Final Research Report

APPLICATION OF THE K_0-STIFFNESS METHOD TO REINFORCED SOIL WALL LIMIT STATES DESIGN

by

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16. ABSTRACT

A new design methodology for estimating reinforcement loads in reinforced soil walls, termed the K_0-Stiffness Method, has been developed. This new method has been demonstrated to more accurately estimate reinforcement loads and strains in reinforced soil walls than do current design methodologies. Step-by-step procedures are provided to lead the designer through the reinforced soil wall internal stability design process using this new methodology. These step-by-step design procedures have been developed with a limit states design approach consistent with current design codes (in North America this is termed Load and Resistance Factor Design, or LRFD). Specifically, consideration has been given to strength and serviceability limit states. Load and resistance factors, based on statistical data where feasible, have been developed for use with this method.

The results of examples from actual wall case histories were summarized and analyzed to assess how well the new methodology performs relative to current design practice. From this analysis of the design examples, the following was observed:

- For geosynthetic walls, the K_0-Stiffness Method has the potential to reduce required backfill reinforcement capacity relative to current design methodology by a factor of 1.2 to 3.
- For steel reinforced soil walls, the reduction in reinforcement capacity relative to what is required by current design methodology is more modest, on the order of 1.0 to 2.1.

Given these findings, use of the K_0-Stiffness Method can result in substantial cost savings, especially for geosynthetic walls, because of reduced reinforcement needs.

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EXECUTIVE SUMMARY

A new design methodology for estimating reinforcement loads in reinforced soil walls, termed the K_0-Stiffness Method, has been developed. This new method has been demonstrated (Allen and Bathurst, 2001) to provide significantly more accurate estimates of reinforcement loads and strains in reinforced soil walls than current design methodologies can produce. The final step in the development process is to apply this new method to reinforced soil wall internal stability design. Step-by-step procedures are provided to lead the designer through the design process with this new methodology. Because current national and international design specifications are moving toward a limit states design approach (in North America this is termed Load and Resistance Factor Design, or LRFD), these step-by-step procedures have been developed with this approach in mind, specifically with consideration for strength and serviceability limit states. Load and resistance factors, based on statistical data where feasible, are developed for use with this method.

Several design examples for both steel and geosynthetic reinforced soil walls are provided. Actual wall case histories are used where reinforcement load data are available. The results of these examples are summarized and analyzed to assess how well the new methodology performs relative to current design practice. From this analysis of the design examples, the following was observed:

- For geosynthetic walls, the K_0-Stiffness Method has the potential to reduce required backfill reinforcement capacity relative to current design methodology by a factor of 1.2 to 3.
- For steel reinforced soil walls, the reduction in reinforcement capacity relative to what is required by current design methodology is more modest, on the order of 1.0 to 2.1.
- For both types of soil reinforcement, as the wall becomes taller or as soil design parameters become more conservative, the reduction in reinforcement required relative to current design methodology becomes smaller.
- Geosynthetic wall reinforcement requirements are reduced when the K_0-Stiffness method is used because it allows more strain to occur in the wall. Designers must be cognizant of this fact when designing reinforced soil walls with this methodology. For applications where increased strain is not acceptable, less cost effective geosynthetic

wall designs may have to be used, or, alternatively, steel reinforced systems must be selected.

The K_0-Stiffness Method provides a more accurate estimate of reinforcement loads, and its use can result in substantial cost savings, especially for geosynthetic walls. However, there is minimal additional margin of safety to accommodate poor construction technique or materials control. The load and resistance factors recommended in this report are intended to accommodate some variation in construction quality, but not wide variations. Therefore, the user of the K_0-Stiffness Method must ensure that a reasonable degree of wall construction quality control is used. If for some reason construction quality cannot be properly controlled, then the user of the K_0-Stiffness Method should increase the value of load factors or decrease resistance factors used in the design to account for that uncertainty.

THE PROBLEM

Allen and Bathurst (2001) developed a new methodology for estimating reinforcement loads in reinforced soil walls. It is called the K_0-Stiffness Method. This new method was developed empirically through analysis of full-scale wall case histories. In most cases, reinforcement loads in these case histories had to be estimated from measured reinforcement strain converted to load through a reinforcement modulus. Therefore, the correct modulus, given time and temperature effects, had to be estimated, at least for geosynthetic walls, to accurately determine the reinforcement loads. Analysis determined that long-term laboratory creep data, which could be used to estimate the creep modulus, was sufficiently accurate for this purpose. For steel reinforced walls, the conversion of strain to load is relatively straightforward.

Once the correct load levels in the reinforcement layers were established, the reinforcement loads obtained from the full-scale walls were compared to what would be predicted with the new method and the current methodologies found in design guidelines and design codes, including the Coherent Gravity Method and the Simplified Method (AASHTO, 1999). All existing design methodologies were found to provide very inaccurate predictions of reinforcement load for geosynthetic walls, and marginally acceptable predictions for steel reinforced structures. Allen and Bathurst (2001) determined that the average and coefficient of variation (COV) of the ratio of predicted to measured T_{max}, the peak reinforcement load in each layer, for the Simplified Method (Allen and Bathurst, 2001) was 2.9 and 85.9 percent, respectively, for geosynthetic walls, and 0.9 and 50.6 percent, respectively, for steel reinforced soil walls when all available case histories were considered. The K_0-Stiffness Method was found to have an average and coefficient of variation of this ratio of 1.12 and 40.8 percent, respectively for geosynthetic walls, and 1.12 and 35.1 percent, respectively for steel reinforced soil walls, a marked improvement. These statistics were based on an empirical database consisting of measured reinforcement strains and loads from nine full-scale field geosynthetic wall cases (13 different wall sections and surcharge conditions, and 58 individual data points) and 19 full-scale field steel reinforced soil wall cases (24 different wall sections and surcharge conditions, and 102 individual data points). An additional five full-scale test wall cases were also analyzed to assess the effect of variables that could not be easily assessed with only the field walls, but they were not directly included in the database. Allen and Bathurst (2001) indicated that the best overall practical indicator of the uncertainty in each method is the comparison of load predictions from the method to the best estimate of the actual loads in the reinforcement layers.

The new methodology considers, directly or indirectly, the stiffness of all wall components relative to the soil stiffness to estimate the distribution and magnitude of T_{max}. As such, it uses working stress principles to estimate the load and strain in the reinforcement. This new methodology was determined to provide a reasonably accurate prediction up to incipient soil failure, making it possible to use the predictions from this method for both a serviceability and strength limit state design (Allen and Bathurst, 2001).

Now that a reinforcement load and strain prediction methodology has been developed, the next step, which is the purpose of this report, is to apply the method to the internal design of reinforced soil walls, making sure that the recommended design approach is compatible with current design codes. Current design codes in North America and worldwide have or are moving toward limit states design (Goble, 1999). Therefore, the design procedures provided herein must be developed in a way that is consistent with limit states design, for example, Load and Resistance Factor Design (LRFD) as described in the AASHTO LRFD Bridge Design Specifications (AASHTO, in press). To accomplish this, the limit states to be evaluated must be clearly defined, and load and resistance factors that account for the uncertainty in the method and material properties must be estimated.

Specific, step-by-step guidance on how to apply the K_0-Stiffness Method to design reinforced soil walls for internal stability (i.e., reinforcement rupture in the backfill and at the connection and pullout) are provided. This includes application of installation damage, creep, and durability reduction factors (or corrosion for steel) to determine the long-term strength available to resist the calculated loads. The design procedures should be widely applicable to reinforced soil walls that utilize granular (non-cohesive) backfill. Examples using some of the full-scale wall case histories presented by Allen and Bathurst (2001) are provided to demonstrate how the new design methodology compares with current practice in design codes.

The scope of this new methodology is limited to granular backfill materials. It is not applicable, given what is known to date, to silt or clay backfills.

THE CALCULATION OF T_{MAX} USING THE K_0-STIFFNESS METHOD

T_{max}, the peak load in each reinforcement layer, can be calculated with the K_0-Stiffness Method as summarized below (Allen and Bathurst, 2001):

$$T_{\max} = 0.5 S_v K_0 \gamma \ (H+S) D_{t\max} \Phi_{local} \Phi_{fb} \Phi_{fs} 0.27 \left(\frac{S_{global}}{p_a} \right)^{0.24} \tag{1}$$

where

- S_v is the tributary area (assumed equivalent to the average vertical spacing of the reinforcement at each layer location when analyses are carried out per unit length of wall)
- K_o is the at rest lateral earth pressure coefficient for the reinforced backfill
- H is the vertical wall height at the wall face,
- S is the average soil surcharge height above the wall top
- D_{tmax} is a distribution factor to estimate T_{max} for each layer as a function of its depth below the wall top relative to T_{mxmx} (the maximum value of T_{max} within the wall)
- S_{global} is the global reinforcement stiffness
- Φ_{local} is the local stiffness factor
- Φ_{fb} is the facing batter factor
- Φ_{fs} is the facing stiffness factor
- p_a is atmospheric pressure (a constant equal to 101 kPa).

The constant p_a is needed simply to preserve dimensional consistency of the equation. K_0, S_{global}, Φ_{local}, Φ_{fb}, Φ_{fs}, and D_{tmax} are further defined below.

K_0 can be determined from Equation 2 below (Holtz and Kovacs, 1981):

$$K_0 = 1 - Sin\phi' \tag{2}$$

where ϕ' is the peak angle of internal soil friction for the wall backfill. For steel reinforced systems, K_0 for design should be 0.3 or greater. This equation for K_0 has been shown to work reasonably well for normally consolidated sands, and it can be modified by using the overconsolidation ratio (OCR) for sand that has been preloaded or compacted. Because the OCR is very difficult to estimate for compacted sands, especially at the time of wall design, the K_0-Stiffness Method was calibrated using only Equation 2 to determine K_0. Because the K_0-Stiffness Method is empirically based, it can be argued that the method implicitly includes compaction effects, and therefore modification of Equation 2 to account for compaction is not necessary. Note also that the method was calibrated using measured peak shear strength data corrected to peak plane strain shear strength values.

The global stiffness, S_{global}, considers the stiffness of the entire wall section, and it is calculated as follows:

$$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^{n} J_i}{H} \tag{2}$$

where J_{ave} is the average modulus of all the reinforcement layers within the entire wall section, J_i is the modulus of an individual reinforcement layer, H is the total wall height, and n is the number of reinforcement layers within the entire wall section.

The local stiffness considers the stiffness and reinforcement density at a given layer and is calculated as follows:

$$S_{local} = \frac{J}{S_v} \tag{3}$$

where J is the modulus of an individual reinforcement layer, and S_v is the vertical spacing of the reinforcement layers near a specific layer.

The local stiffness factor, Φ_{local}, is then defined as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{global}} \right)^a \tag{4}$$

where a = a coefficient which is also a function of stiffness. Observations from the available data suggest that setting a = 1.0 for geosynthetic walls and 0.0 for steel reinforced soil walls is sufficiently accurate.

The wall face batter factor, Φ_{fb}, which accounts for the influence of the reduced soil weight on reinforcement loads, is determined as follows:

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d} \tag{5}$$

where K_{abh} is the horizontal component of the active earth pressure coefficient accounting for wall face batter, and K_{avh} is the horizontal component of the active earth pressure coefficient. This assumes that the wall is vertical and d = a constant coefficient (recommended to be 0.5 to provide the best fit to the empirical data).

The facing stiffness factor, Φ_{fs}, was empirically derived to account for the significantly reduced reinforcement stresses observed for geosynthetic walls with segmental concrete block and propped panel wall facings. It is not yet known whether this facing stiffness correction is fully applicable to steel reinforced wall systems. On the basis of data available at the time of this report, Allen and Bathurst (2001) recommend that this value be set equal to the following:

- 0.5 for segmental concrete block and propped panel faced walls
- 1.0 for all other types of wall facings (e.g., wrapped face, welded wire or gabion faced, and incremental precast concrete facings)
- 1.0 for all steel reinforced soil walls.

Note that the facings defined above as flexible still have some stiffness and some ability to take a portion of the load applied to the wall system internally. It is possible to have facings that are more flexible than the types listed above, and consequently, walls with very flexible facings may require a facing stiffness factor greater than 1.0.

The maximum wall heights available where this facing stiffness effect could be observed were approximately 6.1 m. Data from taller walls were not available. It is possible that this facing stiffness effect may not be as strong for much taller walls. Therefore, caution should be exercised when using these preliminary Φ_{fs} values for walls taller than 6 m.

The soil reinforcement load distribution factor, D_{tmax}, was determined empirically from all of the available field wall case histories. It is shown in Figure 1(a) for geosynthetic reinforced walls and Figure 1(b) for steel reinforced walls. Here D_{tmax} is the ratio of T_{max} in a reinforcement layer to the maximum reinforcement load in the wall, T_{mxmx}. Note that the empirical distributions provided in Figure 1 apply to walls constructed on a firm soil foundation. The distributions that would result for a rock or soft soil foundation may be different from those shown in this figure.

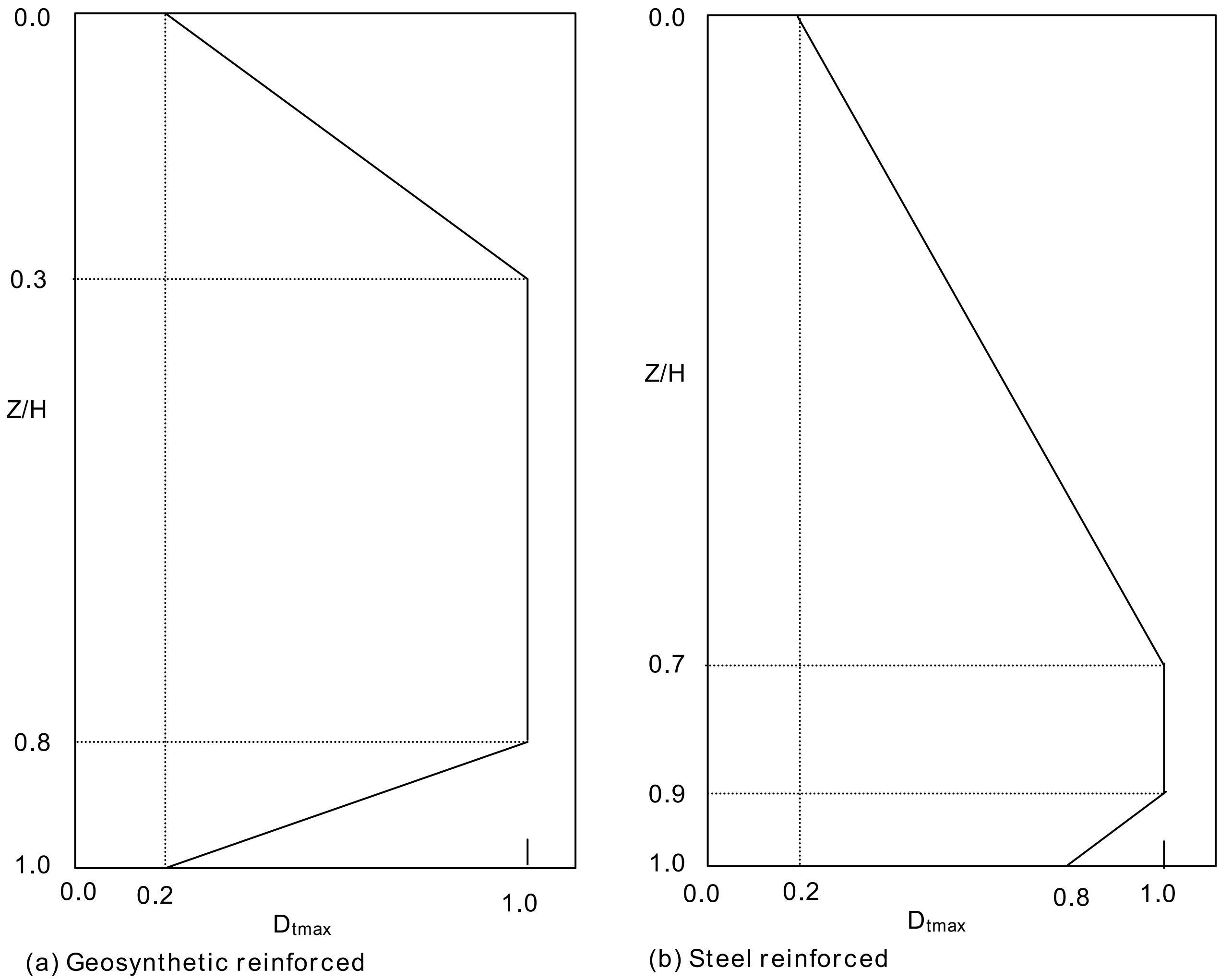


Figure 1. Distribution of T_{max} with normalized depth below wall top.

APPLICATION OF THE K_0-STIFFNESS METHOD TO LIMIT STATES WALL DESIGN

Applicable Limit States and Their Assessment

In general, two limit states are considered: ultimate and serviceability (Ovesen, 1989). These two general categories of limit states are typically subdivided into several specific limit states to accommodate various loading and failure risk scenarios. The ultimate limit state is typically subdivided into two additional subcategories, the strength and extreme event limit states. In geotechnical engineering, the strength limit state is focused on the ability of the structure to resist various combinations of dead and live load. The extreme event limit state is primarily focused on design to resist earthquake loads, impact loads, and scour or flood conditions, in addition to the ever-present dead and live loads. The short- or long-term ultimate capacity of the system to resist loads, factored to account for material variability and other sources of uncertainty, is used in the ultimate limit state in comparison to these loads for design. The serviceability limit state ensures that the structure deformations at working stresses are within acceptable structure performance limits.

The reinforced zone can fail internally by rupture of the reinforcement, failure of the soil, failure of the facing, failure of the connection between the facing and the reinforcement, or by reinforcement pullout. Furthermore, the wall can be considered to have failed if the wall deforms excessively. Allen and Bathurst (2001) indicated that in most geosynthetic wall cases soil failure occurs before reinforcement rupture can occur within the reinforced soil mass, and that once soil failure has occurred, for all practical purposes the wall has failed, too. For steel reinforced walls, the soil reinforcement may reach a limit state before the soil does. In current practice, failure of the reinforced zone has been considered to be a simultaneous failure of both the soil and the reinforcement, and the onset of failure has been calculated with reinforcement rupture capacity. Although this assumption is consistent with the limit equilibrium approach currently used in practice and specified in design code, it is not consistent with the actual failure mechanism observed in full-scale reinforced soil structures.

The K_0-Stiffness Method estimates loads in the reinforcement at working stresses. However, these loads will remain relatively constant for the life of the structure and equilibrium will be maintained if the soil is prevented from failing (Allen and Bathurst, 2001). Estimation of soil failure and reinforcement rupture using loads determined by the K_0-Stiffness Method can be considered strength limit state calculations if soil failure is prevented. Currently, the best way to design for soil failure is to prevent the soil from reaching its peak strain and therefore its peak load. This can be accomplished by selecting a target reinforcement strain, ε_{targ}, that must not be exceeded in design. The worst condition in this regard is a very strong, high peak friction angle soil, as the peak shear strain for this type of soil will be lower than the peak shear strain for most backfill soils. On the basis of plane strain shear strength testing of high shear strength sands, peak shear strains on the order of 2.5 to 6 percent can generally be expected. Full-scale wall laboratory testing has shown that the reinforcement strain at which the soil begins to exhibit signs of failure is on the order of 3 to 5 percent for high shear strength sands that have exhibited peak shear strains on the order of 2 to 3 percent from plane strain shear strength tests (Allen and Bathurst, 2001). Limiting the reinforcement strains to 3 to 5 percent should in general prevent the soil from reaching its strength limit state.

The loads estimated from the K_0-Stiffness Method can also be used to ensure that the reinforcement does not reach its strength limit state. Knowing that the working stress loads estimated from the K_0-Stiffness Method represent equilibrium conditions, reinforcement rupture can be prevented by making sure that the predicted reinforcement load is at or below the long-term strength of the reinforcement selected. For steel, corrosion of the reinforcement during the wall design life must be estimated and taken into account. For geosynthetics, installation damage, creep, and durability strength losses must be estimated and taken into account. This applies to both the backfill and the reinforcement-facing connection.

Pullout failure represents another important strength limit state that must be considered. The load estimated from the K_0-Stiffness Method is the actual long-term load that must be resisted by the reinforcement. Provided the pullout resistance is not exceeded, the wall should remain in equilibrium at the predicted working load.

Reinforcement Tensile Strength and Spacing Design

The reinforcement strength and spacing required to prevent reinforcement rupture at any time during the design life of the structure must be evaluated to provide internal stability of the reinforced soil mass. This can be evaluated with the following equation (AASHTO, in press):

$$\gamma_{EH} T_{\max} \leq \varphi_{rr} T_{al} R_c \qquad (6)$$

where γ_{EH} = soil reinforcement load factor, φ_{rr} = resistance factor for reinforcement rupture, T_{al} = long-term reinforcement design strength, and R_c = reinforcement coverage ratio (b/S_h), which is equal to the reinforcement strip width, b, divided by its horizontal center-to-center spacing, S_h.

For geosynthetic reinforcement, the long-term design strength is determined as follows:

$$T_{al} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D} \qquad (7)$$

where T_{ult} is the ultimate tensile strength of the reinforcement based on the minimum average roll value (MARV), RF_{ID} is the reduction factor for strength loss due to installation damage, RF_{CR} is the reduction factor for strength loss due to creep, and RF_D is the strength reduction factor due to chemical and biological degradation.

For steel reinforcement, the long-term design strength is determined as follows:

$$T_{al} = \frac{A_c F_u}{S_h} \qquad (8)$$

where A_c is the cross-sectional area of the steel reinforcement unit after corrosion losses, and F_u is the ultimate tensile stress of the steel. AASHTO (in press) provides corrosion loss rates that can be used to calculate the cross-sectional area at the end of design life.

Reinforcement Pullout Design

The length and amount of reinforcement required to resist pullout failure must also be evaluated to provide an internally stable reinforced soil mass. T_{max} determined from the K_0-Stiffness Method is used in the pullout capacity calculation. The design methodology for determining pullout resistance provided in AASHTO (1999, in press) can then be used, as shown below:

$$\mathrm{L_e} \geq \frac{\gamma_{\mathrm{EH}} \mathrm{T_{max}}}{\varphi_{po}\, \mathrm{F}^* \alpha\, \sigma_{\mathrm{v}}\, \mathrm{C\, R_c}} \qquad (8)$$

where L_e is the length of reinforcement in the resisting zone, φ_{po} = resistance factor for pullout, F* = pullout friction factor, α = scale effect correction factor (default values of 0.6 for geotextiles, 0.8 for geogrids, and 1.0 for steel reinforcement), σ_v = vertical stress at the reinforcement layer in the resistant zone, and C = reinforcement gross surface area geometry factor (equals two for strip, sheet, and grid reinforcements – i.e., two sides). The default values for the friction factor recommended in AASHTO (in press) are provided in Figure 2.

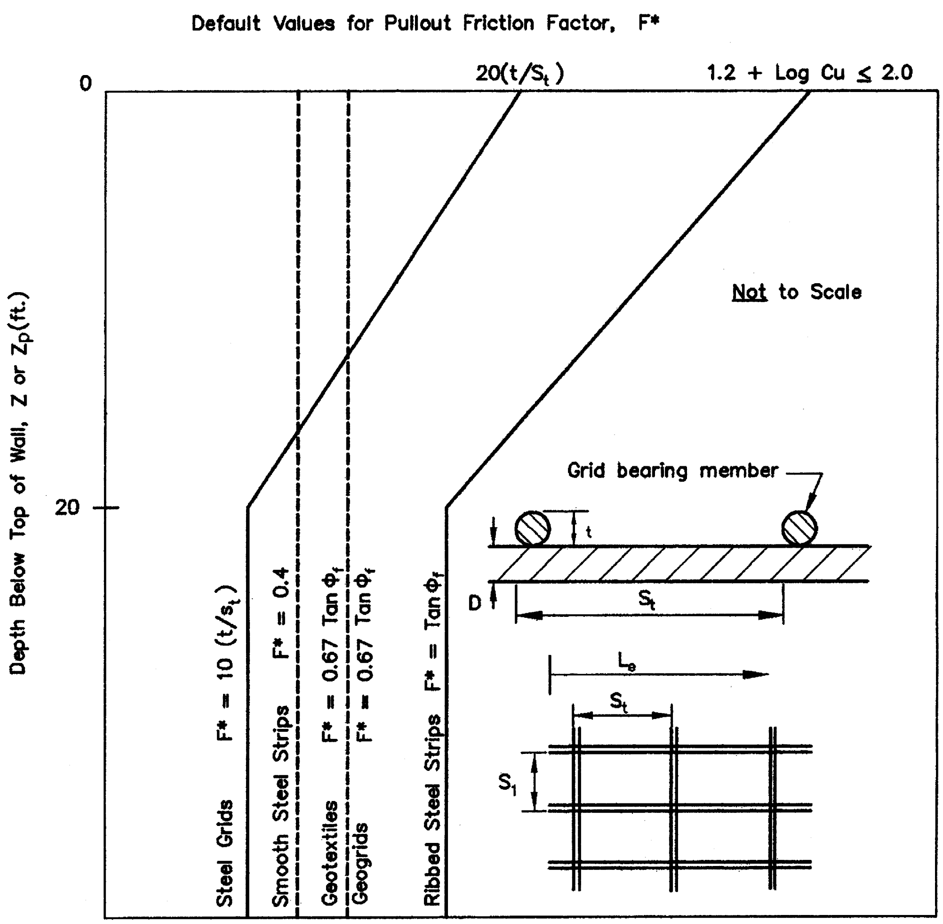


Figure 2. Default values for the pullout friction factor, F* (after AASHTO, in press).

Reinforcement Connection Design

The strength of the connection between the wall facing and the backfill reinforcement and the load applied to the connection must also be checked to ensure that all appropriate limit states are met. The K_0-Stiffness Method was developed to predict reinforcement peak loads (T_{max})

within the backfill zone. However, the loads at the facing-reinforcement connection may be higher or lower than T_{max} in the backfill, depending on the following factors:

1. Facing stiffness – the stiffer the facing, the higher the connection loads tend to be because of the greater ability to develop compaction stresses near the face and due to differential vertical movements, which are more likely between the relatively rigid facing column and the relatively compressible backfill soil.
2. Compressibility of the backfill soil as well as the foundation soil.
3. The care with which the soil is placed and compacted near the face, how the reinforcement layers are installed near the face (e.g., poor construction procedures can result in higher connection stresses).
4. The details of the connection system.

Current design specifications and design guidelines (AASHTO, 1999; Elias, et al., 2001) require that the connections at the facing be designed for loads that are equal to or greater than T_{max}. Therefore, at least a starting point for designing the connection is to use 100 percent of T_{max} for the load. Available full-scale data indicate that the connection load can be as much as 150 percent of T_{max} in the worst case, but the more typical case is a connection load that is 100 percent of T_{max} or less. Note that this connection load issue applies to both geosynthetic and steel reinforced systems.

Typical North American practice for calculating long-term connection strength is provided in AASHTO (in press). For segmental concrete block facing systems with geosynthetic reinforcement, the long-term connection strength must be calculated at each reinforcement level. accounting for the available normal force between facing blocks, if connection strength is a function of normal force. Normal stress at each reinforcement level is calculated on the basis of the vertical stresses that develop in the facing column. For heavily battered walls (greater than 8^{o} from vertical), the hinge height methodology in AASHTO (1999, in press) should be used. The long-term connection strength, T_{ac}, is calculated as follows for geosynthetic/segmental block connections (AASHTO, in press):

$$T_{ac} = \frac{T_{ult} \times CR_{cr}}{RF_D} \tag{9}$$

where CR_{cr} is the long-term connection strength (T_{crc}) extrapolated to the desired design life normalized by the index tensile strength of the lot or roll of material used for the connection testing, T_{lot}. All other variables are as defined previously. If the long-term connection strength is not available, it can be conservatively estimated as follows for geosynthetic/segmental block connections (AASHTO, in press):

$$CR_{cr} = \frac{CR_u}{RF_{CR}} \tag{10}$$

where CR_u = the ultimate strength of the facing–geosynthetic connection determined from laboratory tests, normalized by the lot or roll-specific index tensile strength of the geosynthetic.

For other types of geosynthetic/facing connections, the tensile capacity of the connection must be determined directly through tensile testing of the connection. The long-term resistance of the connection, given creep rupture of the reinforcement or other connecting device, can be determined directly through creep testing of the connection and calculation with Equation 9. It can also be determined with results from short-term tensile tests that are reduced for creep, as shown in Equation 10.

For steel strip reinforcement, the reduced cross-sectional area of the steel strip, after loss from corrosion, that results from the presence of a bolt hole is used to recalculate the reinforcement tensile strength available. The connection strength in this case is the reduced tensile strength or the bolt shear capacity, whichever is less. For welded wire and bar mat reinforcement, provided the weld shear capacity of the reinforcement grid is greater than the tensile strength of the longitudinal members of the grid, the connection strength is approximately equal to the tensile strength of the reinforcement. In both cases, the long-term capacity of the embedded connectors cast into the facing panels must be greater than the reinforcement capacity, per AASHTO (in press).

Estimating Deformations for Serviceability Limit Assessment

Reinforcement strain can be estimated by using the K_0-Stiffness Method to determine T_{max}, and then dividing T_{max} by the reinforcement modulus at the end of construction (EOC) to determine the strain at the EOC. This strain can be compared to an EOC strain criterion, if one is available. However, it is very difficult to estimate deflections directly from strain data. For strains that are on the order of 2 percent or less, the incremental wall face deflections during construction, as functions of normalized depth below the wall top, can be estimated from Figure 3. The maximum deflections in this figure, combined with steel reinforced soil wall face deflections from Christopher (1993), have been plotted against the average peak strain in each wall to provide an approximate relationship between maximum wall face deflection during construction and reinforcement strain at the EOC (see Figure 4). Christopher (1993) also provided a relationship that can be used to estimate wall face deflections, given the global stiffness, S_{global}, of the wall. A simplified version of that relationship is illustrated in the current AASHTO design specifications (AASHTO, 1999; Elias, et al., 2001). Such a relationship, or the data provided in Figure 4, can be used to check the value of the modulus selected for the reinforcement to meet EOC serviceability requirements, with consideration given to both reinforcement strain and wall face displacement.

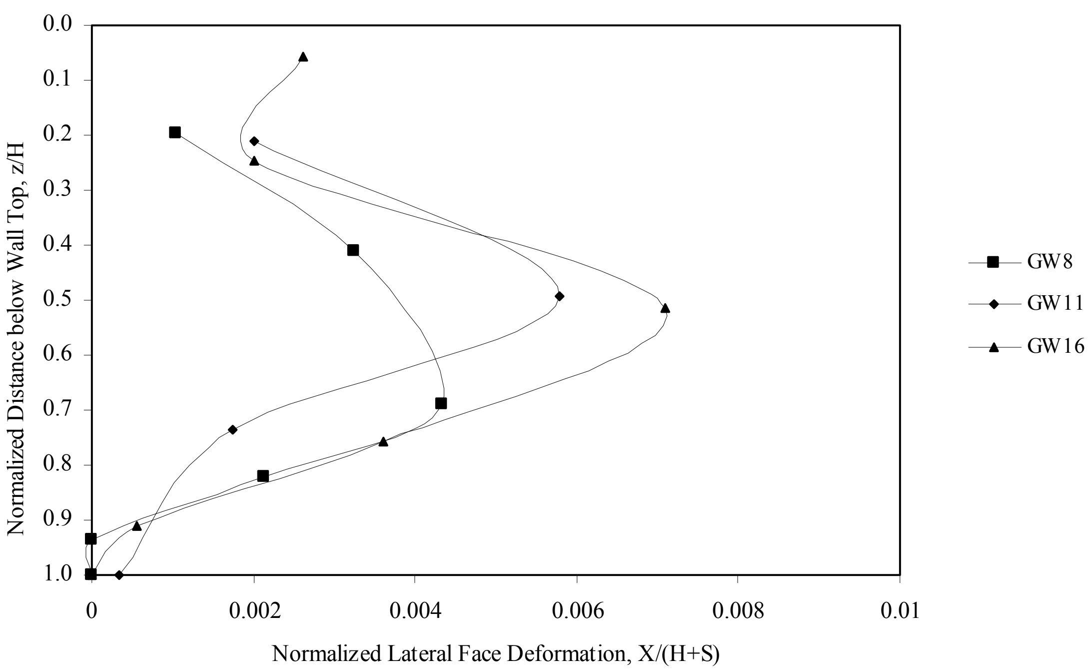


Figure 3. Normalized lateral facing deflections from wall facing survey measurements taken with respect to initial reading (Case study GW8 – incremental concrete panel face; Case study GW11 and GW16 – wrapped face) (after Allen and Bathurst, 2001).

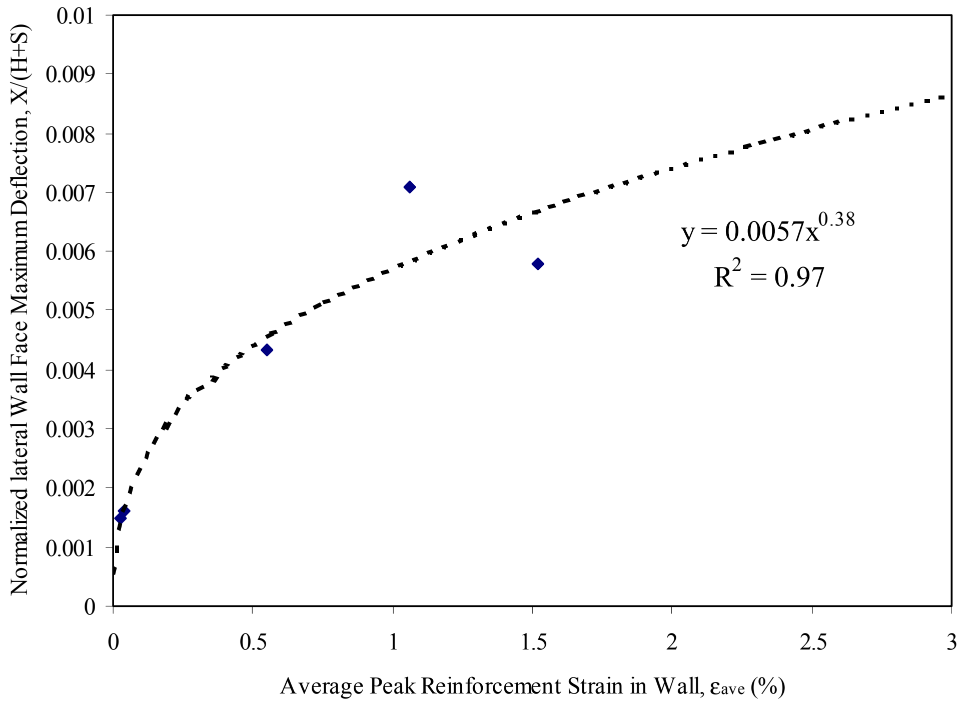


Figure 4. Normalized wall face maximum lateral deflection X at the end of construction versus average peak reinforcement strain in the wall.

Reinforcement post-construction (long-term) strain can also be estimated by using the K_0-Stiffness Method to determine T_{max}, and then dividing T_{max} by the reinforcement modulus at the desired time after construction (e.g., 10,000 hours, or the wall design life) to determine the long-term strain. The strain calculated at the end of construction can then be subtracted from the long-term strain to obtain the post-construction strain. This post-construction strain can then be compared to a long-term creep strain criterion, if one is available. Again, it is difficult to estimate long-term wall face deformations from reinforcement strains. The post construction deformations shown in Figure 5 (which are in turn related to the average post-construction strains measured in each wall in Figure 6) can be used as a guide in setting creep strain limits for geosynthetic walls. One data point is representative of a propped panel wall that had a post-construction strain of approximately 1.5 percent. All of the strain for propped panel walls is post-construction, and these figures show that propped panel wall strains can be treated for practical purposes as creep strain to estimate long-term wall face deformation. Note that one data point in Figure 6 (GW8) exhibited significantly higher post-construction deformations than did the other walls. This data point may demonstrate that creep strains are not the only source of long-term wall face deformation. Other possible sources of long-term lateral wall face deformation may include long-term differential settlements and long-term movements due to slack in the reinforcement-facing connection or slack in the reinforcement layer itself. If post-construction creep strains are greater than desired, T_{ult} of the reinforcement may need to be increased to make the load level lower relative to the creep limit for the material.

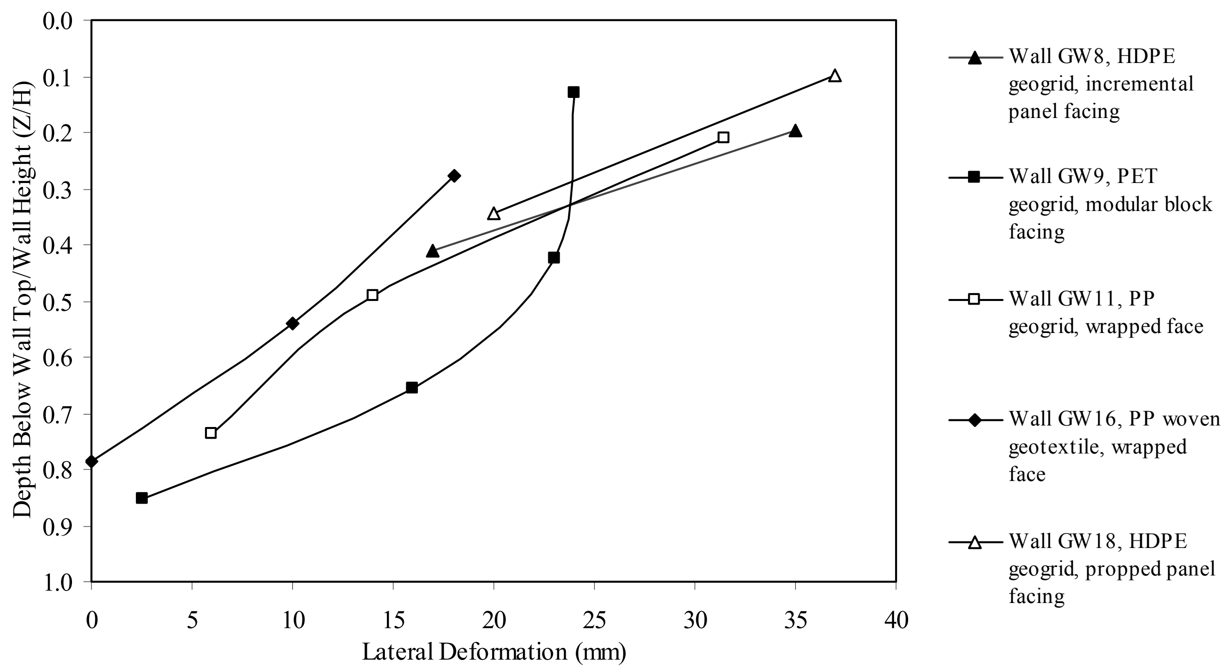


Figure 5. Estimated lateral wall face post-construction deformation for geosynthetic walls at 75 years as a function of normalized depth below the top of the wall (adapted from Allen and Bathurst, 2001).

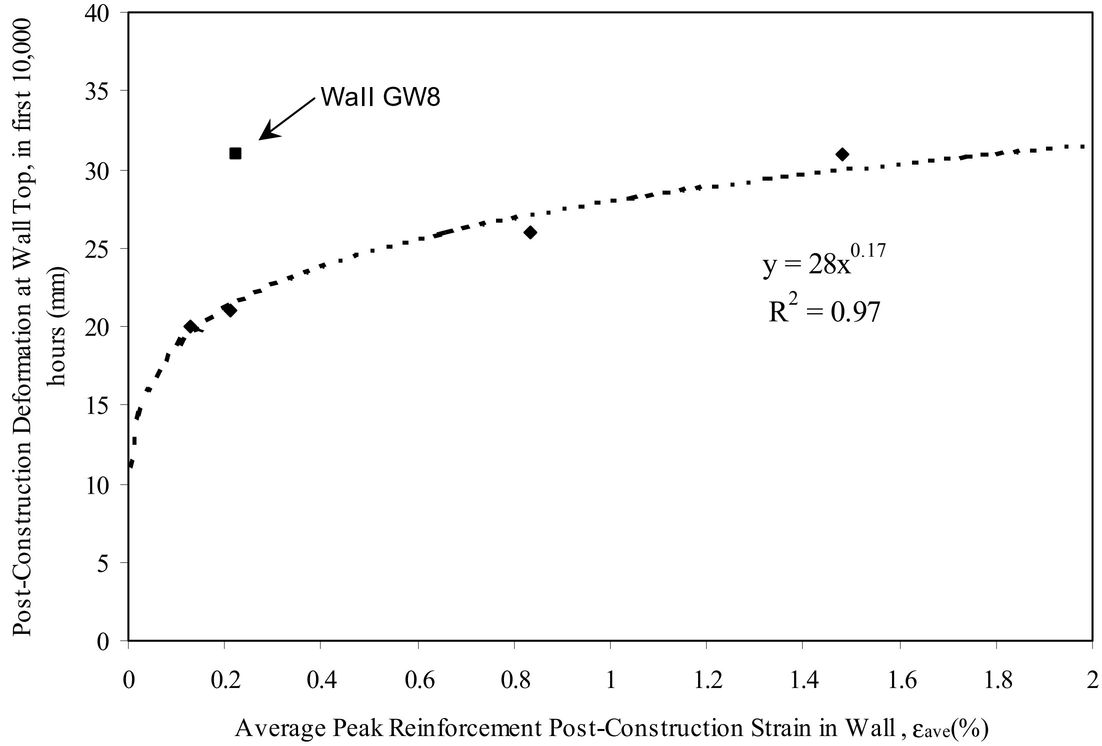


Figure 6. Post-construction maximum wall face deformation, X (at wall top), in first 10,000 hours after the EOC versus average peak reinforcement post-construction strain in the wall.

Estimating Load and Resistance Factors for Limit States Design of Reinforced Soil Walls

A complete design procedure cannot be developed without knowing how to account for the uncertainty in the predicted loads and resistances so that good performance can be ensured. One of two general approaches is currently used in geotechnical engineering practice to account for this uncertainty: allowable stress design (ASD) or limit states design (termed Load and Resistance Factor Design, or LRFD, in North America). The following basic equation can be used to represent limit states design from the North American perspective (AASHTO, in press):

$$\sum \gamma_i Q_i \leq \varphi R_n \quad (11)$$

where γ_i is a load factor applicable to a specific load, Q is a load, the summation of $\gamma_i Q_i$ is the total factored load for the load group applicable to the limit state being considered, φ is the resistance factor, and R_n is the nominal resistance available (either ultimate or the resistance available at a given deformation). The load and resistance factors are used to account for material variability, design model prediction inaccuracy, and other sources of uncertainty. Allowable stress design is represented by the following equation:

$$FS = \frac{R_n}{Q} \tag{12}$$

where FS is the design factor of safety, and all other variables are as defined previously.

In allowable stress geotechnical design practice, factors of safety were typically developed deterministically through experience so that good performance was consistently obtained (Withiam, et al., 1998). In some cases, statistical analyses were performed to justify the safety factors recommended. However, engineering judgment usually played a significant role in establishing the magnitude of safety factors. Because a single factor of safety for a given limit state is used in ASD, such an approach cannot account for the variability of the different loads applied.

For limit states design, it is best if the load and resistance factors are determined through statistical analyses so that a consistent probability of failure is obtained for all limit states. However, because of the lack of adequate data to perform such analyses, the load and resistance factors for geotechnical design have most often been determined by fitting them to ASD. In that way, the use of the resulting load and resistance factors results in the same size foundation or the same wall design as would have been obtained if they had been designed by ASD (Goble, 1999; DiMaggio, et al., 1999). What this means is that load and resistance factors determined in this way include the same engineering judgment that is implicit in the ASD factors of safety. However, the load and resistance factors currently provided in design codes do at least account for the difference in the variability (either assumed on the basis of engineering judgment, or determined in some cases statistically) of the various loads.

Because the K_0-Stiffness Method for soil wall reinforcement load prediction is a completely new methodology, it would do little good to simply calibrate the method to yield the same results as would be obtained by the methods currently specified in design code (e.g., the Simplified Method in AASHTO, 1999). However, given that the K_0-Stiffness Method does appear to improve the prediction accuracy relative to currently used methods such as the Simplified Method, the load and resistance factors for the K_0-Stiffness Method should be no more conservative than the load and resistance factors currently used for the Simplified Method, if those load and resistance factors are correct. Current load and resistance factors provided in the AASHTO design code for reinforced soil wall internal stability design are as follows (Withiam, et al., 1998; AASHTO, in press):

Table 1. Load and resistance factors specified in current US design code (AASHTO, in press).

Limit State	Failure Mode	Reinforcement Type	Load Factor	Resistance Factor
Strength*	Reinforcement or Connector Failure	• Steel strip • Steel grid connected to rigid facing element • Geosynthetic	• 1.35 • 1.35 • 1.35	• 0.75 • 0.65 • 0.90
	Pullout	All	1.35	0.9
Extreme Event*	Reinforcement or Connector Failure	• Steel strip • Steel grid connected to rigid facing element • Geosynthetic	• 1.35 static, 1.0 seismic • 1.35 static, 1.0 seismic • 1.35 static, 1.0 seismic	• 1.00 • 0.85 • 1.20
	Pullout	All	• 1.35 static, 1.0 seismic	• 1.20
Service	N/A	N/A	N/A	N/A

N/A – not applicable at present (nevertheless, load and resistance factors for this limit state would all be equal to 1.0).

*For steel, resistance factors are relative to F_y, the yield stress of the steel (minimum specification values). For geosynthetics, resistance factors are relative to T_{ult} (MARV).

An approximate statistical analysis was conducted where possible to estimate the load and resistance factors that should be used with the K_0-Stiffness Method for internal reinforced soil wall design. Using the approach taken by Nowak (1999) to calibrate the current AASHTO LRFD specifications, an approximate estimate of the load factor to be used with this method can be determined as follows:

$$\gamma_{EH} = \lambda_{QEH}\left(1 + 2\,\mathrm{COV}_{\mathrm{QEH}}\right) \tag{13}$$

where λ_{QEH} = the bias factor for the reinforcement load due to dead load, defined as the mean of the ratio of the measured to predicted load, and COV_{QEH} = the coefficient of variation of the ratio of measured to predicted reinforcement load. The load statistics provided previously, which were obtained from Allen and Bathurst (2001), were based on the ratio of predicted to measured reinforcement load. They defined this ratio in this manner for two reasons: so that a conservative prediction for design corresponds to a ratio of greater than 1.0, and because they viewed the measured load as the independent variable and the load prediction as the dependent variable, a common way to approach such matters. However, the statistical parameters used in Equation 13 and in the equation used to estimate the resistance factor (provided later in this report – see Equation 14) define the bias and COV on the basis of the ratio of the measured to the predicted or nominal value, which is consistent with the calibration of the current AASHTO LRFD design specifications (Nowak, 1999). Therefore, ratios of measured and predicted loads, and the associated statistics, have been re-evaluated to be consistent with equations 13 and 14.

The load data, presented as the ratio of measured to predicted reinforcement load, are plotted as a function of normalized depth below the wall top, z/H, in figures 7 and 8. To ensure that a few outlier data points do not excessively bias the statistics (i.e., result in unreasonably conservative values of load and resistance factors), points that were more than two standard deviations beyond the mean of the ratios were discarded from the dataset, and the remaining data were re-analyzed to calculate parameters needed to determine load and resistance factors. This is a more approximate, though likely conservative, approach than that used for the calibration of the current AASHTO LRFD design specifications to deal with outlier points (D'Appolonia, 1999).

The outlier points were also examined to identify the reason for the unusually poor load prediction. In Figure 7 (geosynthetic walls), two outlier points were identified. These consisted of data obtained near the top and bottom of Wall GW7, Section N (in Allen and Bathurst, 2001). This wall was technically a very steep reinforced slope (heavily battered) subjected to an unusually large soil surcharge. It used a facing that was likely more flexible than any of the other walls in the database used to develop the K_0-Stiffness Method. This combination of factors represents a special wall case that is at the limit of the ability of any current design methodology, including the proposed K_0-Stiffness Method, to accurately predict reinforcement loads. It appears that the unusually heavy surcharge load may have increased the reinforcement load at the wall top. At the bottom of the wall the measured maximum strain was only 0.11 percent, which is below the limit of reliable strain measurement for the Bison coils used in the wall (see Allen and Bathurst, 2001, Chapter 3, for comments on measurement accuracy).

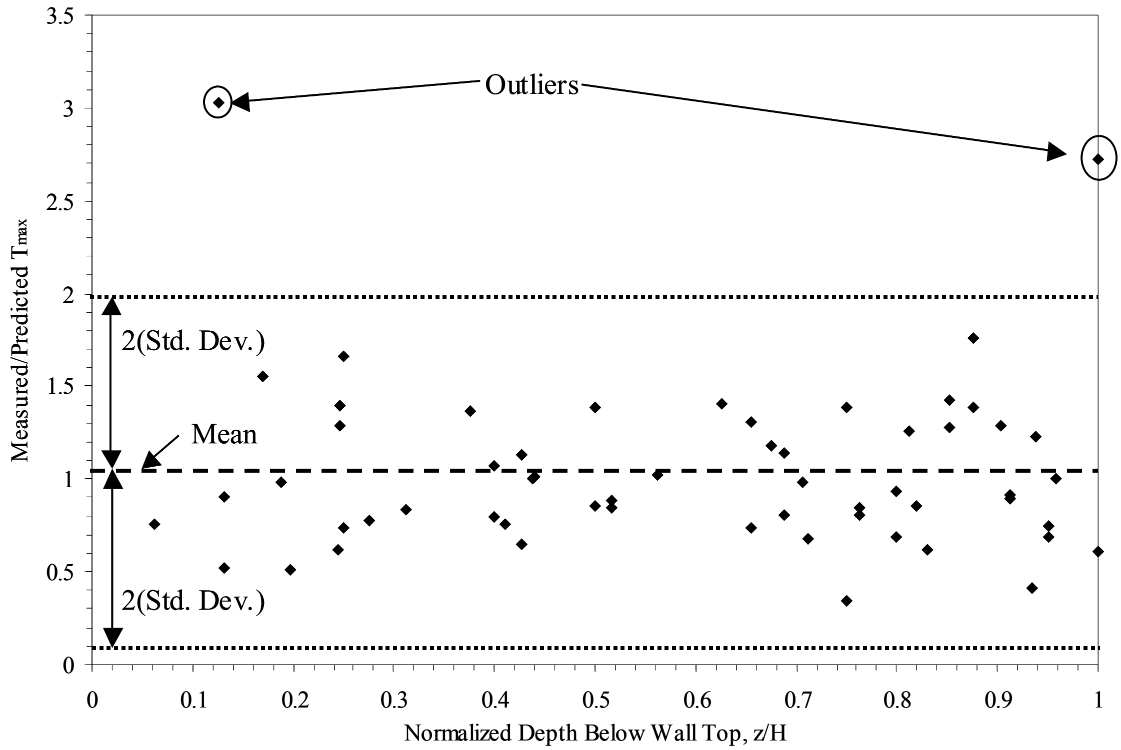


Figure 7. Ratio of measured to predicted T_{max} for the K_0-Stiffness Method versus normalized depth below wall top (geosynthetic walls only).

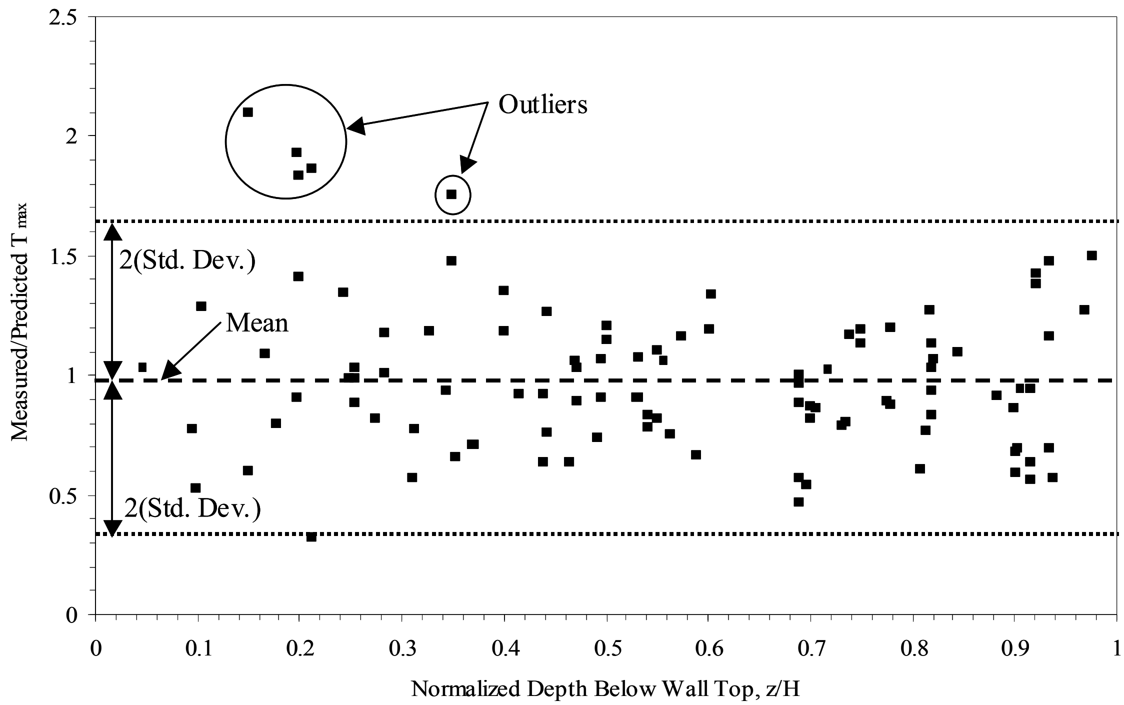


Figure 8. Ratio of measured to predicted T_{max} for the K_0-Stiffness Method versus normalized depth below wall top (steel reinforced walls only).

In Figure 8, five outlier points were identified, all of which were near the tops of the walls. The outlier points came from walls BM1 (without soil surcharge), BM1 (with soil surcharge), BM2 (with soil surcharge), SS6, Section B, and SS11, as designated by Allen and Bathurst (2001). In all of these walls, compaction stresses were likely high. The K_0-Stiffness Method was developed with the assumption of an average level of compaction. Two of the points were located immediately below a large soil surcharge, which may have contributed to the unusually high stresses in the reinforcement. For one of the points (BM1 without a soil surcharge), inconsistency was noted in the pattern of peak reinforcement loads versus depth below the wall top. The point in question exhibited a much higher load than would be expected at that level in the wall, given the overall pattern of reinforcement load distribution. It is possible that gauge bending or gauge malfunction could have contributed to the unusually high load corresponding to this data point.

The analysis of data in Figure 7 for geosynthetic walls revealed a bias (the same as the mean of the ratio of measured reinforcement load to K_0-Stiffness Method predicted load in this case) and COV for geosynthetic walls, with the outliers included, of 1.05 and 45 percent, respectively. Likewise, for the steel reinforced walls, the bias and COV, with the outliers included, were 0.99 and 32.7 percent, respectively. With the outliers removed, the bias and COV for the geosynthetic walls were 0.98 and 32.7 percent, respectively. For the steel reinforced walls, they were 0.95 and 27.1 percent, respectively. Using the dataset without the outlier points, the load factors were 1.62 for geosynthetic walls and 1.46 for steel reinforced walls. Rounding up to the nearest 0.05 results in a recommended load factor of 1.65 for geosynthetic walls and 1.5 for steel reinforced walls. These load factors assume that the peak plane strain soil friction angle is used (measured, or estimated from measured shear strength test results).

More exact statistical analyses may be required if variables are correlated and to fully consider the complexities of this analysis and the uncertainties of the variables that contribute to load in reinforced soil wall design. However, a visual evaluation of the adequacy of the load factors based on the actual data can be conducted. Figure 9 presents a plot of predicted loads multiplied by the proposed load factors versus measured reinforcement loads. This figure illustrates that almost all predicted reinforcement loads from the available full-scale wall database fall above estimated values when the proposed load factors are used. Though more rigorous statistical analyses could be conducted to more accurately determine the load factors required, these load factor values appear to be sufficiently conservative to safely handle the observed uncertainty in the load predictions, even if measured plane strain soil friction angles are used for design.

Note that these load factor values simply provide a starting point for establishing the resistance factors needed to limit the probability of failure to an acceptable level. As will be shown in detail later, the combination of load and resistance factors, and their associated statistical parameters (bias and coefficient of variation), define the probability of failure. However, it is best to establish the load factor at a reasonable magnitude so that the resulting resistance factors are neither excessively high nor low, reflecting as much as possible the real variance in the load and resistance values. Doing so will give designers an intuitive feel for the magnitude of load and resistance factors and greater confidence in their use.

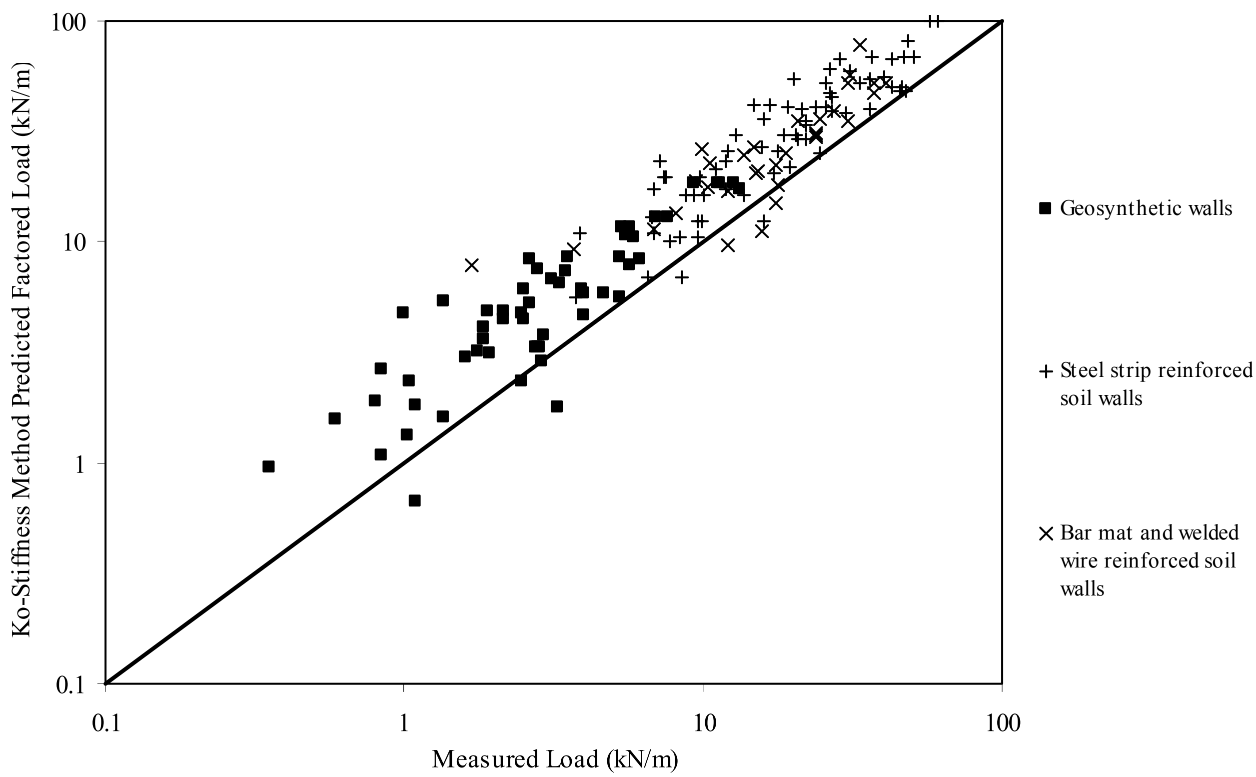


Figure 9. Factored load prediction versus measured reinforcement load using the K_0-Stiffness Method, estimated plane strain soil parameters from measured data, and a load factor Υ_{EH} of 1.65 for geosynthetic walls and 1.5 for steel reinforced walls.

When a load factor is developed for this limit state, it is also important to remember that the K_0-Stiffness Method has been demonstrated to remain reasonably accurate and with a relatively constant COV value up to incipient soil failure (Allen and Bathurst, 2001). This means that the load statistics remain valid at the limit state condition that is being modelled.

To determine the resistance factors needed, consideration was given to the sources of variability and uncertainty in the available resistance terms, as well as to whether lower bound values for the resistance terms were selected. If lower bound values for the resistance terms are used, a resistance factor approaching 1.0 can be used, assuming that the load factor is large enough to limit the probability of failure to an acceptably low level. Otherwise, the resistance factor must be either based on the statistical data and calibrated to yield the desired probability of failure, or must be based on engineering judgment or past practice.

For the reinforced backfill soil failure limit state, a conservative lower bound value for the limit strain of 2.5 percent is recommended for geosynthetic walls. This lower bound corresponds to a resistance factor approximately equal to 1.0. If a higher value is used (for example, one that is based on laboratory test data for the backfill or a material that is similar to the proposed backfill soil), the resistance factor should consider the uncertainty in the soil peak shear strain, ε_p, and how that shear strain is related to the reinforcement strain that could occur in the wall. Good judgment is needed when laboratory soil test data are used to establish ε_{targ}, and an appropriate resistance factor for laboratory data should be used for this purpose. At this writing, more research is needed to fully develop the relationship between ε_{targ} and ε_p.

For steel reinforced soil walls, typical working strains are far below the strains needed to cause the soil to reach a failure condition, even at yield. Above the yield point, depending on the ductility of the steel, the steel can reach strains that are high enough to allow the soil to begin failing. Therefore, F_y of the steel will be used as the criterion to prevent the soil from reaching a failure condition (additional discussion on this issue is provided later). Data on the variability of F_y are available and could be used to establish an appropriate resistance factor for this limit state for steel reinforced walls.

For the reinforcement rupture limit state, the resistance statistics are also available, and a statistical calibration to determine the appropriate resistance factors is possible. The mean value first-order, second moment method (MVFOSM) described by Withiam et al. (1998) and Zhang, et al. (2001) was used for this calibration. The calibration concept, illustrated in Figure 10, is to reduce the resistance values by using a resistance factor so that there is a minimum separation equal to the factor β_τ between the factored resistance distribution and the factored load distribution. The amount of overlap between the two distributions is representative of the probability of failure, i.e., where the load applied exceeds the available resistance. Using this approach, the resistance factor, φ, can be determined as follows, assuming that only the dead load from the soil and the traffic live load, the most common loading scenario for reinforced soil walls, are considered:

$$\varphi = \frac{\lambda_R\left(\gamma_{EH}\left(\frac{QEH}{Q_L}\right)+\gamma_L\right)\sqrt{\frac{1+COV_{QEH}^2+COV_{QL}^2}{1+COV_R^2}}}{\left(\lambda_{QEH}\left(\frac{Q_{EH}}{Q_L}\right)+\lambda_{QL}\right)\exp\left[\beta_\tau\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_{QEH}^2+COV_{QL}^2\right)\right]}\right]} \tag{14}$$

where

- λ_R, λ_{QEH}, and λ_{QL} are the bias factors for the resistance, reinforcement load due to dead load, and live load, respectively (equal to the ratio of measured to predicted resistance or measured to predicted load)
- COV_R, COV_{QEH}, and COV_{QL} are the coefficients of variation for the resistance, reinforcement load due to dead load, and live load, respectively
- Q_{EH}/Q_L is the reinforcement dead load to live load ratio
- γ_{EH} is the load factor for reinforcement load
- γ_L is the load factor for live load
- β_τ is the target reliability index, defined as the number of standard deviations between the mean value and the origin for the safety margin distribution (see Figure 10), which is equal to ln R – ln Q (i.e., the natural logarithm of the resistance values minus the natural logarithm of applied loads; the origin and beyond represent points where the resistance is equal to or less than the applied load, resulting in failure).

All load and resistance statistics are based on the ratio of measured to predicted (or nominal) value. Tables 2 and 3 provide the numerical values needed for the calibration of the reinforcement rupture strength limit state.

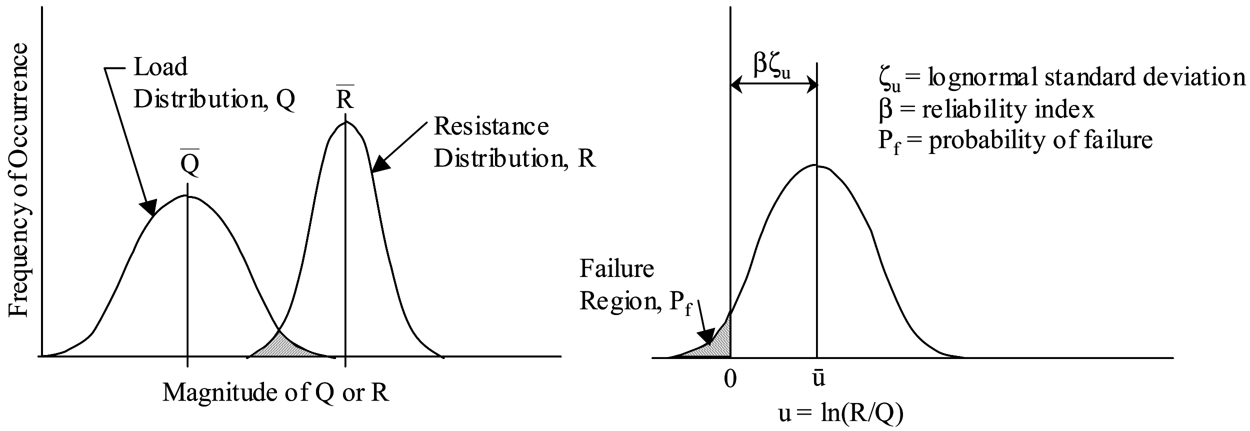


Figure 10. Resistance factor calibration concepts (adapted from Withiam, et al. 1998).

The selection of β_τ for use in calculating the necessary resistance factor depends on the probability of failure (P_f) desired, as the two are directly related. In general, for permanent geotechnical works, a probability of failure of 1 in 100 to 1 in 1,000 is typically considered acceptable, depending on the amount of redundancy in the structure. For example, in pile foundations, the inadequacy of the resistance available for a single pile does not necessarily mean that the entire foundation will fail, as adjacent piles that may have greater capacity could take some of the additional load (Zhang, et al., 2001). Reinforced soil walls depend on many reinforcement layers or strips for their internal stability, and the failure or overstress of a single layer or reinforcing strip will not result in complete failure of the wall. Allen and Bathurst (2001) provided examples of how this redundancy comes into play to provide an internally stable system. D'Appolonia (1999) and Paikowski and Stenersen (2001) determined resistance factors for permanent reinforced soil walls and pile foundations, respectively, by using a probability of failure of 1 in 100 because of this inherent redundancy. Zhang, et al. (2001) indicated that, because of redundancy, an even lower probability of failure may be acceptable for evaluating limit states for a load-carrying element within a group of load-carrying elements to produce the desired probability of failure for the group.

Several methods are available for estimating the probability of failure for a given β_τ value. Withiam et al. (1998) provided a relationship between β_τ and P_f that is an approximation and that tends to be overly conservative at low β values. The approach recommended by Withiam et al. (1998) results in $\beta_\tau = 2.5$ for P_f of 1.0 percent. According to Paikowsky and Stenersen (2001) and Zhang, et al. (2001), the most exact method for determining the relationship between β_τ and

P_f results in β_τ = 2.33 for P_f = 1.0 percent. A value of β_τ of 2.33 was used to determine the resistance factors provided herein for permanent structures. Table 4 provides the resistance factors that result from this analysis.

For temporary structures, a higher probability of failure may be tolerated, depending on the nature of the structure (i.e., the consequences of failure and the potential for error in the characterization of the load and resistance distributions should be considered). No specific guidance on this issue is available. Increasing the probability of failure to approximately 2.5 percent (β_τ of 2.0), or possibly even higher, is conservatively reasonable for temporary reinforced soil structures, especially given their inherent redundancy, and that the consequence of reinforcement overstress is typically manifest as increased deformation rather than catastrophic collapse (see Table 4 for the resulting resistance factors).

Table 2. Load factors and load statistical parameters used for resistance factor calibration (K_0-Stiffness Method, all strength limit states).

Wall Reinforcement Type	Parameter	Value
Geosynthetic	γ_{EH}	1.65
	Number of Data Points, n_{EH}	56
	Bias Factor for the Reinforcement Load, λ_{QEH}	0.98
	COV_{QEH}	0.327
Steel	γ_{EH}	1.5
	Number of Data Points, n_{EH}	97
	Bias Factor for the Reinforcement Load, λ_{QEH}	0.95
	COV_{QEH}	0.271
All	Q_{EH}/Q_L	10
	γ_L	1.75
	Bias factor for the live load, λ_{QL}	1.15
	COV_{QL}	0.18

Table 3. Statistical parameters used for resistance factor calibration (reinforcement rupture and pullout).

Limit State	Reinforcement Type	Number of Data Points Available	Parameter	Value
Soil Failure	Steel Strip[1]	65	Bias Factor for Resistance (F_y), λ_R	1.09
			COV_R	0.059
Reinforcement Rupture	Woven geotextile	162	Bias Factor for Resistance, λ_R	1.08
			COV_R	0.184
	HDPE geogrid[3]	250	Bias Factor for Resistance, λ_R	1.05
			COV_R	0.05
	PP geogrid[3]	230	Bias Factor for Resistance, λ_R	1.05
			COV_R	0.0889
	PET geogrid[3]	634	Bias Factor for Resistance, λ_R	1.06
			COV_R	0.09
	Steel grid[2]	22	Bias Factor for Resistance (F_u), λ_R	1.13
			COV_R	0.081
	Steel strip[1]	65	Bias Factor for Resistance (F_u), λ_R	1.18
			COV_R	0.10
Reinforcement Pullout*	Geogrid	159	Bias Factor for Resistance, λ_R using 0.8Tan ϕ	1.17
			Bias Factor for Resistance, λ_R using 0.67Tan ϕ	1.40
			COV_R	0.41
	Ribbed steel strip (at depth greater than 2 m below wall top)	56	Bias Factor for Resistance, λ_R	1.70
			COV_R	0.29
	Ribbed steel strip (within 2 m of wall top only)	22	Bias Factor for Resistance, λ_R	2.26
			COV_R	0.31
	Smooth steel strip	42	Bias Factor for Resistance, λ_R	3.3
			COV_R	0.45
	Steel grid	38	Bias Factor for Resistance, λ_R	1.32
			COV_R	0.39

*Source of statistical data for pullout is from D'Appolonia (1999).
[1]Source of statistical data is from Anderson (personal communication).
[2]Source of statistical data is from Hilfiker (personal communication)
[3]The bias and COV values for these reinforcement types are approximate.

Table 4. Calculated K_0-Stiffness Method load and resistance factors for the strength limit state (reinforcement rupture and pullout).

Limit State	Load Factor, γ_{EH}	Reinforcement Type	Calculated Resistance factor for $B_\tau = 2.0$	Calculated Resistance factor for $B_\tau = 2.33$
Soil Failure	1.65	Geosynthetic reinforcement	*	*
	1.5	Steel Strip Reinforcement	0.94	0.86
Reinforcement Rupture	1.65	Woven geotextile	0.84	0.75
		HDPE geogrid	0.90	0.81
		PP geogrid	0.88	0.79
		PET geogrid	0.89	0.80
	1.5	Steel grid	0.97	0.88
		Steel strip	1.0	0.91
Reinforcement Pullout (using AASHTO default values	1.65	Geogrid (calc. using 0.67Tan ϕ)	0.79	0.67
	1.5	Ribbed steel strip (at depth greater than 2 m below wall top)	1.2	1.0
		Ribbed steel strip (within 2 m of wall top only)	1.5	1.3
		Smooth steel strip	1.7	1.5
		Steel grid	0.76	0.66

*If lower bound limit reinforcement strain of 2.5% or less is used, assume resistance factor is equal to 1.0.

For the reinforcement rupture limit state, the way the design ultimate tensile strength of the reinforcement is determined has a significant impact on the magnitude of the resistance factor needed. It must be recognized that minimum specification values are typically used to select soil wall reinforcements. For geosynthetic reinforcement, a Minimum Average Roll Value (MARV) is used for T_{ult}. The MARV is defined as the value that is two standard deviations below the mean. The use of these minimum specification values can be taken into account through the bias factor.

On the basis of the data used by Allen and Bathurst (1994) to compare virgin and installation damaged geosynthetic tensile strength, the COV of damaged geosynthetics could be determined directly. All that was left to be determined was the bias caused by using the MARV for the ultimate tensile strength. The bias factor was determined by using the average tensile strength of the undamaged material as the measured T_{ult}, and the average tensile strength minus two standard deviations (again for the undamaged material) as the predicted T_{ult}. It is possible that the bias factor could be greater than this, since the MARV is determined from a much larger data set, but using the available population sample should produce conservative results. In addition, chemical durability could introduce some additional uncertainty. However, previous research (Elias, 2001) has suggested that strength losses due to durability are relatively small, and all strength losses observed to date have been strongly dominated by installation damage effects. Therefore, the resistance factor calculated as described above was considered to be adequate. Uncertainty in extrapolating creep data is also a potential consideration that could affect this resistance factor, but this uncertainty is normally already accounted for in the determination of RF_{CR} using current methodologies (Elias, et al., 2001; WSDOT, 1998). Therefore, additional modification to the resistance factor for extrapolation uncertainty is normally not needed.

The resistance factors for geosynthetic rupture assume an average level of installation damage. For lightweight woven geotextiles or other geosynthetics that are more susceptible to installation damage strength losses and that are subjected to relatively severe installation conditions (i.e., angular gravels), the resistance factor should be lowered by 0.05 from that shown in the table. Doing so accounts for the additional variability in the strength caused by more severe installation damage (see Table 4, comparing the resistance factor for woven geotextiles to the resistance factors for the other geosynthetics).

A similar approach was used for estimating the resistance factor for steel reinforcement. The bias factor was determined as the average tensile strength (measured strength) divided by the minimum specification value (predicted strength).

Note that the resistance factors for ribbed steel strip pullout are greater than 1.0 even for normal static loading. This is the result of the lower bound nature of the model being used to predict the pullout capacity of ribbed steel strips, especially at low overburden pressures (i.e., near the wall top).

Because the K_0-Stiffness Method was calibrated with measured peak soil friction angles, which for many of the wall case histories analyzed were greater than 40^o, selection of a more conservative friction angle for design will add further conservatism to the design. Past design practice for geosynthetic walls suggest that this built-in conservatism due to conservative design friction angle selection can result in an average hidden load factor of 2.0 in the value of T_{max} (the actual "hidden" factor could vary, depending on the strength of the backfill selected and the design friction angle used – see Allen and Bathurst, 2001). If local experience has shown that design soil friction angles are conservative, it would be reasonable to reduce this load factor to 1.2 to 1.5, based on engineering judgment, to take into account this hidden soil strength parameter conservatism and to use this reduced load factor with the resistance factors provided in

Table 4. Note, however, that hidden conservatism resulting from the soil parameter selection is likely greater for geosynthetic walls than it is for steel reinforced walls. Geosynthetic reinforced systems can take greater advantage of high peak shear strengths because of the larger strains in the composite soil mass than can steel reinforced systems (Allen and Bathurst, 2001). If a conservative design soil friction angle is used for steel reinforced systems, it is recommended to reduce the load factor to no less than approximately 1.4, as the reinforcement loads in steel reinforced walls are less dependent on the soil friction angle.

Connection Load and Strength Calibration Issues

The detailed statistical data needed to assess connection load variability are not available at this time. Therefore, a load factor for designing the connections can only be assessed on the basis of experience and previous design practice. Current practice for geosynthetic walls is to use the same load and resistance factors for both the connection and the reinforcement design.

For reinforcement-facing connections in steel reinforced walls, current US design practice (e.g., AASHTO, 1999) in allowable stress design is to increase the factor of safety by 1.15 for steel bar mat and welded wire reinforcement relative to individual strip reinforcement to account for local overstress between longitudinal wires or bars connected together transversely and connected to a rigid facing panel.

On the basis of the potential factors described previously that can affect connection loads in walls with relatively stiff facings, the load factor for connection load design should be greater than or equal to the load factor for designing the reinforcement in the backfill. Theoretically, these connection load variances could occur for any wall with a very stiff facing, regardless of the connection details or reinforcement type. However, in current practice, an increased level of safety is only required for welded wire and bar mats attached to stiff facings. At least some limited evidence indicates that greater variability exists in the connection loads for segmental concrete block facings (Bathurst, et al., 2001) and uneven loading of the longitudinal members of bar mat and welded wire reinforcement attached to stiff facing panels (AASHTO, 1999, in press). Until more is known, a load factor γ_{con} of 1.9 for connection design is suggested for walls with segmental concrete block facings, and a load factor of 1.75 for connection design is suggested for welded wire and bar mat reinforcements connected to any stiff facing, even if a conservative lower bound soil friction angle is used for determining T_{max}.

The statistical analysis needed to determine the resistance factor for connection strength for partially or fully frictional connections to segmental concrete facing blocks is yet to be completed. Intuitively, greater uncertainty may exist in the determination of this type of connection strength than exists in the determination of reinforcement strength in the backfill. Sources of additional uncertainty in the connection strength for partial or fully frictional connections include unevenness and abrasion at the facing-reinforcement load transfer point, geogrid junction strength, and variations in the frictional resistance holding the connection in place. Again, current practice is to not recognize these potential additional uncertainties for these types of connections.

For mechanical connections, variations in the shear strength of the connectors such as bolts, bolt hole sizes, and transverse bar welds need to be considered. Current design specifications in the United States (e.g., AASHTO, in press) do not consider any additional reduction in resistance factors at the connection (other than the uneven load issue discussed above for bar mat and welded wire reinforcement) relative to the resistance factors for the reinforcement in the backfill. This suggests that a resistance factor similar to that recommended for the reinforcement in the backfill could be used at the connection for mechanical connections, given current design practice.

Summary of Recommended Load and Resistance Factors for Design

Recommended load and resistance factors for use with the K_0-Stiffness Method are provided in Table 5. Table 6 compares these load and resistance factors with the load and resistance factors in the current AASHTO specifications (AASHTO in press), by comparing the ratio of (γ_{EH}/ϕ) obtained from the results in Table 5 to what is used in current practice based on the data provided in Table 1.

Table 5. Load and resistance factors recommended for the K_0-Stiffness Method.

Limit State	Failure Mode	Reinforcement Type	Load Factor	Resistance Factor
Strength*	Reinforcement Rupture, φ_{rr}	• Steel reinforcement • Geosynthetic	• 1.5 • 1.65	• 0.90 • 0.80
	Soil Failure, φ_{sf}	• Steel reinforcement • Geosynthetic	• 1.5 • 1.65	• 0.85 • 1.0^{+}
	Pullout, φ_{po}	• Steel Strips (at z > 2 m) • Steel Strips (at z ≤ 2 m) • Steel grids • Geosynthetics	• 1.5 • 1.5 • 1.5 • 1.65	• 1.0 • 1.3 • 0.70 • 0.70
	Connector Rupture, φ_{cr}	• Steel strip, grids, and welded wire • Steel grid connected to rigid facing element • Geosynthetic (wrapped face and stiff or flexible panels) • Geosynthetic (segmental concrete block facings)	• 1.5 • 1.75 • 1.65 • 1.9	• 0.90 • 0.90 • 0.80 • 0.80
Service#	All, φ_s	All	1.0	1.0
Extreme Event 1x	Reinforcement or Connector Rupture, φ_{Eqr} and Soil Failure, φ_{sf}	• Steel strip, grids, and welded wire • Steel grid connected to rigid facing element • Geosynthetic (wrapped face and stiff or flexible panels) • Geosynthetic (segmental concrete block facings)	• 1.5 static, 1.0 seismic • 1.75 static, 1.0 seismic • 1.65 static, 1.0 seismic • 1.9 static, 1.0 seismic	• 1.05 • 1.05 • 0.95 • 0.95
	Pullout, φ_{EQp}	• Steel Strips (at z > 2 m) • Steel Strips (at z ≤ 2 m) • Geosynthetics • Steel grid	• 1.5 static, 1.0 seismic • 1.5 static, 1.0 seismic • 1.65 static, 1.0 seismic • 1.5 static, 1.0 seismic	• 1.25 • 1.6 • 0.90 • 0.85
Temporary	Reinforcement or Connector Rupture, φ_{trr} and Soil Failure, φ_{sf}	• Steel • Geosynthetic	• 1.5 • 1.65	• 0.95 • 0.85
	Pullout, φ_{tp}	• Steel Strips (at z > 2 m) • Steel Strips (at z ≤ 2 m) • Geosynthetics • Steel grid	• 1.5 • 1.5 • 1.65 • 1.5	• 1.2 • 1.5 • 0.80 • 0.75

*For steel, resistance factors are relative to F_u (minimum specification values) of the steel. For geosynthetics, resistance factors are relative to T_{ult} (MARV). For pullout, resistance factors are relative to AASHTO default values (AASHTO, in press) for pullout (see also Figure 2).

+If lower bound default value of 2.5% or less is used.

#Use resistance factors in current LRFD specifications (AASHTO, in press) until a more complete calibration for the Serviceability limit state can be completed.

XDetermined by increasing strength limit state resistance factors by 30%, per overstress allowance in AASHTO (1999).

Appendix A provides the load statistics and resulting load and resistance factors for the AASHTO (1999, in press) Simplified Method using the same data (i.e., Table 3) as is used to calibrate the K_0-Stiffness Method. The load and resistance factors in the current AASHTO (in press) LRFD design specifications were calibrated to current Allowable Stress Design (ASD) practice so that the LRFD wall designs would yield the same degree of conservatism as current ASD practice (D'Appolonia, 1999). Table 6 also provides a comparison of the Simplified Method calibration to the current AASHTO (in press) load and resistance factors. The ratio of (γ_{EH}/φ) discussed previously is used.

Table 6. Ratio of load to resistance factors for the strength limit state for the current AASHTO specifications and for the calibration results from Table 5 and Table A2.

Limit State	Reinforcement Type	Ratio, γ_{EH}/φ		
		Current LRFD Practice per Table 1 (AASHTO, in press)	**Simplified Method, Based on Calibration Provided in Table A2**	**K_0-Stiffness Method, Based on Calibration Provided in Table 5**
Reinforcement Rupture	All geosynthetic walls	1.5	0.9	2.05
	Geosynthetic (segmental concrete block facings at connection)	1.5	0.9	2.35
	Steel grid (attached to rigid facing, at connection)	2.1 (relative to F_y), 2.6 (relative to F_u)	2.0 (relative to F_y), 2.35 (relative to F_u)	1.95 (relative to F_u)
	Steel grid (attached to flexible facing)	1.8 (relative to F_y), 2.2 (relative to F_u)	1.75 (relative to F_y), 2.05 (relative to F_u)	1.7 (relative to F_u)
	Steel strip	1.8 (relative to F_y), 2.1 (relative to F_u)	1.75 (relative to F_y), 2.05 (relative to F_u)	1.7 (relative to F_u)
Reinforcement Pullout (using AASHTO, in press, default values)	Geosynthetic (0.67Tan ϕ)	1.5	0.95	2.35
	Ribbed steel strip (at depth greater than 2 m below wall top)	1.5	1.5	1.5
	Ribbed steel strip (within 2 m of wall top only)	1.5	1.15	1.15
	Steel grid	1.5	2.3	2.0

As can be seen from these results, the case of steel grid pullout in current design practice does not yield the target probability of failure of 1.0 percent used for the calibrations reported herein for the Simplified Method. However, for geosynthetic wall design in general, the current design practice is conservative relative to what is needed to yield a probability of failure of 1.0 percent. Table 6 also shows that for geosynthetic reinforced walls, a more conservative combination of load and resistance factors is needed for the K_0-Stiffness Method than for the Simplified Method, which is not surprising given that the K_0-Stiffness Method is much less conservative than the Simplified Method. For steel reinforcement rupture, the proposed load and resistance factor combination for the K_0-Stiffness Method is less conservative than is currently used for the Simplified Method. This is also not surprising given that the K_0-Stiffness Method is more accurate overall than the Simplified Method in predicting loads. For pullout, the combination of the load and resistance factors for the K_0-Stiffness Method is about the same as current practice for steel strips, but significantly more conservative for steel grids. The key appears to be in the bias of the default method in the current AASHTO specifications for estimating the pullout of steel grids; it is not set low enough to truly be a default pullout estimate (i.e., there is too much risk of overestimating the pullout resistance by using the default procedure). In any case, this can be taken into account through a lower resistance factor, as has been proposed herein.

Note also that because the Simplified Method has greater limitations regarding the range of wall project conditions that are applicable than does the K_0-Stiffness Method (see Appendix A), the size of the dataset used to develop load and resistance factors for the Simplified Method was necessarily smaller than the size of the dataset used to develop load and resistance factors for the K_0-Stiffness Method. For example, the Simplified Method does not successfully model steel reinforced walls with backfill friction angles of greater than 44^{o} plane strain, nor is it well-suited to modelling heavily battered walls and polymer strap walls. Therefore, all of the walls that fit into these "problem" categories had to be removed from the dataset for the Simplified Method to properly determine load and resistance factors. However, the K_0-Stiffness Method was developed with a larger number of wall case studies and a wider range of wall scenarios, and this same larger dataset was used to determine load and resistance factors. Because of the significant difference in the sizes of the datasets, the comparison between the load and resistance factors needed for the Simplified and K_0-Stiffness Methods provided in Table 6 should be considered approximate.

STEP-BY-STEP PROCEDURES FOR SOIL WALL REINFORCEMENT SPACING, STRENGTH, AND STIFFNESS DESIGN

Overview

For a specific, predetermined wall section, the K_0-Stiffness Method can be used directly to estimate loads in the reinforcement and to check that reinforcement loads do not exceed desirable levels. However, for design, it is usually desirable to determine the minimum amount of reinforcement, in terms of properties and spacing, required to produce an internally stable wall. Therefore, the global and local stiffness of the reinforcement must be adjusted to produce the desired level of strain, and to ensure that the ultimate resistance of the reinforcement is great enough to preclude reinforcement rupture within the design life of the wall.

Figures 11 and 12 summarize the process and steps needed to complete an internally stable reinforced soil wall design. Note that the design process is set up to consider specific limit states, and is designed to be compatible with limit states design protocols such as AASHTO Load and Resistance Factor Design (LRFD) (AASHTO, in press).

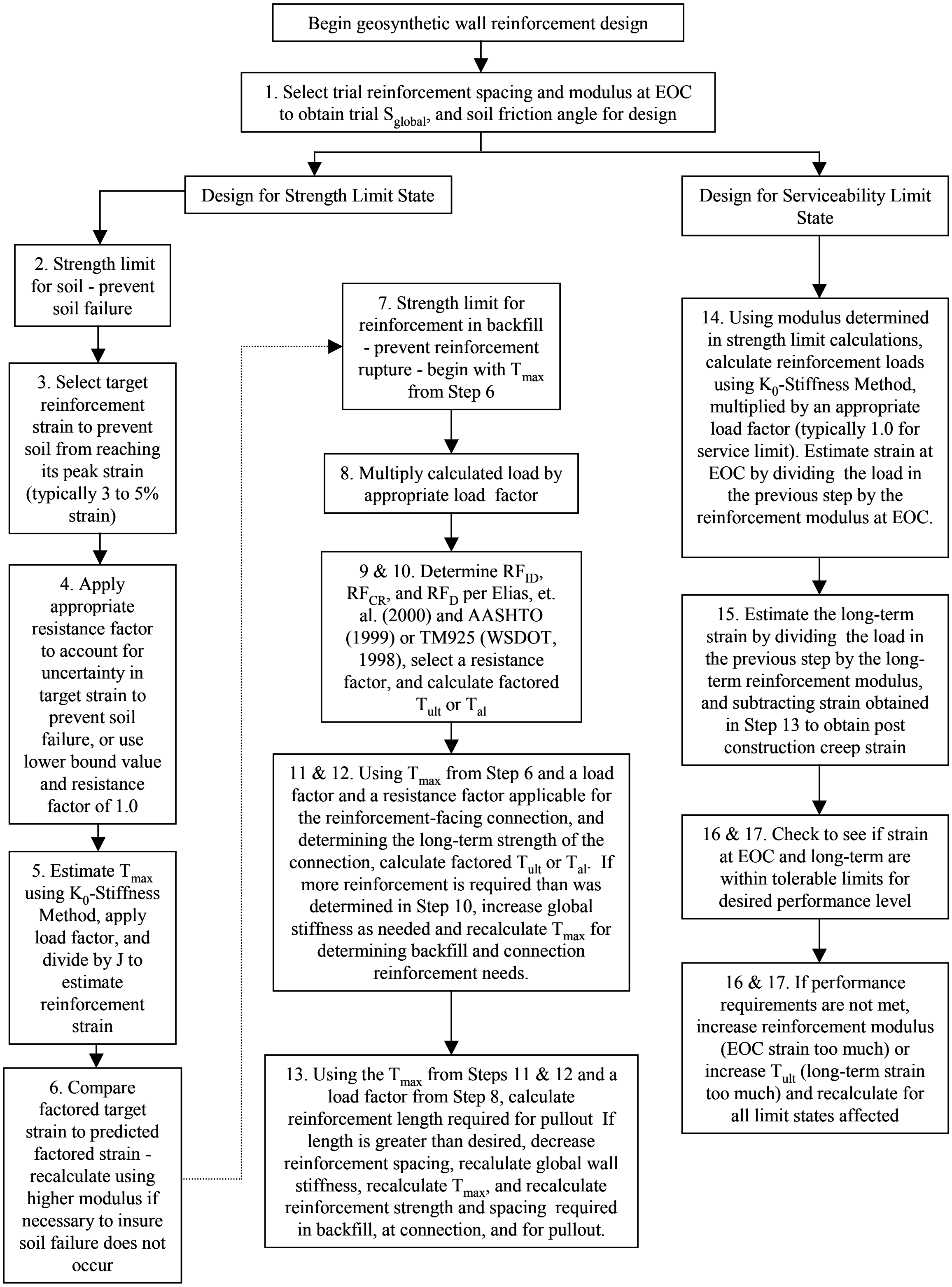


Figure 11. Design flowchart for geosynthetic wall internal stability.

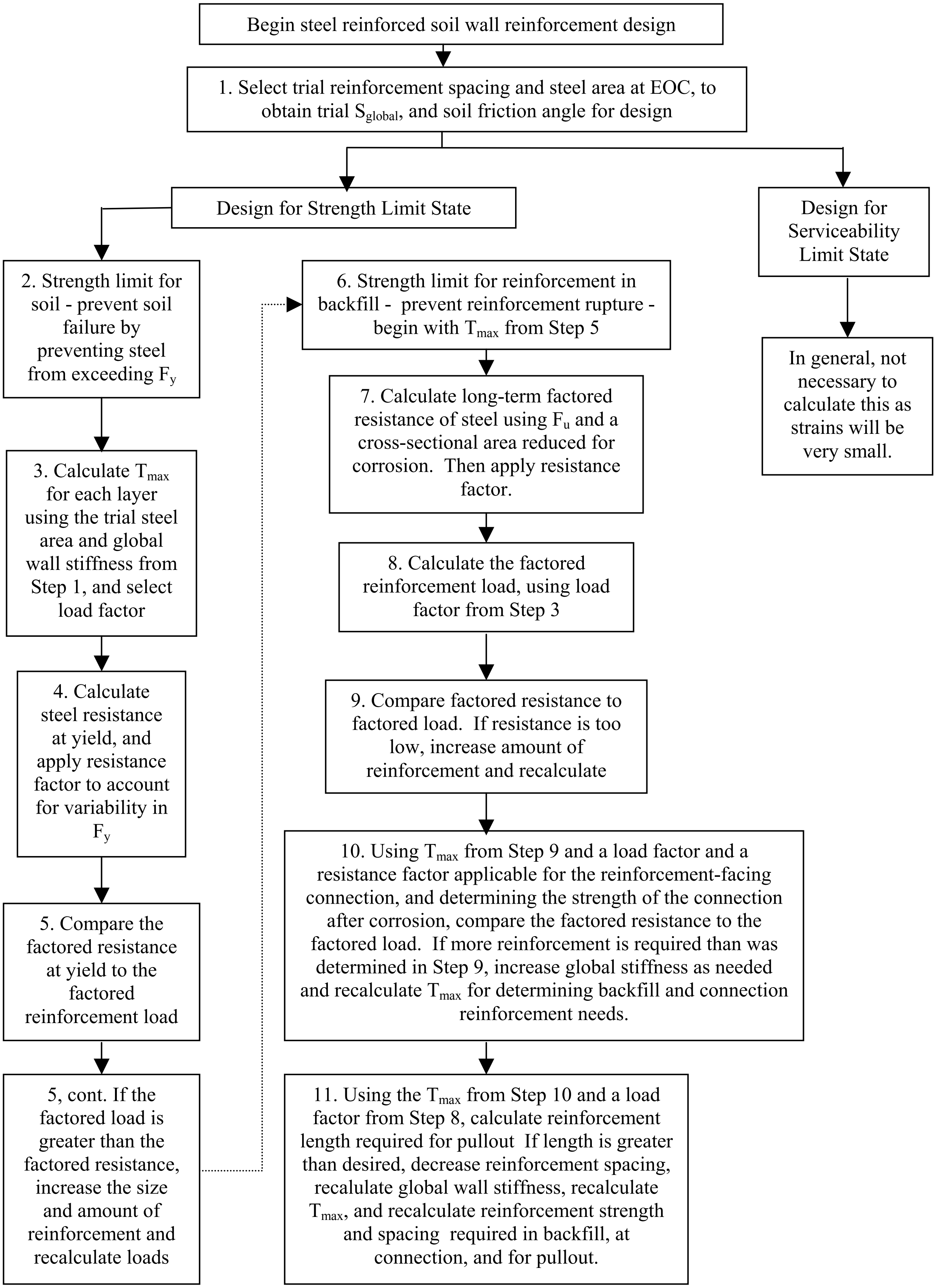


Figure 12. Design flowchart for steel reinforced soil wall internal stability.

Step-by-Step Procedures for Geosynthetic Wall Design

For geosynthetic walls (see Figure 11), two strength limit states (soil failure and reinforcement failure) and one serviceability limit state must be considered for internal reinforcement strength and stiffness design. The design steps, and related considerations, are as follows:

1. Select a trial reinforcement spacing and a trial reinforcement modulus based on the time required to reach the end of construction. If the estimated time required to construct the wall is unknown, an assumed construction time of 1,000 hours should be adequate. Note that at this point in the design, it does not matter how one obtains the modulus. It is simply a value that one must recognize is an end of wall construction modulus determined through isochronous stiffness curves by potential materials suppliers for the constructed wall. Use this modulus to calculate the trial global stiffness of the wall, S_{global}. Also select a soil friction angle for design. Allen and Bathurst (2001) and others (e.g., Zornberg, et al., 1998) show that the reinforcement load is best estimated by using the peak plane strain soil friction angle. Therefore, if the soil shear strength characteristics of the fill likely to be used in the wall are reasonably well known at the time of design, a relatively high design soil friction angle can be used.

Strength Limit State to Prevent Backfill Failure

2. Begin by checking the strength limit state for the backfill soil. The goal is to select a modulus that is stiff enough to prevent the soil from reaching a failure condition.
3. Select a target reinforcement strain, ε_{targ}, to prevent the soil from reaching its peak shear strain. The worst condition in this regard is a very strong, high peak friction angle soil, as the peak shear strain for this type of soil will be lower than the peak shear strain obtained from most backfill soils. As discussed previously, the results of full-scale wall laboratory testing showed that the reinforcement strain at which the soil begins to exhibit signs of failure is on the order of 3 to 5 percent for high shear strength sands (Allen and Bathurst, 2001). This empirical evidence reflects very high shear strength soils and is probably a worst case for design purposes, in that most soils will have larger peak shear strain values than the soils tested in the full-scale walls. A reasonable lower bound

default value for ε_{targ} that should be adequate for granular soils is 2.5 percent. Lower target strains could also be used.

4. Apply a resistance factor to the target strain to account for uncertainty in the target strain to prevent soil failure, as shown below:

$$\varepsilon_{\text{targf}} = \varepsilon_{\text{targ}} \varphi_{sf} \tag{15}$$

where ϕ_{sf} is the resistance factor to account for uncertainties in the target strain, and other variables are as defined previously. If a lower bound value of ε_{targ} is used, a resistance factor of 1.0 will be adequate.

5. Estimate the factored strain in the reinforcement, ε_{reinf}, using the K_0-Stiffness Method as follows:

$$\varepsilon_{re\,\inf} = \left(\frac{T_{\max}}{J}\right)\gamma_{EH} \tag{16}$$

where γ_{EH} is the load factor for the reinforcement load as determined for use with the K_0-Stiffness Method. To determine T_{max}, the facing type must be selected to determine Φ_{fs}. The local stiffness factor Φ_{local} for each layer can be set to 1.0, unless the reinforcement spacing or stiffness within the design wall section will be varied. K_0 must also be determined to determine T_{max} by selecting a design soil friction angle.

6. If ε_{reinf} is greater than ε_{targf}, increase the reinforcement modulus J and recalculate T_{max} and ε_{reinf}. The modulus obtained from this design step is an end-of-construction (EOC) modulus; it will be used later in the design process to check the serviceability limit state. If serviceability is found to be acceptable, this modulus will become the modulus used for specifying the material. To keep things simple, 1,000 hours could be used as a standard EOC time for determining the modulus specified for product selection in much the same way T_{al} is used (see Step 10).

Strength Limit State to Prevent Reinforcement Rupture

7. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term rupture strength of the reinforcement is

greater than the load calculated from the K_0-Stiffness Method. T_{max} calculated from Step 6 is a good starting point for evaluating this limit state. Note that the global wall stiffness for this calculation is based on the EOC modulus of the reinforcement, as the reinforcement loads should still be based on EOC conditions, even though the focus of this calculation is at the end of the service life for the wall.

8. Using the load factor from Step 5, calculate the factored reinforcement load, T_{maxf}:

$$T_{\max f} = (T_{\max})\gamma_{EH} \tag{17}$$

9. Calculate the strength reduction factors RF_{ID}, RF_{CR}, and RF_D for the reinforcement type selected using the approach of Elias, et al. (2001), WSDOT Test Method 925 (WSDOT, 1998), or equivalent protocols. Because the focus of this calculation is to prevent rupture, these factors must be based on reinforcement rupture. Applying a resistance factor to address uncertainty in the reinforcement strength, determine T_{ult}, the ultimate tensile strength of the reinforcement as follows:

$$T_{ult} = \frac{T_{\max f} RF_{ID} RF_{CR} RF_D}{\varphi_{rr}} \tag{18}$$

where the variables are as defined previously. T_{ult} is determined from an index wide-width tensile test such as ASTM D4595 and is usually equated to the MARV for the product.

10. Step 9 assumes that a specific reinforcement product will be selected for the wall, as the strength reduction factors for installation damage, creep, and durability are known at the time of design. If the reinforcement properties will be specified generically to allow the contractor or wall supplier to select the specific reinforcement after contract award, use the following equation:

$$T_{aldesign} = \frac{T_{\max f}}{\varphi_{rr}} \tag{19}$$

where $T_{aldesign}$ is the long-term tensile strength of the reinforcement, accounting for installation damage, creep, and durability. The contractor can then select a product with the required $T_{aldesign}$.

Strength Limit State to Prevent Connection Rupture

11. If the geosynthetic reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. Calculate the long-term connection strength at each reinforcement level, taking into account the available normal force between the facing blocks, if the connection strength is a function of normal force, and apply a resistance factor as appropriate. The long-term unfactored connection strength, T_{ac}, is calculated with Equation 9.
12. Using the reinforcement load from Step 8 and an appropriate load factor for the connection load, determine the adequacy of the long-term reinforcement strength at the connection. Compare the factored connection load at each reinforcement level to the available factored long-term connection strength as follows:

$$T_{\max f} \leq \varphi_{cr} T_{ac} \tag{20}$$

If the reinforcement strength available is inadequate to provide the needed connection strength as calculated from Equation 9, decrease the spacing of the reinforcement or increase the reinforcement strength. Then recalculate the global wall stiffness and re-evaluate all previous steps to ensure that the other strength limit states are met.

Strength Limit State to Prevent Pullout

13. Determine the length of the reinforcement required in the resisting zone by comparing the factored T_{max} value to the factored pullout resistance available by using Equation 8. If the length of the reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

Serviceability Limit State

14. The final steps in this process are checks on the serviceability limit state. Keeping deflection in the wall system to within tolerable levels is the goal for serviceability design. Begin by calculating T_{max} using the K_0-Stiffness Method with the reinforcement modulus and other input values determined at Step 6. Then divide T_{max} by this modulus value to obtain the strain in the reinforcement layer, ε_{eoc}. Since the modulus was determined for the EOC, the strain calculated in this manner is also at the EOC.
15. Long-term post-construction strains can also be determined by recalculating T_{max} for each layer using the K_0-Stiffness Method and a long-term modulus of the reinforcement (i.e., at 10,000 hours after EOC as suggested in Step 16, or at the end of wall design life) determined from isochronous creep stiffness data. Installation damage effects will generally not need to be considered for this serviceability analysis, as demonstrated by the work of Allen and Bathurst (1996). Once T_{max} has been calculated, divide it by the long term modulus, J_{LT}, to determine the total long-term strain. Then subtract the strain at the EOC calculated from Step 15 to determine the post-construction strain.
16. Compare the EOC strain to the maximum tolerable strain criterion, if one is available, or use the calculated strain to estimate the maximum wall deflection (see Figure 4). An empirical relationship is also provided in AASHTO (in press) to estimate maximum wall deflection.
17. Compare the post-construction strain to the maximum tolerable post-construction strain criterion, if one is available. Figure 6 can be used to estimate the total creep deformation at the top of the wall based on the estimated post-construction strain. Allen and Bathurst (2001) suggest that post-construction wall face deformation should be limited to 30 mm in the first 10,000 hours after construction to ensure good long-term wall performance.

Step-by-Step Procedures for Steel Reinforced Soil Wall Design

For steel reinforced soil walls (see Figure 12), one serviceability limit state and two strength limit states (soil failure and reinforcement failure) must be considered for internal reinforcement strength and stiffness design. The design steps and related considerations are as follows:

1. Select a trial reinforcement spacing and steel area that is based on end-of-construction (EOC) conditions (i.e., no corrosion). Once the trial spacing and steel area have been selected, the reinforcement modulus, J, and wall global stiffness, S_{global}, can be calculated. Note that at this point in the design, it does not matter how one obtains the reinforcement spacing and area. They are simply starting points for the calculation. Also select a design soil friction angle for calculating K_0. Note that for steel reinforced wall systems, the reinforcement loads are not as strongly correlated to the peak plane strain soil friction angle as are the reinforcement loads in geosynthetic walls (Allen and Bathurst, 2001). This is likely due to the fact that the steel reinforcement is so much stiffer than the soil. The K_0-Stiffness Method was calibrated to a mean value of K_0 of 0.3 (this results from a plane strain soil friction angle of 44^o, or from triaxial or direct shear testing a soil friction angle of approximately 40^o). It is not recommended to go much higher than this for a design soil friction angle for steel reinforced walls until the soil friction angle issue is more fully resolved for steel reinforced soil walls (see Allen and Bathurst, 2001). Lower design soil friction angles can and should be considered for weaker granular backfill materials.

Strength Limit State to Prevent Backfill Failure

2. Begin by checking the strength limit state for the backfill soil. The goal is to select a reinforcement density (spacing, steel area) that is great enough to keep the steel reinforcement load below yield ($A_sF_yR_c/b$, which is equal to A_sF_y/S_h). F_y is the yield stress for the steel, A_s is the area of steel before corrosion (EOC conditions), and S_h is the horizontal spacing of the reinforcement (use S_h = 1.0 for continuous reinforcement). Depending on the ductility of the steel, once the yield stress has been exceeded, the steel can deform significantly without much increase in load and can even exceed the strain necessary to cause the soil to reach a failure condition. For this reason, it is prudent to limit the steel stress to F_y for this limit state. Tensile tests on corroded steel indicate that the steel does not have the ability to yield to large strains upon exceeding F_y, as it does in an uncorroded state, but instead fails in a brittle manner (Terre Armee, 1979). Therefore, this limit state only needs to be evaluated for the steel without corrosion effects.

3. Using the trial steel area and global wall stiffness from Step 1, calculate T_{max} for each reinforcement layer. Apply the load factor to T_{max}.
4. Determine and apply an appropriate resistance factor to A_sF_y/S_h.
5. Compare the factored load to the factored resistance, as shown in Equation 21 below. If the factored load is greater than the factored resistance, increase A_s and recalculate the global wall stiffness and T_{max}. Make sure that the factored resistance is greater than the factored load before going to the next limit state calculation.

$$T_{\max}\gamma_{EH} \leq \frac{A_s F_y}{b} R_c \varphi_{sf} = \frac{\mathrm{A_s F_y}}{\mathrm{S_h}} \varphi_{\mathrm{sf}} \tag{21}$$

where φ_{sf} is the resistance factor for steel reinforcement resistance at yield, and S_h is the horizontal spacing of the reinforcement.

Strength Limit State to Prevent Reinforcement Rupture

6. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term rupture strength of the reinforcement is greater than the load calculated from the K_0-Stiffness Method. Even though the focus of this calculation is at the end of the service life for the wall, the global stiffness for the wall should be based on the stiffness at the end of wall construction, as reinforcement loads do not decrease because of lost cross-sectional area resulting from reinforcement corrosion. T_{max} obtained from Step 5 should be an adequate starting point for this limit state calculation.
7. Calculate the strength of the steel reinforcement at the end of its service life, using the ultimate strength of the steel, F_u, and reducing the steel cross-sectional area, A_s, determined in Step 5, to A_c to account for potential corrosion losses. Then use the resistance factor φ_{rr}, as defined previously, to obtain the factored long-term reinforcement tensile strength, as shown below:

$$T_{al} = \frac{F_u A_c}{S_h} \varphi_{rr} \tag{22}$$

where F_u is the ultimate tensile strength of the steel, and A_c is the steel cross-sectional area per meter of wall length reduced to account for corrosion loss. The resistance factor is dependent on the variability in F_u, A_s, and the amount of effective steel cross-sectional area lost as a result of corrosion. As mentioned previously, minimum specification values are typically used for design with regard to F_u and A_s. Furthermore, the corrosion rates specified in current North American design codes and guidelines (AASHTO, 1999; Elias, et al., 2001) are also maximum rates based on the available data (Terre Armee, 1991). Recent post-mortem evaluations of galvanized steel in reinforced soil walls also show that AASHTO design specification loss rates are quite conservative (Anderson and Sankey, in press). Furthermore, these corrosion loss rates have been correlated to tensile strength loss, so that strength loss due to uneven corrosion and pitting is fully taken into account. Therefore, a high resistance factor of 0.90 as provided in Table 5, which is based on the variability of the un-aged steel, is reasonable to use in this case, assuming that non-aggressive backfill conditions exist.

8. Selecting an appropriate load factor, calculate the factored reinforcement load, T_{maxf}, using Equation 17.
9. Check to see if T_{ult}, the factored ultimate tensile strength available, is greater than T_{maxf}, the factored load that the reinforcement must carry. If not, increase the steel area, recalculate the global wall stiffness on the basis of the new value of A_s, reduce A_s for corrosion to obtain A_c, and recalculate T_{max} until T_{ult} based on Equation 22 is adequate to resist T_{maxf}.

Strength Limit State to Prevent Connection Rupture

10. If the steel reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. This connection capacity, reduced by the appropriate resistance factor, must be greater than or equal to the factored reinforcement load at the connection. If not, increase the amount of reinforcing steel in the wall, recalculate the global stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

Strength Limit State to Prevent Pullout

11. Determine the length of reinforcement required in the resisting zone by comparing factored T_{max} to the factored pullout resistance available using Equation 8. If the length of reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

Serviceability Limit State

12. The serviceability limit state is generally not an issue for steel reinforced walls. Working strains will generally be on the order of a few tenths of a percent strain, and facing deflections will be small. However, EOC wall face deformation can be evaluated with the wall deflection evaluation guidance provided in the AASHTO specifications (AASHTO, 1999) and in Christopher (1993), as well as by using Figure 4.

Design Sequence and Concluding Remarks Regarding Design Approach

A specific sequence of design steps has been proposed herein to complete the internal stability design of reinforced soil walls. Because global wall stiffness is affected by changes to the reinforcement design to meet various limit states, iterative calculations may be necessary. Depending on the specifics of the wall and reinforcement type, certain limit states may tend to control the amount of reinforcement required. It may therefore be desirable to modify the suggested design sequence to first calculate the amount of reinforcement needed for the limit state that is more likely to control the amount of reinforcement. Then perform the calculations for the other limit states to ensure that the amount of reinforcement is adequate for all limit states. Doing this will hopefully reduce the number of calculation iterations.

For example, for geosynthetic reinforced wrap-faced walls, with or without a concrete facia placed after wall construction, the reinforcement needed to prevent soil failure will typically control the global reinforcement stiffness needed, while pullout capacity is generally not a factor, and connection strength is not applicable. For segmental block-faced or precast panel-faced geosynthetic walls, the connection strength needed is likely to control the global reinforcement stiffness. However, it is also possible that reinforcement rupture or soil failure could control instead, depending on the magnitude of the modulus of a given reinforcement product relative to the long-term tensile strength needed. The key here is that the combination of the required modulus and tensile strength is realistic for the products available. Generally, pullout will not control the design unless reinforcement coverage ratios are low (low coverage ratios for segmental block facings are generally not recommended). If reinforcement coverage ratios are low, it may be desirable to evaluate pullout early in the design process. For steel strip, bar mat, wire ladder, and geosynthetic strip reinforced systems, pullout often controls the reinforcement needed because of the low reinforcement coverage ratios used, especially near the top of the wall. However, connection strength can also be the controlling factor. For welded wire wall systems, the tensile strength of the reinforcement usually controls the global wall reinforcement stiffness needed, though if the reinforcement must be connected to the facing (i.e., the facing and the reinforcement are not continuous), connection strength may control instead. Usually, coverage ratios are large enough for welded wire systems that pullout is not a controlling factor in the determination of the amount of reinforcement needed. In general for all steel reinforced systems, with the possible exception of steel mesh reinforcement, the soil failure limit state does not control the reinforcement design because of the very low strain that typically occurs in steel reinforced systems.

When a trial reinforcement global wall stiffness is selected for design, consideration should be given to the reinforcement configurations that are possible, including typical reinforcement spacing and stiffness values available for the specific type of wall being designed.

COMPARISON OF THE K_0-STIFFNESS METHOD TO PREVIOUS DESIGN PRACTICE

To illustrate the use of the K_0-Stiffness Method and the design steps detailed in the previous sections, examples are provided or summarized herein for both geosynthetic and steel reinforced soil walls selected from actual case histories presented by Allen and Bathurst (2001). The details of these case histories are summarized in Table 7. Typical cross-sections for each of the example case histories are provided in figures 13 through 19. Key input properties for these case histories are provided in the detailed examples and in tables 8 and 9.

First, an attempt was made to predict the reinforcement loads in these case histories using both the AASHTO Simplified Method and the K_0-Stiffness Method. Figures 20 through 27 show predicted and measured loads for each of these case histories. These figures show how these methods either over- or under-predict reinforcement loads relative to the measured loads, providing some insight into the results of the wall designs that follow.

The predictions of reinforcement loads provided in figures 20 through 27 say nothing about the appropriateness of the amount of soil reinforcement used in these wall case histories. To evaluate the amount of reinforcement required to resist the reinforcement loads, each of these case histories is designed in the examples that follow using current design methodology (the AASHTO Simplified Method) and the proposed design methodology (the K_0-Stiffness Method). These wall designs, in terms of the amount of reinforcement required for stability, are then compared to the actual amount of reinforcement used for the wall. Only the amount of reinforcement required for reinforced wall backfill stability is evaluated. It is recognized that other design considerations may control the amount of reinforcement required, such as compound stability (see AASHTO, 1999). For the purposes of these examples, only the stability of the reinforcement in the backfill and at the connection is addressed. See Elias, et al. (2001) and AASHTO (1999, in press) for complete internal stability design procedures.

To form a common basis of comparison between the designs for each wall, the global resistance to demand ratio concept presented by Allen and Bathurst (2001) is used. However, instead of using the total reinforcement load as calculated by the Simplified Method for the demand in the denominator of this ratio, as was done previously, the sum of the measured reinforcement loads from the actual case history will be used as the demand for the denominator.

Interpolation was used to estimate the “measured” reinforcement loads for layers that were not instrumented, so that a total measured reinforcement load for the wall could be obtained. All of these ratios will be calculated at the end of the service life for the wall, and are therefore long-term values.

The resistance to demand ratios are calculated as follows:

$$RD = \frac{R}{D} = \frac{\sum_{i=1}^{n} T_i}{D_m} \tag{23}$$

where R is the total resistance of the backfill reinforcement, D is the total demand, D_m is the total demand measured in the actual case history, and T_i is the tensile resistance of each reinforcement layer. Since only the reinforcement strength at the end of the service life for the wall is considered, T_i is the long-term reinforcement strength reduced to account for installation damage, creep, and durability losses for geosynthetics. It is also the long-term strength based on the ultimate tensile strength after corrosion for steel reinforcements.

Table 7. Summary of case histories used in design examples.

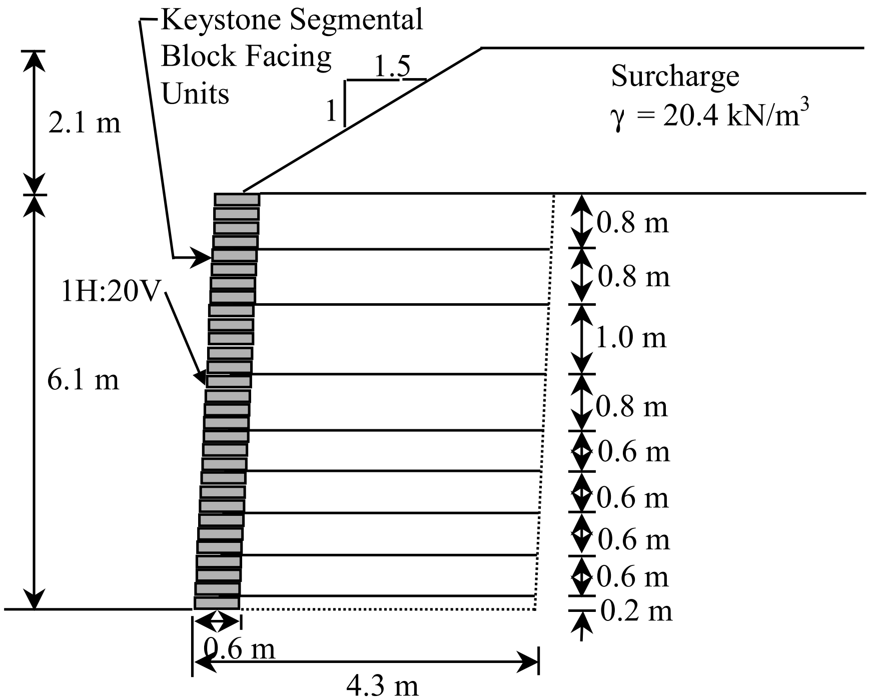
Wall Case History (Case History No.)	Date Wall Built	Wall Height (m)	Surcharge Conditions	Face Batter Angle from Vert. (^{o})	Reinforcement
Algonquin Miragrid Wall (GW9)	1988	6.1	2.1 m sloping surcharge	2.9	Miragrid 5T (PET geogrid)
Algonquin Tensar Geogrid Wall (GW8)	1988	6.1	2.1 m sloping surcharge	0	Tensar SR2 (HDPE geogrid)
Rainier Ave. Geotextile Wall (GW16)	1989	12.6	5.3 m sloping surcharge	2.9	GTF 200 (PP woven geotextile); GTF 375(PP woven geotextile); GTF 500(PP woven geotextile); GTF 1225T (PET woven geotextile)
RMCC Incremental Aluminium Panel Faced Geogrid Wall (GW15)	1989	3	Full test wall top coverage with air bag loading system, up to effective pressure of 60 kPa (actual surcharge pressure was 70 kPa)	0	Tensar SS1, weak direction (PP geogrid)
Algonquin Steel Strip Wall (SS11)	1988	6.1	None	0	50 mm x 4 mm (ribbed steel strip)
Bourron Marlotte Steel Strip Rectangular Test Wall (SS13)	1993	10.5	None	0	60 mm x 5 mm (ribbed steel strip)
Algonquin Steel Bar Mat Wall (BM3)	1988	6.1	None	0	Four W11 bars spaced at 150 mm c-c
Rainier Ave. Welded Wire Wall (WW1)	1985	16.8	0.3 m soil surcharge	0	W4.5xW3.5 for top 13 layers, W7xW3.5 for next 7 layers, W9.5xW3.5 for next 11 layers, and W12xW5 for bottom 7 layers, with all longitudinal wires spaced at 150 mm c-c

Table 8. Summary of design parameters for geosynthetic wall examples (except Example 1).

Design Parameter	**GW8**	**GW15, at 70 kPa surcharge (equiv. To "S" of 3.3 m, considering side wall friction)**	**GW16**
T_{ult} (kN/m)	67.8	12	Zone 1 – 31 Zone 2 – 62 Zone 3 – 92 Zone 4 - 186
S_{global} (kPa)	984	86	1,087
J (kN/m)	750	Layer 4 – 43.1 Layer 3 – 45 Layer 2 – 80 Layer 1 - 90	Zone 1 – 90 Zone 2 – 174 Zone 3 – 311 Zone 4 –1,126
γ (kN/m^3)	20.4	18	21.1
Measured Triaxial or Direct Shear ϕ_{tx}	40^o	50^o	45^o
Estimated Plane Strain ϕ_{ps}	43^o	55^o	54^o
RF_{ID}	1.15 to 1.25	1.0	Zone 1 – 1.3 Zone 2 – 1.1 Zone 3 – 1.05 Zone 4 –1.4
RF_{CR}	3.15	4.0	4.5, except 1.8 for Zone 4
RF_D	1.1	1.3	1.3, except 1.15 for Zone 4
CR_{cr}	0.9/($RF_{ID}RF_{CR}RF_D$)	1.0/($RF_{ID}RF_{CR}RF_D$)	Not applicable
D_m (kN/m)	24.85	5.15	81.69

Table 9. Summary of design parameters for steel soil wall examples (except Example 2).

Design Parameter	**SS13**	**BM3**	**WW1**
Reinforcement Type	Steel Strip, galvanized	Bar Mat, galvanized	Welded Wire Mat, not galvanized
T_{ult} (kN/m)	205 for top 10 layers, 256 for 11th layer, and 306 for bottom 3 layers	98.5	106 for top 13 layers, 166 for next 7 layers, 225 for next 11 layers, and 284 for bottom 7 layers
F_u (MPa)	520	520	550
F_y (MPa)	450	450	450
A_s (mm^2)	300	284	193 for top 13 layers, 301 for next 7 layers, 409 for next 11 layers, and 516 for bottom 7 layers
A_c (mm^2)	215	206	95.8 for top 13 layers, 175.4 for next 7 layers, 259.2 for next 11 layers, and 346.0 for bottom 7 layers
A_c at Connection (mm^2)	164	Not applicable	Not applicable
S_h (m)	0.76 for top 10 layers, 0.61 for 11th layer, and 0.51 for bottom 3 layers	1.5	1.0
R_c	0.079, 0.098, and 0.118 respective of S_h	0.284	1.0
S_{global} (kPa)	136,667	49,687	146,535
Steel Modulus, E (MPa)	200,000	200,000	200,000
γ (kN/m^3)	16.8	20.4	19.2
Measured Triaxial or Direct Shear ϕ_{tx}	37^o	40^o	43^o (but design for 40^o max. per Allen, et al. (2001)
Estimated Plane Strain ϕ_{ps}	40^o	43^o	48^o (but design for 44^o max. per Allen and Bathurst, 2001)
K_a (design)	0.25	0.22	0.27
D_m (kN/m)	379.1	74.1	970.7



Foundation soil is 5 m of dense gravelly sand or fine to medium sand underlain by very dense sandy silt

Figure 13. Cross-section for Algonquin PET geogrid segmental concrete block-faced wall (GW9).

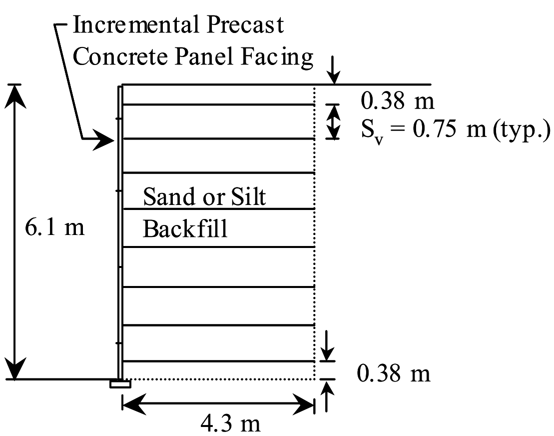
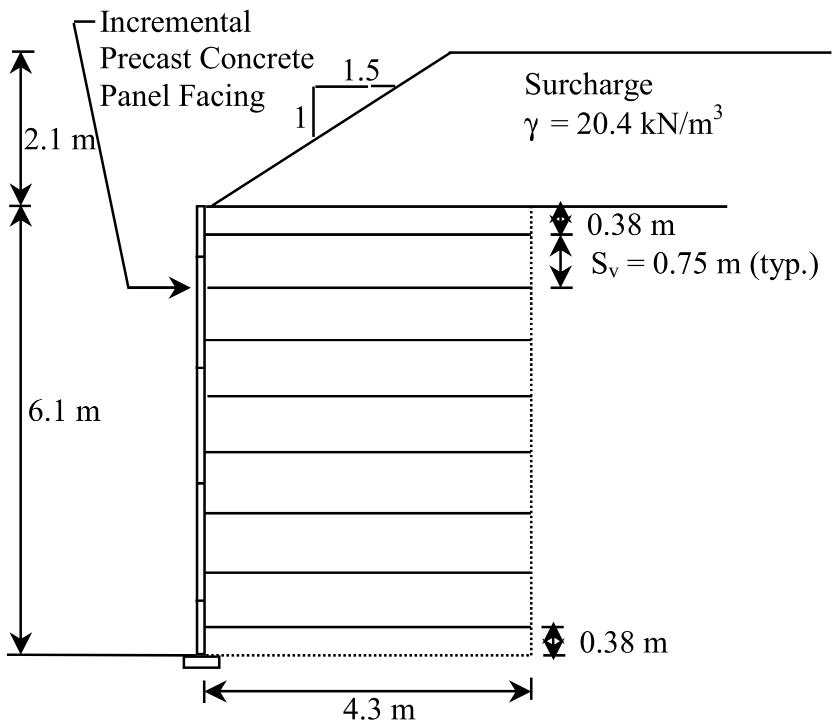
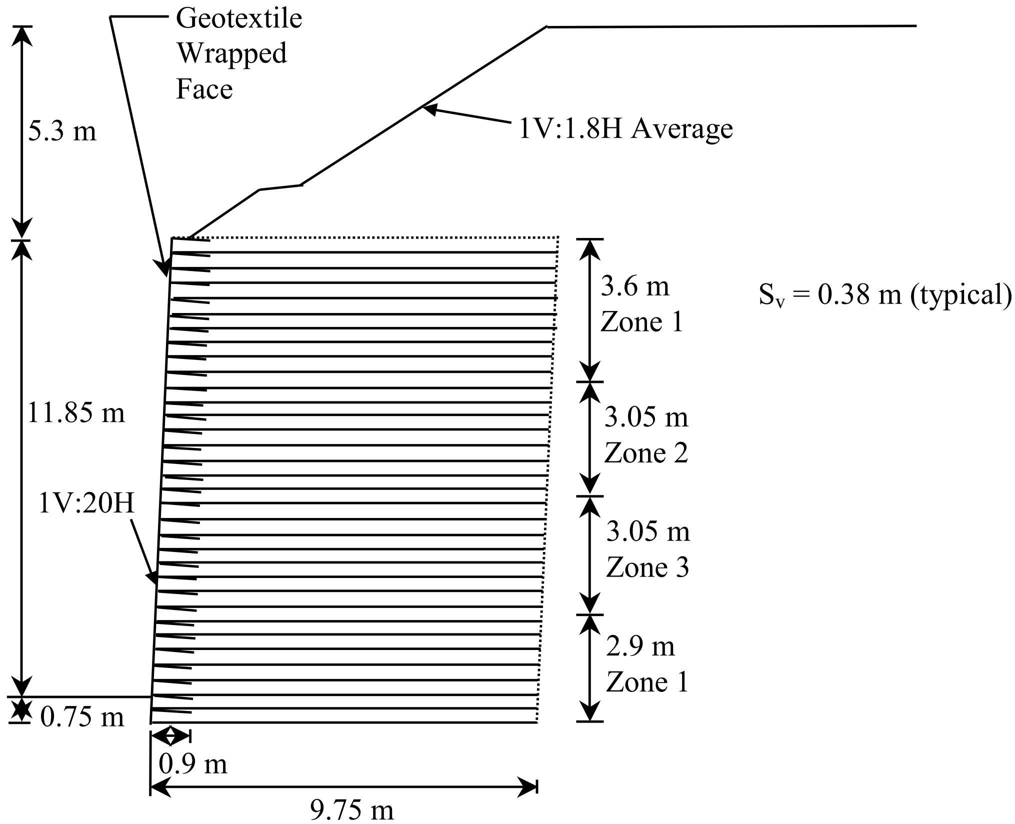


Figure 14. Algonquin steel strip and bar mat walls (SS11, BM3, and BM4).



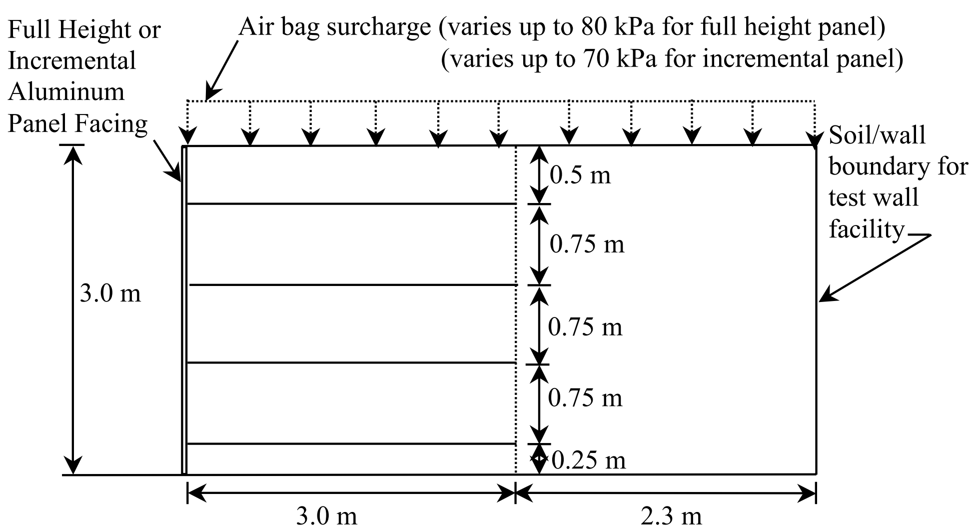
Foundation soil is 5 m of dense gravelly sand or fine to medium sand underlain by very dense sandy silt

Figure 15. Cross-section for Algonquin HDPE geogrid concrete panel wall (GW8).



Foundation soil consists of 6m of dense gravelly sand underlain by 1 to 3 m of soft clayey silt, which is underlain by very dense gravelly sand

Figure 16. Cross-section for WSDOT Rainier Avenue wrap-face geotextile wall (GW16).



Foundation for wall is concrete floor.

Figure 17. Cross-section for RMC incremental panel PP geogrid test wall (GW15).

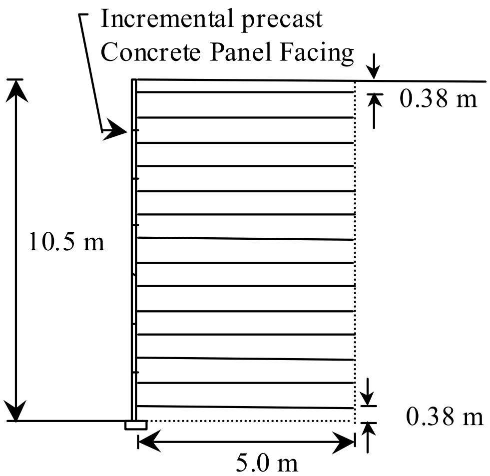


Figure 18. Bourron Marlotte steel strip test wall (SS13).

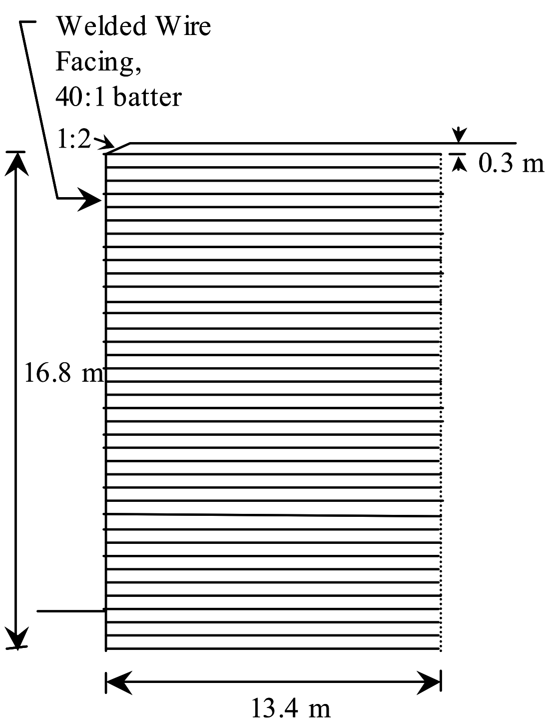


Figure 19. Rainier Avenue welded wire wall (WW1).

Geosynthetic Wall (Example 1)

First, the RD ratio for the actual wall is calculated. The current AASHTO approach using "best estimate" values for degradation mechanisms, or actual measured losses from exhumation and testing of the reinforcement, if reported, can be used to calculate available long-term reinforcement strength for comparison purposes. The available reinforcement capacity, T_i, for the actual reinforcement used in the case history is as follows:

$$T_i = \frac{T^i_{ult}}{RF_{actual}} = \frac{T^i_{ult}}{RF_{ID} \times RF_{CR} \times RF_D} \tag{24}$$

where RF_{actual} is calculated with the "best estimate" values for reduction factors RF_{ID}, RF_{CR}, and RF_D on the basis of site-specific conditions and measurements where available. The *estimated* long-term resistance-demand ratio can now be calculated as follows:

$$RD_{estimated} = \frac{\sum_{i=1}^{n} \frac{T^i_{ult}}{RF_{actual}}}{D_m} \tag{25}$$

Example 1 (Case Study GW9 – Actual Wall).

For the actual wall, calculate the estimated long-term resistance-demand ratio for Case Study GW9 (Figure 11) with surcharge in place. Material properties are as follows:

T_{ult} = 39.2 kN/m (PET geogrid)

S_{global} = 551 kPa

J = 420 kN/m

γ = 20.4 kN/m^3

Measured Triaxial $\phi = 40^o$, Estimated Plane Strain $\phi = 43^o$

K_a at 40^o = 0.20

S = 1.3 m

"Best estimate" and design reduction factors: $RF_{CR} = 1.85$, $RF_{ID} = 1.30$, $RF_D = 1.30$, and

$RF_{actual} = RF_{ID} \times RF_{CR} \times RF_D = 1.85 \times 1.30 \times 1.30 = 3.13$

Width of facing Unit, W_u = 0.6 m

Unit Weight of facing units, γ_u = 18.9 kN/m^3

CR_{cr} = (6.87 + $\sigma_N W_u$Tan 32^o)/(51.4RF_{CR}) for σ_N < 24 kPa, where T_{lot} = 51.4 kN/m

CR_{cr} = (12.3 + $\sigma_N W_u$Tan 13^o)/(51.4RF_{CR}) for $\sigma_N \geq$ 24 kPa, where T_{lot} = 51.4 kN/m

D_m = 24.85 kN/m

The estimated long-term resistance-demand ratio for the actual wall case history is calculated with Equation 25:

$$RD_{estimated} = \frac{\sum_{i=1}^{8} \frac{T_{ult}^{i}}{RF_{actual}}}{D_m} = \frac{8 \times \frac{39.2}{3.13} \text{ kN/m}}{24.85\ kN/\text{m}} = 4.03$$

Example 1, Continued (Case Study GW9 – Typical AASHTO Simplified Method Design – Long-Term).

Using the AASHTO Simplified Method, calculate the estimated long-term resistance-demand ratio for Case Study GW9 (Figure 13), with surcharge in place, using the triaxial soil friction angle of 40^o. Using triaxial shear strengths is consistent with the soil parameters used to develop and calibrate the Simplified Method (Allen, et al. 2001). A complete step-by-step design is not provided for the Simplified Method in this example, but only enough detail is shown to determine the RD ratio for prevention of reinforcement rupture, connection rupture, and pullout.

For reinforcement rupture, the *allowable* long-term design resistance-demand ratio calculated with the AASHTO method can expressed as follows:

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T_{aldesign}^{i}}{D_m} = \frac{\sum_{i=1}^{n} T_{\max} FS}{D_m} = \frac{\sum_{i=1}^{n} (z+S)\gamma K_a S_v FS}{D_m} \tag{26}$$

where $T_{aldesign}$ is the long-term strength of the reinforcement needed, T_{max} is the maximum reinforcement load in each layer, z is the depth of the layer below top of wall, S is the average surcharge depth above the wall, γ is the soil unit weight, K_a is the Coulomb active earth pressure coefficient, but with no wall interface friction, S_v is the tributary area of the reinforcement per unit length of wall, and FS is a safety factor, usually set to 1.5. $RD_{aldesign}$ can be obtained by matching the required reinforcement strength to the demand at each reinforcement layer level (termed "perfect match to demand"), or it can be calculated in accordance with typical practice using constant zones of strength and spacing (termed "typical practice"). Perfect match to demand, using the triaxial soil friction angle, is applied here.

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T_{aldesign}^{i}}{D_m} = \frac{\sum_{i=1}^{8} T_{\max} FS}{D_m} = \frac{155 kN/m}{24.85} = 6.23$$

Similarly, the reinforcement strength needed to provide adequate connection strength can be calculated (see AASHTO, 1999 for detailed calculation procedures). For connection strength,

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T_{aldesign}^{i}}{D_m} = \frac{298 kN/m}{24.85} = 12.0$$

The maximum ultimate reinforcement strength required for connection strength is 151 kN/m. This is a much stronger product than was used for the connection strength testing (T_{lot}, the lot or roll-specific index tensile strength of the product used for the connection testing, was 51.4 kN/m) from which the connection strength relationship used in this example was derrived. Typically, the short-term connection strength as a percentage of T_{ult} decreases somewhat as the weight and strength of the reinforcement product increases. Therefore, $RD_{aldesign}$ to meet connection strength needs could be even higher than 12.0, or modification to the block or connection system would be needed.

For pullout, L_e was determined to be 0.36 m at the wall top, which is significantly less than the minimum L_e allowed of 0.9 m in accordance with AASHTO (1999). The total reinforcement length required in this case is 3.2 m, which is significantly less than 70 percent of the total wall height (4.3 m). Therefore, there is no need to reduce the reinforcement spacing to improve pullout capacity. Connection strength controls the design of this wall.

Example 1, Continued (Case Study GW9 – K_0-Stiffness Method Design – Long-Term).

Calculate the estimated long-term resistance-demand ratio for Case Study GW9 (Figure 13) using the K_0-Stiffness Method with surcharge in place. Note that the K_0-Stiffness Method was calibrated to use the plane strain soil friction angle rather than the triaxial or direct shear soil friction angle, which in this case would be 43°. Therefore, a soil friction angle of 43° will be used.

Steps 1 and 2: Start with reinforcement stiffness and spacing from actual case history.

Steps 3 and 4: Select the target reinforcement strain to prevent soil failure. Use a ε_{targ} of 2.5 percent and treat it as a lower bound. Therefore, the recommended resistance factor is 1.0.

Step 5: Calculate T_{max} for each layer using the K_0-Stiffness Method as follows:

Use Figure 1(a) to determine D_{tmax}.Select Φ_{fs} = 0.5 because the wall is constructed with segmental concrete blocks. Use a coefficient of a = 1.0 for geosynthetics, and all other variables are as defined previously. At the top layer:

$$T_{max} = 0.5 S_v K_0 \gamma \, (H+S) D_{t\,max} \left(\frac{S_{local}}{S_{global}}\right)^a \Phi_{fb} \Phi_{fs} 0.27 \left(\frac{S_{global}}{p_a}\right)^{0.24}$$

$$\mathrm{T}_{max} = 0.5(1.2)(0.32)(20.4)(6.1+1.3)(0.55)\left(\frac{350}{551}\right)^1 (0.952)(0.5)(0.27)\left(\frac{551}{101}\right)^{0.24}$$

$$\mathrm{T}_{max} = 1.94 \text{ kN/m}$$

The determination of T_{max} for all of the layers is summarized below.

S_v (m)	K_o	D_{tmax}	Φ_{local}	Φ_{fb}	Φ_{fs}	Φ_g	T_{max} (kN/m)
1.2	0.32	0.550	0.635	0.952	0.5	0.406	1.94
0.9	0.32	0.899	0.847	0.952	0.5	0.406	3.18
0.9	0.32	1.000	0.847	0.952	0.5	0.406	3.54
0.7	0.32	1.000	1.09	0.952	0.5	0.406	3.54
0.6	0.32	1.000	1.27	0.952	0.5	0.406	3.54
0.6	0.32	1.000	1.27	0.952	0.5	0.406	3.54
0.6	0.32	0.790	1.27	0.952	0.5	0.406	2.79
0.4	0.32	0.397	1.91	0.952	0.5	0.406	1.40

The factored strain for the top layer, using a load factor applied to T_{max} of 1.65, can then be calculated as follows,

$$\varepsilon_{re\inf} = \left(\frac{T_{\max}}{J}\right)\Upsilon_{EH}$$

$$\varepsilon_{re\inf} = \left(\frac{1.94}{420}\right)(1.65)$$

$$\varepsilon_{re\inf} = 0.76\%$$

See Step 10 for T_{max} and ε_{reinf} for the remaining reinforcement layers.

Step 6: Check whether $\varepsilon_{targf} > \varepsilon_{reinf}$.

From Layer 5,

2.5 > 1.39? Yes. Therefore, proceed to Step 7.

Step 7: Determine ultimate tensile strength of reinforcement necessary to prevent reinforcement rupture for the strength limit state. Start with the T_{max} calculated in Step 5.

Step 8: Calculate the factored load, T_{maxf}, for each reinforcement layer.

$$T_{\max f} = (T_{\max})\Upsilon_{EH}$$

$$T_{\max f} = (1.94)(1.65) = 3.2 \text{ kN/m for layer } 8.$$

See the table in Step 10 for the rest of the layers.

Step 9: Calculate the factored ultimate resistance required to prevent reinforcement rupture as follows:

$$T_{ult} = \frac{T_{\max f} RF_{ID} RF_{CR} RF_D}{\varphi_{rr}}$$

$$T_{ult} = \frac{(3.2)(1.3)(1.85)(1.3)}{0.80}$$

$$T_{ult} = 12.5 \text{ kN/m for Layer } 8$$

See Step 10 for the values for the remaining layers.

Step 10. Alternatively, specifying the reinforcement properties generically is desired to allow the contractor or wall supplier to select the specific reinforcement after contract award, use the following equation:

$$T_{aldesign} = \frac{T_{\max f}}{\varphi_{rr}}$$

$$T_{aldesign} = \frac{3.2}{0.80} = 4.0 \text{ kN/m}$$

T_{max} ε_{rein}, T_{maxf}, T_{ult}, and $T_{aldesign}$ for all of the layers are summarized as follows:

Layer No.	**Unfactored Load T_{max} (kN/m)**	**Factored Strain ε_{reinf} (%)**	**Factored Load T_{maxf} (kN/m)**	**Ultimate Strength Required T_{ult} (kN/m)**	**Long-Term Strength Required $T_{aldesign}$ (kN/m)**
8	1.94	0.76	3.21	12.5	4.0
7	3.18	1.25	5.25	20.5	6.6
6	3.54	1.39	5.83	22.8	7.3
5	3.54	1.39	5.83	22.8	7.3
4	3.54	1.39	5.83	22.8	7.3
3	3.54	1.39	5.83	22.8	7.3
2	2.79	1.10	4.61	18.0	5.8
1	1.40	0.55	2.31	9.0	2.9
Total =	**23.5**		**38.7**	**151**	**48.4**

From this information, the RD ratio for the K_0-Stiffness Method can be calculated.
Therefore,

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T^i_{aldesign}}{D_m} = \frac{48.4 \text{ kN/m}}{24.85 \text{ kN/m}} = 1.95$$

Step 11: Connection strength limit state design. Using equations 9 and 10, calculate the long-term connection strength available at each load level. An example calculation at layer 5 is as follows:

$CR_{cr} = (12.3 + \sigma_N W_u \text{Tan } 13^o)/(51.4RF_{CR})$

$\sigma_N = = 18.9 \text{ kN/m}^3 \text{ x } 3.4 \text{ m} = 64.3 \text{ kPa}$, $W_u = 0.6$ m, and $RF_{CR} = 1.85$

Therefore, CR_{cr} = 0.22 (normalized to T_{lot})

See Step 12 for the remaining connection strength calculation results.

Step 12: Determine the T_{ult} and $T_{aldesign}$ needed to ensure that the factored connection strength is greater than T_{maxf}. T_{ult} is determined as follows:

$$T_{ult} = \frac{T_{max}\gamma_{EH}RF_D}{CR_{cr}\varphi_{cr}} \tag{27}$$

$$T_{ult} = \frac{3.54 \times 1.9 \times 1.3}{0.223 \times 0.80} = 49.0\, kN/m \text{ for Layer 5.}$$

T_{max}, CR_{cr}, T_{ult}, and $T_{aldesign}$ for all of the layers are summarized as follows:

Layer No.	**Unfactored Load T_{max} (kN/m)**	**CR_{cr}**	**Ultimate Strength Required T_{ult} (kN/m)**	**Long-Term Strength $T_{aldesign}$ (kN/m)**
8	1.94	0.13	45.5	14.6
7	3.18	0.17	56.6	18.1
6	3.54	0.20	54.3	17.4
5	3.54	0.22	49.0	15.7
4	3.54	0.24	45.6	14.6
3	3.54	0.26	42.6	13.6
2	2.79	0.27	31.6	10.1
1	1.40	0.29	15.0	4.8
Total =	**23.5**		**340**	**109**

From this information, the RD ratio necessary for the K_0-Stiffness Method to meet connection strength needs can be calculated.

Therefore,

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T^i_{aldesign}}{D_m} = \frac{109\text{ kN/m}}{24.85\text{ kN/m}} = 4.38$$

A comparison to the results from Step 10 reveals that connection strength controls the amount of reinforcement needed. T_{ult} from the table of calculation results above shows that layers 4 through 8 will require a slightly stronger product than used in the actual wall, which had a T_{ult} of

39.2 kN/m. This tensile strength requirement is consistent with the modulus used for determination of T_{max} (i.e., products that have a tensile strength in this range typically have a stiffness that is consistent with the value used to determine T_{max}). Note that because long-term data were not available for the connection strength, the short-term connection strength was reduced by RF_{CR} to obtain the long-term connection strength, an approach that is likely conservative given recent long-term testing experience by the authors. Furthermore, because the tensile strength of the required reinforcement resulting from this design is similar to or weaker than the strength of the product actually used to develop the short-term connection strength equation in Step 11, the connection equation used in Step 11 should be adequately accurate, if not conservative for the weaker products (see connection strength discussion for the AASHTO Simplified Method calculations for this example).

Step 13: Check the pullout capacity of the reinforcement to determine whether the current configuration of reinforcement spacing is adequate. The pullout length for layer 8 is determined as follows:

$$\mathrm{L_e} \geq \frac{\gamma_{\mathrm{EH}} \mathrm{T_{max}}}{\varphi_{po}\, \mathrm{F}^* \alpha\, \sigma_{\mathrm{v}}\, \mathrm{C\, R_c}} = \frac{1.65 \times 1.94}{0.7 \times 0.67 \times Tan 43^o \times 0.8 \times 42.84 \times 2 \times 1.0} = 0.11\, m$$

The minimum pullout length allowable is 0.9 m per AASHTO (1999). The distance from the back of the wall face to the active zone boundary for this layer is:

$$L_a = (H - z)Tan(45 - \phi/2) = (6.1 - 0.8)Tan(45 - 43/2) = 2.3\, m$$

L = 2.3 + 0.9 = 3.2 m

0.7H = 4.3 m, which is greater than 3.2 m

The length of the reinforcement needed at the other reinforcement layer locations is less than this. Therefore, pullout does not control the amount of reinforcement needed.

Steps 14 through 17: Serviceability limit state design. Calculate the strain at the EOC as follows:

At layer 5, for end-of-construction, T_{max} = 3.54 kN/m, and J_{EOC} = 420 kN/m. Therefore, ε_{EOC} = 0.84%. At layer 5, for long-term conditions at 10,000 hours after construction, T_{max} = 3.54 kN/m, and long-term J_{LT} = 380 kN/m, determined from extrapolation of isochronous creep data. Therefore, the total long-term strain, ε_{LT} = 0.93 percent. Post-construction strain = 0.93 - 0.84 = 0.09 percent. The calculated strains for all of the layers are summarized below:

Layer No.	Design T_{max} for Layer (kN/m)	$J_{2\%}$ at EOC (kN/m)	EOC Reinforcement Strain (%)	$J_{2\%}$ at 10,000 hrs (kN/m)	10,000 hr Total Reinforcement Strain (%)	Post-Constr. Reinforcement Strain to 10,000 hrs (%)
8	1.94	420	0.46	378	0.51	0.05
7	3.18	420	0.76	378	0.84	0.08
6	3.54	420	0.84	378	0.94	0.09
5	3.54	420	0.84	378	0.94	0.09
4	3.54	420	0.84	378	0.94	0.09
3	3.54	420	0.84	378	0.94	0.09
2	2.79	420	0.67	378	0.74	0.07
1	1.40	420	0.33	378	0.37	0.04

With figures 5 and 6, the wall face deformation can be estimated on the basis of the calculated strains. At the end-of-construction (EOC), the average strain in the wall is approximately 0.7%. The EOC maximum wall face deformation X is therefore:

$$X = 5.7(H+S)\varepsilon_{ave}^{\ 0.38} = 5.7(6.1+1.3)0.7^{0.38} = 37\ mm$$

The average post-construction strain for the first 10,000 hours is approximately 0.07 percent. The 10,000-hour post-construction, maximum wall face deformation X is therefore:

$$X = 528\varepsilon_{ave}^{\ 0.17} = 28(0.07)^{0.17} = 18\ mm$$

All these strains and deformations are within acceptable tolerances.

The final design for reinforcement when the measured plane strain ϕ of 43^{o} is used is summarized as follows: largest T_{ult} required = 56.6 kN/m and $T_{aldesign}$ required = 18.1 kN/m, using J_{EOC} = 420 kN/m. (Note: a lower design reinforcement stiffness could be allowed because the calculated strain was significantly less than the target strain.) If the reinforcement stiffness is reduced to 200 kN/m, factored strains are still at 2.5 percent or less, and the largest T_{ult} within the wall is at 47.4 kN/m ($T_{aldesign}$ is at 15.2 kN/m), resulting in $RD_{aldesign}$ = 3.66. The table below provides a summary of the calculations conducted for all the layers with a reinforcement stiffness of 200 kN/m:

Layer No.	T_{max} (kN/m)	ε_{reinf} (%)	T_{ult} not Considering Connection (kN/m)	T_{ult} Considering Connection (kN/m)	$T_{aldesign}$ (kN/m)
8	1.63	1.34	10.5	38.1	12.2
7	2.66	2.20	17.2	47.4	15.2
6	2.96	2.44	19.1	45.5	14.5
5	2.96	2.44	19.1	41.0	13.1
4	2.96	2.44	19.1	38.1	12.2
3	2.96	2.44	19.1	35.7	11.4
2	2.34	1.93	15.1	26.5	8.5
1	1.17	0.97	7.6	12.5	4.0

However, the larger reinforcement stiffness of 420 kN/m is a more likely value for the products that would be needed for this wall to meet connection strength requirements. Therefore, the values of T_{ult} or $T_{aldesign}$ specified in Step 12 and a minimum reinforcement stiffness at 1,000 hours of 200 kN/m would be used to select reinforcement materials for the wall.

If a more typical design ϕ value of 34^{o} is used rather than the measured ϕ, then the stiffness value must be increased to 600 kN/m, the maximum T_{ult} becomes 81.6 kN/m, and the maximum $T_{aldesign}$ becomes 26.1 kN/m, resulting in $RD_{aldesign}$ = 6.31. Using the AASHTO Simplified Method, for a design ϕ value of 34^{o}, the maximum $RD_{aldesign}$ = 75.7 kN/m, and the maximum T_{ult} required is 200 kN/m.

———————————END OF EXAMPLE 1———————————

<u>Steel Reinforced Soil Wall (Example 2)</u>

First, the RD ratio for the actual wall is calculated. The current AASHTO approach using upper bound specification values for corrosion rates is used to calculate available long-term reinforcement strength for comparison purposes. The focus of this comparison is on the long-term rupture strength. The available reinforcement capacity, T_i, for the actual reinforcement used in the case history is

$$T_i = \frac{A_c F_u}{S_h} \tag{27}$$

where A_c is the steel cross-sectional area of the reinforcement element reduced for corrosion losses, and F_u is the ultimate strength of the steel reinforcement. The RD ratio can be calculated with Equation 23.

Example 2 (Case Study SS11 – Actual Wall).

For the actual wall, calculate the estimated long-term resistance-demand ratio for Case Study SS11 (Figure 14) with surcharge in place. Material properties and reinforcement geometry for the actual wall are as follows:

T_{ult} = 142 kN/m (steel strip, based on the ultimate tensile strength of the steel F_u of 520 MPa, before corrosion)
F_y = 450 MPa (yield stress for steel)
Strip size = 50 mm x 4 mm
A_s = 200 mm^2 (strip area before corrosion)
A_c = 129.2 mm^2 (strip area after corrosion)
S_h = 0.73 m (horizontal strip spacing)
R_c = 0.0694 (reinforcement coverage ratio)
γ = 20.4 kN/m^3 (soil unit weight)
Measured triaxial ϕ = 40^o, estimated plane strain ϕ = 43^o
Using the measured triaxial ϕ, design K_a = 0.22
At the connection with the facing, the bolt hole diameter used is 14.3 mm.
D_m = 143.7 kN/m

The estimated long-term resistance-demand ratio for the actual wall case history is calculated with equations 17 and 21:

$$T_i = \frac{A_c F_u}{S_h} = \frac{0.000129 \text{ m}^2 \times 520{,}000 kPa}{0.73 \text{ m}} = 91.9\, kN/m$$

Therefore,

$$RD_{estimated} = \frac{R}{D} = \frac{\sum_{i=1}^{n} T_i}{D_m} = \frac{8 \times 91.9\, kN/m}{143.7\, kN/m} = 5.12$$

Example 2, Continued (Case Study SS11 – Typical AASHTO Simplified Method Design – Long-Term).

Using the AASHTO Simplified Method, calculate the estimated long-term resistance-demand ratio for Case Study SS11 (Figure 14) using the triaxial soil friction angle of 40°. Use of the triaxial soil friction angle is consistent with how the Simplified Method was calibrated (Allen, et al. (2001). A complete, step-by-step design is not provided for the Simplified Method in this example; only enough detail is shown to determine the RD ratio to prevent reinforcement rupture in the backfill.

The *allowable* long-term design resistance-demand ratio calculated with the AASHTO method can be expressed as follows:

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T^i_{aldesign}}{D_m} = \frac{\sum_{i=1}^{n} \frac{(F_u A_c)}{S_h}}{D_m} \tag{28}$$

where $T_{aldesign}$ is the long-term ultimate strength of the reinforcement, F_u is the ultimate strength for steel, A_c is the area of steel after corrosion, and S_h is the horizontal spacing of the reinforcement elements within a layer. The area and spacing of steel reinforcements must be great enough at each layer that $T_{max}/0.55$, where 0.55 is a safety factor relative to yield, is greater than the long-term yield strength available at each layer, $F_y A_c / S_h$. At the connection with the facing, a reduction factor to account for uncertainties of 0.5 relative to F_u, applied to the net sectional area of the steel, is used. The strength and spacing of the reinforcement in the backfill and at the connection were evaluated to determine $T_{aldesign}$.. They accounted for the reduced section resulting from the presence of the bolt hole, and the amount of reinforcement needed for pullout, keeping pullout from controlling the reinforcement length required. The reinforcement strength and spacing, and $T_{aldesign}$ to address all possible failure modes for each layer, are summarized as follows:

Z (m)	γ (kN/m³)	K_r/K_a	Strip Width (mm)	Strip Thickness (mm)	S_v (m)	$T_{aldesign}$ (kN/m)
0.38	20.4	0.36	50	2	1.05	14.5
1.14	20.4	0.35	50	3	1.05	39.2
1.9	20.4	0.34	50	3	1.05	39.2
2.66	20.4	0.32	50	3	1.05	47.1
3.42	20.4	0.31	50	3	1.05	47.1
4.18	20.4	0.29	50	4	1.05	64.0
4.94	20.4	0.28	50	4	1.05	64.0
5.70	20.4	0.27	50	4	1.05	64.0
Total						**379**

Therefore, RD for the Simplified Method is determined as follows:

$$RD = \frac{R}{D} = \frac{\sum_{i=1}^{n} T_i}{D_m} = \frac{379\, kN/m}{143.7\, kN/m} = 2.64$$

Example 2, Continued (Case Study SS11 – K_0-Stiffness Method Design – Long-Term).

Calculate the estimated long-term resistance to demand ratio for Case Study SS11 (Figure 12) using the K_0-Stiffness Method. Using a plane strain $\phi = 43^o$, which is consistent with how the K_0-Stiffness Method was calibrated, design for the strength limit state.

Step 1: Start with the reinforcement stiffness and spacing from the actual case history.

Steps 2 through 5: Check the strength limit state for the backfill soil by designing the steel so that the factored load is less than the factored yield strength of the steel. Use a resistance factor of 0.85 and a load factor of 1.5. The factored yield strength is determined as follows:

$$T_{\max}\Upsilon_{EH} \leq \frac{A_s F_y}{S_h}\varphi_{sf} \leq \frac{(200\text{ mm}^2)(450\text{ MPa})}{(0.76\text{ m})(1000)}(0.85) = 101\text{ kN/m}$$

Calculate T_{max} for each layer using the K_0-Stiffness Method as follows:

At the top layer, use Figure 1(b) to determine D_{tmax}, select Φ_{fs} = 1 because the wall uses articulating panels, a coefficient "a" equal to 0 for steel, and all other variables as defined previously.

$$T_{\max} = 0.5 S_v K_0 \gamma \; (H + S) D_{t\,\max} \left(\frac{S_{local}}{S_{global}} \right)^a \Phi_{fb} \Phi_{fs} 0.27 \left(\frac{S_{global}}{p_a} \right)^{0.24}$$

$$\mathrm{T_{max}} = 0.5(0.76)(0.32)(20.4)(6.1 + 0)(0.271) \left(\frac{69{,}252}{69{,}025} \right)^0 (1.0)(1.0)(0.27) \left(\frac{69{,}025}{101} \right)^{0.24}$$

$$\mathrm{T_{max}} = 5.3 \text{ kN/m}$$

For the rest of the layers, the determination of T_{max} is summarized as follows:

S_v (m)	K_o	D_{tmax}	Φ_{local}	Φ_{fb}	Φ_{fs}	Φ_g	T_{max} (kN/m)
0.76	0.32	0.27	1.0	1.0	1.0	1.29	5.3
0.76	0.32	0.41	1.0	1.0	1.0	1.29	8.0
0.76	0.32	0.56	1.0	1.0	1.0	1.29	10.8
0.76	0.32	0.70	1.0	1.0	1.0	1.29	13.6
0.76	0.32	0.84	1.0	1.0	1.0	1.29	16.3
0.76	0.32	0.98	1.0	1.0	1.0	1.29	19.1
0.76	0.32	1.00	1.0	1.0	1.0	1.29	19.4
0.57	0.32	0.86	1.0	1.0	1.0	1.29	12.6

The factored load is determined as follows:

$$T_{\max} \mathrm{Y}_{EH} \leq 101 \text{ kN/m}$$
$$(5.27)(1.5) \leq 101 \text{ kN/m}$$
$$7.9 \leq 101 \text{ kN/m? Yes. Steel stress is well below yield.}$$

These calculations are summarized for the rest of the layers in the table in Step 11. Although the steel area could be reduced substantially at this point, before doing so, go on to the next strength limit state to make sure wall reinforcement is adequate to prevent rupture.

Steps 6 and 7: Use T_{max} from Step 5 to evaluate the potential for reinforcement rupture. Calculate the strength of the steel reinforcement at the end of its service life, using the resistance factor φ_{rr} (use 0.90) to obtain the factored long-term reinforcement tensile strength, as shown below:

$$T_{al} = \frac{F_u A_c}{S_h}\varphi_{rr} = \frac{(520\ \mathrm{MPa})(129.2\ \mathrm{mm}^2)}{(0.76\ \mathrm{m})(1{,}000)}(0.90) = 79.6\ \mathrm{kN/m}$$

Steps 8 and 9: Calculate the factored load for the layer.

$$T_{\max f} = (T_{\max})\mathrm{Y}_{EH} = (5.27)(1.5) = 7.9\ \mathrm{kN/m}$$

$7.9 \leq 79.6$? Yes.

The factored reinforcement load $\mathrm{Y}_{EH}T_{max}$, the factored yield strength $A_sF_y\varphi_{sf}/S_h$, the factored long-term ultimate strength T_{al}, and the unfactored long-term ultimate strength for the needed reinforcement configuration $T_{aldesign}$ for all of the layers are summarized in Step 11.

Step 10: Check to make sure that the strength available at the connection to the wall face is adequate. The steel strip cross-sectional area must be reduced to account for the presence of a bolt hole, typically 14.3 mm in diameter. The reduced steel cross-sectional area usually controls the connection strength available, though shear through the bolt and pullout of the steel tab within the panel could be checked (for the purposes of this example, only the reduced strip cross-sectional area is checked). The factored long-term strength of the connection is as follows for the top reinforcement layer:

$$T_{al} = \frac{F_u A_c}{S_h}\varphi_{rr} = \frac{(520\ \mathrm{MPa})(92.2\ \mathrm{mm}^2)}{(0.76\ \mathrm{m})(1{,}000)}(0.90) = 56.8\ \mathrm{kN/m}$$

Compare T_{al} to the factored load for the top layer,

$7.9 \leq 56.8$? Yes.

The results of this calculation for the rest of the layers are summarized in Step 11.

Step 11: Determine whether the amount of reinforcement available is adequate to keep pullout needs from controlling the reinforcement length. Determine F* from Figure 2 for ribbed steel strips. Use a resistance factor of 1.3 for the top 2 m of wall and 1.0 below a depth of 2 m. For the top layer, the length of reinforcement required for pullout is as follows:

$$L_e \geq \frac{\gamma_{EH} T_{max}}{\varphi_{po} \; F^* \alpha \, \sigma_v \; C \, R_c} = \frac{1.5 \times 5.3}{1.3 \times 1.93 \times 1.0 \times 7.8 \times 2 \times 0.066} = 3.1 \, m$$

The distance from the back of the wall face to the active zone boundary per AASHTO (1999) for this layer is as follows (this equation for L_a only applies in the upper one-half of the wall – see AASHTO 1999 for details):

L_a = 0.3 x H_2 = 0.3 x 6.1 m = 1.83 m

See AASHTO (1999) for an equation for H_2. When no soil surcharge is present, H_2 = H, which is the case in this example.

L = 1.83 + 3.1 = 4.9 m
0.7H = 4.3 m, which is less than 4.9 m.

To accommodate this pullout requirement, either the upper layer(s) must be allowed to be longer, which will increase the reinforced soil volume, affecting wall costs, or the reinforcement coverage ratio, R_c, can be increased by decreasing the reinforcement spacing or increasing the strip width (if the manufacturing capability to specify a different strip geometry exists). Since total reinforcement required is being used as a measure to compare these calculations, the reinforcement geometry will be adjusted to address the pullout needs. If S_h is reduced in this top layer to 0.61 m, resulting in a reinforcement coverage ratio of 0.082, L_e becomes 2.5 m and L becomes 4.3 m, which is equal to 70 percent of the wall height. Because the global wall stiffness will increase somewhat because of the additional steel, recalculation results in the following:

Layer No.	Strip Width b (mm)	Strip Thick-ness t (mm)	S_h (m)	Factored Load $T_{max}\gamma_{EH}$ (kN/m)	Factored Yield Strength $A_sF_y\phi_{sf}/S_h$ (kN/m)	Factored Reinforce-ment Resistance T_{al} (kN/m)	Factored Connection Resistance T_{al} (kN/m)	L (m)
8	50	4	0.61	8.0	125	99.1	70.8	4.3
7	50	4	0.76	12.1	101	79.6	56.8	3.5
6	50	4	0.76	16.3	101	79.6	56.8	3.3
5	50	4	0.76	20.5	101	79.6	56.8	3.8
4	50	4	0.76	24.7	101	79.6	56.8	3.6
3	50	4	0.76	28.9	101	79.6	56.8	3.3
2	50	4	0.76	29.4	101	79.6	56.8	2.8
1	50	4	0.76	19.0	101	79.6	56.8	1.6
Total =				**158.8**	**830**	**656**	**468**	

Note that all of the $T_{max}\gamma_{EH}$ values for the top layer, as well as the rest of the layers, have increased slightly relative to what was calculated for the top layer in the previous steps. A comparison of $T_{max}\gamma_{EH}$ to $A_sF_y\varphi_{sf}/S_h$ for the soil failure limit state, and $T_{max}\gamma_{EH}$ to T_{al} for the reinforcement rupture limit state, indicates that the wall reinforcement is over-designed for these four limit states, with the exception of pullout for the top layer. Therefore, reduce the amount of steel to the configuration shown in the table below, revising the calculations. The revised calculation summary for Wall SS11 using the K_0-Stiffness Method is therefore as follows:

Layer No.	Strip Width b (mm)	Strip Thick-ness t (mm)	S_h (m)	Factored Load $T_{max}\gamma_{EH}$ (kN/m)	Factored Yield Strength $A_sF_y\varphi_{sf}/S_h$ (kN/m)	Factored Reinforce-ment Resistance T_{al} (kN/m)	Factored Connection Resistance T_{al} (kN/m)	L (m)
8	60	2	0.76	6.7	60.4	21.6	16.4	4.0
7	40	3	1.05	10.3	43.7	28.2	18.1	4.3
6	40	3	1.05	13.8	43.7	28.2	18.1	4.0
5	50	3	1.05	17.3	54.6	35.3	25.2	4.1
4	50	3	1.05	20.8	54.6	35.3	25.2	4.0
3	50	3	1.05	24.4	54.6	35.3	25.2	3.7
2	50	3	1.05	24.8	54.6	35.3	25.2	3.2
1	40	3	1.05	16.0	43.7	28.2	18.1	2.3
Total =				**134**	**410**	**248**	**172**	

Pullout controlls the amount of steel for layers 5, 7, and 8. For all other layers the resistance at the connection of the strips to the wall facing panels controls the amount of steel needed. Note that the ability to vary reinforcement width "b" and thickness "t" depend on local manufacturing capabilities for steel strips. The strip dimensions used are for illustration purposes only.

From this information, the final RD ratio for the K_0-Stiffness Method can be calculated. Therefore, removing the resistance factor from T_{al} in the table above,

$$T_{aldesign} = T_{al} / \varphi_{rr} = 248/0.9 = 276 \text{ kN/m}$$

$$RD_{aldesign} = \frac{\sum_{i=1}^{n} T_{aldesign}^{i}}{D_m} = \frac{276 \text{ kN/m}}{143.7 \text{ kN/m}} = 1.91$$

END OF EXAMPLE 2

Summary of Additional Examples

To provide additional testing of the K_0-Stiffness Method relative to current design practice, additional case histories were evaluated with both methods, and results were compared to the actual case history designs. The additional case histories evaluated include a concrete panel faced geogrid wall (GW8); a tall, wrap-faced geosynthetic wall (Wall GW16); a tall, steel strip reinforced soil wall; a relatively short, lightly reinforced, full-scale laboratory, incremental panel-faced geogrid wall (GW15) taken to failure; a relatively tall, steel strip reinforced concrete, panel faced wall (SS13); a lightly reinforced bar mat wall (BM3); and a very tall welded wire wall (WW1). Details of these case histories are provided by Allen and Bathurst (2001) and Allen, et al. (2001). Figures 15 through 19 provide a cross-section of each wall to illustrate the wall geometry. Key design properties for these case histories are summarized in tables 8 and 9. The resulting RD ratios (per equations 26 and 28) for all the case histories are summarized in Table 10. Those ratios are analyzed in Table 11 to indicate the reduction in the amount of reinforcement possible by using the K_0-Stiffness Method relative to the AASHTO Simplified Method.

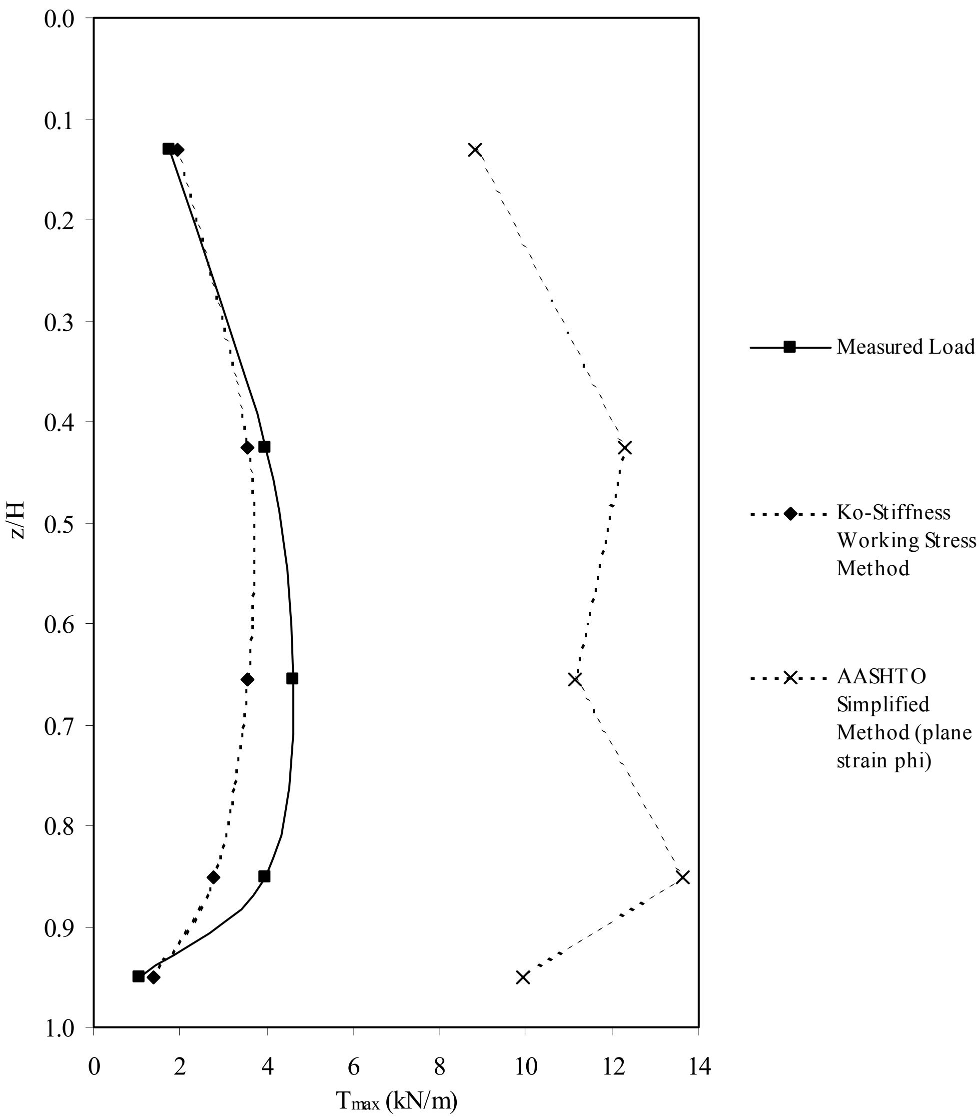


Figure 20. Predicted and measured loads for geogrid wall GW9, with soil surcharge.

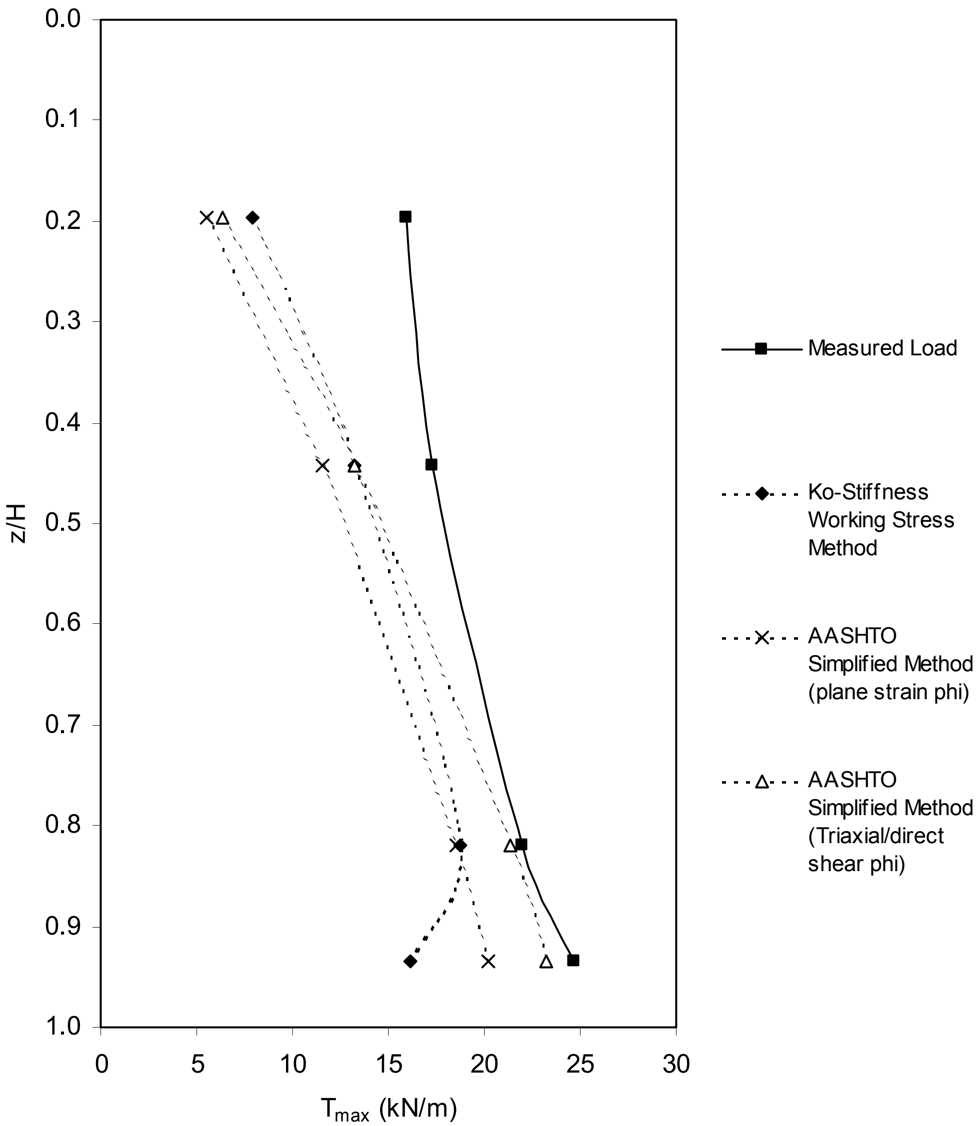


Figure 21. Predicted and measured reinforcement loads for steel strip wall SS11.

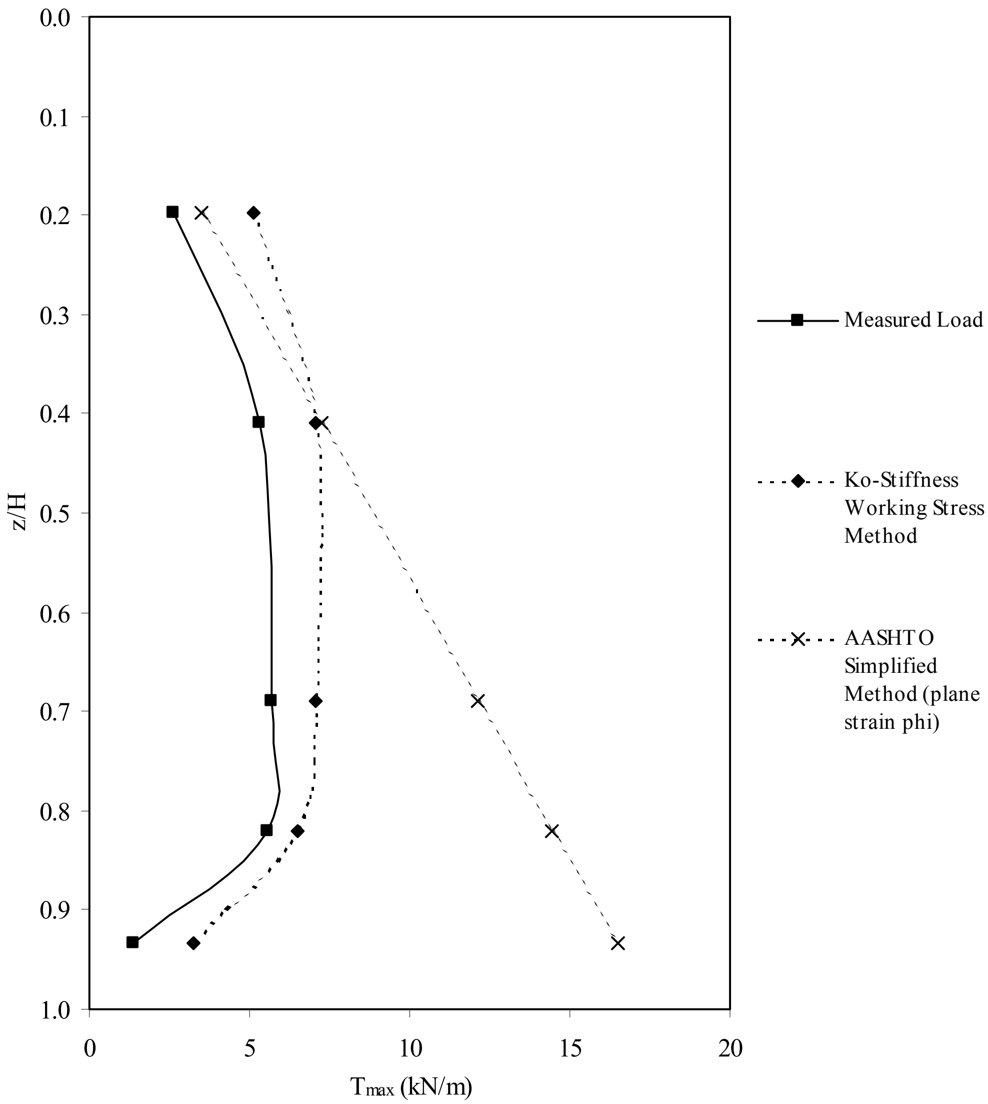


Figure 22. Predicted and measured loads for geogrid wall GW8, with soil surcharge.

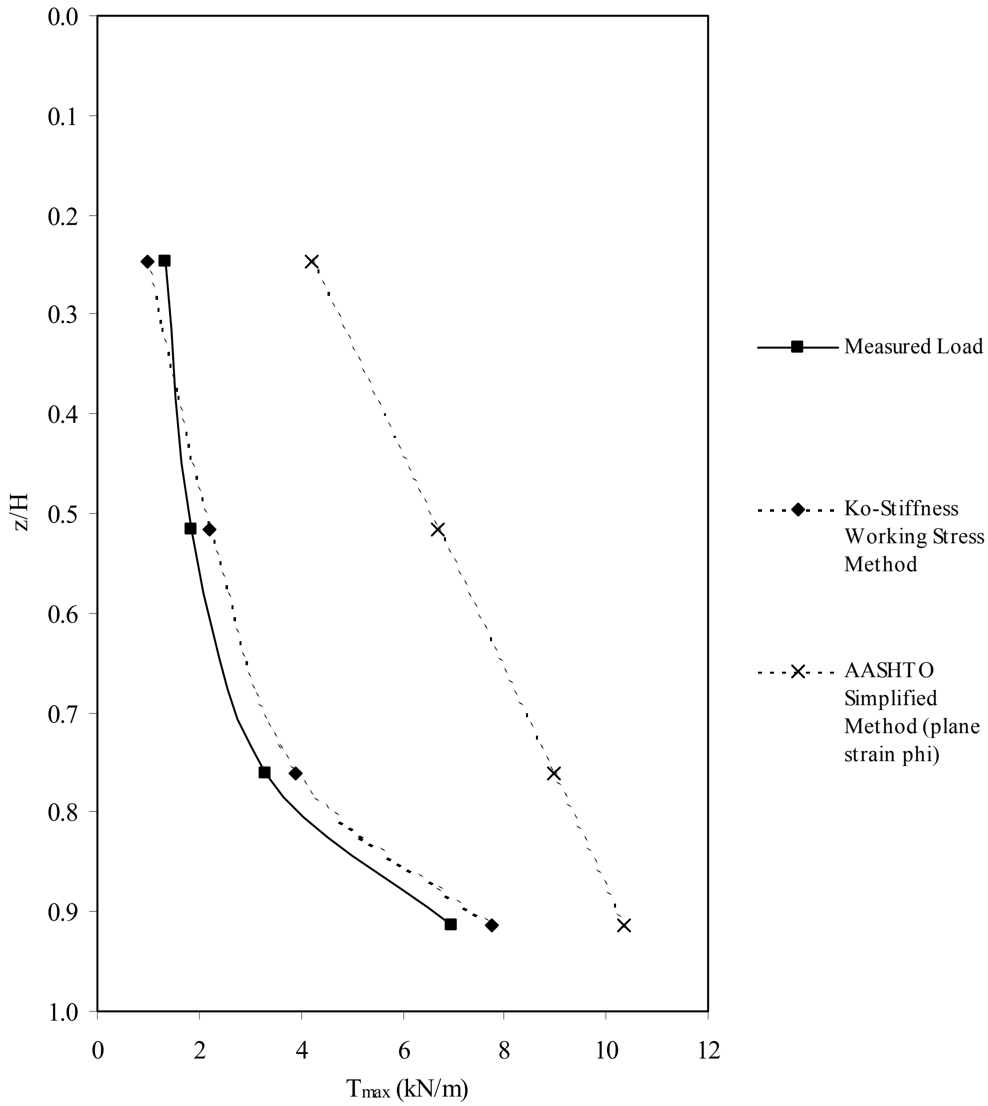


Figure 23. Predicted and measured loads for geotextile wall GW16, with soil surcharge.

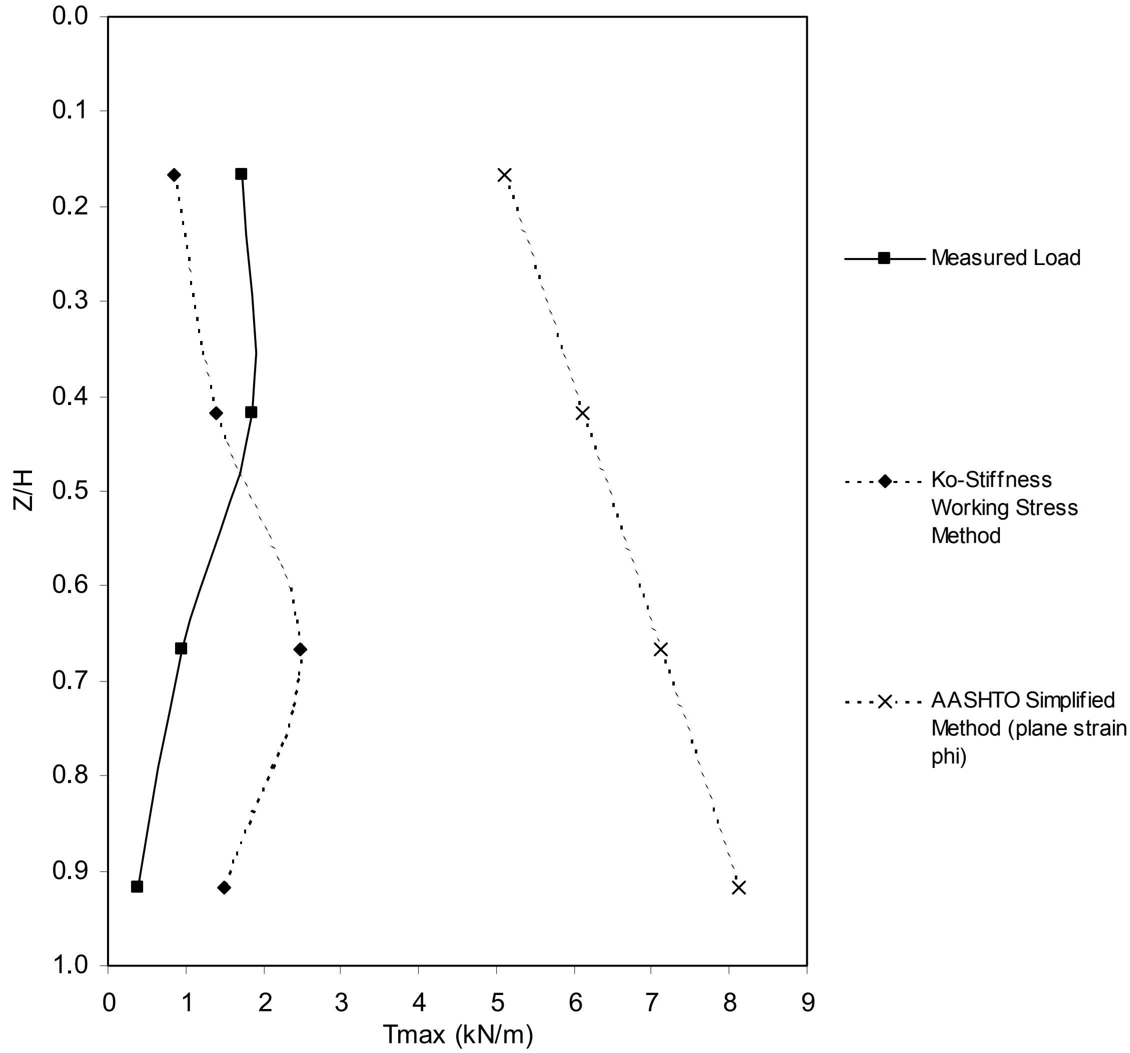


Figure 24. Predicted and measured loads for full-scale geogrid incremental aluminum panel test wall GW15, at 70 kPa surcharge.

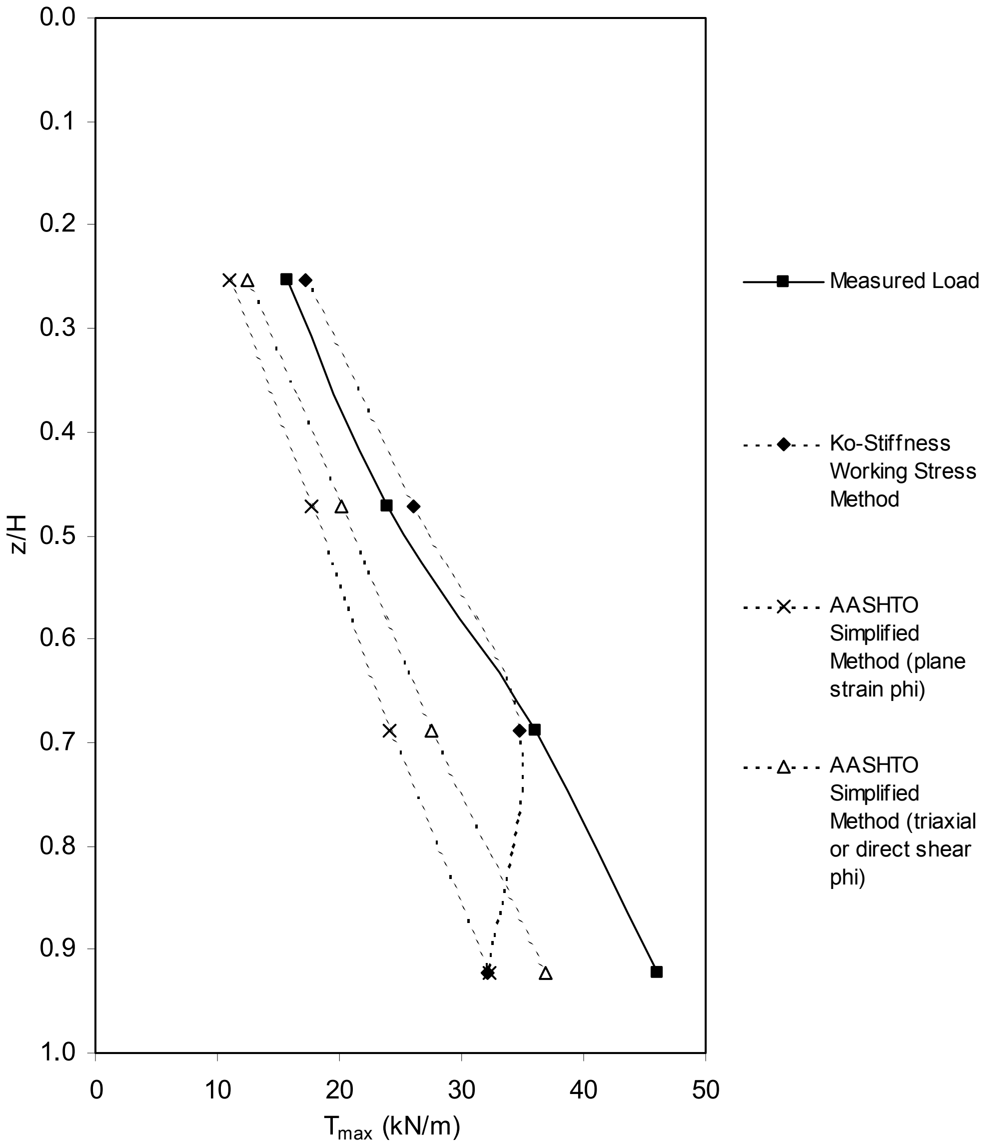


Figure 25. Predicted and measured loads for steel strip wall SS13.

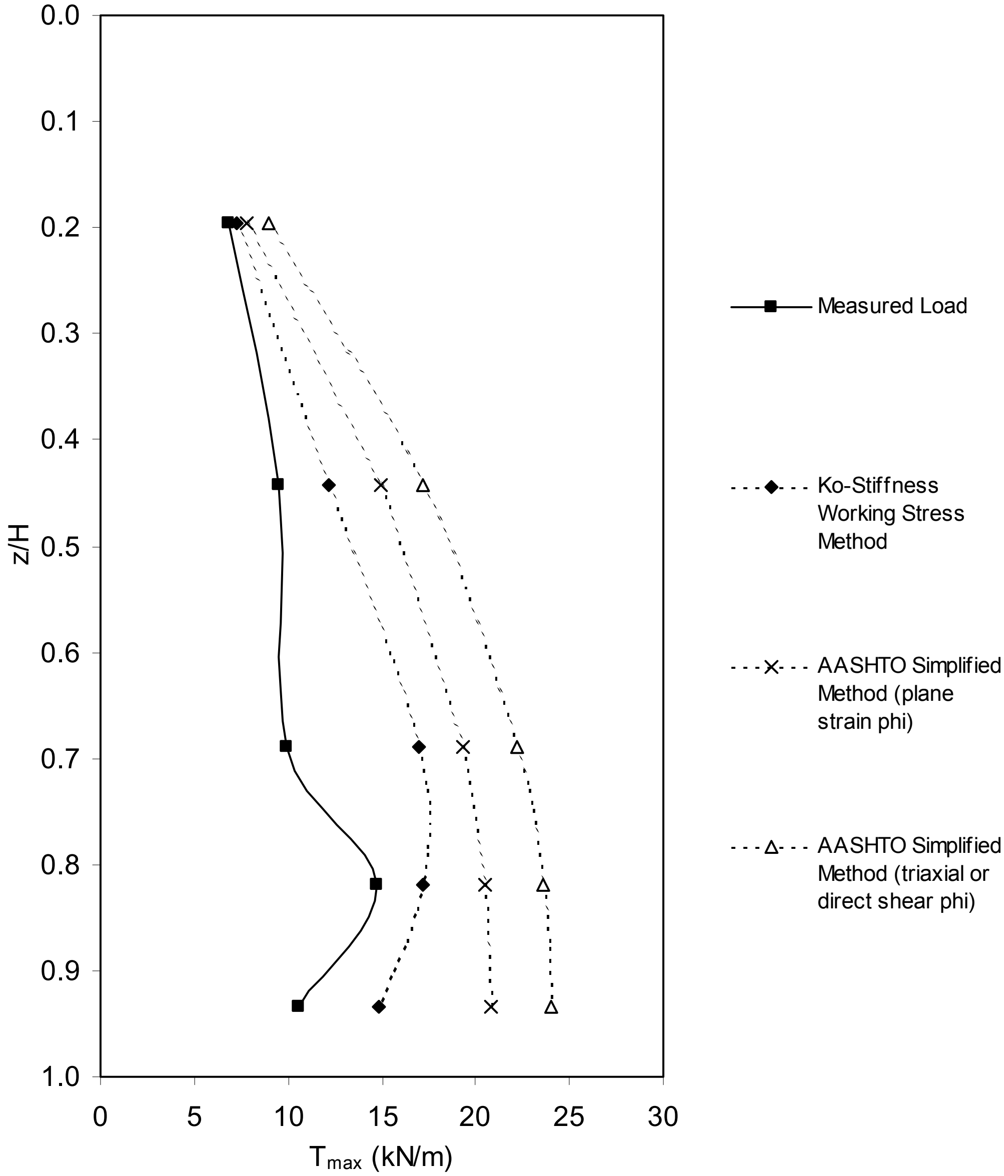


Figure 26. Predicted and measured loads for steel bar mat wall BM3.

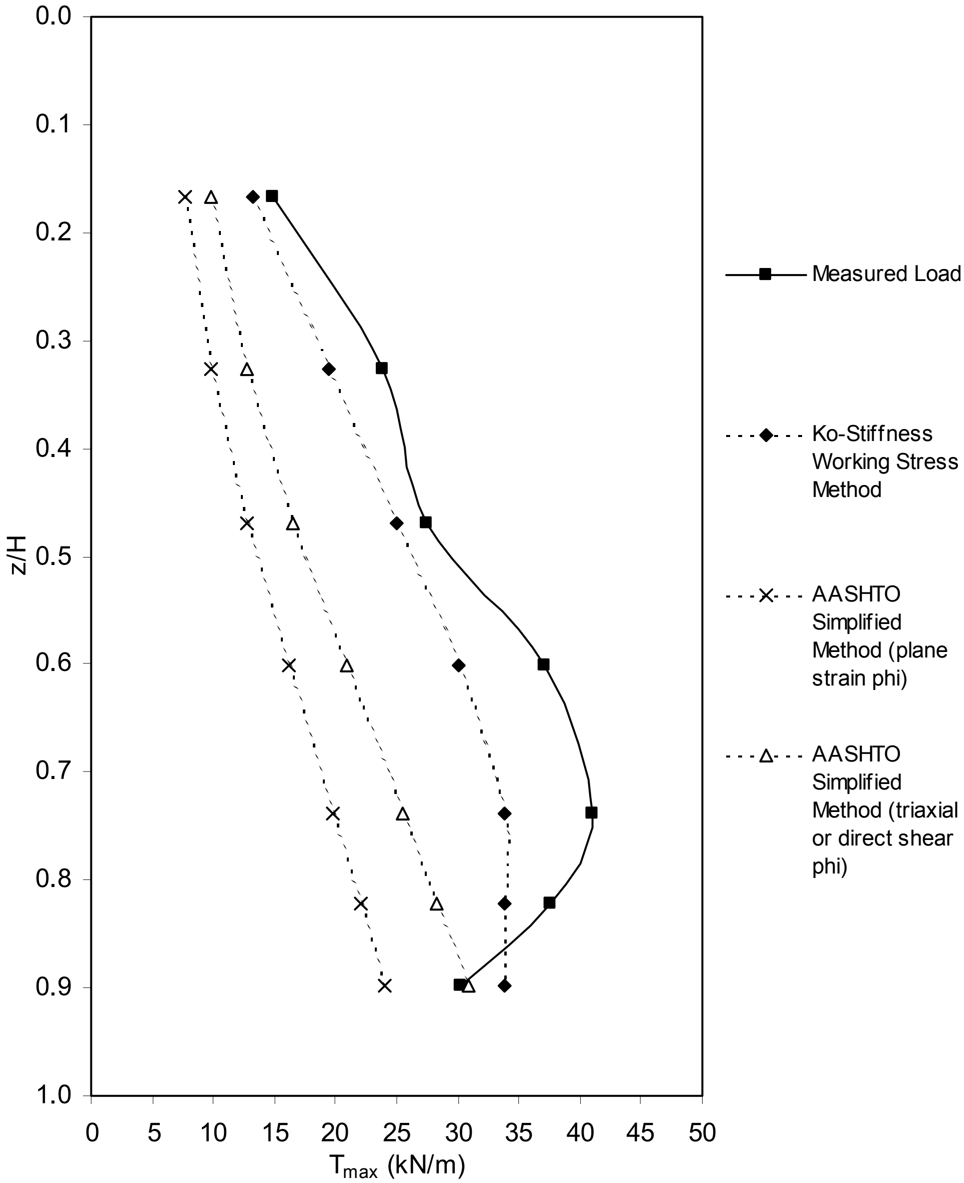


Figure 27. Predicted and measured loads for welded wire wall WW1.

Table 10. Summary of long-term resistance to demand ratios calculated for each design example.

Wall	**For Actual Wall, Using Measured Load, $RD_{estimated}$**	**AASHTO T_{max} Total at Measured (triaxial) ϕ_{tx} (kN/m)**	**For AASHTO Design at Typical Design ϕ of 34° $RD_{aldesign}$**	**For AASHTO Design at Measured (triaxial) ϕ_{tx}, $RD_{aldesign}$**	**For K_0-Stiffness T_{max} Total at Measured (plane strain) ϕ_{ps} (kN/m)**	**For K_0-Stiffness Design at Design ϕ of 34° RD_{stiff}**	**For K_0-Stiffness Design, at Measured (plane strain) ϕ_{ps}, RD_{stiff}**	**Controlling Limit State for K_0-Stiffness Method Design**
GW8	5.25	109	9.50	7.31	56.0	7.68	5.17	Connection Strength
GW9	4.03	103	18.8 (8.24 not considering connection strength)	12.0 (6.23 not considering connection strength)	23.5	6.31	4.38	Connection Strength
GW15 with 70 kPa surcharge	1.79	33.0	20.5	9.6	8.1	11.1	2.56 at actual $J_{2\%}$ (65 kN/m); 3.25 to prevent soil failure ($J_{2\%}$ of 176 kN/m)	Soil Failure (predicted factored strain = 5.3% at actual $J_{2\%}$ of 65 kN/m, measured strain = 4 to 4.5%)
GW16	9.14	374	11.7	6.87	129	9.97	3.25	Reinforcement rupture or soil failure at measured ϕ, soil failure at design ϕ
SS11	5.12	106	3.64	2.64	89.4	3.21	1.91	Pullout, and rupture at connection
SS11 with surcharge (e.g., Fig. 15)	-	155	5.00	3.51	109	3.19	2.01	Rupture at connection
SS13	6.10	286	2.96	2.60	275	2.90	2.05	Pullout and rupture
BM3	7.68	127	6.99	5.45	84.8	7.05	3.82	Pullout and rupture
BM3 with surcharge (e.g., Fig. 15)	-	190	8.89	6.91	100	4.76	3.34	Reinforcement rupture,
WW1	4.39	780	2.91	2.38	765	2.54	1.85	Reinforcement rupture

Table 11. Overall reduction in reinforcement required by the K_0-Stiffness Method relative to what is required by the AASHTO Simplified Method.

Wall Type and Height	**Reinforcement Type**	**Average Surcharge Height (m)**	***AASHTO RD_{al}/ K_0-Stiffness RD_{al}**	**Controlling Limit State**
Segmental concrete Block (6.1 m)	Geogrid	1.3	$^{+}$2.7 to 3.0	Connection rupture
Precast panel (6.1 m)	Geogrid	0	1.2 to 1.4	Connection rupture
Precast panel (3 m)	Geogrid	3.3	1.8 to 3.0	Reinforcement rupture
Wrap Face (12.6 m)	Geotextile	2.65	1.2 to 2.1	Soil failure
Precast panel (6.1 m)	Steel strip	0	1.1 to 1.4	Pullout
Precast panel (6.1 m)	Steel strip	1.3	1.6 to 1.7	Connection rupture
Precast panel (6.1 m)	Bar mat	0	1.0 to 1.4	Pullout
Precast panel (6.1 m)	Bar mat	1.3	1.9 to 2.1	Reinforcement rupture
Precast panel (10.5 m)	Steel strip	0	1.0 to 1.3	Connection rupture
Welded wire (16.8 m)	Welded wire	0.3	1.1 to 1.3	Reinforcement rupture

*Calculated and compared at design $\phi = 34^o$, and at measured ϕ. $^{+}$If connection strength is not considered in the AASHTO calculation, but is considered in the K_0-Stiffness Method calculation, the ratio is 1.3 to 1.4.

ANALYSIS OF EXAMPLES AND IMPLICATIONS FOR DESIGN

The geosynthetic wall examples demonstrated that substantial reductions in the amount of reinforcement required by current design practice are possible when the K_0-Stiffness Method is used (see tables 10 and 11). This difference should be anticipated given the ability of these design methods to predict measured reinforcement loads, as shown for geosynthetic walls in figures 20, 21, 23, and 24. The amount of reinforcement reduction relative to what is required by the AASHTO Simplified Method varied, assuming measured shear strength parameters were used and that the reinforcement strength was perfectly matched to the demand at each layer. Reductions in the amount of reinforcement required ranged from 1.2 to 3. The amount of reinforcement reduction was greatest for stiff faced walls and relatively low height walls with high backfill soil friction angles.

Example 1 (Wall GW9) and Wall GW8 can be compared to demonstrate the effect of a very stiff facing on reinforcement loads and needs. The reduction in reinforcement needed relative to what would be required by the AASHTO Simplified Method was approximately 2.8 for wall GW9, which had a very stiff facing, but only 1.3 for Wall GW8, which had a more flexible facing system. The reduction in reinforcement requirements for Wall GW15, a full-scale wall built in a laboratory environment, was similar to that for Wall GW9, even though wall GW15 had a relatively flexible facing. This may be the result of needing only very lightweight, flexible reinforcement because of the small height of the wall. However, note that the use of such lightweight reinforcement may not be feasible in field walls because of the potential for significant reinforcement damage, and heavier reinforcement may be required in practice.

Another observation that can be made from these geosynthetic wall design examples, as summarized in tables 10 and 11, is that as the design soil friction angle becomes more conservative relative to the measured soil friction angle, the required reinforcement stiffness becomes higher. As the stiffness increases, the calculated reinforcement loads from the K_0-Stiffness Method increase, reducing the difference between amount of reinforcement required by the AASHTO Simplified Method and the K_0-Stiffness Method . Therefore, the use of conservative soil parameters causes the K_0-Stiffness Method to become more conservative more quickly than is the case for the Simplified Method. For geosynthetic walls, it is best to estimate the anticipated peak plane strain soil friction angle as accurately as possible to take full advantage of the K_0-Stiffness Method.

The segmental concrete block faced wall design example demonstrated the effect of connection strength on the amount of reinforcement needed. Note that using short-term connection strength data and reducing them by RF_{CR} to estimate the creep reduced connection strength is likely to be conservative relative to the current protocol referenced by the AASHTO (1999, in press) specifications, which encourage long-term connection tests to be conducted. However, when this conservative determination of connection strength is considered in the wall design, the K_0-Stiffness Method still requires significantly less reinforcement than the AASHTO Simplified Method not considering connection strength.

In general, the controlling strength limit state for the geosynthetic walls was connection rupture for segmental concrete block wall systems (and possibly for the other stiff-faced systems) and reinforcement rupture in the backfill or backfill soil failure for the other systems. Backfill soil failure became more of an issue as the wall became higher and the soil became weaker. That is, the modulus required to keep strains below the target strain to prevent soil failure became high relative to the reinforcement tensile strength required.

One issue that became clear when these example designs were calculated is that knowledge of the typical relationship between the reinforcement modulus and the ultimate tensile strength for the range of products available is very helpful when modulus values are chosen to estimate the reinforcement loads. For the proprietary wall supplier, such knowledge is readily available. For the consultant or government agency engineer, more familiarity with the range of modulus and ultimate tensile strength combinations available may be necessary. As noted in the step-by-step design procedures, the modulus needed for design is an end-of-construction modulus. The time to be used to estimate this EOC modulus can vary with the height of the wall. However, in most cases, the modulus at 1,000 hours will be sufficiently accurate for design purposes. Geosynthetic reinforcement suppliers should be prepared to provide these longer term modulus data if this methodology is fully implemented.

, The potential reduction in the amount of reinforcement required for the steel reinforced examples was not as great as it was for geosynthetic walls in the majority of cases. For the taller walls, the reduction in the amount of reinforcement required was relatively small. In general for both steel and geosynthetic reinforced walls, as the wall became taller or as the design soil friction angle used became lower, the reinforcement stiffness increased, and the amount of reinforcement required by the K_0-Stiffness Method increased relative to the AASHTO Simplified Method.

For steel reinforced walls with discontinuous reinforcement and with little or no soil surcharge above the wall top, pullout requirements strongly controlled the amount of reinforcement required, at least in the upper part of the wall. Since the focus of these comparisons between the Simplified Method and the K_0-Stiffness Method was the amount of reinforcement required, to simplify the comparison regarding the pullout issue, the reinforcement length was forced to be at 70 percent of the wall height, the minimum width required in general by AASHTO (1999, in press). This caused the amount of reinforcement required to meet pullout needs to increase, if pullout controlled the design. The reinforcement length in the upper portion of the walls could have been increased instead by 0.3 to 0.9 m to meet these pullout requirements, allowing the amount of reinforcement required to be significantly less. Though it is possible that a small increase in the reinforcement length in the upper portion of the wall may be more cost effective than increasing the amount of reinforcement and keeping the reinforcement length at 70 percent of the wall height, for the purposes of these examples, the latter approach was used. Note that increasing the amount of reinforcement to address pullout requirements increases the global wall stiffness and therefore increases the reinforcement loads, further exacerbating the pullout problem. However, in spite of this, the AASHTO Simplified Method is slightly more conservative than the K_0-Stiffness Method when little or no soil surcharge is present (i.e., overburden pressure is low).

For those examples in which pullout strongly controlled the amount of steel reinforcement needed (Walls SS11 and BM3), a second example design was performed with the soil surcharge shown in Figure 13 for Wall GW8. As can be observed from Table 10, pullout no longer controlled the amount of reinforcement needed for the K_0-Stiffness Method, but did tend to control (at least marginally and only at some reinforcement locations) the amount of reinforcement needed in the AASHTO Simplified Method. Because pullout was no longer controlling the design, the amount of required steel reinforcement decreased significantly in relation to what the AASHTO Simplified Method would require. In this case, the amount of reinforcement required could be reduced by a factor of 1.6 to 2.1 when the K_0-Stiffness method was used to perform the design.

These analyses were conducted to demonstrate the amount of reinforcement reduction relative to current practice possible through the use of the proposed design methodology. However, especially for the shorter walls, unusually thin or lightweight reinforcing materials would be needed to properly match the demand, even when all limit states are considered. For geosynthetic reinforcement, significant susceptibility to installation damage must be considered, and it may be more practical to select a geosynthetic with greater strength than required than to select a geosynthetic that would be too susceptible to installation damage. For steel reinforcement, concerns about the applicability of the corrosion model to small diameter bars or wires, or very thin strips, as well as possible limitations in current manufacturing capabilities, may limit minimum reinforcement sizes. New steel reinforced soil wall systems may need to be developed to better utilize the potential advantages of the K_0-Stiffness methodology.

Although the K_0-Stiffness Method produces less conservative designs for geosynthetic walls, this is accomplished by allowing the geosynthetic to exhibit relatively large strains to take full advantage of the strength of the soil. Although these larger strains are designed to be within the margins of safety for all of the geosynthetic wall components (i.e., soil, reinforcement, and facing), these larger strains may not be acceptable from a serviceability standpoint, depending on the specific application. While the K_0-Stiffness Method generally provides less reduction in the reinforcement required for steel reinforced systems, depending on the specific wall geometry and soil properties used, it does reveal the much lower strains that steel reinforced wall systems produce, demonstrating the applicability of steel reinforced soil walls in applications where low strain may be desirable (e.g., bridge abutments or walls that support deformation intolerant structures).

The Simplified Method provides a large amount of built in, hidden safety in the design, especially for geosynthetic walls. Because of this, walls designed with the Simplified Method can accommodate poor construction technique or material quality and still perform well. The examples mentioned previously showed that the Simplified Method has a built-in factor of safety of 2 to 3 for geosynthetic walls, even if measured soil strengths are used. The K_0-Stiffness Method provides a much more accurate estimate of reinforcement loads, and its use can result in substantial cost savings. However, the K_0-Stiffness Method provides less margin of safety to accommodate poor construction technique or materials control than does the AASHTO Simplified Method. The load and resistance factors recommended herein are intended to accommodate some variation in construction quality, but not wide variations. Therefore, it is incumbent on the user of the K_0-Stiffness Method to ensure that a reasonable degree of wall construction quality control is used. If for some reason construction quality cannot be properly controlled, then the user of the K_0-Stiffness Method should increase load factor values or decrease the resistance factor values used in the design to account for that uncertainty.

It is also important to not extrapolate the K_0-Stiffness Method to design situations that are significantly beyond the geometric conditions and material properties within the database of case histories that were used to develop the method. For example, this method was based on case histories that used only granular backfills with relatively low silt content. Therefore, this method should not be used for silt or clay backfills at this time. Additional research should be conducted to develop modifications to the method, if needed, to fully address such conditions.

The K_0-Stiffness Method can be used to justify the use of less reinforcement than has been required by past and current design practice. The temptation will be to increase reinforcement spacing to reduce the number of reinforcement layers required, maximizing the reduction of wall construction cost. Current North American design practice is to limit the vertical spacing of reinforcements to a maximum of 0.8 m (AASHTO, 1999). On the basis of the analyses performed by Allen and Bathurst (2001), it appears that a reasonably accurate prediction of the reinforcement load can be obtained with a larger vertical spacing of reinforcement, on the order of 1.0 m or more. A maximum reinforcement vertical spacing of 1.0 m is recommended with the K_0-Stiffness Method as a reasonable limit to ensure that the wall behaves as predicted and that the empirical and theoretical bases for the method are not violated. However, this large vertical spacing limit only applies to stiff facing systems, and not, for example, to flexible facings such as geosynthetic wrap or welded wire panels. For flexible facings, smaller vertical spacing of the reinforcement is needed to control facing deformations and stresses. Furthermore, very wide reinforcement spacing should not be combined with relatively low wall heights, as it is important that enough layers be present to cause the reinforced soil mass to behave as a coherent unit. A minimum of 2 to 4 layers, depending on the wall height, is recommended if this design methodology is used.

The K_0-Stiffness Method demonstrates the effect that reinforcement stiffness has on the reinforcement load levels within the wall, in that as stiffness increases, the reinforcement load level increases. The empirical evidence strongly supports this conclusion. It is recognized that the reinforcement strength and stiffness required to prevent soil failure and reinforcement rupture in the backfill may not control the amount of reinforcement required for internal stability, depending on the wall system and the specifics of the design. Reinforcement loads must be calculated each time significant changes are made to the reinforcement design to accommodate the needs of other limit states, as increasing the amount of reinforcement increases the reinforcement loads. As shown by the design examples provided previously, an increase in the global stiffness by a factor of 2.5 can result in an increase in the reinforcement loads of approximately 30 percent. Because of this, some iteration in the calculations to complete a wall design may be needed. If a wall design is specified generically (this tends to be the case for geosynthetic wall designs), not only must a minimum reinforcement modulus and strength be specified, but a maximum allowable reinforcement modulus may also be needed to make sure that reinforcement stresses are not significantly higher than considered for the design. In that case, the minimum reinforcement modulus should be specified to ensure that serviceability requirements are met and to prevent soil failure, and the minimum reinforcement tensile strength, connection strength, and pullout length should be based on the load required for the maximum reinforcement EOC modulus considered acceptable for the wall.

A SIMPLIFIED PROCEDURE USING THE K_0-STIFFNESS METHOD

The AASHTO (1999) Simplified Method utilizes a series of design curves to modify the earth pressure coefficient and account for the effect that various reinforcement types have on the magnitude of T_{max}. A similar approach can be taken for the K_0-Stiffness Method so that it will have a similar "look and feel," require no or very little iteration, and be more easily implemented. This simplified K_0-Stiffness approach for calculating T_{max} is as follows:

$$T_{\max} = 0.5 S_v K_0 \gamma \, (H + S) \left(\frac{S_{local}}{S_{global}} \right)^a \left(\frac{K_{abh}}{K_{avh}} \right)^{0.5} \Phi_{fs} \Phi_{\mathrm{dj}} \tag{29}$$

where Φ_{dj} is a combined global stiffness - load distribution factor obtained from Figure 28, and all other variables are as defined previously. The detailed data used to generate the curves in Figure 28 are provided in Appendix B. Note that interpolation between curves provided in Figure 28 is not recommended. Furthermore, the actual global stiffness of the wall should be less than or equal to the global stiffness associated with the curve selected for design. This will cause this simplified approach to be somewhat more conservative than what would be calculated with the full K_0-Stiffness Method (Equation 1). If a more accurate determination of the reinforcement load is desired, Equation 1 should be used instead.

Typically, the 500 kPa curve will apply to small and moderate-sized geosynthetic walls. The 1,000 kPa curve typically applies to geosynthetic walls 9 to 12 m high, whereas the 2,000 kPa curve may apply to geosynthetic walls that are taller than 12 m. The 10,000 kPa curve typically applies to polymer strap walls. All curves with greater global stiffnesses apply to steel reinforced systems. The 50,000 and 100,000 kPa curves typically apply to moderate-sized, steel reinforced walls. Curves associated with greater global stiffnesses typically apply to relatively large steel reinforced walls. Figure 28 shows that for global wall stiffnesses greater than 100,000 kPa, the stiffness-distribution factor does not increase nearly as much as it does for lower stiffnesses.

The difference in the shape between the curves applicable to geosynthetic walls versus those applicable to steel reinforced walls is due to the difference in the D_{tmax} used (see Figure 1). The overlap in the curves for the highest geosynthetic wall global stiffness and the lowest steel reinforced wall global stiffnesses indicates that some type of transitional D_{tmax} distribution may be needed for intermediate global wall stiffness values. Continuing research will be used to refine these distributions, assessing the effect that global wall stiffness, and possibly the amount of reinforcement coverage (i.e., the coverage ratio, R_c) has on D_{tmax}, as suggested by Allen and Bathurst (2001).

The step-by-step procedures provided previously for the K_0-Stiffness Method still generally apply, except that no reiteration should be needed, provided the right design curve from Figure 28 has been selected. All limit states should still be checked. The load and resistance factors developed for the full K_0-Stiffness Method apply to this simplified approach as well.

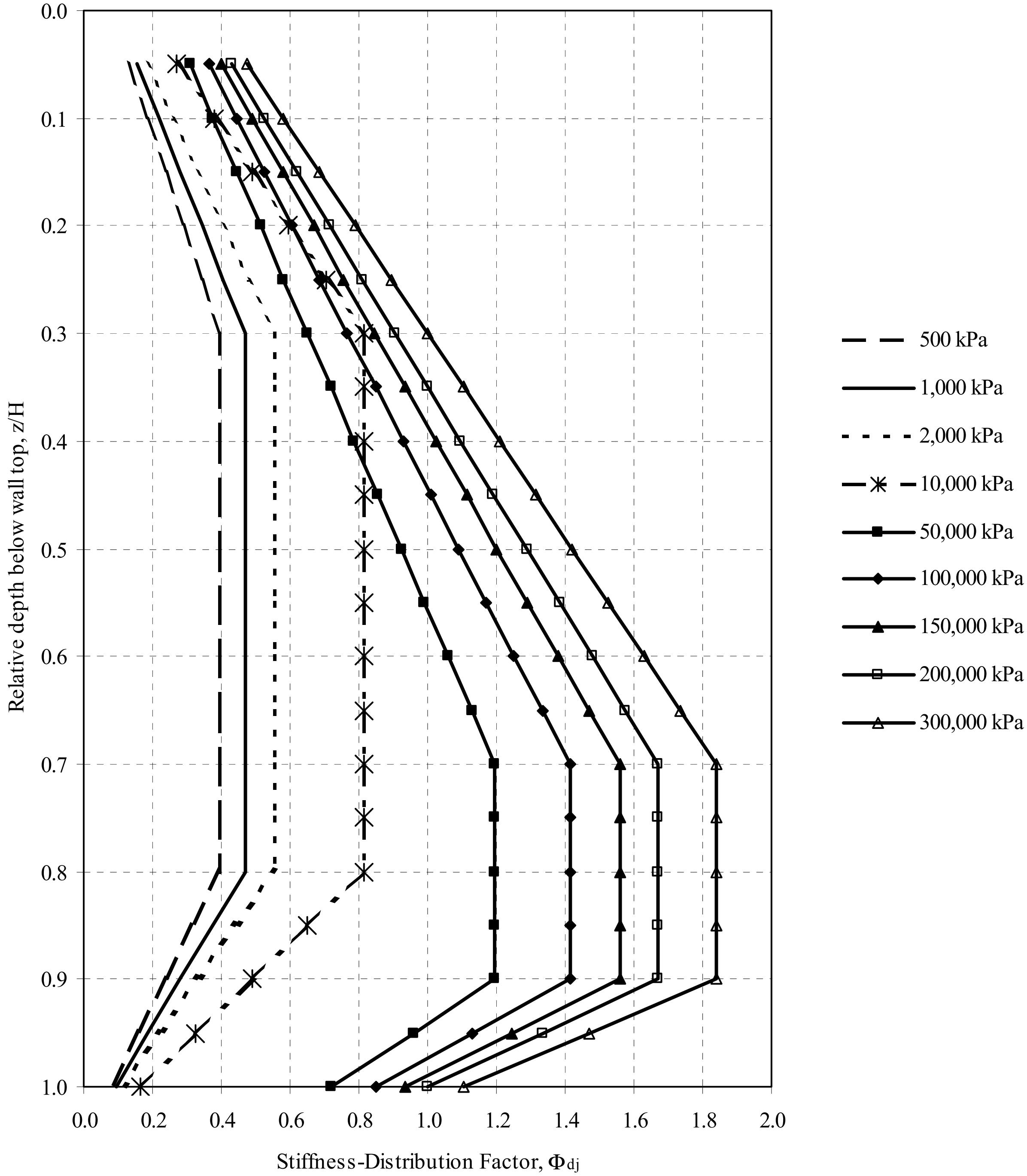


Figure 28. Stiffness-distribution factor for Simplified K_0-Stiffness Method.

CONCLUSIONS

Step-by-step procedures have been presented to demonstrate the application of the K_0-Stiffness Method to reinforced soil wall design. These procedures have been developed with a limit states approach, so that they can be more easily incorporated into current design code. Recommendations based on statistical data for load and resistance factors that account for material property and design model uncertainty have been provided to consistently produce a probability of failure of 1 percent. The K_0-Stiffness Method was developed and calibrated assuming that measured plane strain soil parameters would be used. Therefore, use of conservative lower bound shear strength values will add conservatism to the design and further decrease the probability of failure.

Simplified statistical techniques have been used to develop the load and resistance factors recommended for use with the K_0-Stiffness Method. A more rigorous statistical calibration to confirm the magnitude of these load and resistance factors has not yet been performed. Nevertheless, the load and resistance factors provided herein should provide reasonably conservative designs.

A comparison of backfill reinforcement designs developed with current design methodology (e.g., the Simplified Method) and the K_0-Stiffness Method demonstrates that the K_0-Stiffness Method may produce designs with significantly less (by a factor of 1.2 to 3) reinforcement for geosynthetic walls. These soil reinforcement reductions resulting from the use of the K_0-Stiffness Method should be considered the minimum possible. In practice, it is more typical to use conservative soil parameters for design and to not perfectly match the reinforcement to the demand. Therefore, greater reductions in the amount of reinforcement relative to current design practice are possible. For steel reinforced walls, reduction in reinforcement appears to be less than for geosynthetic walls (typically, the amount of reinforcement is reduced by a factor of 1.0 to 1.4 but can be as high as 2.1). However, the general approach and level of safety used will be consistent for all reinforced soil walls, regardless of the reinforcement type.

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NOMENCLATURE

a = a coefficient which is also a function of stiffness
AASHTO = American Association of State Highway and Transportation Officials
A_c = cross sectional area of the steel reinforcement unit after corrosion losses (m^2)
A_s = area of steel before corrosion (m^2)
b = width of reinforcement unit (mm)
C = reinforcement gross surface area geometry factor for pullout (dimensionless)
COV = coefficient of variation (%)
COV_R = coefficient of variation for the resistance (%)
COV_{QEH} = coefficient of variation for the reinforcement load due to dead load (%)
COV_{QL} = coefficient of variation for the reinforcement load due to live load (%)
CR_{cr} = long-term connection strength extrapolated to the desired design life normalized by the index tensile strength of the lot or roll of material used for the connection testing (dimensionless)
CR_u = the ultimate strength of the facing–geosynthetic connection determined from laboratory tests, normalized by the lot or roll specific index tensile strength of the geosynthetic
d = constant coefficient related to facing batter
D = total reinforcement demand within a wall (kN/m)
D_m = total reinforcement demand measured in the actual case history (kN/m)
D_{tmax} = reinforcement load distribution factor (dimensionless)
EOC = end of wall construction
F* = pullout friction factor
F_u = ultimate tensile strength of the steel (kPa)
F_y = yield stress for the steel reinforcement (kPa)
FS = design factor of safety
H = vertical wall height at the wall face (m)
HDPE = high density polyethylene
i = counter (1,2,3 …n)
J, J_i = modulus of an individual reinforcement layer (kN/m)
J_{ave} = average modulus of all the reinforcement layers within the entire wall section (kN/m)
J_{EOC} = reinforcement modulus at end of wall construction (kN/m)
J_{LT} = long term reinforcement modulus (kN/m)
K_a = coulomb active earth pressure coefficient (dimensionless)
K_{abh} = horizontal component of active earth pressure coefficient accounting for wall face batter (dimensionless)
K_{avh} = horizontal component of active earth pressure coefficient assuming the wall is vertical (dimensionless)
K_o = at-rest lateral earth pressure coefficient for the reinforced backfill (dimensionless)
L_e = length of reinforcement in the resisting zone (m)
LRFD - load and resistance factor design
MARV = minimum average roll value
n = number of reinforcement layers within the entire wall section
n_{EH} = number of data points
OCR = overconsolidation ratio
p_a = atmospheric pressure (a constant equal to 101 kPa)

P_f = probability of failure (%)
PET = polyester
PP = polypropylene
Q = load (kN/m)
Q_{EH} = reinforcement dead load (kN/m)
Q_L = reinforcement live load (kN/m)
R = resistance in general, or total resistance of the backfill reinforcement (kN/m)
R_c = reinforcement coverage ratio (dimensionless)
RD = resistance to demand ratio(s)
$RD_{aldesign}$ = allowable long-term resistance-demand ratio (dimensionless)
$RD_{estimated}$ = estimated long-term resistance-demand ratio (dimensionless)
RF_{actual} = total strength reduction factor for installation damage, creep and durability estimated based on actual site conditions, and direct measurement from exhumed samples where feasible
RF_{CR} = reduction factor for strength loss due to creep
RF_D = strength reduction factor due to chemical and biological degradation
RF_{ID} = reduction factor for strength loss due to installation damage
R_n = nominal resistance available (kN/m)
S = average soil surcharge height above the wall top, or equivalent height of uniform surcharge pressure (m)
S_{global} = global wall reinforcement stiffness (kN/m^2)
S_h = horizontal spacing of the reinforcement (m)
S_{local} = local stiffness (kN/m^2)
S_v = tributary area (assumed equivalent to the average vertical spacing of the reinforcement at each layer location when analyses are carried out per unit length of wall) (m^2/m)
t = strip or reinforcement thickness (mm)
T_{ac} = long-term connection strength (kN/m)
T_{al} = long-term reinforcement design strength (kN/m)
$T_{aldesign}$ = the long-term tensile strength of the reinforcement, accounting for installation damage, creep and durability (kN/m)
$T^i_{aldesign}$ = long-term strength of the reinforcement in layer i (kN/m)
T_{crc} = long-term connection strength extrapolated to the desired design life (kN/m)
T_i = tensile resistance of each reinforcement layer (kN/m)
T_{lot} = index tensile strength of the lot or roll of material used for the connection testing (kN/m)
T_{max} = peak load in each reinforcement layer (kN/m)
T_{maxf} = factored load in each reinforcement layer (kN/m)
T_{mxmx} = maximum value of T_{mx} within the wall (kN/m)
T_{ult} = ultimate tensile strength of the reinforcement based on the minimum average roll value (kN/m)
T_{ultf} = factored ultimate tensile strength of the reinforcement (kN/m)
T^i_{ult} = short-term (index or ultimate) strength of the reinforcement for layer i (kN/m)
W_u = width of facing unit (m)
X = maximum wall face lateral deformation
z, z_p = depth below top of wall (m)
α = scale effect correction factor for pullout
β = reliability index

β_τ = = target reliability index (dimensionless)
ε_{ave} = average peak reinforcement post-construction strain in wall (%)
ε_{eoc} = strain in reinforcement layer at end-of-construction (%)
ε_{LT} = total long-term strain (%)
ε_{reinf} = factored strain in the reinforcement (%)
ε_{targ} = target reinforcement strain (%)
ε_{targf} = factored target reinforcement strain (%)
ε_p = soil peak shear strain (%)
ϕ = friction angle of the soil (degrees)
ϕ' = peak angle of internal soil friction for the wall backfill
φ_{cr} = resistance factor for connection rupture
φ_{EQr} = resistance factor for reinforcement rupture during seismic loading
φ_{EQp} = resistance factor for reinforcement pullout during seismic loading
φ_{po} = resistance factor for pullout
φ_{ps} = soil friction angle estimated at plane strain condition (degrees)
φ_{rr} = resistance factor for reinforcement rupture
φ_s = resistance factor for the service limit state
φ_{sf} = resistance factor for backfill soil failure
φ_{tp} = resistance factor for reinforcement pullout for temporary structures
φ_{trr} = resistance factor for reinforcement rupture for temporary structures
ϕ_{tx} = soil friction angle measured using triaxial or direct shear test (degrees)
Φ_g = global stiffness factor (dimensionless)
Φ_{fb} = facing batter factor (dimensionless)
Φ_{fs} = facing stiffness factor (dimensionless)
Φ_{local} = local stiffness factor (dimensionless)
γ = soil unit weight (kN/m^3)
γ_{con} = load factor for connection design (dimensionless)
γ_{EH} = soil reinforcement load factor
γ_i = load factor applicable to a specific load (dimensionless)
γ_u = unit weight of facing units (kN/m^3)
λ_R = bias factor for the resistance (dimensionless)
λ_{QEH} = bias factor for the reinforcement load due to dead load (dimensionless)
λ_{QL} = bias factor for the reinforcement load due to live load (dimensionless)
σ_N = normal vertical stress on the reinforcement-facing connection (kPa)
σ_v = vertical stress at the reinforcement layer in the resistant zone (kPa)
ζ_u = lognormal standard deviation

APPENDIX A

CALIBRATION DATA FOR AASHTO SIMPLIFIED METHOD

A calibration was performed for the AASHTO (1999) Simplified Method using the same resistance statistics (see Table 4) as for the calibration of the K_0-Stiffness Method. This was done so that a comparison could be made between the current load and resistance factors in the AASHTO specifications and the load and resistance factors that result from a calibration using the data gathered for this reinforced soil wall study. The load statistics for the Simplified Method are summarized in Table A1. These load statistics were developed based on the data provided in Figures A1 and A2. Simplified Method reinforcement load predictions used triaxial or direct shear soil shear strength parameters, which are consistent with the original development of the Simplified Method and the minimum requirements in the AASHTO specifications (AASHTO, 1999, in press; Allen, et al., 2001).

Figures A1 and A2 provide plots of the ratio of measured to predicted reinforcement load T_{max} as a function of normalized wall height z/H, to demonstrate the distribution of the data used to develop the statistical parameters. Due to the limitations of the Simplified Method, some of the walls included in the dataset used to evaluate the K_0-Stiffness Method had to be removed from the dataset used to evaluate the Simplified Method. Therefore, the heavily battered walls (technically steep reinforced slopes) designated as wall GW7 by Allen and Bathurst (2001) and the PET strap wall, designated as Wall GW19 were removed from the geosynthetic wall dataset to determine load and resistance factors for the Simplified Method, as the use of the Simplified Method to design these types of walls is currently discouraged in the AASHTO specifications. Allen, et al. (2001) also discourage the use of the Simplified Method in combination with triaxial or direct shear design soil friction angles greater than 40^o (plane strain friction angles of greater than 44^o) for steel reinforced walls due to the tendency to seriously underpredict reinforcement loads in that situation. Therefore, steel reinforced walls cases which had a high backfill soil friction angle were removed from the dataset used to determine the Simplified Method load and resistance factors for steel reinforced walls.

Using the reduced datasets described above, points that were more than two standard deviations beyond the mean of the ratios were additionally discarded from the datasets, to insure that a few outlier data points do not excessively bias the statistics, resulting in unreasonably conservative load and resistance factors. Furthermore, outlier points identified in this manner were also evaluated as to the likely cause of their unusually poor load prediction. The outlier point for the geosynthetic walls in Figure A1 is from the wall designated by Allen and Bathurst

(2001) as GW10. The point in question is located near the top of the wall. This wall was very lightly reinforced, and the wall backfill was exhibiting signs of failure, including the development of cracks in the backfill. This condition most strongly affected the top layer. The outlier points for the steel reinforced walls in Figure A2 were from the top layers in wall SS11 and BM5. In both cases, compaction stresses were likely to be unusually high, which may explain why the load prediction was poor in these two cases.

Table A1. Load statistical parameters used for resistance factor calibration (Simplified Method, all strength limit states).

Load Type	Parameter	Value
Geosynthetic Reinforcement Load*	Number of Data Points, n_{EH}	36
	Bias Factor for the Reinforcement Load, λ_{QEH}	0.27
	COV_{QEH}	0.548
Steel Reinforcement Load*	Number of Data Points, n_{EH}	55
	Bias Factor for the Reinforcement Load, λ_{QEH}	0.93
	COV_{QEH}	0.291
Live Load	Bias factor for the live load, λ_{QL}	1.15
	COV_{QL}	0.18
	Q_{EH}/Q_L	10
	γ_L	1.75

*Bias and COV are based on the Simplified Method using triaxial or direct shear strength parameters, per the AASHTO specifications.

The load factors for geosynthetic and steel reinforcement are calculated as follows:

$$\gamma_{EH} = \lambda_{QEH}\left(1 + 2\,\mathrm{COV_{QEH}}\right) \tag{A1}$$

Based on the statistical parameters provided in Table A1, for the geosynthetic reinforced walls, this results in a load factor of 0.60 for geosynthetic walls and 1.5 for steel reinforced walls. The resulting resistance factors for the Simplified Method are provided in Table A2. Figure A3 demonstrates the adequacy of the proposed load factors to produce a consistently conservative estimate of reinforcement load.

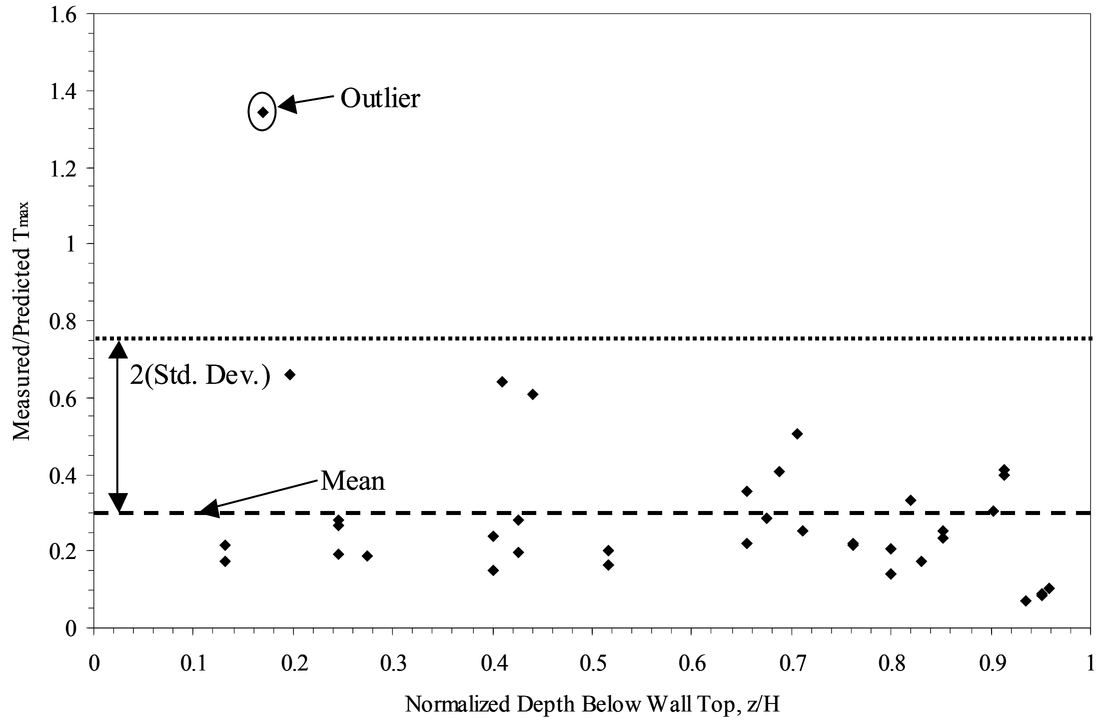

Figure A1. Ratio of measured to predicted T_{max} for the AASHTO Simplified Method versus normalized depth below wall top (geosynthetic walls only).

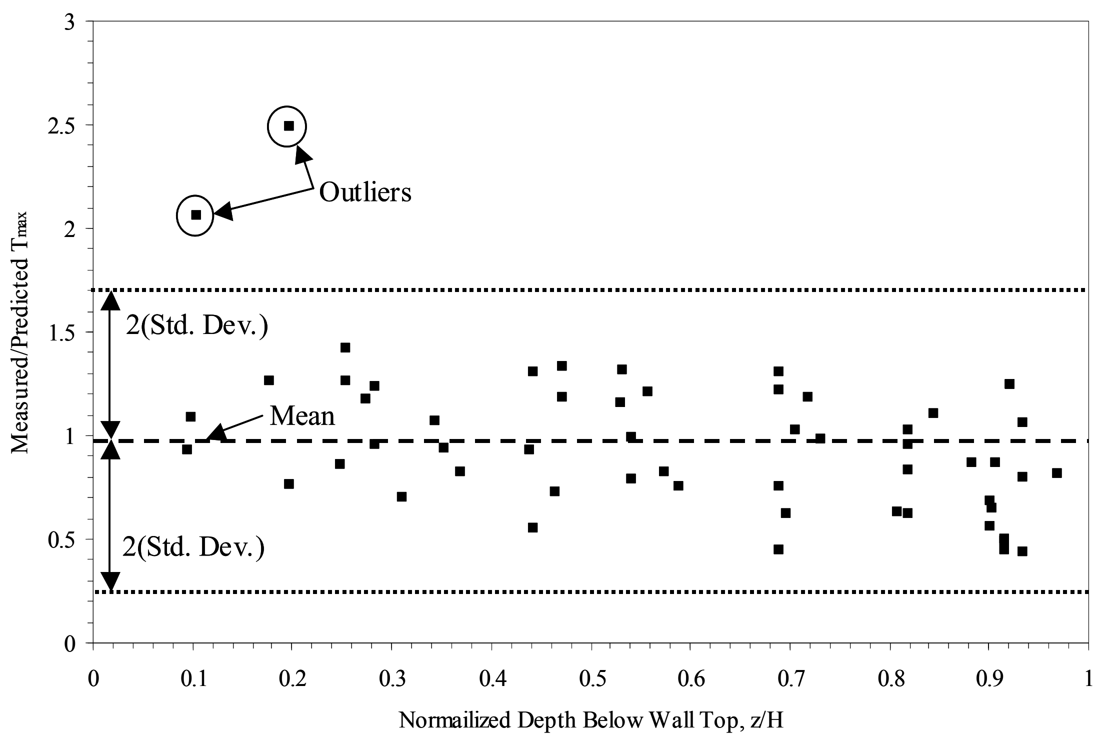

Figure A2. Ratio of measured to predicted T_{max} for the AASHTO Simplified Method versus normalized depth below wall top (steel reinforced walls only).

Table A2. Calculated AASHTO Simplified Method load and resistance factors for the strength limit state (reinforcement rupture and pullout).

Limit State	Load Factor, γ_{EH}	Reinforcement Type	Calculated Resistance factor for $B_\tau = 2.0$	Calculated Resistance factor for $B_\tau = 2.33$
Reinforcement Rupture	0.60	Woven geotextile	0.72	0.67
		HDPE geogrid	0.74	0.71
		PP geogrid	0.73	0.70
		PET geogrid	0.74	0.70
	1.5	Steel grid	0.94	0.85
		Steel strip	0.94	0.85
Reinforcement Pullout (using AASHTO default values	0.60	Geogrid (using 0.67Tan ϕ)	0.74	0.65
	1.5	Ribbed steel strip (at depth greater than 2 m below wall top)	1.15	1.0
		Ribbed steel strip (within 2 m of wall top only)	1.49	1.3
		Smooth steel strip	1.72	1.46
		Steel grid	0.76	0.66

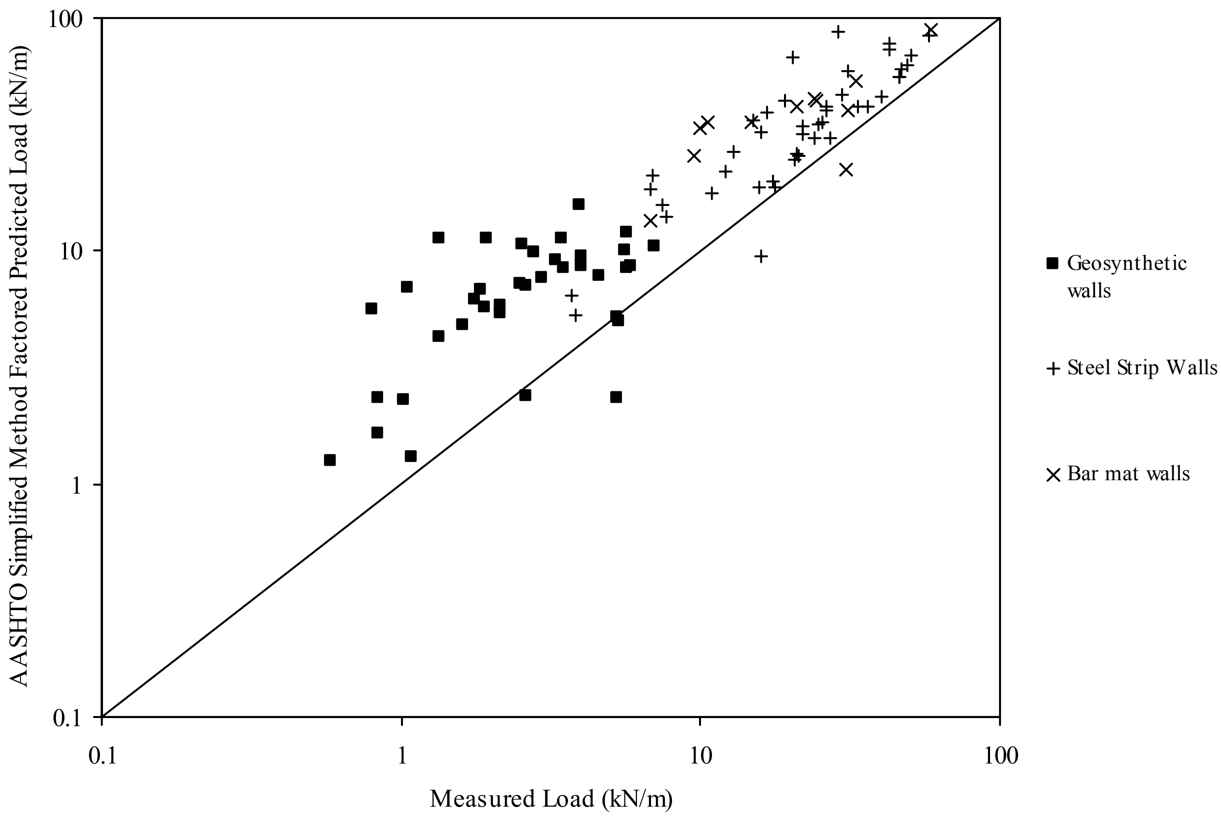

Figure A3. Measured vs. factored predicted load for the AASHTO Simplified Method.

Note that for the geosynthetic walls, most of the factored load prediction values plot well above the one to one correspondence line. This indicates that the scatter in the geosynthetic wall load predictions are too great to simply factor away the inherent excess conservatism in the Simplified Method with regard to geosynthetic wall reinforcement loads. The one geosynthetic wall data point, and the two steel reinforced wall data points, that are located well below the one to one correspondence line are the outliers identified earlier that were removed from the data used to determine the load factors for the Simplified Method.

APPENDIX B

DATA FOR SIMPLIFIED K_0-STIFFNESS METHOD

Table B1. Simplified K_0-Stiffness Method stiffness-distribution factor, Φ_{dj}, for various global wall stiffness values, S_{global} as a function of relative reinforcement depth below wall top.

Relative Depth below Wall Top z/H	**Stiffness-Distribution Factor, Φ_{dj}**								
	Upperbound S_{global} (kPa)								
	500	**1,000**	**2,000**	**10,000**	**50,000**	**100,000**	**150,000**	**200,000**	**300,000**
0.05	0.13	0.16	0.18	0.27	0.31	0.36	0.40	0.43	0.47
0.10	0.18	0.22	0.26	0.38	0.38	0.44	0.49	0.52	0.58
0.15	0.24	0.28	0.33	0.49	0.44	0.53	0.58	0.62	0.68
0.20	0.29	0.34	0.41	0.60	0.51	0.61	0.67	0.72	0.79
0.25	0.34	0.41	0.48	0.70	0.58	0.69	0.76	0.81	0.89
0.30	0.40	0.47	0.55	0.81	0.65	0.77	0.85	0.91	1.00
0.35	0.40	0.47	0.55	0.81	0.72	0.85	0.93	1.00	1.10
0.40	0.40	0.47	0.55	0.81	0.79	0.93	1.02	1.10	1.21
0.45	0.40	0.47	0.55	0.81	0.85	1.01	1.11	1.19	1.31
0.50	0.40	0.47	0.55	0.81	0.92	1.09	1.20	1.29	1.42
0.55	0.40	0.47	0.55	0.81	0.99	1.17	1.29	1.38	1.52
0.60	0.40	0.47	0.55	0.81	1.06	1.25	1.38	1.48	1.63
0.65	0.40	0.47	0.55	0.81	1.13	1.33	1.47	1.57	1.73
0.70	0.40	0.47	0.55	0.81	1.20	1.41	1.56	1.67	1.84
0.75	0.40	0.47	0.55	0.81	1.20	1.41	1.56	1.67	1.84
0.80	0.40	0.47	0.55	0.81	1.20	1.41	1.56	1.67	1.84
0.85	0.32	0.37	0.44	0.65	1.20	1.41	1.56	1.67	1.84
0.90	0.24	0.28	0.33	0.49	1.20	1.41	1.56	1.67	1.84
0.95	0.16	0.19	0.22	0.33	0.96	1.13	1.25	1.34	1.47
1.00	0.08	0.09	0.11	0.16	0.72	0.85	0.93	1.00	1.10