Review of NCMA Segmental Retaining Wall Design Manual for Geosynthetic-Reinforced Structures

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The National Concrete Masonry Association (NCMA) recently introduced a design manual for the analysis, design, and construction of geosynthetic-reinforced soil retaining walls that use dry-stacked masonry concrete units as the facing system. Important features of the manual are addressed, including methods of analysis, interpretation of long-term design strength of the geosynthetic reinforcement, selection of factors of safety, partial materials factors, and proposed test methods that address stability aspects of the facing system. Differences between current FHWA and AASHTO guidelines and the NCMA manual are discussed, and deficiencies in these earlier standards with respect to mortarless masonry wall systems are identified.

The use of dry-stacked columns of interlocking modular concrete units as the facing treatment for geosynthetic-reinforced soil retaining wall structures has increased dramatically in recent years (*1–3*). An example of a completed project is illustrated in Figure 1. The National Concrete Masonry Association (NCMA) recently adopted the term "soil-reinforced segmental retaining wall" to identify this type of retaining wall system (*4*,p.336). Reinforced segmental retaining wall systems offer advantages to the architect, engineer, and contractor. The walls are constructed with segmental retaining wall units (modular concrete block units) that have a wide range of aesthetically pleasing finishes and provide flexibility with respect to layout of curves, corners, and tiered wall construction. The base course of modular units is typically seated on a granular bearing pad, which offers cost advantages over conventional poured-in-place concrete walls and some types of reinforced concrete panel wall systems that routinely require a concrete bearing pad.

The mortarless modular concrete units are easily transportable and therefore facilitate construction in locations where access is difficult. The mortarless construction and typically small segmental retaining wall unit size and weight allows installation to proceed rapidly. An experienced installation crew of three or four persons typically can erect 20 to 40 m^2 of wall face a day. The economic benefit due to these features is that reinforced segmental retaining walls of more than 1 m in height typically offer a 25 to 40 percent cost savings over comparable conventional cast-in-place concrete retaining walls (*4*,p.336).

Conventional methods of geosynthetic-reinforced soil retaining wall design are available in publications prepared by FHWA (*5*,p.287) and AASHTO (*6*). These two documents adopt analysis and design methodologies for earth retaining structures that are based on concepts familiar to geotechnical engineers. For example, these earlier guidelines adopt limit equilibrium methods, conventional earth pressure theory, and factors of safety against a number of potential failure mechanisms and partial material factors applied to geosynthetic reinforcement properties.

The authors have prepared a design manual on behalf of the NCMA that addresses design and construction aspects of geosynthetic-reinforced segmental retaining wall structures. The NCMA manual adopts an overall approach that is similar to recommendations found in the FHWA and AASHTO guidelines but that extends and refines the methods of analysis and design to consider explicitly all performance aspects of dry-stacked segmental retaining wall units. Hence, an important feature of the NCMA manual is that it allows the designer to quantify performance differences between reinforced wall options built with different modular concrete unit types and in combination with different geosynthetic reinforcement materials. A generic step-by-step methodology is introduced in the manual to help the designer optimize the structure.

The NCMA manual also offers guidelines for the analysis, design, and construction of unreinforced (gravity) segmental retaining wall structures. However, this paper is restricted to a discussion of structures that include horizontal layers of extensible geosynthetic reinforcement to increase the mass of the composite retaining wall system and to stabilize the dry-stacked facing units.

Nevertheless, the NCMA manual recognizes that there are common performance features of both unreinforced and reinforced segmental retaining wall systems and provides the designer with a consistent and integrated approach to the analysis, design, and construction of both classes of structure. The approach adopted by the authors in preparing the manual has been to view reinforced segmental retaining wall systems as a modification to unreinforced systems that allows the safe construction of taller and more heavily surcharged segmental retaining walls.

This paper gives an overview of important aspects of the NCMA manual. However, because of space constraints it focuses on analyses that are unique to dry-stacked modular

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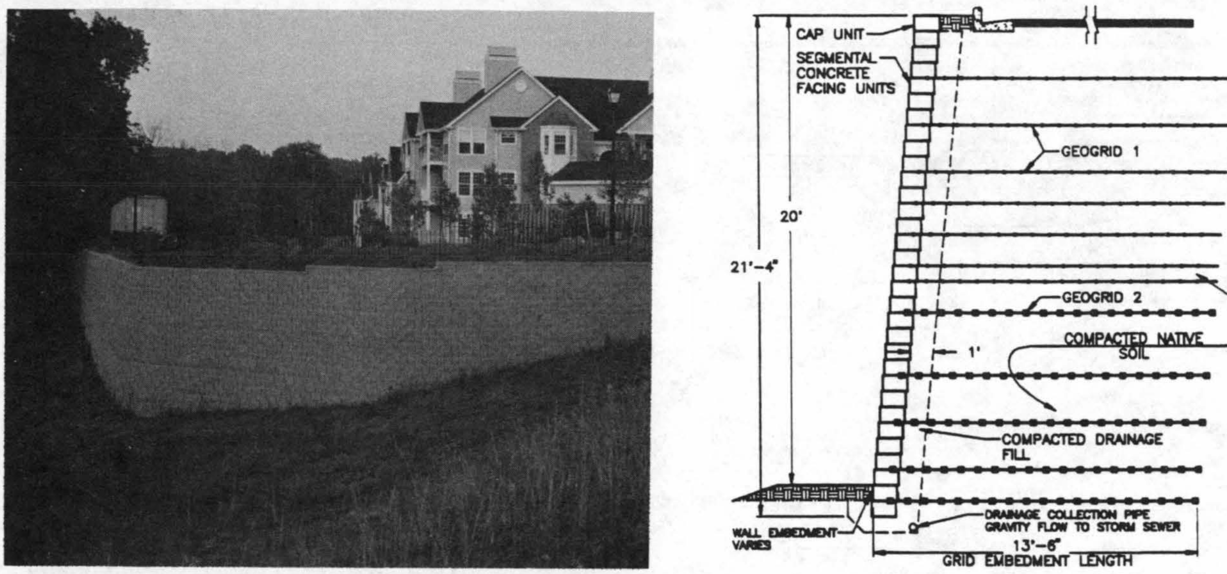

FIGURE 1 Example project from NCMA design manual.

concrete unit construction or exceptions to FHWA and AASHTO guidelines that simplify calculations or reduce current conservativeness in analysis and design.

SEGMENTAL RETAINING WALL UNITS

Modular concrete facing units are produced using machine-molded or wet-casting methods and are available in a wide range of shapes, sizes, and finishes. Examples of some commercially available segmental units are illustrated in Figure 2. Most proprietary units are 8 to 60 cm high, 15 to 80 cm wide (toe to heel), and 15 to 180 cm long. The modular units typically vary from 14 to 45 kg each and may be solid, hollow, or hollow and soil-infilled.

The units may be cast with a positive mechanical interlock in the form of shear keys or leading/trailing edges. Alternatively, the connections may be essentially flat frictional interfaces that include mechanical connectors such as pins, clips, or wedges. The principal purpose of the connectors is to assist with unit alignment and to control wall facing batter during construction. Segmental retaining walls are constructed with a stepped face that results in a facing batter ω that ranges from 3 to 15 degrees. Most facing systems are between 7 and 12 degrees. Shear transfer between unit layers is developed primarily through shear keys and interface friction. However, for interface layers under low normal pressures, a significant portion of shear transfer may be developed by mechanical connectors.

The physical requirements for mortarless dry-cast concrete units with respect to mix design, minimum compressive strength, and water absorption are documented in a separate publication by NCMA (7).

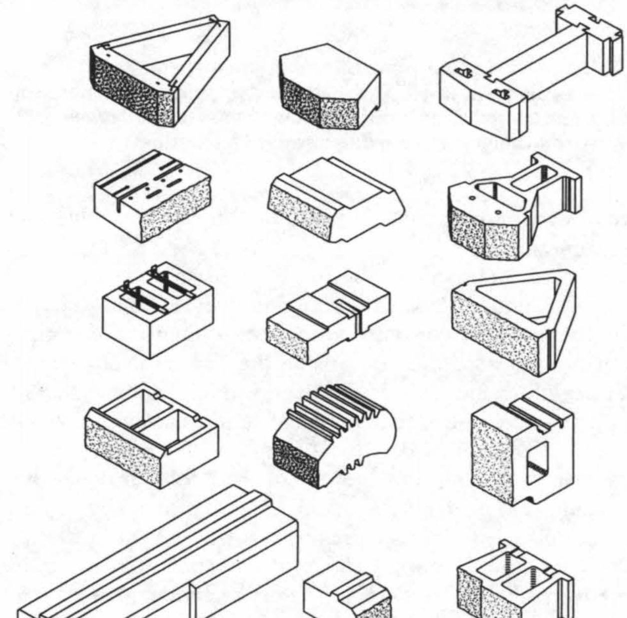

FIGURE 2 Examples of segmental retaining wall units.

NCMA ANALYSIS AND DESIGN METHODOLOGY

Figure 3 (*top*) shows principal components of a geosynthetic-reinforced soil segmental retaining wall. The geosynthetic reinforcement layers in the reinforced soil zone are extended through the interface between facing layers to create an es-

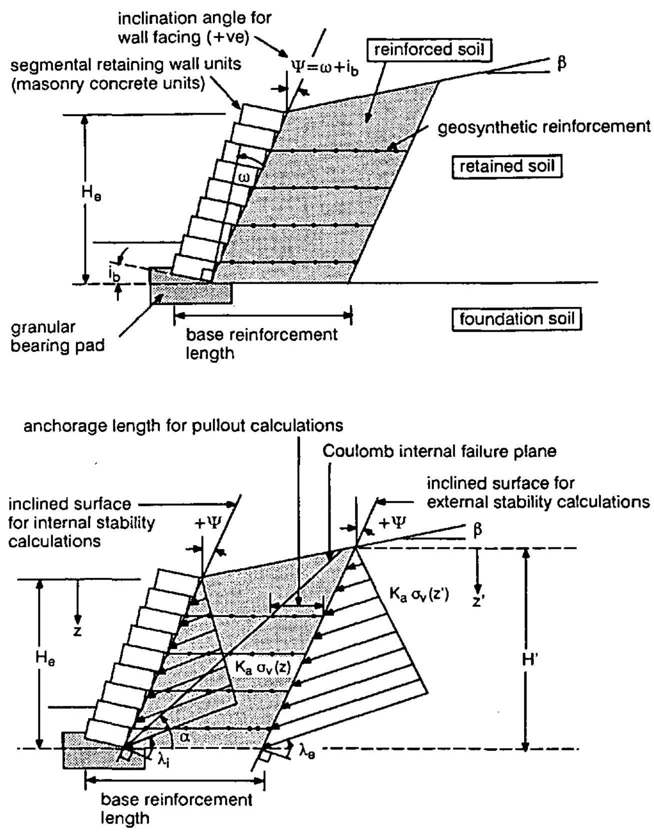

FIGURE 3 Principal components, geometry, and earth pressures assumed in NCMA method: *top*, principal components and geometry for segmental retaining wall systems; *bottom*, principal geometry and earth pressure distributions.

sentially frictional connection with the dry-stacked column of masonry units.

Potential failure mechanisms for geosynthetic-reinforced segmental retaining walls are summarized in Figure 4. External failure mechanisms consider the stability of an equivalent gravity structure comprising the facing units, geosynthetic reinforcement, and reinforced soil fill. Internal stability calculations are restricted to potential failure mechanisms within the reinforced soil zone. Local stability calculations are focused on the stability of the dry-stacked column that forms the facing and the connections with the reinforcement layers. Design of the maximum unreinforced wall height at the top of the structure [Figure 4 (*bottom*)] is carried out using the stability analyses and factors of safety recommended for conventional (gravity) segmental retaining walls.

Not illustrated in Figure 4 is the requirement that global stability of the structure be satisfied as is the case for all retaining wall systems. Conventional slope stability methods of analyses that have been modified to include the stabilizing influence of horizontal layers of geosynthetic reinforcement can be used for this purpose (*5*,p.287).

Coefficient of Active Earth Pressure

The NCMA manual assumes that the retained soil and the soil in the reinforced soil zone are both at a state of incipient collapse corresponding to an active earth pressure condition. This assumption is consistent with FHWA and AASHTO guidelines and is reasonable given that a dry-stacked column of modular concrete units is outwardly flexible and the geosynthetic reinforcement materials are extensible. The calculation of active earth pressure coefficient K_a for external stability calculations in the AASHTO document is based on the following expression:

$$K_a = \cos\beta \, \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad (1)$$

and is called the Rankine solution in this paper. Parameter ϕ is the peak friction angle of the retained soil, and β is the slope angle from the horizontal. The active earth forces are assumed to act parallel to the backslope. Although not explicitly stated in the AASHTO document, most engineers assume that the same approach applies to active earth pressures and force inclination during internal stability calculations. The FHWA document also recommends that Equation 1 be used for internal stability calculations and that the direction of active earth forces be taken parallel to the backslope angle. However, the FHWA guidelines recommend that a classical Coulomb wedge solution be used to calculate an equivalent coefficient of active earth pressure K_a in external stability calculations.

The line of action of active earth forces is also taken as parallel to the backslope angle during external stability calculations, according to the FHWA guidelines. The distribution of lateral earth pressures in both documents is assumed to be triangular because of soil self-weight and constant with depth below any uniformly distributed surcharge pressure.

The facing batter ω for segmental retaining wall systems typically ranges from 3 to 15 degrees; for most systems it falls between 7 and 12 degrees. In addition, the wall footing may be inclined at some angle i_b, which results in a farther net wall face inclination y from the vertical where $y = w + i_b$ (see Figure 3).

Rankine earth pressure theory as used in the AASHTO and FHWA guidelines for internal stability calculations cannot explicitly consider the reduction in lateral earth pressure developed within the reinforced soil zone due to neither an inclined wall facing nor the interface shear resistance that may be mobilized at the back of the rough wall units. Furthermore, the use of different lateral earth pressure theories for external and internal stability calculations in the FHWA document is inconsistent. It is also noted that in a number of case studies it has been demonstrated that Rankine active earth pressure theory consistently overestimates measured lateral earth pressures at the back of vertical or near-vertical wall facings under in-service conditions and hence overestimates tensile forces in the geosynthetic reinforcement (*1*,*8*,*9*,p.15).

In the NCMA manual a single formulation is used to calculate the coefficient of active earth pressure for both internal and external earth pressures. Coefficient K_a is based on the Coulomb wedge solution (*10*) for an inclined wall face at angle ψ and mobilized interface friction angle λ:

$$K_a = \frac{\cos^2(\phi + \psi)}{\cos^2(\psi)\cos(\psi - \lambda)\left[1 + \frac{\sin(\phi + \lambda)\sin(\phi - \beta)}{\cos(\psi - \lambda)\cos(\psi + \beta)}\right]^2} \qquad (2)$$

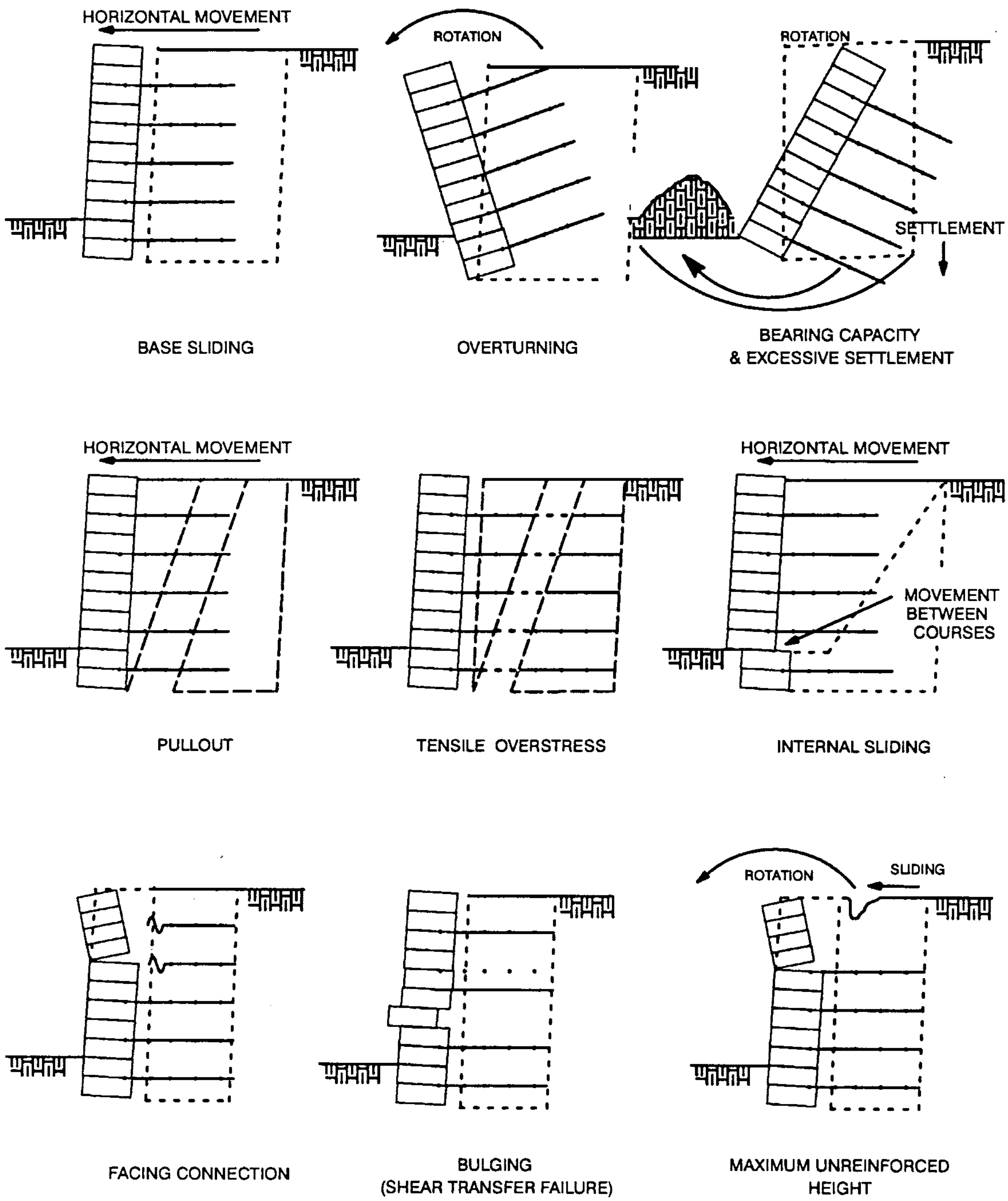

FIGURE 4 Assumed failure mechanisms for external (*top*), internal (*middle*), and local (*bottom*) stability analyses.

In the NCMA manual the inclination of the interface surface for both internal and external stability calculations is taken as parallel to the line connecting the heels of the dry-stacked facing units (surfaces at angle ψ to the vertical in Figure 3). The distributions of lateral earth pressures are taken as being triangular because of soil self-weight and constant with depth for a uniform distributed surcharge pressure as in the FHWA and AASHTO methods.

Unlike the FHWA and AASHTO methods, however, shear resistance is assumed to be mobilized along the interface surfaces identified on Figure 3 (*middle*). Outward movement of the facing and reinforced soil zone is assumed to generate positive interface shear at the back of the facing units ($+\omega_i$) and at the back of the reinforced soil zone ($+\gamma_\theta$). For internal stability calculations the interface shear angle acting between the inclined surface (ψ) and the reinforced soil is taken as $\lambda_i = 2\phi/3$. This assumption is consistent with the mobilized friction angle that is assumed to operate at the interface formed by compacted granular soil in contact with concrete walls in conventional retaining wall design. Interface friction is assumed to be fully mobilized at the back of the reinforced soil zone (i.e., $\lambda_\theta = \phi$ where ϕ is the lesser of the peak friction angle for the retained and reinforced soil materials).

To simplify calculations in the NCMA manual, only the horizontal component of lateral earth pressures due to soil self-weight and any uniformly distributed surcharge loading are considered for external and internal stability calculations.

It should be noted that Equations 1 (AASHTO) and 2 (NCMA) yield the same solution for the case of a horizontal backslope, a vertical facing and no interface shear resistance (i.e., $\psi = \beta = \lambda = 0$). The method recommended by FHWA for external stability calculations and the NCMA method yield the same solution for $\beta = \lambda = \phi$.

Figure 5 shows the relative magnitude of horizontal earth pressures used in internal and external stability calculations based on the Rankine solution (FHWA, AASHTO) and the Coulomb solution (NCMA). The NCMA approach results in lower values of horizontal earth pressure with increasing wall inclination, which is consistent with the notion that earth forces should diminish with wall batter. The resulting conservativeness in design for inclined wall faces based on the Rankine solution may be significant. Figure 5 illustrates that the Coulomb solution is about 55 percent of the Rankine solution for a facing inclination of $\psi = 15$ degrees, $\phi = 35$ degrees, and a horizontal backslope.

Orientation of Internal Failure Plane

In the AASHTO and FHWA guidelines the internal failure plane is assumed to propagate up into the reinforced soil mass from the heel of the wall face at an angle α to the horizontal [Figure 3 (*middle*)] where

$$\alpha = \frac{\pi}{4} + \frac{\phi}{2} \tag{3}$$

Here ϕ is the peak friction angle of the reinforced soil. This orientation is inconsistent with the theory that is used to develop Equation 1 for $\beta > 0$. In the NCMA manual the orientation of the potential internal failure plane is consistent with the Coulomb wedge theory used to arrive at Equation

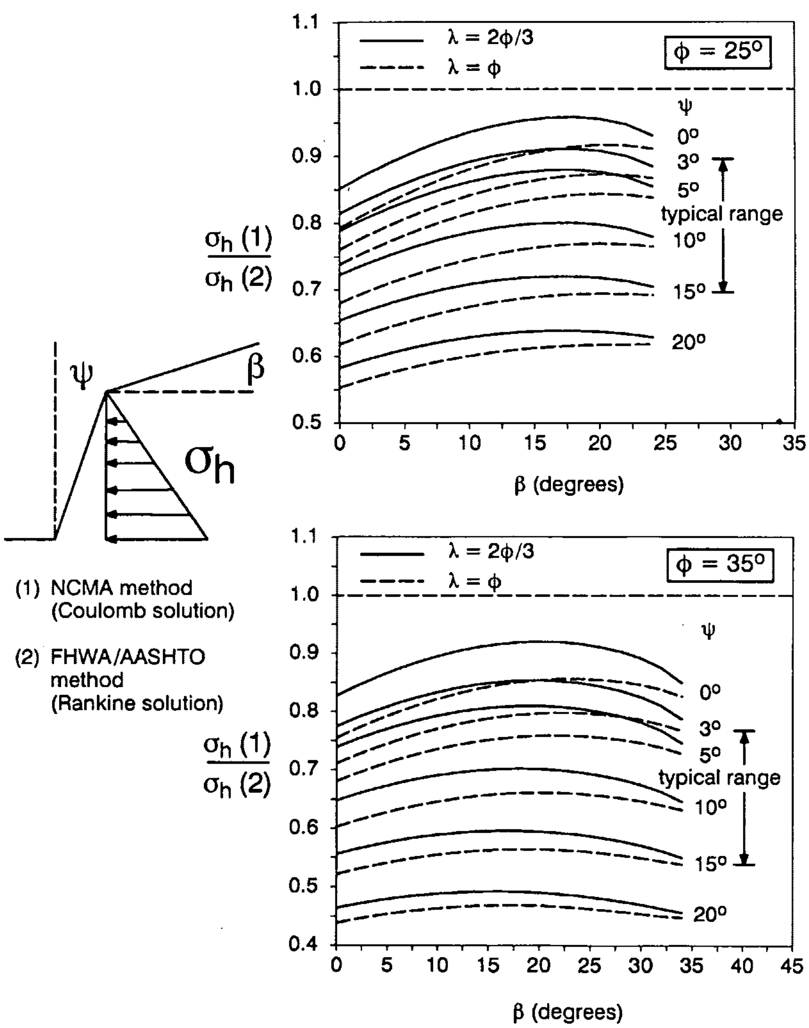

FIGURE 5 Ratio of Coulomb solution to Rankine solution for the calculation of horizontal earth pressures for internal stability analyses.

2. Orientation α is calculated according to the following equation:

$$\tan(\alpha - \phi) = \{-\tan(\phi - \beta) + \sqrt{\tan(\phi - \beta)[\tan(\phi - \beta) + \cot(\phi + \psi)][1 + \tan(\lambda_1 - \psi)\cot(\phi + \psi)]}\} \div \{1 + \tan(\lambda_1 - \psi)[\tan(\phi - \beta) + \cot(\phi + \psi)]\} \quad (4)$$

and has been taken from Jumikis (*10*). Equation 4 degenerates to Equation 3 for the case $\beta = \psi = \lambda_i = 0$. In the NCMA manual, values of $\alpha = \phi(\phi, \beta, \psi, \lambda_i)$ can be taken directly from a series of tables or interpolated between values in the tables.

An implication of Equation 4 to internal stability calculations is that internal failure planes are shallower than those calculated using Equation 3 as illustrated in Figure 6. To satisfy pullout criteria, some reinforcement layer lengths close to the crest of the wall may be longer than those calculated using the AASHTO approach. However, the NCMA method does not require that all reinforcement layers have the same length as required in the AASHTO document. NCMA requires that the minimum length of all reinforcement layers be at least equal to the base length of the reinforced mass required to satisfy all external stability requirements but not less than 0.6H for critical structures or 0.5H for noncritical structures (see later discussion). The designer is permitted to increase locally the width of the reinforced soil zone and the length of individual layers near the crest of the wall as required to satisfy pullout criteria.

Base Eccentricity and Minimum Reinforcement Lengths

The requirement that the net vertical load transferred to the base of the reinforced soil zone must act within the middle third of the base of the composite structure is not a requirement in the NCMA manual. This so-called eccentricity criterion, which is found in current AASHTO and FHWA guidelines, is not considered to be applicable to flexible wall structures such as geosynthetic-reinforced segmental retaining walls. The notion that tensile contact pressures can develop at the base of a reinforced soil mass is counterintuitive and has not been observed in instrumented structures (*1,11*).

However, experience with reinforced soil walls with narrow reinforcement zones is not available in North America, and hence a value of 0.6H for the minimum base width of the reinforcement zone is recommended in the NCMA manual for critical structures and 0.5H for noncritical structures regardless of the result of external stability calculations [Figure 4 (*top*)]. This criterion is less conservative then the minimum base reinforcement length of 0.7H or 2.4 m (whichever is less) that currently appears in the AASHTO guidelines. The empirical constraint of 1 m on the minimum anchorage length that appears in AASHTO is reduced to 0.6 m in the NCMA manual for critical structures and 0.3 m for noncritical structures. This new criterion also helps to eliminate undue conservativeness that may result from the calculation of the internal failure plane using Equation 4, which is shallower than α calculated using Equation 3 found in AASHTO and FHWA.

Hinge Height Concept

The maximum height that a column of dry-stacked facing units can be placed without toppling over or leaning into the retained soil mass is called the hinge height (H_h) in the NCMA manual. Segmental retaining wall systems composed of dry-stacked columns of concrete units are typically constructed at some inclination $\psi > 0$. The effect of inclination is that the column weight above the base of the wall or above any other interface may not correspond to the weight of the facing units above the reference elevation. The concept is illustrated in Figure 7. Hence, for walls with $\psi > 0$, the normal stress acting

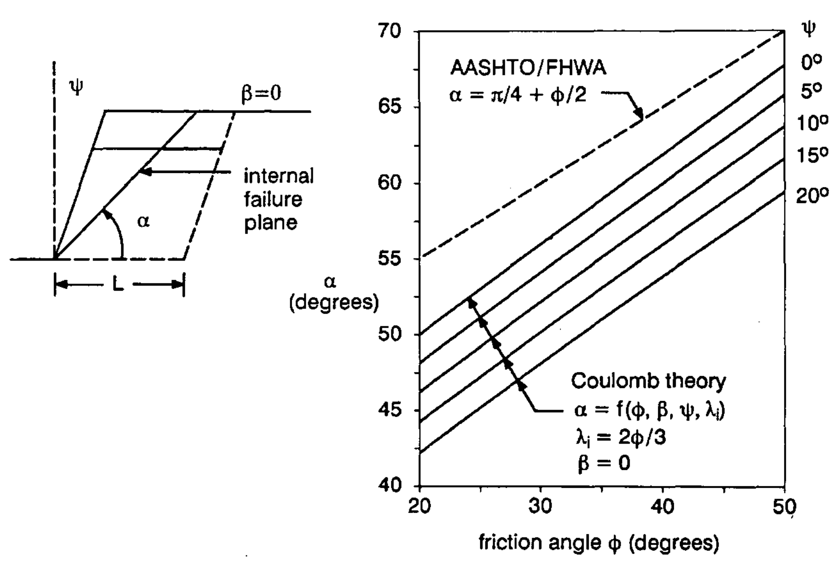

FIGURE 6 Comparison of internal failure plane orientation based on AASHTO/FHWA and NCMA recommendations.

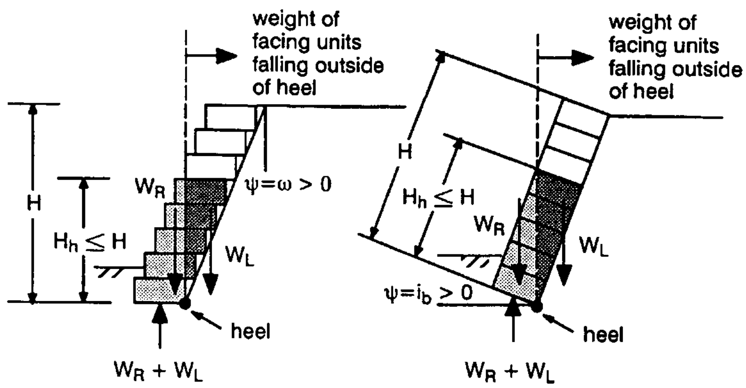

FIGURE 7 Hinge height concept: *left*, base inclination $i_b = 0$; *right*, facing setback angle $\omega = 0$.

at the interface is limited to the lesser of the hinge height or the height of the wall above the interface.

An important consequence of the hinge height is that it may control the magnitude of frictional shear resistance available between facing units and the frictional connection strength at the geosynthetic-facing unit interface. The hinge height also directly influences the toppling resistance (i.e., overturning resistance) of the unreinforced column at the crest of the wall [Figure 4 (*bottom*)]. The results of hinge height calculations using moment equilibrium with respect to the heel of a dry-stacked column of facing units are illustrated in Figure 8. The figure shows that for a typical solid unit with a block width to height ratio of 2, the number of units corresponding to the hinge height diminishes rapidly with increasing wall inclination.

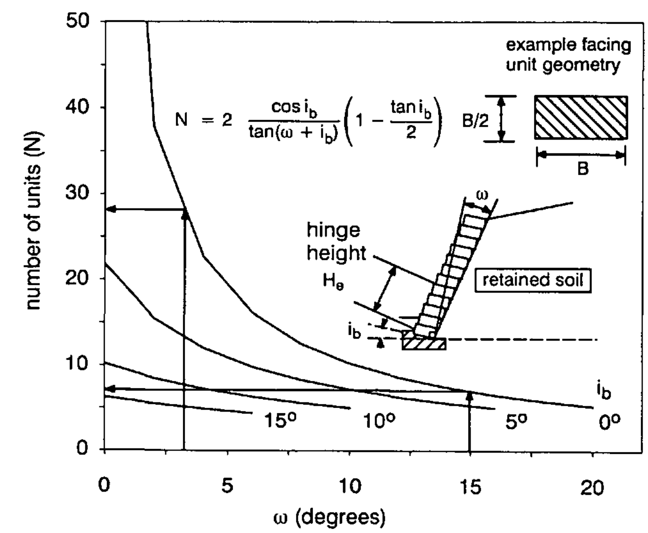

FIGURE 8 Influence of wall inclination on number of facing units within hinge height.

Interface Shear Transfer

The NCMA methodology assumes that (unbalanced) lateral earth pressures act against the back of the dry-stacked column of segmental wall units. The calculation of the magnitude and distribution of lateral pressures has been described earlier. These distributed loads must be transferred as shear forces between units in order that the wall system remains stable. The calculation of required shear transfer is carried out using a continuously supported beam analog in which the lateral earth pressure is taken as the distributed load and the reinforcement layers are taken as the supports. The magnitude of shear capacity available at the interface of concrete units can be established only from the results of full-scale direct shear testing. The NCMA manual includes a test method for the determination of the direct shear resistance between units (NCMA Test Method SRWU-2). The test results are reported as equivalent Mohr-Coulomb strength parameters (a_u, λ_u), which can be used to estimate the ultimate interface shear strength V_U on the basis of the applied interface normal stress σ_n where

$$V_U = a_u + \sigma_n \tan \lambda_u \tag{5}$$

It should be noted that the shear capacity at a geosynthetic-modular concrete unit interface may be reduced by the presence of a geosynthetic inclusion. Consequently, shear tests must be carried out to quantify the ultimate strength of segmental units with and without a geosynthetic between course layers.

Connection Strength Between Modular Units and Geosynthetic Reinforcement

The connection between the geosynthetic reinforcement and the dry-stacked column of modular concrete units is a critical construction detail in reinforced segmental retaining wall design. Most connections are essentially frictional in nature, although a portion of pullout resistance may also be developed by the bearing action of transverse geogrid members against concrete keys or mechanical connectors.

In addition to differences in interface geometry and connection type, the connection performance will be influenced by (a) hollow or solid masonry concrete construction, (b) whether the hollow core is left empty or infilled with granular soil, (c) tolerances on block dimensions, (d) quality of construction, and (e) thickness, structure, and polymer type of the geosynthetic. Because of the large number of variables, the tensile capacity of a geosynthetic-reinforcement connection can be established only from large-scale tests carried out using a representative range of normal stresses. The NCMA manual contains a test to perform and interpret the results of connection tests (NCMA Test Method SRWU-1). The method was based on earlier work reported by Bathurst and Simac (*12*). The method of test recommended by NCMA has the following features:

- Test specimens must be at least 1 m wide in order to model the effect of block joints on connection performance. Lesser widths are permitted if it can be demonstrated that the connection performance is the same as for wider models. At least one running joint must be located at the center of pull.
- Tests must be performed on actual specimens of segmental retaining wall units since variations in the dimensions of nominal identical units from different plants or different molds must be expected.
- Over a range of normal stresses the relationship between interface pressure σ_n and connection capacity can be expressed by a Mohr-Coulomb friction law using parameters (a_{cs}, λ_{cs}). Failure envelopes are based on a peak (ultimate) load criterion and a deformation criterion (20-mm displacement). Different values for strength parameters may be required over different ranges of normal pressure to reflect the nonlinear failure envelope that often results from connection testing (*12*). The range of normal pressures applied in a test series must include the normal pressure anticipated at each connection.

The maximum design connection force is assumed to be equal to the maximum tensile force calculated for the reinforcement layer using a contributory area approach (i.e., the same concept as in the AASHTO and FHWA methods but with earth pressures calculated using K_a from Equation 2). The connection forces are not reduced with increasing wall elevation as recommended by FHWA.

FACTORS OF SAFETY

Recommended minimum factors of safety are summarized in Table 1. The NCMA manual preserves the factor of safety approach that is common practice for geotechnical engineers in North America for the design of earth structures. Never-

TABLE 1 Recommended Minimum Factors of Safety for Design of Geosynthetic-Reinforced Soil Segmental Retaining Walls

FAILURE MODE		CRITICAL APPLICATIONS	NON-CRITICAL APPLICATIONS
Base Sliding	FS_{sld}	1.5	1.5
Overturning	FS_{ot}	2.0	2.0
Bearing Capacity	FS_{bc}	2.0	2.0
Global Stability	FS_{gl}	1.5	1.3
Tensile Over–stress	FS_{to}	1.2	1.0
Pullout (peak load criterion)	FS_{po}	1.5	1.5
Pullout (serviceability criterion)	FS_{po}	1.0	N/A
Facing Shear (peak load criterion)	FS_{sc}	1.5	1.5
Facing Shear (serviceability criterion)	FS_{sc}	1.0	N/A
Connection (peak load criterion)	FS_{cs}	1.5	1.5
Connection (deformation criterion)	FS_{cs}	1.0	N/A

NOTES:

1. The minimum factors of safety given in this table assume that stability calculations are based on **measured site-specific soil/wall data.** Measured data are defined as the results of tests carried out on actual samples of soils and geosynthetic products for the proposed structure and actual samples of masonry concrete units (i.e. the same molds, forms, mix design and infill material or same broad soil classification type (e.g. G,S) if applicable).
2. The designer should use larger factors of safety than those shown in this table or conservative estimates of parameter values when **estimated data** are used. **Estimated data** include bulk unit weight and shear strength properties taken from the results of ASTM methods of test (or similar protocols) carried out on samples of soil having the same USCS classification as the project soil and the same geosynthetic product. **Estimated data** for facing shear capacity and connection capacity analyses shall be based on laboratory tests carried out on the same masonry concrete unit type under representative surcharge pressures for the project structure (and the same broad soil classification type (e.g. G,S) if applicable).
3. For critical structures, minimum factors of safety based on serviceability *and* peak load criteria must be satisfied for pullout, facing shear and facing connection failure modes.
4. Design of the maximum unreinforced wall height at the top of the structure is carried out using the stability analyses and factors of safety recommended for conventional (gravity) segmental retaining walls.
5. Minimum wall embedment depths as a function of wall height follow recommendations given in AASHTO/FHWA. In no case shall the minimum wall embedment depth be less than 0.45m (1.5 ft) for critical structures or 0.15 m (0.5 ft) for non-critical structures.

theless, the NCMA manual introduces recommended minimum factors of safety for failure mechanisms not previously addressed (e.g., interface shear failure and connection failure). In addition, the manual distinguishes in some cases between minimum recommended factors of safety on the basis of whether a structure is critical or noncritical. A noncritical structure in NCMA manual terminology is "a structure in which loss of life would not occur as a result of wall failure nor would failure result in significant property damage or loss of necessary function of adjacent services or structures." A critical structure is clearly the converse. Permanent structures are usually considered critical structures and are designed for a life of 75 to 100 years. Similarly, transportation-related structures would normally be considered critical structures.

The footnotes to Table 1 explain that the factors of safety listed are minimum values. The recommended minimum factors of safety should be used only if design parameters are taken from laboratory tests using materials identical to those proposed in the field. The designer should adjust factors of safety upward when design parameters are estimated from laboratory tests carried out on similar materials.

Factors of safety in the table for external modes of failure, tensile overstress, and pullout in noncritical structures using measured data are consistent with FHWA recommendations.

LONG-TERM DESIGN STRENGTH OF GEOSYNTHETIC REINFORCEMENT

The long-term design strength (LTDS) of the geosynthetic reinforcement is viewed by some as the most important single parameter in geosynthetic-reinforced soil wall design. However, its calculation is often a source of unease with many designers because of questions about durability, construction damage, and creep.

In the NCMA design manual two approaches are available to the designer for calculating the LTDS for a candidate reinforcement: Methods A and B. Space constraints in this paper prevent a complete description of the methods, but the reader is referred to the NCMA manual for a complete description. A brief statement of the two methods follows.

Method A

Method A has been adapted from a recent publication by FHWA for the design, analysis, and construction of reinforced earth slopes and embankments on firm foundations (*13*,p.98). This document is a consensus geosynthetics manufacturing industry standard that is based on the AASHTO (*6*) and FHWA (*5*,p.287) guidelines referenced earlier in the paper and selected Geosynthetic Research Institute (GRI) standards (*14*). The modification relates to the introduction of an overall factor of safety for uncertainties as proposed by AASHTO for reinforced soil retaining wall design.

Method B

Method B was developed by the authors to provide a comprehensive treatment of the calculation of long-term design load for geosynthetics in soil reinforcement applications. Method B borrows heavily from the work of Jewell and Greenwood (*15*) and is similar to European practice for the calculation of LTDS. The principal difference between the two approaches is that Method B decouples the factor of safety against overall uncertainty from the calculation of LTDS. In addition, specific calculation steps are contained in Method B that allow the designer to estimate LTDS from product-specific creep data using a common framework that is independent of the geosynthetic product type.

Method B in the NCMA manual recommends that LTDS be related to a maximum load, T_{lim}, which is the estimated maximum in-isolation, constant load that will just prevent the cumulative elastic and plastic strain in the reinforcement from exceeding a maximum strain value over the design life of the structure. In no case is the design maximum strain value allowed to be greater than 10 percent. The definition adopted in this manual is similar to the serviceability state criterion that appears in the current AASHTO guidelines.

The LTDS is calculated as

$$\mathrm{LTDS} = \frac{T_{\mathrm{lim}}}{\mathrm{FC} \times \mathrm{FD} \times \mathrm{FB}} \tag{6}$$

where

FC = partial material factor for construction site installation damage,
FD = partial material factor for chemical degradation, and
FB = partial material factor for biological degradation.

The definition of LTDS in the NCMA manual differs from AASHTO and GRI Standards of Practice GG4 and GT7 by restricting all uncertainties in the calculation of LTDS to factors related directly to the long-term strength of the geosynthetic reinforcement under in-service conditions. A so-called overall factor of safety is not included in Equation 6 because this overall uncertainty is independent of the presence of the geosynthetic reinforcement in the structure. The philosophy adopted in the NCMA manual is that the degree of uncertainty in soil properties should be accounted for by basing the selection of factors of safety on estimated values or site-specific data as noted in Table 1. Uncertainty in external loading is best treated by using conservative estimates of parameters that contribute to destabilizing forces.

CONCLUDING REMARKS

This paper has summarized the most important features of the NCMA design manual for analysis and design of geosynthetic-reinforced soil retaining walls that use dry-stacked masonry concrete units as the facing system. The emphasis in the manual has been to present the designer with a comprehensive and rational approach to the design and analysis of modular masonry wall systems. The methodology is sufficiently detailed to allow the designer to quantify the influence of candidate facing units on the stability of otherwise identical geosynthetic-reinforced soil walls. This feature is not available in current FHWA and AASHTO guidelines. Finally, it should be noted that the manual also includes an integrated design and analysis approach for conventional (gravity) structures that use unreinforced backfills and contains construction guidelines and sample material specifications.

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