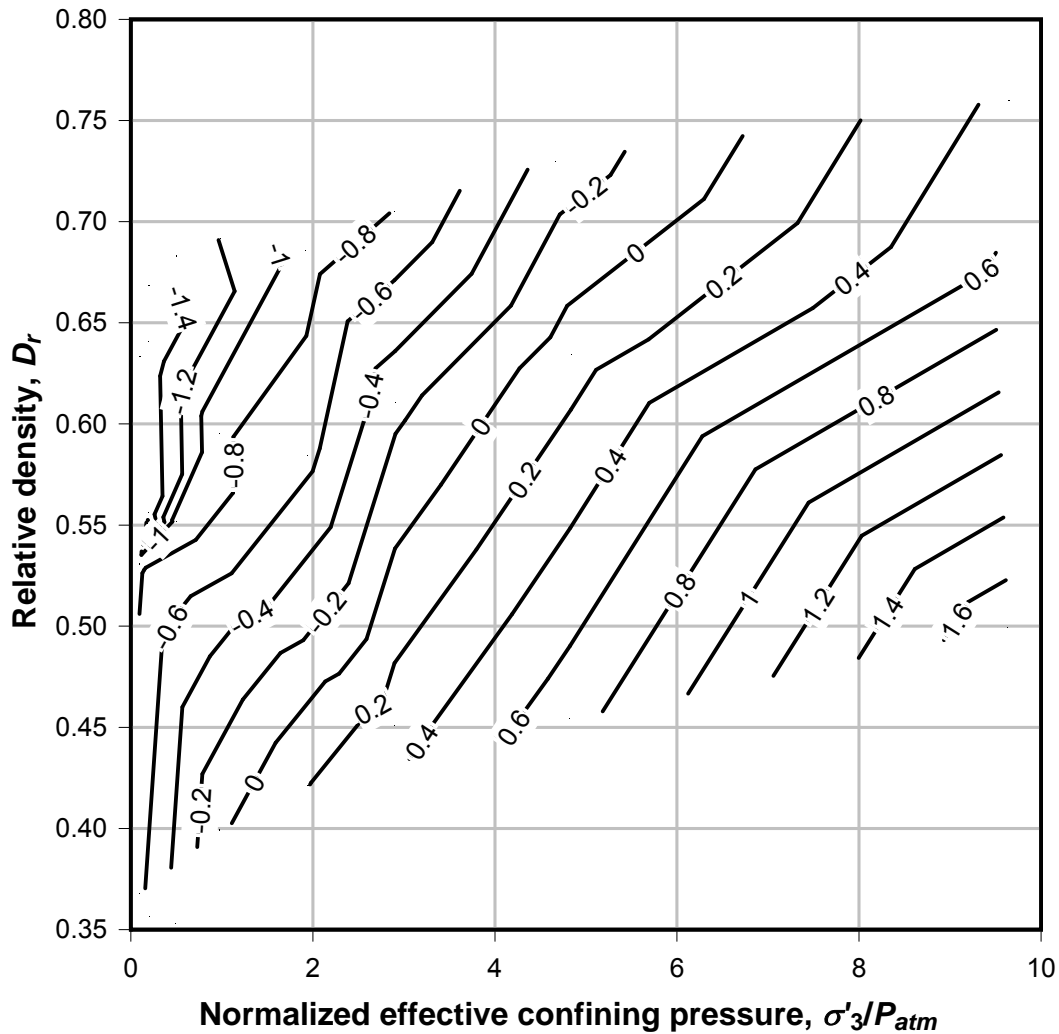


Figure 0.25 Peacock diagram showing the relationship between volumetric strain at failure (shown by the contour lines), relative density and confining pressure.

4.3 Summary

In this chapter the material characteristics for the ribbed steel strip reinforcements and reinforced backfill materials were presented along with a discussion on the testing procedures used to measure the material properties. Tension testing was performed on coupons of the reinforcement material to provide the yield strength and modulus of elasticity, which were determined equal to 700 MPa and 208 GPa, respectively. Axial symmetric triaxial compression tests were performed on 15 cm diameter specimens of the reinforced backfill material compacted to relative densities of 35, 55, and 65 percent and tested at effective confining pressures ranging from 10 to 1,000 kPa. Peak friction angles ranging from 54 to 39 degrees were measured. The dilation angles were calculated using the methods by Rowe (1962), and Tatsuoka (1987) and resulted in dilation angles ranging from 45 to -0.3 degrees with ratios of dilation angles calculated using the method by Rowe (1962) to those computed using the method by Tatsuoka (1987) varied from 0.75 to 0.9.

The stress-strain and volumetric behavior of the reinforced backfill material was compared to the behavior of two soils reported by Verdugo and Haz (2007) and found to behave relatively similar. Finally, the Duncan and Chang (1970) hyperbolic model was calibrated to the stress-strain and volumetric behavior recorded during triaxial testing. Modulus numbers ranging from 726 to 1200, and modulus exponents ranged from 0.385 to 0.525 were observed. The appropriateness of the Duncan and Chang (1970) stress strain model was visually analyzed and determined acceptable at working stress levels from approximately one to four percent depending on the effective confining stress. In the following chapter the interaction between the reinforced backfill material and the ribbed steel strip reinforcements is discussed and data from laboratory pullout tests are presented.

Chapter 5 Single-Strip Soil-Reinforcement Interaction

In-situ and laboratory pullout tests are usually performed to measure the apparent friction coefficient, f^* , between a MSE wall reinforcement strip and the surrounding backfill (as discussed in Section 2.2.1.3). In current MSE wall design, conservative design models for f^* are typically used rather than performing pullout tests on specific backfill-reinforcement combinations as this can make up for poor field quality control on compaction. The f^* design models were calibrated with a wide variety of soils including fine sands with friction angles as low as 35 degrees and therefore under-predict the apparent friction coefficient when high strength sand-gravel mixtures are used as backfill.

Since a major focus of this study is to measure the interaction between a specific backfill and specific reinforcement, it was deemed necessary to perform pullout tests and produce specific f^* design models for the backfill under consideration. This chapter discusses the design of a laboratory pullout test apparatus, the methods used in conducting the pullout tests, and the observed interactions between the backfill soil and the ribbed reinforcement. Additionally, two new f^* design models will be presented: one calibrated to the specific backfill being tested, and a second calibrated using all available data from pullout tests on gravels and sand-gravel mixtures.

5.1 Pullout Test Apparatus for Single Reinforcement Strips

A pullout test apparatus was designed to measure the load-displacement behavior of single reinforcement strips. This system consisted of a pullout box, air bladder, hydraulic actuator, reaction frame, two load cells, string potentiometer, and data acquisition system (DAQ).

The pullout test box was designed to accommodate a 2 m long segment of reinforcement strip within a compacted rectangular prism of backfill soil with dimensions of 0.356 m (14 in.) in height, 0.456 m (17.75 in.) in width, and 2.16 m (84 in.) in length. Refer to Appendix C for a full set of pullout box design schematics. The pullout box was designed to house an air bladder capable of applying 300 kPa (gauge) uniform pressure to the top of the backfill. The structure of the box consists of MC 4 x 13.8 channel sections, 9.5 mm (3/8 in.) thick steel plates and 12.7 mm (1/2 in.) A354 ($f_u = 150$ ksi) threaded rods configured as shown in Figure 0.1 and Figure 0.2. Design calculations were performed following AISC (2010).

The pullout box was secured to a reaction beam along with a 2200 kN (500 kips) capacity hydraulic actuator as shown in Figure 0.3. The actuator was connected to the pullout strip with a 100 kN (22.5 kip) load cell (Figure 0.4), except when the required pullout load was predicted to exceed the capacity of the load cell. For the tests where the 90 kN load cell was removed, 4 strain gauges attached to the reinforcement strip (2 on each side), were used to measure the load in the reinforcement as shown in Figure 0.5. The loads measured with the strain gauges agreed with the load cells that accompany the actuator.

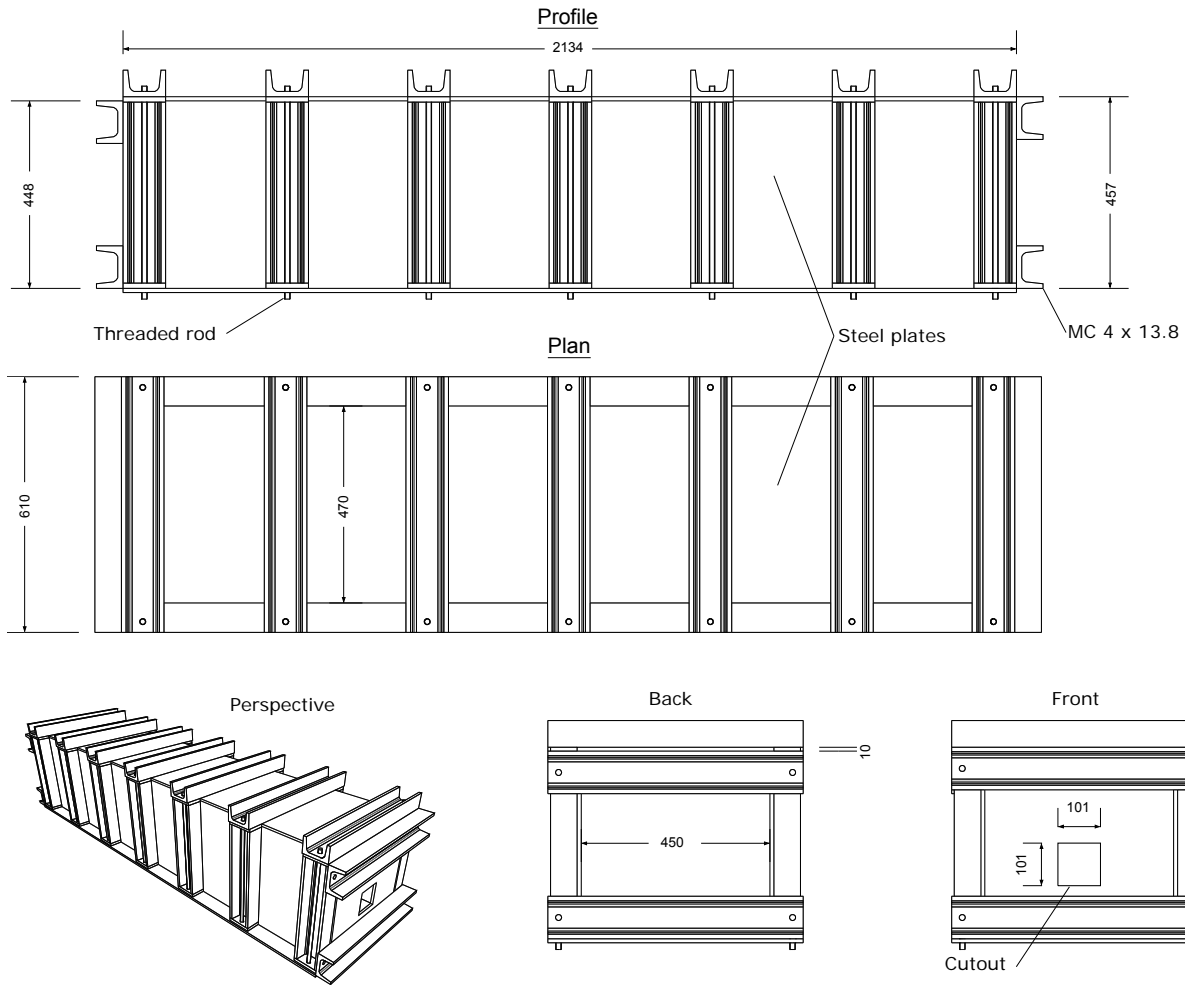


Figure 0.1 Schematic of the pullout box with dimensions in mm.



Figure 0.2 Pullout box before loading.

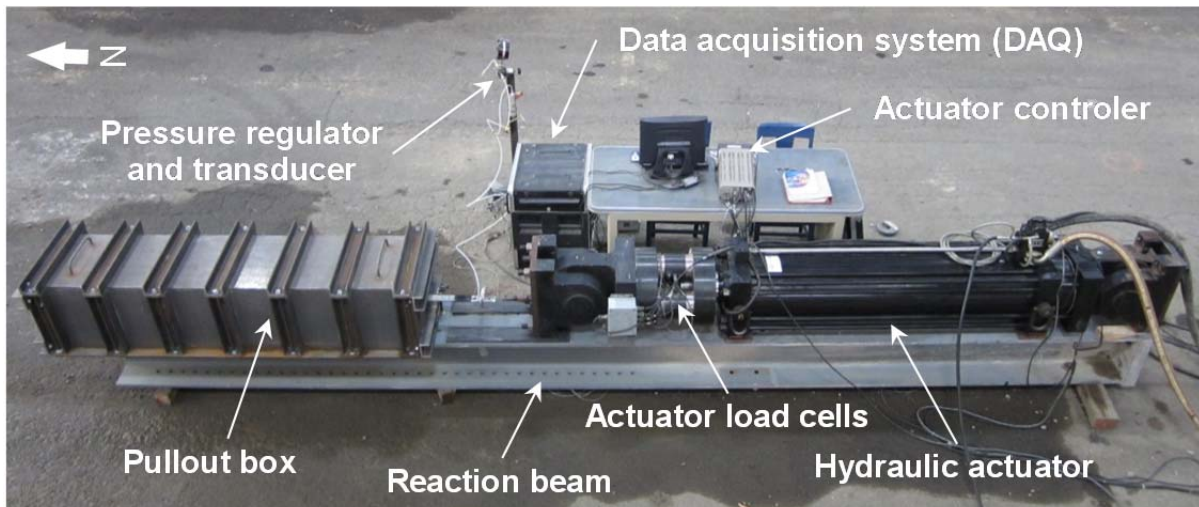


Figure 0.3 Reinforcement pullout testing system.

The air bladder used consisted of a common queen size inflatable mattress with a modified valve capable of being attached to a pneumatic regulator. The integrity of the air bladder was checked periodically throughout each test by momentarily shutting off the air supply while still being able to measure the pressure inside the bladder.

All tests were performed in a displacement controlled manner with a displacement rate of 1 mm (0.04 in) per minute. The displacement of the reinforcement strip was independently measured using a string potentiometer (M/N: LX-PA-15-P10K; S/N: 38120817), shown in Figure 0.5 secured to the reaction beam.

5.2 Test Program

The two main motivations for performing these single strip pullout tests were:

- to expand the current pullout test database for high strength reinforced gravel backfill; and,
- to produce baseline results for comparison with subsequent multi-strip pullout tests (not included in this thesis).

Currently, the number of well documented pullout tests for high strength sand-gravel mixtures using ribbed steel strip reinforcements is small. By combining the results from the present study with those reported in the literature, new f^* design models for use with gravels and sand-gravel mixtures were able to be developed.

The present study is a part of a larger group of research projects aimed at analyzing the behavior of tall MSE walls. Due to the large stresses developed in tall MSE walls, small reinforcement spacing, both vertical and horizontal, is required to develop sufficient resistance. However, the effect of reinforcement spacing on the backfill-reinforcement interaction is not well understood. Research is currently being carried out that employs a larger pullout apparatus used to perform tests with multiple reinforcement strips at varying horizontal and vertical

spacing. In subsequent research, the results from the present study will be used as a baseline in the analysis of these multi-strip pullout test results.



Figure 0.4 Load cell used for the majority testing (looking east).

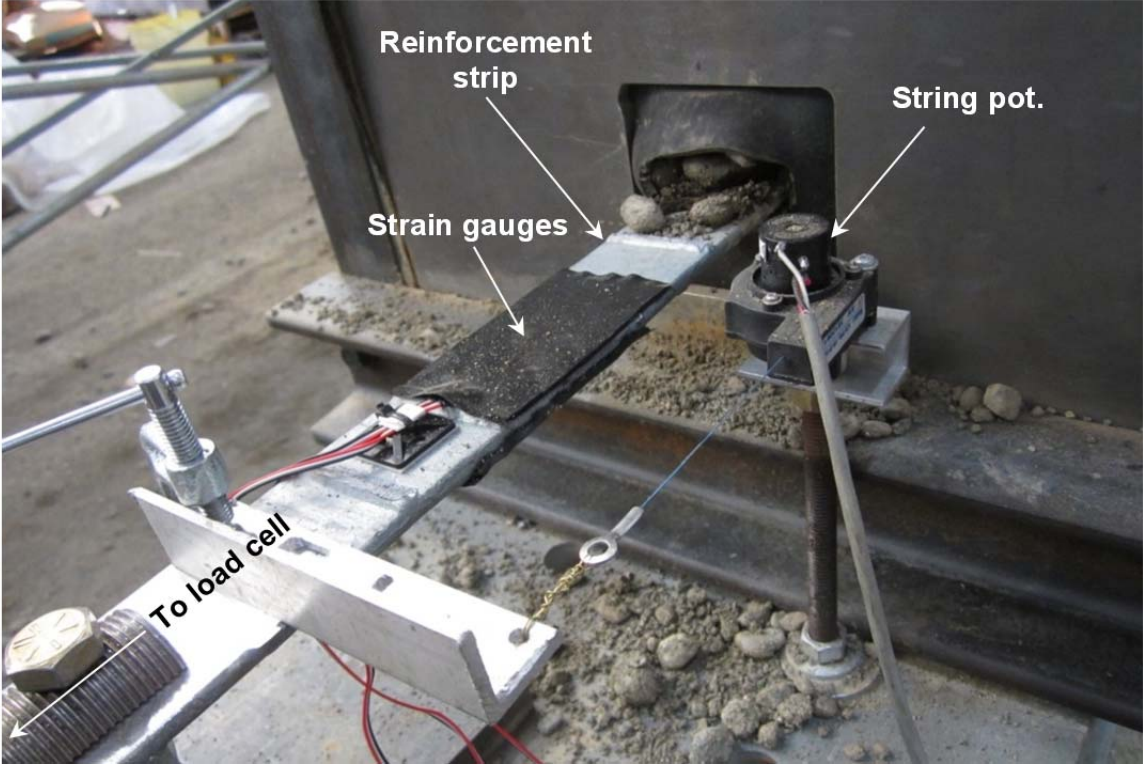


Figure 0.5 Strain gauges and string potentiometer (looking northwest).

The test program for the present study included 20 pullout tests at vertical effective stresses ranging from 10 to 300 kPa. All tests had the same target relative density of 65 percent corresponding to a relative compaction of 95 percent and a dry unit weight of 21.3 kN/m³ (135.6 pcf). Each of the twenty pullout test specimens were given a unique designation consisting of the vertical effective stress the specimen was subjected to, and a subset chronologic indication number. For example, the fourth test performed at a vertical effective stress of 100 kPa was given the designation of 100-4. Table 0.1 provides the designations of each test considered in the present study.

Table 0.1. Pullout tests considered in the present study.

Single strip pullout test designations	
10-1	100-5
20-1	100-6
50-1	150-1
50-2	150-2
50-3	150-3
75-1	150-4
100-1	200-1
100-2	200-2
100-3	250-1
100-4	300-1

5.3 Compaction of Reinforced Backfill

In contrast to the CD triaxial test specimens discussed in Chapter 4, all pullout test specimens were targeted to achieve the same relative compaction of 95 percent of maximum dry density (ASTM D-1557) (i.e, 65 percent relative density). Specimen compaction was carried out using the vibratory plate compactor shown in Figure 0.6a (Bomag M/N: BVP 18/45) with a mass of 90 kg (6.17 slugs), a vibratory frequency of 90 Hz, a manufacture rated dynamic force of 18 kN (4050 lb), and a total force of 18.9 kN (4140 lb). Each lift had a target compacted thickness of 9 cm (3.5 in.). The soil for each lift was weighed and brought to optimum water content (6.4 percent) before being placed in the box and compacted. The same compactive effort was used for each test. The compactor was passed over the top of the soil five times starting from the south end of the box and ending at the north. Then the compactor was rotated 180 degrees and passed over the soil five more times in the opposite direction. After the second lift was compacted, the surface of the backfill was inspected to make sure it was level, then the reinforcement strip was placed and the final two lifts were compacted.

Compaction verification was completed with a Nuclear Density Gauge (NDG) on the first test specimen. The initial NDG testing was performed by FEI (2012a) by taking 3 measurements: one approximately 0.3 m (1 ft.) from the north end of the box, one at approximately the center and one approximately 0.3 m from the south end of the box. The average relative compaction was found to be 94 percent which corresponded to a dry unit weight of 21.03 kN/m³ (133.9 pcf). After reviewing the initial NDG test results it was decided that no further density verification

was needed as long as the same compaction procedure was strictly followed for the remaining tests. However, after observing relatively high variability in the peak pullout load between tests with the same vertical effective stress, as described subsequently, it was suspected that variations in density were possible

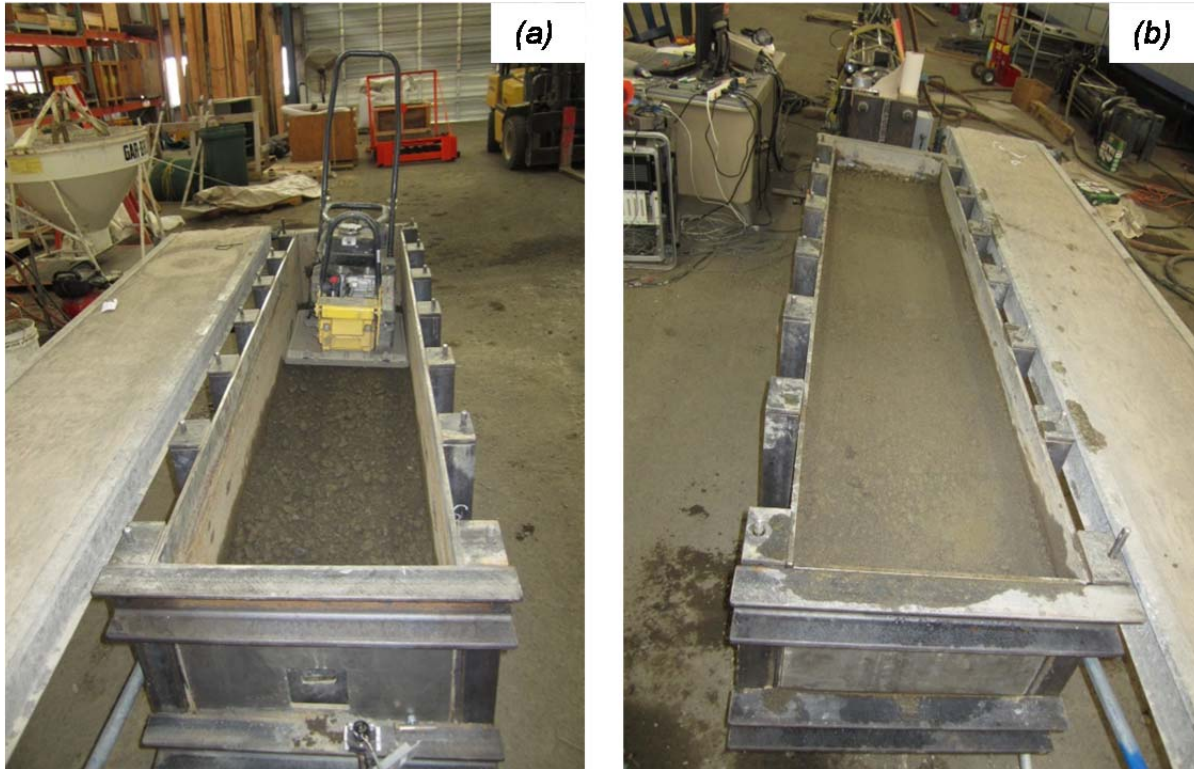


Figure 0.6 Compaction of backfill soil inside pullout box (a) before compaction of second lift (looking north), and (b) after compaction of fourth (final) lift (looking south).

In order to frequently measure the dry unit weight of each specimen, a lower cost alternative to the NDG was required. Due to the availability of testing equipment, the Balloon Density Apparatus (BDA) shown in Figure 0.7 was selected. The dry unit weights of two more specimens were measured with both the NDG (FEI 2012b; c) and the BDA. In order to test the same soil mass with each device, the balloon density tests were conducted between where the source rod of NDG was driven and where the photon sensor was located as shown in Figure 0.8.

On average, the dry unit weight was 0.55 kN/m^3 (3.5 pcf) higher when measured with the BDA. Since the NDG test is the standard test used in industry, the average difference (0.55 kN/m^3) was subtracted from the dry unit weights of subsequent test specimens measured with the BDA. Figure 0.9 shows the variation in the measured relative compaction within and between specimens. The maximum and minimum recorded value was 103 and 88 percent, respectively, with a global median and global mean was 95.5 and 96.1 percent, respectively. The consequences of these variations are discussed in Section 5.4.



Figure 0.7 Balloon density apparatus (BDA).

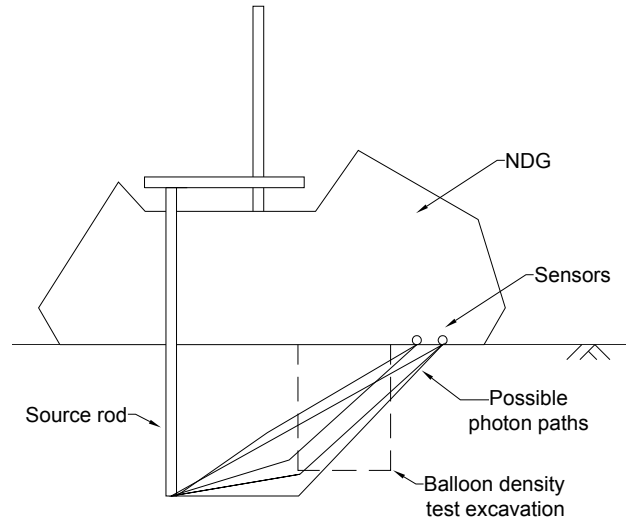


Figure 0.8 Location of balloon density excavation relative to the nuclear density gauge (modified from Holtz et al. (2011) and Troxler Electronic Laboratories Inc.).

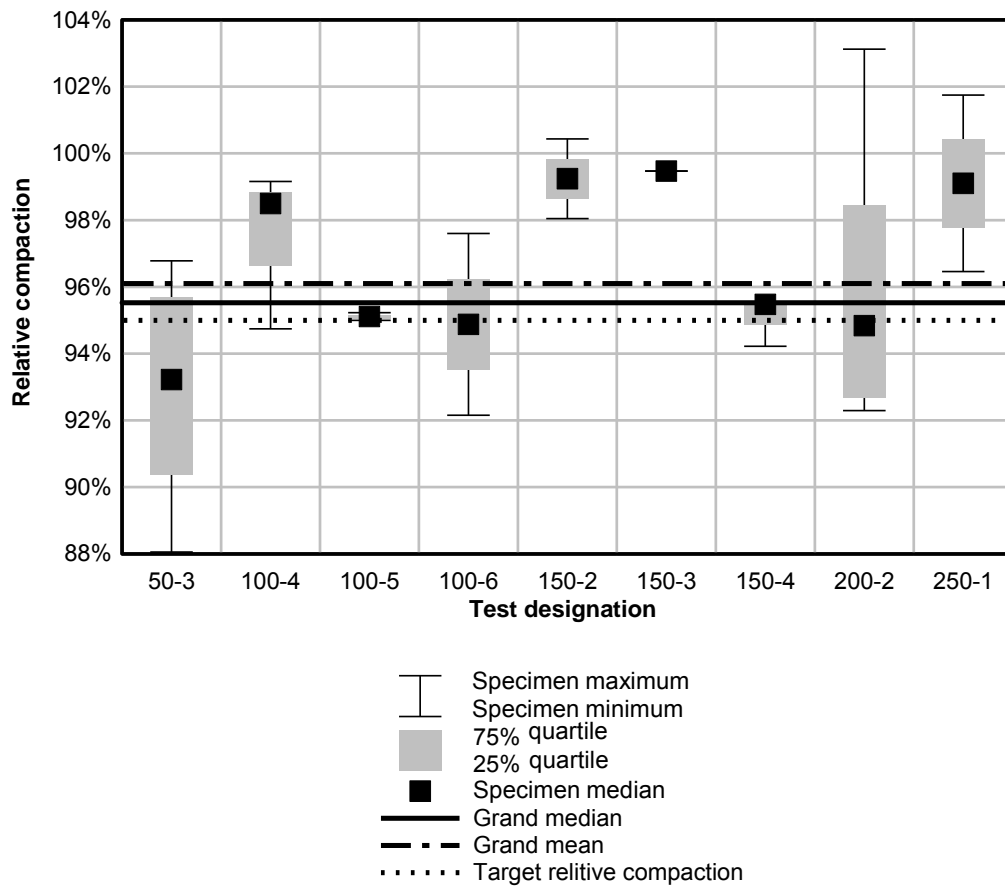


Figure 0.9 Variation in corrected relative compaction for pullout test specimens measured with the BDA.

5.4 Single Reinforcement Strip Pullout Resistance

As mentioned previously, each pullout test was conducted at a specific vertical effective stress ranging from 10 kPa to 300 kPa. The vertical effective stress was the single controlled variable for the pullout test program.

5.4.1 Interpretation of Pullout Tests

In general, the load-displacement behavior was categorized into three different parts: initial non-slip resistance, monotonic accumulation of resistance, and a transition to the development of reinforcement slip with a reduction in resistance (Figure 0.10). Initially, load accumulated with little to no displacement. The load at which movement of the strip relative to the soil was initiated was referred to as the initial non-slip resistance and is shown in Figure 0.10(b). The initial non-slip resistance develops from the passive rib resistance and the static friction between the reinforcement and backfill. Although, a correlation of the non-slip resistance with effective vertical stress was expected this was not observed as discussed subsequently in Section 5.4.2.

As the reinforcement strips began to displace under the applied loading, the resistance increased monotonically until slippage between the reinforcement and the surrounding soil occurred. The initial of the reinforcement strips evaluated in this study slippage typically occurred between 8 and 16 mm of displacement.

After the initial slippage occurred, the pullout resistance generally continued to increase with small sporadic reductions as slipping continued. Larger cycles of reductions followed by gradual increases in resistance were also observed in many of the tests. These resistance cycles were attributed to larger particles being overridden or crushed by the reinforcement ribs resulting in losses of passive resistance and dilation-induced confining pressures. Figure 0.11 shows a broken gravel particle found near a reinforcement rib subsequent to testing. Pullout resistance typically decreased noticeably with continued local variations at approximately 60 mm of displacement.

5.4.2 Pullout Test Results

Table 0.2 summarizes the results of each pullout test conducted. Some data, such as unit weight, initial non-slip resistance, measured initial stiffness, and maximum pullout resistance for certain specimens were not available. For example, a faulty potentiometer was used to measure displacement for the first seven tests conducted. The initial potentiometer was not a string potentiometer as shown in Figure 0.5, it was a spring-loaded plunger type potentiometer. For these seven tests, the plunger was initially contracted at the beginning of each test, then, as displacement occurred, the plunger would begin to release. However, the faulty potentiometer did not produce a change in resistance (and thus voltage) for the first 5 mm of travel. This non-responsive range was only encountered for four of the seven tests including tests 10-1, 50-1, 75-1, and 200-1. Neither the initial non-slip resistance nor the initial stiffness was measured for those tests. Additionally, the air bladder used for the test 300-1 lost pressure prior to achieving peak resistance at approximately 5 mm of displacement. Consequently, neither the peak resistance nor the apparent friction coefficient could be reported for that test.

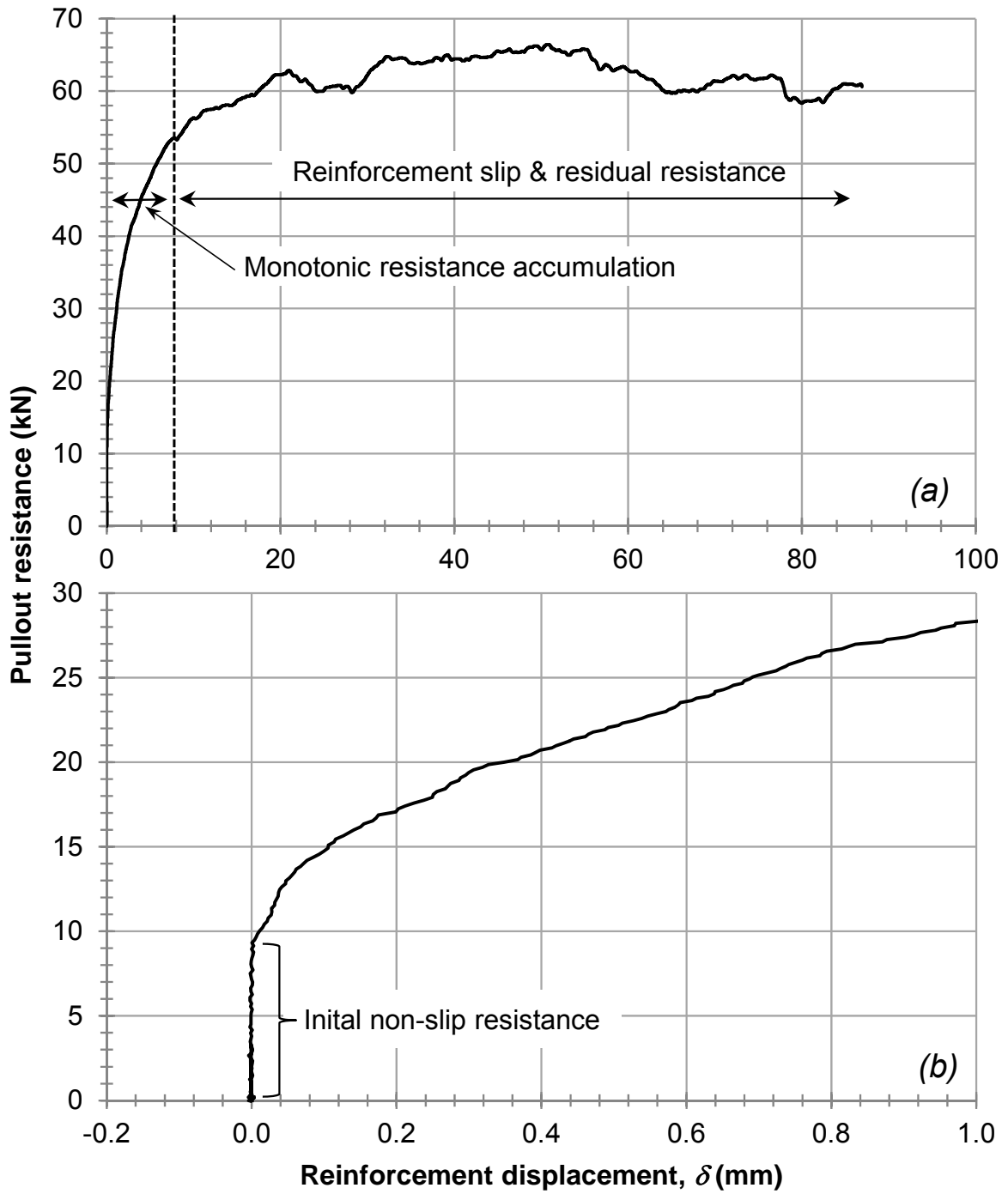


Figure 0.10 Pullout resistance vs. reinforcement strip displacement curve for test 200-2 showing (a) the full range in displacement, and (b) an expanded view of the initial load-displacement and non-slip resistance.



Figure 0.11 Photo of a gravel particle crushed by a reinforcement rib.

Figure 0.12 shows the load-displacement measurements for specimens tested at $\sigma'_v = 10$ and 20 kPa. The 10 kPa increase in σ'_v produced an increase of 15 kN in the peak reinforcement pullout resistance. The load-displacement curves for tests conducted at $\sigma'_v = 50$ and 75 kPa are shown in Figure 0.13; it should be noted that the vertical effective stress for specimen 50-1 was 57.2 rather than its intended target value of 50 kPa and therefore should not be directly compared to tests 50-2 and 50-3 based on effective vertical stress. Pullout test specimen 50-3 ($\gamma_d = 21.3$ kN/m³) produced the smallest maximum pullout resistance of 38.5 kN, specimen 50-2 initially exhibited significantly stiffer behavior than observed for specimen 50-3 suggests a denser specimen. However, unit weight was not measured for specimens 50-1, 50-2, nor 75-1. Figure 0.14 shows the load displacement measurements for tests performed at $\sigma'_v = 100$ kPa. Peak pullout resistance values ranging from 54.6 kN for specimen 100-6 ($\gamma_d = 21.3$ kN/m³) to 80.3 kN for specimen 100-4 ($\gamma_d = 22.4$ kN/m³) were observed. The peak reinforcement load for specimen 100-5 was 9.3 kN less than that observed for specimen 100-4 despite having the same measured unit weight of 22.4 kN/m³. The measured pullout resistances and reinforcement displacements for specimens tested at $\sigma'_v = 150$ kPa are shown in Figure 0.15. Specimen 150-4 behaves much stiffer than specimens 150-3 and 150-2 despite having a lesser unit weight. Figure 0.16 presents the load-displacement measurements for specimens tested at $\sigma'_v = 200, 250,$ and 300 kPa. The peak reinforcement load measured for Specimen 200-1 was 12.2 kN larger than that measured for specimen 250-1 ($\gamma_d = 22.7$ kN/m³), and 28 kN larger than that measured for 200-2 ($\gamma_d = 21.3$ kN/m³) suggesting that over-densification may have occurred during the compaction of specimen 200-1.

The initial non-slip resistance varied between 0.9 and 14.7 kN as shown in Figure 0.17. From basic knowledge of frictional behavior, it was speculated that the initial non-slip resistance would correlate with σ'_v . Using the Spearman-Rank test with the initial non-slip resistance effective confining pressure data, the null hypothesis of non-correlation was rejected at a significance level of five percent (p-value = 0.021) suggesting the presence of a correlation. However, the significant variability of the relationship between initial non-slip resistance and σ'_v can be seen in Figure 0.17. The cause of this variability was suspected to be local density variations within the soil specimens.

In order to exclude the effect of the initial non-slip resistance, the initial stiffness was determined by measuring the rate of load accumulation between 0.25 mm and 1 mm of displacement. Values of initial stiffness ranged from 4.9 to 23.0 kN/mm.

Generally, peak reinforcement load, P_{max} , increased with increases in σ'_v (Figure 0.18). However, variations in P_{max} of up to 26 kN were observed between tests with equal effective vertical stresses.

Table 0.2 Summary of pullout test results.

Test Designation	γ_d (kN/m ³)	RC (%)	σ'_v (kPa)	Initial Non-Slip Resistance (kN)	Meas. Initial Stiffness, (0.25 – 1.0 mm) (kN/mm)	P_{max} (kN)	f^* (-)
10-1	N/A	N/A	10	N/A	N/A	18.6	9.92
20-1	21	94	23.7	4.6	10.7	33.5	7.56
50-1	N/A	N/A	57.2	N/A	N/A	53.1	4.96
50-2	N/A	N/A	50	5.7	12.4	45.7	4.89
50-3	N/A	N/A	50	0.9	12.5	38.5	4.11
75-1	N/A	N/A	75	N/A	N/A	57.3	4.09
100-1	N/A	N/A	100	11.2	12.1	71.2	3.81
100-2	N/A	N/A	100	7.1	12.4	63.3	3.38
100-3	N/A	N/A	100	7.5	13.4	65.1	3.48
100-4	21.85	98	100	6.0	18.0	80.3	4.29
100-5	21.35	95	100	7.4	14.4	71.0	3.80
100-6	20.75	93	100	5.0	16.8	54.6	2.92
150-1	N/A	N/A	150	5.8	17.2	80.1	2.86
150-2	22.25	99	150	7.6	7.5	66.4	2.37
150-3	22.25	99	150	4.1	4.9	61.9	2.21
150-4	21.25	95	150	9.2	15.6	69.4	2.47
200-1	N/A	N/A	200	N/A	N/A	92.8	2.48
200-2	20.75	93	200	14.7	13.7	64.8	1.73
250-1	22.15	99	250	8.2	9.7	80.6	1.72
300-1	N/A	N/A	300	7.5	23.0	N/A	N/A

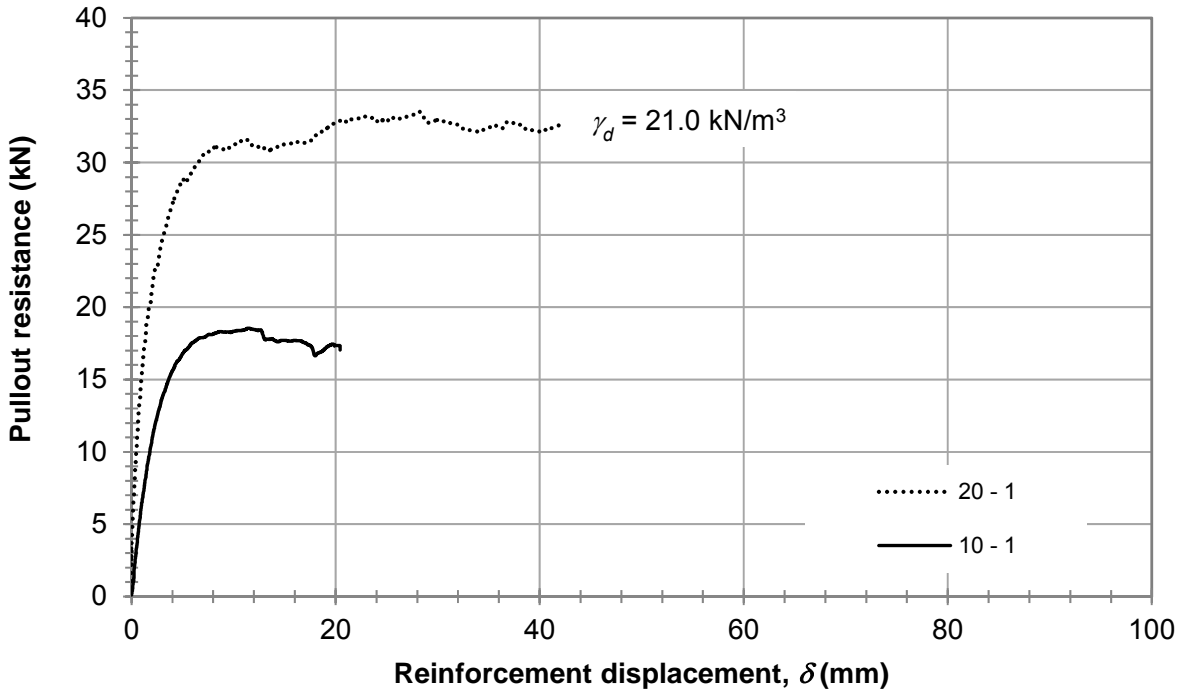


Figure 0.12. Pullout resistance versus reinforcement displacement for tests with vertical effective stresses of 10 and 20 kPa. Note, the unit weight of 10-1 was not measured.

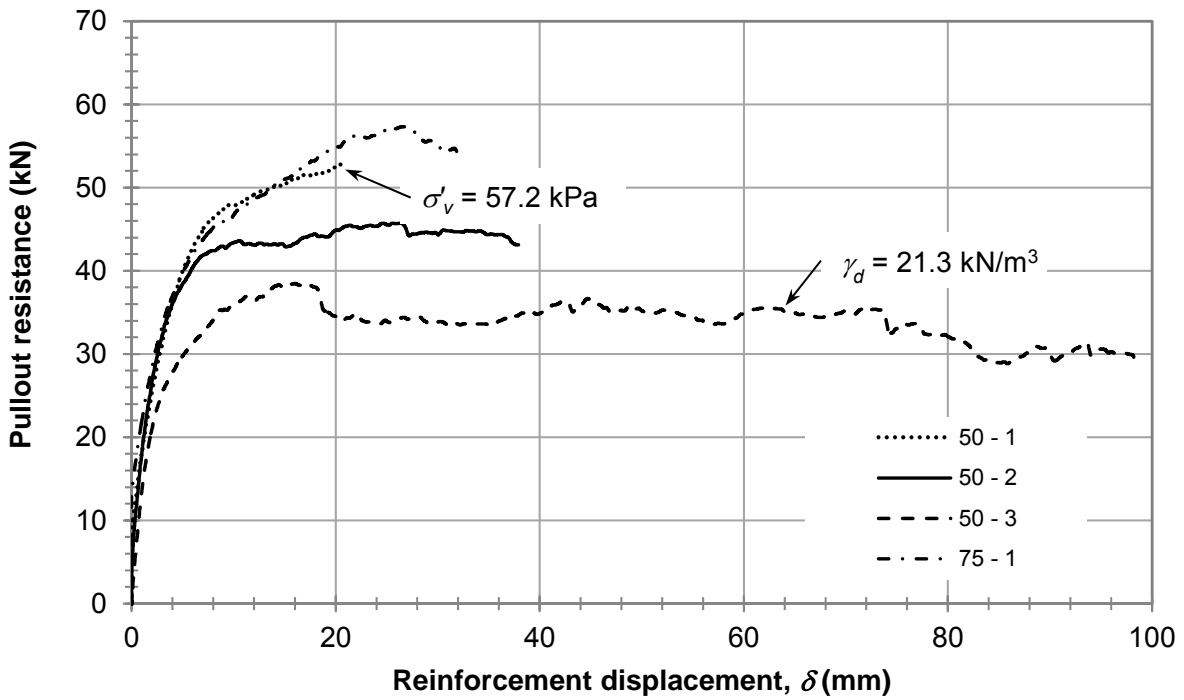


Figure 0.13. Pullout resistance versus reinforcement displacement for tests with vertical effective stresses of 50 and 75 kPa. Note, the unit weight of 50-1, 50-2, and 75-1 was not measured.

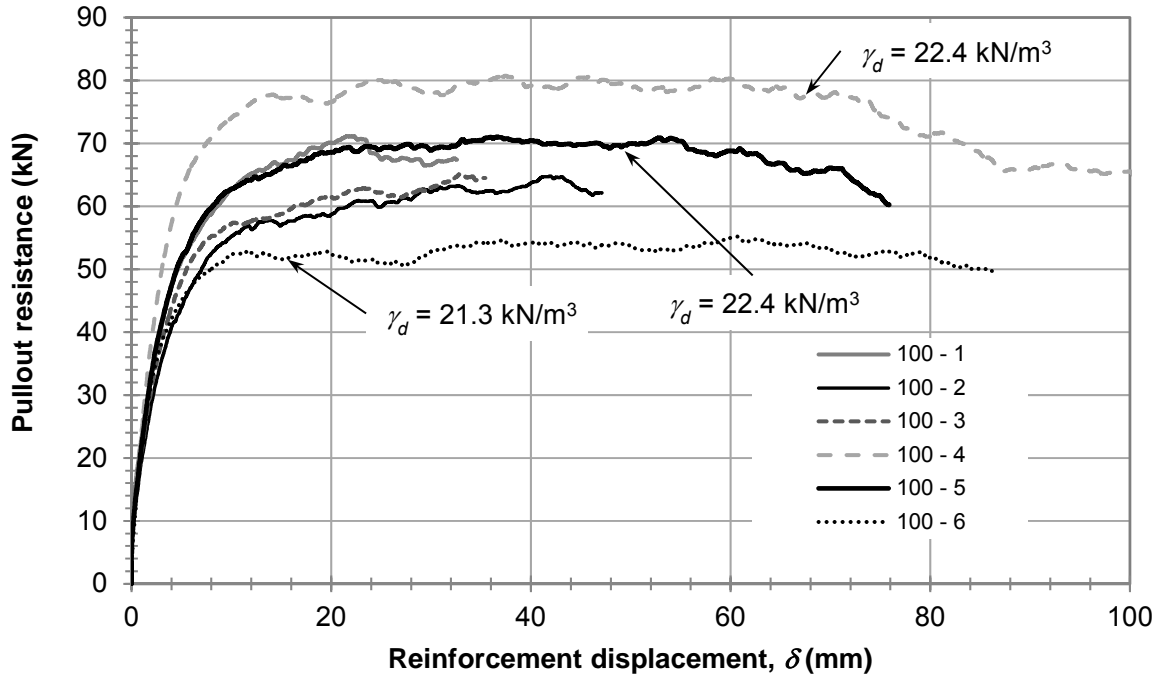


Figure 0.14. Pullout resistance versus reinforcement displacement for tests with vertical effective stresses of 100 kPa. Note, the unit weight of 100-1, 100-2, and 100-3 was not measured.

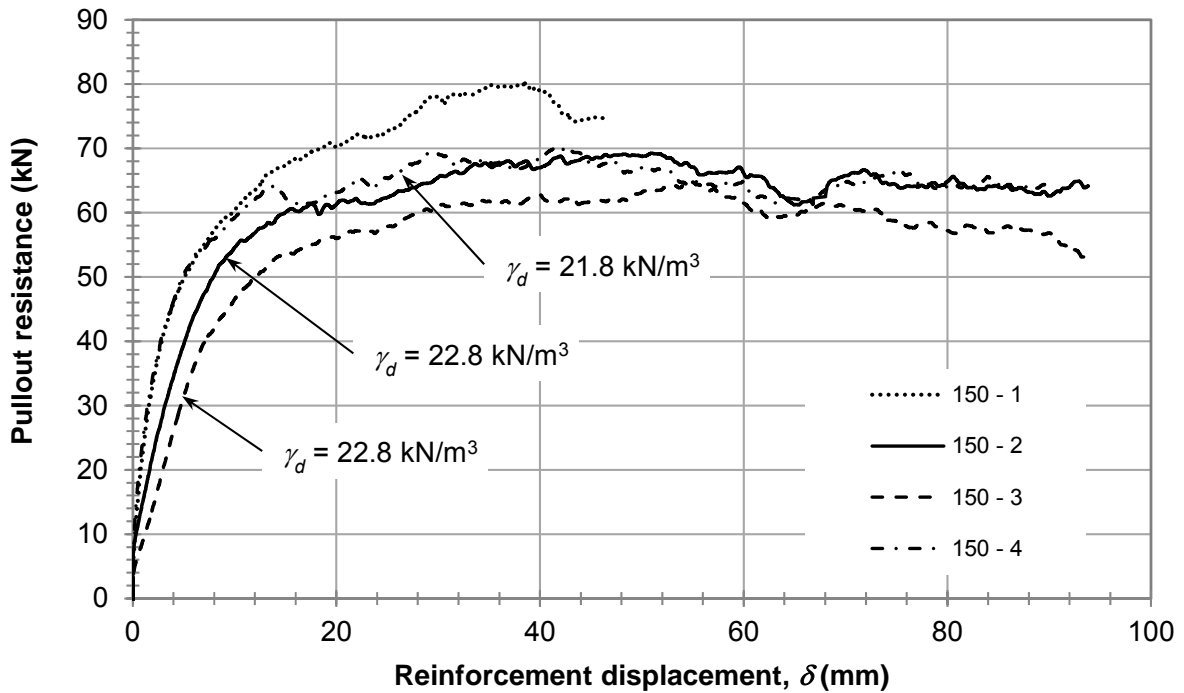


Figure 0.15. Pullout resistance versus reinforcement displacement for tests with a vertical effective stress of 150 kPa. Note, the unit weight of 150-1 was not measured.

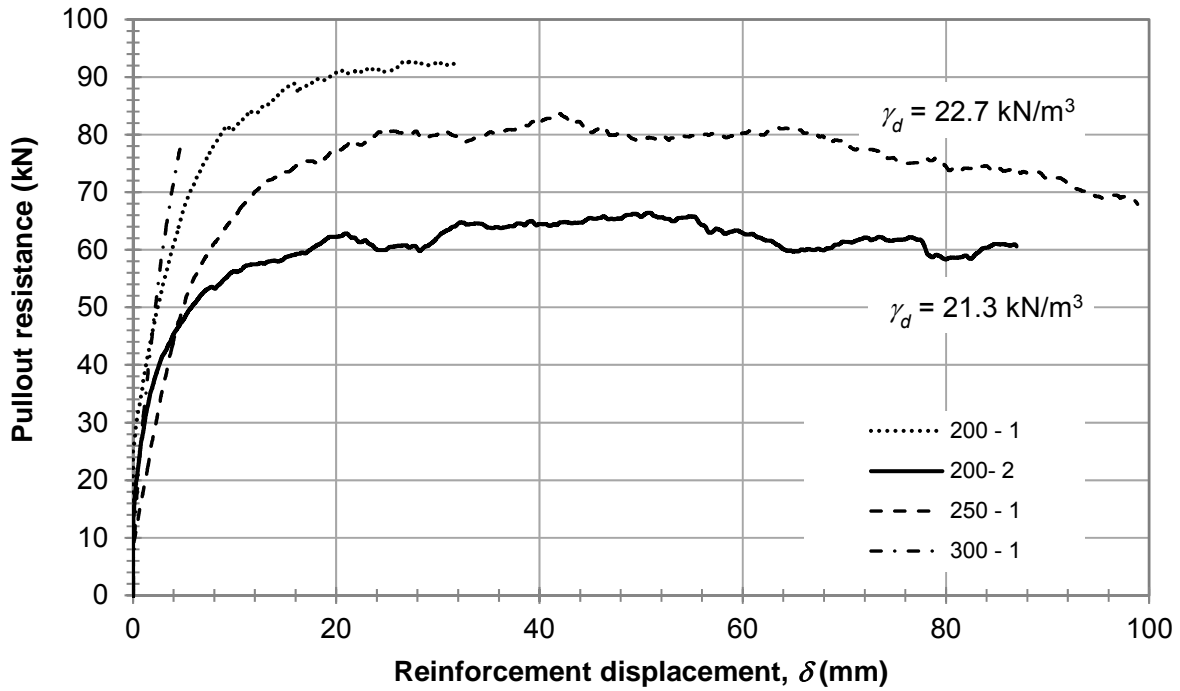


Figure 0.16. Pullout resistance versus reinforcement displacement for tests with vertical effective stresses of 200, 250 and 300 kPa. Note, the unit weight of 200-1, and 300-1 was not measured.

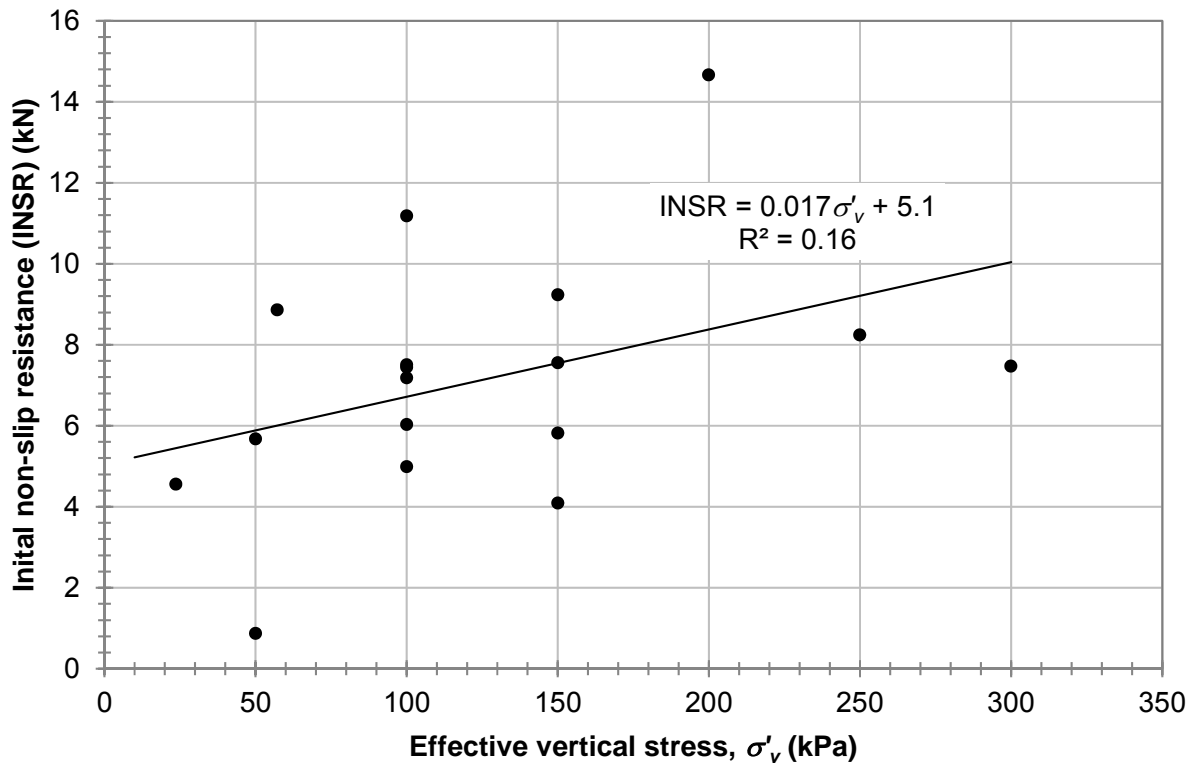


Figure 0.17. Initial non-slip resistance versus effective confining stress.

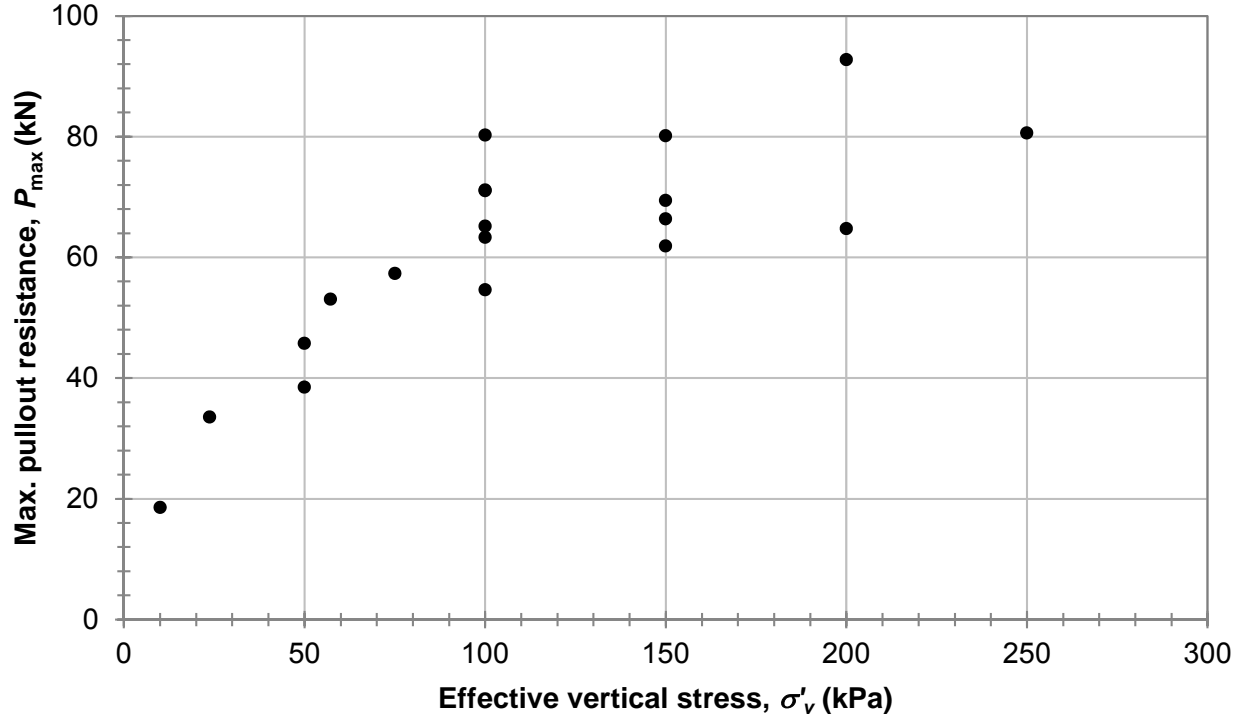


Figure 0.18. Maximum pullout resistance versus effective confining stress.

The uncertainty in the computed apparent friction coefficient, f^* , values reported in Table 0.2 were quantified using the procedure outlined in Kline and McClintock (1953). The 95 percent uncertainty intervals for each f^* input variables ($w_{\sigma'_v}$, $w_{P_{max}}$, w_b , and w_L) were estimated and used to compute combined f^* uncertainty intervals, w_{f^*} , for each test using the first order-second moment approach:

$$w_{f^*} = \left[\left(\frac{df^*}{d\sigma'_v} w_{\sigma'_v} \right)^2 + \left(\frac{df^*}{dP_{max}} w_{P_{max}} \right)^2 + \left(\frac{df^*}{db} w_b \right)^2 + \left(\frac{df^*}{dL} w_L \right)^2 \right]^{1/2} \quad (3.5)$$

The uncertainty interval for the vertical overburden stress, $w_{\sigma'_v}$, was estimated to be ± 0.4 kN/m² based on the fluctuation in airbladder pressure recorded by the pressure transducer. The uncertainty interval for the maximum pullout resistance, $w_{P_{max}}$, was estimated to be ± 0.01 kN based on the manufacture's specifications for the 100 kN load cell. The uncertainty intervals for the width and embedded length of the reinforcement, $w_b = \pm 0.0005$ m and $w_L = \pm 0.02$ m, respectively, were estimated using the author's judgment.

As shown in Figure 0.19, the f^* uncertainty interval is larger for tests conducted at lower effective vertical confining pressures and smaller for tests conducted at larger vertical confining

pressures. The computed f^* uncertainty interval values ranged from 0.76 at $\sigma'_v = 10$ kPa to 0.05 at $\sigma'_v = 250$ kPa with a mean and median values of 0.15 and 0.10, respectively.

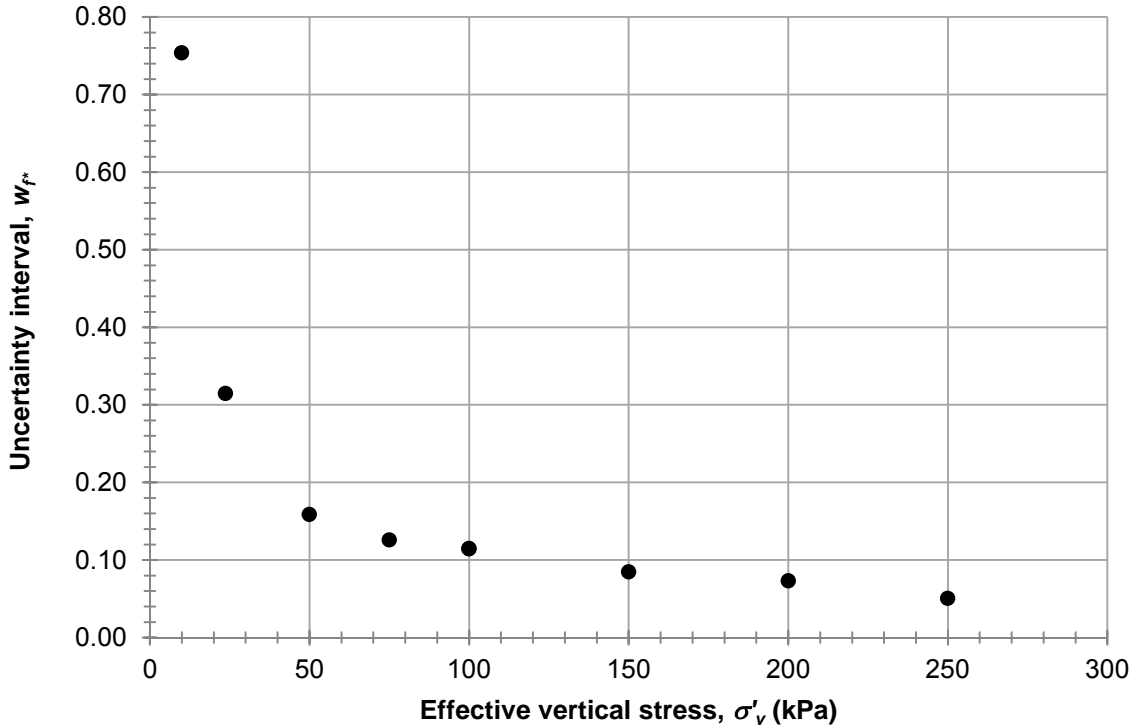


Figure 0.19. Average estimated f^* uncertainty intervals for various effective vertical stresses.

Multi-linear regression (MLR) modeling was performed on the measured pullout resistance data that was associated with a known dry unit weight in order to quantify the combined effect of compaction and vertical effective stress on the maximum pullout resistance. Two models were investigated, including a two-parameter MLR model, which included both the dry unit weight in kN/m³ and the vertical effective stress in kPa, and a one-parameter MLR model, which included just the vertical effective stress. Based on Figure 0.20, the MLR modeling was performed using the natural logarithm of vertical effective stress. The effect of dry unit weight, a proxy for relative compaction and relative density, should influence the maximum pullout resistance; however, the results of the two-parameter MLR model shows that the dry unit weight is not statistically significant at the significance level of 5 percent, as shown in Table 0.3. This could be due to the lack of data compounded with the small variation in dry unit weight associated with relative compaction values largely greater than 93 percent. If the relative compaction, and therefore relative density, varied over a larger range, it would be likely that the dry unit weight would be a statistically significant parameter. Despite the use of a statistically insignificant prediction variable, the global p-value and adjusted R^2 of the two-parameter model was 0.01 and 0.653, indicating moderately strong predictive power.

Table 0.3 Summary of multiple linear regression (MLR) modeling on maximum pullout resistance.

Model Parameters		Fitted Estimate	Standard Error	<i>t</i> statistic	<i>p</i> -value
<i>Two-parameter MLR; global p-value = 0.010; global adjusted R² = 0.653</i>					
Intercept	[kN]	-155.42	110.47	-1.41	0.2023
Ln(σ'_v)	[kPa]	7.05	5.66	1.24	0.2534
Dry unit weight, γ	[kN/m ³]	13.53	5.39	2.51	0.0404
<i>One-parameter MLR; global p-value = 0.004; global adjusted R² = 0.629</i>					
Intercept	[kN]	-20.21	20.64	-0.98	0.3560
Ln(σ'_v)	[kPa]	17.67	4.38	4.03	0.0038

Nonetheless, removal of the dry unit weight to form the one-parameter MLR model appeared warranted, and resulted in a strengthening of the significance of the vertical effective stress variable, as shown in Table 0.3. Figure 0.20 shows that the effective predictive power is essentially the same between the one and two-parameter MLR models as compared to data that represent those pullout tests with and without dry unit weight measurements. However, the one-parameter model clearly shows improved generality with respect to all of the pullout test data (i.e., the model predicts all of the data equally well), despite the use of training data that were associated with known dry unit weights, only. Figure 0.21 compares the measured maximum pullout resistance and those predicted using the two MLR models; based on this comparison, it appears that the use of the one-parameter MLR model is most appropriate for use with this specific backfill gradation until additional pullout resistance data with a larger variation in dry unit weight can be obtained.

5.4.3 Hyperbolic Modeling of Pullout Resistance-Displacement Behavior

The prediction of the load displacement behavior of MSE wall reinforcement strips at any vertical effective stress is desirable because it allows for the prediction of wall deformation under working loads and it provides a way to predict the response of sacrificial reinforcement strips scheduled to be pulled out at the SeaTac MSE walls in the near future. Hyperbolic curves were fit to the pullout load-displacement data in order to provide a general working stress displacement estimation method.

To calibrate the model, a least sum of squares regression was used on the general hyperbolic model, given by:

$$P = \frac{\delta}{\frac{1}{a_1} + \frac{\delta}{a_2}} \quad (3.6)$$

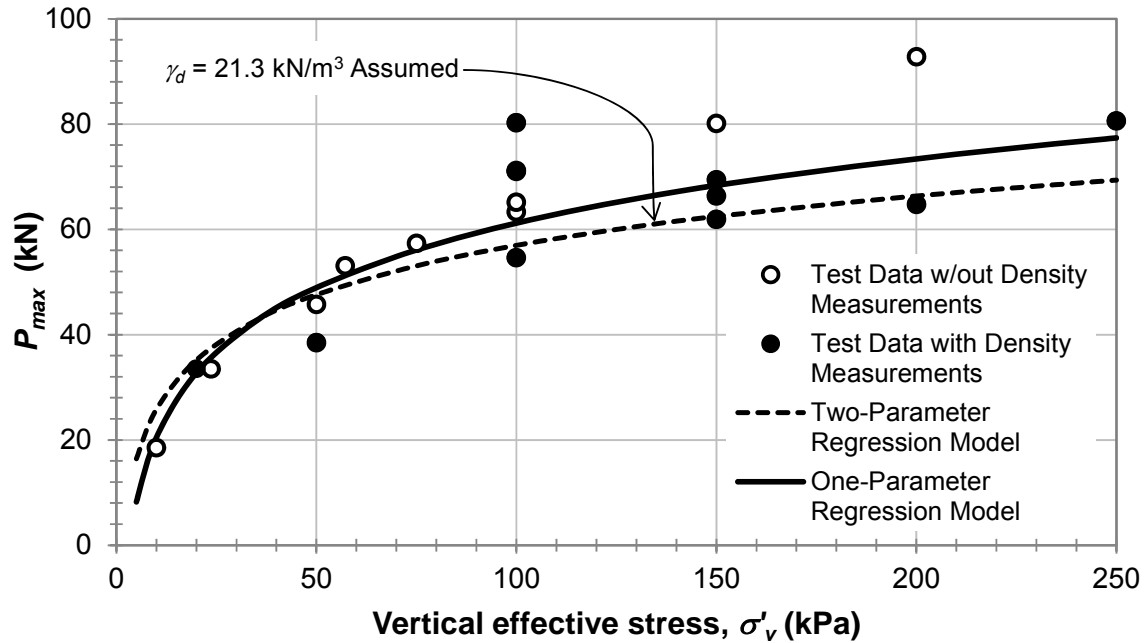


Figure 0.20. Comparison of measured and predicted maximum pullout resistance as a function of vertical effective stress.

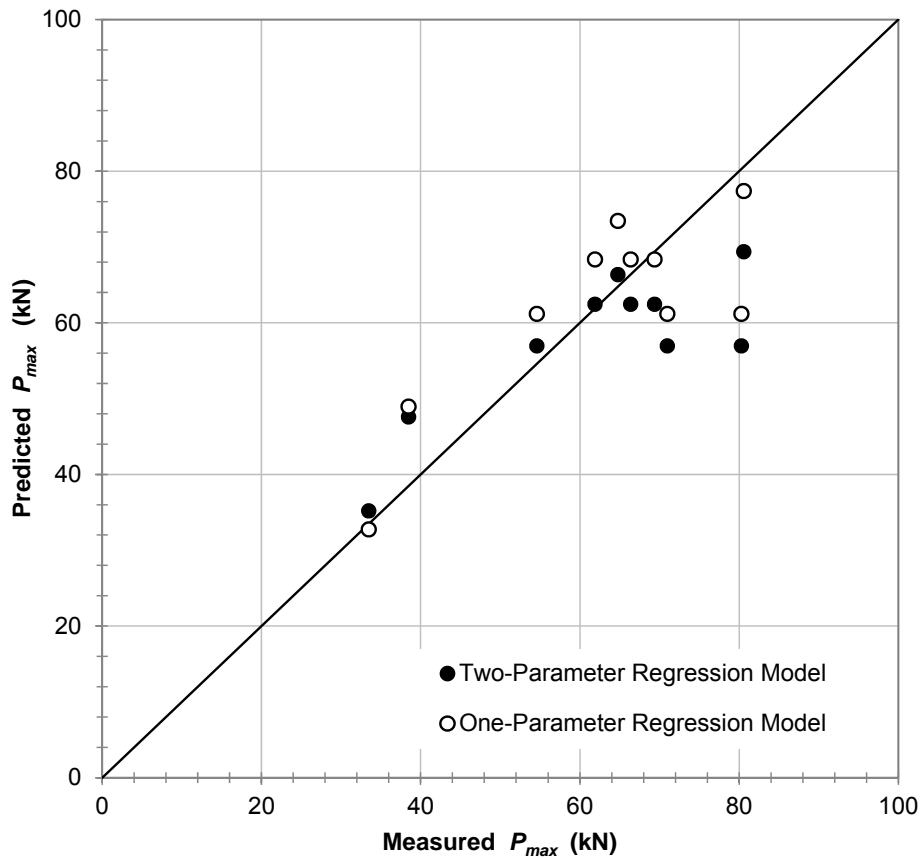


Figure 0.21 Predicted maximum pullout resistance versus measured maximum pullout resistance.

where P is the pullout resistance at displacement δ , a_1 is a coefficient related to the initial stiffness of the soil-reinforcement interaction (initial slope of the load-displacement curve), and a_2 is the asymptotic coefficient related to the maximum pullout resistance.

The variation of coefficients a_1 and a_2 with vertical effective confining pressure are shown in Figure 0.22 and Figure 0.23, respectively. The variation of the coefficient a_1 with vertical effective stress appears to be relatively random when considering all data as shown in Figure 0.22(a). However, upon removing those test data for which compaction data was either unavailable or did not meet requirements, a linear relationship was observed and was characterized with a coefficient of determination of 0.47 (Figure 0.22b). The relationship used to correlate the initial stiffness coefficient to vertical effective can be expressed as:

$$a_1 = 0.053\sigma'_v + 30.8 \quad (3.7)$$

As shown in Figure 0.23(a), a much stronger relationship exists between a_2 and vertical effective stress such that a log increase in vertical confining stress results in a 51 kN increase in maximum the maximum resistance coefficient. However, in order to capture the load-displacement behavior at the relative density of interest the same subset of pullout tests used to calibrate the initial stiffness coefficient, a_1 , was used to calibrate the maximum resistance coefficient, a_2 . Figure 0.23(b) shows the fitted logarithmic relationship along with the reduced dataset of a_2 coefficients. The models produced by the full dataset and the reduced dataset are fairly similar, though a reduction in logarithmic slope was observed; that is, for every log cycle increase of vertical effective stress resulted in a 44 kN increase in maximum reinforcement coefficient. The calibrated relationship for a_2 can be described using:

$$a_2 = 44.3\text{Log}(\sigma'_v) - 29.2 \quad (3.8)$$

In order to compare the calibrated model to measured behavior, the measured load-displacement data was compared to the load-displacement predicted using Equations 3.6, 3.7, and 3.8. As shown in Figures 5.23 through 5.26, the model does relatively well at approximating the measured behavior at displacements greater than 2 mm. However, it is difficult for the hyperbolic relationship to capture the initial non-slip portion of the measured load-displacement curve.

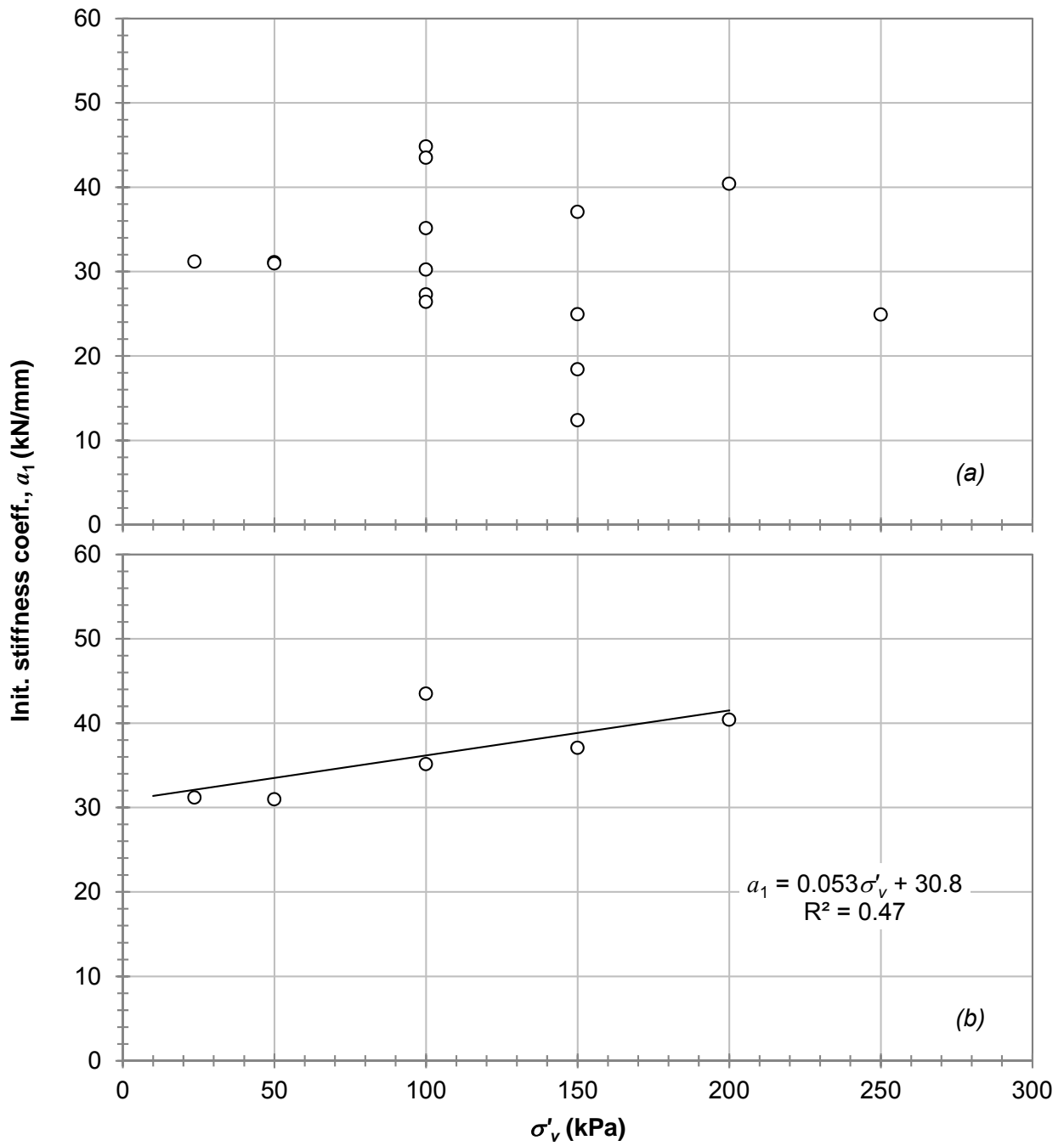


Figure 0.22 Variation of the initial stiffness coefficient, a_1 , with effective vertical stress for (a) all test data, and (b) data meeting density requirements.

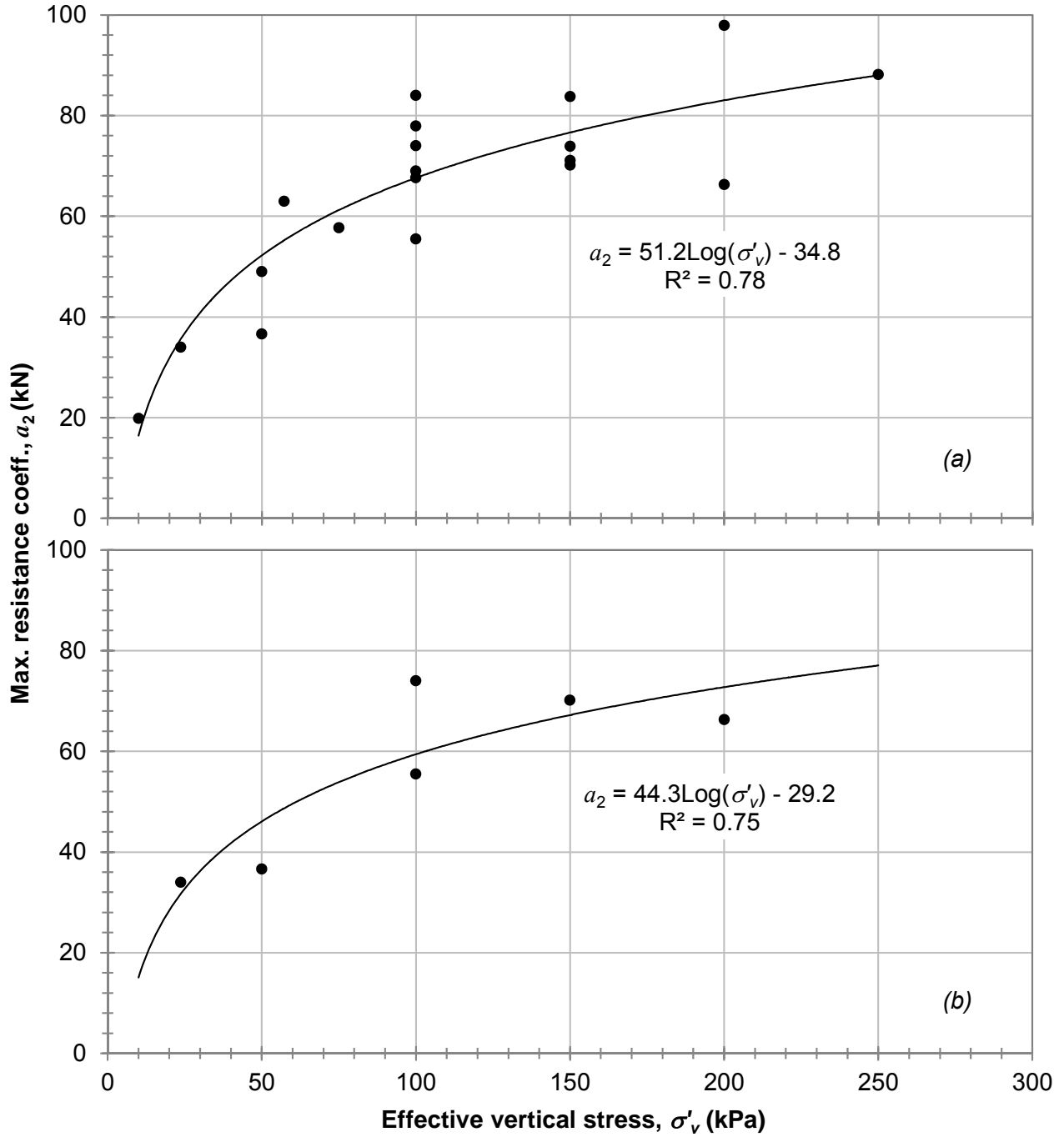


Figure 0.23 Variation of the maximum pullout resistance coefficient, a_2 , with effective vertical stress for (a) all test data, and (b) data meeting density requirements.

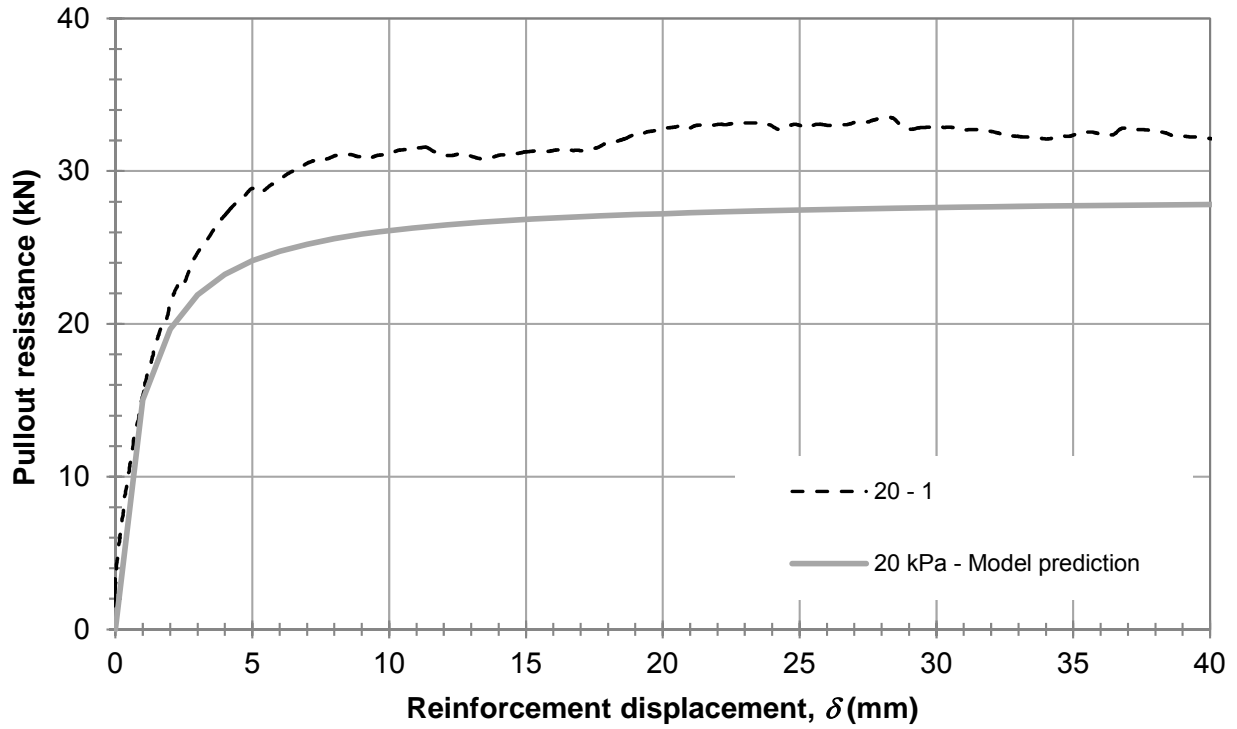


Figure 0.24 Comparison of measured pullout load-displacement behavior and the hyperbolic model prediction for $\sigma'_v = 20$ kPa.

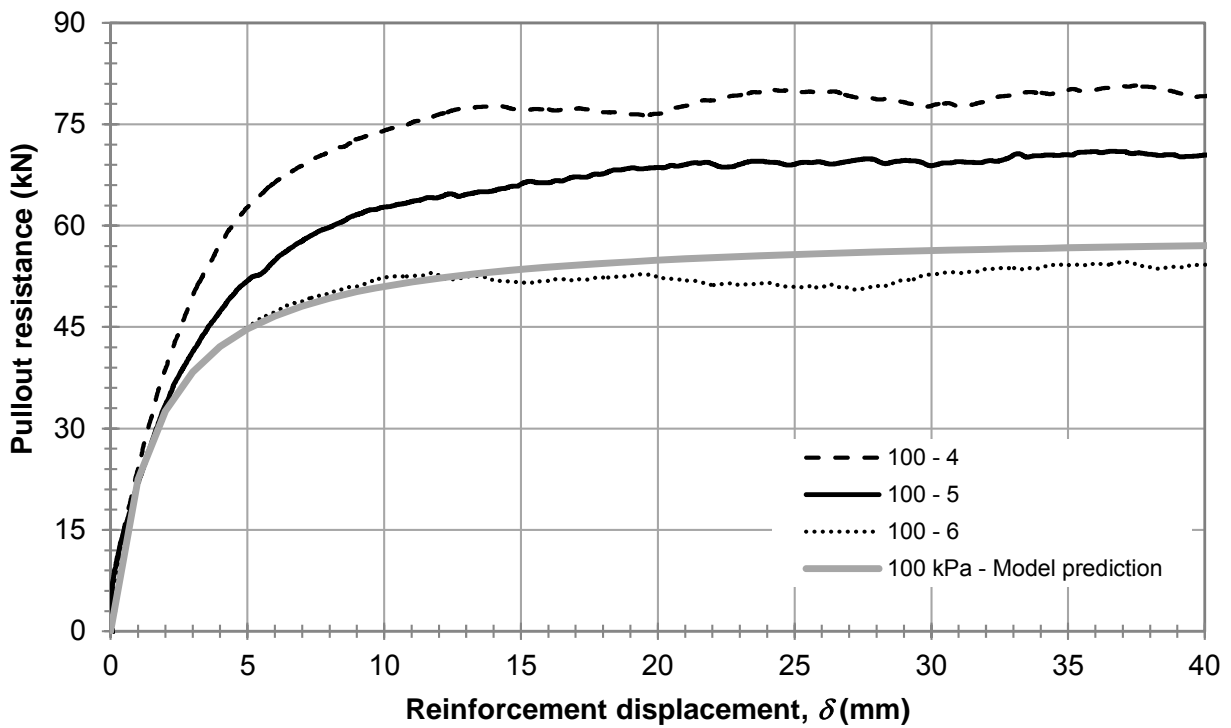


Figure 0.25 Comparison of measured pullout load-displacement behavior and the hyperbolic model prediction for tests conducted at $\sigma'_v = 100$ kPa.

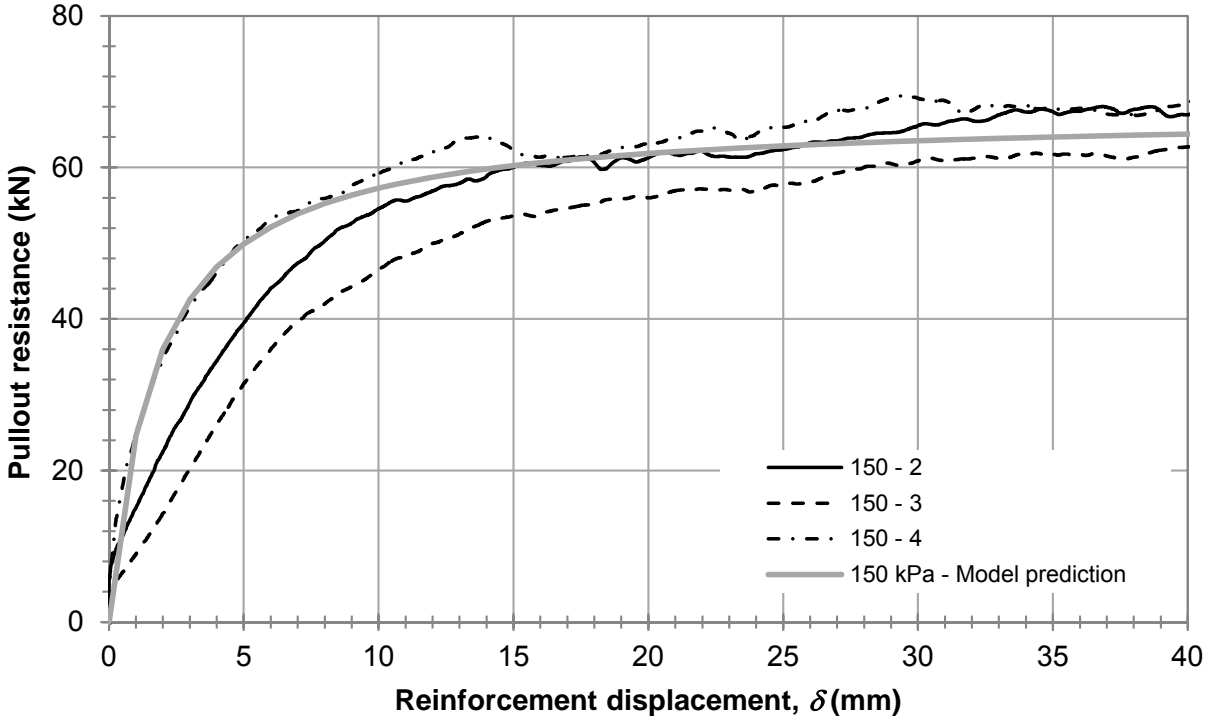


Figure 0.26 Comparison of measured pullout load-displacement behavior and the hyperbolic model prediction for tests conducted at $\sigma'_v = 150$ kPa.

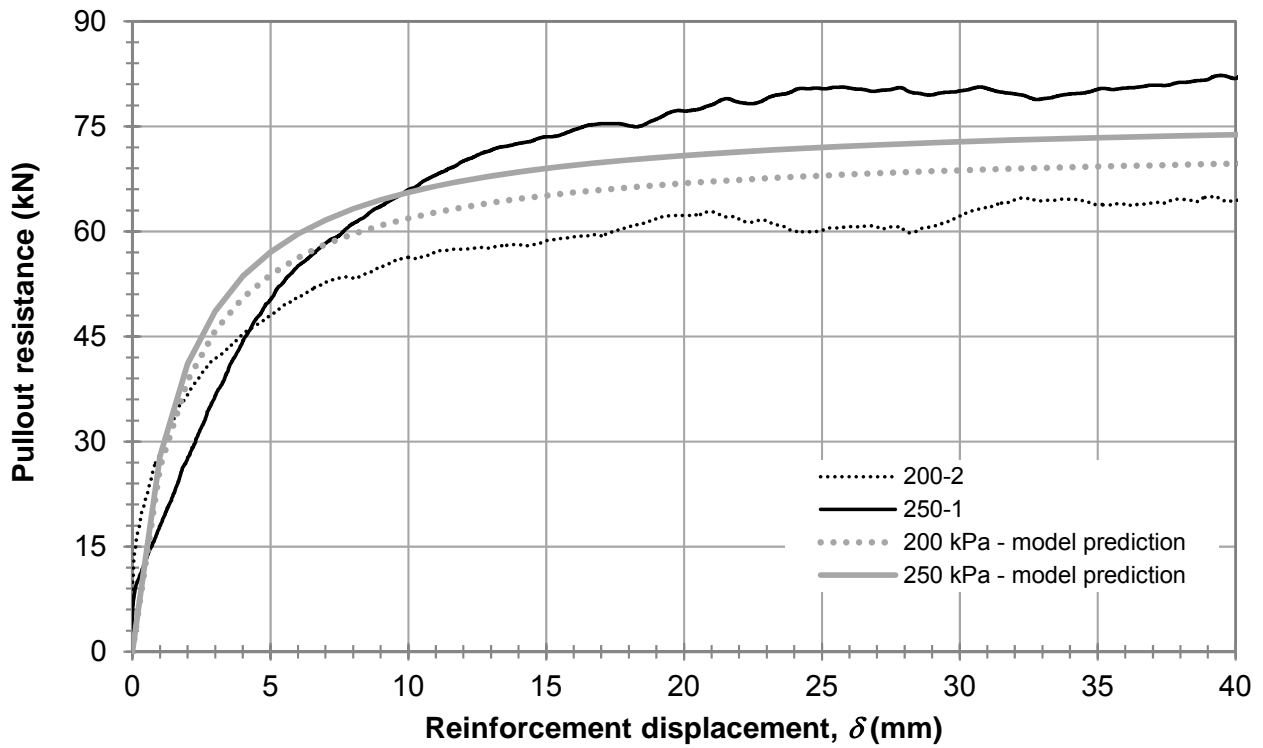


Figure 0.27 Comparison of measured pullout load-displacement behavior and the hyperbolic model prediction for tests conducted at $\sigma'_v = 200$ and 250 kPa.

5.5 Pullout Resistance Design Models for Gravel and Sand-Gravel Mixtures

The apparent friction coefficient, f^* , is an empirical design parameter used in determining the pullout resistance of MSE wall reinforcements as discussed in Section 2.2.1.2. Since the introduction of the apparent friction coefficient into accepted MSE wall design procedures, the same design model has been used to determine f^* as a function of depth. As specified by AASHTO (2010), values of f^* are not permitted to be taken as larger than 2. This limit applies to all soils types, even highly dilative soils such as gravels and sand-gravel mixtures. As shown in Figure 2.26, values of f^* well above the current design model have been measured for gravels and sand-gravel mixtures.

In order to better represent the relationship between the apparent friction coefficient and vertical effective stress for gravels and sand-gravel mixtures, a new gravel-specific design model was developed. Apparent friction coefficients of five soils were used in the calibration of the new model and are presented in Figure 0.28. The soils reported by McKittrick (1978) consisted of two gravels (G-1 and G-2) and a sand-gravel mixture (SG-1). Specific gradation information for each of these soils were not provided, however, the friction angles were reported equal to 47, 46, and 37 degrees, respectively. The soil taken from Boyd (1993) is a poorly graded gravel with the gradation presented in Figure 0.29. Though a friction angle of 35 degrees was assumed for the design of the wall, the results of strength tests were not reported by Boyd (1993).

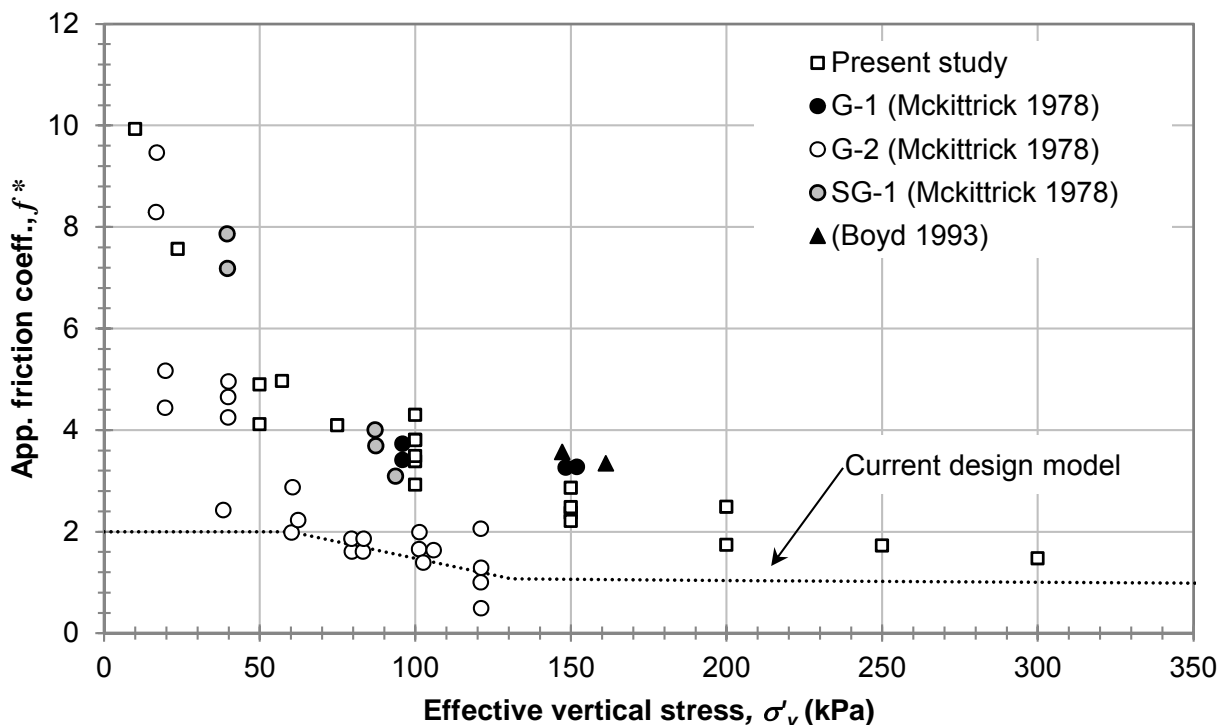


Figure 0.28 Apparent friction coefficient versus effective vertical stress for pullout tests with gravel and sand-gravel mixtures.

A new gravel-specific f^* model was calibrated using the pullout resistance data reported for the five different soils. An ordinary least squares regression with the form of an assumed function was used in the calibration. The model is presented in Figure 0.30(a) which indicates that at $\sigma'_v = 0, f^* = 17$, and then f^* decreases with increases in σ'_v to eventually converge to an asymptote of $f^* = 1$. The proposed general gravel model is expressed as:

$$f^* = \frac{194}{12 + \sigma'_v} + \tan \phi' \quad (3.9)$$

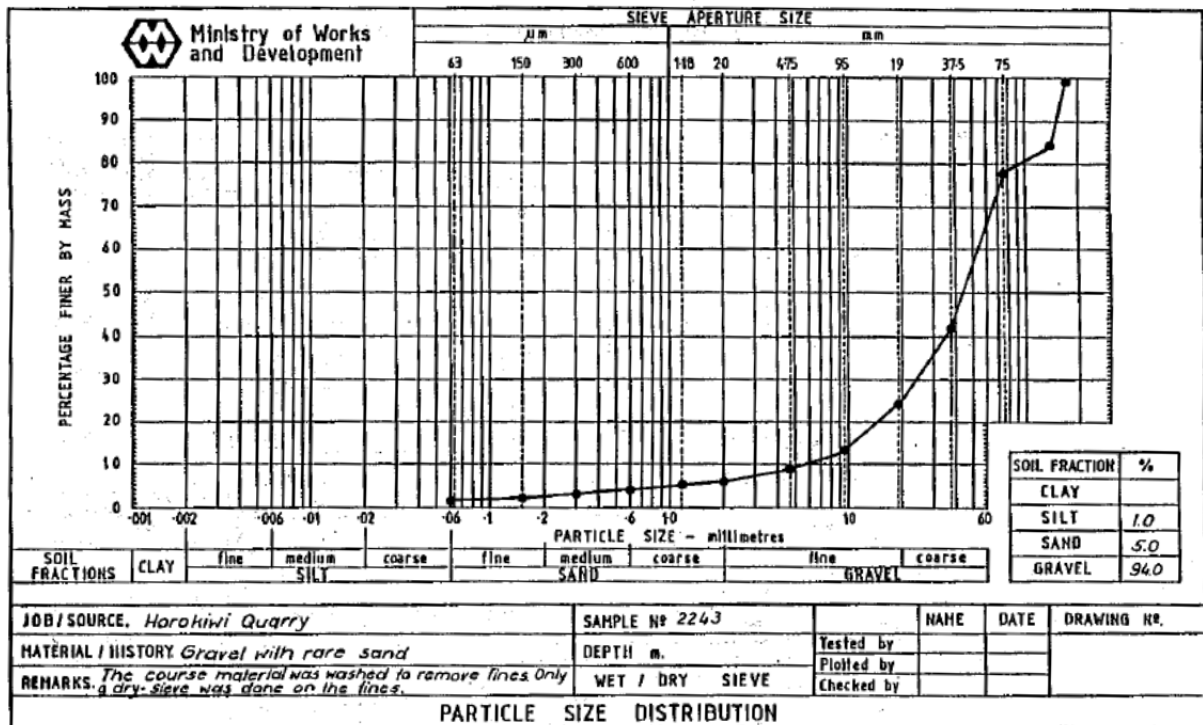


Figure 0.29 Particle size distribution of poorly graded gravel (after Boyd 1993).

Though the conservative design model discussed in Section 2.2.1.4 is typically used in design, AASHTO allows more aggressive f^* distributions if adequate pullout testing is completed. Using the same calibration procedure as was used for the general gravel model, a new backfill-specific model was calibrated for use with single strips. The backfill-specific model is presented in Figure 0.30(b), in which similar behavior to the general model was observed. The main difference between the two models is the rate of reduction in f^* which is greater for the general model than for the backfill-specific model. The proposed backfill-specific design model is proposed as:

$$f^* = \frac{286}{21 + \sigma'_v} + \tan \phi' \quad (3.10)$$

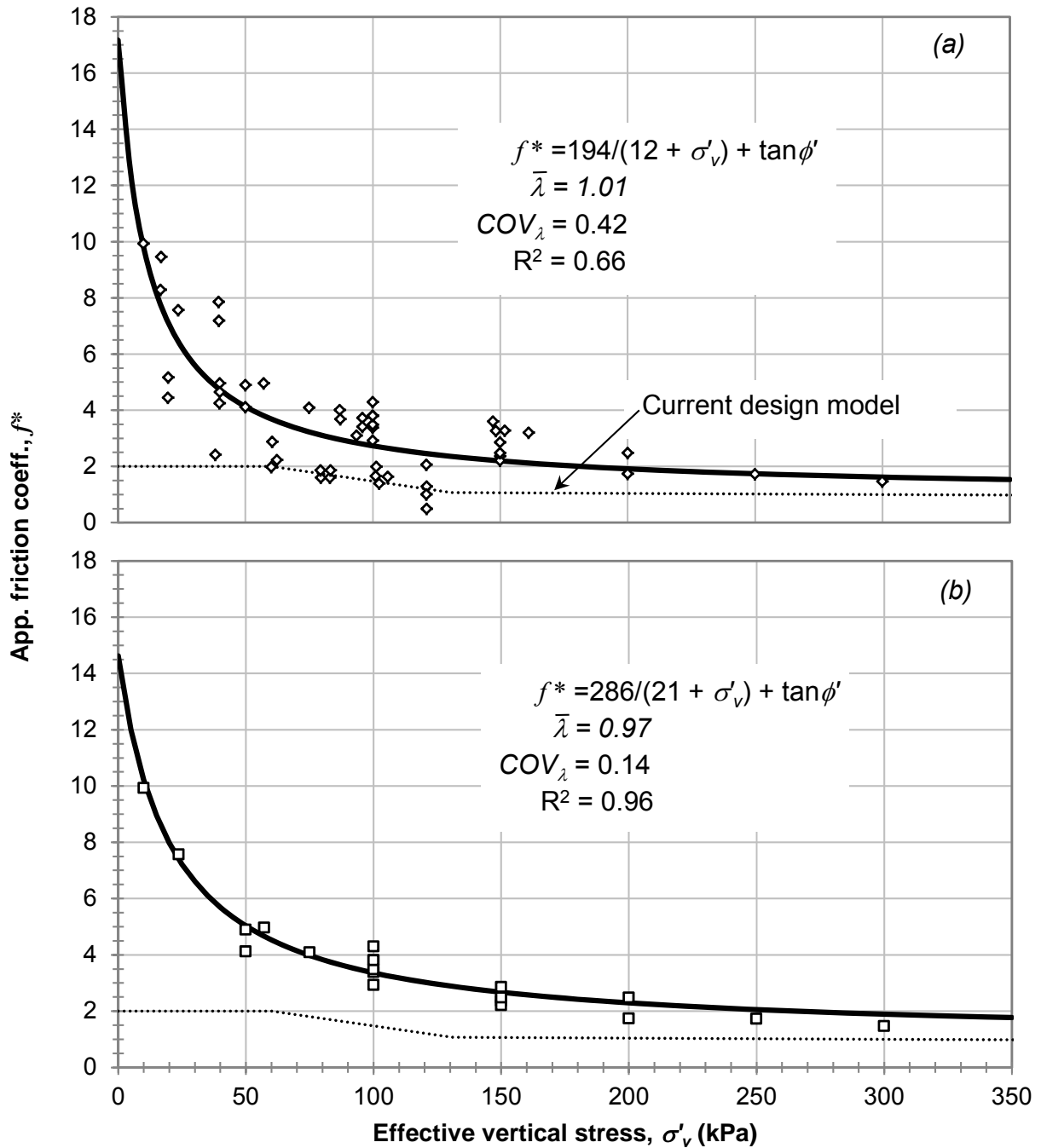


Figure 0.30 f^* design models for (a) general gravel and sand-gravel backfills, and (b) specific design model for backfill tested during the present study.

Table 0.4 presents the results of a statistical analysis performed to analyze the quality of fit of the AASHTO and general gravel f^* models to the general gravel pullout dataset (Dataset A), and the quality of fit of the AASHTO, general gravel, and backfill -specific f^* design models to the dataset of pullout results from the present study (Dataset B). Overall, the sample biases

ranged from 0.20 to 4.96 with an average sample bias between 0.97, and 2.45. The AASHTO design model produced the highest average biases for both datasets, indicating the under-prediction of f^* values and thus a more conservative model. For Dataset B, the global gravel and backfill-specific models produced similar f^* predictions, though the backfill-specific model produced a smaller average bias. For Dataset A, the global gravel model produced biases ranging from 0.20 to 1.65 with an average bias of 1.01, whereas AASHTO model bias values ranged from 0.45 to 4.96 with an average of 2.23, indicating more conservative and less accurate predictions of f^* . For the global gravel model and Dataset A, a coefficient of variation (COV) of 36 percent and coefficient of determination, R^2 , of 0.66 were computed, indicating a better fit to the data when compared to the AASHTO model, which produced a COV and R^2 of 44 percent and 0.40, respectively. The use of the AASHTO model in predicting the f^* values for Dataset B resulted in biases ranging from a minimum of 1.37 to a maximum of 4.96, a COV of 32 percent, and a R^2 value of 0.65. In comparison, the global gravel and backfill- specific models produced biases ranging from 0.97 to 1.57, coefficients of variation of approximately 15 percent, and coefficients of variation of 0.94 and 0.96, respectively, indicating much more accurate and precise predictions of f^* when compared to the AASHTO model.

Table 0.4. Summary statistics on the performance of different f^* design models considered in the present study.

Dataset/Design Model	Mean bias	Range in bias	COV (%)	R^2
A: General gravel data base				
AASHTO	2.23	4.96	0.45	44
Global gravel	1.01	1.65	0.20	36
B: Present study (all tests)				
AASHTO	2.45	4.96	1.37	32
Global gravel	1.17	1.57	0.90	15
Backfill Specific	0.97	1.28	0.75	14

5.6 Summary

This chapter presented the results of twenty pullout tests on MSE wall ribbed steel strip reinforcements embedded in the sand-gravel backfill analyzed in Chapter 4. The primary conclusions and contributions of this chapter include:

1. A new apparatus was designed and constructed for performing full scale laboratory pullout tests on ribbed steel strip reinforcements;
2. Twenty pullout tests were performed on the steel ribbed strips and backfill discussed in Chapter 4;
3. The effects of compaction and vertical effective stress on maximum pullout resistance was quantified statistically;

4. Hyperbolic coefficients were fit to the load-displacement data from pullout test results for future possible use in a working stress displacement prediction model.
5. A database of ribbed steel strip pullout tests in gravels and sandy gravels was developed by combining available data in literature with the pullout results from the present study;
6. Two new pullout resistance design models for use with gravels were calibrated:
 - a. using all of the available pullout tests data for gravels in the literature; and
 - b. using the pullout test results from the present study exclusively to form a backfill-specific design model.
7. The performance of the two new pullout resistance models were compared to the current AASHTO design model and found to produce much more accurate and precise predictions of the apparent friction coefficient.

Chapter 6 Multi-Strip Soil-Reinforcement Interaction

The main focus of this research is to determine whether or not closely-spaced reinforcements interact with one another, and what role spacing plays on the degree of interference. This chapter describes the experimental program developed to observe the pullout resistance response of closely-spaced reinforcements and presents the preliminary findings results from the experimental program.

6.1 Pullout Test Apparatus for Multiple Reinforcement Strips

6.1.1 Experimental Setup

In order to observe the pullout resistance of closely-spaced reinforcements, a specially designed multi-strip pullout box was designed and constructed. The multi-strip pullout box assembly consisted of two main components: the pullout box that contained the compacted reinforced soil specimen and the main reaction frame assembly. A three dimensional rendering of the two components is presented in Figure 6.1. The interior dimensions of the reinforced soil sample were 1.52 meters (5 feet) in width, 1.52 meters (5 feet) in height and 1.83 m (6 feet) in length; this corresponds to a sample size of 4.25 cubic meters of compacted soil, approximately 11 times larger than the single strip pullout apparatus by volume. The large pullout box assembly was designed to accommodate nine reinforcement strips. The compacted soil was retained with 12.7 mm (0.5 inch) thick steel plates while the structural support was provided by 22 steel flange sections (W8 x 21), 0.2 meters in height. The interior dimension of the steel pullout box is approximately 0.10 m taller than the soil sample to allow the placement of an air bladder that can be pressurized during testing as shown in Figure 6.2. By pressurizing the bladder, the vertical or normal stress could be increased to assess the effect of overburden pressure on the interaction of variably-spaced reinforcements.

The large pullout box was designed using Allowable Stress Design (ASD) and a maximum bladder pressure of 250 kPa. Initial estimates of expected reinforcement strip loads indicated that at pressures greater than approximately 250 kPa the metallic reinforcement would yield during testing. The main steel wide-flanged beam support sections were sized using a combined loading of the 250 kPa bladder pressure and the self-weight of the soil. The vertical to horizontal ratio of soil pressures were assumed to be 1 to 1 for the purpose of sizing members, reasonable given the compactive effort provided to the soil during placement. The front of the box contained two horizontal channel sections (MC10 x 33.6) that were 0.254 meters in depth and 1.73 meters in length, placed above and below the reinforced pullout zone (Figure 6.1). These were designed to transfer the loads that develop during pullout testing to the reaction frame assembly. The four vertical channel supports were constructed of the same channel section but were substantially shorter (0.69 meters in length). Additional renderings and shop drawings for the large pullout box and reaction frame assembly are presented in Appendix D.

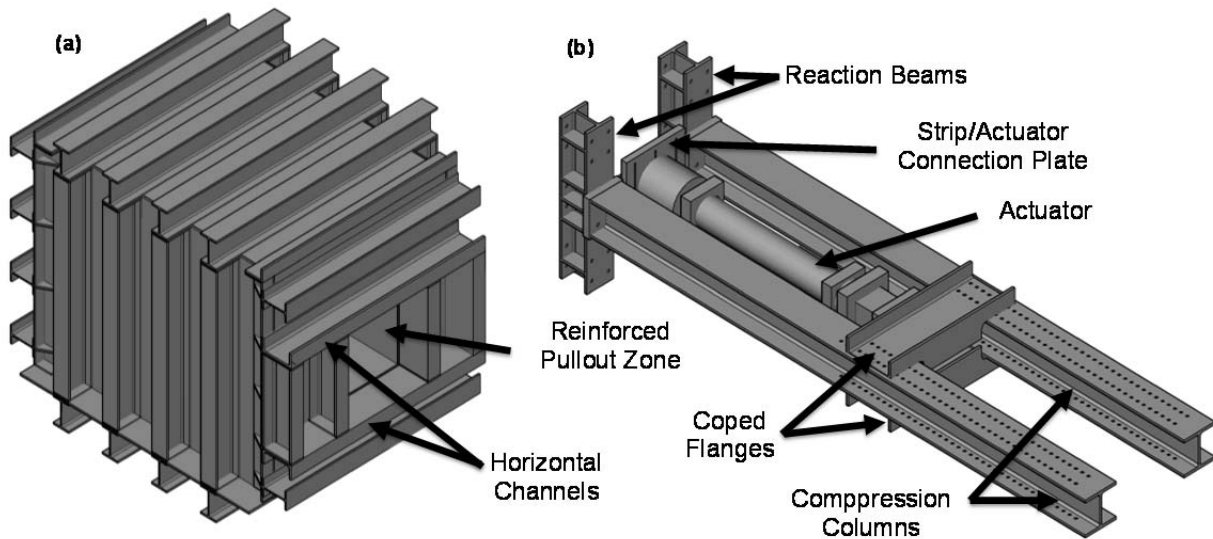


Figure 6.31 Three dimensional rendering of the (a) multi-strip pullout box and (b) reaction frame assembly.

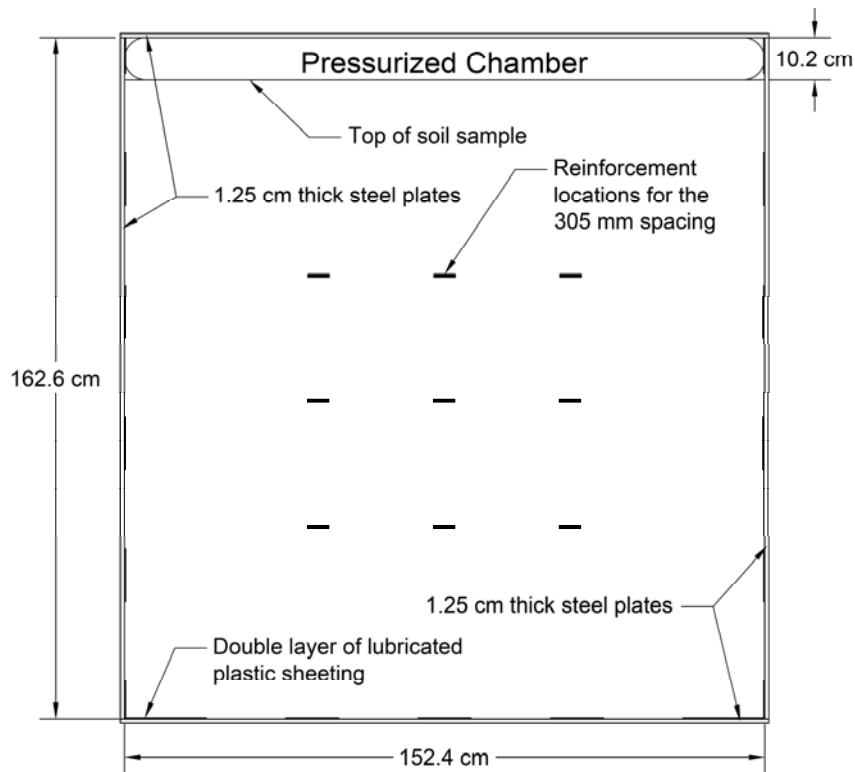


Figure 6.2 Cross section of the large multi-strip pullout box.

The reaction frame assembly, presented in Figure 6.1(b), consisted of two 6.1 meter (20 feet) long main columns (W12 x 120, Grade A572) which were elevated to the center of the large pullout box using six support plinths. The 2224 kN (500 kip) capacity actuator was connected to the two main reaction beams using two coped flange sections. Also fastened to the end of the

main columns were two 1.63 meter long (64 inch) reaction beams (W12 x 120) which rest against the front channel sections of the large pullout box. Figure 6.3 presents pictures of the reaction frame/large pullout box assembly.



Figure 6.3 Photos of the multi-strip pullout box assembly.

Located at the end of the main piston of the actuator was a 50 mm thick steel connection plate that allowed the metallic reinforcement strips to be attached to the actuator (Figure 6.4). The connection plate was grooved to allow six angle sections (L3 x 2 x 5/8) to bolt to it for fixity, but remain adjustable to accommodate the desired reinforcement spacing. The metallic reinforcement strips were then positioned between the angle sections and created a system that could be adjusted to match the desired test spacing without compromising structural integrity. The spacing between the the angle sections were just slightly larger than the thickness of the reinforcement strips to allow the strips to move freely during testing. The connection plate is presented in Figure 6.4.

The open area at the front of the large pullout box, termed herein as the reinforced pullout zone, was designed to facilitate various reinforcement configurations and measures 711 mm square. Various reinforcement spacing configurations were accommodated using four steel plates: a 102 mm, a 203 mm, a 254 mm plate, and a specially designed plate that could accommodate either 152 mm or 305 mm spacing. After placement of the steel plate a perforated rubber membrane was positioned on the interior of the box and fixed in place. Steel channel reinforcements were then bolted on to the horizontal channel sections of the large pullout box. Figure 6.5 present pictures of the reinforced pullout zone after a completed test.

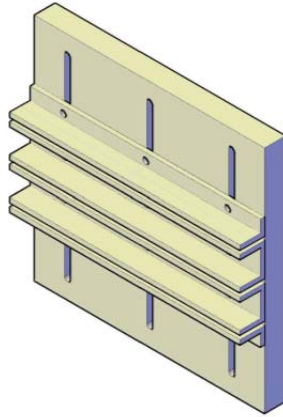


Figure 6.4 Connection plate between actuator and metallic reinforcement strips.



Figure 6.5 View of reinforced front panel section during test.

The displacement rate of the actuator was set to 1 mm per minute (0.04 inches per minute) during testing, which equals the displacement rate used for the single strip pullout tests described in Chapter 5. Strain gages were attached to each metallic reinforcement strip and used to measure the load in the strip. Each reinforcement strip was instrumented with four strain gages: two gages on the bottom and two gages on the top. The gages were placed at the same longitudinal location on the strip, within a tenth of a meter from the connection plate, and were spaced evenly in the transverse direction. The strain gages were oriented such that the primary strain direction was in the longitudinal direction. The flexural bending strains that develop from small variations in strip placement could be eliminated in the data by averaging the readings from the top and bottom gages. Three string potentiometers were attached to the connection plate to determine the displacement of the connection plate. Experimental observations were

recorded using a data acquisition system (Figure 6.3), which included the strain gage data, bladder pressure, connection plate displacement, and actuator response.

6.1.2 Sample Preparation and Deconstruction

During sample preparation the lid of the large pullout box was removed, the interior of the box was coated with a spray lubricant, and one layer of 6 mil plastic sheeting was laid out inside the pullout box. Prior to placement of a second layer of six mil plastic sheeting, lubricant was sprayed on top of the first layer of sheeting. After placement of the second layer of plastic sheeting, five straps with a load rating of 9 Mg (10 tons) were laid across the box to facilitate sample deconstruction. Figure 6.6 shows the large pullout box prior to soil placement.

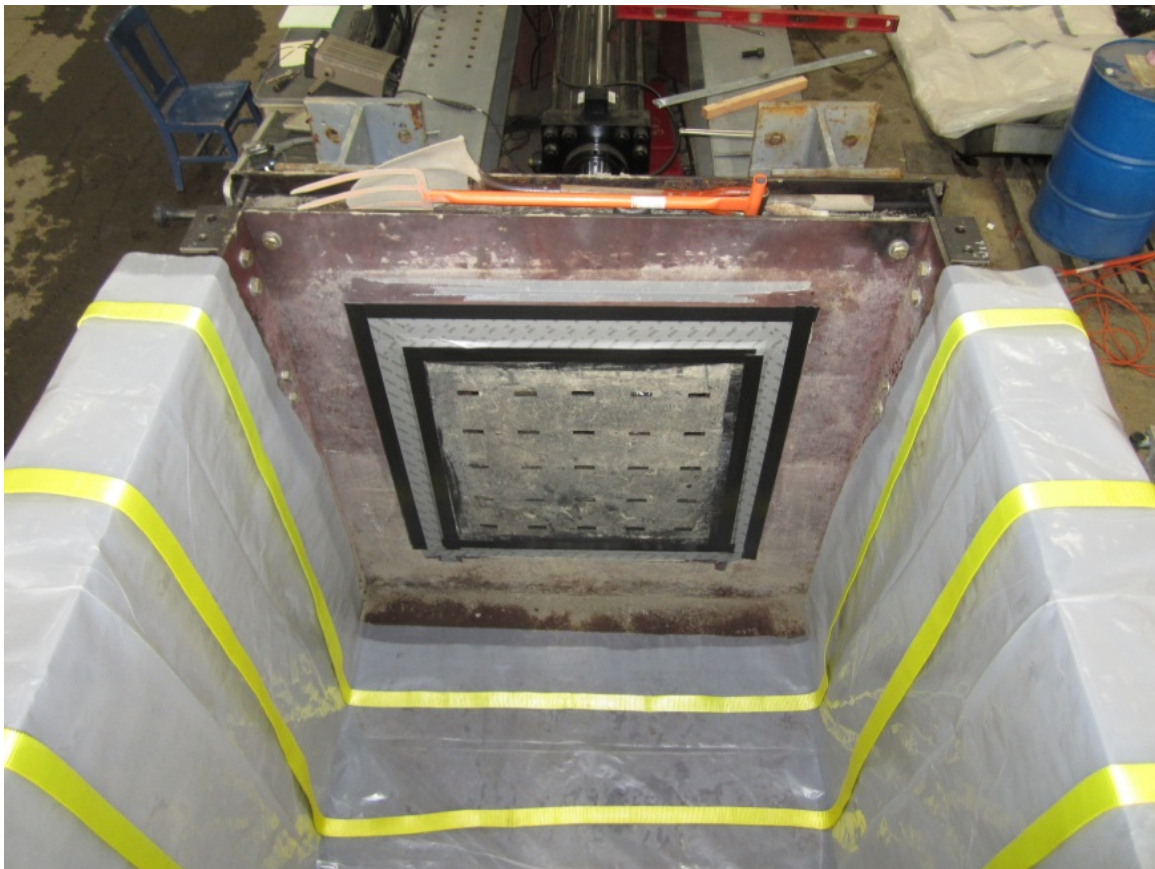


Figure 6.6 Picture of the multi-inclusion pullout box with lid removed prior to soil placement.

Soil was placed into the box and compacted using a vibratory plate compactor (Bomag M/N: BVP 18/45) in 0.1 to 0.15 meter (four to six inch) lifts, depending on the reinforcement spacing. Density was verified using Balloon Density Gage (BDG) tests which were performed for selected lifts and locations. Figure 6.7 presents the BDG being conducted on a lift of compacted

soil. Reinforcement strips were placed on lift elevations as dictated by test requirements; a view of strip placement on a compacted lift is shown in Figure 6.8.



Figure 6.7 Balloon density gage testing.



Figure 6.8 View of reinforcement strips placed on a compacted lift of fill soil.

After the completion of a test the top and rear plates of the large pullout box were removed to deconstruct the sample. Figure 6.9 presents a view of the back of the sample after the rear plate has been removed and in which the lift thicknesses are visible. Deconstruction of the sample typically began by lifting up on the yellow straps embedded beneath the. The straps loosened the densely compacted soil sample and minimized the labor required to fully deconstruct the sample.



Figure 6.9 Soil specimen after removal of rear plate and support sections.

6.2 Test Program

The multi-strip test program was developed to compare the effects of reinforcement spacing on pullout resistance at different vertical effective stresses. The vertical and horizontal reinforcement spacing, S_v and S_h respectively, were chosen based on the typical spacing employed in the lower tiers of the SeaTac MSE wall, where observations indicated potential evidence for frictional interference and/or load concentration in closely-spaced reinforcement

strips. The test program consisted of multi-strip pullout tests with applied pressures of 30, 100, and 200 kPa and reinforcement spacing that ranged from 102 to 406 mm. The maximum pressure was limited to 200 kPa to limit the onset of yielding in the steel reinforcement strips. Table 6.1 shows the multi-strip test program and indicates which tests have been completed to-date.

Table 6.5 Proposed and completed multi-inclusion pullout tests.

S_v (mm)	S_h (mm)	Applied Pressure (kPa)		
		30	100	200
102	102		P	
152	152	C	C	C
203	203		C	
254	254		C	
305	305	C	C	P
406	406		P	

P = Proposed, C = Completed

6.3 Preliminary Results for Multi-Strip Pullout Resistance

The following section discusses the preliminary results of pullout tests conducted using the large multi-inclusion pullout box. The entire laboratory testing program has not yet been completed, and therefore the following discussion will concentrate on a portion of the test program.

6.3.1 Pullout Load-Displacement Response

A comparison of the multi- and single strip pullout tests showed indicated that some frictional resistance is impacted by close spacing. For ease of comparison, the selected test designation used in this study consists of:

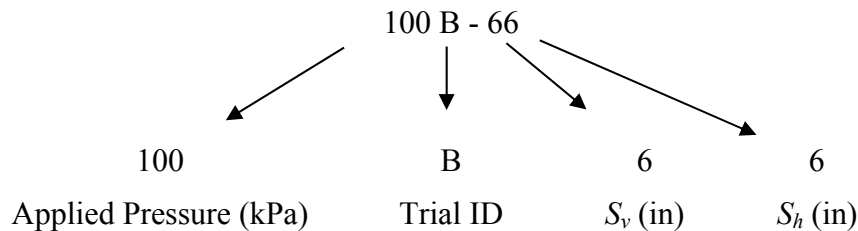


Figure 6.10 presents the pullout load-displacement response for a multi-strip test conducted with an applied pressure of 100 kPa and a reinforcement spacing of 152 mm (six inches). Also shown is the result of a single strip pullout tests (100-6) conducted with fill compacted to same degree of relative compaction. The general pullout load-displacement response of the multi-strip pullout test is similar to the single strip pullout test, as the initial response is fairly stiff, achieves a peak pullout load, and then slowly sheds its load exhibiting

stick-slip behavior as indicated with the variably increasing and decreasing pullout resistance along the general softening trend. However, the preliminary test results in Figure 6.10 indicate that at low displacements, for example less than 10 mm, the single strip pullout response is stiffer than the response of strips placed in close proximity. This is likely the result of a larger soil mass being mobilized during testing. Figure 6.10 also indicates that some reinforcement strips acted stronger than the single strip, and others weaker. Although variations due to variability in compaction explain some differences, it is observed that strips that are confined by other strips in the multi-strip setup exhibit greater frictional resistance than single isolated strips, as described below.

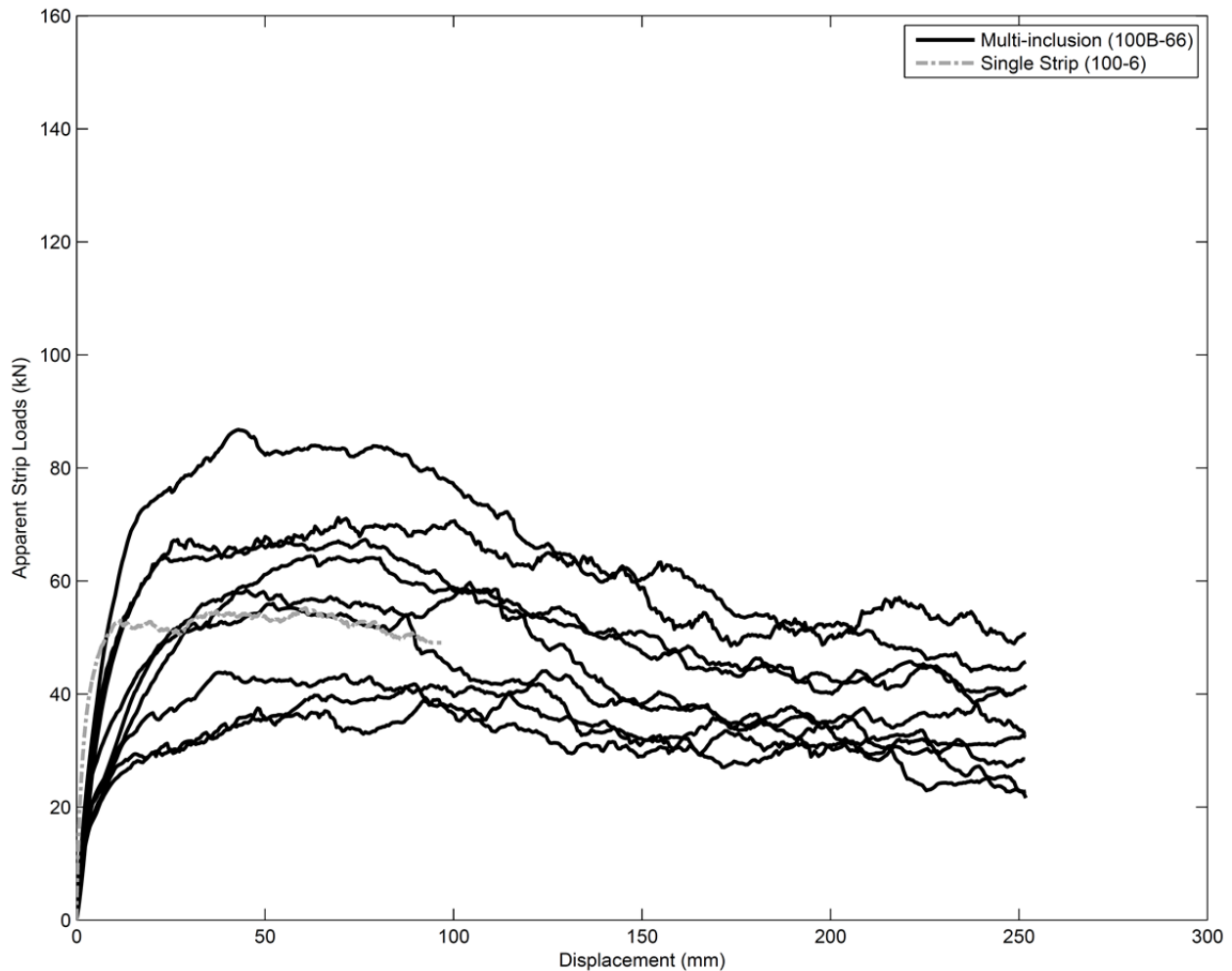


Figure 6.10 Test results from a 100 kPa multi-inclusion pullout test at a reinforcement spacing of 152 mm (100B-66).

6.3.2 Interactions Between Closely-spaced Reinforcement Strips

For convenience in making comparisons, reinforcement strips were numbered sequentially in order of position as shown in Figure 6.11. This layout of the strips allows for comparison between rows and columns to quantify the effects of vertical and horizontal spacing.

Figure 6.12 provides the initial multi-strip load-displacement results presented in Figure 6.10 separated into rows and columns. The forces in the metallic reinforcement strip were calculated by multiplying the averaged strain gage data by the Young's modulus produced from coupon tests and the cross sectional area of the strip.

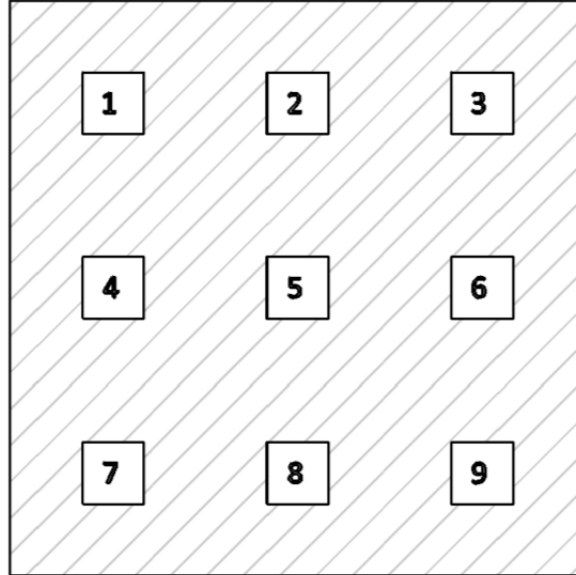


Figure 6.11 Strip number layout.

A comparison of total load-displacement response recorded by the actuator is compared to the summation of the loads of the nine metallic strips in Figure 6.12(a). In general, the summation of the loads is greater than that measured by the actuator and may be due to uncertainties in measurements of the cross sectional area of the reinforcement strip. The total, or global, load-displacement behavior of the multi-inclusion pullout test follows the same trend as a single strip pullout test on this material. At initial displacements the load response is steep and approaches a peak value then reduces to some residual load.

Figures 6.12(b), 6.12(c), and 6.12(d) present the load-displacement response of the metallic reinforcement strips on a row by row basis for a 152 mm test at an applied pressure of 100 kPa. For this test, strips two, five, and eight all exhibit larger peak loads in relation to the other strips in the same row. Additionally, figures 6.12(b), 6.12(c), and 6.12(d) also indicate that the peak loads of the middle row (strips 4, 5 and 6) are greater than the top and bottom rows (strips 1, 2, 3, 7, 8, and 9). The peak load of strips 1 through 9 for each multi-strip pullout test was used to develop vertical and horizontal load multipliers that quantify the effect of reinforcement spacing. To develop the vertical load multiplier, the peak load of the load-displacement behavior for the middle row (strip 4, 5, or 6) was divided by the average of the top and bottom strip (strips 1, 7 or 2, 8 or 3, 9 respectively). The same method was used to develop a horizontal load multiplier where the middle column (strip 2, 5, or 8) was divided by the left and right column of strips. The vertical and horizontal load multipliers are indicators of the quantity of load increase due to reinforcement spacing effects.

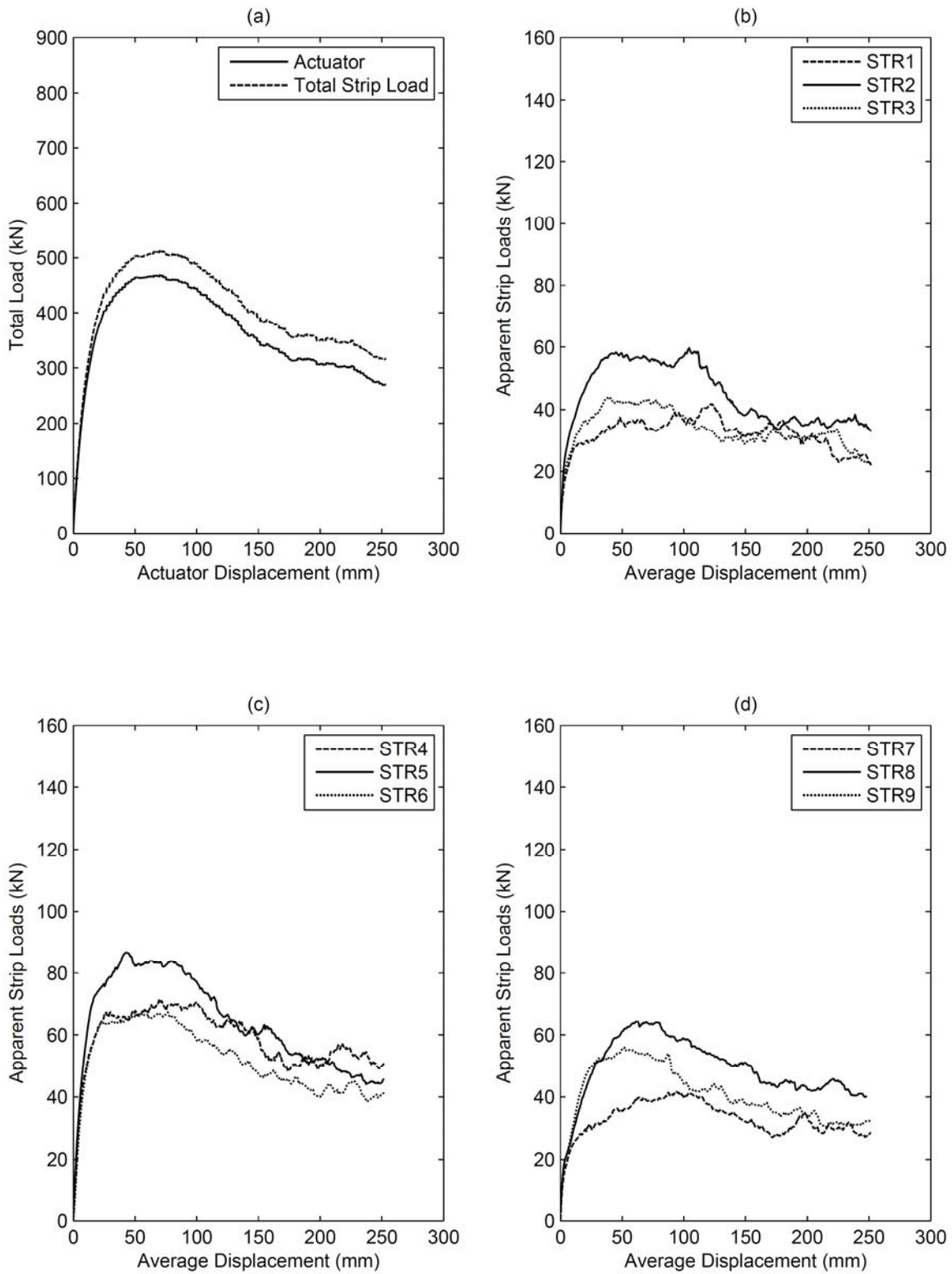


Figure 6.12. Load displacement behavior of a multi-strip pullout test with 152 mm spacing at an applied pressure of 100 kPa: (a) comparison of total loads, (b) strips 1 through, (c) strips 4 through 6, and strips 7 through 9.

Preliminary load multiplier results are presented in Figure 6.13 and suggest that the pullout resistance in metallic reinforcements is affected by adjacent reinforcement proximity. The scatter in the data is likely due to variations in moisture content during compaction and sample preparation. However, trend lines plotted in Figure 6.13 suggest that reinforcement spacing interaction effects are greatest at smaller spacing and reduce as the spacing is increased. The trend line for the vertical load multiplier is approximately 1.4 at a spacing of 152 mm and reduces to around 1.0 at 305 mm spacing. This trend may suggest that the vertical reinforcement spacing effects on peak load are negligible at reinforcement spacing greater than 305 mm, or 16 times the maximum particle size of the soil. Similarly, the trend line for the horizontal load multiplier is approximately 1.25 at a spacing of 152 mm and reduces to 1.15 at 305 mm which may suggest that horizontal reinforcement spacing effects on peak load are negligible at reinforcement spacing greater than 685 mm, or 36 times the maximum particle size. The remaining portion of the test program will help to clarify some of the trends noted thus herein.

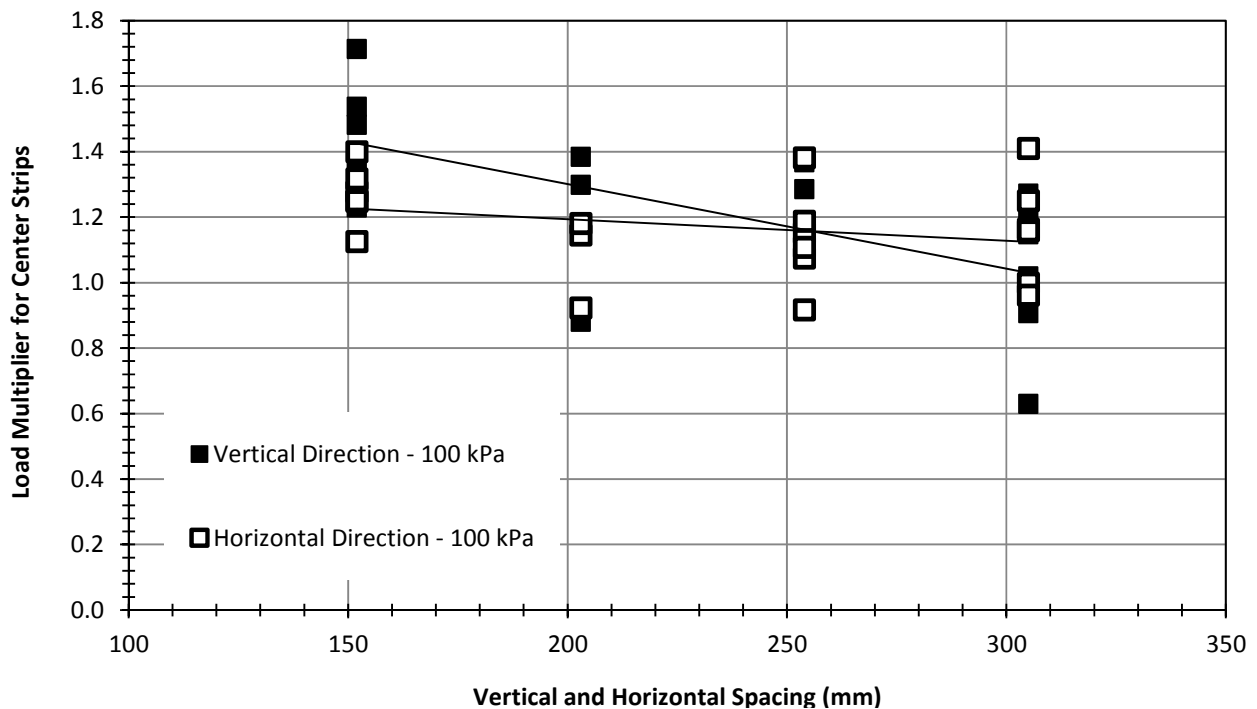


Figure 6.13. Effect of reinforcement spacing on the peak load during pullout testing at an applied pressure of 100 kPa.

6.4 Summary

This chapter presented the design and use of a large multi-strip pullout box including preliminary pullout test results on MSE wall ribbed steel strip reinforcements embedded in the sand-gravel backfill analyzed in Chapter 4. The preliminary conclusions and contributions of this chapter include:

1. A new large volume multi-strip pullout apparatus was designed and constructed for performing full scale laboratory pullout tests on closely-spaced ribbed steel strip reinforcements;
2. Initial test results indicate variability similar to the pullout response of single-strip reinforcements;
3. Preliminary test results indicate that frictional interference appear to exist in the vertical direction at vertical pressures of 100 kPa and may increase the peak pullout loads of reinforcement strips as much as 40% at a spacing of 152 mm;
4. Interaction effects appear to reduce as the vertical reinforcement spacing approaches 305 mm.
5. Interaction effects may exist in the horizontal direction but at are reduced magnitude.

Chapter 7 Conclusions

7.1 Summary of Research Investigation

The main objectives of this thesis were to characterize the constitutive behavior of a sandy gravel backfill material used in two very tall MSE walls, to develop a better understanding of the soil-reinforcement interaction between the sandy gravel backfill and the ribbed steel strip reinforcements, and to evaluate the possible interaction of closely-spaced reinforcements on pullout resistance. Laboratory tests were performed on the backfill and reinforcement materials separately to determine their constitutive properties. Full-scale laboratory pullout tests were performed and the results used to produce a backfill-specific design models that can be used to predict peak reinforcement pullout resistance. Full-scale laboratory pullout tests were conducted on multiple reinforcement strips to assess the possibility of frictional interference of closely-spaced reinforcements.

7.2 Conclusions

7.2.1 Laboratory Test Program

The following summarizes the findings of the laboratory testing program:

- Tension testing was performed on coupons cut from the ribbed steel strip reinforcements and the following representative constitutive properties determined: yield strength, $f_y = 526$ MPa; and, Young's modulus, $E = 208$ GPa
- Consolidated drained axisymmetric triaxial strength tests were performed on the reinforced backfill material compacted to 35, 55 and 65 percent relative density. The friction angles of the material at a confining pressure of one atmosphere were found equal to 47.9, 45.3 and 40.0 degrees, respectively. The reduction in peak friction angle with a log-cycle increase in confining pressure was equal to 2.1, 4.7, and 6.8 degrees for relative densities of 35, 55, and 65 percent, respectively.

7.2.2 Single-Strip Pullout Test Program

The following summarizes the findings of the single-strip pullout testing program:

- A large, single strip pullout test apparatus was designed and constructed in order to evaluate the soil-reinforcement interaction of ribbed steel strip reinforcements.
- Twenty pullout tests were performed at effective vertical stresses ranging from 10 to 300 kPa resulting in peak reinforcement loads ranging from 18.6 to 93.8 kN.
- Multi-linear regression was used to quantify the combined effect of vertical effective stress and compaction on the maximum pullout resistance. Although both variables control the peak pullout resistance, the effect of dry unit weight was found to be statistically insignificant, likely due to the small range in dry unit weight evaluated and

experimental error. A regression model requiring the use of the vertical effective stress as the single independent variable was recommended for use until further tests could be obtained.

- New backfill-specific and gravel models for the prediction of the apparent friction coefficient with normal effective stress were developed for use with ribbed steel strip reinforcements.
- The performance of the proposed f^* design models were compared to the current AASHTO design model and found to produce significantly more accurate predictions of the apparent friction coefficient.

7.2.3 Multi-Strip Pullout Test Program

The following summarizes the *preliminary* findings of the multiple-strip pullout testing program:

- A new large volume multi-strip pullout apparatus was designed and constructed for performing full scale laboratory pullout tests on closely-spaced ribbed steel strip reinforcements.
- Initial test results indicate variability similar to the pullout response of single-strip reinforcements.
- Preliminary test results indicate that frictional interference appear to exist in the vertical direction at vertical pressures of 100 kPa and may increase the peak pullout loads of reinforcement strips as much as 40% at a spacing of 152 mm.
- Interaction effects appear to reduce as the vertical reinforcement spacing approaches 305 mm.
- Interaction effects may exist in the horizontal direction but at are reduced magnitude.

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Appendix A Principal Stress Difference-Axial Strain Curves

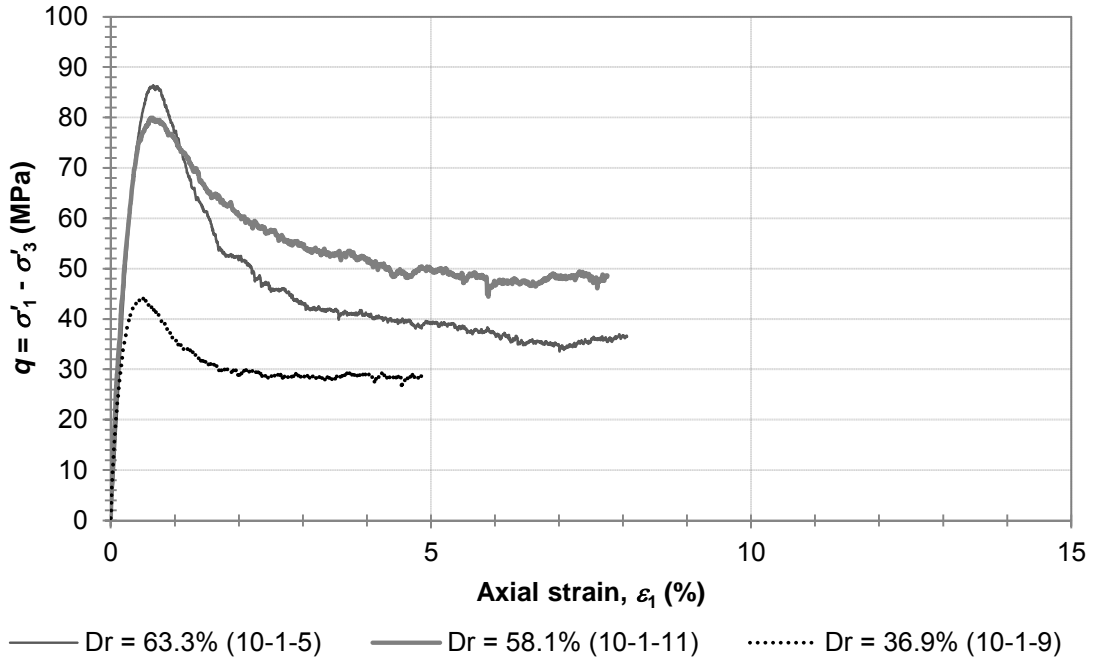


Figure A.1 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 10 kPa.

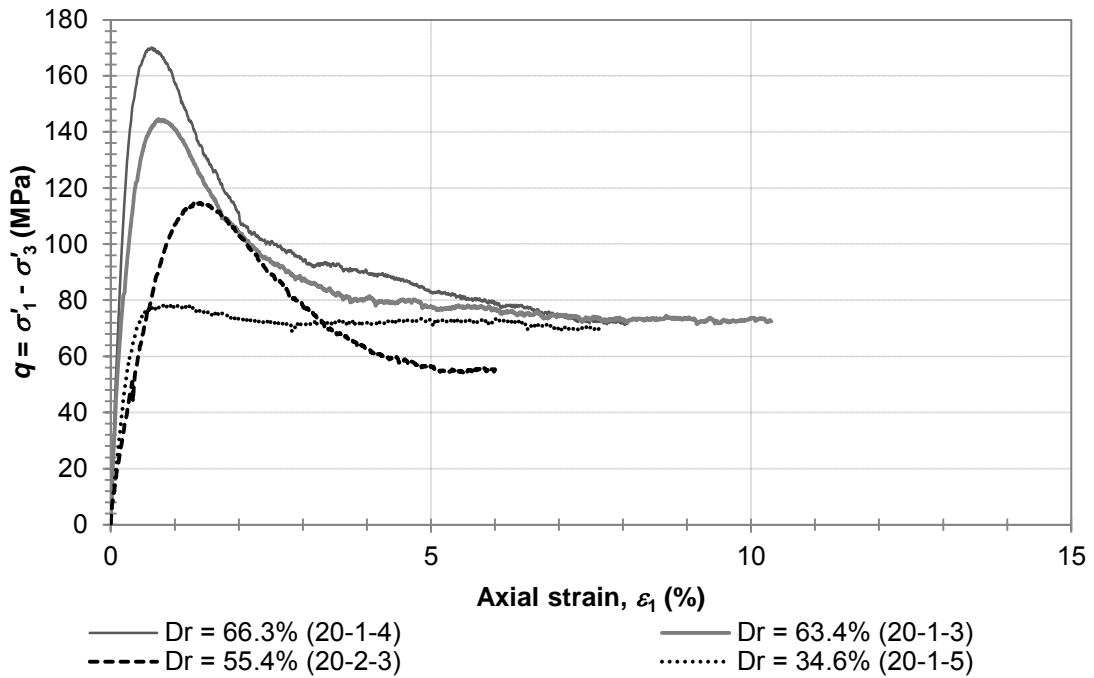


Figure A.2 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 20 kPa.

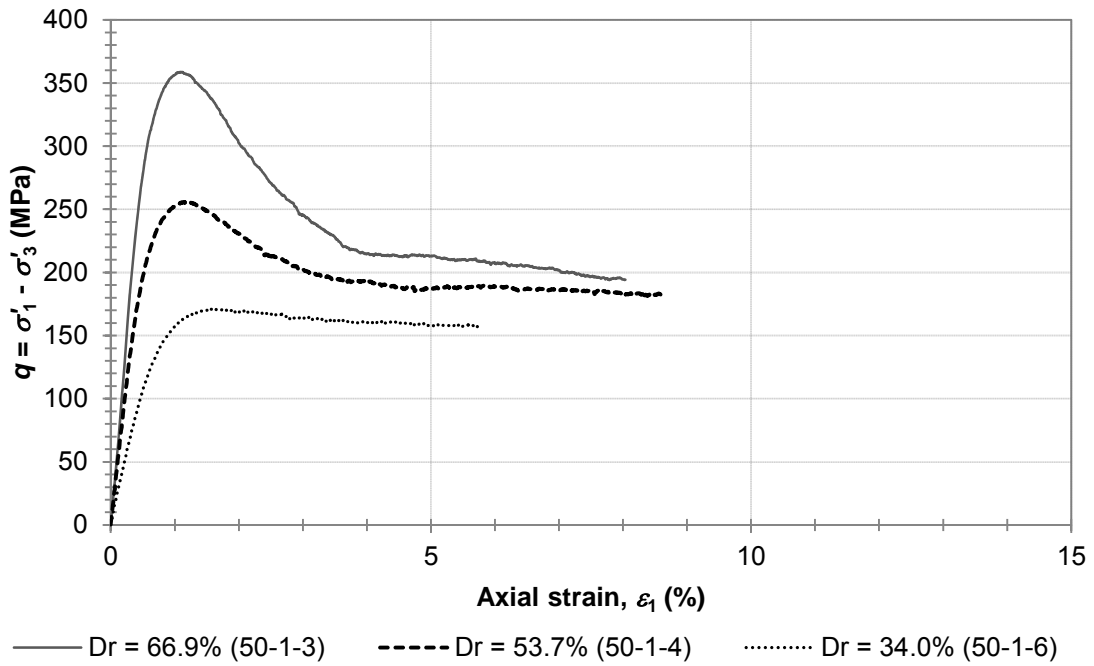


Figure A.3 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 50 kPa.

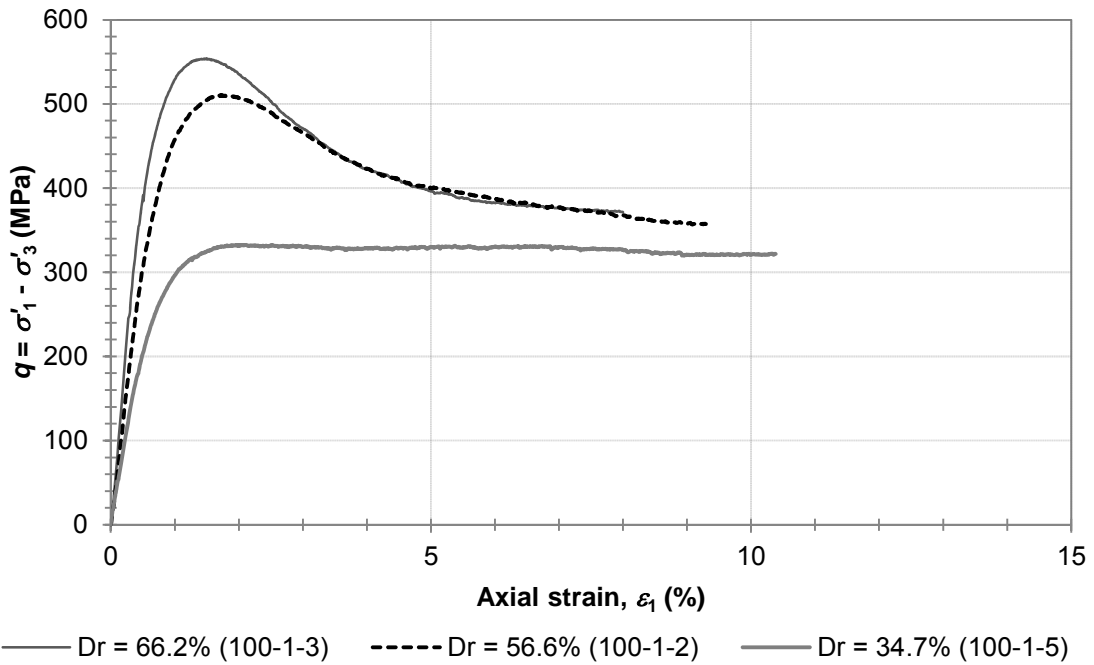


Figure A.4 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 100 kPa.

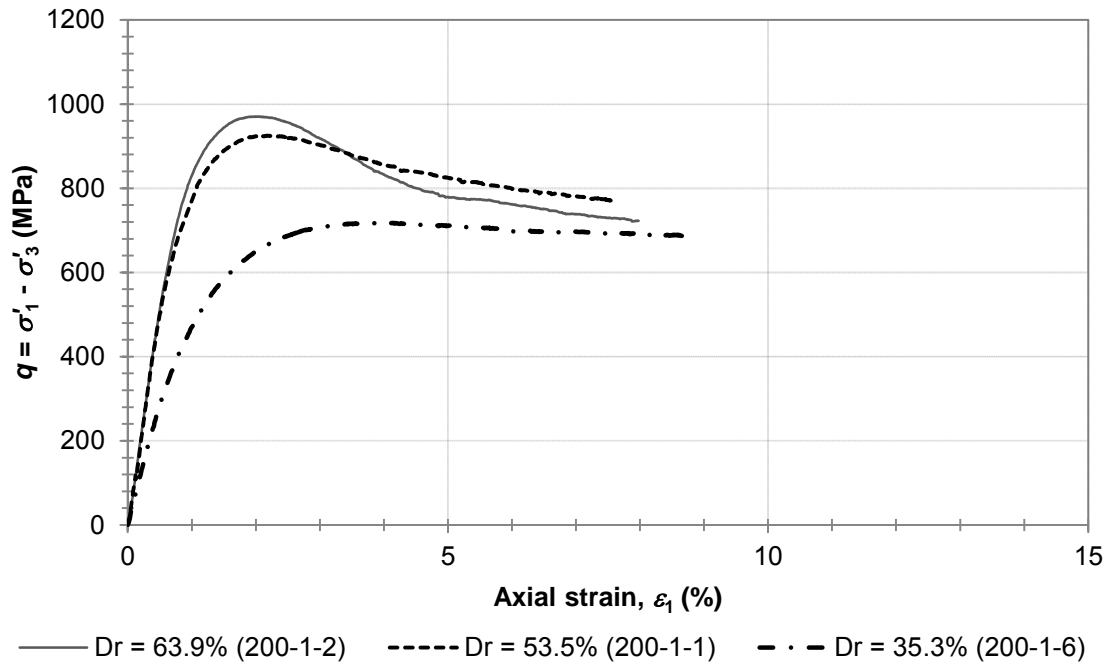


Figure A.5 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 200 kPa.

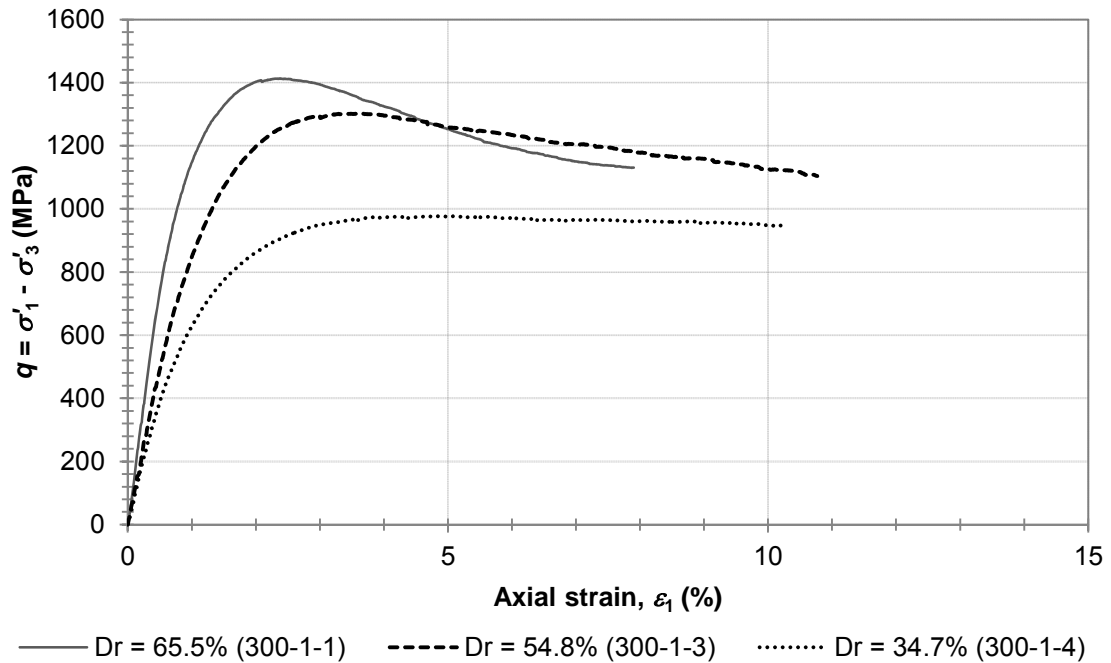


Figure A.6 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 300 kPa.

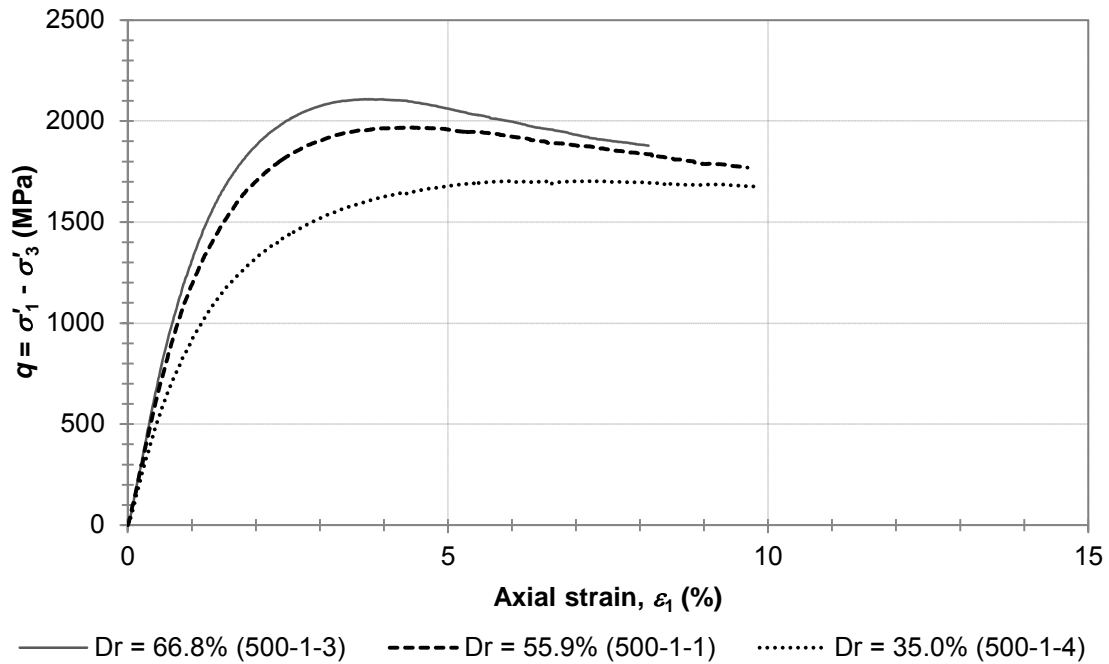


Figure A.7 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 500 kPa.

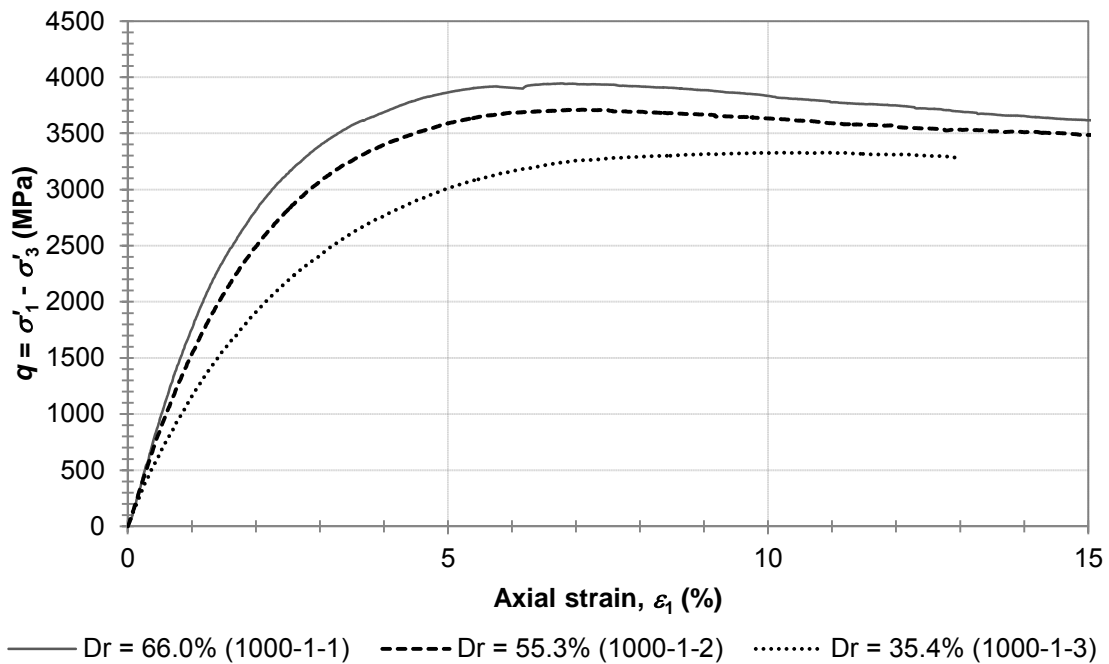


Figure A.8 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 1000 kPa.

Appendix B Volumetric Strain-Axial Strain Curves

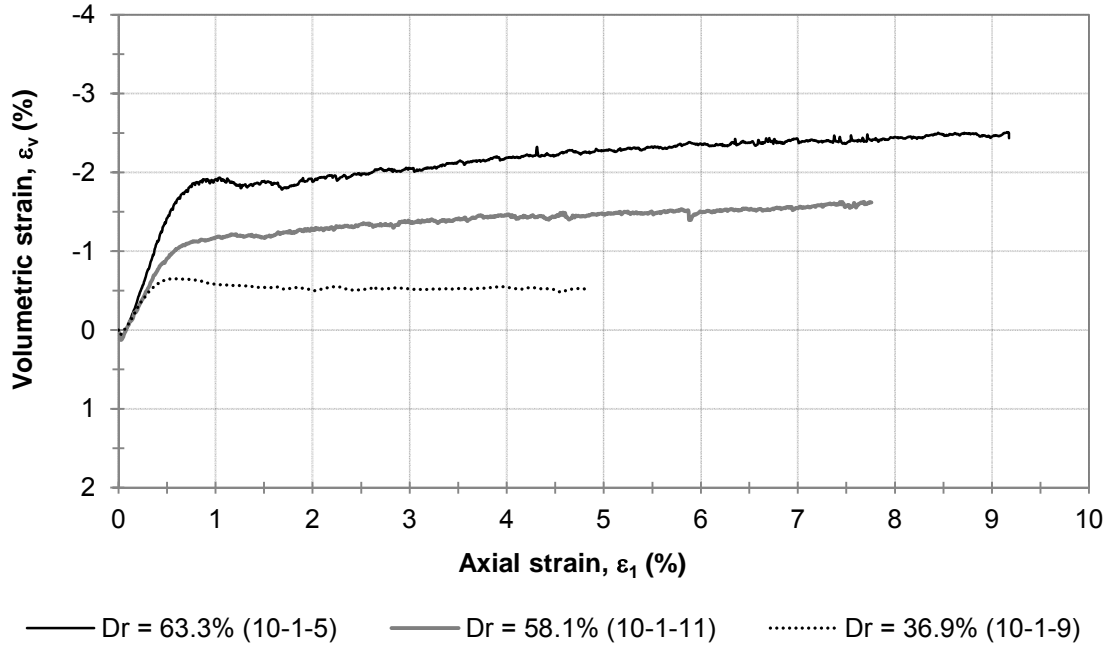


Figure B.1 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 10 kPa.

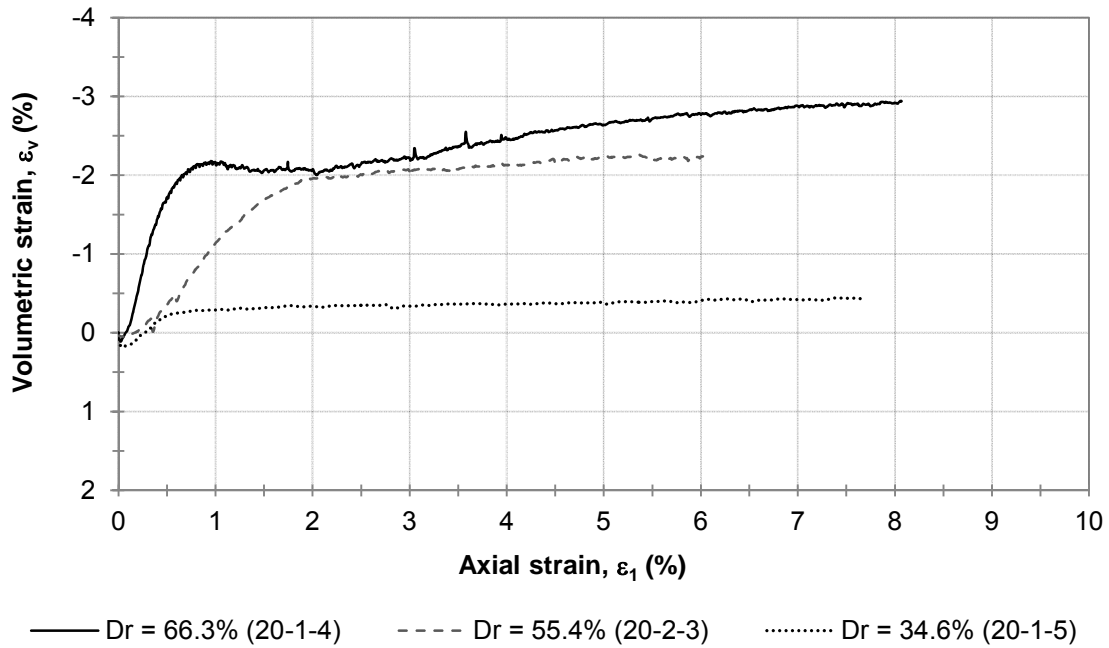


Figure B.2 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 20 kPa.

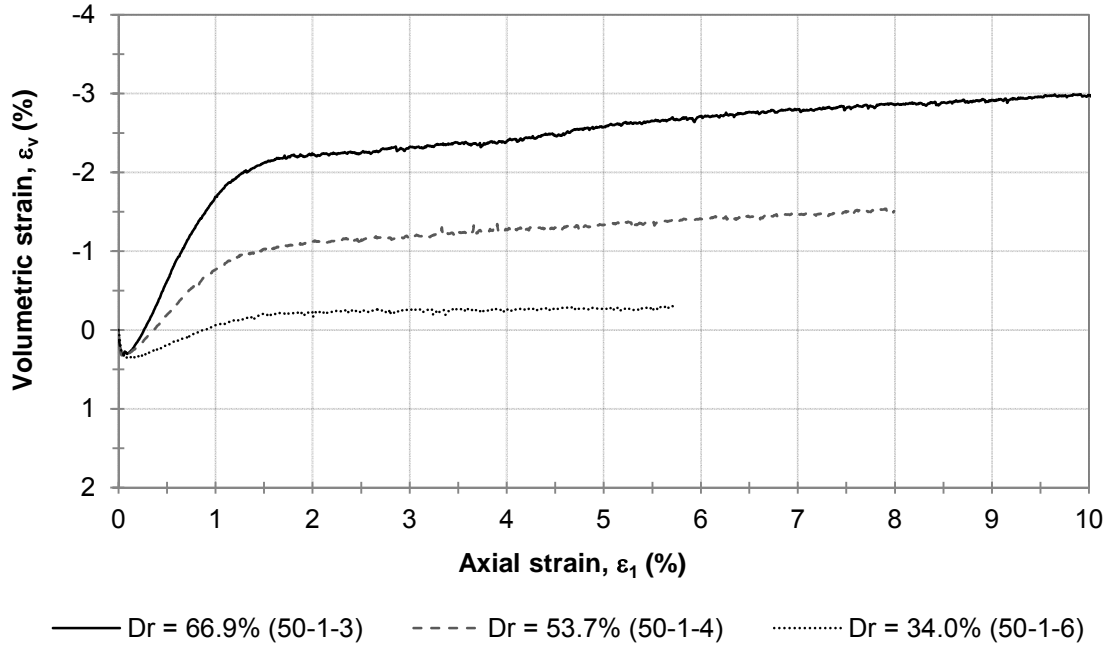


Figure B.3 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 50 kPa.

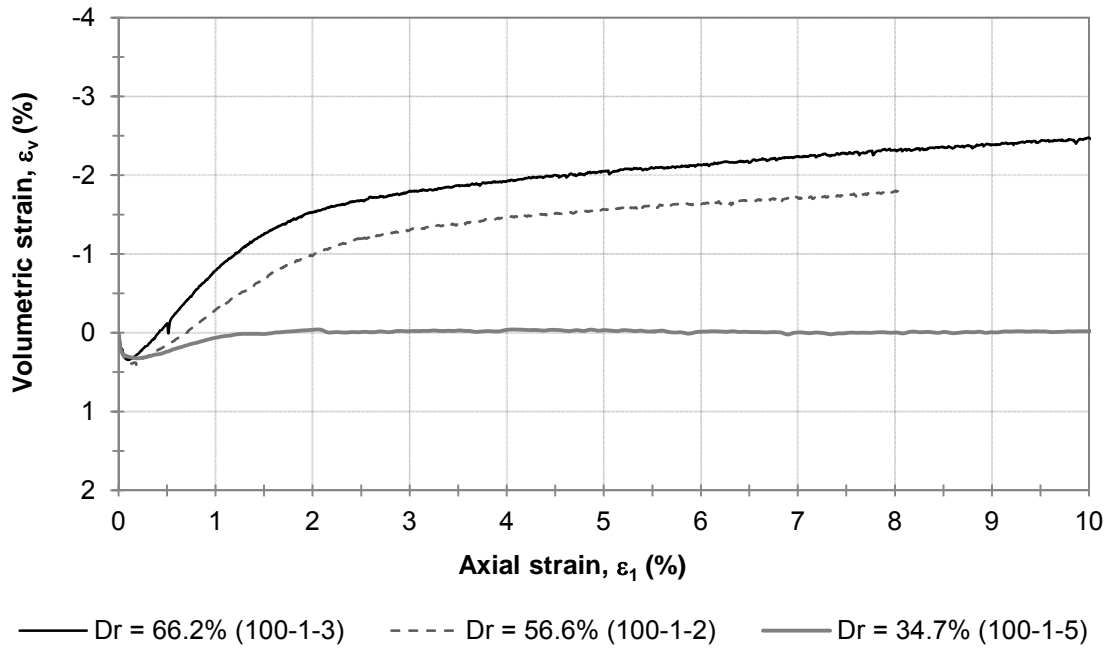


Figure B.4 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 100 kPa.

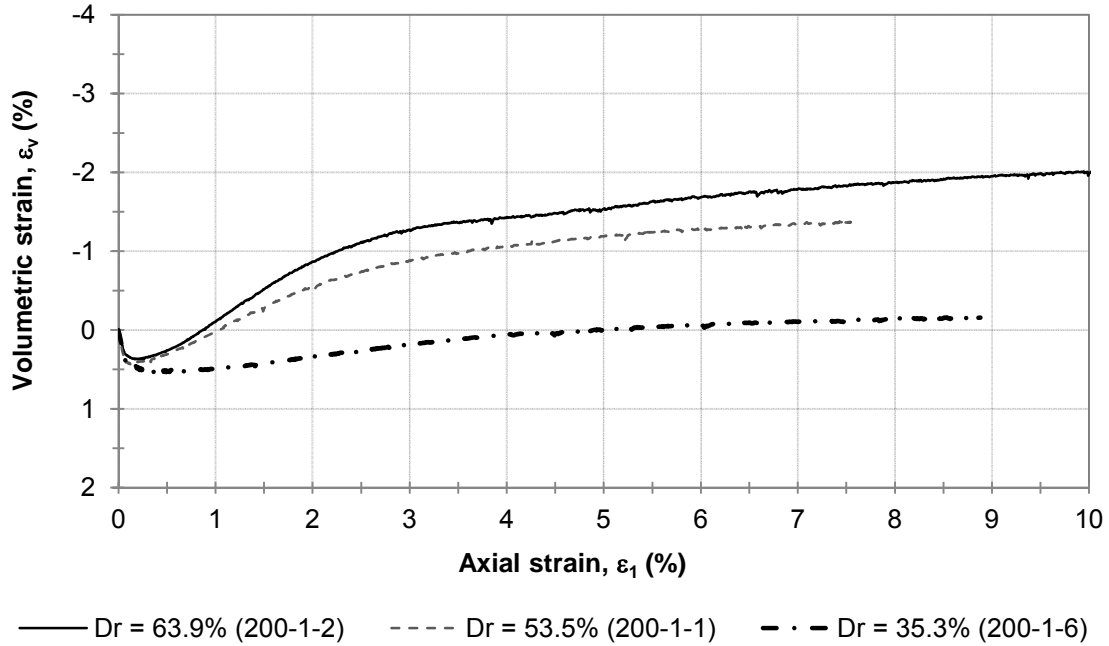


Figure B.5 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 200 kPa.

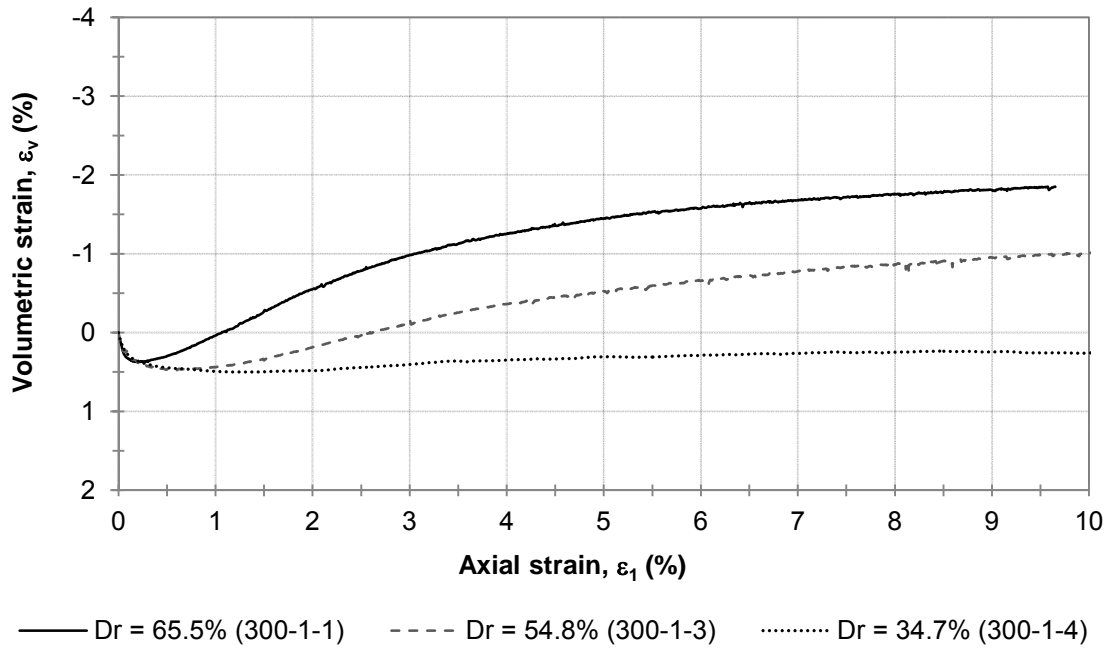


Figure B.6 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 300 kPa.

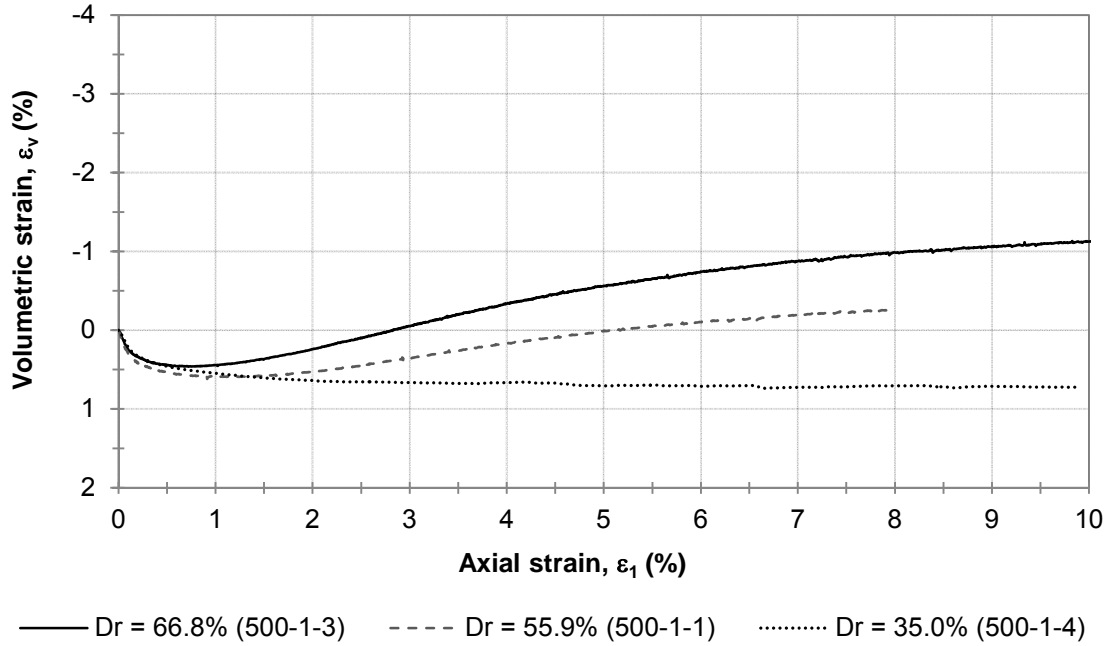


Figure B.7 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 500 kPa.

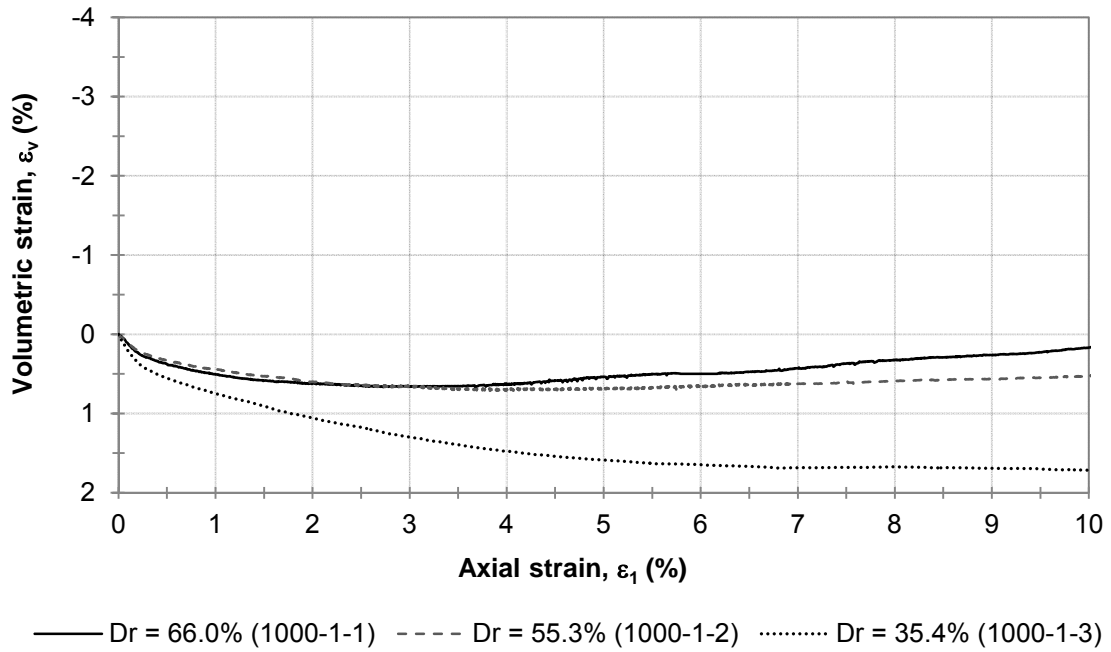


Figure B.8 Effective principal stress difference versus axial strain for tests at an effective confining pressure of 1000 kPa.

Appendix C Single-Strip Pullout Box Design Schematics

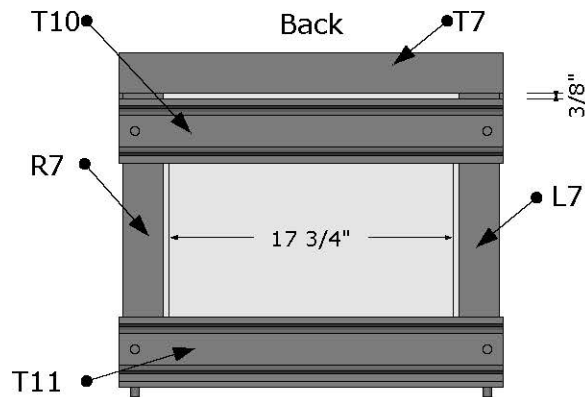
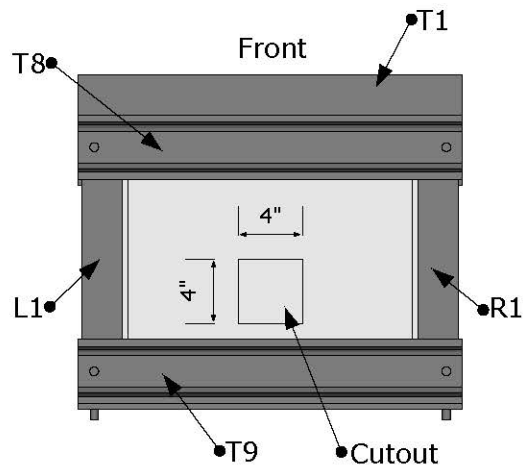
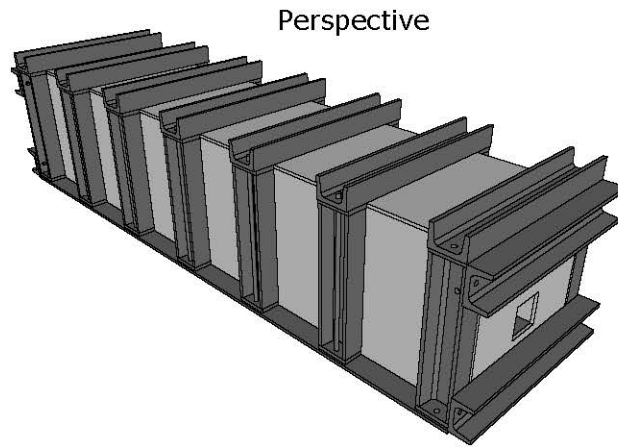


Figure C.1 Front, back, and perspective views of assembled pullout box.

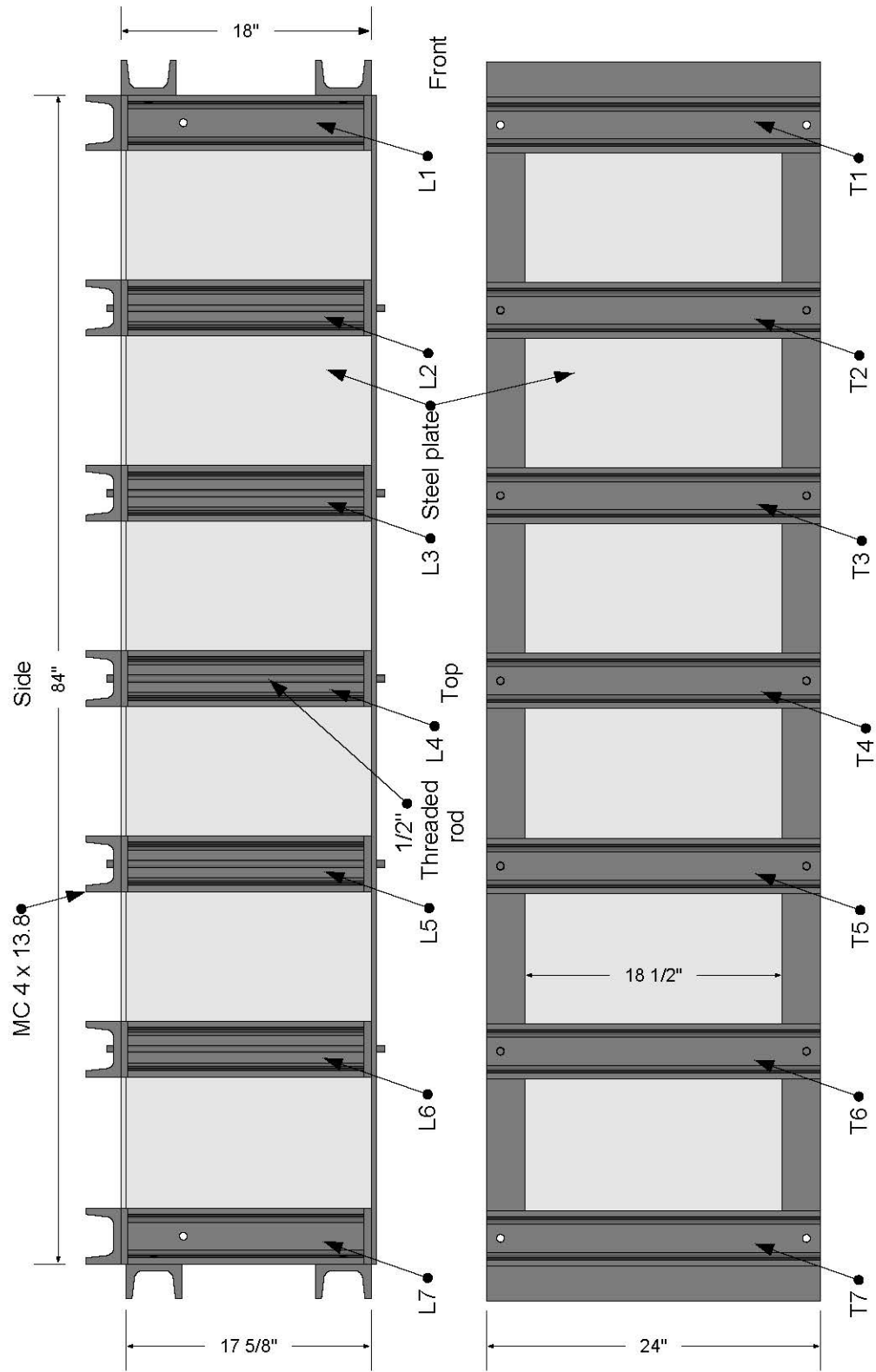
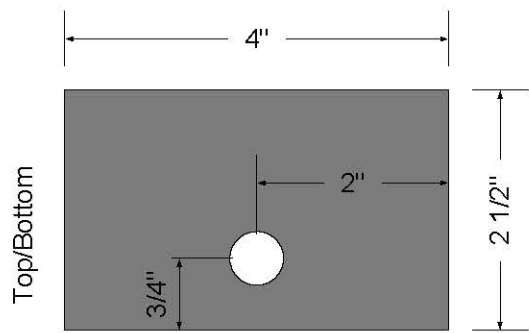
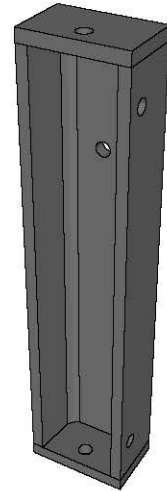


Figure C.2 Side and top view of assembled pullout box.



Perspective



Note: All Holes are 9/16" Dia. unless noted otherwise.

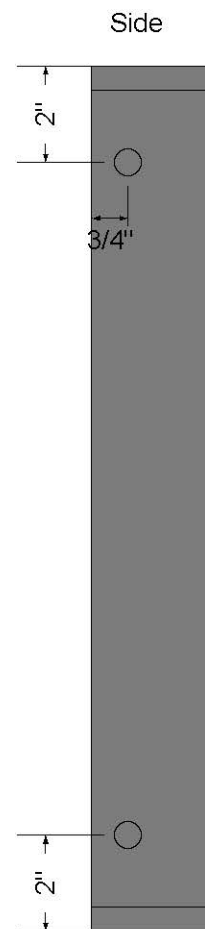
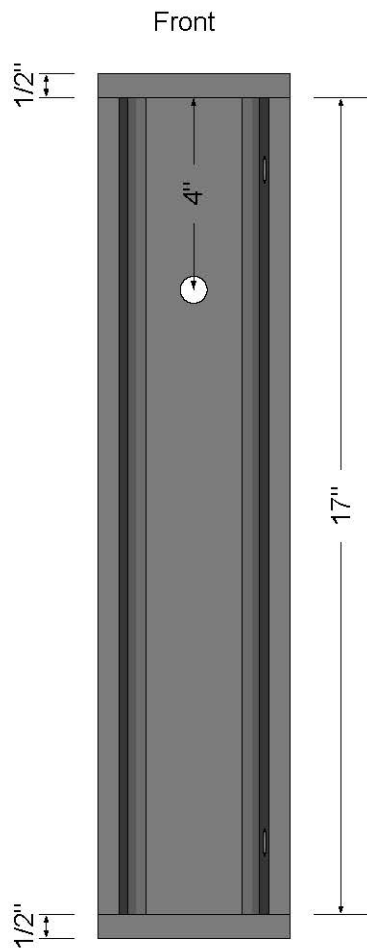
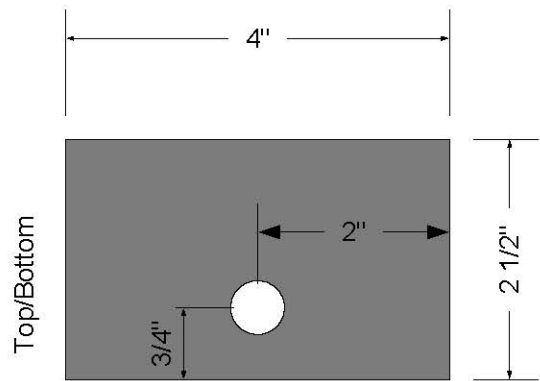
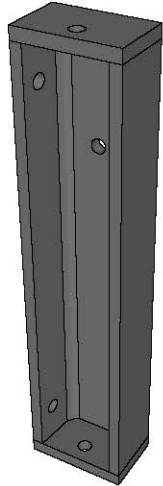
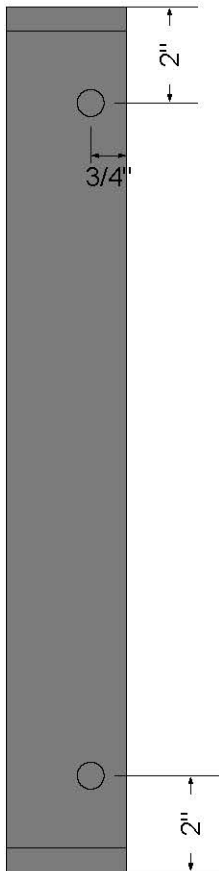


Figure C.3 Schematic of pullout box part L1, quantity = 1.

Perspective



Side



Front

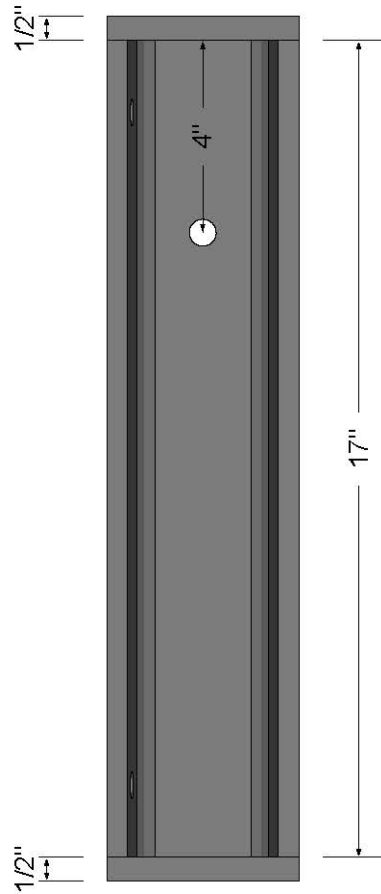


Figure C.4 Schematic of pullout box part R1, quantity = 1.

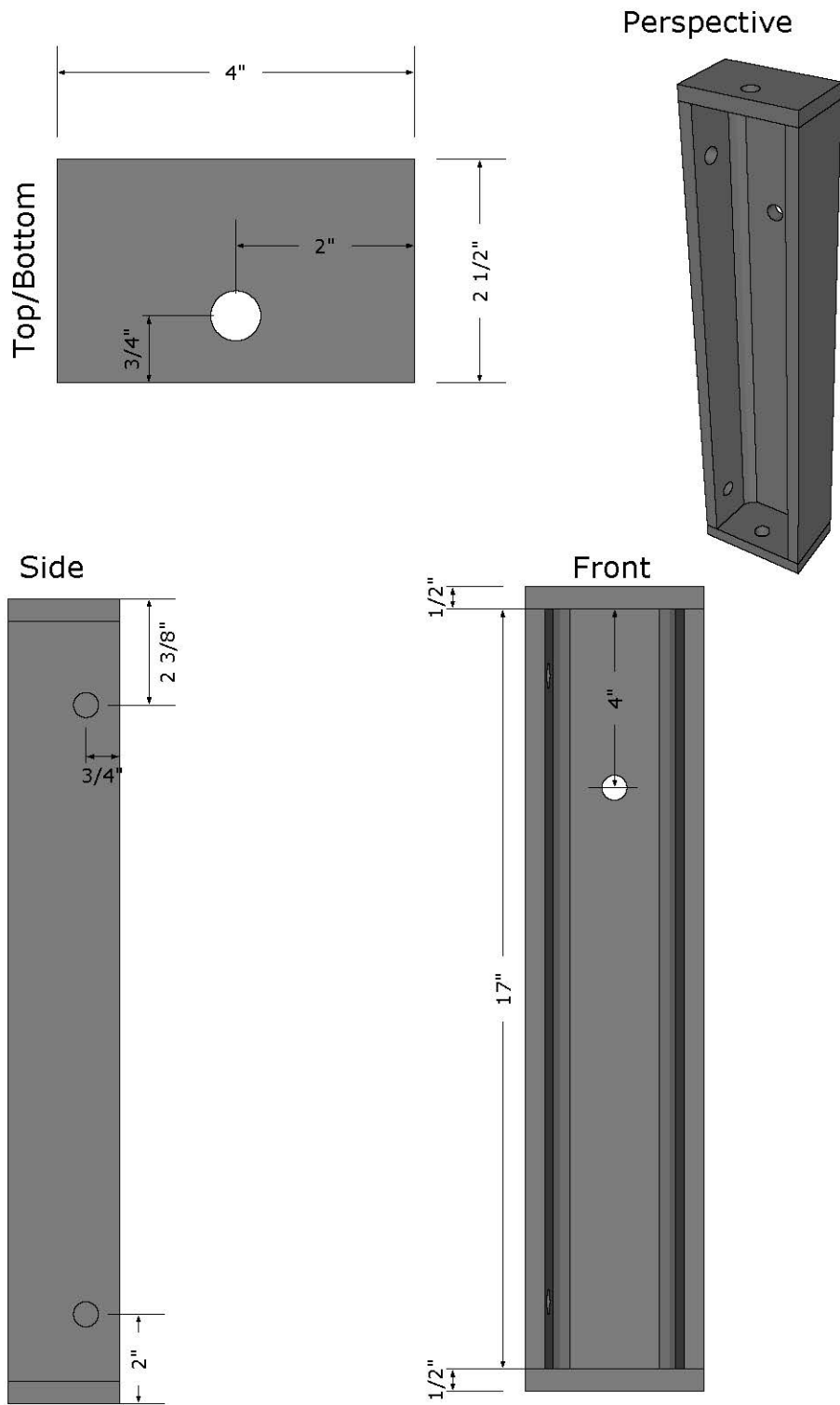


Figure C.5 Schematic of pullout box part L7, quantity = 1.

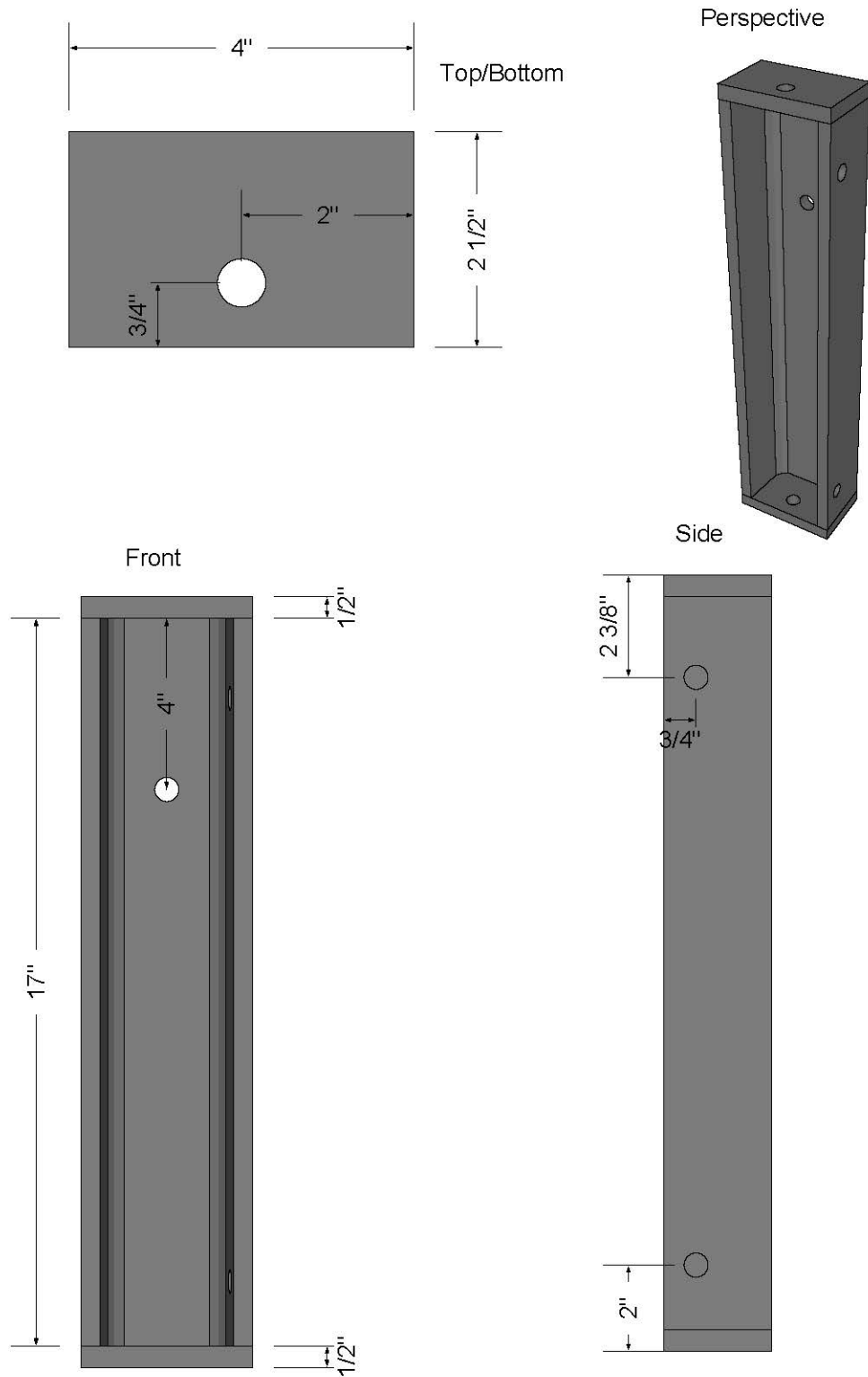


Figure C.6 Schematic of pullout box part R7, quantity = 1.

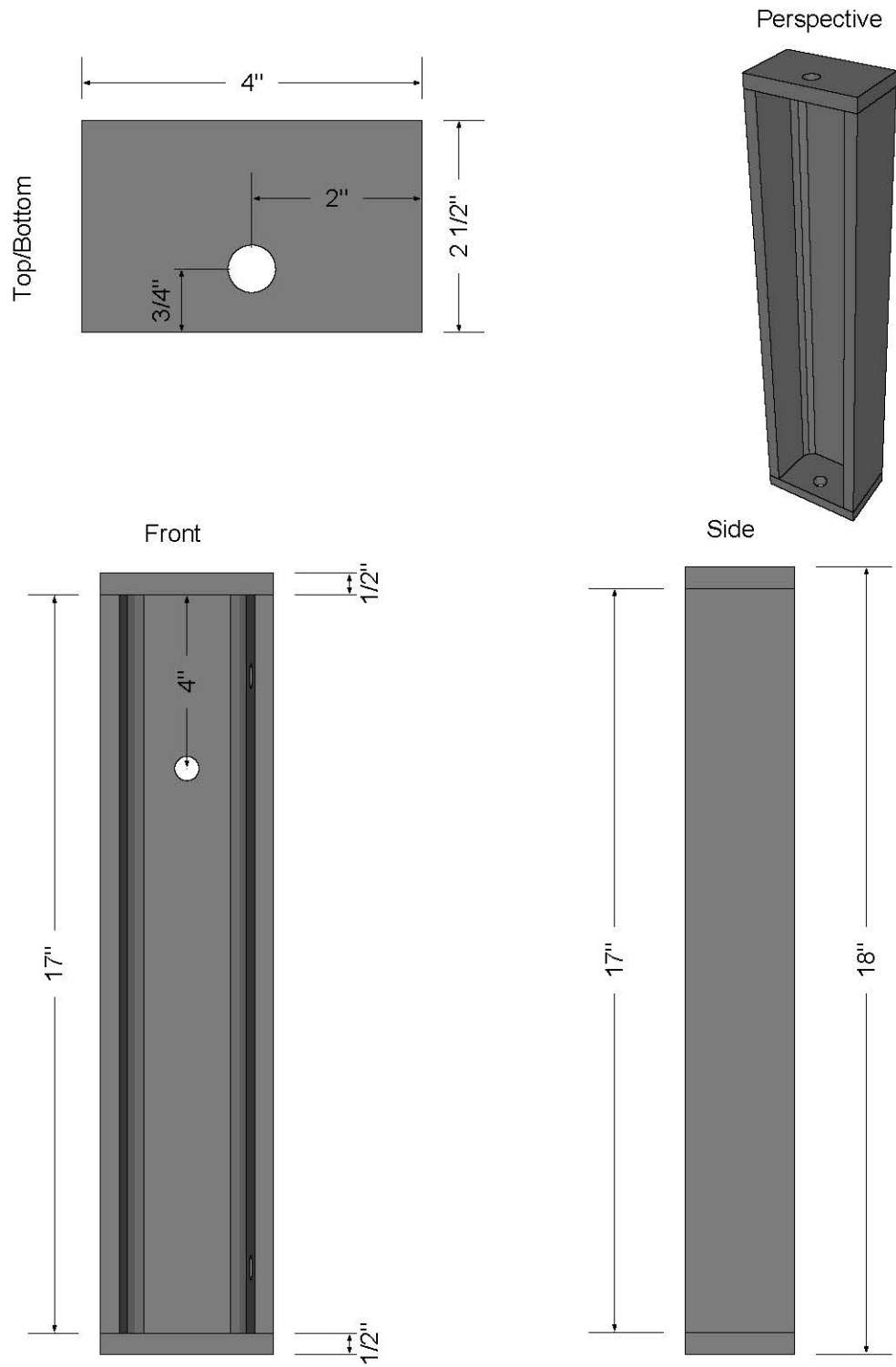


Figure C.7 Schematic of pullout box part R2-R6 and L2-L6, quantity = 10.

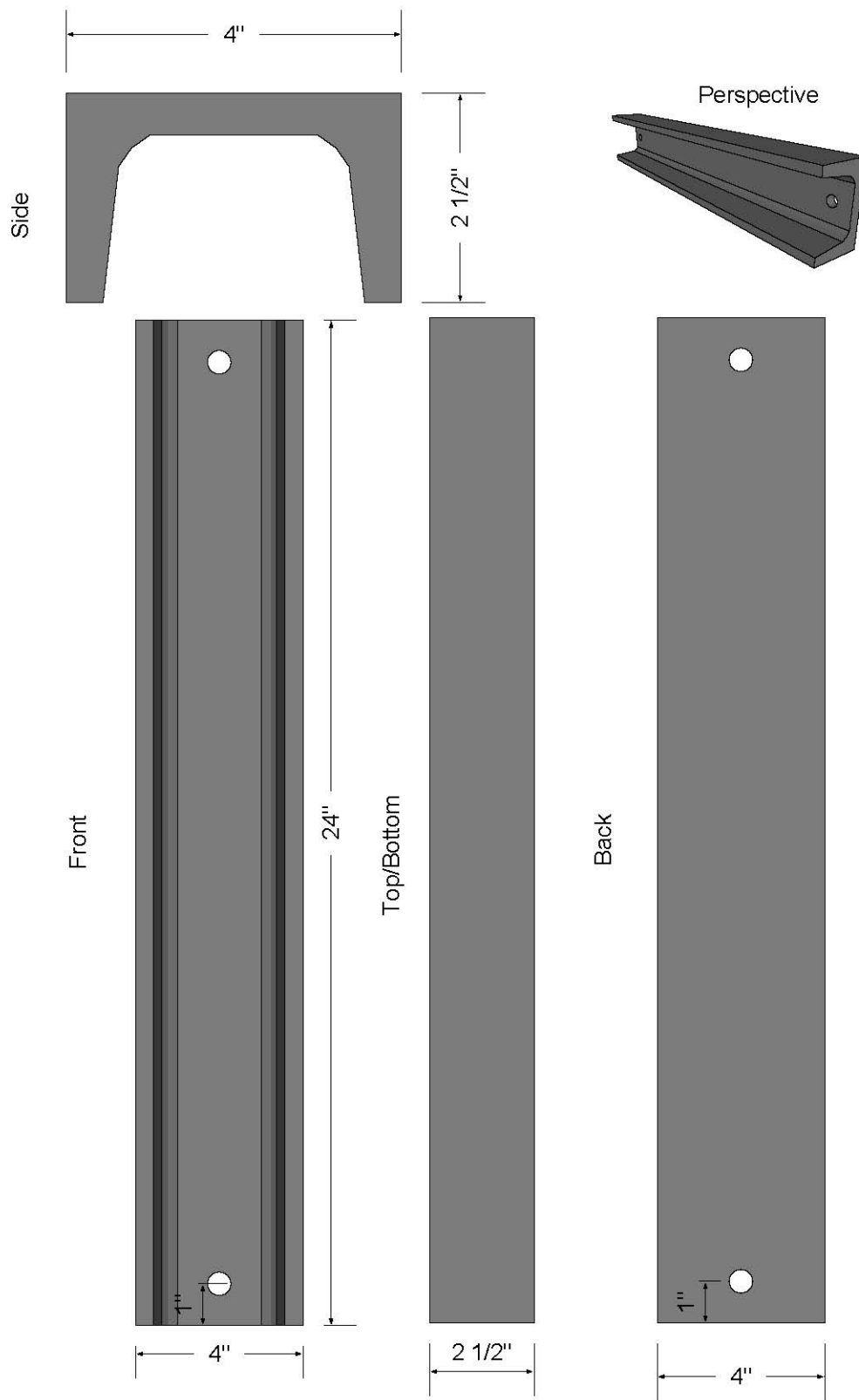


Figure C.8 Schematic of pullout box part T1 - T11, quantity = 11.

Note: All Plates are 3/8" thick

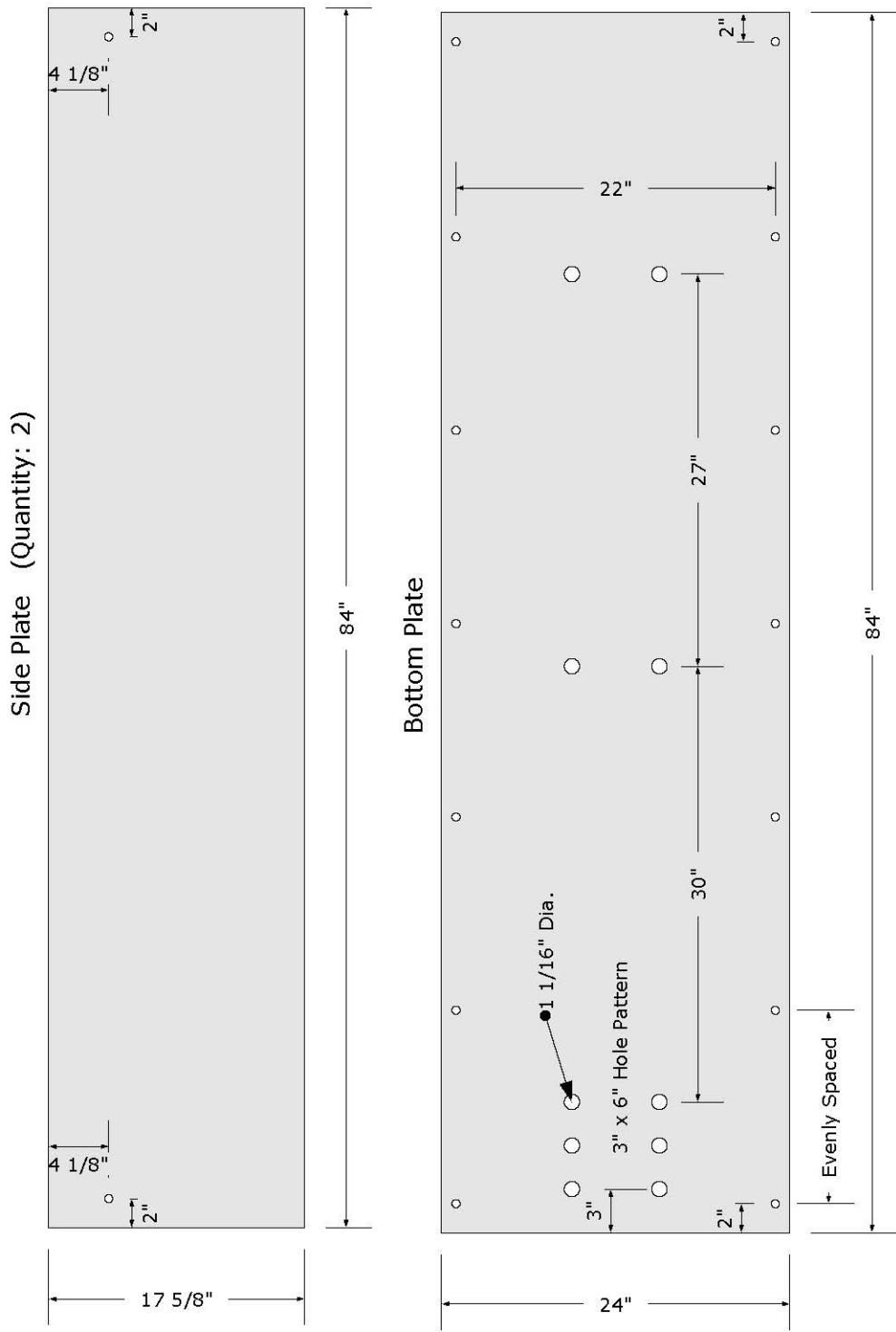


Figure C.9 Schematic of side plate and bottom plate.

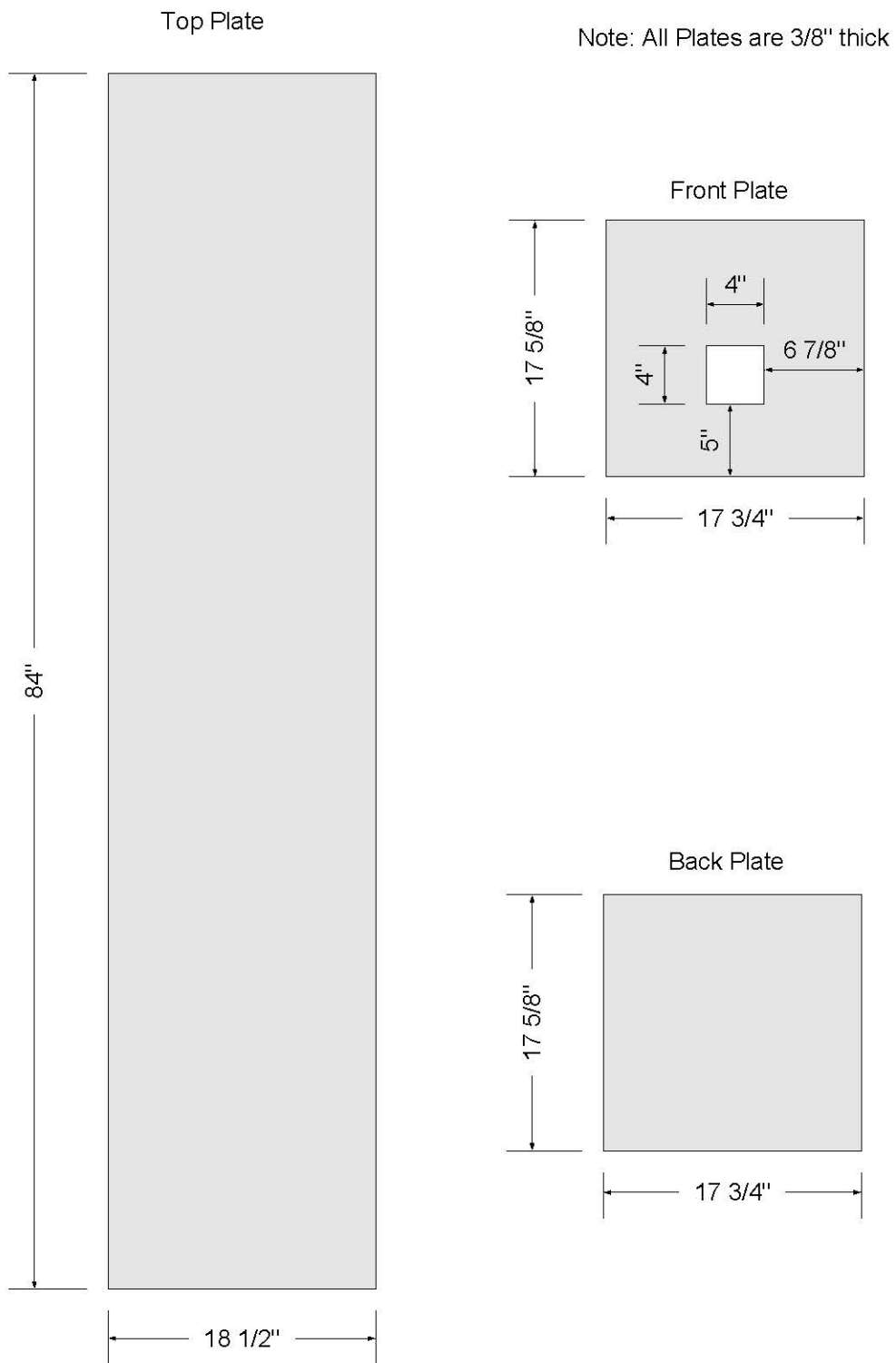
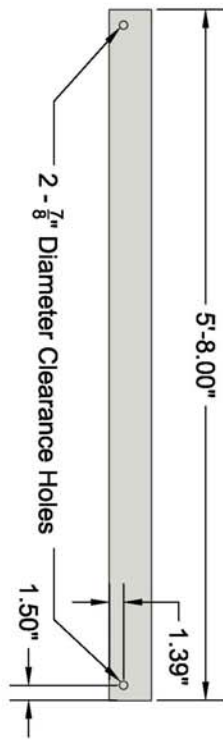
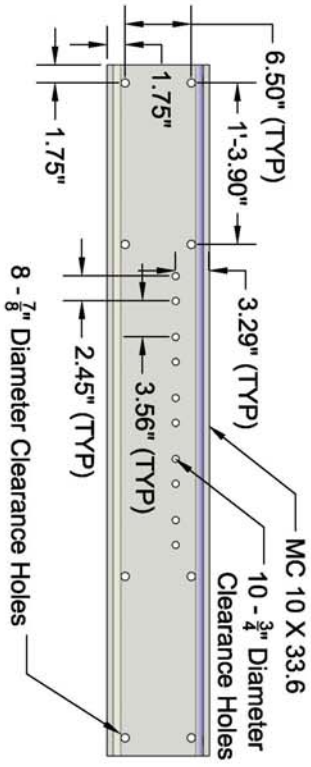
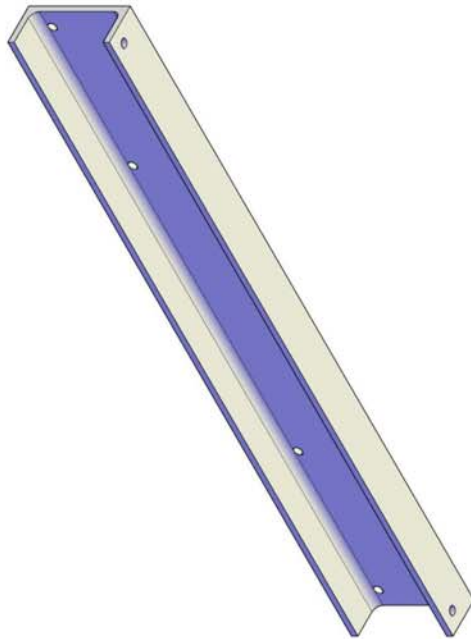


Figure C.10 Schematic of pullout box top, front, and back plates.

Appendix D Multi-Strip Pullout Box Design Schematics

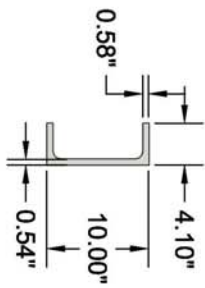


TOP VIEW



FRONT VIEW

SIDE VIEW



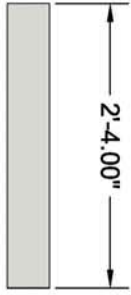
Part ID: Main Face Channel

Drawn by: AWS

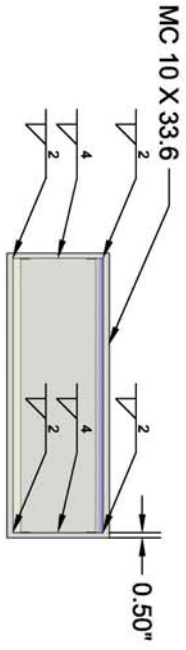
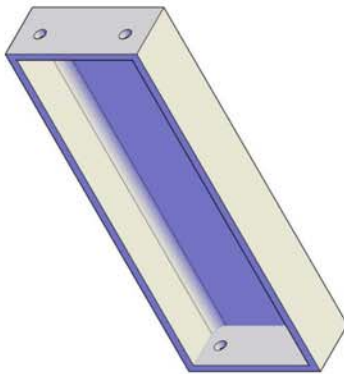
Project: Multiple Pull-out Box

Date: 11/18/12

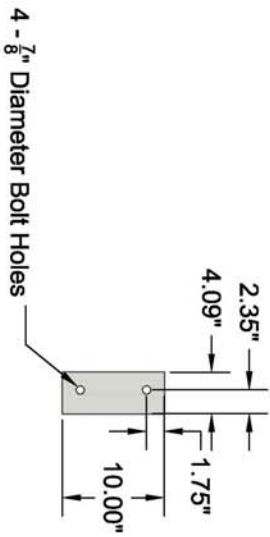
Quantity: 2



TOP VIEW



FRONT VIEW



SIDE VIEW

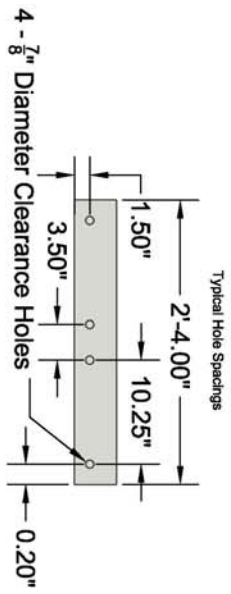
Part ID: Support Face Channel1

Drawn by: AWS

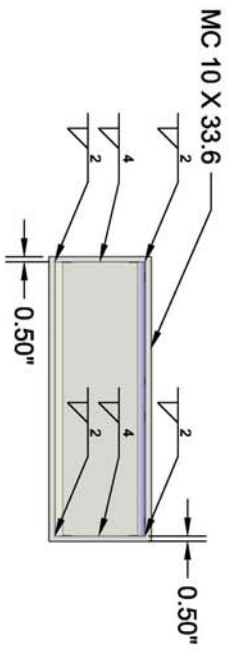
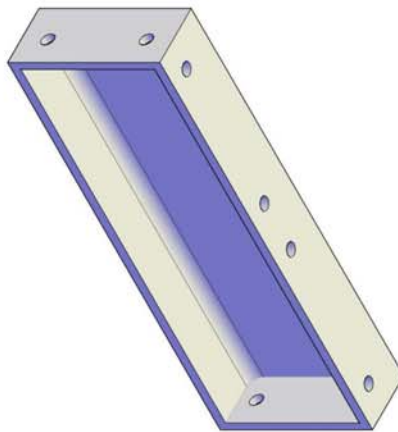
Project: Multiple Pull-out Box

Date: 11/18/12

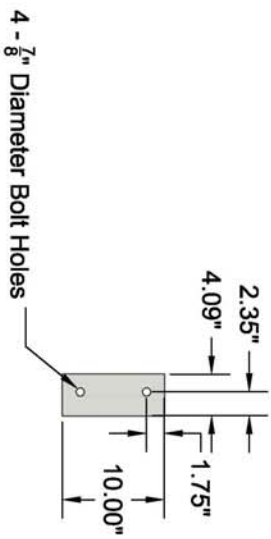
Quantity: 2



TOP VIEW



FRONT VIEW



SIDE VIEW

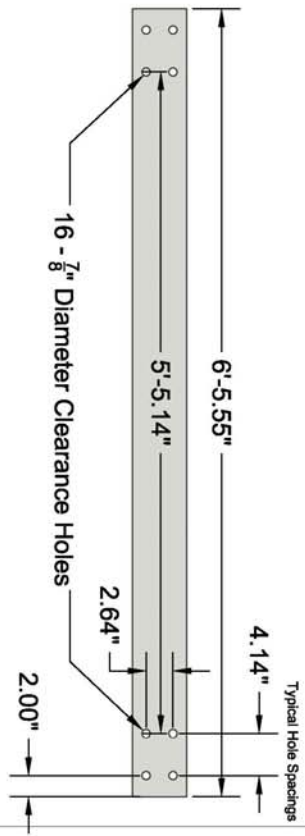
Part ID: Support Face Channel2

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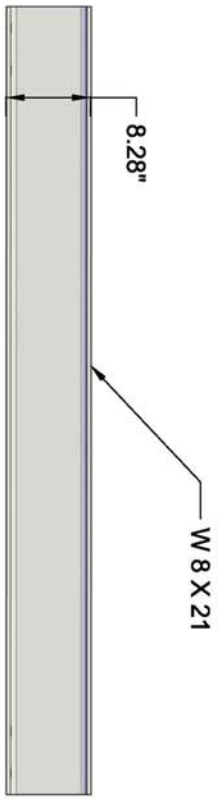
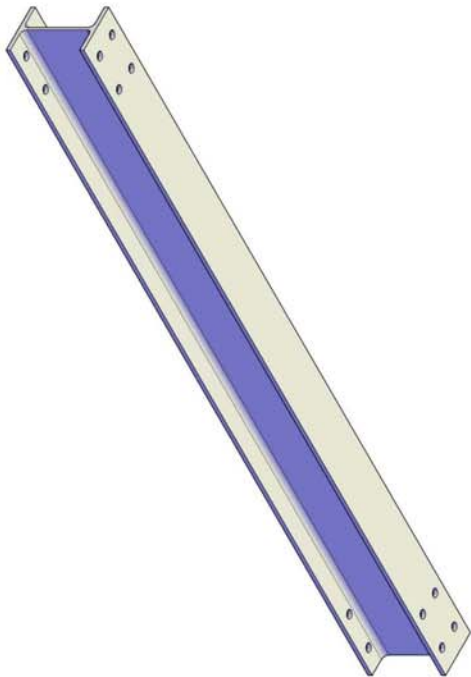
Project: Multiple Pull-out Box

Date: 11/18/12

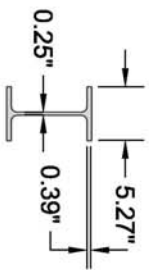
Quantity: 2



TOP VIEW



FRONT VIEW



SIDE VIEW

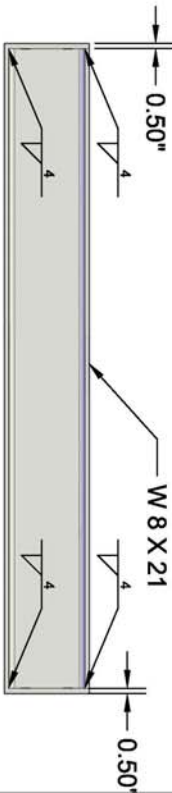
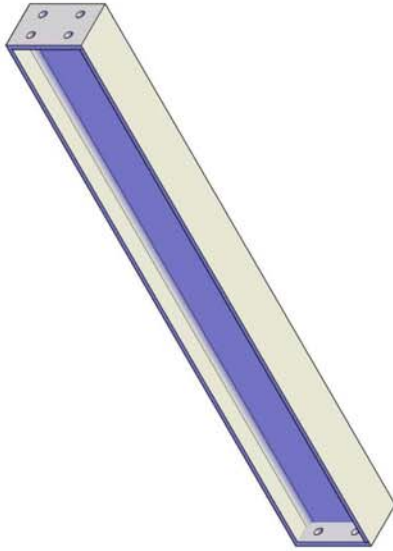
Part ID: Top Support Beam

Drawn by: AWS
Project: Multiple Pull-out Box

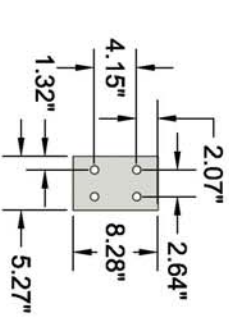
Date: 11/18/12
Quantity: 8



TOP VIEW



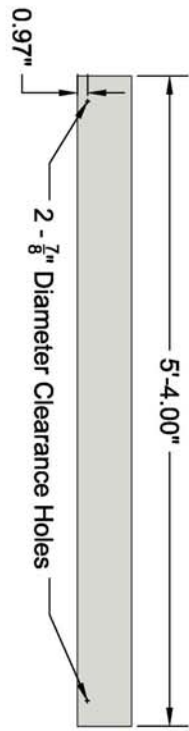
FRONT VIEW



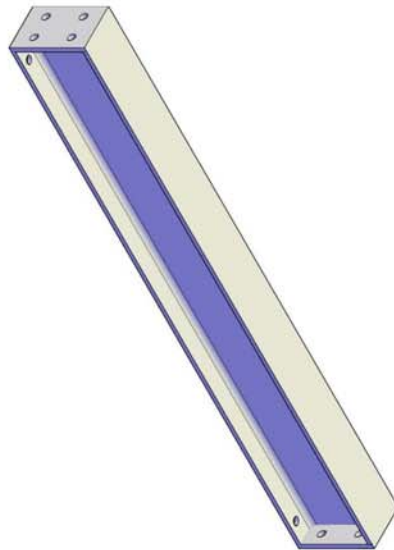
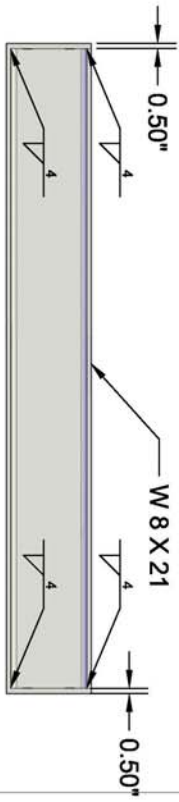
SIDE VIEW

Part ID: Side Support Beam	Drawn by: AWS	Date: 11/18/12
	Project: Multiple Pull-out Box	Quantity: 4

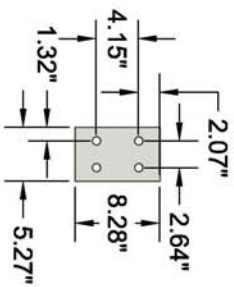
TOP VIEW



FRONT VIEW



SIDE VIEW



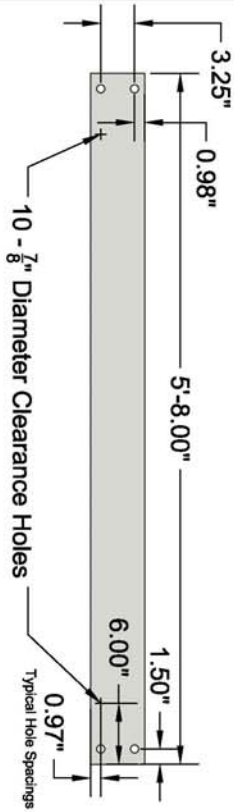
Part ID: Side Support Beam

Drawn by: AWS

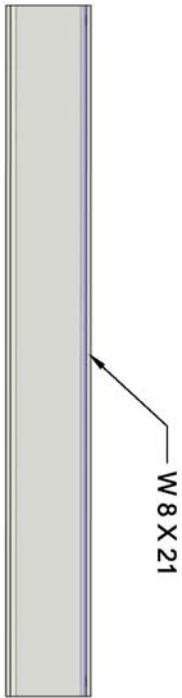
Project: Multiple Pull-out Box

Date: 11/18/12

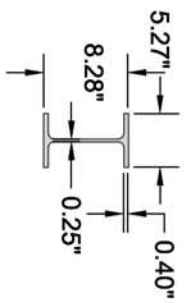
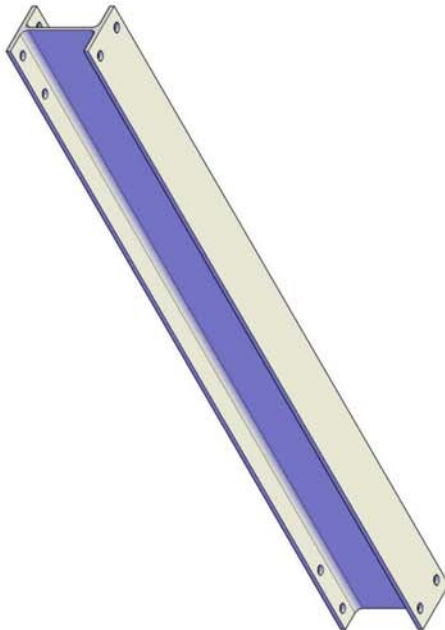
Quantity: 4



TOP VIEW



FRONT VIEW

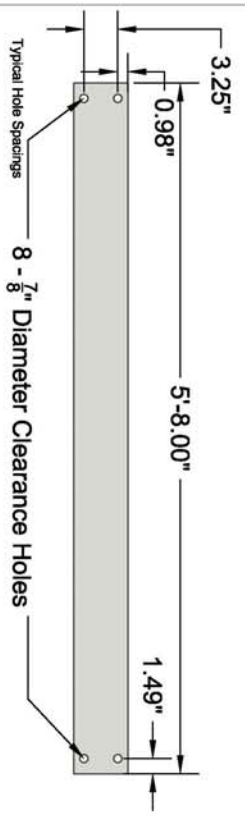


SIDE VIEW

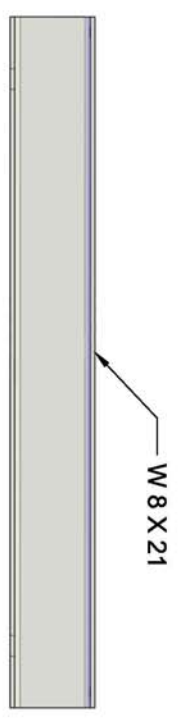
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Drawn by: AWS
Project: Multiple Pull-out Box

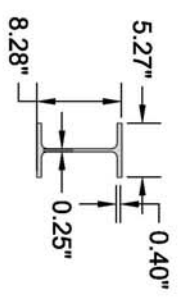
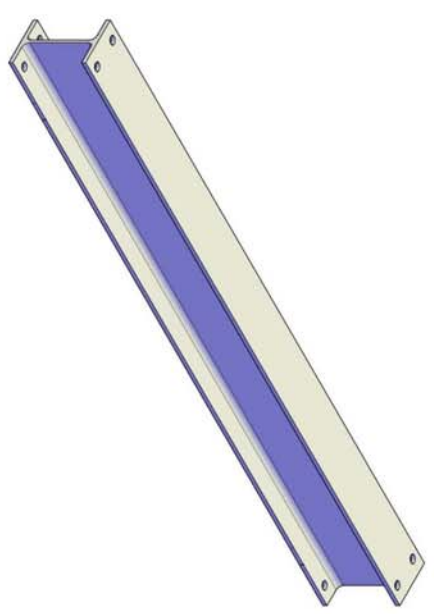
Date: 11/18/12
Quantity: 4



TOP VIEW



FRONT VIEW

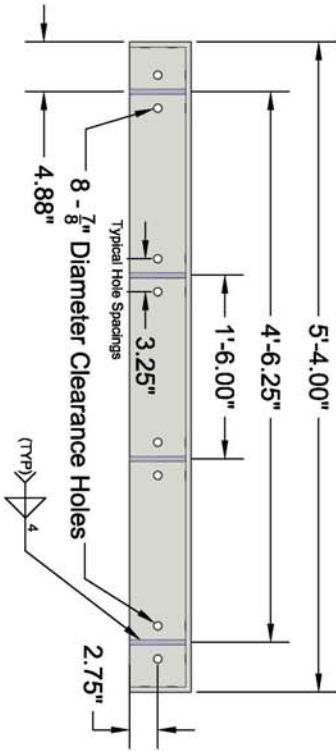


SIDE VIEW

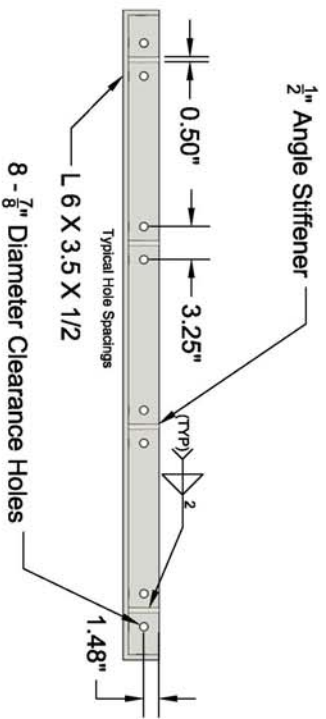
Part ID: Rear Support Beam

Drawn by: AWS
Project: Multiple Pull-out Box

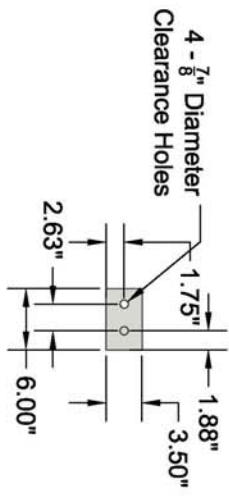
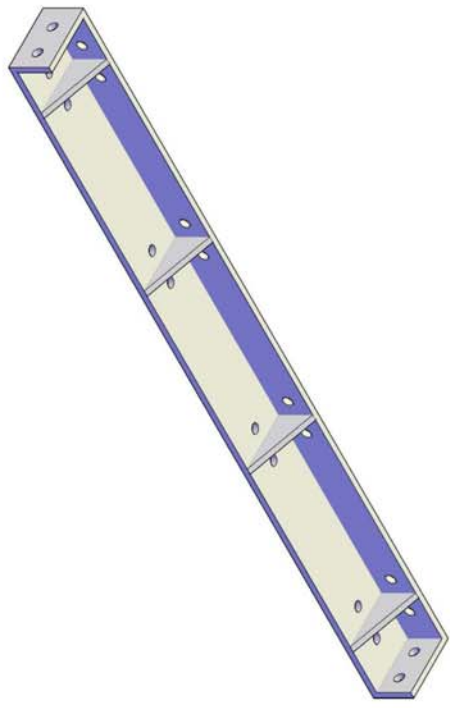
Date: 11/18/12
Quantity: 2



TOP VIEW



FRONT VIEW



SIDE VIEW

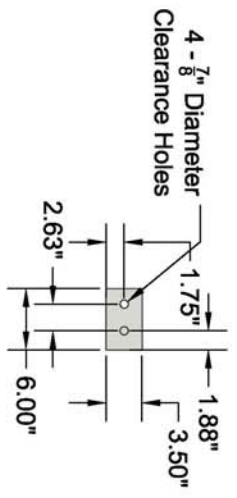
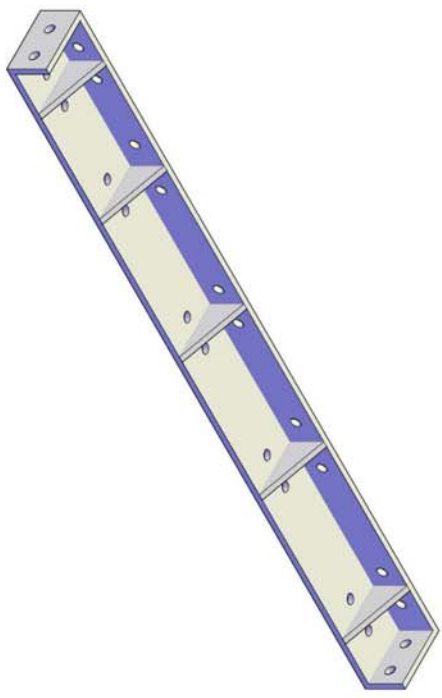
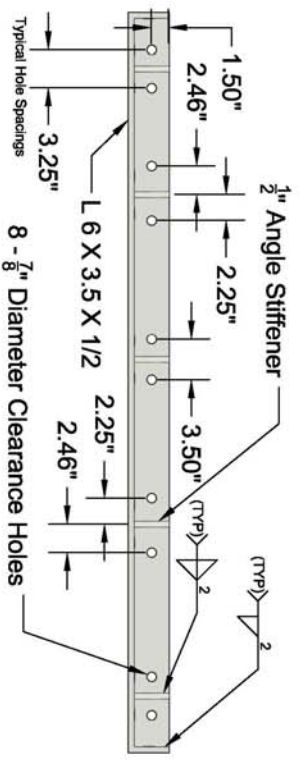
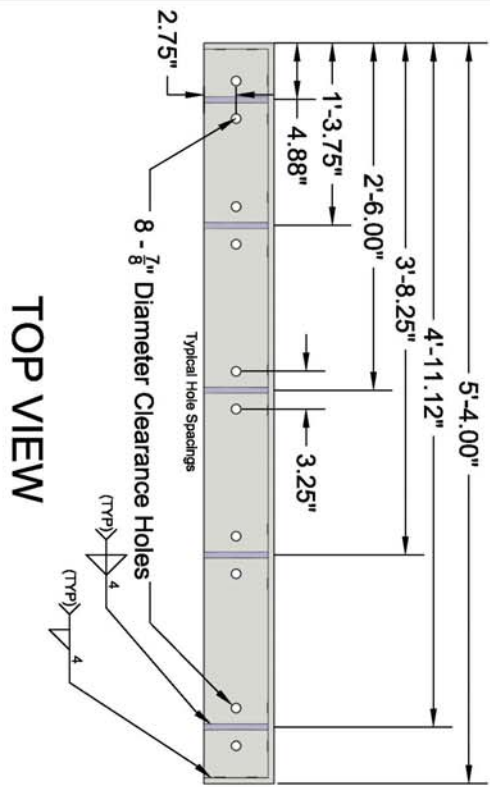
Part ID: Rear Support Angle

Drawn by: AWS

Project: Multiple Pull-out Box

Date: 11/18/12

Quantity: 2



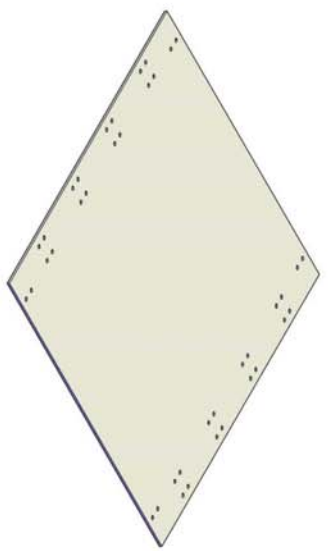
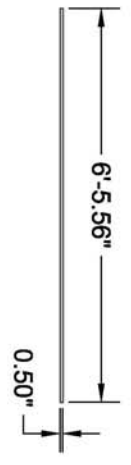
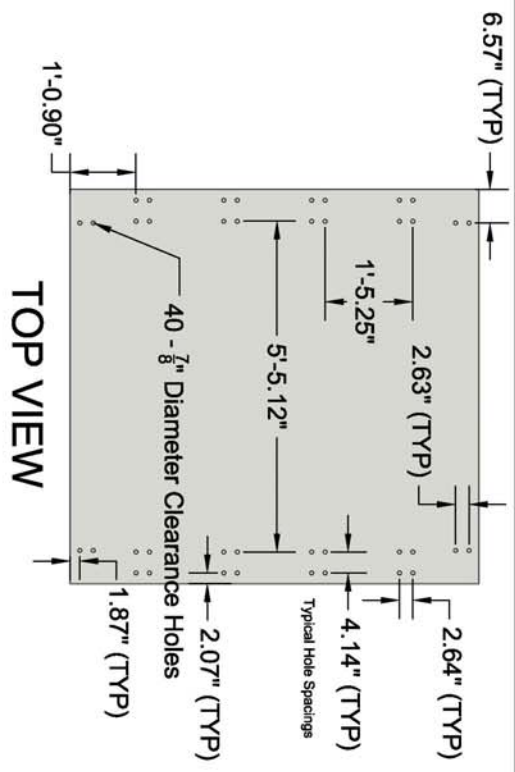
Part ID: Front Support Angle

Drawn by: AWS

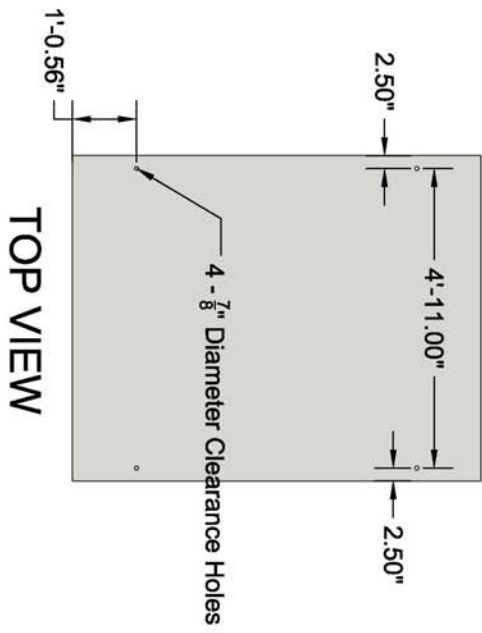
Project: Multiple Pull-out Box

Date: 11/18/12

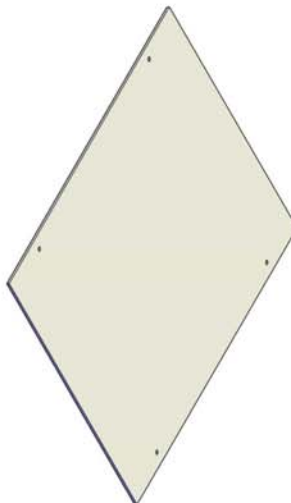
Quantity: 2



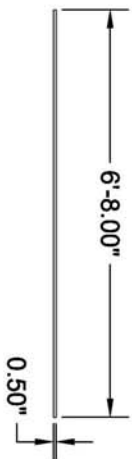
Part ID: Base Plate	Drawn By: AWS	Date: 11/18/12
	Project: Multiple Pull-out Box	Quantity: 1



FRONT VIEW

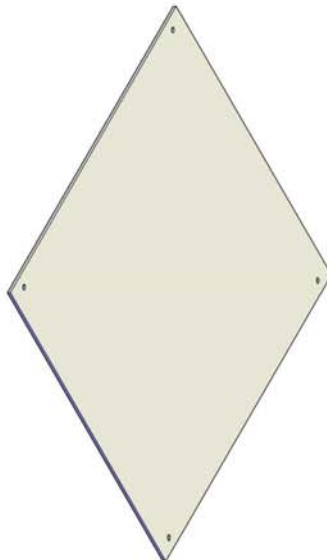
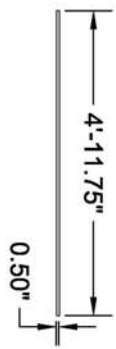
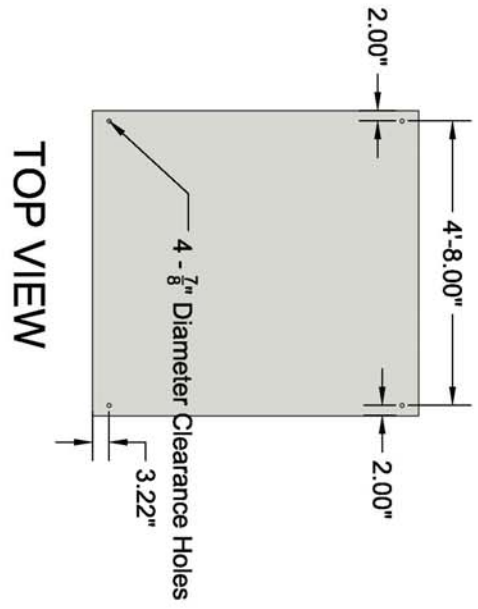


SIDE VIEW



Part ID: Side Plate

Drawn by: AWS
 Project: Multiple Pull-out Box
 Date: 11/18/12
 Quantity: 2



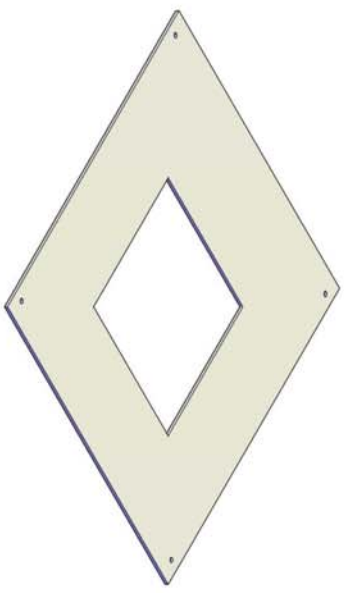
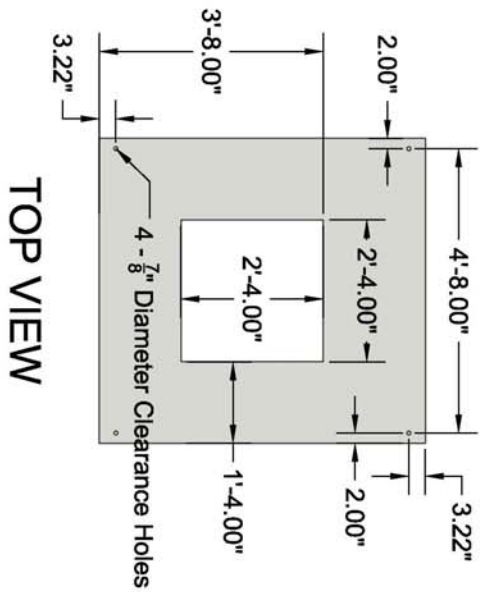
Part ID: Rear Plate

Drawn by: AWS

Project: Multiple Pull-out Box

Date: 11/18/12

Quantity: 1

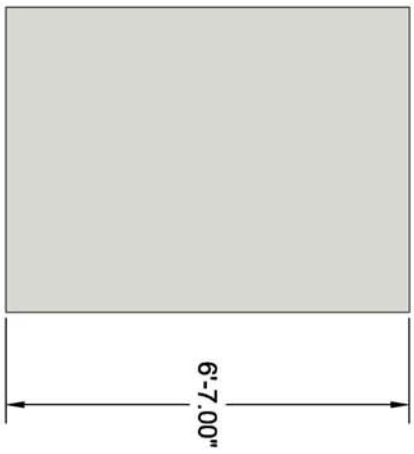


SIDE VIEW

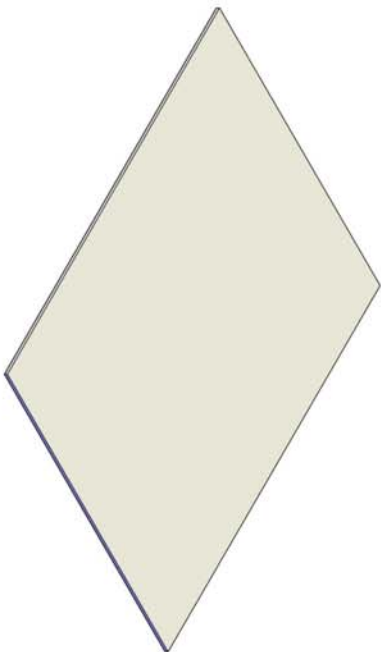


Part ID: Front Plate

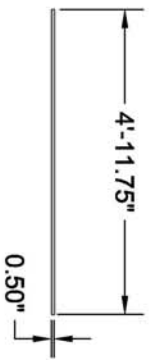
Drawn by: AWS
 Project: Multiple Pull-out Box
 Date: 11/18/12
 Quantity: 1



TOP VIEW



SIDE VIEW



FRONT VIEW

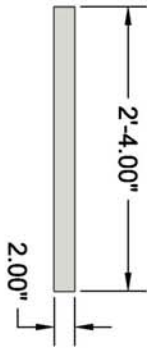
Part ID: Top Plate

Drawn by: AWS

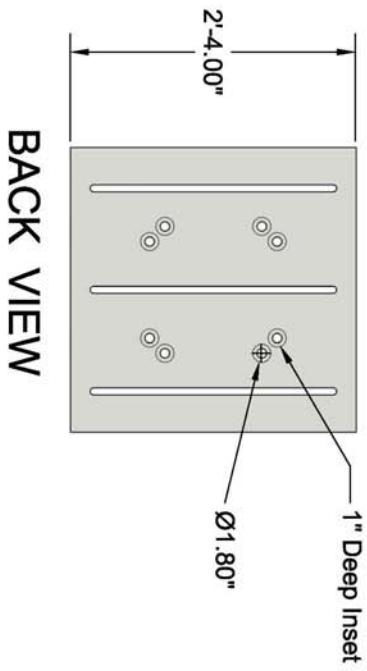
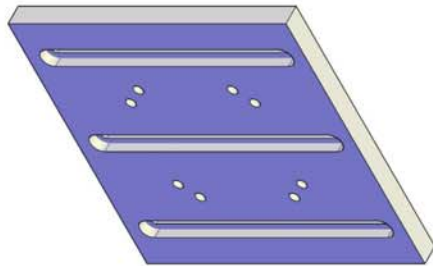
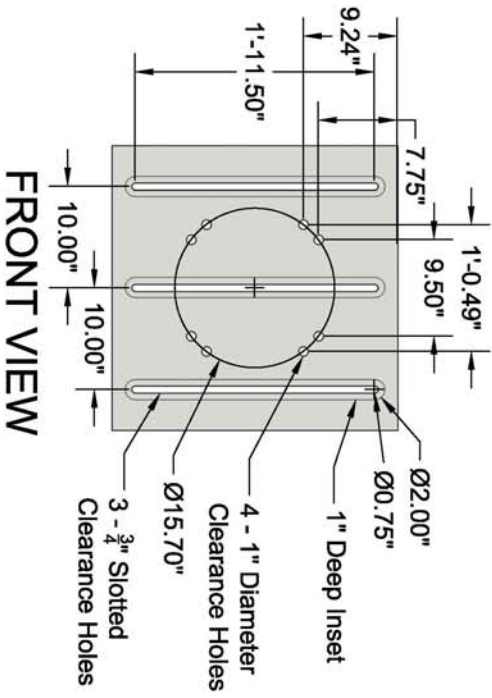
Project: Multiple Pull-out Box

Date: 11/18/12

Quantity: 1

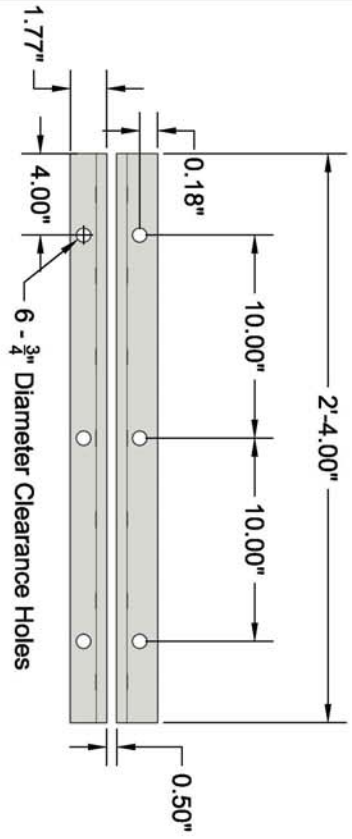


TOP VIEW

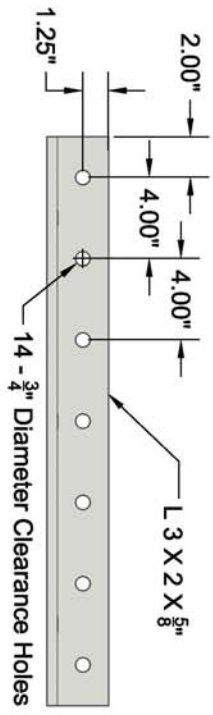
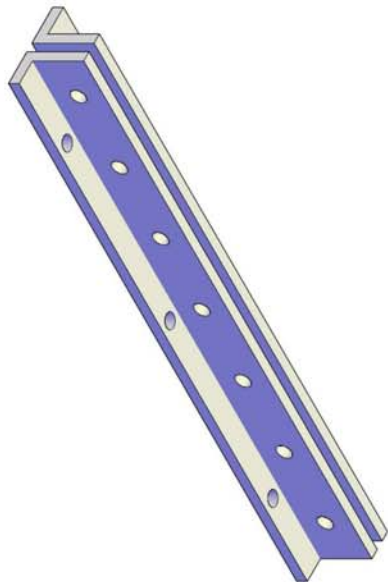


Part ID: Connection Plate 1

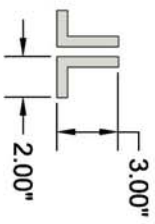
Drawn by: AWS
 Project: Multiple Pull-out Box
 Date: 11/18/12
 Qty: 1



TOP VIEW



FRONT VIEW



SIDE VIEW

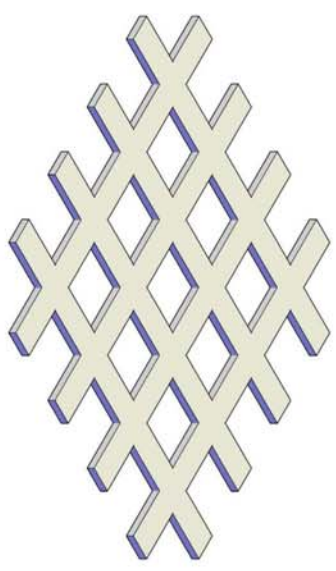
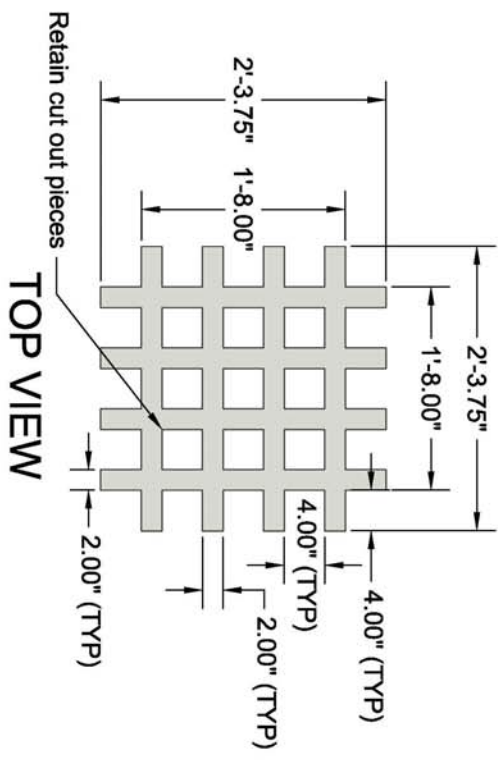
Part ID: Connection Angle

Drawn by: AWS

Project: Multiple Pull-out Box

Date: 11/18/12

Quantity: 3

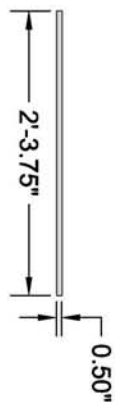
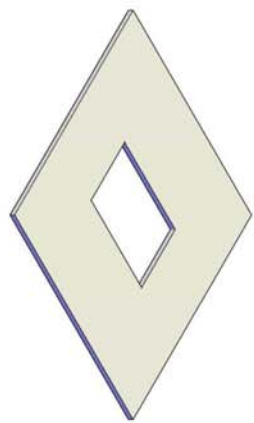
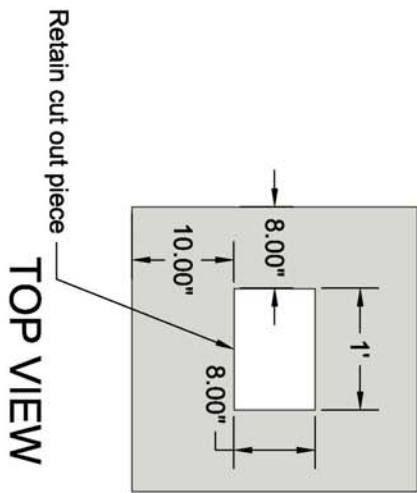


FRONT VIEW

SIDE VIEW



Part ID: Pull-out Plate 1	Drawn by: AWS	Date: 11/20/12
	Project: Multiple Pull-out Box	Quantity: 1



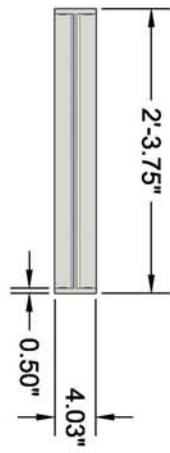
Part ID: Pull-out Plate 2

Drawn by: AWS

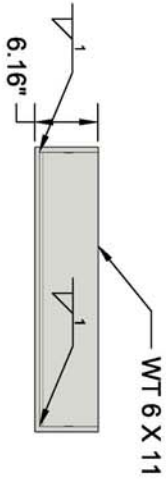
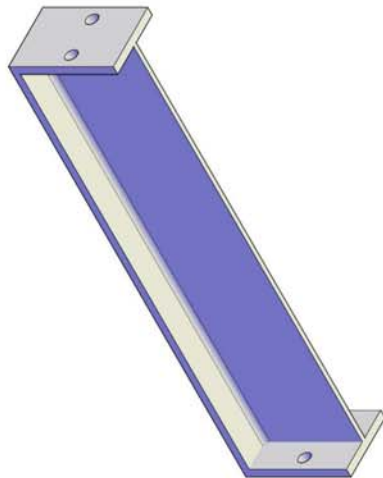
Project: Multiple Pull-out Box

Date: 11/20/12

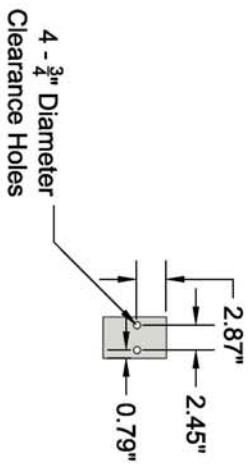
Quantity: 1



TOP VIEW



FRONT VIEW



SIDE VIEW

Part ID: Pull-out Reinforcement

Drawn by: AWS
 Project: Multiple Pull-out Box
 Date: 11/20/12
 Quantity: 3