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RECENT DEVELOPMENTS IN GEOSYNTHETIC REINFORCED SEGMENTAL RETAINING WALLS IN NORTH AMERICA

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SYNOPSIS: Geosynthetic reinforced segmental retaining walls are a recent technology that has gained wide popularity in North America for reasons of performance, aesthetics, cost and expediency of construction. These systems are identified by the use of a column of dry-stacked (mortarless) modular concrete units to form a hard facing. The facing column is attached to horizontal layers of geosynthetic reinforcement to create a composite reinforced soil mass. The paper focuses on the analysis and design of these systems and discusses important performance issues related to the stability of the facing column and the connections. Test protocols developed by the author and co-workers to obtain quantitative data for facing column connection and interface shear behaviour are described and the implications of example data to design of these systems identified. The design methodologies and specialized performance test protocols discussed in this paper have been adopted by the National Concrete Masonry Association (NCMA) in North America.

KEYWORDS: Segmental, Modular block, Retaining wall, Geosynthetics, Reinforced structures.

1. INTRODUCTION

The use of dry-stacked columns of interlocking modular concrete units as the facing for geosynthetic reinforced soil retaining wall structures has increased dramatically in North America since their first appearance in the mid-1980's (Bathurst and Simac 1994). Examples of completed projects are illustrated in **Fig 1, 2** and **3**. The National Concrete Masonry Association (NCMA) in the USA has adopted the term *Soil-reinforced Segmental Retaining Wall* (SRW) to identify these types of retaining wall systems.

Reinforced segmental retaining wall systems offer advantages to the architect, engineer and contractor as described below. 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. For some large wet-cast units or transportation related projects, concrete levelling pads may be used to maintain wall alignment and batter. The mortarless modular concrete units are easily transportable and therefore facilitate construction in difficult access locations. 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 can typically erect 20 - 40 square metres of wall face per day. The economic benefit due to these features is that reinforced segmental retaining walls in excess of 1 m in height typically offer a 25 to 45% cost saving over comparable conventional cast-in-place concrete retaining walls (Bathurst and Simac 1994). At the time of writing, the majority of reinforced segmental retaining wall structures have been built using polymeric geogrid materials as the geosynthetic reinforcement. Nevertheless, the design methodologies reviewed in this paper do not preclude the use of some woven geotextiles which may introduce further economies for these systems.

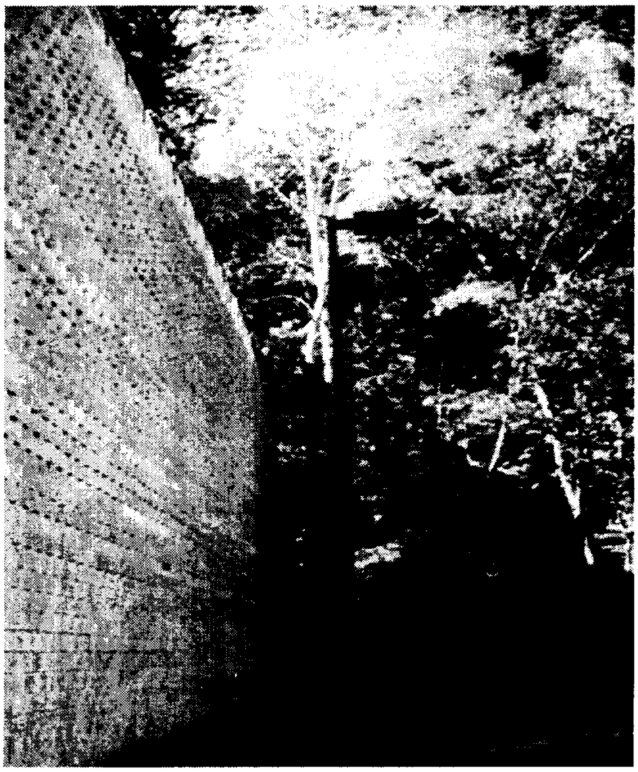


Fig 1. Example 12 m high geosynthetic reinforced segmental retaining wall (Anderson et al. 1991)

Fig 2. Connection detail (Bathurst and Simac 1994)

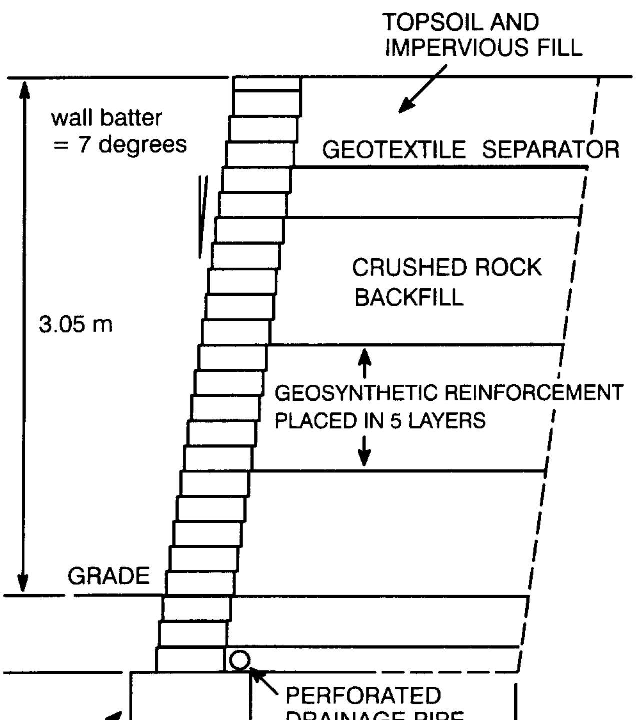


Fig 3. Example cross-section of geosynthetic reinforced segmental retaining wall (after Wetzel et al. 1995)

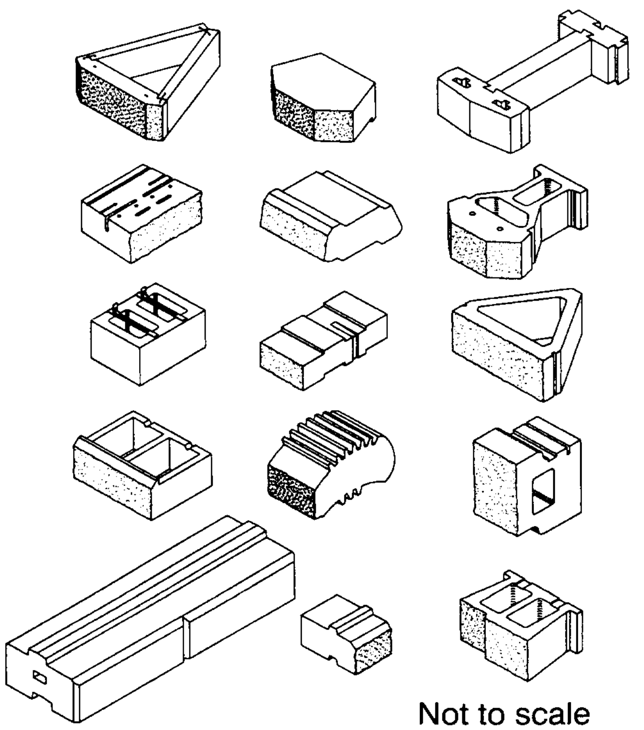


Fig 4. Examples of some segmental retaining wall units (NCMA 1997)

However, the discrete nature of the dry-stacked column of modular concrete units that is the distinguishing feature of reinforced segmental retaining walls introduces additional and unique design considerations. Conventional engineering practice for geosynthetic reinforced soil retaining walls prior to 1993 did not fully address all performance issues for modular systems since they were developed largely with the use of precast concrete panel systems in mind.

The paper reviews recently developed methods of analysis and design of geosynthetic reinforced soil retaining walls that use dry-stacked (mortarless) modular concrete units as the facing system. The paper is limited to structures constructed in non-seismic areas. Test protocols developed by the author and co-workers to obtain quantitative data for facing column connection and interface shear behaviour are also described and the implications of example data to design of these systems identified.

Analysis and design methods described in this paper for routine structures were adopted by the National Concrete Masonry Association (NCMA) in the USA in 1993 (Simac et al. 1993) and updated in 1997. The NCMA is an umbrella organization whose mandate is to support and advance the common interest of its North American members in the manufacture, marketing, research, and application of concrete masonry products.

2. 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 **Fig 4**. Most proprietary units are 80 to 600 mm in height, 150 to 800mm in width (toe to heel), and 150 to 1800mm in length. The modular units typically vary from 14 to 48kg each. The modular concrete units may be solid, hollow, or hollow and soil infilled.

The units may be cast with a positive mechanical interlock in the form of concrete shear keys or leading/trailing edges. Alternatively, interlocking between layers may be developed by essentially flat frictional interfaces that may include mechanical connectors such as pins, clips or wedges. The principal purpose of mechanical connectors is to assist with unit alignment and to control wall facing batter during construction. Recently a number of systems have been developed that include a molded plastic connector that is mated to a slot cast into the top surface of the facing units (e.g. Austin and Martin 1996). These plastic connectors are designed to interlock with the apertures of the geosynthetic reinforcement (typically integrally formed polyolefin uniaxial geogrids).

Segmental retaining walls are usually constructed with a stepped face that results in a facing batter that ranges from 3 to 15 degrees. The majority of 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 (e.g. close to the wall crest) a significant portion of shear transfer may be developed by mechanical connectors.

Typical specification limits for dry-cast masonry concrete blocks in retaining wall applications are as follows: minimum compressive strength = 21 MPa; maximum water absorption 6 - 7% and; maximum dimensional tolerance = 3 mm (Bathurst and Simac 1994). The physical requirements with respect to mix design can be found in separate publications by the NCMA and the American Society for Testing and Materials (ASTM) standards (e.g. ASTM C 90). Methods to sample and test concrete masonry units for compressive strength, absorption, unit weight (density), moisture content and dimensions are provided in ASTM C 140. A compressive strength of 21 MPa is more than adequate from a structural point of view. However, a minimum compressive strength of 40MPa has been required by at least one state department of transportation in the USA. Higher strengths are possible by adjusting the mix design and manufacturing process. Reinforcement steel is not used in dry-cast masonry units or wet-cast units for reinforced segmental retaining wall applications in North America.

3. ANALYSIS AND DESIGN

Methodologies for the analysis and design of segmental retaining walls in the United States can be found in guidelines published by three different organizations: the Federal Highway Administration (FHWA); the American Association of State Highway and Transportation Officials (AASHTO) and; the National Concrete

Masonry Association (NCMA). In order to be consistent with conventional North American practice for the design and analysis of retaining wall structures, stability calculations in these guidelines adopt a limit-equilibrium approach together with the assumption of c - ϕ soils. The cited references all adopt a gravity structure approach for external stability calculations and variations of the "tied-back wedge" approach for internal stability calculations.

A critical review of the design and analysis methodologies found in the first edition of the NCMA guidelines (1993) and AASHTO/FHWA (Christopher et al. 1989) documents in effect at that time can be found in papers by Bathurst et al. (1993) and Simac et al. (1993). The NCMA guidelines published in 1993 were essentially a refinement of earlier FHWA and AASHTO guidelines (1992 or earlier). These FHWA and AASHTO guidelines were developed based on experience with geosynthetic and steel strip reinforced soil retaining wall systems that used primarily precast concrete panels. In 1996 and 1997, AASHTO and FHWA documents were revised (AASHTO 1996, 1997; Elias and Christopher 1997). These documents now include some of the recommendations found in the first edition of the NCMA manual. In 1997, the NCMA design manual for reinforced segmental retaining walls was modified to bring some features, such as the calculation of the long-term design strength for the geosynthetic reinforcement, in agreement with current AASHTO/FHWA recommendations. Nevertheless, the most current AASHTO and FHWA guidelines do not explicitly consider all potential failure mechanisms for reinforced segmental retaining wall systems. It can be argued that it is possible to design a wall according to AASHTO and FHWA recommendations and still generate facing column instability.

The NCMA methodology has the advantage that the designer can quantify the influence of different candidate facing units on the stability of otherwise identical geosynthetic reinforced soil walls. The NCMA manual also includes an integrated design and analysis approach for conventional (gravity) structures that use unreinforced backfills. Hence the NCMA guidelines offer a unified approach for unreinforced and reinforced segmental retaining wall systems consistent with the notion that both types are essentially gravity structures. Finally, the NCMA document reduces some conservatism found in the FHWA and AASHTO guidelines with respect to the choice of earth pressure theory, base eccentricity criteria and minimum reinforcement lengths. Based on the comments made above, the analytical approach described in this paper for routine structures is based on the NCMA guidelines which were prepared by the author and co-workers (Simac et al. 1993; Bathurst et al. 1993) and recently updated by the NCMA (1997).

4. MODES OF FAILURE

Potential failure modes for reinforced segmental retaining wall structures are illustrated in **Fig 5**. External failure mechanisms consider the stability of an equivalent gravity structure comprising the facing units, geosynthetic reinforcement and reinforced soil fill. Not included in **Fig 5** is global instability which involves failure mechanisms passing through or beyond the reinforced soil mass. Conventional slope stability methods of analysis that have been modified to include the stabilizing influence of horizontal layers of geosynthetic reinforcement can be used for this purpose (e.g. Elias and Christopher 1997). Modes of failure that require special considerations in reinforced segmental retaining wall design and analysis are illustrated in the last five diagrams of **Fig 5**. The reinforcement layers are placed between the masonry units to form a tensile load carrying connection. The modular unit-geosynthetic reinforcement connection capacity can control the spacing and the selection of polymeric reinforcement type. Similarly, adequate unit to unit interface shear capacity is required to prevent internal sliding mechanisms that propagate through to the face of the structure and/or to prevent local bulging of the facing units.

5. APPLICATION OF EARTH PRESSURE THEORY

Limit-equilibrium approaches are routinely adopted for the design and analysis of reinforced segmental retaining walls. Earth pressure distributions and important wall geometry parameters are illustrated in **Fig 6**. In addition to the wall batter (ω) that is generated by the built-in setback of the units, the base course may be inclined at some angle i_b which results in a further net wall face inclination (ψ) from the vertical ($\psi = \omega + i_b$). The Cou-

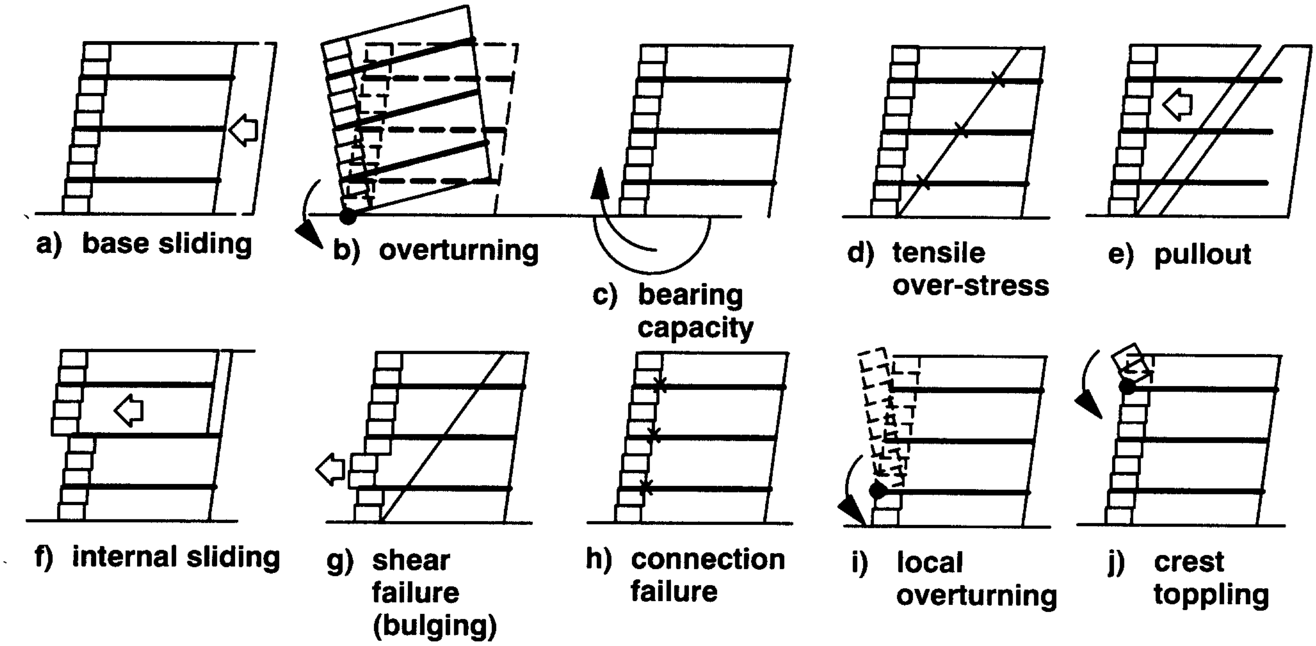


Fig 5. Modes of failure for reinforced SRW structures: external (a,b,c); internal (d,e,f); and facing (g,h,i,j) (after Bathurst and Cai 1995)

lomb approach has been adopted by the NCMA for all stability calculations because it can explicitly accommodate the contribution to lateral earth pressure of wall inclination angle (ψ), backslope angle (β) and shear mobilized at the interfaces between the reinforced soil and retained soil zones (interface friction angle δ_e), and the facing column and reinforced soil zone (interface friction angle δ_i). The Coulomb equation using the notation introduced above is expressed as:

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

where ϕ is the peak friction angle of the soil. Outward movement of the facing and settlement of the reinforced soil mass is assumed to generate positive interface shear at the back of the facing units ($+\delta_i$). For internal stability calculations the interface shear angle acting between the inclined surface (ψ) and the reinforced soil is taken as $\delta_i=2\phi/3$. Interface friction is assumed to be fully mobilized at the back of the reinforced soil zone (i.e. $\delta_e = \phi$ where ϕ is the lesser of the peak friction angle for the retained soil and reinforced soil materials).

In the NCMA manual only the horizontal component of lateral earth pressure due to soil self-weight and any uniformly distributed surcharge loading is considered for external and internal stability calculations. This approach simplifies calculations and results in the conservative assumption that the vertical component of earth pressures does not contribute to resisting forces in stability calculations. Boussinesq solutions are used to calculate any additional lateral stresses due to line loads and other finite distributed surface loads.

Lateral earth pressures are integrated over the contributory area of each reinforcement layer (S_v in **Fig 7**) to calculate the maximum tensile load (T_{max}) to be carried by each layer: e.g. $T_{max} = S_v K_a \cos(\delta-\psi)\, \sigma_v(z)$. This tensile load is used in the calculation of factors of safety against reinforcement pullout, tensile over-stress, and connection failure. Generally, the value of T_{max} will increase with depth of the layer below the crest of the wall, particularly if uniform layer spacings are adopted.

To be consistent with Coulomb theory, the orientation of potential failure planes (**Fig 6**) through the reinforced soil zone are calculated as $\alpha = f(\phi, \beta, \psi, \delta_i)$. The closed-form solution can be found in geotechnical engineering textbooks. The internal failure plane is used to locate the active wedge that must be restrained by the anchorage zone in pullout capacity calculations. For design purposes the internal failure plane is assumed to

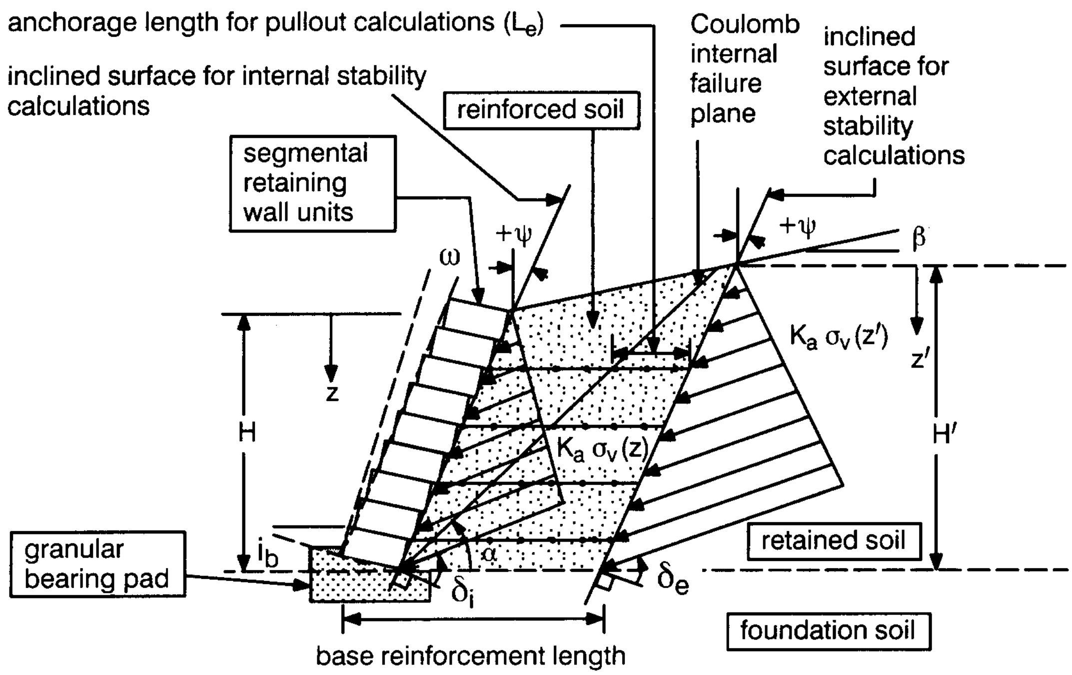


Fig 6. Principal components, geometry and earth pressures assumed in NCMA method (after Bathurst et al. 1993)

propagate from the heel of the lowermost facing column unit. An implication of the Coulomb approach to internal stability calculations is that internal failure planes (α) are shallower than those calculated using the Rankine solution which is often used in conventional retaining wall design (Bathurst and Simac 1994). In order to satisfy pullout criteria, some reinforcement layer lengths close to the crest of the wall may have to be longer than those required at the base of the wall. The NCMA (1997) allows the designer to locally increase 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. However, NCMA (1997) 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.

6. CALCULATION OF ALLOWABLE TENSILE LOAD

The calculation of maximum allowable strength for geosynthetic reinforcement in North America is based on the application of partial factors to a reference index strength. Strategies used to select appropriate values for partial factors and factor of safety expressions for tensile over-stress and anchorage failure for other geosynthetic reinforced soil retaining wall structures are equally applicable to segmental wall design. The following expression is used in the current NCMA manual (NCMA 1997) to calculate the maximum allowable strength (T_a) for a reinforcement layer and follows the same method used in the FHWA guidelines for reinforced slopes (Berg 1992):

$$T_a = \frac{T_{index}}{RF_{CR} \times RF_{ID} \times RF_D \times FS_{UNC}} > T_i \qquad (2)$$

Here T_{index} refers to the ultimate tensile strength of the reinforcement (minimum average roll value) from wide-width strip tests (ASTM D 4595) or single rib geogrid tensile tests (GRI GG1). Partial factors of safety are defined as: RF_{CR} = partial factor of safety for creep deformation (ratio of T_{index} to creep limiting strength interpreted from constant load tests). Typical values vary from 1.5 to 5.0; RF_{ID} = partial factor of safety for installation damage (minimum value not less than 1.05); RF_D = partial factor of safety for durability (minimum value not less than 1.1); FS_{UNC} = partial factor of safety for overall uncertainty (typical value of 1.5). The quantity T_i refers to the maximum tensile load acting in the reinforcement layer (**Fig 7**). The minimum allowable value for the factor of safety against tensile over-stress is unity (i.e. $FS_{OS} = T_a/T_i > 1$; **Table 1**).

7. CALCULATION OF ANCHORAGE CAPACITY

Pullout resistance (R_{PO}) is defined as the maximum force required to cause uniform pullout of the entire embedded length of geosynthetic in a laboratory test (e.g. GRI GG5) and is calculated as:

$$R_{PO} = 2\, L_e\, C_i\, \sigma_v \tan \phi \tag{3}$$

where: L_e = anchorage length beyond the internal failure plane (**Fig 6**); C_i = coefficient of shear stress interaction; σ_v = average vertical stress acting over the geosynthetic in the anchorage zone. The factor of safety against anchorage failure (FS_{PO}) is calculated using:

$$FS_{PO} = R_{PO}/T_i \tag{4}$$

The minimum allowable value for factor of safety against anchorage failure is $FS_{PO} = 1.5$ (**Table 1**).

8. CONNECTION DESIGN

The lateral earth pressures that develop at the back of the facing column will require that the reinforcement layers carry load and that these loads will be transferred to the facing column through the connection system. The conventional approach to assign static tensile loads is based on a contributory area method and is illustrated in **Fig 7**. In FHWA (Elias and Christopher 1997) guidelines and AASHTO (1997) interim guidelines the connection loads are assumed to diminish to 80% of the maximum internal tensile load (T_i) as the top of the wall is approached. Bathurst and Simac (1997) reviewed measured data from two instrumented modular block walls and concluded that the connection forces cannot be assumed to be reduced at the back of the facing column. In the NCMA method of design the connection load is assumed to be equal to 100% of the tensile load calculated using internal stability analyses ("tied back wedge" approach). It is clear from the geometry and distribution of earth pressure illustrated in **Fig 7** that the tensile load transmitted to each connection will increase in magnitude with increasing reinforcement spacing, depth of connection below the crest of the wall and increasing magnitude of lateral earth pressure using the NCMA approach.

A methodology to quantify the connection capacity of any modular block-geosynthetic system was first introduced by Bathurst and Simac (1993) and subsequently adopted by the NCMA. The approach describes connection capacity using Mohr-Coulomb failure criteria for both peak (ultimate) load and serviceability (deformation) conditions. The model assumes that connection capacity varies with the magnitude of normal load transmitted across the connection. For the characterization of peak capacity:

$$T_{ultconn} = a_{cs} + N \tan \lambda_{cs} \tag{5}$$

and for serviceability design;

$$T_{sconn} = a'_{cs} + N \tan \lambda'_{cs} \tag{6}$$

where: $T_{ultconn}$ and T_{sconn} are connection capacities (kN/m); a'_c and a'_{cs} are minimum available connection capacities (kN/m); N is the normal load (kN/m); and λ_c and λ'_{cs} are equivalent friction angles (degrees). The service-

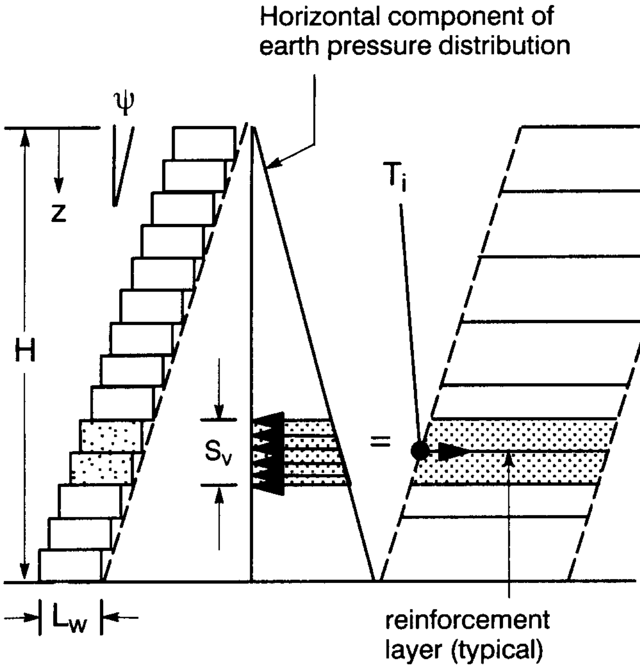


Fig 7. Calculation of tensile load, T_i, in a reinforcement layer (Note: S_v = contributory area of reinforcement layer)

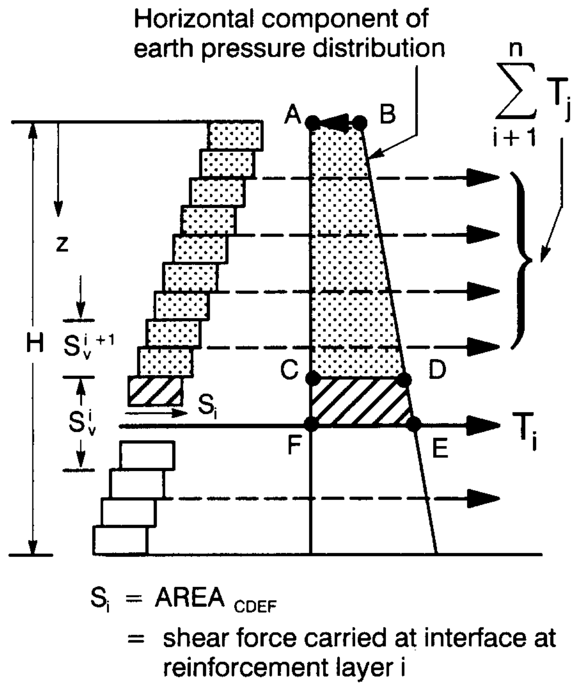


Fig 8. Calculation of interface shear force acting at a reinforcement elevation (n = total number of reinforcement layers)

ability connection capacity T_{sconn} is based on a connection capacity at a prescribed displacement of the reinforcement at the back of the connection (typically taken as 19 mm). Linear failure envelopes can be used over different ranges of normal load N for a particular connection system as described later in the paper.

9. INTERFACE SHEAR DESIGN

The discrete nature of the facing column requires that the shear load transmitted across each facing column interface not exceed the shear capacity of the interface. The approach adopted in current NCMA guidelines for statically loaded structures is to consider the facing column as a continuously supported beam with the reinforcement layers providing the reactions and the horizontal component of the earth pressure distribution taken as the distributed load. The reaction spacings are related to the contributory areas introduced earlier to partition distributed loads (earth pressures). The shear load (S_i) to be transmitted at an interface can be referenced to **Fig 8**. In current AASHTO (1997) interim guidelines this particular mode of failure is not addressed.

Similar to the trend in connection loads, the approach illustrated in **Fig 8** leads to larger interface shear loads transmitted across each facing unit with increasing reinforcement spacing, increasing depth of interface layer below the crest of the wall and increasing magnitude of lateral earth pressure.

The interface shear capacity for a segmental facing unit system can be conveniently described using a Mohr-Coulomb type law based on peak (ultimate) and serviceability design criteria. For the characterization of peak capacity:

$$V_u = a_u + N \tan \lambda_u \tag{7}$$

and for serviceability design;

$$V'_u = a'_u + N \tan \lambda'_u \tag{8}$$

where: V_u and V'_u are interface shear capacities (kN/m); a_u and a'_u are minimum available shear capacities

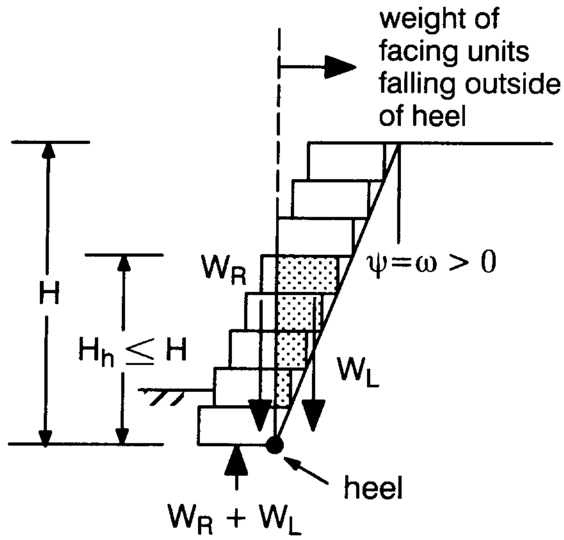


Fig 9. Hinge height concept (after Bathurst et al. 1993)

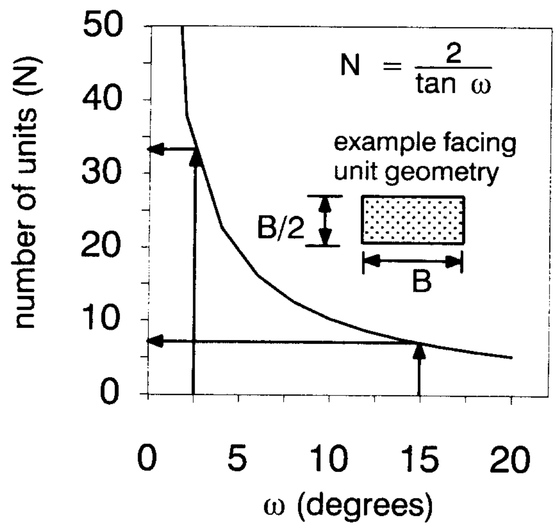


Fig 10. Influence of wall inclination on number of facing units within hinge height (after Bathurst et al. 1993)

(kN/m); N is the normal load (kN/m); and λ_u and λ'_u are the equivalent interface friction angles (degrees). Shear capacity envelopes based on a serviceability criterion are shear capacity values recorded after a prescribed relative displacement criterion.

The results of shear testing presented later in the paper show that the presence of a geosynthetic inclusion within the shear interface may reduce available interface shear capacity. Hence, the interface shear parameters used for design should be based on capacity envelopes that include the presence of the geosynthetic if this condition results in shear capacity envelopes that are lower that those for the block to block interface without a geosynthetic inclusion.

10. HINGE HEIGHT

Segmental retaining wall structures are constructed with facing units that routinely include a setback leading to a facing inclination angle $\psi > 0$. The effect of facing inclination is that the column weight above any interface may not correspond to the sum of the individual weights of the facing units above the reference elevation. A reduction in facing column weight will reduce the magnitude of the normal force N used to quantify connection and interface shear capacity in Equations 5 through 8. The effective height H_h of the facing column used to calculate the normal load N is called the *hinge height.* The hinge height is equal to the isolated height of stacked facing units that will just initiate toppling of the column in the direction of the retained soil mass and can be determined from moment equilibrium with respect to the heel of a dry-stacked column. The concept is illustrated in **Fig 9**. **Fig 10** 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. For vertical wall batters, small setbacks and/or low height walls the hinge height effect may not occur in which case the total height of column above the interface is used in facing stability calculations.

11. SUMMARY OF DESIGN CRITERIA FOR STATIC LOADING CONDITIONS

A summary of minimum factor of safety values for reinforced segmental retaining walls is given in **Table 1**. The factors of safety correspond to all potential failure mechanisms described earlier and have been taken from the NCMA (1997) guidelines. Factors of safety based on critical and non-critical structures (Simac et al. 1993) have been removed in the most recent guidelines by the NCMA. It is interesting to note that the current NCMA

Table 1: Recommended minimum factors of safety for design of geosynthetic reinforced soil segmental retaining walls (NCMA 1997).

Base Sliding	FS_{SLD}	1.5	Tensile Over-stress	FS_{OS}	1.0
Overturning	FS_{OT}	2.0	Pullout	FS_{PO}	1.5
Bearing Capacity	FS_{BC}	2.0	Facing Shear	FS_{SC}	1.5
Global Stability	FS_{GL}	1.3 – 1.5	Connection	FS_{CS}	1.5

guidelines consider overturning about the base of the structure as a potential failure mechanism. The most recent FHWA and AASHTO guidelines omit this mechanism because it has never been observed in practice. The NCMA was reluctant to follow the FHWA and AASHTO guidelines with respect to overturning since local toppling of the upper unreinforced facing column is a potential failure mechanism. In addition, many state engineers in the USA still prefer to carry out a base overturning stability calculation for geosynthetic reinforced structures since they are required to carry out a similar calculation for traditional gravity structures.

12. LABORATORY CONNECTION TESTING

A test methodology for connection testing was proposed by Bathurst and Simac (1993) and has been adopted by the NCMA as Test Method SRWU-1. At the time of writing the test protocol with minor changes is under ballot as an ASTM method of test.

A test apparatus that has been used by the author to carry out connection testing according to the NCMA protocol is illustrated in **Fig 11**. A brief description of the apparatus and test methodology is presented here.

The test apparatus allows tensile loads of up to 50 kN to be applied to the geosynthetic while it is confined between two block layers. The facing blocks are laterally restrained and surcharged vertically. The blocks are seated to engage any shear key or mechanical connector that is used to connect the blocks and create the facing

1 masonry concrete block
2 geosynthetic
3 loading platen
4 gum rubber mat/air bag
5 roller clamp
6 lateral restraining system
7 guide rail
8 extensometer clamp
9 surcharge actuator
10 loading frame
11 spacers
12 platform
13 wire-line extensometer
14 computer-controlled hydraulic actuator
15 load cell

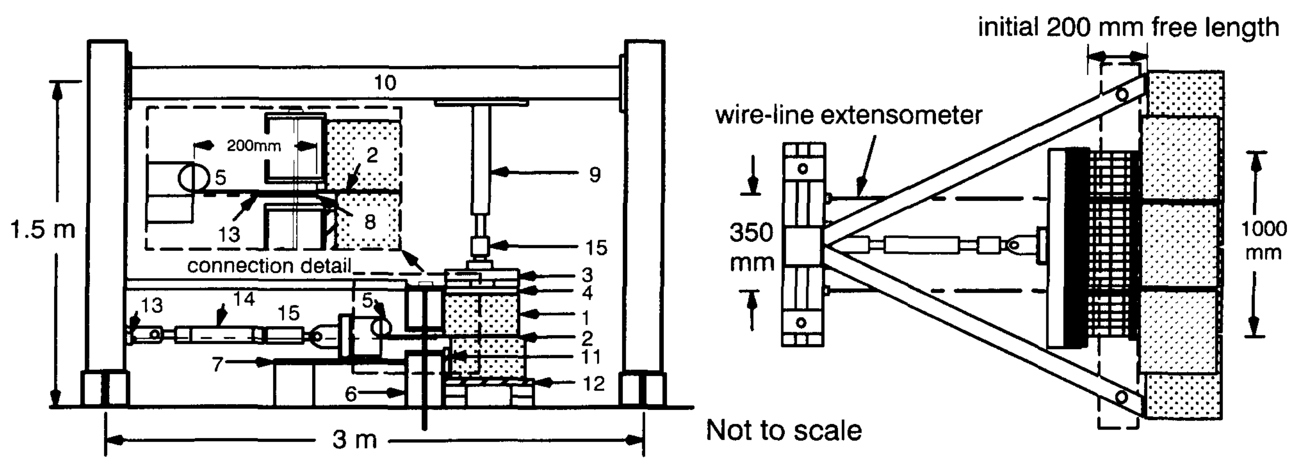


Fig 11. Schematic of RMC connection test apparatus showing typical masonry concrete block units and geosynthetic reinforcement (after Bathurst and Simac 1993)

unit setback. The hollow portions of each block and spaces between blocks are infilled with select granular fill and lightly compacted as specified for the field installation. Strips of geosynthetic reinforcement 1 m wide are attached to a roller clamp and the geosynthetic extended over the facing block. The next course is then placed over the geosynthetic layer simulating the running joint technique that would be used in the field. Two wire-line LVDT(s) are connected to the geosynthetic to measure geosynthetic displacement at the back of the block. Variable wall heights above the connection are simulated by placing one block course over the interface, infilling with a select stone (if required), and applying an additional surcharge load using the vertically-oriented hydraulic jack shown in **Fig 11**. Maximum surcharge loads equivalent to a 15 m high wall are possible with the apparatus used by the author. A gum rubber mat is placed over the top layer of blocks to ensure a uniform distribution of vertical surcharge pressure. The connection force is applied at a constant rate of displacement (20 mm/minute) using a horizontal computer-controlled hydraulic actuator. The load and displacements recorded by the actuator and the LVDTs are read continuously during the test by a microcomputer/data acquisition system. Each test is continued until there is a sustained loss in connection strength due to pullout or failure of the geosynthetic, or rupture of the modular blocks. The peak (ultimate) connection capacity and the connection load at a prescribed displacement measured at the back of the facing units is taken from the load-displacement trace for each test.

Following each test, the blocks are removed and the geosynthetic examined to confirm failure modes. A virgin specimen of geosynthetic is used for each test. The only variable in a series of connection tests is the magnitude of surcharge load.

The development of the test protocol described here and the reasons for selection of test details has been described in the papers by Bathurst and Simac (1993, 1997) and are summarized here.

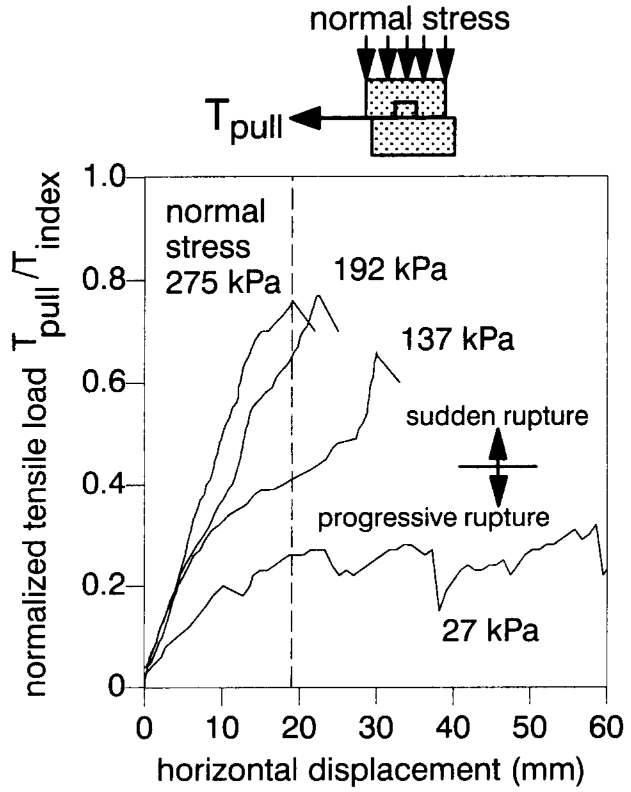
The specimen width of 1 m was selected to ensure that the effect of the running joints in a connection system is captured in the test procedure. Nominally identical tests carried out with only a single block above and below the reinforcement were shown to give higher connection capacities. Hence, tests that do not reproduce the running joint construction give connection capacity values that are non-conservative for design.

The 200 mm free length of reinforcement between the back of the connection and the 20 mm/minute rate of loading (i.e. 10% strain/minute) was selected to be consistent with the ASTM 4595 method of test that is routinely used as an index strength test for geosynthetic reinforcement products. It is convenient to imagine the connection test method described here as a large in-isolation tensile test carried out with one set of poor clamps. The difference in index strength determined from the ASTM 4595 method of test and peak connection capacity using our method can be directly attributed to the reduced efficiency of the connection formed by the block-geosynthetic system.

A displacement criterion is required to ensure that design connection capacity is not developed at the expense of unacceptable wall movement. A displacement (serviceability) criterion based on connection capacity at 19 mm displacement is recommended in the NCMA guidelines. However, as experience with modular block-geosynthetic reinforced soil wall structures performance increases, other displacement criteria may become appropriate.

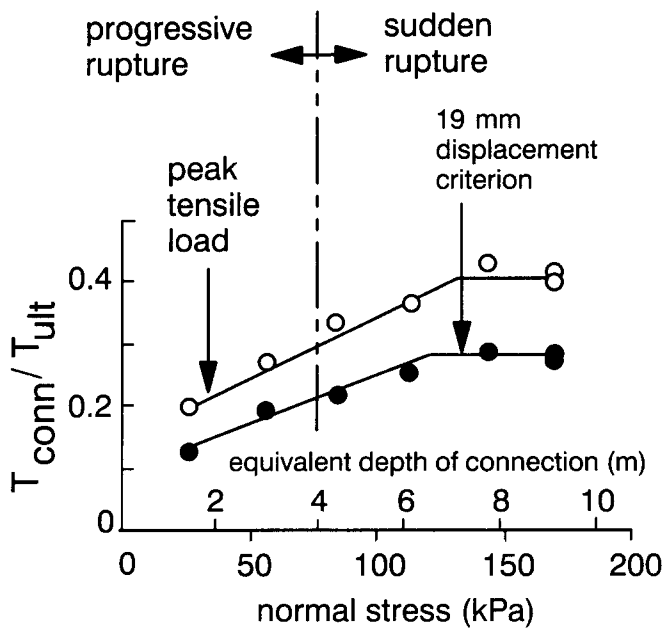
Example connection test results are illustrated in **Fig 12**. Connection loads have been normalized with respect to the in-isolation index tensile strength of the example geosynthetic (ASTM 4595). The data in the figure shows that for this particular system the peak connection strength may develop after 19 mm of displacement has occurred. This result is typical of most connection systems on the market today that derive connection capacity through a combination of geosynthetic-block interface friction (and geosynthetic-granular fill friction for infilled blocks) and interlock with shear keys or other forms of mechanical connectors.

Failure envelopes based on a different set of data are presented in **Fig 13**. The data shows that after a threshold normal load level there is no increase in connection capacity. This result is typical of most connection systems which derive a large portion of connection capacity through friction. Hence, performance-based failure envelopes are typically quantified using bi-linear failure envelopes with a maximum capacity cut-off. An explanation for this behavior is that there is a limiting efficiency of the connection that is imagined to be a poor "clamp". In comparison to the roller clamp that is used to apply the tensile load to the connection, the block system is not able to to distribute load to the geosynthetic reinforcement in a uniform manner. Localized load concentrations develop as the geosynthetic connection is tensioned, particularly in the vicinity of shear keys, mechanical pins and other points of contact. These load concentrations result in a tensile strength for the system that is less than



Strong uniaxial polyethylene geogrid in combination with solid concrete units (2m long) constructed with continuous concrete key

Fig 12. Example normalized connection load-displacement curves (after Bathurst and Simac 1993)



Medium strength woven polyester geogrid-300mm wide by 450mm long hollow core masonry concrete block (granular infill)

Fig 13. Example normalized connection strength curves (after Bathurst and Simac 1993)

that achieved for the nominally identical in-isolation tension test that distributes applied loads more evenly to the geosynthetic.

The connection performance is also influenced by the type of geosynthetic. For example, a relatively inflexible geogrid with a thick vertical cross-section profile may not be able to conform to the interface geometry of block systems with sharp shear key geometries whereas a flexible geogrid or geotextile is better able to conform to this geometry. However, large percentages of index strength have been measured by the author using continuously formed rigid geogrids in combination with specially designed connection rakes that match individual geogrid apertures and are inserted into the top of specially molded concrete facing units. An example of this type of connection has been reported by Austin and Martin (1996).

Fig 14 shows a summary of more than 1200 connection test results carried out on more than 200 different concrete block-geosynthetic systems available on the market today. The blocks ranged from solid dry-cast masonry units to hollow masonry units filled with a compacted uniformly graded crushed stone infill (top size of 19 mm). The geosynthetics included almost all types of geosynthetics that are routinely used in reinforced soil wall systems (i.e. flexible woven geogrids, stiff continuous biaxial and uniaxial geogrids and woven polypropylene and polyester geotextiles). The figure shows that peak connection capacity varies over a large range. Despite attempts by the author and co-workers to characterize connection performance according to type of connection (i.e. type of block and type of geosynthetic) there is no general trend in the data set. Indeed, within a particular product line of geosynthetic reinforcement in combination with a specific block type there is often no consistent pattern to normalized connection capacity. However, as a general rule, connection capacity at a given normal load does increase with geosynthetic index strength when the geosynthetic products being compared are of the same construction type and from the same manufacturer. These observations highlight the need to perform product-specific testing of any candidate connection system and the need to quantify connection

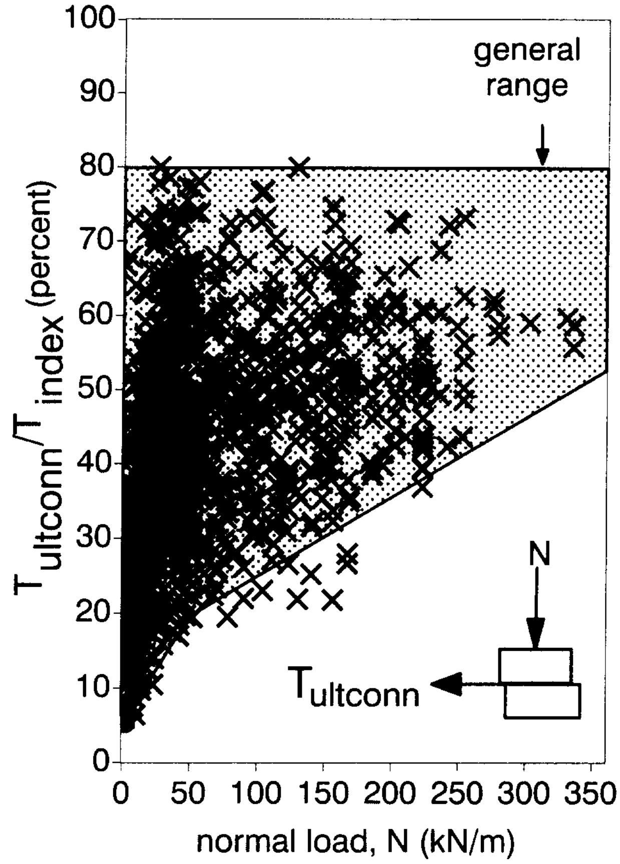


Fig 14. Summary of connection test data (peak load capacity) (Bathurst and Simac 1997)

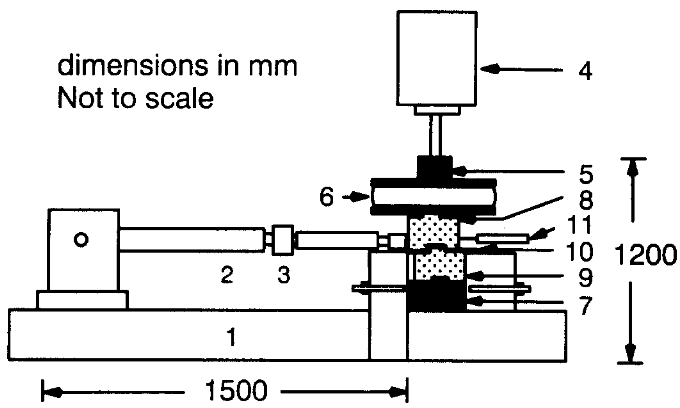


Fig 15. Schematic of interface shear test apparatus (with geosynthetic inclusion) (after Bathurst and Simac 1994)

capacity in a generic manner using Mohr-Coulomb failure envelopes of the type introduced earlier.

13. LABORATORY SHEAR TESTING

A test protocol for interface shear testing has been developed by the author and co-workers and has been adopted by the NCMA as Test Method SRWU-2. The test apparatus used by the author to perform the tests according to the NCMA method is illustrated in **Fig 15**. A brief description of the apparatus and test methodology is presented here.

The test apparatus allows horizontal loads of up to 160 kN to be applied across the interface between two block layers. The segmental units are laterally restrained at the bottom and surcharged vertically. Tests can be performed with or without the presence of a geosynthetic inclusion. A centrally located running bond (joint) is formed by placing a single block over the joint between two underlying units. This arrangement simulates the staggered construction procedure used in the field. Variable wall heights above the interface are simulated by applying additional normal load using the air bag arrangement shown in **Fig 15**. The blocks are seated to engage any shear key or mechanical connector that is used to connect the blocks and control the facing unit setback. The hollow portions of each block and spaces between blocks are infilled and lightly compacted with a select granular aggregate as specified for the field installation. All blocks are visually inspected prior to testing to confirm that they are free of defects.

The horizontal (shear) force is applied at a constant rate of displacement using a computer-controlled hydraulic actuator. The load and displacements are measured by the actuator and displacement transducers and recorded continuously during the test by a microcomputer/data acquisition system. Each test is continued until large shear displacements are achieved, usually as a result of failure of the concrete shear key or other mechanical connection system. The interface shear load recorded at peak capacity (ultimate failure) and after a prescribed relative displacement of the top block is used to quantify shear capacity of the system.

Following each test, the blocks are removed and the units examined to confirm failure modes. In order to minimize the number of blocks used in a test series, blocks used for the lower courses are re-used as the top layer block in subsequent tests. However, the interface along which shear occurs is initially undamaged in each test. In addition, when interface shear testing with a geosynthetic inclusion is undertaken, a virgin specimen of geosynthetic is used in each test.

The test method described here is simply a direct shear test. While the connection test protocol described earlier requires that at least one running joint be constructed this is not a practical arrangement for interface shear testing because of the difficulty of applying a uniform horizontal load to more than one facing unit. In addition, many of the facing units available on the market today have very efficient concrete shear keys that can develop very large shear capacities and hence would be difficult to fail.

A displacement criterion of 2% of the height of the facing unit is recommended in the NCMA guidelines to ensure that interface shear capacity is achieved after a minimum amount of relative movement between layers. Based on available wall performance data (Bathurst and Simac 1994) a maximum out-of-vertical alignment of 2% of the wall height is a reasonable value that ensures that active earth pressures can develop in the retained soils without visual distortion of the facing column. For some systems with integrally cast continuous shear keys and matching slot construction the 2% displacement criterion is not achieved prior to peak shear capacity.

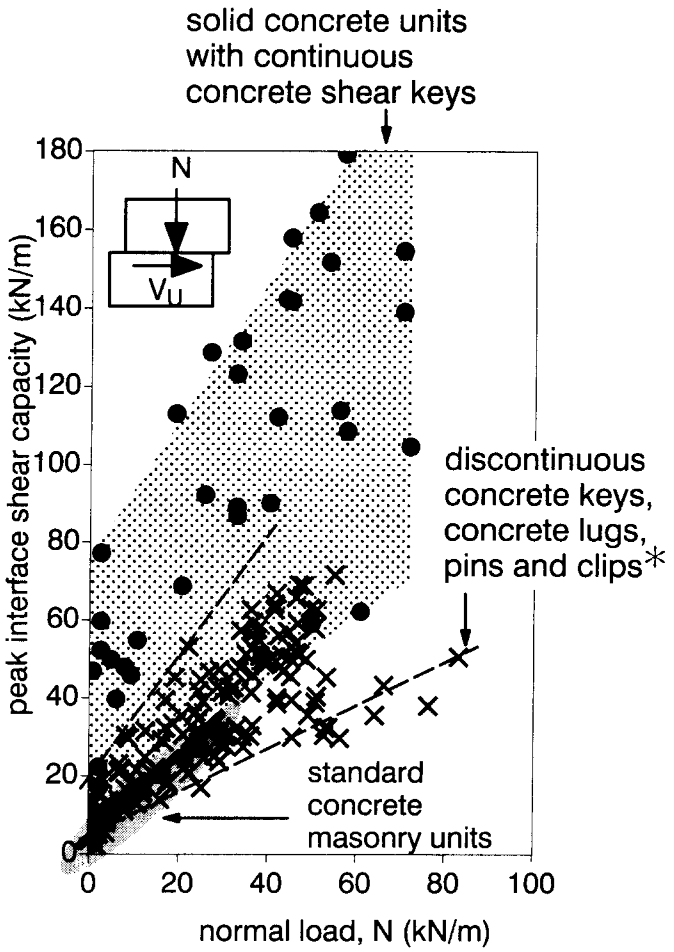
Similar to observations made with respect to connection capacity, interface shear capacity varies widely between different block systems on the market today. Examples of peak interface shear data (without a geosynthetic layer inclusion) are illustrated in **Fig 16.**The data in the figure is taken from tests on 16 different segmental retaining wall units that are on the market today. The test results have been broadly grouped based on the type of connector that is used to achieve wall batter and to develop a portion of shear capacity. Also included in the figure are standard masonry concrete units (190 mm high by 190 mm toe to heel by 390 mm long) that were infilled with a crushed stone. These standard masonry units do not have any type of shear connector and consequently gave some of the lowest shear capacities of all the units tested, particularly at low normal (surcharge) loads. The largest shear capacity values recorded were for large (i.e. wide toe to heel dimension) solid concrete units with one or more parallel continuous integral concrete shear keys.

The connection formed with most segmental facing unit systems on the market today is created by extending the geosynthetic reinforcement layer across the interface to the front of the facing column. It can be expected that the presence of the geosynthetic layer at the interface will modify the shear capacity between block units.

The results of interface shear testing of a typical hollow infilled masonry concrete unit are illustrated in **Fig 17**. Shear capacity data is shown with different geogrid products placed in the interface and for nominally identical tests without a geogrid layer inclusion. The data shows that the presence of the geosynthetic may increase *or* decrease interface shear capacity from values for the block to block interface alone. As a general observation, the inclusion of a relatively rigid, thick cross-section geogrid will reduce the shear capacity of block systems. This can be explained by the reduction in shear key engagement caused by the geogrid inclusion and lower interface friction values for smooth polymeric surfaces in contact with concrete surfaces when compared to rough concrete to concrete surfaces alone. However, for systems that rely on concrete shear keys, the presence of a flexible geosynthetic inclusion may not reduce interface shear capacity and in some cases may *increase* shear capacity as illustrated in **Fig 17**. The explanation for this observation is that a flexible woven geogrid (or woven geotextile) can act as a cushion to minimize point loads that can develop against the shear keys. These point loads are the result of concrete surface asperities and minor variations in block dimensions and alignment. **Fig 17** also shows that shear capacity envelopes may be best described in some instances using bi-linear curves that include a maximum shear capacity cut-off at some threshold magnitude of shear capacity V_u.

14. INTERPRETATION OF CONNECTION TEST RESULTS FOR DESIGN

Connection capacities determined from laboratory performance tests must be factored down for design to account for the effects of creep, chemical and biological degradation. The approach adopted by the NCMA



* includes solid concrete units and hollow masonry units infilled with a uniform 19 mm size crushed stone

Fig 16. Peak interface shear capacity versus normal load for selected modular block units without a geosynthetic inclusion (from Bathurst and Simac 1997)

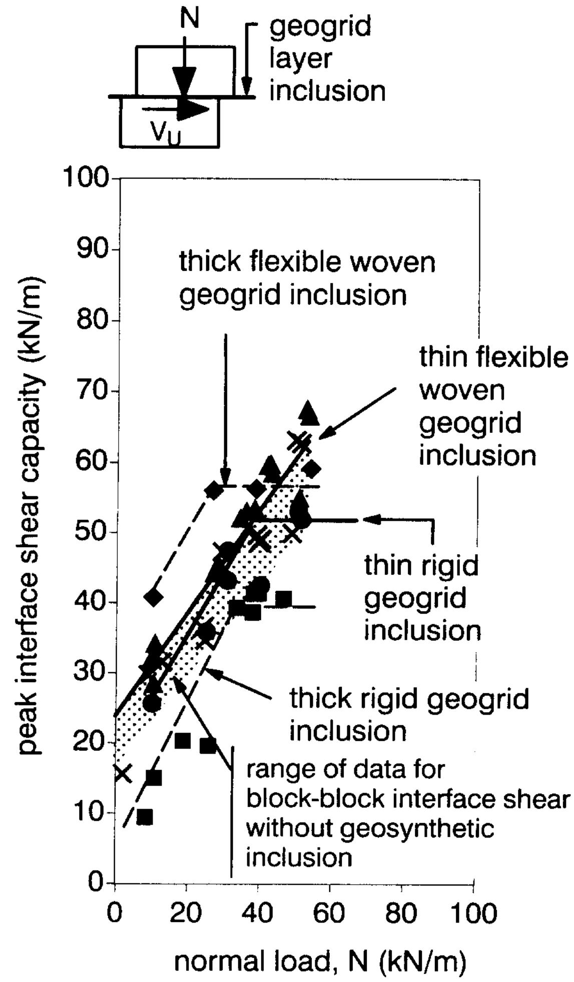


Fig 17. Example peak interface shear capacity versus normal load data showing influence of geogrid inclusion (hollow masonry unit with discontinuous concrete key and 19 mm uniform size crushed stone infill) (from Bathurst and Simac 1997)

(1997) to calculate the long-term connection strength T_{ac} for design is expressed by the following equation:

$$T_{ac} = \frac{T_{ultconn}}{FS} \leq T_{sconn} \tag{9}$$

where FS = 1.5 is a lumped reduction factor to account for chemical and biological degradation of the reinforcement.

Some confusion in terminology exists in the current AASHTO (1997) interim guidelines and FHWA (Elias and Christopher 1997) guidelines regarding the interpretation of T_{sconn}. This value is referred to as the connection load at *pullout* of the geosynthetic. Pullout implies that the geosynthetic pulls out of the connection while remaining intact. Based on a large number of tests by the author, pullout of intact geosynthetic specimens occurs only under conditions of very low normal loads applied to systems with no pins or clips to penetrate the geosynthetic or systems with a low profile concrete shear key or no connection system at all (e.g. standard concrete masonry units). In practice, peak connection capacity of the system typically occurs due to complete and sudden rupture of the geosynthetic *or* progressive rupture of the geosynthetic as it tears at localized points of contact with the block and/or granular infill in the connection (i.e. at concrete shear keys, pins or clips, or the geosynthetic structure ruptures at points of contact with the granular infill). An exception to this general observation are block systems with a smooth concrete interface. Very few of these concrete block systems exist on the market today.

In the NCMA approach developed by the author and co-workers, the value of T_{sconn} is the connection capacity based on a serviceability (displacement) criterion (typically 19 mm) measured at the back of the connection and is used to ensure that the mobilization of connection capacity is not developed at the expense of excessive extension of the geosynthetic within the connection interface. By adopting a displacement-based load criterion the particular type of failure mechanism does not enter into the determination of serviceability failure envelopes for design.

15. INTERPRETATION OF INTERFACE SHEAR TEST RESULTS FOR DESIGN

Current AASHTO and FHWA design guidelines for reinforced segmental retaining walls do not consider interface shear failure mechanisms. The NCMA manual uses the following equation to calculate minimum acceptable design interface shear capacities:

$$S_{design} = \frac{V_u}{FS} \leq V'_u \qquad (10)$$

where $FS \geq 1.5$ and V'_u is the interface shear capacity based on a serviceability criterion (i.e. relative displacement equal to 2% of the height of the unit).

16. COMPUTER AIDS

The analysis and design steps described in the paper lend themselves to solution using computer programs. A number of proprietary computer packages are available from manufacturers of modular block systems and the suppliers of geosynthetic reinforcement materials. A generic software package called SRWall (ver 2.0) is available from the NCMA that is a full generic implementation of the current NCMA (1997) methodology (Bathurst and Simac 1995). The program has been developed for the Microsoft Windows operating system. A typical graphics user interface (GUI) from the program is illustrated in **Fig 18.**

17. CONCLUDING REMARKS

The paper has focused on recent developments in the design, analysis and performance testing of geosynthetic reinforced soil retaining walls that employ modular concrete units as the wall facing system. The recommendations for routine structures contained in this paper have been adopted by the NCMA. The design strategies reported here are generic in nature and consider all potential modes of failure for these systems. Limit-equilibrium based methods of analyses together with Coulomb active earth pressure theory are key features of stability calculations and are consistent with the conventional approach used by geotechnical engineers to design retaining wall structures in North America. The paper extends the general approach used in earlier FHWA and AASHTO guidelines to examine facing instability modes of failure not considered in older design methodologies. Facing stability calculations related to facing connection performance and interface shear allow the designer to quantify performance differences between nominally identical geosynthetic reinforced soil retaining wall systems built with different modular facing units.

Experience with the design of structures that are fully compliant with NCMA guidelines has shown that the combination of hinge height concept, interface shear capacity and facing connection requirements, controls ver-

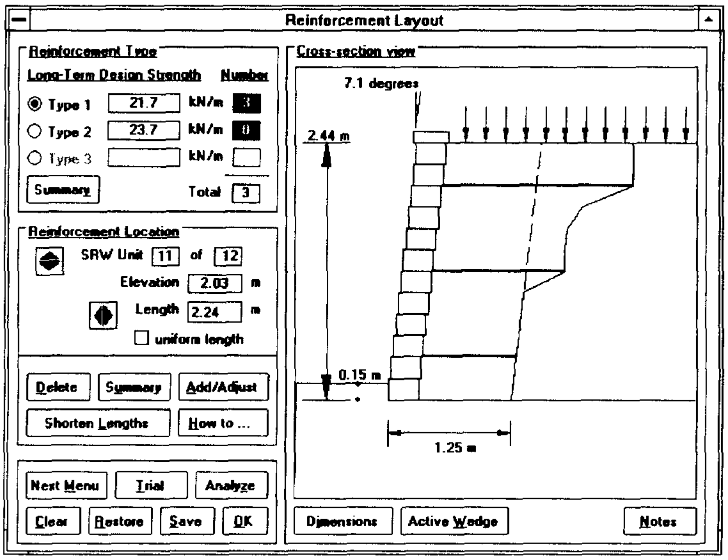


Fig. 18 Program SRWall.

tical reinforcement spacing and results in designs with multiple layers of relatively low strength reinforcement as opposed to a lesser number of stronger layers. This result is desirable from the point of view of creating a composite facing-reinforced soil mass with redundant reinforcement elements.

To date, the majority of structures have been constructed with geogrid reinforcement materials. However, the results of connection tests carried out by the author using woven geotextiles shows that their connection strength is comparable to that of geogrid materials used in similar applications. The generally cheaper price of geotextiles will undoubtedly lead to their more frequent use as the reinforcement material in future soil reinforced segmental retaining wall structures.

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