Pullout and Tensile Behavior of Crimped Steel Reinforcement for Mechanically Stabilized Earth (MSE) Walls

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PULLOUT AND TENSILE BEHAVIOR OF CRIMPED STEEL REINFORCEMENT
FOR MECHANICALLY STABILIZED EARTH (MSE) WALLS

by

Oscar E. Suncar

A thesis submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

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Logan, Utah

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ABSTRACT

Pullout and Tensile Behavior of Crimped Steel Reinforcement for Mechanically Stabilized Earth (MSE) Walls

by

Oscar E. Suncar, Master of Science
Utah State University, 2010

Many research studies made on hundreds of MSE walls have shown that in order to get lower values of lateral earth pressure coefficients from an active condition on the backfill soil, thus lower exerted loads and stresses on the reinforcement, the wall needs to yield. This is typical of extensible polymer-based wall systems, such as geosynthetics. Steel systems, on the other hand, are very rigid and do not allow enough deformation on the wall to generate the active condition.

For this research, steel reinforcement for MSE walls that behaves similar to geosynthetics was developed. This was done by using crimps on steel bars that would allow the wall to deform as the crimps straighten. A pullout box was designed and constructed, where tensile and pullout tests were performed on the crimped reinforcement. Different crimp geometries on different bar diameters were tested under a range of confining pressures. From this, force-displacement curves were
developed for these crimp geometries that could be used to predict deflections on walls with crimped reinforcement.

In addition, the pullout resistance of the crimps in the straighten process was evaluated. This way, the crimps would not only be used to allow the wall to yield, but also as a pullout resistance mechanism. The pullout resistances per crimp for different tensions on the crimp and under a range of overburden pressures were evaluated. By combining the pullout resistance of the crimps and the force-displacement curves, a new internal stability design method was introduced where crimped reinforcement is used to resist both pullout and rupture failure.

Also presented here are the pullout resistances of round bars with improved deformations of different diameters. These were found to have the same pullout resistance of square deformed bars with the same cross-sectional area. Round bars are preferred over square bars because they are more corrosion resistant and have longer design life.
DEDICATION

I dedicate this thesis first to God; through him everything is possible. Then to my beautiful family; you are the inspiration of my life and the reason to keep giving the best out of me; thank you for your unconditional support. At last but not least, to my friends and all others that have supported me through all these years, encouraging and motivating me to achieve greater things; I am deeply grateful to you.

Oscar E. Suncar
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Thanks to Ken Jewkes, for all his time and technical support building up the pullout box. His innovative thinking, outstanding construction abilities, and willingness to work hard are gratefully acknowledged.

Thanks are also due to the Hilfiker Retaining Wall Company, who generously provided the financial support and sponsorship of this project. Specifically, thanks to Mr. Bill Hilfiker, who also provided his ideas, expertise, and professional insight during this process.

Finally, very special thanks and admiration to my colleague Bernardo A. Castellanos. His hard work and dedication were very valuable for this project. He has also been an example of a true friend; without him the time at Utah State University would not have been so gratifying.

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Since the French engineer Henri Vidal developed the concept, soil reinforcement has become one of the most used solutions for earth foundation and retaining systems. It consists of the improvement of the mechanical properties of a soil mass using reinforcement elements such as metal bars, welded wire mats, geosynthetics or other anchorage systems. This benefit effect has been demonstrated by the successful construction of numerous retaining walls with different types of reinforcing and facing elements, as well as reinforced embankment slopes and natural or cut slopes (Sampaco, 1996).

The original concept developed by Vidal (1969) used steel strips as reinforcement with facing elements made of either metallic elements or segmental concrete panels, and was introduced to the U.S. in 1972 with the construction of a Reinforced Earth Wall on California’s State Highway 39. Since its introduction, approximately 700,000 m$^2$ of MSE wall facing area were built every year in the U.S. as of 1997, and keeps growing every year, using a wide range of different and improved reinforcing and facing configurations (ASCE, 1997).

Many efforts have been made over the years to develop design procedures by the American Association of State Highway and Transportation Official (ASSHTO), the U.S. Federal Highway Administration (FHWA) and some States’ Departments of
Transportation. These design procedures for MSE walls are based on limit equilibrium concepts but working stress observations were also used to adjust the models to fit what has been observed in extensive full-scale structures. In addition, theoretical and numerical methods have been implemented to verify the results.

Current design procedures identify two main types of MSE Wall systems: extensible and inextensible. Extensible walls are those that are considered to move and allow deformations on the face of the wall, thus decreasing the stresses on the reinforcement and the face itself. Inextensible walls on the other hand, are considered to be very stiff with little or no deformation. Steel systems fall into the category of inextensible walls while polymer-based systems, such as geosynthetic walls, into the extensible.

Extensible systems are usually used in shorter retaining wall structures and in most reinforced soil slope projects (ASCE, 1997). They are very cost effective and can be used in numerous applications. However, geosynthetics are greatly affected by the environment and by time, which limit their ultimate strength and construction capabilities. Steel systems, on the other hand, can be used in taller and more demanding structures because the engineer has more control over stresses and deformations.

Each MSE wall type must satisfy the internal and external stability criteria. External stability is evaluated by considering the reinforced mass as a semi-rigid block with active pressure applied behind the wall. This semi-rigid block is checked against
failure for conventional stability criteria: overturning, sliding, bearing capacity and deep stability.

The internal stability evaluation involves checking for a sufficient factor of safety against tension and pullout failure of the reinforcement, and the integrity of the facing-reinforcement connections. Tension failure or rupture of the reinforcement is evaluated by comparing the maximum tensions on the wall to the tensile capacity of the reinforcement material. One important consideration when doing this evaluation is accounting for the effective cross-sectional area of the reinforcement after corrosion losses or for the effective strength of the reinforcement after other environmental degradations and long term effects.

Pullout failure is assessed by checking if the reinforcement extends far enough behind the potential failure plane and develops sufficient resistance at the soil-reinforcement interface to prevent being pulled out of the resisting mass. Pullout resistance depends on two essential soil-reinforcement interaction mechanisms: friction and passive earth pressure resistance. Both mechanisms are very complex and are not yet fully understood, thus pullout resistance cannot be accurately computed theoretically, but is rather estimated from pullout test data.

One of the most important parameters required in designing for internal stability is the lateral earth pressure coefficient, \( K \), which is defined as the ratio of the horizontal to vertical earth pressure. This variable determines the magnitude of the forces that are exerted to the reinforcement by the soil, thus the amount of steel or geosynthetic reinforcement required to resist these loads. It depends on the amount of yielding that
can take place on the wall, as well as the type of reinforcement. Therefore, higher values of $K$ are expected for stiffer, inextensible reinforcing systems such as steel than for relatively extensible systems such as geosynthetics.

Another important parameter is the potential failure plane, which defines the location of the locus of maximum tension on the reinforcement and consequently, the length of the reinforcement, since the pullout resistance of the reinforcement must be developed behind this plane. This parameter also depends on the amount of yielding that can take place on the wall, and varies for inextensible and extensible systems.

Objectives and Scope of This Research

Many research studies have been made over hundreds of MSE Wall projects during and after construction and with different configurations in order to determine potential failure planes, lateral earth pressure coefficients, deformations, maximum forces and stresses and environmental and ground water influences on reinforcement of MSE walls. These studies have shown that in order to get lower values of lateral earth pressure coefficients, thus lower exerted loads and stresses on the reinforcement, the wall needs to yield. This means that the reinforcement must be extensible and allow certain amount of deformation for the soil to reach an active condition. This situation is typical of polymer-based wall systems, such as geosynthetics. Steel systems on the other hand, are very rigid and do not allow enough deformation on the wall to generate an active condition.
The main purpose of this research is to develop steel reinforcement for MSE walls that behaves similar to geosynthetics. This would be done by using crimps on steel bars that would allow the wall to deform as the crimps straighten. Then, the reinforcement would become less stiff, the potential failure plane would resemble that of extensible systems, and the lateral earth pressure coefficients would be equal to the active case. Once this has been accomplished, the loads on the reinforcement would decrease significantly, reducing the steel requirements, thus the total cost of the structure.

Different crimp shapes and bar diameters were tested in the air and in soil under different confining pressures. The objective was to obtain an ideal shape that would allow the sufficient amount of deformation on the reinforcement while not compromising the wall stability, and would completely straighten to be able to use the full allowable strength of the steel, which is about half of the yield stress. With different confining pressures, the effects of the soil and the different pressures at every height of an MSE wall were simulated. Once an adequate shape was found, the tensile behavior of different sizes of crimps for different round bar diameters was determined. The bar diameters that were expected to be used from top to bottom of a wall of up to 50 feet in height were tested.

Another objective of this research was to evaluate the pullout resistance of round straight bars of different cross-sectional area with different forms of deformations. The goal was to increase the pullout capacity of round steel bars by using improved deformations on them, and to find round bars that would have similar pullout
resistance of square deformed bars, such as a ribbed steel strips (known as RECO Straps), which have more surface area and have traditionally been the soil reinforcement with the most pullout capacity. Since corrosion affects the thickness of the reinforcement in every direction, for the same corrosion depth, reinforcement with a square cross section, such as a steel strap, would have a greater loss of cross-sectional area than a round bar with same original cross section area. Thus, it would be ideal to have a round bar with closely the same cross-sectional area than a RECO strap, and similar pullout resistance. The RECO strap’s pullout resistance was also tested for comparison.

The studies have also shown that in most MSE walls, as described in Chapter 2, the steel is only stressed to about 30 % of its yield stress. This is because bars of large cross-sectional area have been used due to limitations on the code, and since they have more surface area and consequently more pullout. By pushing the bars to its limit and designing for half the yield stress, and finding ways to improve pullout resistance, adequate bar sizes and spacing could be used to additionally reduce steel requirements and total expenditures.

Initially, the crimps’ benefit of allowing deformations on the wall was going to be combined with the improved deformations on round bars for increased pullout resistance, as described above, in order to obtain an optimal crimped steel reinforcement to use for internal stability design of MSE walls. Due to expected difficulties on industrially manufacturing the improved bars, the inventory benefits of using rebar (structural steel) as soil reinforcement, having found their relatively good
pullout resistance compared to smooth bars, and the fact that the crimps infer that they significantly contribute to pullout resistance, a new scope was given to this research.

The pullout resistance contribution of the crimps while they straighten was then tested on rebar, under different confining pressures that simulate layers of reinforcement along the wall height. From this, a new internal stability design approach was developed by using a steel rebar with a series of crimps as MSE wall reinforcement. In this approach, each crimp contributes not only to the deformation of the wall, but to the pullout resistance depending on the tension exerted on them, i.e. how much they have been straightened. The contribution of the portions of straight rebar along the total length of the reinforcement were also considered for the pullout resistance, thus their pullout resistance was also evaluated in this research.

This way, as stated before, the crimps would allow deformations on the wall to develop an active condition, and would also contribute to the pullout resistance, making it a more practical and economical internal stability design approach. In addition, by using rebar many production and availability problems would be avoided, since it is also used as structural steel.

Nevertheless, the benefit of the crimps can still be applied to other steel reinforcement with different pullout capacity mechanisms, in order to help them behave similar to geosynthetics. For instance, crimps could be placed on the longitudinal bars of welded wire mats or steel grids to allow enough deformations and develop an active condition on the wall, while taking advantage of the transverse wires for pullout
resistance. The tensile behavior of crimps on longitudinal bars of welded wire mats were also tested as part of this study but the results are not included here.

In order to perform the tests on the crimped bars, tensioning them until failure in the air and in soil, and to evaluate the pullout resistance of the straight bars and the crimped bars in the straightening process, a closed pullout box was designed and built at Utah State University. It was designed to resist pressures of 30 psi, and was adapted in a way that both tension and pullout testing could be performed in it, sometimes without opening the box. More of the details of the pullout box and the testing procedures are described in Chapter 3. The crimped and deformed bars were all provided by the Hilfiker Retaining Wall Company.

In summary, this research had four main purposes. First, to design and build a closed pullout box to accommodate the tension and pullout testing. Second, to evaluate the pullout resistance of differently deformed straight bars for comparison. Third, to test different crimp shapes on different bar sizes and under different pressures to develop force-displacement curves. Also, to further assess whether the crimps fully straighten without braking, being able to utilize half the yield strength of the steel, and whether they allow enough or too much deformation for the active case to occur at each level of the wall. The fourth and final objective was to evaluate the pullout resistance of the crimps on the process of straightening and develop tension vs. crimp pullout resistance curves at various overburden pressures. Then, to a new internal stability design approach using the tension vs. crimp pullout resistance and force-displacement curves is presented.
Chapter 2 tackles the most important concepts about soil reinforcement and MSE wall design. Chapter 3 fully describes the design and construction process of the pullout box, as well as the type of soil used for the test and its properties. It also gives an explanation of the tests’ methodologies. Chapter 4 talks about the straight bar pullout tests performed: the process, the bars tested and an analysis. On Chapter 5, all the tests made on crimped bars are presented. On this chapter, the process of the tension tests, the pullout tests and the combined tension/pullout tests made on the crimped bars are carefully explained, and the results are presented. Chapter 6 explains the new internal stability design approach using crimped bars, and goes through the process of defining the design parameters and developing the design curves from the data. Chapter 7 then exposes the general conclusions of this research and some recommendations are made.
CHAPTER 2

LITERATURE REVIEW

Introduction

This chapter discusses the most important concepts related to soil reinforcement and MSE Wall Design. The MSE wall components and the behavior of most MSE wall structures are presented, as well as the parameters used for design, how to estimate them and the assumptions made in the design process.

Regardless of the type of system used, all MSE walls behave and are analyzed the same way, needing to meet the same design requirements. These requirements are basically three: external stability, internal stability and acceptable deformations. This chapter will talk about all of them but will focus on the internal stability due to the nature of the objectives and scope of this research.

The current state of knowledge on MSE wall design is basically contained in the AASHTO Bridge Design Specifications. All design codes make reference to these specifications for external stability evaluation, load factors and other design parameters, but when it comes to internal stability evaluation, there are two available design methods: the Simplified Method, developed by the AASTHO, and the K-stiffness method, under development by the Washington State Department of Transportation. Their main difference relies on the way each method estimates the reinforcement loads. The K-Stiffness method takes into account the stiffness difference between the layers of reinforcement, the friction along the base of the wall, among other factors that the
AASHTO does not. In this chapter, this and other differences between the two main methods for internal stability design are also described.

Principal MSE Wall Components

Soil reinforcement systems are composite structures built using alternating horizontal layers of reinforcing elements that improve the mechanical properties of the compacted soil that surrounds them. Although different reinforcement materials and configurations are implemented into MSE wall systems, they all contain three main components:

1. the reinforcing elements,
2. the backfill soils, and
3. the facing elements (Anderson et al., 1995). Figure 2.1 shows the principal components of MSE wall systems.

As stated previously, there are various types of reinforcing systems and different configurations that are available in the industry today. The most common reinforcing elements used are steel strips, steel grids and welded wire mats, considered as inextensible reinforcement, and geosynthetics, such as geotextiles and geogrids, which are categorized as extensible reinforcement. This classification is based on the deformation of the reinforcement at failure, which for inextensible reinforcement is much less than the deformability of the soil, while for extensible reinforcement is comparable or more. Each reinforcement type behaves differently from the other and has its unique advantages and disadvantages, thus careful considerations should be
made of the project requirements before the selection of a reinforcement system (Conder, 2002).

Another important requirement is the selection of the backfill material to be used in the wall. The type of backfill chosen depends upon the design requirements of the wall and the availability of soils around the construction area and their costs. For most soil structures, well-graded cohesionless are used due to their free drainage and stability. Cohesive backfills are also used, though they are usually undesirable due to their poor drainage, low strength, compressibility, and creep properties (Mitchell and Christopher, 1990).

The facing elements are another important requirement in the design of MSE walls. There are numerous types of reinforcing facing elements as well, which are chosen based on the amount of the settlement the wall can tolerate or whether the primary settlement occurs before or after the placement of the facing elements, and on aesthetic considerations. Some of the most common facing elements used today are stiff or rigid facings, such as concrete panels with different shapes and forms, concrete blocks rigid facings, such as concrete panels with different shapes and forms, concrete blocks, steel and timber facings; flexible wall facings, such as welded wire, expanded metal, geosynthetics, gabions, and similar facings; and post-construction facings, such as shotcrete, prefabricated panels of wood or concrete, among other materials, that can be attached to wrapped face walls (welded wire, geosynthetic or geogrid walls) after they have been constructed.
Flexible walls can tolerate large amounts of settlement, while stiff facings, like concrete panels, can tolerate little to no settlement, but they generally are more aesthetic. To be able to use stiff concrete as facing elements, large surcharges are often used to allow primary settlement to occur before the placement of the panels.
The location of the locus of maximum tensile forces for MSE walls, also called the potential failure surface, has been reported to be closely to the Coulomb Failure Plane (Lee et al., 1973), parabolic (Schlosser and Long, 1974) and a logarithmic spiral (Juran, 1977). These conflicted observations indicate that the location and shape of the potential failure surface varies depending upon the geometry of the wall and the type and stiffness of the reinforcing elements.

Christopher et al. (1989) utilized the currently available field and experimental data to distinguish two types of potential failure planes depending on the stiffness of the reinforcement system used. Thus, two main types of MSE wall systems were identified: inextensible and extensible. For inextensible reinforcement, such as steel straps and welded wire mats, a bilinear failure surface is defined as shown in Figure 2.2 (a). For extensible reinforcement, such as geotextiles and geogrids, the failure surface closely resembles the Coulomb failure plane as shown in Figure 2.2 (b). These failure surfaces are the ones used for design by current design codes.

The potential failure plane divides the reinforced soil structure into two zones (Figure 2.3): the active zone, in which the tangential shear stresses exerted by the soil on the reinforcement are directed towards the face while the soil tries to spread laterally; and the resistant zone, in which the tangential shear stresses are directed towards the free end of the reinforcement, mobilized trying to prevent the sliding of the reinforcements (Sampaco, 1996).

It is important to note that the stability of the entire reinforced wall depends on the available resistance occurring within these two distinct zones, as it will be described
Figure 2.2 Potential failure surfaces (after Anderson et al., 1995).

Figure 2.3 Tension distribution along reinforcement and between zones in a MSE wall.
in the next section. For the whole system to be stable, the reinforcement must be of adequate size to prevent rupture, and must extend far enough behind the failure plane to develop sufficient pullout resistance to keep the reinforcement from pulling out of the soil mass and restrain the active zone from sliding along the potential failure surface.

Stability Requirements

The first of a series of requirements for the successful design and construction of MSE walls is stability. As stated before, regardless of the components used, MSE walls must have the adequate external and internal stability.

External stability

External stability is evaluated by considering the reinforced soil mass as a semi-rigid gravity retaining wall with active pressure applied behind it. Then, conventional limit equilibrium methods are used to check the performance of the wall against sliding, overturning, bearing capacity and deep or overall stability. Figure 2.4 shows the four mechanisms of external failure in MSE walls. The AASHTO Bridge Design Specifications give detailed guidelines on how to go through the analysis of each one of these stability criteria. Other design codes refer to these specifications for the external stability design as well.

For the calculations of the active forces acting behind the wall, the AASHTO uses Coulomb’s earth pressure theory, assuming no wall friction at the interface of the wall
(taken as a rigid mass) and the soil by setting the interface friction angle equal to the angle of the fill to the horizontal. It also states that the friction angle of the retained soil shall be used for all calculations unless specific data is missing, in which a maximum friction angle of $30^\circ$ may be used.

For calculations of the bearing pressure acting beneath the MSE wall, the AASHTO assumes a uniform base distribution over an effective width, with the eccentricity of the resultant of the vertical forces taken into account.

**Internal stability**

Safety against structural failure or internal stability is evaluated with respect to pullout and rupture of the reinforcement. Both failure modes are critical: tensile failure can lead to progressive collapse of the reinforced structure as the load is transmitted to the remaining elements; pullout resistance on the other hand can lead to progressive deformation of the wall due to redistribution of stresses (Budhu, 1999). Figure 2.5 shows the two internal failure modes.

In this section, the estimation of the forces that are exerted from the soil to the reinforcement will be explained first, starting with the concepts of lateral earth pressures and finishing with how each internal stability method comes up with the estimation of reinforcement loads. Then, the performance check of the reinforcement with respect to rupture and pullout of the reinforcement will be presented, as well as the considerations taken and how the parameters used are estimated.
Figure 2.4 Failure mechanisms for external stability evaluation of MSE walls (after Christopher et al., 1989)

(1) Sliding
(2) Overturning
(3) Bearing Capacity
(4) Deep Stability

Figure 2.5 Internal failure modes for MSE walls (after Anderson et al., 1995).

(1) Tension Failure
(2) Pullout Failure
Lateral earth pressures

When considering internal stability, it must be understood that the stresses of the reinforcement are transferred into the soil differently upon the system selected. The soil-to-reinforcement relative movement required to mobilize the design tensile force depends mainly upon the load transfer mechanism, the extensibility of the reinforcement material, the soil type, and confining pressure.

Extensible systems require large amounts of strains to mobilize the strength of the reinforcement and thus larger internal deformations occur. On the other hand, inextensible systems require very small strains to mobilize the strength of the reinforcement, much smaller than the strains required to mobilize the strength of the soil (Anderson et al., 1995).

Bonaparte and Schmertmann (1988) analyzed and tested different types of reinforcement to determine a relationship between reinforcement stiffness and mobilized lateral earth pressures coefficients. According to them, equilibrium strains for inextensible steel reinforcement are in the order of 0.01 % to 0.1 %. Meaning by equilibrium strains, the strains in which the reinforcement tension balances the lateral earth forces. On the other hand, even for the stiffest geosynthetics currently used, the equilibrium strains are in the order of 20 times greater than those for steel reinforcements.

The latter implies that for inextensible reinforcement, the lateral earth pressures are higher than those for geosynthehtics. This is because with steel reinforcement the soil never reaches an active condition, since the steel would need to undergo a tensile strain
on the order of its initial yield strain of 0.2% in order for the soil to reach an active plastic state. This amount of horizontal strain required to induce an active plastic state on soil was determined on an unreinforced compacted granular fill, and can range from far less than 0.5% to 1% to 2% depending on the stress path. Then again, for geosynthetic reinforcement the lateral stresses were found to be virtually those corresponding to active conditions, because the strains were 20 times higher than the ones for steel reinforcement (Bonaparte and Schmertmann, 1988).

Furthermore, Terzaghi (1936) gave rough quantitative values of amounts of yield that is needed on a conventional retaining wall for the active condition to develop in the soil:

(a) If the mid-height point of the wall moves outward a distance roughly equal to 1/20 of 1 per cent of the wall height, an arching active case is attained.

(b) If the top of the wall moves outward a distance roughly equal to ½ of 1 per cent of the wall height, the totally active case is attained (Taylor, 1948).

In addition, Table C.3.11.1-1 of the AASHTO Bridge Design Specifications also gives some values of movements of the top of the wall (Δ) relative to the total height of the wall (H), required to reach active earth pressure conditions for different soils in conventional retaining walls. These are presented in Table 2.1.

Nevertheless, MSE walls behave differently than conventional retaining walls, thus no one have ever predicted what amount of yielding is needed to attain an active case on an MSE wall.
In the author’s opinion, it would be a good assumption that to be able to reach the active case in an MSE wall, which has been demonstrated that develops for geosynthetic reinforcement, the horizontal strain needed to reach an active case on unreinforced soil and the amount of yield for an active condition to develop on a conventional wall should both be exceeded. However, further tests and full-scale MSE wall evaluations should be performed to get an accurate value of the yielding needed for an active condition to develop in different types of MSE walls.

Nevertheless, in order to obtain current design values for lateral earth pressures, numerous studies have been made evaluating the lateral earth pressure coefficient, $K$, which is the ratio of horizontal to vertical soil pressure, on full scale inextensible and extensible systems. As stated before, extensible reinforcement allow more yielding to occur near the top of the wall, which mobilizes the active lateral pressure. On the other hand, inextensible reinforcing elements have small lateral displacements near the top of

<table>
<thead>
<tr>
<th>Type of Backfill</th>
<th>Values of Δ/H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dense sand</td>
<td>0.001</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>0.002</td>
</tr>
<tr>
<td>Loose sand</td>
<td>0.004</td>
</tr>
<tr>
<td>Compacted silt</td>
<td>0.002</td>
</tr>
<tr>
<td>Compacted lean clay</td>
<td>0.010</td>
</tr>
</tbody>
</table>
the wall. This seems to develop the at-rest lateral coefficient in the upper part of the wall, with the active state of stress developing in the lower part of the wall. The at-rest condition, and sometimes higher values of $K$, develops in the upper part of the wall also due to overstressing from heavy compaction. The active condition is reached in the lower part due to arching effect between the base of the reinforced soil structure and the upper section of the wall (Schlosser, 1990).

The appropriate lateral earth pressure coefficient value to use in the estimation of reinforcement loads in the design process depends upon the soil reinforcement system chosen. Christopher et al. (1989) compiled design envelopes of $K$ for various types of reinforcement from case histories and are now used for design using the Simplified Method of the AASHTO (Figure 2.6). Furthermore, Allen and Bathurst (2003) also developed a method, the K-Stiffness Method, which estimates reinforcement loads from case histories, considering the stiffness of the reinforcement and other factors not taken into account by the Simplified Method. Both methods, with their figures and equations used for predicting reinforcement loads are presented in the next section.

**Maximum reinforcement load**

The maximum reinforcement loads and stress are assumed to be located at the boundary between the active zone and the resistant zone in Figure 2.3. Here, the potential for reinforcement rupture and pullout must be evaluated.

There are basically two approaches for the calculation of the maximum tension acting on the failure plane, as mentioned before: the Simplified Method and the K-Stiffness Method. The Simplified Method by the AASHTO assumes that each
reinforcement element provides the lateral restraint for an area that extends half the
distance to the adjacent element above, below and on both sides. Then, the maximum
tension could be expressed as:

\[ T_{max} = \sigma_H S_v S_h \]  \hspace{1cm} (2.1)

where:

\[ \sigma_H = \text{Factored horizontal soil stress at the reinforcement level (ksf)}. \]
\[ S_v = \text{Vertical spacing of the reinforcement (ft) (Should not be greater than 2.7 ft unless sufficient full scale data is available to support the use of higher spacing)}. \]
\[ S_h = \text{Horizontal spacing of the reinforcement (ft)}. \]

Generally, the loads are calculated per unit width of wall, making the maximum
reinforcement load equal to:

\[ T_{max} = \sigma_H S_v \]  \hspace{1cm} (2.2)

The horizontal soil stress at any depth, \( z \), from the top of the wall can be
calculated using the following equation:

\[ \sigma_H = \gamma_p \left( \sigma_v K_r + \Delta \sigma_H \right) \]  \hspace{1cm} (2.3)

where:

\[ \gamma_p = \text{Load factor for vertical earth (AASHTO, 2004)}. \]
\[ K_r = \text{Horizontal pressure coefficient as specified in Figure 2.6}. \]
\[ \sigma_v = \text{Pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, any surcharge loads present, and any increase in vertical stress due to concentrated vertical loads (ksf) (AASHTO, 2004)}. \]
\[ \Delta \sigma_H = \text{Horizontal stress at reinforcement level resulting from any applicable concentrated horizontal surcharge load (ksf)}. \]
The AASHTO states that the maximum friction angle used to calculate the horizontal forces within the reinforced soil mass shall be assumed to be 34° unless results from triaxial or direct shear tests methods (AASHTO T 234-74 and T 236-72) specifies a higher friction angle, and it should never be more than 40°.

As mentioned before, the other approach for estimating loads on the reinforcement is the K-Stiffness method, developed by the Washington Department of Transportation, which will be explained next. This approach differs from the one just exposed above in that it estimates the maximum reinforcement load by adjusting the

![Variation of the coefficient of lateral stress ratio kr/ka with depth in a MSE wall (AASHTO, 2004).](image)
maximum at rest lateral earth forces at each reinforcement layer with a load distribution factor \( (D_{\text{tmax}}) \) and some others empirical factors that take into account the stiffness of the facing and reinforcement, the facing batter, among others.

To calculate the maximum factored load at each reinforcement level, also on a per unit width of wall basis, the following equation shall be used:

\[
T_{\text{max}} = 0.5S_V K \sigma_V D_{\text{tmax}} \phi_{fs} \phi_{local} \phi_{fb} + \gamma_p \Delta \sigma_H S_V
\]  

(2.4)

where:

\( S_V \) = Vertical spacing (ft.). (Should also not be greater than 2.7 ft unless sufficient full scale data is available to support the use of higher spacing).

\( K \) = Index lateral earth pressure coefficient equal to \( K_0 = 1 - \sin \phi \) (dim.).

\( \sigma_V \) = Pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, any surcharge loads present, and any increase in vertical stress due to concentrated vertical loads (ksf) (WSDOT, 2006).

\( D_{\text{tmax}} \) = Distribution factor to calculate \( T_{\text{max}} \) for each layer as a function of depth (dim.) (Figure 2.7).

\( \Phi_g \) = Global stiffness factor (dim.).

\( \Phi_{local} \) = Local stiffness factor (dim.).

\( \Phi_{fb} \) = Facing batter factor (dim.).

\( \Phi_{fs} \) = Facing stiffness factor (dim.).

\( \Delta \sigma_H \) = Horizontal stress increase resulting from concentrated horizontal surcharge (ksf).

The following equations shall be used to calculate the reinforcement stiffness, facing stiffness and batter factors:
Global stiffness factor:

This factor considers the stiffness of the entire wall section.

\[ \phi_g = 0.25 \left( \frac{S_{global}}{P_a} \right)^{0.25} \]  

(2.5)

where:

\[ P_a = \text{Atmospheric pressure (2.11 ksf).} \]

\[ S_{global} = \text{Global reinforcement stiffness.} \]

\[
(2.6)
\]

where:

\[ J_{ave} = \text{Average stiffness of all reinforcement layers within the entire wall section on a per foot of wall width basis (kips/ft).} \]

\[ J_i = \text{Stiffness of an individual reinforcement layer on a per foot of wall width basis (kips/ft).} \]

\[ H = \text{Total wall height (ft.).} \]

\[ n = \text{Number of reinforcement layers within the wall section (dim.).} \]

Local stiffness factor:

This factor considers the stiffness and reinforcement density at a given layer.

\[ \phi_{local} = \left( \frac{S_{local}}{S_{global}} \right)^a \]  

(2.7)

where:

\[ a = 1.0 \text{ for geosynthetic walls and 0.0 for steel reinforced walls.} \]

\[ S_{global} = \text{Global reinforcement stiffness.} \]

\[ S_{local} = \text{Local reinforcement stiffness.} \]
(\( S_{\text{local}} = \frac{J}{S_V} \) )

where:

\( J \) = Stiffness of an individual reinforcement layer on a per foot of wall width basis (kip/ft.).

\( S_V \) = Vertical spacing (ft.).

**Face batter factor:**

This factor takes into account the influence of the reduced soil weight on reinforcement loads.

\[ \phi_{fb} = \left( \frac{K_{abh}}{K_{avh}} \right)^d \]  

(2.9)

where:

\( K_{abh} \) = Horizontal component of the active earth pressure coefficient accounting for wall face batter (dim.) (WSDOT, 2006).

\( K_{avh} \) = Horizontal component of the active earth pressure coefficient assuming that the wall is vertical (dim.) (WSDOT, 2006).

\( d \) = Constant coefficient recommended to be 0.25 to provide the best fit to empirical data (dim.).

**Facing stiffness factor:**

This factor considers the reduced stress due to facing stiffness in geosynthetic systems.

\[ \phi_{fs} = \eta F_{f}^{c} \]  

(2.10)

where:
\( \eta, k \) = Dimensionless coefficients determined from empirical data, 0.5 and 0.14, respectively.

\[
F_f = \frac{1.5H^2}{ELb_w^2h_{eff}} P_a
\]

(2.11)

where:

- \( b_w \) = Thickness of the facing column.
- \( L \) = Unit length of the wall.
- \( H \) = Total wall face height.
- \( E \) = Modulus of the facing material.
- \( h_{eff} \) = Equivalent height of an un-jointed facing column that is 100% efficient in transmitting moment throughout the facing column.
- \( P_a \) = Atmospheric pressure used to preserve dimensional consistency (2.11 Ksf.)

\( D_{max} \) shall be determined using Figure 2.7. This distribution only applies to wall constructed in firm soil. The rock and soft soil distribution should be different; it becomes more triangular as the soil becomes more compressible.

The K-Stiffness method was calibrated to use plane strain soil friction angle, thus if other test methods are used, such as triaxial and direct shear, some equations have to be used to convert the given friction angle to plane strain, and may be found in the WSDOT, 2006 (Equations 15-1 and 15-2). This method also limits the soil friction angle to 44°.

In general, as appreciated on both internal stability methods, when using less stiff reinforcement, such as geosynthetics, the loads can be reduced significantly.
Reinforcement strength

After calculating the maximum reinforcement loads at each reinforcement layer, the capacity of the reinforcement to resist these loads must be checked. Both internal stability methods use the following equation:

\[ T_{\text{max}} \leq \phi T_{\text{al}} R_c \]  \hspace{1cm} (2.12)

where:

\[ T_{\text{max}} \] = Applied factored load to the reinforcement (kips/ft).

\[ \phi \] = Resistance factor for reinforcement tension as specified either in the AASHTO, 2004 (Section 3) or the WSDOT, 2006 (Section 15).

\[ T_{\text{al}} \] = Nominal long-term reinforcement design strength per unit of reinforcement width (kips/ft).
Reinforcement coverage ratio as specified in the AASHTO, 2004 (Figures 11.10.6.4.1-1 and 11.10.6.4.1-1) ($R_c = \frac{b}{s_h}$, where $b =$ unit width of reinforcement and $s_h =$ Horizontal spacing).

The resistance factor for reinforcement tension ($\Phi$) given by the codes is about 0.75, but when combined with the load factors that are also applied during design, the yield strength of the reinforcement reduces to about half.

$T_{al}$ is evaluated differently for steel reinforcement and geosynthetic reinforcement. For steel reinforcement, the following equation applies, per unit of reinforcement width:

$$T_{al} = \frac{A_c F_y}{b}$$

where:

$T_{al} =$ Nominal long term reinforcement design strength (kips/ft).

$F_y =$ Minimum yield strength of steel (ksi).

$A_c =$ Area of reinforcement corrected for corrosion (in$^2$).

$b =$ Unit width of reinforcement (ft).

As noted on Equation 2.13, the steel reinforcement must account for the loss in cross-sectional area due to corrosion of the steel. To calculate this corrected area, a thickness of the reinforcement at the end of the service life is estimated using the following equation, which considers corrosion loss:

$$E_c = E_n - E_s$$

where:

$E_c =$ Thickness of metal at the end of service life (mil.).
\[ E_n = \text{Nominal thickness of steel reinforcement at construction (mil.).} \]

\[ E_s = \text{Sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure (mil.).} \]

The corrosion rates that may be used for the above equation are summarized in Yannas (1985) and should be taken as follows, for nonaggressive materials:

- Loss of galvanizing = 0.58 mil./yr. for first 2 years.
  = 0.16 mil./yr. for subsequent years.
- Loss of carbon steel = 0.47 mil./yr. after zinc depletion.

If the backfill soil used in the MSE walls does not meet the requirements of the AASHTO for nonaggressive materials, other considerations must be considered, such as using galvanized coating, as specified in the AASHTO, 2004 (Section 11.10.6.4.2).

The service life of the structure must be taken as a minimum of 75 years for permanent MSE walls, 36 months for temporary walls, and more than 100 years for permanent walls for which its failure would have severe consequences.

For geosynthetic reinforcement, the long term design strength can be calculated as:

\[ T_{al} = \frac{T_{ult}}{RF} \]  \hspace{1cm} (2.15)

where:

\[ T_{al} = \text{Nominal long-term reinforcement design strength (kips/ft).} \]
\[ T_{\text{ult}} = \text{Minimum average roll value (MARV) ultimate tensile strength (kips/ft).} \]

\[ RF = \text{Combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging (dim.).} \]

\[ (RF = RF_{ID} \times RF_{CR} \times RF_{D}) \quad (2.16) \]

where:

\[ RF_{ID} = \text{Strength reduction factor to account for installation damage (dim.) (Given by the manufacturer).} \]

\[ RF_{CR} = \text{Strength reduction factor to prevent long-term creep rupture of reinforcement (dim.) (Given by the manufacturer).} \]

\[ RF_{D} = \text{Strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim.) (Given by the manufacturer).} \]

The use of geosynthetic reinforcement to MSE walls design and construction is limited to some wall applications, soil conditions and polymer type to be able to assure that the degradation due to environmental factors will be minimal. Further product-specific durability studies shall be carried out to determine the short and long term properties of the geosynthetic if the circumstances considered safe under the AASHTO, 2004 and WSDOT, 2006 are not met.

**Pullout resistance**

The reinforcement pullout resistance should be checked in every level to prevent pullout failure. The pullout resistance of the reinforcing elements must be developed behind the potential failure surface (Figure 2.2), which is calculated as an effective pullout length of the reinforcement, and is limited to not less than 3 ft. The total
reinforcement length is equal to \( L_a + L_e \) and should be more than 0.7 the height of the wall, as shown in Figure 2.2. \( L_a \) depends of the type of reinforcement, thus of the potential failure surface. Both internal stability methods use the same approach.

The effective pullout length shall be determined using the following equation:

\[
L_e \geq \frac{T_{\text{max}}}{\Phi F^* \alpha \sigma_v CR_c}
\]

where:

- \( L_e \) = Length of reinforcement in resisting zone (ft.).
- \( T_{\text{max}} \) = Applied factored load in the reinforcement.
- \( \Phi \) = Resistance factor for reinforcement pullout.
- \( F^* \) = Pullout friction factor (dim.)
- \( \alpha \) = Scale effect correction factor (dim.)
- \( \sigma_v \) = Unfactored vertical stress at the reinforcement level in the resistant zone (ksf).
- \( C \) = Overall reinforcement surface area geometry factor based on the gross perimeter of the reinforcement and is equal to 2 for strip, grids and sheet type reinforcements, i.e., two sides (dim.)
- \( R_c \) = Reinforcement coverage ratio as specified in the AASHTO, 2004 (Figures 11.10.6.4.1-1 and 11.10.6.4.1-1) \((R_c = \frac{b}{S_h})\) where \( b \) = unit width of reinforcement and \( S_h \) = Horizontal spacing).

\( F^* \alpha \sigma_v CR_c \) is the ultimate pullout resistance, \( Pr \), per unit reinforcement width.

The values for \( Pr \), or \( F^* \) and \( \alpha \) shall be determined from the product specifics’ pullout tests in the project backfill material or equivalent soil or they can be determined empirically or theoretically. Next section will explain the pullout testing procedures that the AASHTO requires for the estimation of these parameters.
If standards backfill materials as specified in AASHTO, 2004 (Article 7.3.6.3), with the exception of uniform sand ($C_u < 4$), is used, in the absence of test data, conservative values from Table 2.2 and Figure 2.8 can be used for $\alpha$ and $F^*$, respectively. If ribbed steel is been used and the $C_u$ is unknown at the moment of the design, it could be assumed as 4 to determine $F^*$.

From looking at Figure 2.8, it can be seen that Ribbed Steel Strips are the soil reinforcement type with most pullout resistance. Steel Grids, on the other hand, offer the least.

Table 2.2 Default Values for the Scale Effect Correction Factor $\alpha$ (AASHTO, 2004)

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>Default Value for $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Steel Reinforcement</td>
<td>1.0</td>
</tr>
<tr>
<td>Geogrids</td>
<td>0.8</td>
</tr>
<tr>
<td>Geotextiles</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Figure 2.8 Default values for the pullout friction factor $F^*$ (AASHTO, 2004).
Estimating Pullout Resistance

As discussed earlier, internal stability is evaluated against rupture of the reinforcement and pullout failure. The first component can be easily verified on the basis of the actual stress imposed on the reinforcement of known cross-sectional area and its allowable tensile stress. For the second component, on the other hand, the mechanism of failure has not been fully understood yet. This is especially true for reinforcing elements in which more than two components contribute to the pullout resistance, as opposed to steel straps or geotextile sheets, in which only the friction between the reinforcement and the soil offers resistance to pullout (Sampaco, 1996).

Pullout resistance is dependent upon many factors, such as soil-reinforcement interaction mechanisms, embedment length of reinforcement, overburden pressure and soil types. The soil reinforcement mechanisms are basically two: friction against longitudinal elements, and passive soil resistance against transverse elements and deformations (Figure 2.9). The frictional resistance is developed through contact between soil particles and the reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextiles and some geogrid layers.

Passive resistance is developed when a bearing surface in a different plane than the pullout force is pulled into the soil. Unlike frictional resistance, passive resistance is not well understood due to its complexity. Passive resistance is generally considered to be the primary interaction for rigid geogrids, bar mats, and wire mesh reinforcements.
The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance (Sampaco, 1996).

Figure 2.9  Soil-Reinforcement interaction mechanisms (FHWA, 2001).
Figure 2.10 Bearing failures of transverse bars in grid reinforcement (Anderson et al., 1995).

(a) Bearing Capacity Failure

\[ N_q = e^{\left(\pi \tan \phi'\right)} \tan^2 \left(45 + \phi'/2\right) \]

(b) Punching Shear Failure

\[ \theta_1 = \theta_2 = \frac{\pi}{4} + \frac{\phi'}{2} \]

\[ N_q = e^{\left(\pi/2 + \phi'\right) \tan \phi'} \tan \left(45 + \phi'/2\right) \]
For grid reinforcement, passive resistance of transverse bars has been estimated assuming either that the bars behave as strip footings rotated to the horizontal (Figure 2.10 (a)), or that it resembles the base pressure in deep foundations (Figure 2.10 (b)). From these assumptions, values of $N_q$ are estimated and included into Terzaghi’s general bearing capacity equation for calculating the passive resistance of the bars.

Because neither of the above mentioned mechanisms and factors can be accurately computed theoretically, the current design practice uses the pullout resistance data obtained from pullout tests conducted in a laboratory test cell or field prototype walls. Soil-interaction coefficients and pullout capacity equations have been developed from these data using different approaches, methods and evaluation criteria. These try to predict the pullout resistance as a function of overburden pressure, length and size of the reinforcement, the number of longitudinal and transverse elements, or the embedment length.

The above mentioned equations use different interaction parameters, and it is therefore difficult to compare the pullout performance of different reinforcements for a specific application. The AASHTO and FHWA have developed a normalized definition of pullout resistance for design and comparison purposes. This approach was mentioned on the previous section, where the pullout resistance, $Pr$, of the reinforcement per unit width of reinforcement was given by:

$$Pr = Le \times F^* \alpha \sigma_y CR_c$$  \hspace{1cm} (2.18)

Appendix A of the FHWA publication No. FHWA-NHI-00-043 about MSE walls and Reinforced Slopes Design and Construction guidelines, gives two procedures to
determine the pullout friction factor, $F^*$ and the scale effect correction factor, $\alpha$, from the above equation. The first procedure is empirical, in which $F^*$ is defined as:

$$F^* = \tan \rho + F_q \alpha_\beta$$

(2.19)

where:

- $\tan \rho$ = Apparent friction coefficient for the specific reinforcement
- $\rho$ = Soil-reinforcement interface friction angle.
- $F_q$ = Embedment bearing capacity factor
- $\alpha_\beta$ = Structural geometric factor for passive resistance

The determination of all these parameters is provided in Table 5 of Chapter 3 of FHWA (2001) with $\alpha$ estimated analytically using direct shear test data. However, due to the complexity of this analytical method, it is better to obtain $\alpha$ directly from pullout test data. If pullout test data is not available, a default value of 1.0 can be used for inextensible reinforcements, and a default value of 0.6 to 0.8 can be used for extensible reinforcements. In addition, some semi-empirical relationships for $F^*$ can be found in Chapter 3 of FHWA (2001).

The other procedure to determine $F^*$ and $\alpha$ is through experimental tests. Two types of tests are used to obtain pullout resistance parameters: the direct shear test, and the pullout test. The direct shear test is useful for obtaining the peak or residual interface friction angle between the soil and reinforcement material. ASTM D-5321 should be used for this purpose. In this case, $F^*$ would be equal to $\tan \rho_{\text{peak}}$ for sheet and smooth strip type reinforcement only. However, $\alpha$ cannot be derived directly but must be assumed or analytically derived.
Pullout tests are preferred over direct shear tests, since you can obtain $F^*$ and $\alpha$ values for all types of reinforcements, not only for the ones mentioned above. There is not an ASTM standard yet for pullout testing, but the AASHTO recommends using the test procedures GRI GG-5 and GRI GT-6, using the strain rate method. The pullout movement should be around 1 mm (0.04 inch) per minute. For extensible reinforcement, it is recommended that specimen deformation be measured at several locations along the length of the specimen in addition to the deformation at the front of the specimen. For all reinforcement materials, it is recommended that the specimen tested for pullout has a minimum embedded length of 24 inches.

In addition, it is recommended that for inextensible reinforcements, a maximum deflection of 3/4 inch measured at the front of the specimen be used to select $P_r$ if the maximum value for $P_r$ if the maximum value for $P_r$ or rupture of the specimen does not occur first. For extensible reinforcement, it is recommended that a maximum deflection of 5/8 inch measured at the back of the specimen be used to select $P_r$ if the maximum value of $P_r$ or rupture of the specimen does not occur first. It is also acceptable to use the same recommendation of the 5/8 inch measured on the back of the reinforcement for inextensible reinforcement as well. This allowable deflection criteria is based on a need to limit the structure deformations, which are necessary to develop sufficient pullout capacity.

Long-term pullout tests to assess soil/reinforcement creep behavior should be conducted when silt or clay reinforced backfill is being used. Soil properties and
reinforcement type will determine if the allowable pullout resistance is governed by creep deformations.

From the pullout test data, a normalized pullout versus mobilized reinforcement length curve should be established as shown in Figure 2.11. For this, the mobilized reinforcement length can be obtained in extensible reinforcement by using strain or deformation measuring devices attached to the reinforcement surface at various points back from the grips. A section of the reinforcement is considered to be mobilized when the deformation measuring device indicates movement at its end. For inextensible reinforcement, the mobilized length of reinforcement would be nearly equal to the embedded length; in other words, the deflections at the front and the back of the reinforcement specimen are closely the same. Tests must be run at several confining pressures to develop the $P_r$ versus $\sigma_v \cdot L_p$ plot. The value of $P_r$ to be plotted at each confining pressure versus $\sigma_v \cdot L_p$ is the lesser of either the maximum value (sustainable load) of $P_r$, the load which causes rupture of the specimen or the value of $P_r$ at the maximum deflection criteria.

Acceptable Deformations

Another requirement for the successful design and construction of MSE walls is the acceptable deformations the wall can undergo. This implies a limit to the deformations required to mobilize the working shear resistance of the soil, the working tensile resistance of the reinforcement and the pullout resistance of the reinforcement. Design considerations falling under this criterion are the integrity of the facing elements,
Figure 2.11  Experimental procedure for obtaining $F^*$ and $\alpha$ using pullout test data (FHWA, 2001).
the visual impact of the wall, the serviceability of the wall for supported structures, and the possible lead to progressive failure of the structure (Sampaco, 1996).

To satisfy this criterion, several design guidelines have been established by the design codes. One of them, as mentioned before, is the limit on the maximum amount of deformation at which the maximum pullout capacity of reinforcements is evaluated in the pullout tests, since it would also limit the amount of movement on full scale structures. Nevertheless, in extensible reinforcement large deflections occur at the front of the specimen during pullout testing compared to the deflection criteria applied to the back of the specimen. Since pullout tests do not model well the reinforcement deflections which occur in full scale structures, the deflection criteria of 5/8 inch applied to the back of the reinforcement specimen is acceptable and the full scale structure won’t be unstable (FHWA, 2001).

Other limitations are established by the AASHTO on selecting backfill material, the facing-reinforcement connection strength, especially on flexible walls and extensible reinforcement, and lateral wall displacements during construction.

Backfill material selection involves a restriction on the use of poor quality, fine grained soils (greater than 15 % passing Sieve No. 200 and plasticity index greater than 6). These types of soils delay the effective stress transfer during construction, since they are normally poorly drained, thus increasing the possibility of movement. In addition, post-construction deformations may occur due to an observed plastic behavior of this type of backfill material.
With respect to lateral wall displacements, no method is presently available to definitely predict lateral displacements, most of which occur during construction. The horizontal movements depend on compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-panel connection details, and details of the facing system. A rough estimate of probable lateral displacements of simple structures that may occur during construction can be made based on the reinforcement length to wall-height ratio and reinforcement extensibility as shown in Figure 2.12.

This figure indicates that increasing the length-to-height ratio of reinforcements from its theoretical lower limit of 0.5H to 0.7H, decreases the deformation by 50 percent. It further suggests that the anticipated construction deformation of MSE structures constructed with polymeric reinforcements (extensible) is approximately three times greater than if constructed with metallic reinforcements (inextensible) (FHWA, 2001).

The K-Stiffness method adds another limitation to its internal stability design: soil failure. Soil failure is considered when the soil reaches its peak shear strain, because at strains higher than that, the soil lowers its friction angle. It is only applied for extensible reinforcement, because only these could allow greater strains than the soil’s peak shear strain. By preventing soil failure, additional lateral movements on the wall are avoided since larger deformations would be needed for the development of the required lengths to mobilize reinforcement tensile strength and pullout resistance.
The K-Stiffness method states that the reinforcement strain ($\varepsilon_{\text{rein}}$) should be limited to the maximum soil strain which can be assumed for granular soils to be 3% for flexible faced wall and 2% for stiff faced wall.

Current Use of Steel in MSE walls

As mentioned in the introduction of this thesis, current practice suggests that the strength of steel is over conservatively used. Figure 2.13 shows the tensile stress of steel reinforcement normalized to its yield strength for different walls in the country. As observed, the steel reinforcement is being stressed to less than 30% of its yield strength.
The reason for this is the limitations on the allowable strength by design codes and the small pullout resistance obtained and the corrosion protection that require cross sections to be larger.

Figure 2.13 Current use of steel in MSE walls.
CHAPTER 3

THE PULLOUT BOX

Introduction

This chapter presents the details of the steel box used to conduct the pullout and tension tests made on the straight and crimped bars. The pullout box design and construction was a joint effort from Dr. James Bay, Professor in the Civil and Environmental Engineering Department at Utah State University, Bernardo Castellanos and the author, graduate students of the same department, and Ken Jewkes, in charge of the department’s mechanical shop.

The box was made out of 60 ksi steel and built in the Soil Structures Laboratory at Utah State University. It was almost 8 ft long, holding up about 10 ft³ of sand and designed to be able to resist pressures of up to 30 psi, which represents around 30 feet of soil overburden. The vertical pressure was applied through a rubber air bladder placed directly on top of the sand and secured inside the box. Pullout tests were performed on bars of up to 7.25 feet of length at different overburden pressures. In addition, the box had a removable bulk head located at 30 inches from the front plate that served as a reaction plate to carry out the tension tests.

In order to perform the tests, a general procedure was followed for each of the three (3) different test configurations: tension only, pullout only and a combined tension/pullout test, as explained later in the chapter. The displacement rate of the tests was controlled by pulling the bars with a screw jack attached to a variable speed
motor. The data acquisition system consisted of linear position sensors (LVDTs) located either at the front or back of the bars, and a 10 ton load cell connected to a computer with data acquisition software.

Box Components

Figure 3.1 shows a 3-D view of the pullout box, Figure 3.2 shows pictures of the box and Figure 3.3, 3.4, 3.5, and 3.6 are the plans showing the exact dimensions of the box and its pieces. In addition, in this section all the components of the box will be explained in detail and the construction process as well. In general, the box consisted of a closed section that was filled with sand and had the bar to be tested in the middle; the bar came out of this section and got attached to a load cell and then to a screw jack using different adapters for each bar type; the box was then pressurized and the bar was pulled with the screw jack, controlled by a variable speed motor, to carry out the different kinds of tests.

Steel frame

The main component of the box was the steel frame, made of ½ and ¼ inch 60 ksi steel plates welded together. The side plates of the box were ½ inch thick, 1 ft high, going 113 inches continuously from the back plate to the front plate where the screw jack and the motor were attached (Front Plate 3). The box could be divided into two sections: a back section, which was the closed box, where the sand was held and the bars were put into place; and a front section, that was open and was used to
Figure 3.1 3-D view of the pullout box.
Figure 3.2  Pictures of the pullout box: (a) completed pullout Box, (b) front section, (c) back section
Figure 3.3  Plan view of the pullout box.
Figure 3.4 Steel frame components: (a) front part plan view (b) middle part plan view (c) channels’ top and side view.
Figure 3.5  Front views of Front Plates 1 and 2 of the pullout box.
Figure 3.6  Front views of the Front Plate 3, Middle Plate & Bulk Head.
accommodate the proper attachments of the bars to the pulling and data acquisition systems, and made room for the bars as they came out of the closed section of the box (Figure 3.2). The back section, or the closed section of the box, was 88 inches long, 19 inches wide and was supported by six channels separated 8 inches from each other and with dimensions as shown in Figure 3.4 (c). The channels had ¼ inch slots used to keep the side plates in place before they were welded together. The front part of the box, which was the remaining 25 inches of the total length of the box and the same width, was not supported but remained stable thanks to the continuity of the side plates and the counter weight of the back section filled up with sand.

The back section consisted of the side plates, a ¼ inch thick bottom plate, a ½ inch thick back plate, two ¼ inch thick top plates and a ½ inch thick front plate (Front Plate 1). Figures 3.5 and 3.6 show the plans of all the individual plates, except for the ones than did not need machinery, and Figure 3.3, Figure 3.2 and Figure 3.1 (3-D view) show how they all came together. The bottom plate lied on the channels and was welded to the side plates on the inside. The back plate was welded to the bottom and side plates. The top plates were removable; they had handles and were placed on top of a rubber air bladder, which applied the pressure to the sand. Once the box was pressurized, these top plates were kept in place by six more channels on top of them which also had slots that connected to the side plates and were secured using fine-threaded rods that went from the top channels to the bottom channels through holes and were tightened on both sides with nuts.
The front plate of this section (Front Plate 1) had a 2 ½ x 2 ½ inch hole that allowed the bar to come out of the pressured box and get attached to the pulling device. It was also welded to the bottom and side plates. The inside of the box came to be 18 inches wide, 87 inches and 11 inches of height, holding up about 10 ft³ of sand.

A middle plate and a bulk head were also placed inside this section of the box to shorten it, allowing pullout testing of short bars and making a space between them where a linear position sensor (LVDT) was placed to measure the back of extensible bars (Figure 3.7 (a)) during pullout testing, as requested by the AASHTO. For the latter, two holes were drilled to one of the side plates between the middle plate and the bulk head to be able to reach the LVDT without opening the box while it was pressured (Figure 3.7 (b)). To pressurize the box, a steel plate needed to be placed on top of this middle section to allow the air bladder to go through. In addition, the bulk head served as a reaction plate where the bars would get tightened with nuts for tension testing. Figure 3.6 shows the plans for the middle plate and bulk head.

![Figure 3.7](image1.jpg)  
(a) Top picture of the middle section, (b) holes on the side plates between the Middle Plate and the Bulk Head.
The front section consisted of the side plate, which came continuously from the back section, the front plate of the closed section of the box (Front Plate 1), two front steel plates that held the motor and the screw jack in place (Front Plates 2 and 3), and two channels that were welded transversally to the front plate of the closed box and the front plates of this section (Figure 3.2 (b)). These channels were used to avoid buckling of the side plates when the loads were very high.

This section’s front plates (Front Plates 2 and 3) were held together with nuts and bolts that went through 2 ½ inch long slots on the last front plate (Front Plate 3) (Figure 3.6). These slots permitted the motor and the screw jack to move up and down and make any testing configuration suitable (Figure 3.8). The front plate of the closed section (Front Plate 1) also had a big enough hole to allow the bars to move up and down (Figure 3.5).

Figure 3.8 Picture of the motor and the screw jack attached to Front Plate 3.
Air bladder

As stated before, the air bladder was made of rubber. It was manufactured by The Perma-Type Company, Inc. It was almost the same dimensions as the inside of the box (87 inches long by 18 inches wide) and had an almost 3 ½ ft long hose in the middle that ended in an air valve. The bladder went on top of the sand, and its hose went through a hole on the top plate of the box (Figure 3.10). The air pressure was supplied by an air compressor with a digital control panel attached to the bladder’s air valve through a very long hose. Figure 3.9 shows the air control panel.

Figure 3.9  Air control panel.

Figure 3.10  Picture of the closed box with the hose coming out that supplied air to the rubber bladder.
Data acquisition system

Load cell

The load cell used was a 10 ton capacity load cell manufactured by Omega, model LC-702. It was 6.5 inches long, with 1 ½ inch diameter threaded rods on both ends. Two excitations were used to get a more accurate output when lower loads were applied to it: 0.01 v for loads lower than 10,000 lbs and 0.1 v for loads higher than 10,000 lbs. Figure 3.11 shows the load cell. The load cell was powered by a DC regulated power supply, and was turned on for at least 30 min before the tests to avoid any thermal effects on the load cell’s measurements.

![Load cell: (a) side view, (b) top view.](image)
Linear position sensors or linear variable differential transformers (LVDTs)

The Linear Position Sensors (LVDTs) were manufactured by GeoTac, model LS3. Their displacement range went from -0.05 to 3 inches. They were held in place by a magnetic base, rubber bands and spring clamps, and were located either at the back of the screw jack (measuring the front displacement of the bars) or at the back of the bars, depending on the test configuration. They were initially attached to the front of the bars directly with spring clamps for measurements (can be seen in Figure 3.2 (b)), but was discontinued to do due to bending found in the system. By measuring at the back of the screw jack, we were measuring the actual displacement of the system without any bending effects. Figure 3.13 shows a picture of the LVDTs.

Data acquisition software

The LVDTs and the Load Cell were connected to an ADIO module, which supplied the power from a DC regulated power supply, and sent the information to a computer through a Network module. Then, GeoTac’s GP data acquisition software was used to record the data. Figure 3.12 shows a picture of the system.

Figure 3.12  Part of the data acquisition system: (a) ADIO module, (b) network module, (c) computer with data acquisition software.
Figure 3.13  LVDT measuring (a) the displacement of the back of an extensible reinforcement and (b) the front of the reinforcement specimen.

Pulling system

Screw jack

The screw jack was an ActionJac Machine Screw Worm Gear Screw Jack, manufactured by Nook Industries, model 10-MSJ-UK. It had a clevis end, a little over 1 ft of total travel, a maximum load capacity of 10 tons, a torque needed to raise one lb of 0.0221 in-lbs and a longitudinal travel of one inch per 48 turns of the worm. During the tests, the screw jack’s worm (the back of the screw jack) was uncovered in order to make measurements of its displacement, as explained before. Figure 3.14 shows a picture of the screw jack.
Figure 3.14  Screw Jack used for pulling the bars: (a) drawing of the screw jack, (b) front of the screw jack, (c) back of screw jack (without the worm cover).

Motor

The motor was a variable speed motor with an attached controller (Figure 3.15). It could run backwards and forwards, and was attached to the screw jack through a set of sprockets meshed with chains that gave a torque increase of about 6.6, for a gear ratio of 6.6:1. This was done going from a 10 teeth sprocket to a 22, then shifting to 10 teeth on the same shaft, and finally changing to a 30 teeth sprocket (Figure 3.16). In addition, when the motor’s capacity had been overcome, a hand crank was attached to the motor’s spinning shaft to help it. The sprockets were covered with a wire mesh to prevent any accidents when running the tests.

Attachment of the bars to the pulling and data acquisition systems

Different bar sizes and types were used during this project. Therefore, many different pieces had to be built in order to connect these bars to the pulling and data
Figure 3.15  Variable speed motor.

Figure 3.16  Chains and sprockets.
acquisition systems. Figure 3.17 shows a picture of all the parts connected together. The connecting pieces showing consisted of the following:

- **Screw jack to load cell adapter:** Since the screw jack had a clevis end and the load cell a threaded 1 ½ inch diameter rod, in order to connect the two a piece had to be built that could have a pin going through itself and through the screw jack’s clevis on one end, and then a threaded hole that got screwed in by the load cell’s threaded rod on the other end. The pin was also used to assure that the bars were unloaded by taking it completely out during the tests. Figure 3.18 shows a picture of this piece and the pin.

- **Load cell to bar adapters coupler:** From the load cell, which had a 1 ½ inch diameter rod end, different bar sizes and types had to be attached. For this, a female coupler was built so that the different adapters for the bar sizes and types, which were pieces of a 1 ½ inches threaded rod, could get connected to the 1 ½ inch threaded rod end of the load cell. This way, only one coupler was used for connecting all the bars. It was 2 inches in diameter, 3.5 inches long and had a 1 ½ inches threaded hole that went all the way through (Figure 3.19).
Figure 3.17   Attachment of the bar to the pulling and data acquisition systems.

Figure 3.18   Load cell to screw jack connector: (a) side view, (b) top view, (c) Pin.
Figure 3.19  Coupler.

Figure 3.20  Round bar adapters. (a) top view, (b) side view.
• **Bar adapters:** The round bar sizes that were tested included: 5/8”, 1/2”, 3/8”, and 1/4” bars. In addition, a rectangular shaped RECO strap was also tested. To be able to build an adapter for each of the round bar sizes, a 1 ½ inch diameter threaded rod was cut into pieces and the different bar sizes were threaded into it. This way, the bars, also threaded, would just screw into these pieces. Figure 3.20 shows a picture of these different adapters. Furthermore, for the RECO strap, a special design was machined to a threaded rod piece so that the RECO Strap would attach to it using a pin (Figure 3.21). Once the bars were screwed into their corresponding adapter, this would then be screwed into the coupler, which would then connect it with the load cell and the screw jack (Figure 3.17).
Soil Data

As mentioned in the literature review, the backfill for MSE walls is recommended to be a well-graded cohesionless soil due to its drainage and stability. Following these guidelines, the soil used for the tests was a well-graded dry sand. Furthermore, to be able to model the compacting conditions used on full scale structures, the soil was compacted to a 95% of the AASHTO T 99 in dry conditions.

In order to obtain the properties of the sand used, such as unit weight, friction angle and the maximum dry density (AASHTO T 99), four tests were performed: Dry density, triaxial tests, standard proctor test (AASHTO, 2001b) and modified proctor test (AASHTO, 2001a). Both proctor tests were performed in order to compare how much increase in dry density is obtained using higher compaction energy.

**Dry density**

All the soil used for the tests was air dried by laying a small layer of sand on the floor and letting it stand, then turning it over from time to time until it was completely dry. Once it was dry, the dry density was determined by using a mold with a known weight and volume. The mold was filled with the sand and weighted, then the weight of the mold was subtracted and this was divided by the known volume of the mold. The dry unit weight found was 100 pcf.
Triaxial tests

In order to determine the friction angle of the soil, three triaxial tests were performed at three confining pressures: 5 psi, 10 psi and 30 psi. The soil was strained up to a 30 % strain. A peak friction angle of 46.08º, 44.74º and 42.22º were found at 5, 10, and 30 psi, respectively, for an average peak friction angle of 44.34º. The residual friction angle was about 38º. Figure 3.22 shows the results of the triaxial tests.

Maximum dry density

Since the sand for the tests was always going to be dry, the AASHTO T 99 and AASHTO 180 were only performed on the completely dry sand, without adding any water and getting moisture-density curves. Three tests were averaged for each of the compacting energies. The maximum dry densities found were 110.44 pcf for the AASHTO T 99 and 115.74 pcf for the AASHTO T 180. As noted, there was not much difference between the two compacting energies used, because there was no water added to the sample. Then, the 95 % of the AASHTO T 99 was selected as standard for the tests, which was 104.92 pcf. Table 3.1 shows the summary of the results from these tests.

Test Methodology

Three different types of tests were conducted inside the pullout box just presented: pullout, tension and a combination of tension and pullout. The pullout tests were performed to short (up to 30 inches long) and long bars (up to 87 inches long) to
Figure 3.22  Triaxial tests results.

Table 3.1  Maximum Dry Density Results

<table>
<thead>
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<th>AASHTO T 99</th>
<th>AASHTO T 180</th>
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<td></td>
<td>Test 1</td>
<td>Test 2</td>
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<td>Mold Weight (Kg)</td>
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<tr>
<td>Mold + Soil Weight</td>
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<td>5.96</td>
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<tr>
<td>(Kg)</td>
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<td>56.9438</td>
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<tr>
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<td>110.489</td>
</tr>
<tr>
<td>γave (pcf)</td>
<td>110.44</td>
<td></td>
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<tr>
<td>95%</td>
<td>104.92</td>
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</table>
determine the pullout resistance per embedded length of the bars. Tension tests were
performed to crimped bars to obtain force-displacement curves. A combination of
tension and pullout tests were performed also to the crimped bars to obtain the pullout
contribution of the crimps in the process of straightening.

Regardless of the type of test performed, there were some basic setup and test
procedures, which will be explained next.

**Preparing the bars**

The bars were supplied by the Hilfiker Retaining Wall Company. In order to be
able to pull the bars to test them, they had to be threaded (Figure 3.23). Some of them
were already threaded when they arrived, but most of them were not. For pullout tests,
the bars only needed to be threaded on the front, but for the tension tests they also had
to be threaded on the back to tight them up with nuts. For the combined
tension/pullout tests, the threads of the back of the bar had to be long enough to be
able to perform 5 pullout tests with 1 tension test in between each one without opening
the box, as explained later in Chapter 5.

The threads were made using pipe threaders for every bar size. Figure 3.24
shows a picture of the threaders.

**Filling the box**

The box was filled up in 4 layers of the same thickness. This was done to have
better control over the compaction of the soil in the box. For this, some marks were
placed on the inside walls of the box indicating where the layers of soil should level up
Figure 3.23  Threaded end of a rod.

Figure 3.24  Threaders for different bar diameters.

(Figure 3.25). This way, knowing the desired dry density of the soil, a weight of soil per layer was calculated, then this weight of soil was placed loose inside the box and it was compacted until the soil leveled up to the marks on the walls. Since the soil was sand, it was compacted just by hitting the sides of the box with a hammer. Figure 3.26 shows a picture of the hammer. The bar was placed in the middle of the box, during the placement of the second layer of sand.
In general, the process for filling the box was as follows:

1. *Divide the box into 4 equal layers and make marks on the wall.*

2. *Calculate the weight of sand needed per layer.* To obtain the desired dry density of the sand, which was 104.92 pcf for a 95% of the AASHTO T 99 (Table 3.1), and knowing the dimensions of the inside of the box, which were 87 inches long, 18 inches wide and 11 inches tall, then the weight came to be 261.48 pounds per layer, for a total of 1045.92 pounds.

![Figure 3.25 Leveling up the sand inside the box.](image)

![Figure 3.26 Large hammer used for hitting the sides of the box.](image)
Figure 3.27  Pictures showing: (a) the bar put in place in the middle of the box, and (b) sheet of rubber used to avoid the soil to come out of the front plate during the tests.

However, when the box was shorten to 30 inches for the tension, tension/pullout and pullout of short bars tests, the weight per layer was 90.17 pounds, for a total of 360.68 pounds.

3. **Place the soil per layer and compact it to the marks.** After calculating what was the weight per layer, 5 gallon buckets were used to weight the soil and deposit the correct amount of soil per layer. Then, the sand was compacted hitting the sides of the box with a hammer until it reached the marks on the side of the walls and was leveled. The same was repeated for the 4 layers of sand, stopping after the first layer to put the bar to be tested in place, as explained in the next step, before continuing with the other layers.

After a few tests, it came to notice that by just hitting the box three times with the hammer between channels on each side of the box (15 times on
each side for long bar tests, 6 times for short bar tests) was sufficient to get the amount of compaction desired. This remained consistent through every test.

4. *Put the bar to be tested in place.* After the first layer of sand was compacted, the bar was screwed in and held in place in the middle of the box while the second layer of sand was deposited. This was done to get better compaction of the sand around the bars. Holding the bar was not necessary for the tension tests because the bars went from the front of the box all the way through to the back of the bulk head and remained supported.

Before the bar was screwed in place, it had to go through a small aperture on a sheet of rubber placed in the inside part of the front plate of the closed box. This sheet of rubber did not allow the sand to come out of the box during the tests (Figure 3.27). At this point, it was best to screw just the bar adapter to the bar first and then rotate the bar and the adapter together to screw them into the coupler, which was already connected to the load cell. After the bar was screwed in properly, then the screw jack was moved forward (moving the bar forward too) to put the bar as far back as the screw jack could go to get as more travel on the front of the box as possible. The remaining layers of sand were placed and compacted afterwards.
Closing and pressurizing the box

After filling the box, the air bladder was laid down on top of the sand. If a pullout test of an extensible bar, such as a crimped bar, was going to be performed, before putting the air bladder the LVDT that measures the back of the bar had to be put in place inside the space between the middle plate and the bulk head, as described in the next section and shown in Figure 3.13 (a).

After the air bladder was in place, the two top plates of the box were positioned, and then the channels were set up on top of these and the fine-threaded rods got tightened with nuts. It had to make sure that the hose of the air bladder was out of the box through the hole on the top plate (Figure 3.10), and that there was a big enough gap between the top plate and the top layer of sand so that the air bladder had enough room to fill itself up and that the top plate was not applying any pressure on the sand. If there was not a big enough gap, some sand was removed from the top layer of sand to avoid the top plate from applying any unwanted pressure to the sand.

Once the box was closed, all the nuts tightened and the box secured, the air bladder’s hose was connected to the air panel and the box was pressurized to the desired pressure.

Setting up the pulling and data acquisition system

LVDTs

Depending on the test that was being carried out, the location of the LVDTs changed. As stated before, for the pullout tests of long and short bars, the LVDT was
placed at the back of the screw jack, measuring the displacement of the front of the bars (Figure 3.13 (a)), and for the pullout tests of the crimped bars it was placed at the section between the middle plate and the bulk head to measure the displacement of the back of the bar.

The LVDTs were held in place by a magnetic base, rubber bands and spring clamps. The spring clamps were used to hold the tip of the LVDT at the back of the bar together with the bar. Initially, the LVDT at the front was also attached to the bar with spring clamps as shown in Figure 3.2b, but due to observed bending of the bars at this location, it was switched to measuring the displacements at the back of the screw jack, which gave more accurate measurements of total displacements.

Once the LVDTs were in place, a zero was taken in the data acquisition software for each of the LVDTs. Also, the calibration factors given by the manufacturer of the LVDTs had to be input into the program correctly to be sure the LVDTs were measuring properly.

**Load cell**

As mentioned before, the Load Cell had to be connected to power well in advance the beginning of the tests to avoid any thermal effects affecting the measurements. Also, the calibration factors given by the manufacturer had to be input correctly into the program. Then, a zero value was taken by removing the pin from the piece that connected the screw jack to the load cell (Figure 3.17), which assured that the load cell was completely unloaded.
In addition, the excitation that went through the load cell had to be set in the program to 0.01 v for loads lower than 10,000 pounds and to 0.1 v for loads above 10,000. Doing this gave more accurate measurements of lower loads.

**Motor**

The motor had a controller to adjust its turning speed. It also had two directions: backward and forward. When put to forward, the screw jack pulled on the bar, and when put in backward position, the screw jack either unloaded the bar or pushed it into the box. To start the motor, the green button on Figure 3.15 was pushed, making sure the AC switch was on. Then to stop it the red button had to be pushed.

**Running the tests**

Once the box was pressurized, the load cell and the LVDTs were in place, zero values had been taken, then a name was assigned to the test in the data acquisition program and the motor was put in forward position for the screw jack to start pulling and get the test started. During the tests, the displacements and the loads were monitored through the computer’s monitor. This way, and adjusting the motor’s speed, the displacement rate could be controlled, which was tried to kept under 0.04 inches per minute for the pullout tests. The tension tests were run a little bit faster, since it was not critical to run them slow. The data acquisition program took a reading from the load cell and the LVDTs every second and stored it.

Depending on the type test being performed, the bar was either pulled until a desired displacement was reached, such as when doing pullout tests, or until an specific
load was applied, such as when doing tension tests or straightening a crimp to a certain load. After the desired displacement or load was reached, the motor was put to backward position to unload the bars until the monitored load was close to zero and then the pin was pulled to make sure the system was completely unloaded.

When the combined tension/pullout test was being done, as will also be explained in Chapter 5, after unloading the system the test configuration was changed to either tension or pullout. This means that if a tension test had finished, the nut on the back of the bar was removed and the LVDT was repositioned there through the side holes of the box. This was done because the pressure remained the same and the box remained closed. Then, the data acquisition system was reset and the test was run again. On the other hand, if a pullout test had just finished, then the LVDT at the back of the bar was removed and the nut was screwed in and tightened through the side holes of the box. The data acquisition system was then reset and the test was rerun.

When a pullout test alone was being done, as will also be explained in Chapter 4, after unloading the system, the pressure was increased, the data acquisition system was reset and the test was run again.

Once all the tests had been done on a bar, or a bar had been tensioned to failure, the box was unpressurized and opened, the three top layers of the box were completely removed, and the bar was taken apart. The pullout box was then refilled, a new bar was placed and the entire setup and testing procedures were repeated.
CHAPTER 4
STRAIGHT BAR PULLOUT BEHAVIOR

Introduction

As described in the literature review, the internal stability of MSE walls is checked against two failure criteria: tension failure or rupture of the reinforcement and pullout failure (Figure 2.5). The first one can be easily evaluated by comparing the maximum tensions on the wall to the tensile capacity of the reinforcement given its cross-sectional area and the allowable yield strength of the reinforcement material. For steel reinforcement systems, one important consideration when doing this evaluation is accounting for the effective cross-sectional area of the reinforcement after corrosion. Since corrosion affects the thickness of the reinforcement in every direction, for the same corrosion depth, reinforcement with a square cross section, such as a steel strap, would have a greater loss of cross-sectional area than a round bar with same original cross section area, as shown in Figure 4.1.

The second internal mode of failure depends on the soil-reinforcement interaction mechanisms in which pullout resistance is developed. These mechanisms depend on many factors, but are essentially two: friction and passive earth pressure resistance. Friction depends mainly of the interface between the soil particles and the reinforcement surface. Passive resistance, on the other hand, is dependent upon bearing elements normal to the direction of the force. Both mechanisms are very
complex and are not yet fully understood. Thus, pullout resistance cannot be accurately computed theoretically, but rather estimated from pullout test data.

Among current types of steel reinforcement, ribbed steel straps are the ones that have the highest pullout capacity (Figure 2.8). This is mainly because they have more surface area than the others, thus creating more interface friction, and have deformations that generate some significant passive resistance.

From the above, it could be said that the ideal type of steel reinforcement to use would be one with a round cross-sectional area, thus less affected by corrosion, and as much pullout resistance as a ribbed steel strap.

This chapter presents a new type of round deformed bars that can create as much pullout resistance as straps having closely the same cross-sectional area. The pullout capacity of this type of bar was tested and compared with the pullout capacity
of a RECO strap (ribbed steel strap). In addition, for comparison purposes smooth bars were also tested for pullout capacity, as well as rebar (structural steel bars).

The pullout capacity of the rebar also needed to be evaluated for the new internal stability method presented in Chapters 5 and 6, in which MSE reinforcement consists of series of crimps made on steel rebar. The rebar were selected for this due to its huge availability and because they were found to have a considerable pullout resistance. American and European types of rebar were tested for comparison, as both share the same availability, but have different deformations on them that indicated different pullout capacities.

Pullout resistances were evaluated using the pullout box and the test procedures described in Chapter 3, under six different confining pressures: 5 psi, 10 psi, 15 psi, 20 psi, 25 psi, and 30 psi, except for the 5/8” and 1/2” diameter rebar, which were tested only for 3.3, 10, and 30 psi, and 5, 10, 20, and 30 psi, respectively.

Methodology

The procedures described in Chapter 3 for filling out the box, placing the bar, setting up the data acquisition system, pressurizing the box and running the tests were applied. In general, the procedure consisted of the following: One bar was put inside the box, the box was pressurized to the first confining pressure and then the bar was pulled until 3/4 inches of displacement in order to obtain its pullout resistance; once this test was over, the system was unloaded, the pressure was increased and the bar was pulled again at the new confining pressure; this was then repeated for all confining pressures.
Since the bars tested consisted of straight, inextensible steel reinforcement, the displacements were only measured at the front of the bars and were pulled to 3/4” inches of displacement, as requested by the AASHTO and mentioned in Chapter 2.

Plots of Pullout Resistance vs. Displacement were created using the information from the load cell and the LVDT located at the back of the screw jack. These plots were then divided by the embedded length at each displacement point to get a Pullout Resistance per embedded length vs. Displacement plot of each bar at each confining pressure. Then, to be conservative, the maximum pullout resistance per embedded length of reinforcement was taken as the value at 3/4 inch displacement of the front of the bars, even if there was a higher value before this point.

In addition, the pressure applied by the weight of the two layers of sand above the bar was accounted for in the calculations and specified in the plots. This pressure corresponded to 0.2986 psi or 43 psf.

For comparison purposes, plots with the pullout resistances of bars with the same cross-sectional area were also created at certain confining pressures. For instance, the plots for the pullout resistance per unit length at 30 psi (4363 psf) of the RECO strap, 5/8” smooth bar, 5/8” rebar and 5/8” bar with washers, which have the same cross-sectional area of 0.3 in², were put together in one. Furthermore, the ratios of pullout resistance of deformed bars over pullout resistance of smooth bars of the same cross-sectional area were calculated to better appreciate the effects of the deformations.
To finalize, a general plot which includes the maximum pullout resistances of all the bars tested at every confining pressure was elaborated and the results shown on the plot were analyzed.

Bars Tested

Five types of bars were tested: a steel strap (RECO strap), smooth round bars, round bars with washers spaced 2 inches from each other, traditional American rebar, and European rebar.

RECO straps

RECO straps are hot rolled ribbed steel strips. They are one of the most common type of soil reinforcement, and certainly the one that generates more pullout resistance. This type of soil reinforcement was patented by The Reinforcing Earth Company, from which it gets its name, RECO. They have a yield strength of 60 ksi and a square cross section with nominal dimensions of 2 inches wide and 0.15 inches thick, for a cross-sectional area of 0.3 in$^2$. They are hot rolled with transverse ridges that alternate separations of 6 inches and 2 inches on both sides.

Their pullout resistance comes from friction on its extensive surface area and from the passive resistance generated by their transverse ridges. Due to their superior pullout resistance compared to other soil reinforcement, this bar was tested first and was used as a point of comparison. Figure 4.2 shows a picture of the RECO strap tested.
Smooth bars

The smooth bars were tested to be able to compare its pullout resistance with the others and appreciate how much increase comes from the improved deformations on the bars. For this, 5/8” and 1/4 “diameters were tested. These sizes were selected predicting that they would be required for the bottom and the top part of a MSE wall, respectively. Figure 4.3 shows a picture of one of the smooth bars tested.

Round bars with washers spaced 2”

The round bars with washers consisted of conventional washers (3/16 “ thick and 7/16”) of outside diameter, welded to the surface of smooth bars every 2 inches. The bars tested were of 5/8” and 1/4” in diameter. These bars represented one of the main objectives of this research. They are a new type of soil reinforcement, with a
different design for more pullout capacity, that were thought to be able to surpass the pullout capacity of the RECO straps in the 5/8” diameter size, which has the same cross-sectional area. If so, they would be preferred over the straps due to their lower loss in cross-sectional area from corrosion. Figure 4.4 and 4.5 show pictures of these round bars with washers.

Figure 4.3 Picture of a 5/8” diameter smooth bar.

Figure 4.4 Picture of 5/8” diameter bar with washers spaced 2”. 
Rebar

The rebar consisted of two types: American and European, and were tested in 5/8”, 1/2” and 3/8” diameters. These bars are hot rolled steel bars with deformations, primarily used as structural steel. They have very similar shape, differing in that the American rebar have deformations in about a 45 degree angle from the horizontal, while the European bars have them totally perpendicular. Here, they were tested for pullout resistance as part of the development of a new internal stability method presented in Chapters 5 and 6. In this method, the rebar would be combined with crimps made on them, which would also contribute to the pullout resistance, to become a new type of soil reinforcement. Additionally, the 1/2” and 3/8” diameters were tested expecting they would be required for the middle section of a MSE wall. Figure 4.6 and Figure 4.7 show pictures of the two types of rebar tested.
Figure 4.6  Picture of 5/8” diameter American rebar.

Figure 4.7  Picture of a 5/8” diameter European rebar.
Pullout Tests Results

In this section, the results of the pullout tests for each of the bars tested are individually presented, showing for each bar the results for all confining pressures together. The order in which the results are presented is in groups of bars of similar cross-sectional area in descendent order (from larger to smaller area). This way, a better appreciation of the pullout resistance, under the same steel requirements, is assessed. The units of the Pullout Resistance are given in lbs/in.

RECO strap

Figure 4.8 shows the pullout resistance of the RECO strap tested under different confining pressures.

![Graph](image)

Figure 4.8 Pullout resistance per unit length of a RECO strap.
5/8” diameter bars

Figures 4.9, 4.10, 4.11, and 4.12 show the pullout resistances of the different 5/8” diameter bars tested under different confining pressures.

Figure 4.9 Pullout resistance per unit length of a 5/8” diameter smooth bar.

Figure 4.10 Pullout resistance per unit length of a 5/8” diameter bar with washers spaced 2”.
Figure 4.11  Pullout resistance per unit length of a 5/8” diameter American rebar.

Figure 4.12  Pullout resistance per unit length of a 5/8” diameter European rebar.
1/2” diameter bars

Figures 4.13 and 4.14 show the pullout resistances of the different 1/2” diameter bars tested under different confining pressures.

Figure 4.13  Pullout resistance per unit length of a 1/2” diameter American rebar.

Figure 4.14  Pullout resistance per unit length of a 1/2” diameter European rebar.
**3/8” diameter bars**

Figure 4.15 shows the pullout resistances of the 3/8” diameter American rebar tested under different confining pressures.

![Figure 4.15](image)

**1/4” diameter bars**

Figures 4.16 and 4.17 show the pullout resistances of the different 1/4” diameter bars tested under different confining pressures.

![Figure 4.16](image)
Analysis of Results

In this section, comparison plots between the pullout resistances of bars of the same cross-sectional area are presented and discussed. Then, a general plot with the maximum pullout resistances per unit length of all the bars tested, taken as the value at 0.75 in of displacement, at every confining pressure is also exposed and analyzed.

Comparison plots

To be able to compare the pullout resistances of the bars of the same cross-sectional area, they were plotted together at 10 psi and 30 psi. This way, the pullout resistances of the bars could be compared at low and high confining pressures. In addition, the ratios of the maximum pullout resistances per unit length at 0.75 inches of...
displacement of each bar to the pullout resistances of the smooth bars with the same cross-sectional area are also presented in tables. This was done to see how much improvement is obtained with the different soil-reinforcement mechanisms of the bars.

5/8” diameter bars

From Figures 4.18 and 4.19, it can be appreciated that the 5/8” bar with washers has a greater peak pullout resistance than the RECO strap at 10.3 psi, and closely the same at 30.3 psi. It is also noted that after the peak, both curves from the 5/8” bar with washers and the RECO Strap start to descend and at approximately 0.75 inches of displacement they come together, for a maximum pullout resistance at this point of around 32 lbs/in at 10.3 psi and 60 lbs/in at 30.3 psi.

Figure 4.18 Pullout resistances of all tested 5/8” diameter bars at 10.3 psi.
The reason for this behavior may come from the different soil-reinforcement mechanisms of both bars. The RECO strap is mostly dependant on the friction generated by its extensive surface area, while the round bars rely on the passive resistance generated by the washers as they push through the soil. Nevertheless, it is demonstrated here that the round bars generate as much resistance as the RECO strap.

Also observed in the figures above is that the two types of rebar yield very close results at low confining pressures but differ quite a bit at high confining pressures. It seems that having the deformations in an angle, as in the American rebar, generates more friction and pullout capacity than having them perpendicular, with their difference becoming more significant as the confining pressure increases.
On Table 4.1, the ratios of the pullout resistance of the bars with deformations to the pullout resistance of the smooth bars are presented. It can be seen that the bars with the washers can generate as much as 6 times the pullout resistance of the smooth bars. In addition, the rebar can only generate a little more than double the smooth bar’s pullout.

1/2" diameter bars

Figures 4.20 and 4.21 show the comparison between the 1/2" American and European rebar. The same behavior of the 5/8” rebar is observed. The rebar have the same values at 10.3 psi, but differ quite a bit at 30.3. From this, it is confirmed that the American rebar have a higher pullout resistance than the European rebar, regardless of the bar size. Thus, the 3/8” rebar comparison should show the same results and won’t be compared.

Table 4.1 Ratios of Pullout Resistance Per Unit Length of Deformed Bars to Pullout Resistance of Smooth bars

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<table>
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</tbody>
</table>
Figure 4.20  Pullout resistances of all tested 1/2" diameter bars at 10.3 psi

Figure 4.21  Pullout resistances of all tested 1/2" diameter bars at 30.3 psi
1/4” diameter bars

Figures 4.22 and 4.23 show the pullout test results of the smooth 1/4” bar and the 1/4” bar with the washers spaced 2”. As noted and shown in Table 4.2, the ratios of the pullout resistance of the bar with washers to the pullout resistance of the smooth bar at the two shown confining pressures are of an average of 6.5, even more that in the 5/8” bars.

Observed behavior of the washers

After performing the tests, and looking at the bars with the washers spaced at 2”, it was observed that the paint that the bars had originally was scratched on a large portion of the bar between the washers, with no paint being removed near the washers, as seen in Figure 4.4. It seems that a lot of friction was being generated in this location, away from the washers, indicating that the washers were not causing any effects to this
portion of the bar. It could then be inferred that if another washer was placed in between the washers, making the spacing only 1”, the pullout resistance should go up as more passive resistance would be generated by pushing more soil. However, it should be evaluated if getting more pullout resistance from these bars would be beneficial by shortening the design lengths but not pushing them below the allowable in the design codes.
General plot

To conclude, a final plot compiling the maximum pullout resistances per unit length of all the bars at every confining pressure is shown next. It is observed how the RECO Strap and the 5/8” bar with washers have closely the same pullout resistance, and also how the 1/4” bar with washers can generate more pullout resistance than bars with more than double its size, such as the 1/2” rebar and the 5/8” smooth bar. The units were converted to lbs/ft to make it easier to refer to when designing a MSE wall.
Figure 4.24 Pullout resistance per unit length of reinforcement of all bars tested.
CHAPTER 5
CRIMPED BAR TENSILE AND PULLOUT BEHAVIOR

Introduction

The main purpose of this research was to develop steel reinforcement for MSE walls that behaves similar to geosynthetics. The reason for this was first, that steel reinforcement is preferred over geosynthetic because steel reinforcement is less affected by environmental degradation and long term effects, such as creeping and strength softening, and taller structures can be built where the engineer has more control over stresses and deformations.

Second, that it has been demonstrated, through a plenty number of studies on full scale walls, that the loads exerted on the reinforcement in geosynthetic systems, considered as extensible, are significantly lower than in steel reinforcement systems, classified as inextensible. This is because extensible systems allow the wall to yield and the soil to reach the lateral strains needed to develop an active condition. Inextensible systems, on the other hand, allow little to no deformation, which induces at-rest pressures on the top of the wall, corresponding to almost half of the vertical pressures, when active pressures are around 0.2 to 0.3 the vertical pressures. The at-rest pressures at the top of the wall reduce linearly to about an active condition after the first 20 ft (Figure 2.6), due to some arching of the wall.

Then, in order to make steel reinforcement behave similar to geosynthetics, it had to somehow become extensible. This was done by producing crimps on the
reinforcement that would straighten as the loads were increasing. This way, the wall would move enough to get the soil to reach an active condition.

The first step of evaluating the new crimped steel reinforcement was assessing the correct shape of the crimps that would allow enough deformations while not compromising the wall stability, and would additionally completely straighten without braking, permitting the full use of the allowable strength of the steel. For this, the pullout box described in Chapter 3 was used, which tries to simulate field conditions and different soil overburden pressures.

Once an adequate shape of the crimps that would not affect the resistance of the steel was obtained, the tensile behavior of different sizes of crimps was evaluated at different confining pressures. Load-displacement curves were developed in order to be able to estimate how much deflection is obtained from each crimp at each layer of a MSE wall, and to evaluate the number of crimps necessary at each layer of reinforcement to develop an active condition when designing a MSE wall. Using these curves, the tensile behavior of the crimps could be combined with the pullout mechanisms of different types of steel reinforcement, such as welded wire mats or round bars with washers, and a steel reinforced MSE wall could be designed with loads as low as those in extensible systems.

After the tensile behavior of the crimps was assessed, the pullout resistance contribution of the crimps was then investigated. The reason for evaluating the pullout resistance of the crimps was to determine if it was significant enough to be able to rely only on them to prevent pullout failure. Then, the crimps would not only contribute to
deformations on the wall, but also to prevent pullout failure. Additionally, the pullout resistance evaluation was done on crimped rebar, which are structural steel bars. They were done on these bars because they can be easily obtained, and they offer an additional pullout resistance over just using smooth bars. This way, it is not only relied on the crimps for pullout resistance, but also on the straight bar pullout resistance of the rebar.

It was acknowledged that this pullout resistance would change when the crimps started deforming due to the increasing loads, and that the crimps would deform differently depending on the confining pressure, thus the pullout resistance was determined at different tensions and confining pressures on the bars until they were completely straight. By doing this, the effects of the confining pressure on the pullout contribution of the crimps were also assessed.

On the literature review it was shown that in order to develop an active condition on the wall, the top of the wall only needed to move around 0.2 to 0.5 % the wall’s height, which for most walls is less than 1 inch. Taking this into account, the height of the crimps, which would be essentially how much it would deform, had to measure around one inch. On the other hand, if higher pullout resistances were desired, the crimps would have to be bigger in order to generate more opposition to movement. Thus, if a crimped bar were to be used as MSE wall reinforcement, the two considerations mentioned were needed to be accounted for, and a balance had to be found where the deformations from the crimps were not too much and their pullout resistance was adequate. As a result, different heights of crimps were tested to be able
to decide later which one gives the best output whether adequate deflections or more pullout resistance is the governing design parameter.

In summary, different crimp shapes and sizes were evaluated to be able to come up with an adequate crimp shape. Then, the tensile behavior and the pullout resistance of the crimps at different tensions and overburden pressures were also determined. From this, a new internal stability design approach was then implemented using just a steel rebar with a series of crimps as MSE wall reinforcement. This new internal stability method is described in Chapter 6. Nonetheless, the tensile behavior of the crimps could be applied to any current steel reinforcement system to add compliance to the wall, not necessarily having to rely on them for pullout resistance.

Methodology and Sequence of the Tests

As mentioned before, the first step was to perform tension tests to crimped bars to assess if they fully straightened without braking, and to observe its behavior at different soil overburden pressures.

In order to perform the tension tests, the procedures described in Chapter 3 were followed. The crimped bars measured a little bit more than 30 inches, allowing it to be held with nuts on the back of the bulk head of the pullout box. Once the bars were in place, they were just pulled to failure, either with soil pressure around it, or in the air. At first, the confining pressures used were 5 psi, 10 psi, and 30 psi.

The first bars tested resulted in braking before they got straightened, both in air and soil. The soil and confining pressure effects on the deformations were observed to
be of very little significance. Thus, the subsequent tension tests were only done in the air to try and understand why they were braking.

After the adequate shape of the crimped bars was defined, and the decision of also evaluating the pullout resistance of the crimps on rebar was made, the tension tests were made in conjunction with pullout tests. As shown in Figure 5.1, since the distribution of the forces in the reinforcement is of a maximum tension at the failure plane and then decreasing as the bars offer opposition to movement, at a same overburden pressure, if the bar has different crimps on it, each of the crimps would be subjected to a different load, and thus they would have different pullout resistances.

Figure 5.1  Tension distribution for a crimped bar in a MSE wall.
Then, to try to model this behavior in a lab test and come up with the pullout resistances of each different crimp confined at the same overburden pressure, but with different loads on each one, thus deformed differently, the following procedure was followed:

1. The bar was put in place, the box was pressurized to 3.3 psi and a LVDT was located at the back of the bar.

2. With no tension on the crimped bar (0 lbs), a pullout test was performed.

3. After the pullout test finished, the bar was unloaded. Then, using the holes on the sides of the box, the LVDT on the back of the bar was removed and another one was placed at the front, and a nut was screwed in and tightened at the threaded rear end of the crimped bar to be able to pull on it. The box remained unopened.

4. The bar was pulled to a force at which the bar was stressed to 1/8 of half its yield (0.5 Fy). For instance, for a 1/2” diameter bar, 0.5 Fy = 5.890 lbs, then 1/8 = 736 ≈ 750 lbs.

5. Once the tension test was over, the bar was unloaded and another pullout test was performed with the crimp being deformed or tensioned to this load.

6. The same procedure was repeated for 1/4, 1/2, and 1 times half of the yield stress (0.5 Fy). Since the limiting stress on the bars was half their yield stress, the pullout resistance of the crimps was evaluated at 4 equally increased loads until half of the yield stress was reached.
7. Steps 1 to 6 were then repeated for 10 psi and 30 psi. The first confining pressure was 3.3 psi for the first set of crimped bars, but it was changed to 5 psi as explained later on.

8. The entire process was repeated for the different crimp heights and different bar diameters.

From the data obtained performing the described procedures, Load-Displacement curves of the crimps and Pullout Resistance per Crimp vs. Tension on the crimps curves were developed for different confining pressures. The Pullout Resistance of one crimp was defined as the difference between the pullout force obtained at 0.75 inches of displacement during the pullout tests of the crimped bars and the pullout force that the 30 inches of embedded length of straight rebar generate at the same displacement. It was defined this way to ease the calculations for pullout resistances and lengths when designing a MSE wall using crimped reinforcement. The pullout resistance of the crimped reinforcement would then just be the pullout contribution of the whole length of straight bar, as if there were no crimps, and the sum of the contributions of each of the crimps depending on the tension on them, as described in chapter 6. Furthermore, the pullout force at 0.75 was selected as the pullout resistance of the crimps and also the straight bars from recommendations by the FHWA for pullout tests of extensible and inextensible reinforcement explained in Chapter 2.

The bars tested were of 5/8", 1/2", 3/8", and 1/4" in diameter. The first bars tested were 5/8" and 1/4" smooth bars with a sharp bent, as will be seen later. As stated before, these bars broke due to their abrupt geometry, but served as a starting
point for finding an adequate bent and to look into the effects of confining pressure. At this point the pullout resistance of the crimps was not thought to be evaluated, but the subsequent set of crimped bar tested were made on rebar for this purpose. 5/8”, 1/2”, and 3/8” crimped rebar were tested for their tensile and pullout behavior.

Two crimp sizes were tested in the 5/8” diameter rebar, one considered large and another small relative to the other. They were tested at 3.3, 10, and 30 psi. From the data obtained from these two crimps, it was concluded that for the 5/8” diameter bars the pullout resistance contribution of the crimps tested were not significant enough to make it suitable for pullout design relative to the large loads they could withstand. However, if 5/8” crimped bars were to be used to add compliance to the walls combined with another mechanism for pullout, the load-displacement curves obtained of the crimps could be used to estimate the tensile behavior. Additionally, from the initial tests on the 5/8” diameter rebar at 3.3 psi, it was concluded that the pullout contribution of the crimps at this confining pressure was actually below the straight bar pullout resistance, meaning that at very low confining pressures the crimps created a gap between the bar and the surrounding soil that decreased the pullout resistance, and in that case it would be better to use just a straight rebar. From this moment on, the following bars were tested at 5, 10, and 30 psi, where a positive contribution from the crimps was found.

On the 1/2” rebar three crimp sizes were tested, having found a good pullout resistance contribution on the large and medium crimps but not on the small crimps. From the data on the 1/2" diameter bars it was determined that the confining pressure
is of little significance in the tensile behavior of the crimps, so the 3/8” diameter bars were only tested at 10 psi, which was an approximate average value. For this bar size only two crimps sizes were tested at 10 psi and the values for other confining pressures were estimated. Furthermore, for this bar size only two crimps were tested from the information obtained in the 1/2" diameter rebar that the small crimp size did not have a good pullout resistance contribution.

In brief, Load-displacements and Pullout Resistance per Crimp vs. Tension curves were developed for each crimp size and bar diameter at different confining pressures.

Presentation and Analysis of Tension Tests Results

In this section, the Load-Displacement curves for all the crimped bars tested are going to be presented. First, a CAD drawing and a picture of each crimped bar are shown, and then the load-displacement curves extracted directly from the data for each confining pressure. A piecewise load-displacement curve is then presented which compiles all confining pressures of each crimped bar. This piecewise load-displacement curve has been cleaned up and only selected points connected by lines are shown for a better appreciation and to make it easier to estimate the tensile behavior of the crimps.

**Crimped smooth bars**

The first set of bars tested were 5/8” diameter smooth bars with a sharp bent, as shown in Figure 5.3. Four different crimp sizes were tested, but just the larger and the smaller ones were analyzed under two different confining pressures: 15 and 30 psi. The
other ones were only tested in the air to see how they behaved and were plotted all
together to appreciate the effect of the crimp’s height. Figure 5.2 shows a picture of one
of the 5/8” smooth bars with these initial bents. Figure 5.3 shows a CAD drawing with
the dimensions of the four different crimp sizes tested, which were conveniently
classified as 2”, 1.5”, 1.25”, and 1.00” crimp. They were classified this way due to
approximate measurements of the crimps’ heights before they were tested or drawn,
but as seen in the drawings these were not their real heights. From the drawings, it can
be appreciated how the bars are bent closely to a 45° angle.

Due to stress concentrations and bending on the sharp bents, the 5/8” diameter
smooth bars failed at around half of their yield stress, as shown in Figure 5.4. They all
had the same type of failure and broke at the same spot, the middle break, regardless of
the confining pressure. Figure 5.5 and Figure 5.6 show the force-displacement curves of
the 1” and 2”, crimps respectively, tested at two confining pressures and without
confinement. It is observed that the confining pressure is of little significance for the

![Figure 5.2 Picture of 5/8” smooth bar with sharp bent.](image)
small crimp, while it becomes more significant for the large crimp but not enough to be considered a relevant factor affecting displacements. Also, the 1” crimp failed at a much higher load than the 2” crimp. This occurred because the 1” crimp straightened more than the 2” crimp, thus stress concentrations and bending effects were reduced and the bar’s strength was less affected.

Additionally, Figure 5.7 shows force-displacement curves of all the different crimp sizes tested on the 5/8” diameter smooth bars without any confinement. It can be noted that the crimps with shorter heights had less displacement but failed at higher loads than the crimps with higher heights due to the reasons explained before.

After the 5/8” smooth bars, 1/4” smooth bars with similar bends were tested. These ones had a different behavior though: they did not break at the bends but at the
threads on the back section of the bar used to tighten them to the bulk head. This means that the strength of the bar was not affected by the stress concentrations and the bending, probably because the smaller bars are more ductile. They were also classified as 1.25”, 1.00”, 0.625”, and 0.5 “crimps after the measurement of their heights before testing them. The shape of the crimps was proportionally the same as the 5/8” diameter bars, so they were not drawn. Figure 5.8 shows a picture of a set of 1/4” diameter crimped smooth bars. Figure 5.9 shows force-displacement curves of the four different crimp sizes tested without confinement. The same behavior of the crimps on the 5/8” bars is seen here again, the smaller crimp heights had less displacement but failed at higher loads.

Figure 5.4 Picture of 5/8” smooth bars with sharp bents after failure.
Figure 5.5  Load-displacement curves for the 1” crimp on a 5/8” diameter smooth bar at two different confining pressures.

Figure 5.6  Load-displacement curves for the 2” crimp on a 5/8” diameter smooth bar without confinement and at two different confining pressures.
Figure 5.7  Load-displacement curves for all crimp sizes tested on 5/8” smooth bars without confinement.

Figure 5.8  Picture of 1/4” smooth bar with sharp bent.
Figure 5.9 Load-displacement curves for all crimp sizes tested on 1/4" smooth bars without confinement.

**Crimped rebar**

After failing on getting the crimped smooth bars to fully straighten, a smoother bent was tried on rebar. The crimps were now bent using a radius of about 5 times the diameter of the bars, similar to how structural steel is bent. 5/8”, 1/2”, and 3/8” diameters were tested. As stated before, two crimp sizes were tested for the 5/8” diameter rebar, three crimp sizes for the 1/2” diameter and two crimp sizes for the 3/8” diameter. The 5/8” bars were tested at 3.3, 10, and 30 psi, while the 1/2” bars were tested at 5, 10, and 30 psi, and the 3/8” bars only at 10 psi, as explained before. Figure 5.10 shows a picture of a 5/8” diameter rebar with one of the large crimps. Figure 5.11 shows a CAD drawing with the dimensions of all the crimps sizes tested on the 5/8”
diameter rebar. Since the same crimp sizes were tested at different confining pressures, even though they were made the same way, they differed a little bit from one to another, and thus their dimensions’ deviation is annotated on the drawings. The crimps on these bars were classified as large and small.

Figure 5.10  Picture of Large Crimp on 5/8” diameter rebar.

Figure 5.11  CAD drawings of the different crimp sizes on the 5/8” rebar.

All units are in inches.
The next set of figures shows the load-displacement curves of the large and small crimps tested on the 5/8” diameter rebar. As mentioned, the same type of crimp, either the large or the small, was tested at 3.6, 10.3, and 30.3 psi. Additionally, the tests were combined with pullout tests made at specific tensions, as explained before; therefore the load-displacement curves are segmented. This means that load-displacement curves of the load-unload process made for each tension were put together to get a complete load-displacement curve of the crimped bar as if no interruptions had been made. Then, Figure 5.16 and Figure 5.21 show the plots of load-displacement curves compiling every confining pressure tested for each crimp type, large or small. These curves were made as piecewise line curves to make it easier to estimate the tensile behavior of the crimps.

![Figure 5.12 Load-displacement curve for Large Crimp on 5/8” diameter rebar without confinement.](image)
Figure 5.13  Load-displacement curve for Large Crimp on 5/8” diameter rebar at 3.63 psi.

Figure 5.14  Load-displacement curve for Large Crimp on 5/8” diameter rebar at 10.3 psi.
Figure 5.15  Load-displacement curve for Large Crimp on 5/8” diameter rebar at 30.3 psi.

Figure 5.16  Load-displacement curves for Large Crimp on 5/8” diameter rebar at all three confining pressures and without confinement.
Figure 5.17  Load-displacement curve for Small Crimp on 5/8” diameter rebar without confinement.

Figure 5.18  Load-displacement curve for Small Crimp on 5/8” diameter rebar at 3.63 psi.
Figure 5.19  Load-displacement curve for Small Crimp on 5/8” diameter rebar at 10.3 psi.

Figure 5.20  Load-displacement curve for Small Crimp on 5/8” diameter rebar at 30.3 psi.
Figure 5.21  Load-displacement curves for Small Crimp on 5/8” diameter rebar at all three confining pressures and without confinement.

The next set of figures show the load-displacement curves for the three crimp sizes tested on the 1/2” rebar at three different confining pressures: 5 psi, 10 psi, and 30 psi. They were classified as Large, Medium, and Small crimps. First, a picture of a 1/2” rebar with the Large Crimp is shown (Figure 5.22), and then a CAD drawing with the dimensions of the three types of crimps (Figure 5.23). As with the 5/8” rebar, since the same crimp size was repeated in order to be tested at different confining pressures, there were some differences in the dimensions, which are shown in the drawings. Figure 5.27, Figure 5.31, and Figure 5.35 show plots of the load-displacement curves of all confining pressures together of each crimp size. Again, these are piecewise line curves.
Figure 5.22  Picture of Large Crimp on 1/2" diameter rebar.

Figure 5.23  CAD drawings of the different crimp sizes on the 1/2" rebar.

All units are in inches.
Figure 5.24  Load-displacement curve for Large Crimp on 1/2” diameter rebar at 5.3 psi.

Figure 5.25  Load-displacement curve for Large Crimp on 1/2” diameter rebar at 10.3 psi.
Figure 5.26  Load-displacement curve for Large Crimp on 1/2” diameter rebar at 30.3 psi.

Figure 5.27  Load-displacement curves for Large Crimp on 1/2” diameter rebar at all three confining pressures.
Figure 5.28  Load-displacement curve for Medium Crimp on 1/2” diameter rebar at 5.3 psi.

Figure 5.29  Load-displacement curve for Medium Crimp on 1/2” diameter rebar at 10.3 psi.
Figure 5.30  Load-displacement curve for Medium Crimp on 1/2” diameter rebar at 30.3 psi.

Figure 5.31  Load-displacement curves for Medium Crimp on 1/2” diameter rebar at all three confining pressures.
Figure 5.32  Load-displacement curve for Small Crimp on 1/2” diameter rebar at 5.3 psi.

Figure 5.33  Load-displacement curve for Small Crimp on 1/2” diameter rebar at 10.3 psi.
Figure 5.34  Load-displacement curve for Small Crimp on 1/2” diameter rebar at 30.3 psi.

Figure 5.35  Load-displacement curves for Small Crimp on 1/2” diameter rebar at all three confining pressures.
At last, the load-displacement curves for the two crimp sizes on 3/8” diameter rebar are presented. These crimp sizes were classified as large and medium. Figure 5.36 shows a picture of a 3/8” diameter rebar with one of the large crimps, and Figure 5.37 the CAD drawing with the dimensions. These drawings do not show any deviation since the crimps were tested at only one confining pressure, 10 psi, as stated before. Figure 5.40 and Figure 5.41 show piecewise load-displacement curves of the large and medium crimps.

Figure 5.36   Picture of Large Crimp on 3/8” diameter rebar.

Figure 5.37   CAD drawings of the different crimp sizes on the 3/8” rebar.
Figure 5.38  Load-displacement curve for Large Crimp on 3/8" diameter rebar at 10.3 psi.

Figure 5.39  Load-displacement curve for Medium Crimp on 3/8" diameter rebar at 10.3 psi.
Figure 5.40  Piecewise load-displacement curve for Large Crimp on 3/8” diameter rebar at 10.3 psi.

Figure 5.41  Piecewise load-displacement curve for Medium Crimp on 3/8” diameter rebar at 10.3 psi.
Presentation and Analysis of Pullout Tests Results

In this section, the Pullout Resistance per Crimps vs. Tension of each crimp size tested on every bar diameter at different confining pressures are presented. First, the pullout test results directly from the test are shown for each confining pressure and at the given loads. Then, the Pullout Resistance per Crimp, as defined earlier in this chapter, is plotted vs. the tension on the crimp for each crimp size at each confining pressure on 5/8”, 1/2”, and 3/8” diameter rebar. In addition, the same Pullout Resistance is plotted vs. Vertical Stress for the given loads; in other words, the horizontal axis and the legend were switched to give another perspective of the results.

5/8” diameter rebar

Two crimp sizes were tested at 3.63, 10.3 and 30.3 psi. The procedure for the combined tension/pullout test explained earlier was applied, and a pullout resistance for each crimp while they straightened (at different loads or deformations until half of yield was reached) was obtained for the three confining pressures. The two crimp sizes dimensions are shown in Figure 5.11. Figure 5.45 and Figure 5.50 show the Pullout Resistance per Crimp vs. Tension for the two crimp sizes tested at three confining pressures. It is observed how the pullout contribution of the crimp was negative for 3.3 psi. This means that at this confining pressure the pullout obtained in the crimp was lower than for a straight rebar. For this reason, the pullout resistance on the next crimped bars was tested at 5 psi, 10 psi, and 30 psi.
Figure 5.42  Pullout test results for Large Crimp on 5/8” diameter rebar at 3.63 psi.

Figure 5.43  Pullout test results for Large Crimp on 5/8” diameter rebar at 10.3 psi.
Figure 5.44  Pullout test results for Large Crimp on 5/8” diameter rebar at 30.3 psi.

Figure 5.45  Pullout resistance per crimp vs. tension for Large Crimp on 5/8” diameter rebar at three confining pressures.
Figure 5.46  Pullout resistance per crimp vs. vertical stress for Large Crimp on 5/8” diameter rebar at different tensions on the crimp.

Figure 5.47  Pullout test results for Small Crimp on 5/8” diameter rebar at 3.63 psi.
Figure 5.48 Pullout test results for Small Crimp on 5/8” diameter rebar at 10.3 psi.

Figure 5.49 Pullout test results for Small Crimp on 5/8” diameter rebar at 30.3 psi.
Figure 5.50  Pullout resistance per crimp vs. tension for Small Crimp on 5/8” diameter rebar at three confining pressures.

Figure 5.51  Pullout resistance per crimp vs. vertical stress for Small Crimp on 5/8” diameter rebar at different tensions on the crimp.
1/2" diameter rebar

Three crimp sizes were tested at 5.3, 10.3, and 30.3 psi. The procedure for the combined tension/pullout test explained earlier was applied, and a pullout resistance for each crimp while they straightened (at different loads or deformations until half of yield was reached) was obtained for the three confining pressures. The three crimp size dimensions are shown in Figure 5.23. Figure 5.55, Figure 5.60, and Figure 5.65 show the Pullout Resistance per Crimp vs. Tension for the three crimp sizes tested at three confining pressures. It can be seen that for the small crimp, the pullout contribution is very little compared to the others.

![Figure 5.52 Pullout test results for Large Crimp on 1/2" diameter rebar at 5.3 psi.](image)
Figure 5.53  Pullout test results for Large Crimp on 1/2” diameter rebar at 10.3 psi.

Figure 5.54  Pullout test results for Large Crimp on 1/2” diameter rebar at 30.3 psi.
Figure 5.55  Pullout resistance per crimp vs. tension for Large Crimp on 1/2” diameter rebar at three confining pressures.

Figure 5.56  Pullout resistance per crimp vs. vertical stress for Large Crimp on 1/2” diameter rebar at different tensions on the crimp.
Figure 5.57  Pullout test results for Medium Crimp on 1/2\textquotedbl{} diameter rebar at 5.3 psi.

Figure 5.58  Pullout test results for Medium Crimp on 1/2\textquotedbl{} diameter rebar at 10.3 psi.
Figure 5.59  Pullout test results for Medium Crimp on 1/2” diameter rebar at 30.3 psi.

Figure 5.60  Pullout resistance per crimp vs. tension for Medium Crimp on 1/2” diameter rebar at three confining pressures.
Figure 5.61  Pullout resistance per crimp vs. vertical stress for Medium Crimp on 1/2” diameter rebar at different tensions on the crimp.

Figure 5.62  Pullout test results for Small Crimp on 1/2” diameter rebar at 5.3 psi.
Figure 5.63  Pullout test results for Small Crimp on 1/2” diameter rebar at 10.3 psi.

Figure 5.64  Pullout test results for Small Crimp on 1/2” diameter rebar at 30.3 psi.
Figure 5.65  Pullout resistance per crimp vs. tension for Small Crimp on 1/2” diameter rebar at three confining pressures.

Figure 5.66  Pullout resistance per crimp vs. vertical stress for Small Crimp on 1/2” diameter rebar at different tensions on the crimp.
3/8” diameter rebar

Two crimp sizes were tested only at 10 psi, while other values for the other confining pressures were estimated as will be explained in Chapter 6. The procedure for the combined tension/pullout test explained earlier was applied, and a pullout resistance for each crimp while they straightened (at different loads or deformations until half of yield was reached) was obtained. The two crimp size dimensions are shown in Figure 5.37. Figure 5.55, Figure 5.60, and Figure 5.65 show the Pullout Resistance per Crimp vs. Tension for the three crimp sizes tested at three confining pressures.

Figure 5.67  Pullout test results for Large Crimp on 3/8” diameter rebar at 10.3 psi.
Figure 5.68  Pullout resistance per crimp vs. tension for Large Crimp on 3/8” diameter rebar at 10.3 psi.

Figure 5.69  Pullout test results for Medium Crimp on 3/8” diameter rebar at 10.3 psi.
Figure 5.70  Pullout resistance per crimp vs. tension for Medium Crimp on 3/8” diameter rebar at 10.3 psi.
CHAPTER 6
DESIGN PARAMETERS

Introduction

In this chapter the parameters needed for MSE wall design are going to be developed from the data and the curves obtained on the pullout and tension tests. As stated before, the primary objective of this research was to be able to design steel reinforced MSE wall systems as extensible systems by allowing them to deform using crimps. The crimps make the walls less stiff permitting them to yield and work under active pressures instead of at rest pressures, thus lowering the working loads.

Then, in order to design MSE walls, explicit curves showing the tensile behavior of different crimps on different bar diameters and at different confining pressures had to be developed to be able to predict the amount of deformation a wall was going to be subjected to at each level of reinforcement. Additionally, the crimps were also evaluated for their pullout resistance while they straightened in order to optionally rely on them for pullout failure design. Thus, pullout resistance curves were also developed for the different crimps on different bar diameters, at different tensions and confining pressures.

Since it would be nearly impossible and extremely time consuming to evaluate every confining pressure and every bar diameter that could be used in a MSE wall in laboratory tests, values for a wider range of bar diameters and confining pressures were interpolated or extrapolated and estimated from the limited amount of testing
performed and exposed in the previous chapters. Furthermore, the tensile and pullout behavior of the different crimp sizes did not show any patterns that could be used for coming up with design equations as a function of either crimp height, tension on the crimps, bar diameter or overburden pressure; mainly due to the complexity of the straightening process of the crimps and their interaction with the soil surrounding them. As a result, the design curves presented here are only suitable for similar crimp geometries and bar diameters to the ones tested here and for similar soil properties. Additionally, these curves were only developed for a specific range of overburden pressures and tensions on the crimps. Nevertheless, they should be sufficient to be able to design MSE walls of up to 50 ft in height or more using crimped reinforcement.

The force displacement design curves of crimped reinforcement are going to be presented first, then the pullout resistance design curves of the crimped rebar and to finalize an overview of how to design MSE walls using just crimped rebar as reinforcement, in which the crimps are used to both add compliance to the wall and for pullout design.

**Force-Displacement Design Curves**

As seen in Chapter 5, the confining pressure is of little significance when it comes to estimating the deflection of the crimps. Therefore, for each crimp size, a single force-displacement curve was obtained by fitting it to the values for the three confining pressures tested. Below, these force-displacement curves for each crimp size and for each bar diameter are presented. Even though the tests were performed on rebar,
these displacement curves could be used for any smooth or deformed bars since the straighten process is not much affected by the deformations on the bar. Additionally, force displacement curves for 1/4” diameter smooth bars are included here, but these were extrapolated from observations on the other bar sizes tested and assuming similar crimp geometries.

Figures 6.1 and 6.2 show the Tensile Behavior Design Curves for the Large and Small crimps on the 5/8” diameter rebar; Figures 6.3, 6.4, and 6.5 for the Large, Medium, and Small crimps on the 1/2” diameter rebar; Figures 6.6, and 6.7 for the Large and Medium crimps on the 3/8” diameter rebar; and finally Figures 6.8 and 6.9 for the Large and Medium crimps on the 1/4” diameter smooth bars.

Crimp Pullout Resistance Design Curves

In this section, the crimp pullout resistance design curves for each crimp of each diameter size were developed for a large range of confining pressures and tension loads on the crimp. To be able to come up with the design curves, the plots of Crimp Pullout Resistance (obtained at 0.75 inch displacement) vs. Vertical Stress for different loads were used to interpolate to a larger range of confining pressures. Then, all these values were plotted with Crimp Pullout Resistance on the Y axis and Tension Load on the Crimp on the X axis, and lines were fitted for each confining pressure. The result was a plot where for each crimp type and bar diameter you could come up with the crimp pullout resistance given a tension load on the crimp and the confining pressure. Here only design curves for the Large and Medium crimps on the 1/2” and 3/8” diameter rebar are
Figure 6.1  Tensile behavior design curve for Large Crimp on 5/8” diameter rebar.
Figure 6.2  Tensile behavior design curve for Small Crimp on 5/8” diameter rebar.

For well-graded sand
- Friction angle: 37 °
- γ = 100 pcf

ALL UNITS IN INCHES
Figure 6.3  Tensile behavior design curve for Large Crimp on 1/2” diameter rebar.

For well-graded sand

- Friction angle: 37°
- $\gamma = 100$ pcf

Tension Force in the Crimp (lbs)
Displacement of the Crimp (in)

- 10.3 psi (1483 psf)
- 30.3 psi (1483 psf)
- 5.3 psi (783 psf)
Figure 6.4  Tensile behavior design curve for Medium Crimp on 1/2" diameter rebar.
Figure 6.5  Tensile behavior design curve for Small Crimp on 1/2” diameter rebar.
Figure 6.6  Tensile behavior design curve for Large Crimp on 3/8” diameter rebar.

For well-graded sand
- Friction angle: 37°
- $\gamma = 100$ psf

10.3 psi (1483 psf)
Figure 6.7  Tensile behavior design curve for Medium Crimp on 3/8" diameter rebar.
Figure 6.8  Tensile behavior design curve for Large Crimp on 1/4” diameter smooth bar.

For well-graded sand

- Friction angle: 37 °
- $\gamma = 100$ pcf
Figure 6.9  Tensile behavior design curve for Medium Crimp on 1/4” diameter smooth bar.

For well-graded sand

- Friction angle: 37˚
- ρ = 100 pcf
presented, as well as for the same crimp geometries on 1/4" diameter smooth bars, which were extrapolated from observations on the other bar sizes and assuming similar crimp geometries.

The reason for only developing design curves for the Large and Medium crimps was that for the small crimp size the pullout resistance was found not to be sufficiently large to be able to take advantage of it. Furthermore, the design curves for the 5/8” diameter rebar crimps were not developed because their pullout contribution was not significant compared to the large loads they would have to withstand to take full advantage of the strength of steel.

Since the pullout resistances of the crimps on the bars were only evaluated until half of their yield strengths, as explained in Chapter 5, then for the design curves it was assumed that after that the pullout resistance continued decreasing linearly to zero at a certain “critical load,” after which the crimp would not contribute more to pullout and one could only rely on the pullout resistance of the straight bar. For the 1/2” the critical load was assumed to be 8500 pounds, for the 3/8” bars 5000 pounds and for the 1/4” bars 2500 pounds. These critical loads were determined by simply following the trends of the pullout resistance curve fits until they intersected the axis at the “critical load.”

Figures 6.10 and 6.11 show the Pullout Resistance Design Curves for the Large and Medium crimps on the 1/2” diameter rebar; Figures 6.12 and 6.13 for the Large and Medium Crimps on the 3/8” diameter rebar; and Figures 6.14 and 6.15 for the Large and Medium crimps on the 1/4” diameter smooth bars.
Figure 6.10 Pullout resistance design curves for Large Crimp on 1/2" diameter rebar.

For well-graded sand  
Friction angle: 37°  
γ = 100 psf
Figure 6.11  Pullout resistance design curves for Medium Crimp on 1/2” diameter rebar.
Figure 6.12  Pullout resistance design curves for Large Crimp on 3/8" diameter rebar.
Figure 6.13  Pullout resistance design curves for Medium Crimp on 3/8” diameter rebar.

For well-graded sand
- Friction angle: 37 °
- γ = 100 psf
Figure 6.14  Pullout resistance design curves for Large Crimp on 1/4\" diameter smooth bar.

For well-graded sand
- Friction angle: 37°
- \( \gamma = 100 \text{ psf} \)
Figure 6.15  Pullout resistance design curves for Medium Crimp on 1/4” diameter smooth bar.
MSE Wall Internal Stability Design Using Crimped Reinforcement

In this section a brief description of how to use the design curves from Chapter 6 to evaluate the internal stability of a MSE wall is presented. More information on the new model for internal stability using crimped reinforcement can be obtained in Castellanos (2009).

The internal stability design consists of evaluating the safety of the reinforcement against rupture and pullout. Once the maximum working loads that are going to be exerted on the reinforcement layers of a wall have been calculated, after estimating horizontal and vertical spacing, the tensile design of the reinforcement on a MSE wall is made by limiting the yield stress of the steel to a given percentage and calculating a cross-sectional area that satisfies equilibrium. The reinforcement has got to be able to resist the loads without reaching a certain percentage of its yield strength. In addition, the reinforcement needs to be able to develop enough resistance along its length to avoid being pulled out of the soil mass.

The difference in the new internal stability method using crimped bars is going to be in calculating the loads on the reinforcement and estimating the pullout resistance of the crimps. The loads on the reinforcement are going to be reduced since the lateral earth pressure coefficients are going to be the ones corresponding to the active condition of the soil, because the wall is going to be yielding. The amount of deformation on the wall, and the fact that it yields, could be determined and verified by estimating the loads on each crimp of each layer of reinforcement and thus the
corresponding deflections for the given loads using the force-displacement curves obtained in this research. Then, the total deflection of the reinforcement would be to the summation of all the deflections of each of the crimps. The same is repeated for each reinforcement layer and added to obtain the total deflection of the wall. It has to be taken into consideration the reduction of tension longitudinally along a reinforcement layer as the forces are being taken by the pullout resistance of both the straight portion of the bars and the crimps; thus they tension decreases linearly through the straight portions of the reinforcement bar and as a step function at each crimp, moving away from the locus of maximum tension (Figure 6.16). In Castellanos (2009) examples of the design of walls using crimped reinforcement can be obtained, where the compliance of the walls can be observed with the calculated deflections, which clearly surpass what is needed to get active conditions on the soil.

The evaluation of the pullout resistance of the crimps is also made taking into account the linear decrease of the tensions forces from the point of maximum tension by the pullout resistance of the straight portion of the bar and then by the step reduction of the force from the contribution of each crimp to pullout resistance, as shown in Figure 6.16. This contribution of each crimp is going to be a function of the effective vertical stress and the tension in that crimp. It can be evaluated using the design curves obtained in Chapter 5. The location of the point of maximum tension is going to be depending on the failure surface of the wall, defined by assuming a rankine active failure plane behind the wall as it is done for extensible reinforcement (Figure 2.2
b). The pullout resistance can only be developed behind this failure surface, which is called the resisting zone. The moving part of the wall is called the active edge.

The total length of reinforcement needed in each layer is then going to be defined as the summation of the length of the reinforcement in the active edge for extensible reinforcement ($L_a$) and the length of reinforcement in the resisting zone ($L_e$) (Figure 6.16). The length of reinforcement in the resisting zone is determined by decreasing the tension at the locus of maximum tension linearly by the contribution of the pullout resistance of the straight portion of the bar as it moves away and then by a stepped value at each crimp until it becomes zero. It is defined as the length at which no more pullout resistance is needed. Since the pullout resistance of each crimp was defined as the difference between the resistance of the crimp including the straight portion of the bar and the pullout resistance of just the straight portion of the bar, when doing the analysis of the pullout resistance of the crimped reinforcement, the
contribution of the straight portion of the bar is calculated considering the total length of the bar, and then to that is added the pullout resistance of each of the crimps obtained from the design curves to come up with the total pullout resistance of each reinforcement bar.
Conclusions

In this research, a pullout box was designed and constructed to be able to perform tensile and pullout tests on straight and crimped steel reinforcement. From the straight bar pullout tests, the following was concluded:

1. Improved round bars can match pullout behavior of square bars (straps) with nearly the same cross-sectional area. Round bars have less cross-sectional area loss over time due to corrosion than square bars.

2. By welding washers spaced at 2” to round smooth steel bars, almost 5 times the pullout resistance of smooth bars is obtained.

Tensile tests were performed on different crimp geometries on 5/8”, 1/2”, 3/8”, and 1/4” diameter bars. From these tests, force-displacement curves were obtained for each crimp geometry and bar diameter at different overburden pressures. It was concluded that:

1. Crimps with a sharp bent broke at half of the yield strength of the steel. Stress concentrations and bending effects were the main reason.

2. The ideal crimp shape is one with a smooth radius of approximately 5 times the radius of the bar, similar to how structural steel is bent.

3. Crimp deflection is proportional to crimp height. The crimp is almost straight after stressed to half of its yield strength.
4. The tensile behavior of the crimps is not significantly affected by confining pressure.

5. Been able to almost fully straighten the crimps suggests that they would succeed in adding compliance to an MSE Wall.

Pullout tests on the crimped reinforcement were performed in between the tension tests, stopping at a given load and then pulling on the bar. The objective was to obtain the pullout resistance of the crimps in the straighten process, and at different confining pressures. From these tests, the pullout resistances per crimp for different crimp geometries at different tension loads on the crimp and under a range of confining pressures were obtained. It was concluded the following:

1. The combined Tension-Pullout Test procedure is very time consuming and can only be performed for a limited amount of tension on the crimps.

2. The pullout resistance of the crimps is a very complicated, nonlinear process. It is proportional to the crimp height, bar diameter, tension on the crimp and confining pressure.

3. At very low confining pressures, the pullout resistance of the crimp is lower than for a straight bar. This is because a gap is created that prevents contact between soil particles and the crimp.

4. Crimps can generally be relied on for pullout resistance. After reaching half of the yield strength, for practical purposes the pullout resistance of the crimps is zero.
Using the information from the tensile and pullout behavior of the crimps a new internal stability design approach was introduced at which the crimps can be used to add compliance to the wall and as pullout resistance mechanisms.

Nevertheless, the crimps could always be used in combination with other steel reinforcement that rely on a different pullout resistance mechanism, making them extensible to be able to decrease lateral stresses (i.e. crimps on welded wire mats or on round bars with washers).

In summary, from this research, three ways to significantly reduce steel on MSE walls were obtained:

1. Decreasing horizontal stresses using crimps.
2. Reducing corrosion losses using round bars.
3. Improving pullout resistance of round bars.

Recommendations

The following recommendations are given:

1. To continue to test different crimp geometries with different tension loads on and at different confining pressures to better understand the straighten process and the pullout resistance mechanism of the crimps.
2. To design MSE walls using crimped reinforcement in order to calculate how much deflection is expected using the different crimp geometries and evaluate which crimp geometry is more effective, i.e. sufficient pullout resistance contribution plus acceptable, not extreme, deflections.
3. To evaluate costs of using crimps in combination with other soil reinforcement, or just using crimped rebar alone to see which is the most cost effective.

4. To build a MSE wall using crimped reinforcement in order to obtain real data to calibrate and verify results and assumptions made in this research; such as having a Rankine active failure surface and the actual loads acting on the reinforcement.
REFERENCES


