Geosynthetic Reinforced Segmental Retaining Wall Structures in North America

Richard J. Bathurst
Department of Civil Engineering
Royal Military College of Canada
Kingston, Ontario, Canada K7K 5L0

Michael R. Simac
Earth Improvement Technologies
100 Mayflower Court
Cramerton, NC 28032 USA

ABSTRACT: The Paper reviews recent research on the analysis, design and construction of geosynthetic reinforced soil retaining walls that use dry-stacked modular concrete units as the facing system (geosynthetic reinforced segmental retaining walls). These systems have gained wide popularity in North America for reasons of performance, aesthetics, cost and expediency of construction. However, the discrete nature of these modular block systems requires that special attention be paid to the design and construction of the facing elements. Some consequences of the extension of limit-equilibrium (pseudo-static) methods to the stability of segmental retaining wall structures are reviewed. Design methodologies, construction and specification recommendations described in this paper for routine structures have been recently adopted by the National Concrete Masonry Association (NCMA).

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 (e.g. Crowe et al. 1989, Anderson et al. 1991, Berg 1991, Simac et al. 1991, Hill and Berg 1993, Kemp et al. 1993). Examples of completed projects are illustrated in Figure 1, 2 and 3. The National Concrete Masonry Association (NCMA) in the USA has recently adopted the term *Soil-reinforced Segmental Retaining Wall* (SRW) to identify this type of retaining wall system.

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 exper-

Fig. 1 First major geosynthetic reinforced soil segmental retaining wall structure built in Canada (7 m height) (Crowe et al. 1989).

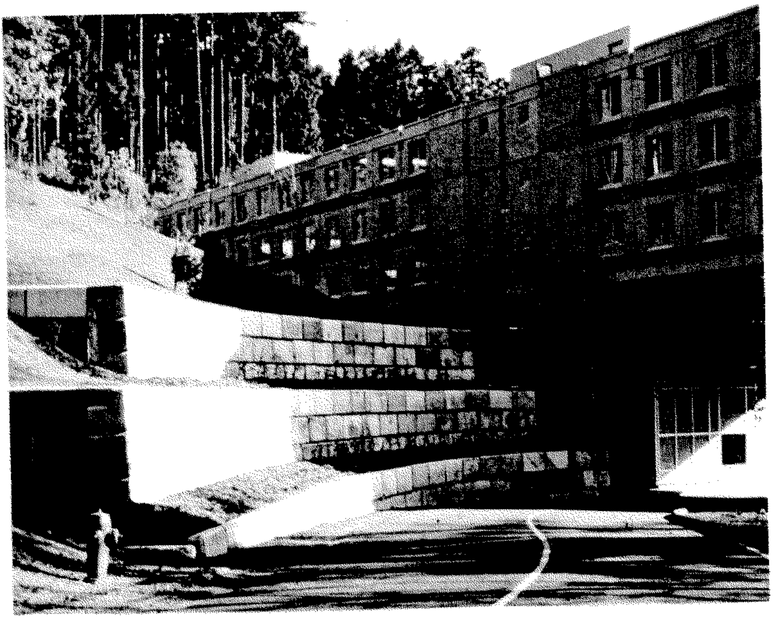


Fig. 2 Tiered geosynthetic reinforced soil segmental retaining wall with 0.75m x 0.75m x 1.5m solid concrete units (Bowden 1991).

ienced installation crew of three or four persons can typically erect 20–40 square meters 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 (Berg 1991, Simac et al. 1991, Anderson et al. 1991, Geotechnical Fabrics Report 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.

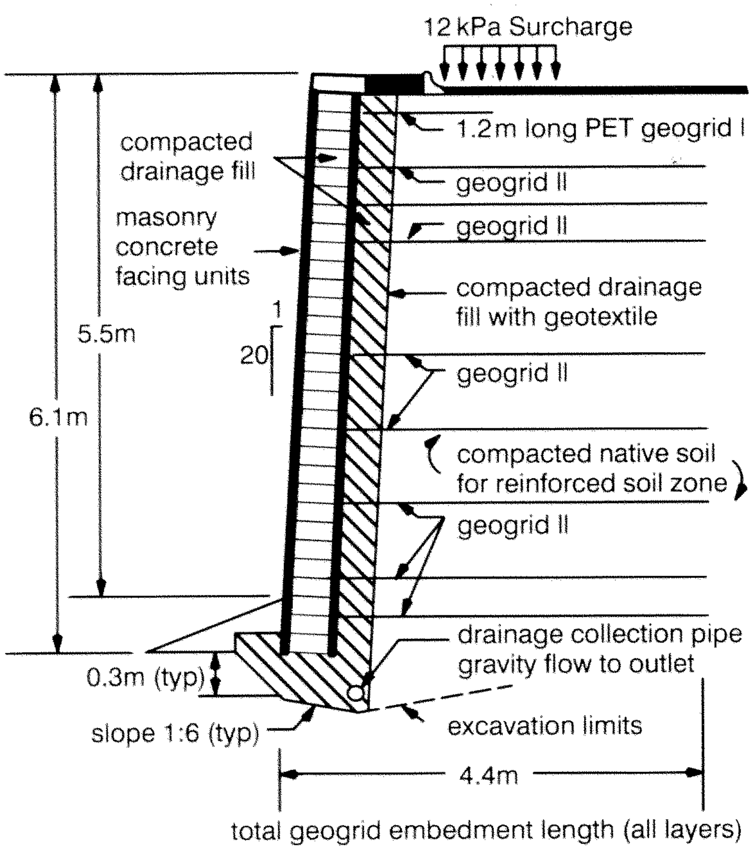


Fig. 3 Typical geosynthetic reinforced soil segmental retaining wall cross-section (after Simac et al. 1991).

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 recent research on the analysis, design, construction and specification of geosynthetic reinforced soil retaining walls that use dry-stacked modular concrete units as the facing system. Design methodologies and construction recommendations described in this paper for routine structures have been recently adopted by the National Concrete Masonry Association (NCMA) in the USA (Simac et al. 1993a). 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 Figure 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. 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

Table 1: Specifications for segmental retaining wall masonry units (NCMA 1991, ASTM C 55).

Minimum Compressive Strength (MPa)	Maximum Water Absorption (%)	Dimensional Tolerances (mm)
20.7	6–7*	3

* Minnesota Department of Transport (Hill and Berg 1993)

layers under low normal pressures (e.g. close to the wall crest) a significant portion of shear transfer may be developed by mechanical connectors.

Specification limits for dry-cast masonry concrete blocks in retaining wall applications are given in Table 1. The physical requirements with respect to mix design can be found in separate publications by the NCMA (1991) 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. The compressive strength of masonry concrete units meeting the minimum value in Table 1 is more than adequate from a structural point of view. However, a minimum compressive strength of 40 MPa has been required by at least one state department of transportation in the USA and this strength has been achieved 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.

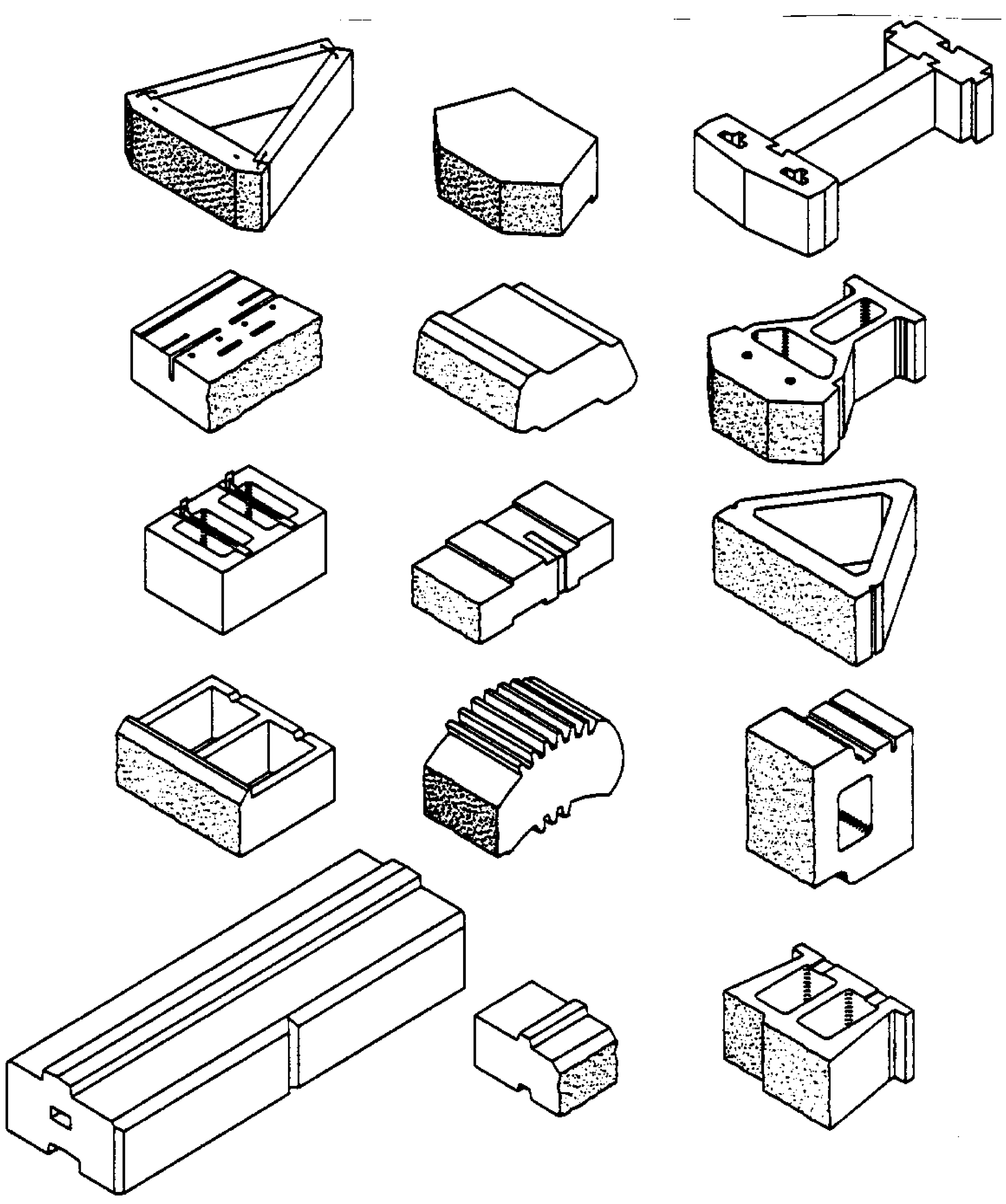


Fig. 4 Examples of segmental retaining wall units.

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) (Christopher et al. 1989); the American Association of State Highway and Transportation Officials (AASHTO 1990a,b; 1992); and the National Concrete Masonry Association (NCMA) (Simac et al. 1993). In Canada, guidelines based on the FHWA and AASHTO documents are found in the 3rd Edition of the Canadian Foundation Engineering Manual (1992). 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 NCMA guidelines and the earlier FHWA and AASHTO references can be found in papers by Bathurst et al. 1993a and Simac et al. 1993b. The NCMA guidelines are essentially a refinement of earlier FHWA and AASHTO guidelines. The FHWA and AASHTO guidelines were developed based on experience with geosynthetic reinforced soil retaining wall systems that used primarily precast concrete panels. 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 authors and co-workers (Simac et al. 1993a,b; Bathurst et al. 1993a).

4 MODES OF FAILURE

Potential failure modes for reinforced segmental retaining wall structures are illustrated in Figure 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 Figure 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. Christopher et al. 1989). Modes of failure that require special considerations in reinforced segmental retaining wall design and analysis are illustrated in the last four diagrams of the figure. The reinforcement layers are placed between the masonry units to form an essentially frictional 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 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 Figure 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 choice of Rankine or Coulomb earth pressure theory varies between guidelines. The Coulomb approach has been adopted by the authors 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). In addition, other research has demonstrated that Rankine theory overestimates lateral pressures acting within instrumented geosynthetic reinforced soil retaining wall structures (e.g. Berg et al. 1986, Bathurst and Simac 1991). 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 in-

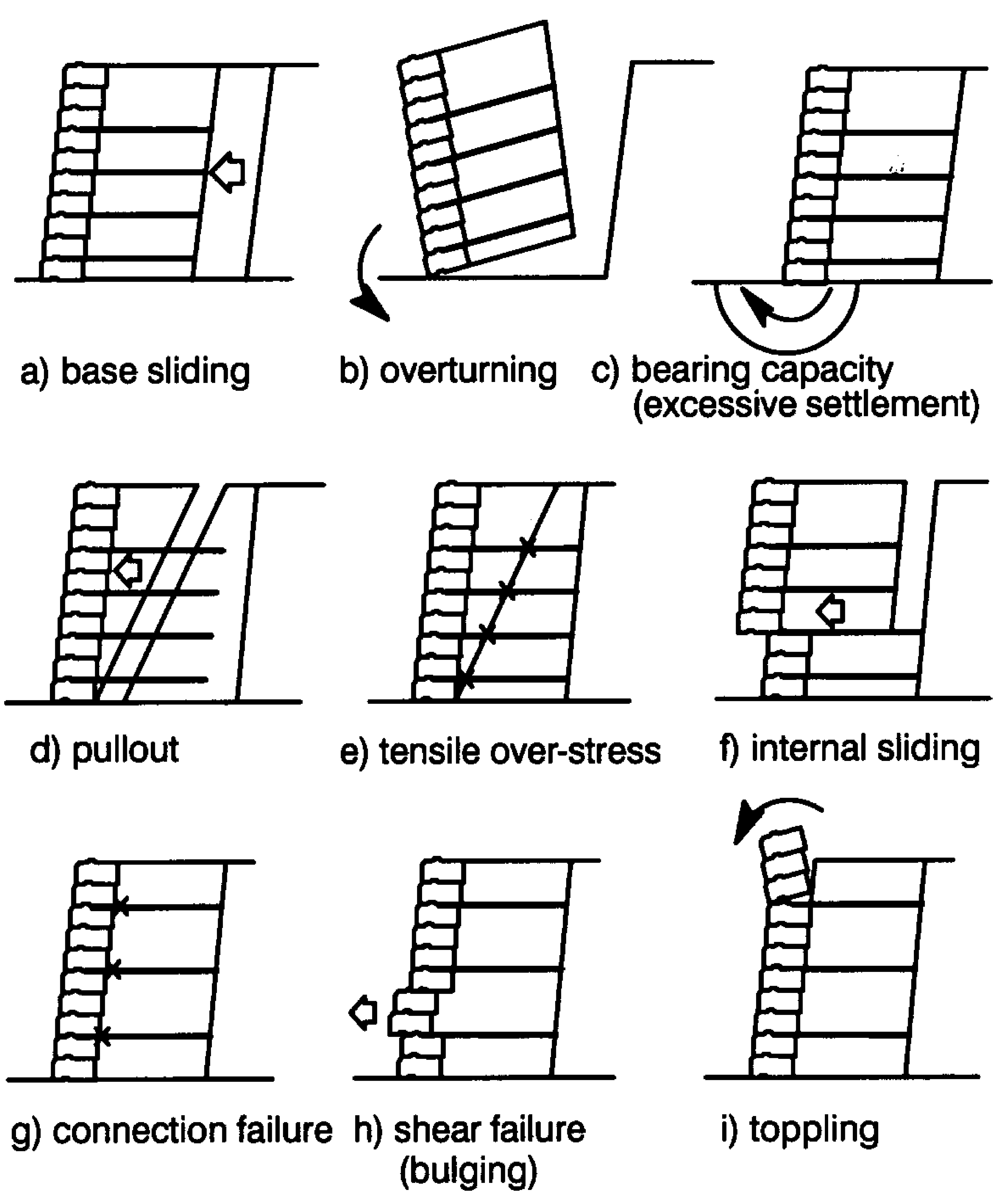


Fig. 5 Modes of failure: external (top row); internal (middle row); facing (bottom row) (Simac et al. 1993a).

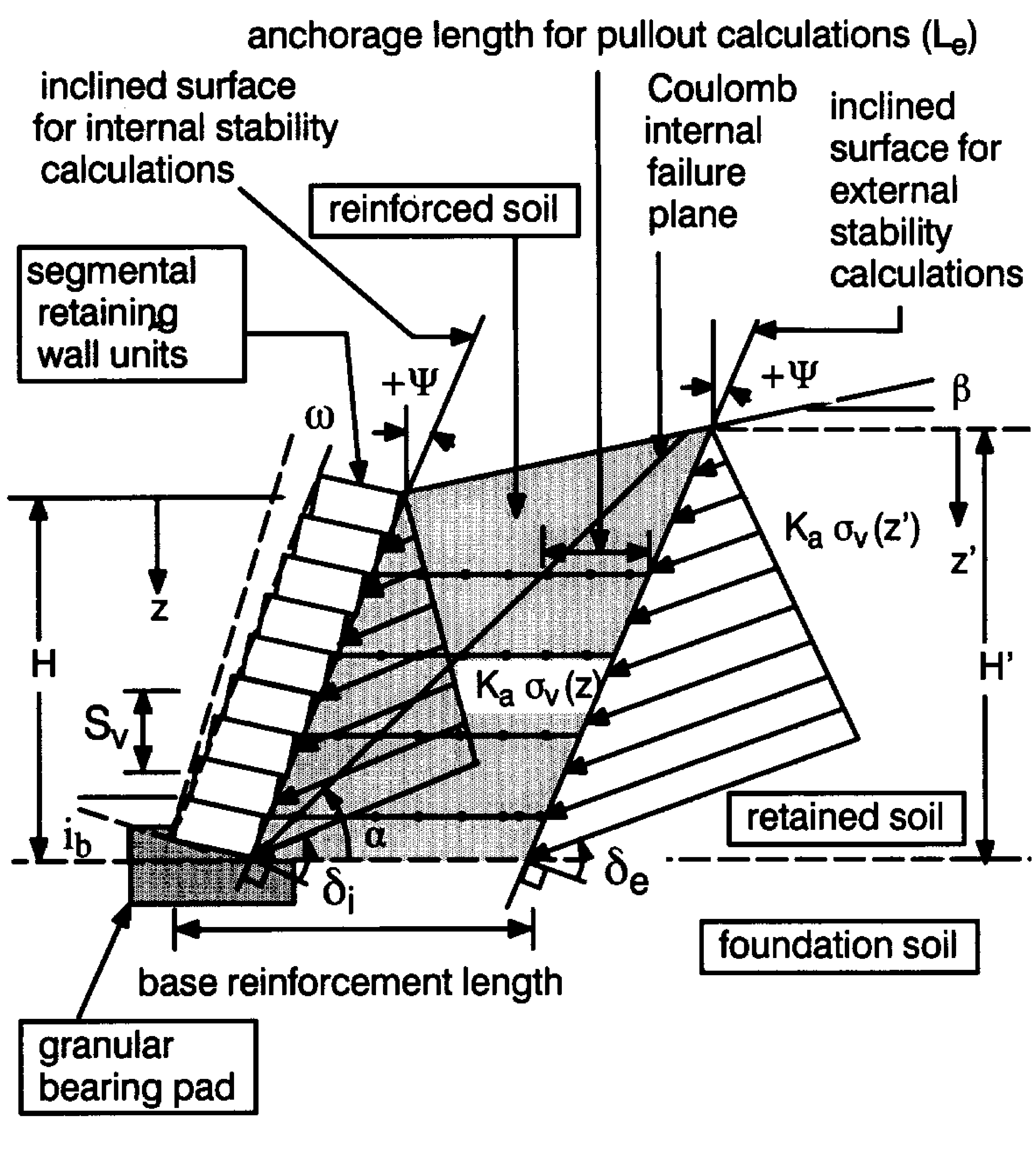


Fig. 6 Principal components, geometry and earth pressures assumed in NCMA method (Bathurst et al. 1993a).

terface shear angle acting between the inclined surface (ψ) and the reinforced soil is taken as $\delta_i=2\phi/3$. This general approach is consistent with conventional retaining wall design but is slightly more conservative than in AASHTO (1992) guidelines that recommend $\delta_i=3\phi/4$ for granular soil in contact with prefabricated modular concrete panels. 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 assumption simplifies calculations and results in the conservative assumption that the vertical component of earth pressures does not contribute to resisting forces in stability calculations. The influence of the choice of Rankine theory (equation 1) or Coulomb theory (equation 2) on the magnitude of horizontal earth pressure is illustrated in Figure 7. Coulomb earth pressures are 15% to 45% lower than those calculated by Rankine theory for typical segmental retaining wall geometries. 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 Figure 6) 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 (Figure 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 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 are shallower than those calculated using the Rankine solution as illustrated in Figure 8. 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. However, the NCMA method does not require that all reinforcement layers have the same length as required in the AASHTO documents. 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 non-critical structures (see Table 2). The designer is permitted 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.

6 CALCULATION OF ALLOWABLE TENSILE LOAD AND ANCHORAGE CAPACITY

The calculation of long-term design strength (LTDS) in North America for reinforced soil structures 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

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

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

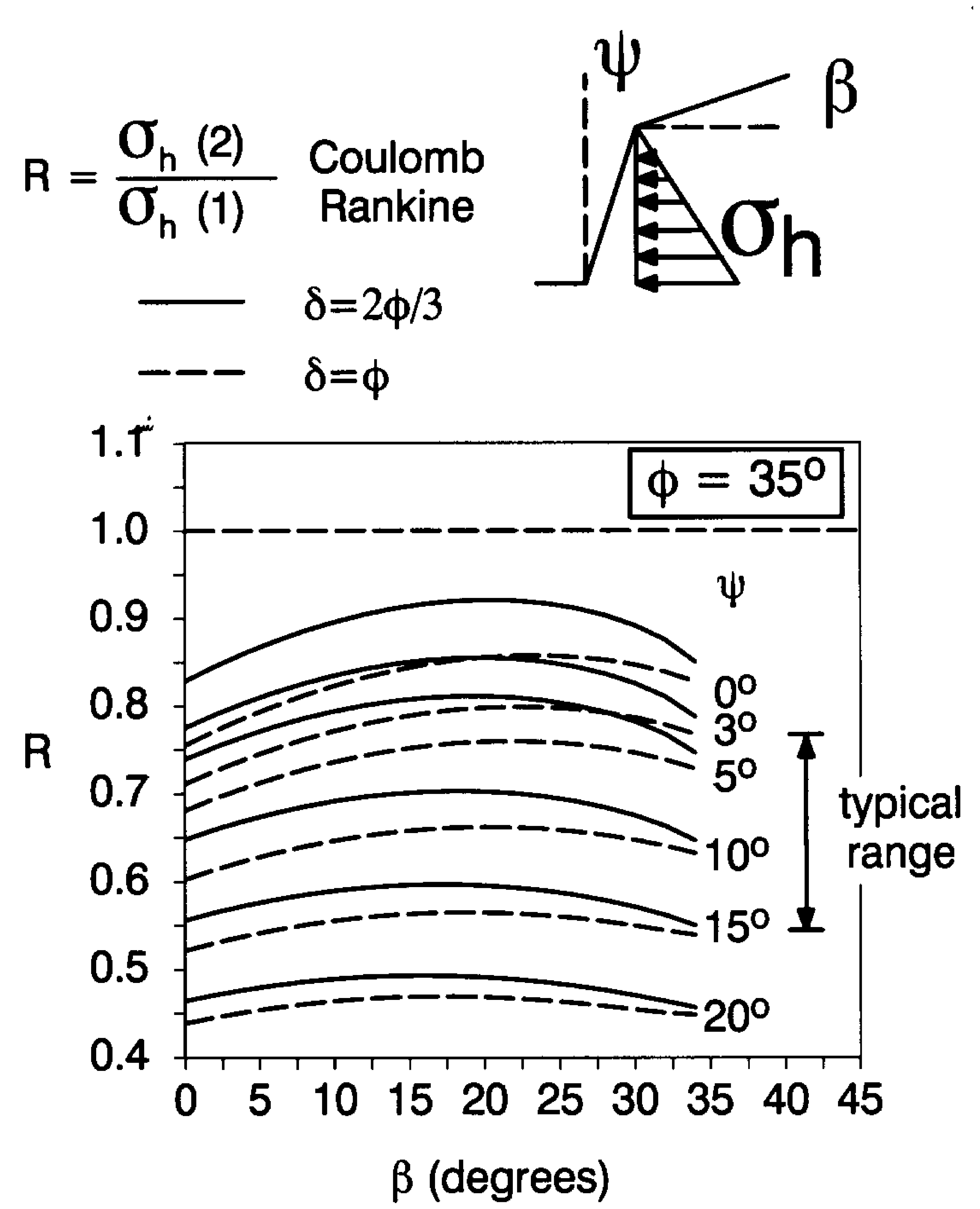


Fig. 7 Ratio of Coulomb solution to Rankine solution for the calculation of horizontal earth pressures (Bathurst et al. 1993a).

retaining wall structures are equally applicable to segmental wall design. A brief description of internal stability calculations follows for completeness. The reader is referred to the references at the end of the paper for a more thorough treatment of the topic and for details of the variations to the general approach illustrated here.

The following expressions can be used to calculate the maximum allowable strength (T_a) for a reinforcement layer and are adapted from FHWA guidelines (Christopher et al. 1989, Berg 1992) and Geosynthetic Research Institute (GRI) standards:

$$LTDS = \frac{T_{index}}{FS_{CR} \times FS_{ID} \times FS_{CD} \times FS_{BD} \times FS_{MU}} \quad (6)$$

$$T_a = LTDS/FS_{OS} > T_{max} \quad (7)$$

Here T_{index} refers to the minimum average roll value from

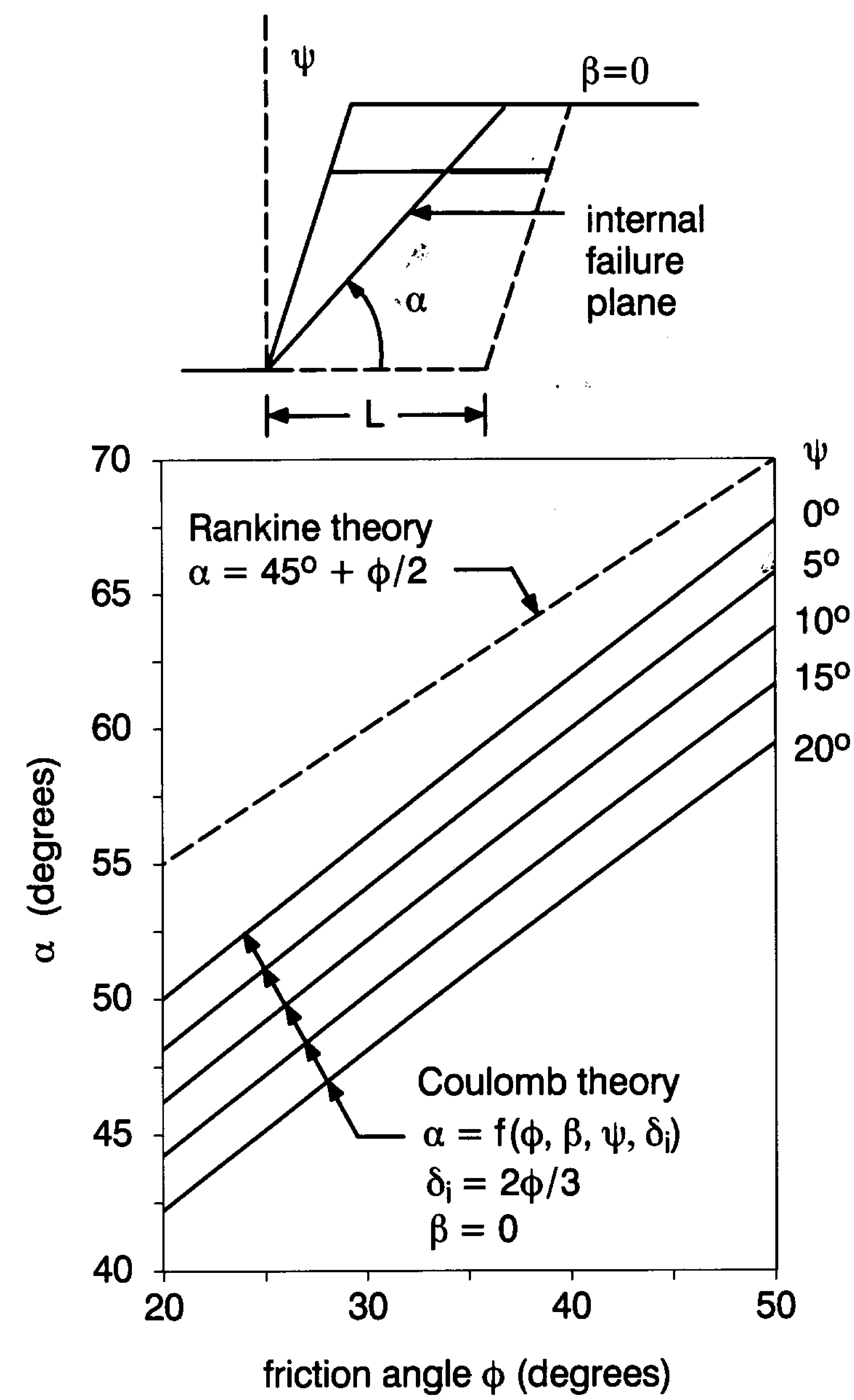


Fig. 8 Comparison of internal failure plane orientation based on Rankine and Coulomb earth pressure theory (Bathurst et al. 1993a).

wide width strip tests (ASTM D 4595). Partial factors of safety are defined as: FS_{CR} = partial factor of safety for creep deformation (ratio of T_{index} to creep limiting strength taken at 10% of in-isolation strain); FS_{ID} = partial factor of safety for installation damage; FS_{CD} = partial factor of safety for chemical degradation; FS_{BD} = partial factor of safety for biological degradation; FS_{MU} = partial factor of safety for material uncertainty (1.5 minimum).

Pullout resistance (R_{PO}) and the factor of safety against anchorage failure (FS_{PO}) are calculated as:

$$R_{PO} = 2\, L_e\, C_i\, \sigma_v \tan \phi \quad (8)$$

$$FS_{PO} = R_{PO}/T_{max} \quad (9)$$

where: L_e = anchorage length beyond the internal failure plane (Figure 6); C_i = coefficient of shear stress interaction; σ_v = average vertical stress acting over the geosynthetic in the anchorage zone. An alternative but more complicated equation for geosynthetic reinforcement pullout capacity has been reported by Christopher et al. (1989). However, equation (8) offers simplicity with no proven loss in accuracy. Minimum factors of safety against tensile over-stress (FS_{OS}) and anchorage failure (FS_{PO}) are given in Table 2 based on both peak load and deformation criteria.

7 FACING STABILITY

The discrete nature of the dry-stacked modular column construction in reinforced segmental retaining wall design requires performance data and analytical models that are unique to these systems. Calculation of interface shear capacity and connection capacity developed between the modular units and the polymeric reinforcement requires that an estimate of the normal stress transmitted between stacked units be made. The magnitude of normal stress will be dependent on: the height of wall above any interface; the wall inclination; and the magnitude of vertical load transmitted to the column by wall-soil shear over the design life of the structure. A lower bound estimate of normal load can be made by calculating the *hinge-height* of the unsupported facing column. The calculation is related to the maximum number of units to satisfy moment equilibrium about the heel of the column. The general expression for hinge height H_e is:

$$H_e = 2[(W_u - G_u - 0.5\, H_u \tan i_b) \cos i_b]/\tan(\omega + i_b) \quad (10)$$

where: W_u is the width of the modular unit; G_u is the distance to the centre of gravity of the unit measured from the front face; H_u is the height of the unit; i_b is the base inclination angle; and ω is the wall batter. The concept is illustrated in Figure 9 which shows that for a typical solid unit with a block width to height ratio of two, the number of units corresponding to the hinge height diminishes rapidly with increasing wall inclination. Clearly, as the wall batter $\psi = \omega + i_b \rightarrow 0$ the hinge height will be restricted to the height of the column above the interface.

7.1 Interface shear capacity

In order to satisfy minimum factors of safety against block to block sliding, the shear capacity of any interface between units must be determined. The calculation of required shear capacity is based on a continuously supported beam analog in which the lateral earth pressure is taken as the distributed load and the reinforcement layers as the supports. Unit to unit interface sliding also contributes to the resistance to internal sliding as illustrated in Figure 5f. The magnitude of shear capacity available at the interface of concrete units

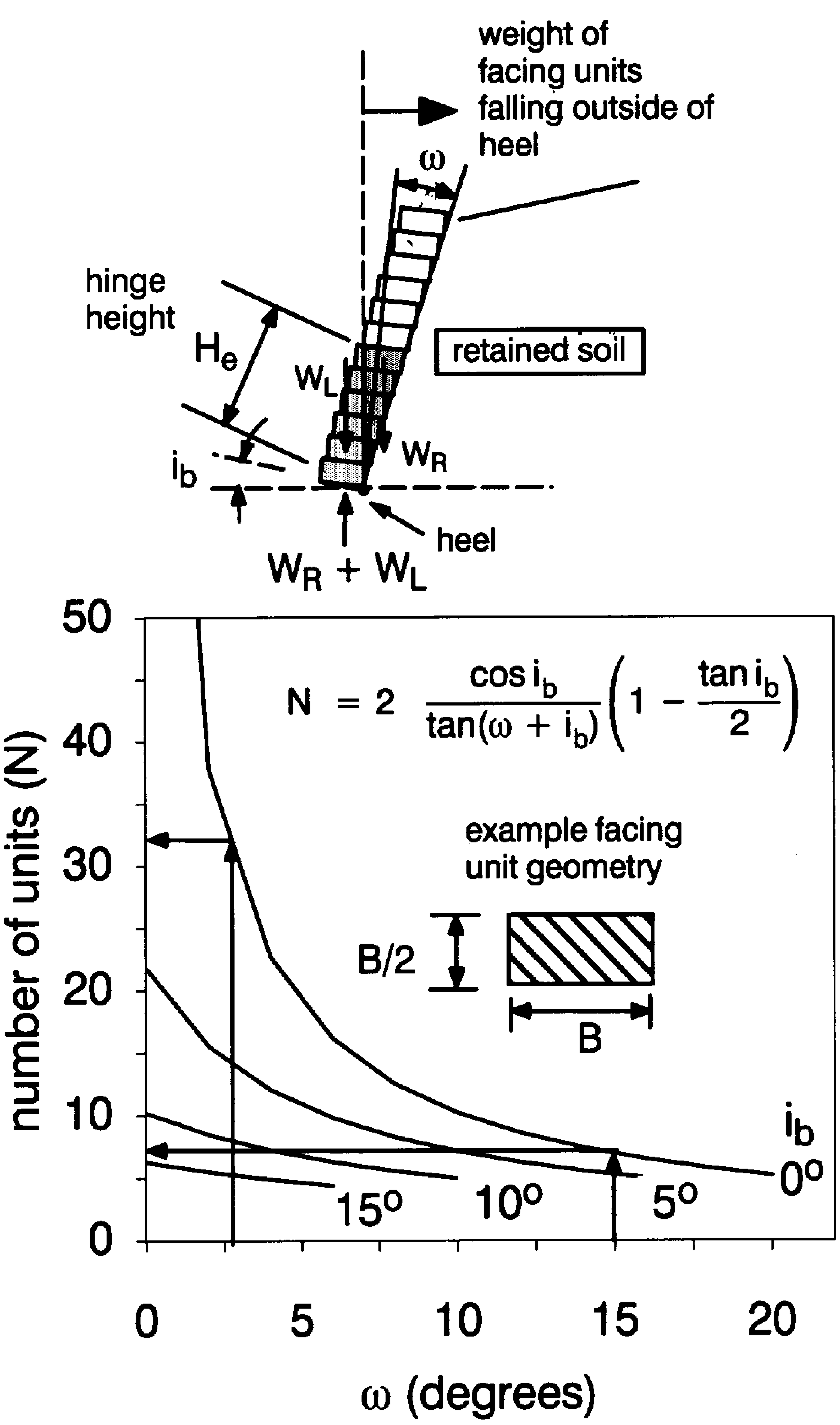


Fig. 9 Influence of wall inclination on number of facing units within hinge height (after Bathurst et al. 1993a).

can only be established by full-scale direct shear testing. The authors have developed a large-scale test apparatus to quantify interface shear under varying normal load (Figure 10). Peak interface shear capacity test data is illustrated in Figure 11 for six different modular units using a method of test developed by the authors. The data in Figure 11 has been grouped into four categories. The hollow units were infilled with a compacted angular stone with a top size of 18mm. The toe to heel dimension of the units ranged from 300 to 600mm. For a given connection type the relationship between peak interface shear capacity and normal load can be described by a simple Coulomb friction law:

$$S_{SU} = a_{su} + N_n \tan \delta_{su} \tag{11}$$

In fact, interface shear capacity may be developed by shear key interlock and to a lesser extent by mechanical connectors that are part of some systems. The authors have had some success in predicting the shear capacity of units that have a continuous concrete shear key or concrete trailing edge, based on the concrete strength, failure mechanism and dimensions of the unit. The data set in Figure 11 is limited but does illustrate that: a) there is a wide range of interface shear capacity between different types of units; and b) concrete shear keys or concrete trailing edges provide a more efficient shear connection than purely frictional interfaces. However, the fully-mobilized interface shear capacity can be expected to be reduced by the presence of any geosynthetic reinforcement inclusion that is present as part of the wall facing-geosynthetic reinforcement connection detail. Figure 12 demonstrates the reduction in peak interface shear capacity for a solid unit that results from the presence of a

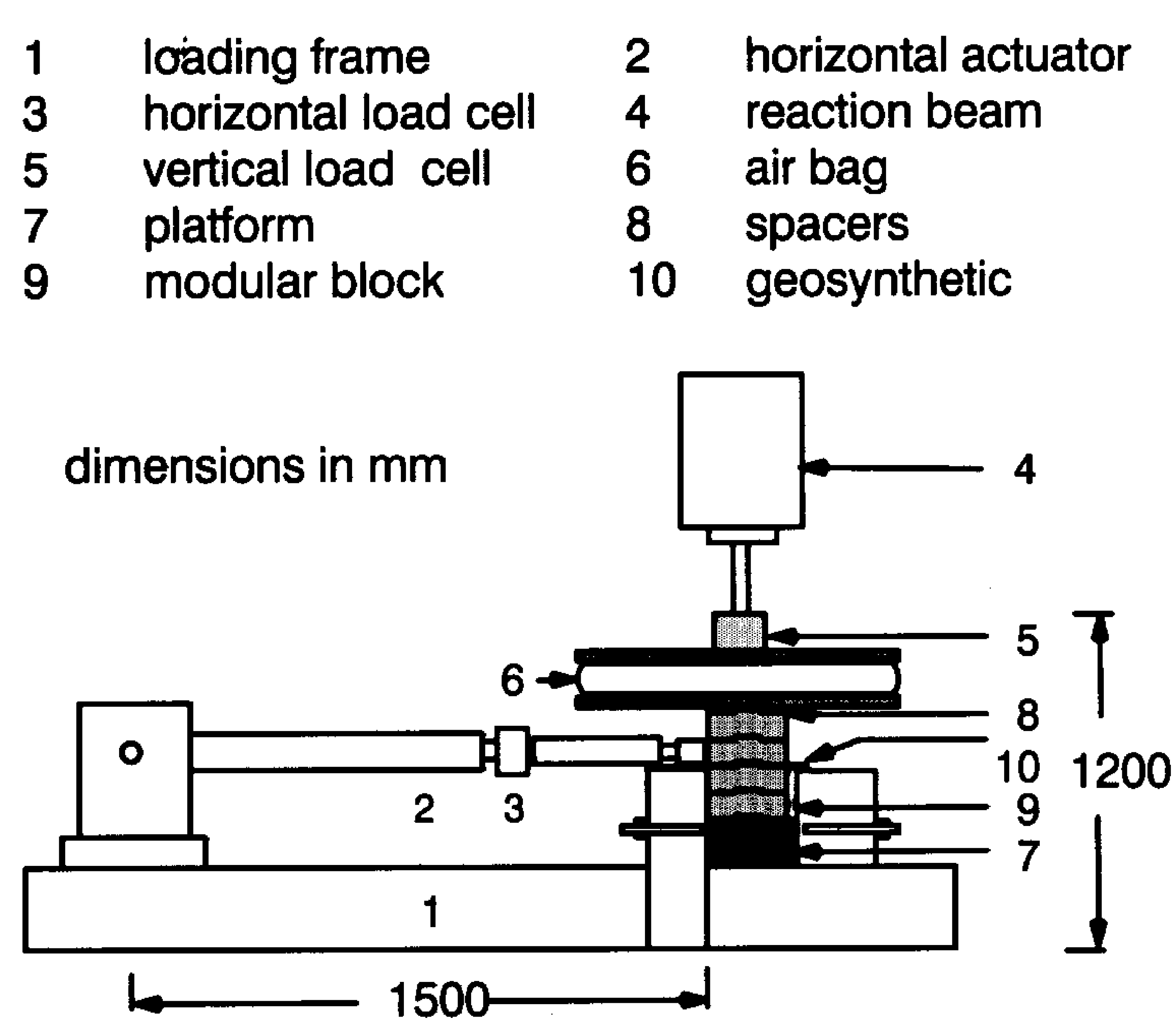
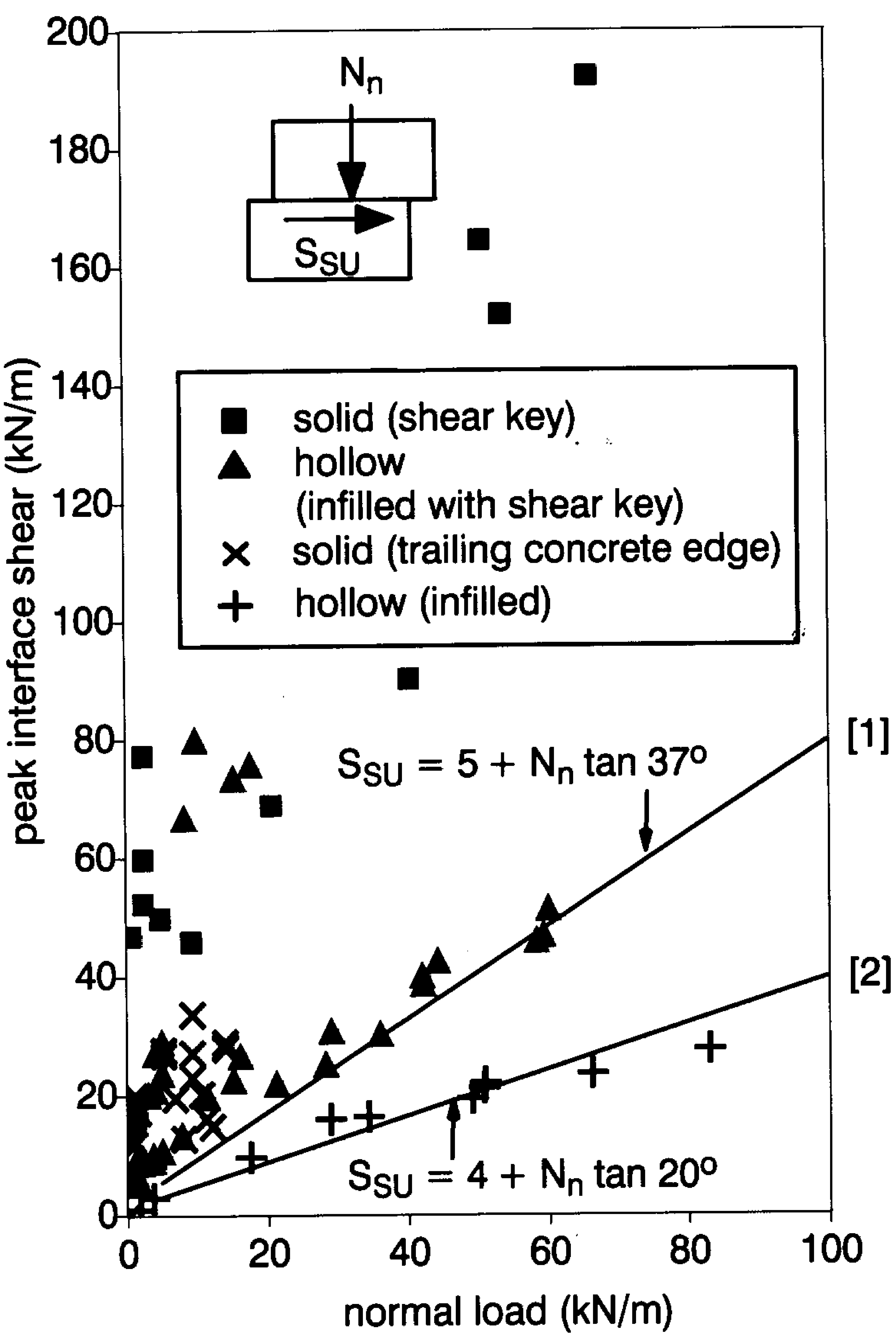


Fig. 10 Schematic of shear connection test apparatus (with geosynthetic inclusion).

polyester geogrid inclusion. The data in Figures 11 and 12 demonstrates that shear tests should be carried out with and without a geosynthetic between course layers. The design interface shear capacity (V_U) is calculated as:

$$V_U = S_{SU}/FS_{SC} \tag{12}$$

with the choice of factor of safety (FS_{SC}) based on either a peak or deformation (serviceability) criteria (Table 2). The choice of deformation criterion can be based on maximum permissible lateral wall deformations during wall construction. The test protocol developed to generate the data presented in this section has been adopted by the NCMA (1993a).



[1] lower bound limit on units with concrete shear key or concrete trailing edge (solid and hollow infilled)
[2] hollow infilled unit with flat interface

Fig. 11 Example peak interface shear versus normal load data.

7.2 Facing connection performance

The load-deformation properties of the connection formed by placing the reinforcement between block courses have been quantified by the results of full-scale laboratory connection testing as described by Bathurst and Simac (1993a). The tensile load-deformation properties are influenced by: a) geometry and type of geosynthetic-facing unit interface (i.e. continuous keys, trailing edges, dowels or pins); b) quality of the concrete; c) whether the facing units are hollow or solid; d) whether the hollow core is left empty or infilled with a granular soil; e) tolerances on block dimensions, f) quality of construction; and g) thickness, structure and polymer of the geosynthetic. A test apparatus developed at the Royal Military College of Canada (RMC) to carry out prototype-scale connection tests is illustrated in Figure 13. A similar device has been developed by researchers at the University of Wisconsin-Platteville (UWP) (Buttry et al. 1993). A protocol for carrying out these tests has also been developed by the authors and has been adopted by the NCMA (1993b). A slightly modified version

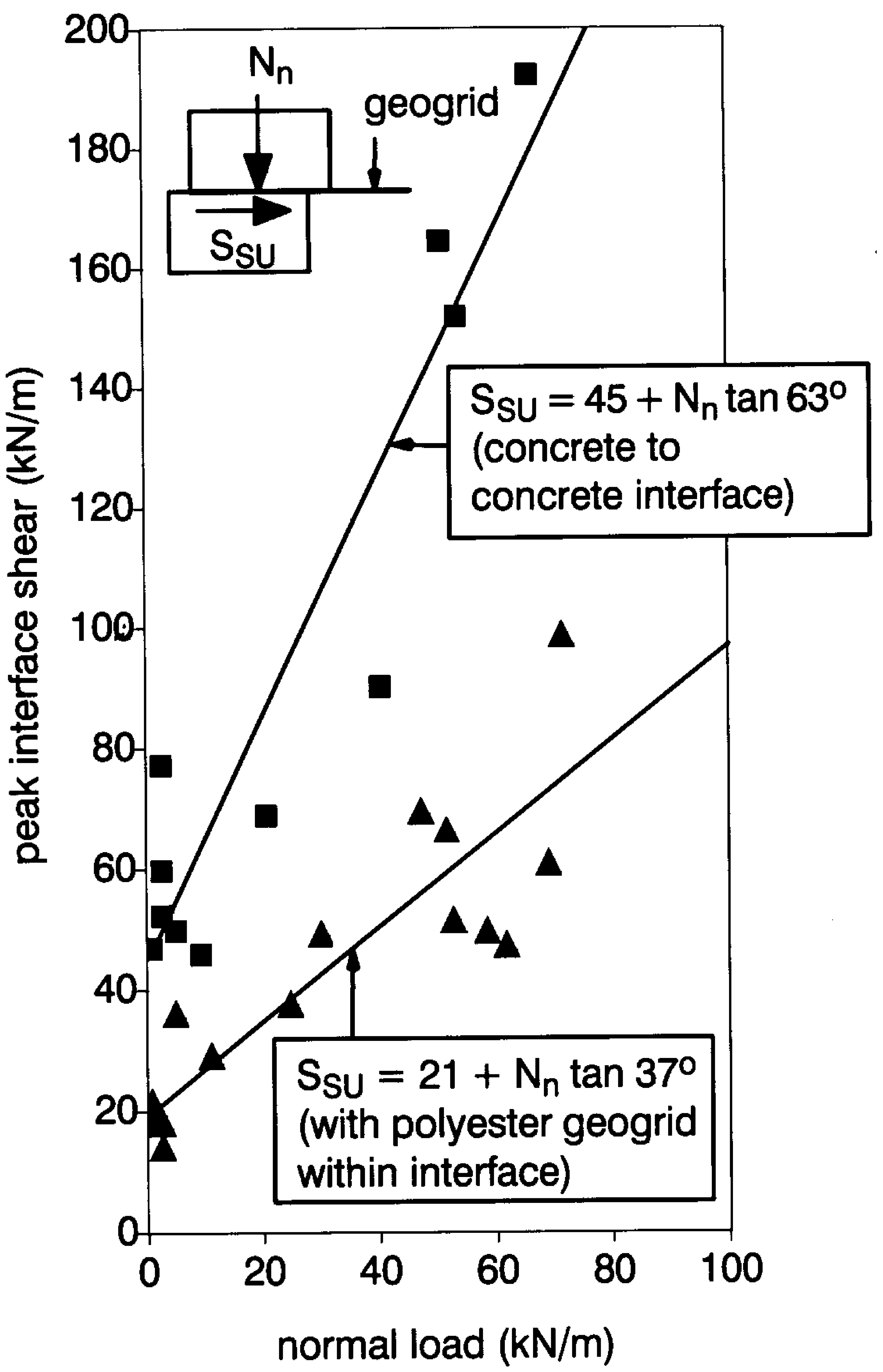


Fig. 12 Influence of geosynthetic layer inclusion on peak interface shear capacity (solid unit with continuous concrete shear key).

of this procedure is under ballot as a ASTM Standard Test Method at the time of writing. Typical test results are illustrated in Figure 14. Connection strengths have been normalized with respect to the ultimate (index) strength of the product determined according to the ASTM D 4595 (Wide-Width Strip) Test Method. In fact, the rate of loading, free length of the reinforcement, laboratory ambient temperature and relative humidity required in the NCMA method of test have been selected to match the ASTM D 4595 method of test. Essentially, the facing connection test method developed at RMC can be thought of as the standard in-isolation tensile test performed with one set of poor clamps (i.e. the block-geosynthetic connection). In this way meaningful comparisons between connection configurations can be made and their connection capacities related to a widely used index strength value. The maximum connection strength may be due to: a) pullout of the reinforcement through the interface; b) rupture of the reinforcement; c) failure of the concrete units; or d) combinations of these mechanisms.

Connection strength envelopes based on peak capacity and a 20 mm displacement criterion measured at the connection point at the back of the facing units are plotted in Figure 15. The 20 mm displacement criterion has been used by the authors to be consistent with the displacement criterion for pullout testing of geosynthetics in soil recommended by AASHTO (1990b). The bi-linear shape of the connection

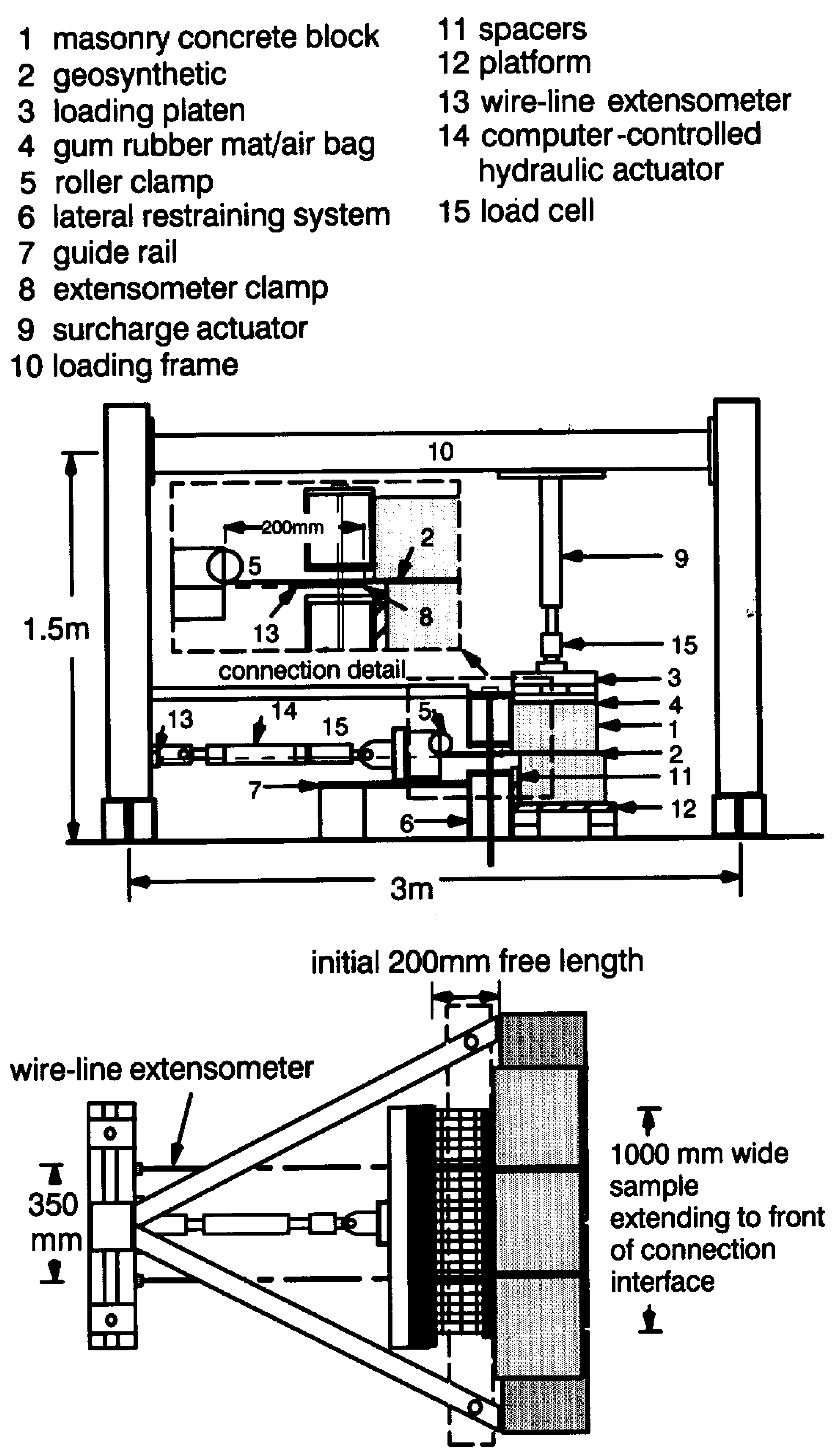


Fig. 13 Schematic of RMC connection test apparatus showing typical masonry concrete block units and geosynthetic reinforcement (Bathurst and Simac 1993a).

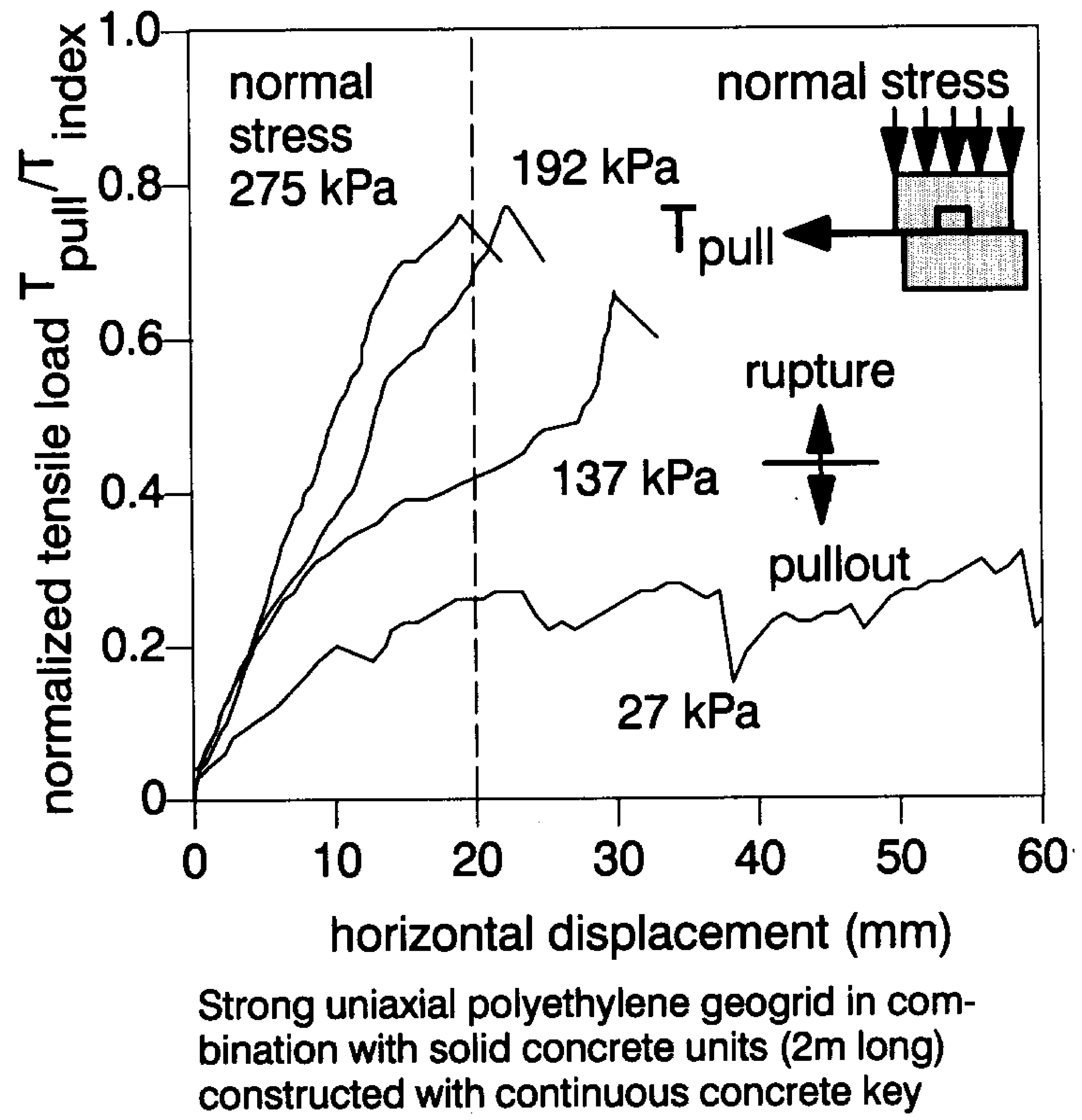


Fig. 14 Example normalized connection load-displacement curves (Bathurst and Simac 1993a).

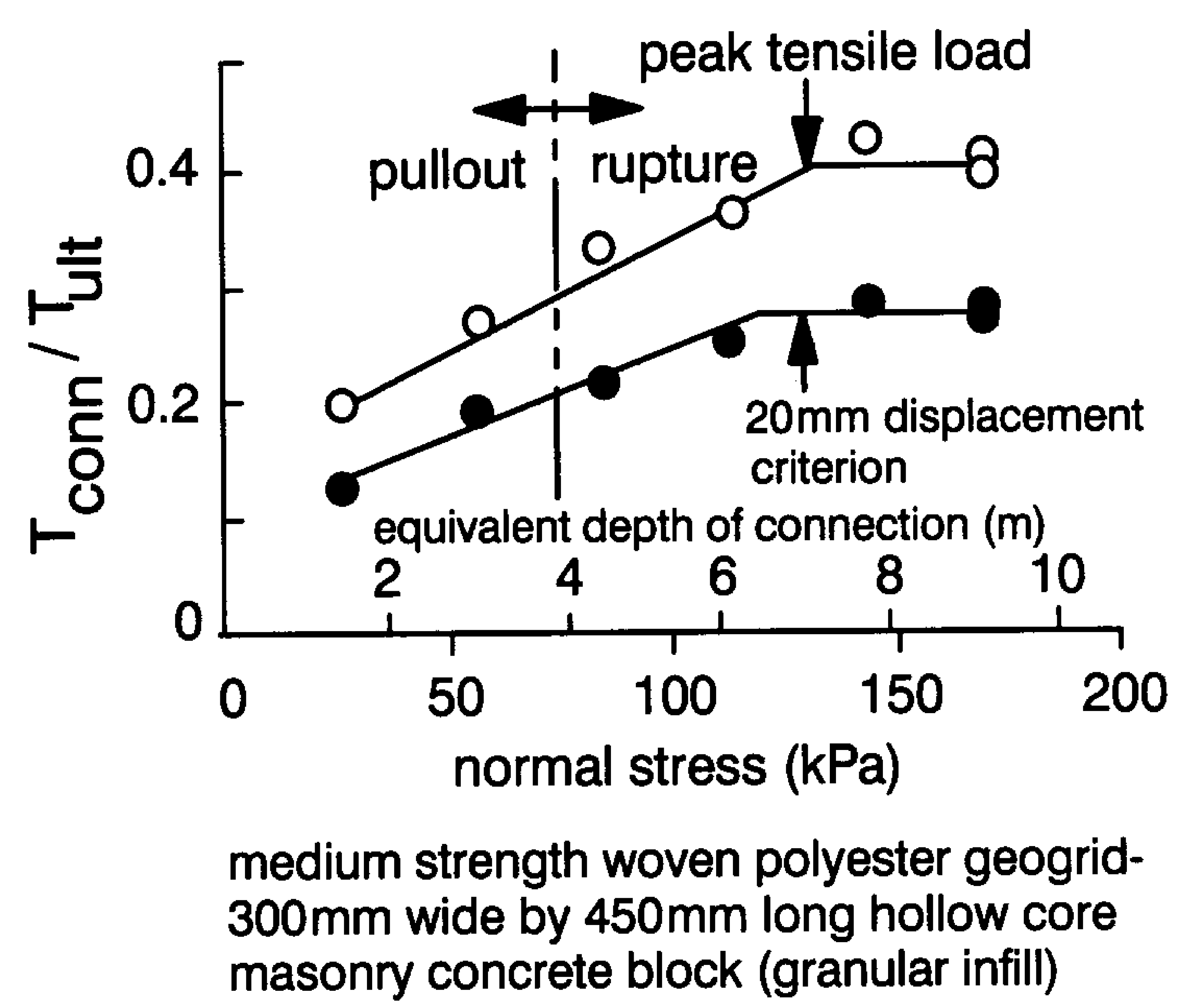


Fig. 15 Example normalized connection strength curves (Bathurst and Simac 1993a).

envelopes plotted in the figure are typical of most modular block-geosynthetic reinforcement material combinations investigated by the authors. An explanation for the bi-linear trend in the data is that the polymeric reinforcement inclusion suffers mechanical damage when placed between concrete block units, particularly after large surcharge pressures have been applied. Mechanical damage to the reinforcement occurs as a result of (typically): rough concrete surfaces; irregularities in unit dimensions; joints between the blocks; and sharp edges. These conditions lead to pinching and crushing of the reinforcement during construction and result in uneven load transfer within the connection. The initial mechanical damage increases at higher confining pressures and prevents the further increase in connection capacity with depth that might otherwise be anticipated for an essentially frictional connection.

The results of more than 1350 tests from RMC and UWP are summarized in Figure 16. The data represents 29 different segmental unit types, 20 different polymeric reinforcement materials and 131 different combinations of segmental unit and geosynthetic reinforcement. The reinforcement types include polypropylene geotextiles, and polyester and HDPE geogrids. The data in Figure 16 shows that there can be a significant reduction in connection capacity when compared to the index strength of the reinforcement for the reasons stated above. The data highlights the need to perform product-specific testing to determine accurate project data.

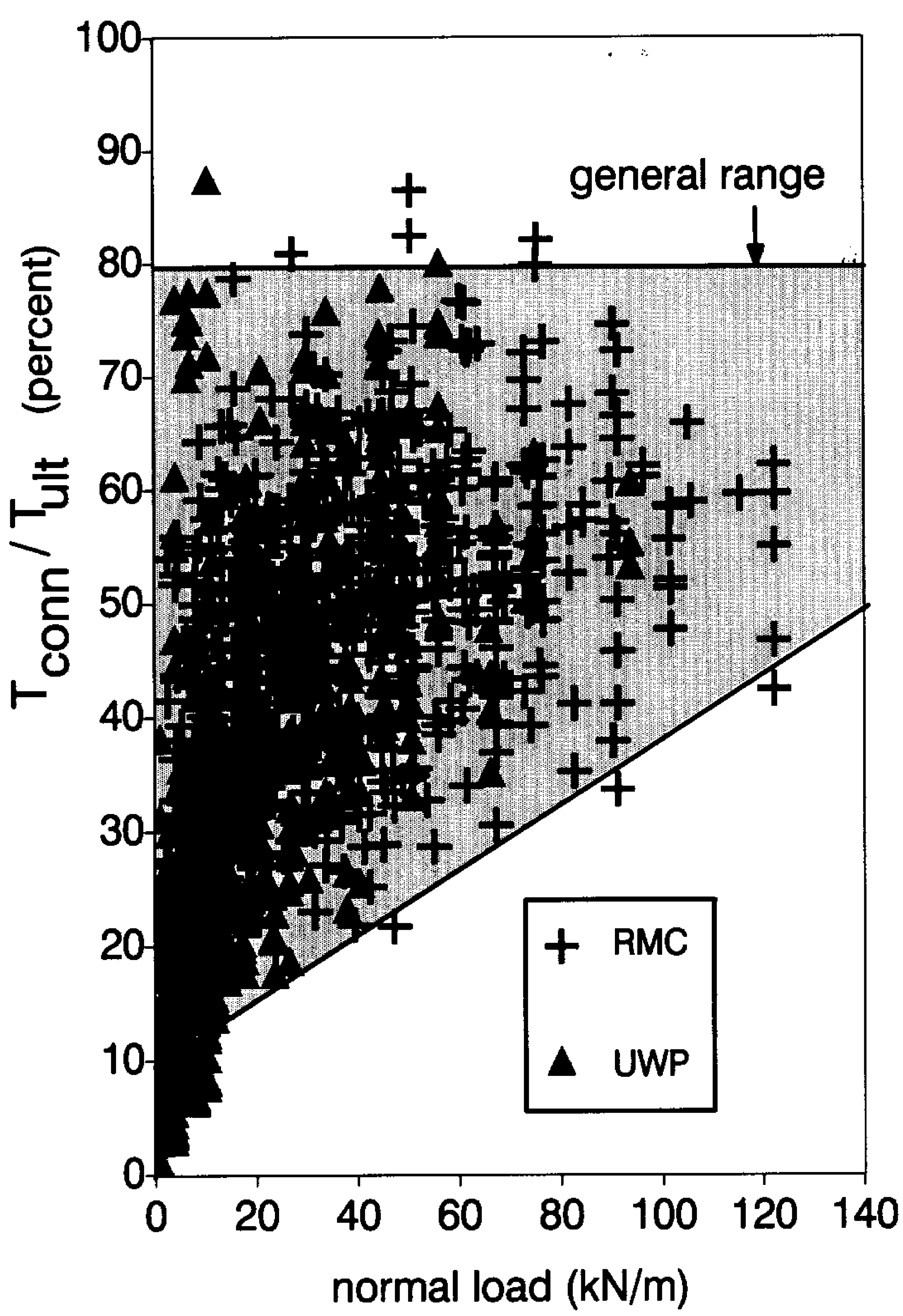


Fig. 16 Summary of connection test data.

Based on connection capacity envelopes of the type shown in Figure 15, a design capacity envelope can be developed for critical structures that is the lesser of the peak load envelope, divided by a factor of safety value (FS_{CS}), or the deformation envelope. The maximum tensile load in any reinforcement layer (T_{max}), calculated according to the contributory area approach, must fall within this design envelope.

8 SUMMARY OF DESIGN CRITERIA

A summary of principal design criteria for reinforced segmental retaining walls is given in Table 2. The factors of safety correspond to all potential failure mechanisms described earlier and have been taken from the NCMA guidelines (Simac et al. 1993a). Many of the deformation limits used in serviceability criteria in the NCMA guidelines have been selected to meet values recommended in FHWA and AASHTO guidelines.

9 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 GEOWALL (ver 2.0) has been recently released that implements many of the recommendations found in the NCMA guidelines. This program is an upgrade of the program described in the paper by Bathurst and Simac (1993b). The NCMA intends to release a computer program called NCMAWall in early 1995 that will provide a full implementation of the NCMA guidelines or, alternatively, allow the user to design according to FHWA recommendations. The program has been developed for the Microsoft Windows operating system. A typical graphics user interface (GUI) from the program is illustrated in Figure 17.

10 CASE STUDIES

A number of geosynthetic reinforced segmental retaining walls in North America have been monitored. The data from these structures is useful to verify design methodologies and provide performance data.

10.1 Algonquin Wall

A research project sponsored by the Federal Highway Ad-

Table 2: Recommended minimum factors of safety for design of geosynthetic reinforced soil segmental retaining walls.

FAILURE MODE		APPLICATIONS CRITICAL	APPLICATIONS NON-CRITICAL
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_{OS}	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 is used. Estimated data includes 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 0.45 m for critical structures or 0.15 m for non-critical structures.
6. A non-critical 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 75 to 100 year life. Similarly, transportation-related structures would normally be considered critical structures.

(after Simac et al. 1993a)

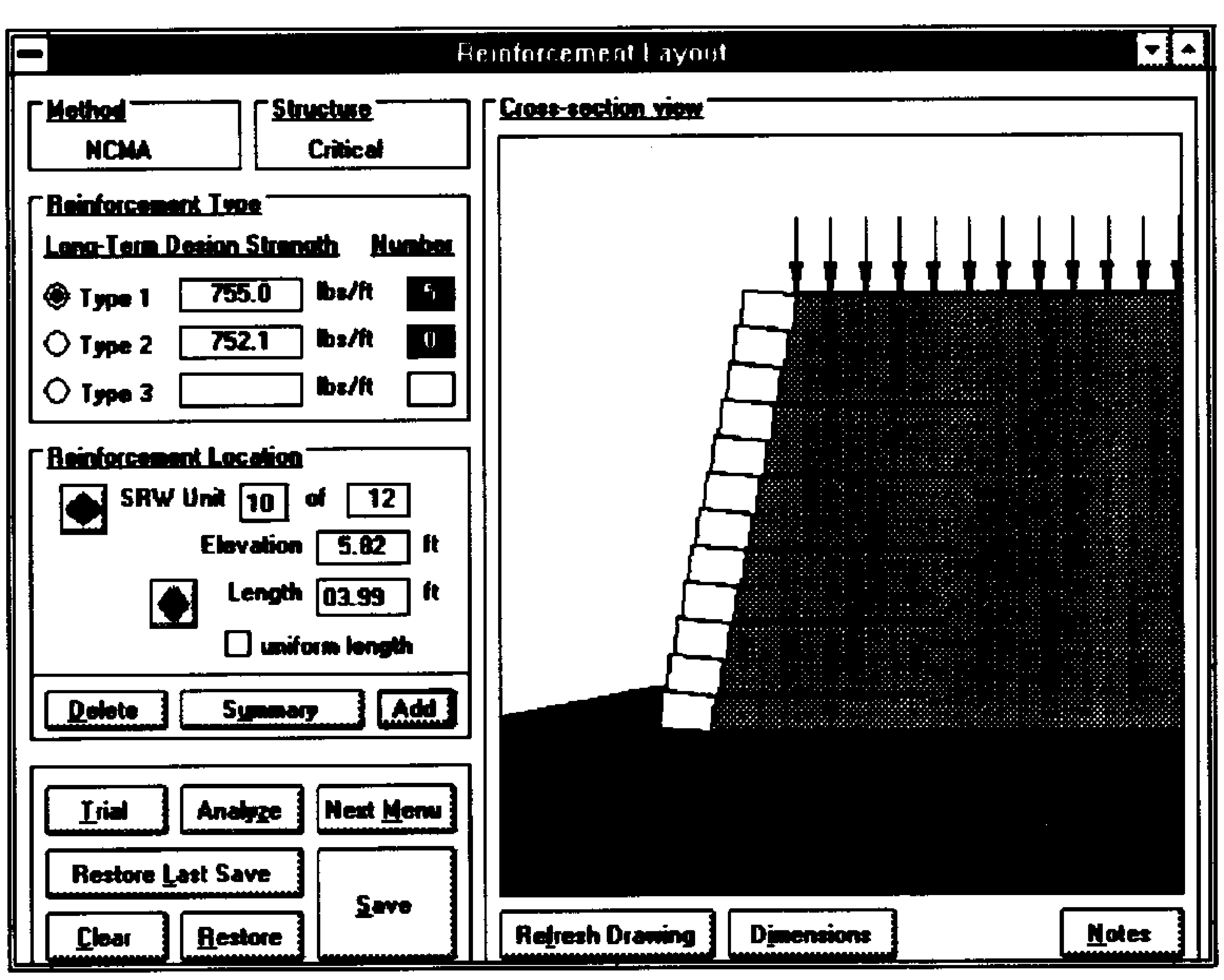


Fig. 17 Program NCMAWall.

ministration (FHWA) was initiated in the late 1980's in Algonquin, Illinois, USA that involved the full-scale construction and testing of several proprietary retaining wall systems using both extensible (polymeric) and in-extensible reinforcement with a variety of facing types (Christopher et al. 1989, Simac et al. 1990, Bonczkiewicz et al. 1991, Bathurst et al. 1993b, Christopher 1993). The wall that is discussed here is illustrated in Figure 18 and 19 and had a total length of 15 m and a crest height of 6.1 m. The wall was heavily instrumented and was monitored for a period of 479 days following completion.

The facing comprised 47.5 kg hollow (soil infilled) modular masonry concrete facing units (Keystone Standard) with the nominal unit dimensions shown in Figure 20. A pair of 13 mm diameter pins was used to provide an automatic setback of 10 mm per course. The resulting setback gave a target facing batter of 20 vertical to 1 horizontal. The reinforcement for this project was a high-tenacity woven polyester geogrid distributed by Nicolon/Mirafi (Miragrid 5T). The reinforcement was placed over the masonry units and pre-tensioned by hand. A uniform size pea gravel was used to infill the hollow portions of the modular block units and the spaces between adjoining units. The pea gravel was also extended to a distance of 600 mm behind the facing columns to create a drainage layer which is a common recommended practice in reinforced segmental retaining wall design. The reinforced and retained soil comprised a cohesionless well-graded sandy gravel with a peak friction angle of 40 degrees (Christopher 1993) and this material was compacted to 95% of standard Proctor.

The wall was designed using a computer program that implements a conventional limit-equilibrium approach together with Coulomb-type soils (Bathurst and Simac 1993b). The design methodology was based on recommendations contained in AASHTO and FHWA guidelines in effect at the time of the project (Christopher et al. 1989 and AASHTO 1990a,b). The exceptions to AASHTO/FHWA guidelines were the use of 4.3 m lengths of reinforcement (versus 3.5m required) and the use of an overall factor of safety (FS_{MU}) of 1.0 to calculate the long-term allowable design strength of the reinforcement. The 4.3 m lengths were chosen to be consistent with a number of other structures at the site (Christopher et al. 1989). The number and vertical arrangement of the geogrid reinforcement was selected to give a factor of safety of 1.1 with respect to over-stressing of the reinforcement beyond its serviceability limit load of 15.5 kN/m (strain < 10%) based on in-isolation creep tensile testing of the geogrid material together with typical partial factors of safety for construction damage and durability (Simac et al. 1990). With the surcharge in place the same factor of safety was reduced to 0.8. The resulting deficiency in internal design with respect to over-stressing of the reinforcement was purposely carried out to encourage large reinforcement strains and wall movements and hence examine conservatism in current methods of design used in North America at the time of the project.

The wall was constructed in six weeks using a three-man crew with a front end loader and self-propelled steel drum roller. The wall was constructed with a routine standard of care using conventional construction procedures interrupted only by installation of instrumentation. A complete description of construction activities can be found in the pa-

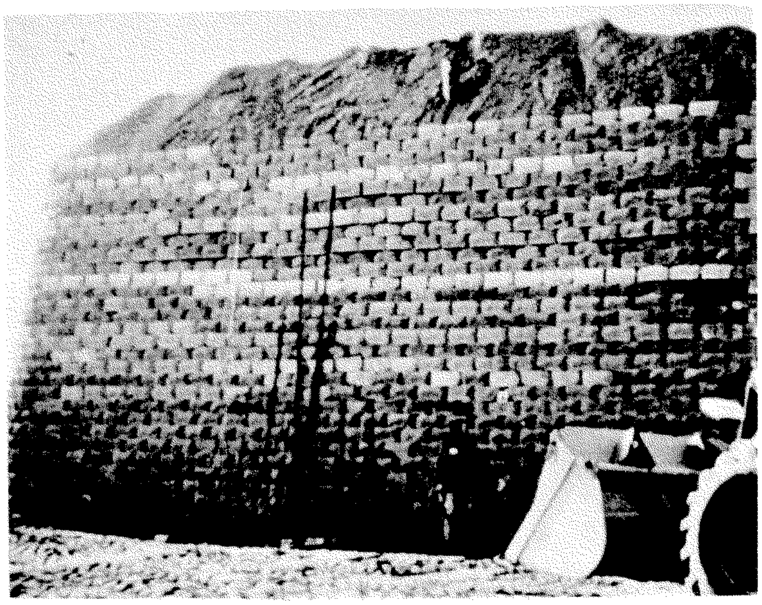


Fig. 18 Algonquin wall.

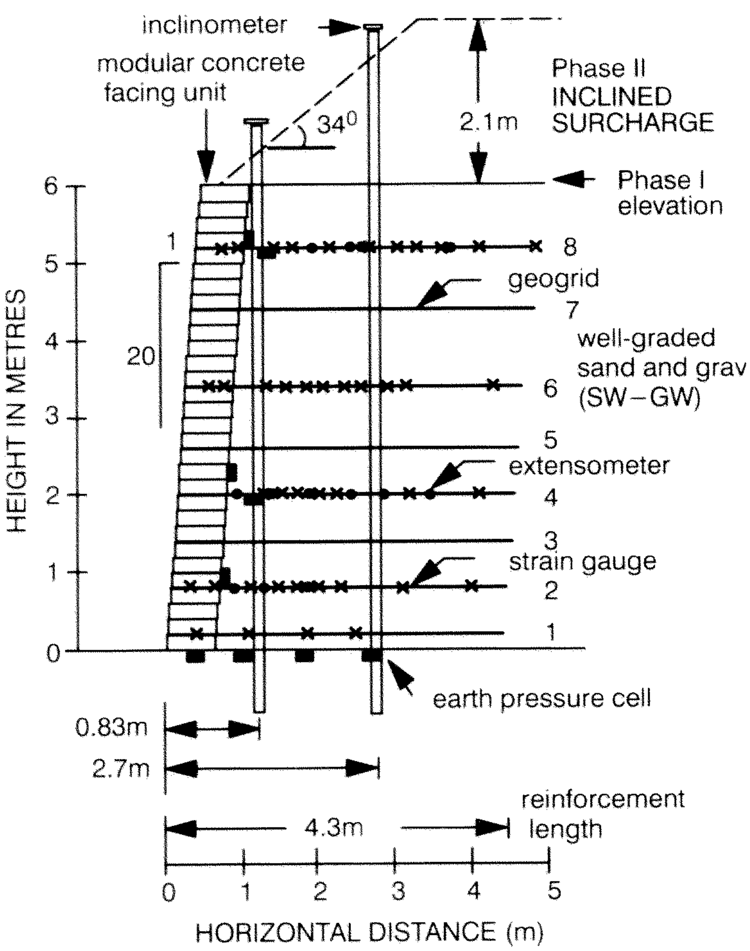


Fig. 19 Cross-section and instrumentation of Algonquin wall.

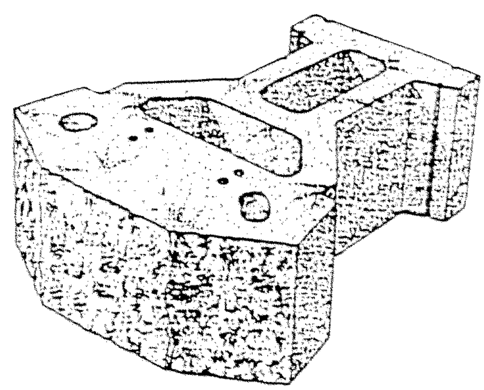


Fig. 20 Modular block unit: 200 mm high by 600 mm deep (toe to heel dimension) by 460 mm wide (Algonquin wall).

Table 3: Comparison of measured and predicted lateral earth pressures in Algonquin wall (3.8 m below wall crest).

Measured	Rankine (δ=0)	Coulomb (δ=0)	(δ=2/3ϕ)*	(δ=ϕ)*
end of construction				
max 15.2				
min 10.2				
avg 12.7	16.6	18.5	12.8	11.9kPa
surcharge in place				
max 15.8				
min 9.92				
avg 12.7	18.1	20.2	14.0	13.0kPa

* NCMA method

per by Simac et al. (1990). A full description of the instrumentation program and all test data gathered from the project is reported in the paper by Bathurst et al. (1993b).

The results of inclinometer readings are summarized in Figure 21. The data shows that post-construction outward wall movements were generated over the bottom half of the wall. Lateral earth pressures recorded for the bottommost pressure cell mounted against the back of the facing column are summarized in Table 3. Only the data from this lateral earth pressure cell was considered reliable by field technicians (Simac et al. 1990). However, all cells gave signals

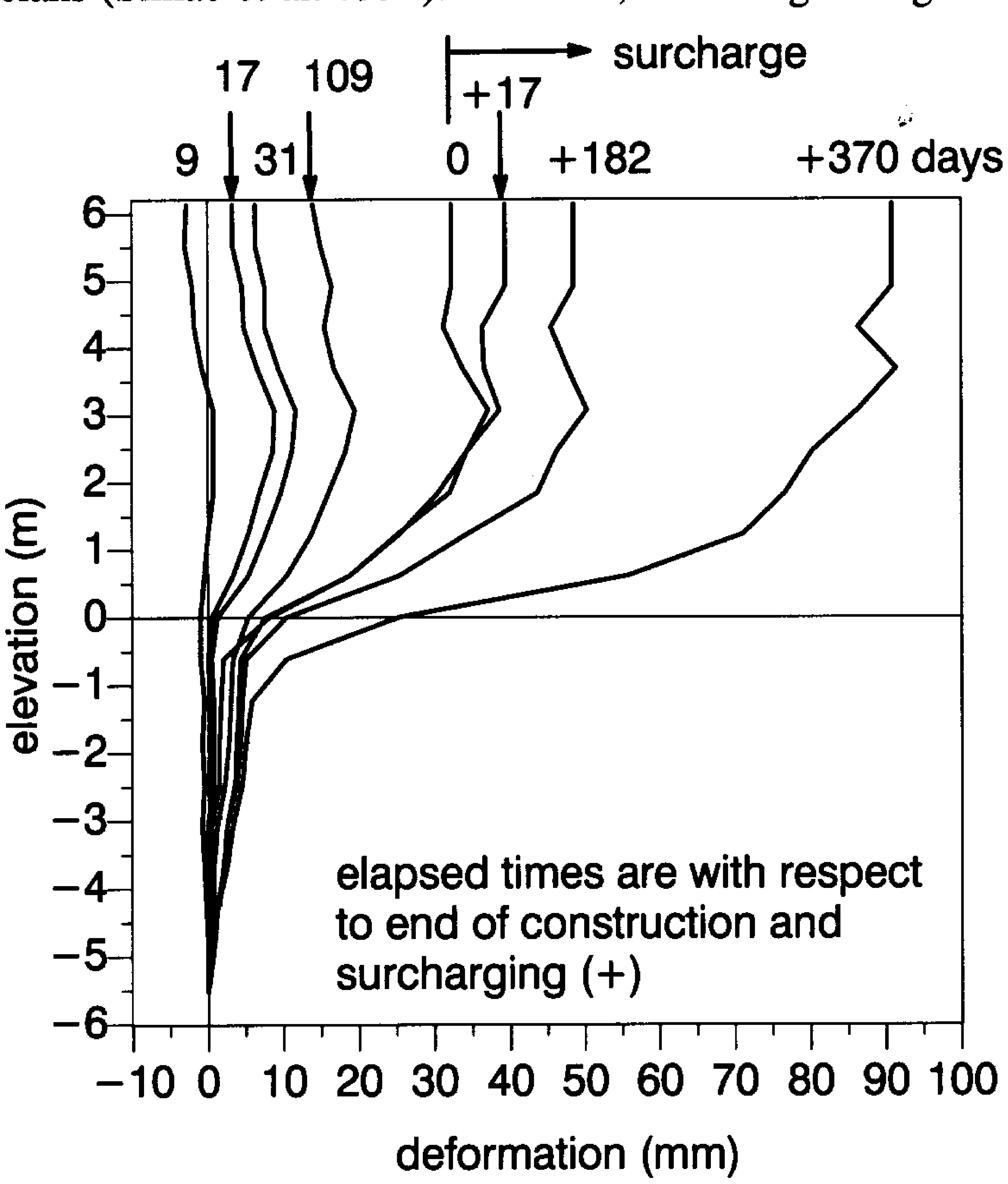


Fig. 21 Post-construction front inclinometer deformations (Bathurst et al. 1993b).

indicating that lateral earth pressures are developed directly against the facing column. In addition, strain gauges mounted directly on the reinforcement indicated load transmission within the facing connections. Hence, the facing connections for these segmental retaining wall structures must be designed to carry load and the horizontal interface between stacked facing units must be designed to transmit shear. The data shows that the most accurate prediction of the horizontal component of active lateral earth pressure is given by Coulomb solutions with the inclination of active earth thrust taken as $\delta=\phi$ or $\delta=2/3\phi$. Based on the experience with the cells located at higher elevations, it is not possible to investigate the influence of any compaction-induced stresses that may have developed over the top half of the wall. Compaction-induced stresses could lead to greater pressures than those predicted by active earth pressure theory. However, routine practice in the field is to minimize compaction stresses by using light-weight compaction equipment directly behind the facing. Finally, an advantage of most modular block systems is that they provide some lateral compliance against compaction loads that may assist to minimize compaction-induced stresses. Further high quality field monitoring of lateral pressures at the back modular block facing columns is required to explore this issue further.

Reinforcement strains inferred from extensometer readings are summarized in Figure 22. The data shows that the maximum reinforcement strains in the reinforcement were not greater than about 2%. With the exception of one reading at layer 4, all strains in the reinforcement layers were less than 1%. The maximum strain reading of 2% is well within the creep limit of the reinforcement. Despite being designed for strains as great as 10%, the loads in the reinforcement did not approach design loads. The over-prediction of reinforcement loads by conventional limit-equilibrium methods that employ active earth pressure theory has been noted by the first author in large-scale model tests of other geosynthetic reinforced soil walls that used full-height and flexible facing systems. In addition, significant footing restraint has been demonstrated to occur in carefully monitored laboratory experiments and this mechanism may be responsible for reducing the total tensile load carried by extensible reinforcement layers (Bathurst 1993). Based on data from the Algonquin wall and other carefully monitored geosynthetic reinforced wall structures there is increasing confidence that Coulomb earth pressure theory results in conservative (i.e. safe) estimates of reinforcement tensile loads.

The distribution of base pressures at selected locations below the facing column and reinforced soil zone is given in Figure 23. The peak pressure readings agree well with computed pressures based on bulk unit weight of the reinforced soil and surcharge materials (γz+q). However, vertical pressure concentration towards the front of the wall was not observed as would be predicted by considering eccentricity of the resultant vertical load from the reinforced zone and a trapezoidal vertical pressure distribution. An implication of this data to design is that a base eccentricity criterion of the type employed for rigid footings may be unwarranted for flexible reinforced soil structures. Eccentricity criteria can be shown to control the minimum required width of the reinforced soil zone in earlier design guidelines and hence adds to excessive conservatism in the calculation of minimum reinforcement lengths.

The pressures recorded at the base of the facing column provide a check on the calculation of normal force transferred across modular block interfaces. For the Algonquin wall the steep facing batter results in the hinge height being equal to the height of the facing column above any interface (H_e=H). The measured footing loads are plotted in Figure 24 together with theoretical values calculated as the facing column weight alone and the facing column weight plus additional down drag force due to facing column-soil interface shear. The figure shows that the column self-weight predicted using just the column self-weight is accurate.

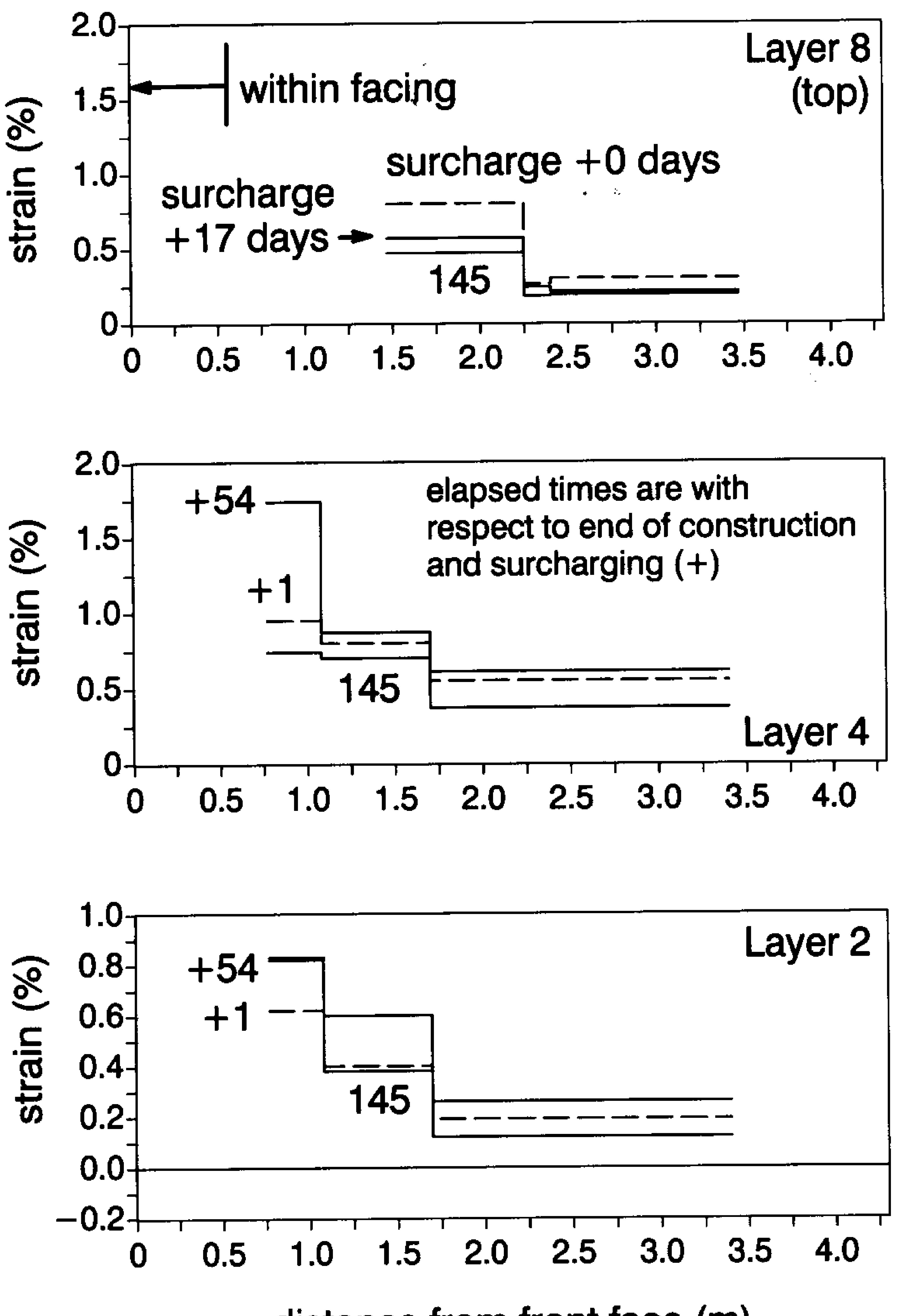


Fig. 22 Summary of reinforcement strains at selected times (Bathurst et al. 1993b).

Additional forces assumed to be developed by interface shear result in over-prediction of interface normal force which would result in unconservative design for interface shear and connection capacity design. It should be noted that the Algonquin wall was constructed on a very stiff foundation which can be expected to minimize relative downward movement of the reinforced soil mass relative to the facing column. For example, the maximum vertical settlement of the wall was measured as 1.5mm with a distortional settlements of about 0.01% along the length of the wall.

10.2 California Wall

A temporary geosynthetic-reinforced segmental retaining wall was constructed in 1990 at the site of a deep excavation in California (Figure 25 and 26). The wall was used to retain a sand backfill to a height of 18m and had a base length of 76m. The modular units were identical to the units used in the Algonquin wall (Figure 20) but were specially fabricated to give a zero setback (i.e. no wall batter). The modular units were founded on a compacted granular base and reinforced with 23 layers of a polyester geogrid with a long-term design strength of 59kN/m. Once the retaining wall was completed it was used as the formwork for the basement wall that was cast against the modular unit facing. The wall was designed to satisfy all limit-equilibrium based

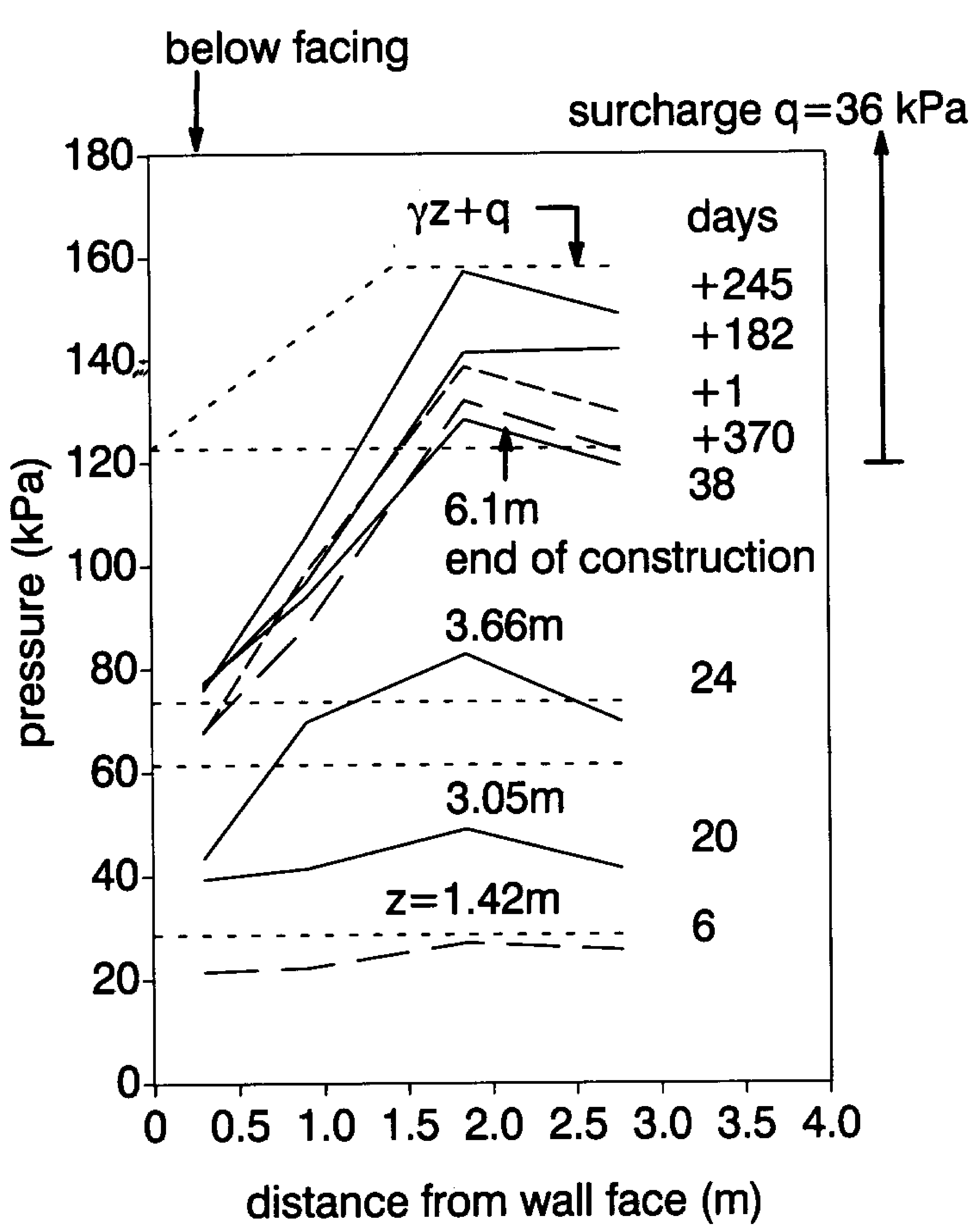


Fig. 23 Vertical pressure at base of reinforced soil mass (after Bathurst et al. 1993b).

performance criteria in accordance with AASHTO and FHWA guidelines. However, the vertical out-of-alignment that developed as wall height increased resulted in encroachment of the wall into the excavation. In order to monitor this movement, measurements were taken at six stations along the wall as the height of the facing column increased. The data in Figure 27 represents outward movements at the crest of the wall *during* construction normalized with respect to current wall height. The data shows that out-of-alignment movements do not increase monotonically with increasing wall height during construction. Rather, crest movements are largely controlled by lower portions of the wall. Bulging of the wall structure over the lower half of the wall height at the California project is consistent with the trend of post-construction movement observed for the Algonquin wall (Figure 21). The development of outward movements will also be influenced by quality of construction, compaction-induced stresses immediately adjacent to the back of the facing units, block interface shear stiffness and lateral earth pressures developed under soil self-weight. The maximum crest movement as a result of construction activity was approximately 1.3% of the height of the wall and it is reasonable to assume that conditions of active earth pressure were developed behind the structure. Based on the magnitude of maximum outward movement (i.e. 130mm) the wall was stepped back after achieving a height of 14.3m in order to minimize further movement into the excavation that would result by continu-

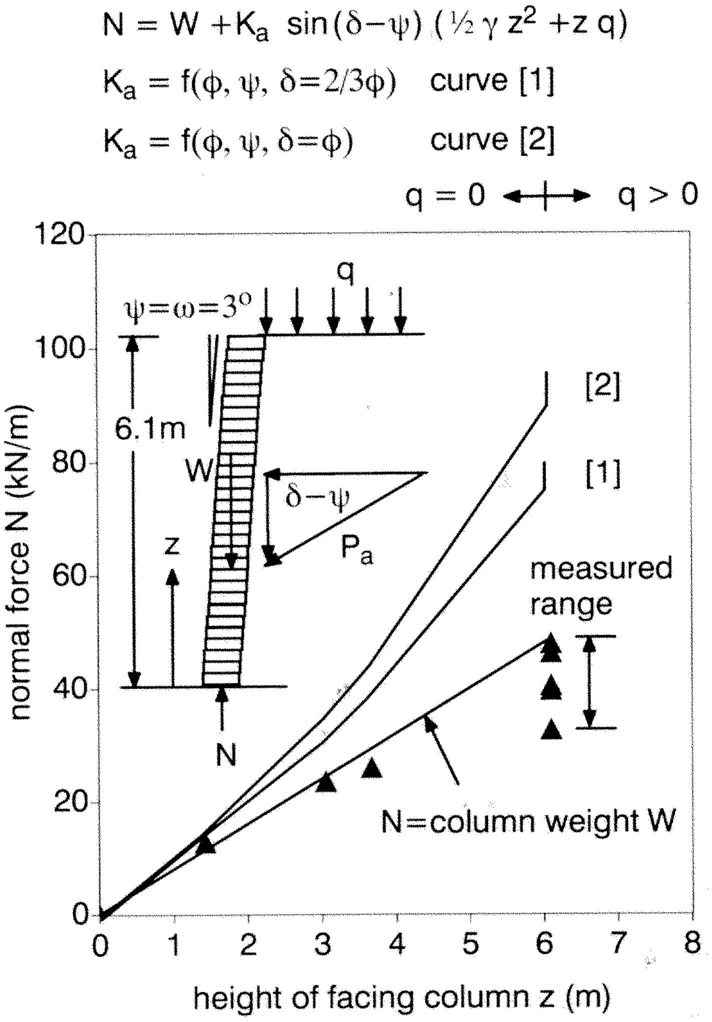


Fig. 24 Comparison of normal force at base of wall with predicted values (Algonquin wall).

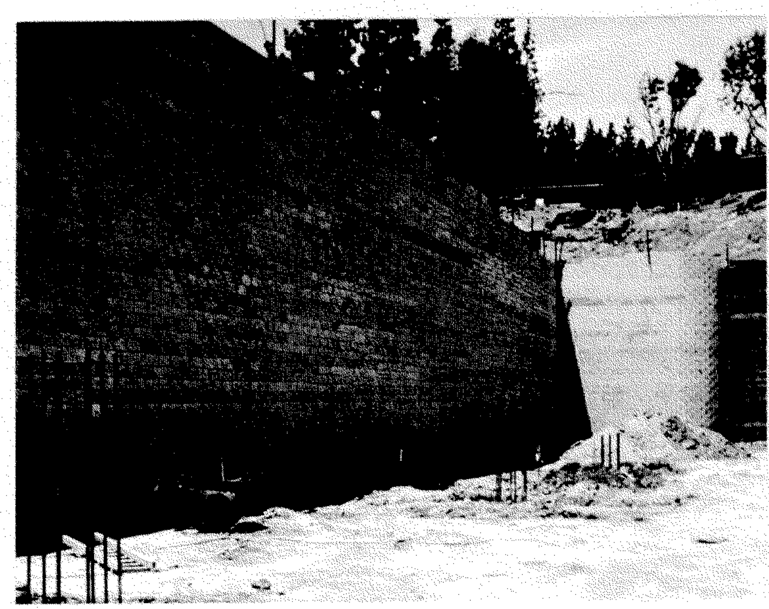


Fig. 25 California wall.

Fig. 26 Facing detail (California wall).

ing with a single tier wall.

The data contained in Figure 27 may be useful to estimate how much pre-batter should be constructed into a reinforced segmental retaining wall face in order to ensure that the end-of-construction wall batter is not less than the target wall value. Based on the limited data available, a value of 1.0% of the height of the wall for structures 8 m or less and 1.5% of the height of the wall for structures greater than 8 m in height may be reasonable construction guidelines. These recommendations are very preliminary and assume that structures are built with the same reinforcement density and length, and the same routine standard of care that was employed at the projects cited. The limits include an additional margin for post-construction creep equal to that recorded for the Algonquin wall (0.25%) after 109 days. The data in Figure 27 and the preliminary construction guideline are less than values predicted by a similar chart in the FHWA guidelines (Christopher et al. 1989). The recommendations contained in the FHWA guidelines were based on finite element modelling (Mitchell and Christopher 1990) and predict end-of-construction movements of about $\Delta H/H = 1.7\%$ for both the UWP wall and the California wall.

10.3 University of Wisconsin-Platteville (UWP) Wall

An instrumented segmental wall 3.5 m high and 37 m long was constructed at the University of Wisconsin-Platteville in 1993. Preliminary results of this structure have been reported by Wetzel et al. (1995). The wall is illustrated in Figure 28. The wall was divided into three sections 7.62 m in length and each section was reinforced with a different geosynthetic (HDPE geogrid, woven polyester geogrid and a woven polyester geotextile). The wall was designed according to NCMA guidelines with the long-term design strengths of the materials taken as 15.3, 15.3 and 13.1 kN/m, respectively. The wall units were seated on a compacted granular footing. The granular footing and backfill comprised a washed crushed limestone with a top size of 19 mm and a peak friction angle of 38 degrees.

The vertical out-of-alignment of the wall due to construction activities is estimated to be about 8 mm at the end of construction. Hence, construction-induced outward movements are about 0.2% of the height of the wall with essentially all construction-induced movement occurring within the lower half of the wall height. As in the earlier cited case studies, bulging of the lower half of the wall occurred resulting in larger relative outward movements at intermediate stages of wall construction.

A load cell was incorporated into the facing column at 0.75m above the base of the wall. Vertical load transmitted across the interface between units was monitored during construction and for a period of six months thereafter. Measured normal force readings reported by Wetzel et al. (1995) are compared against: the weight of the facing column

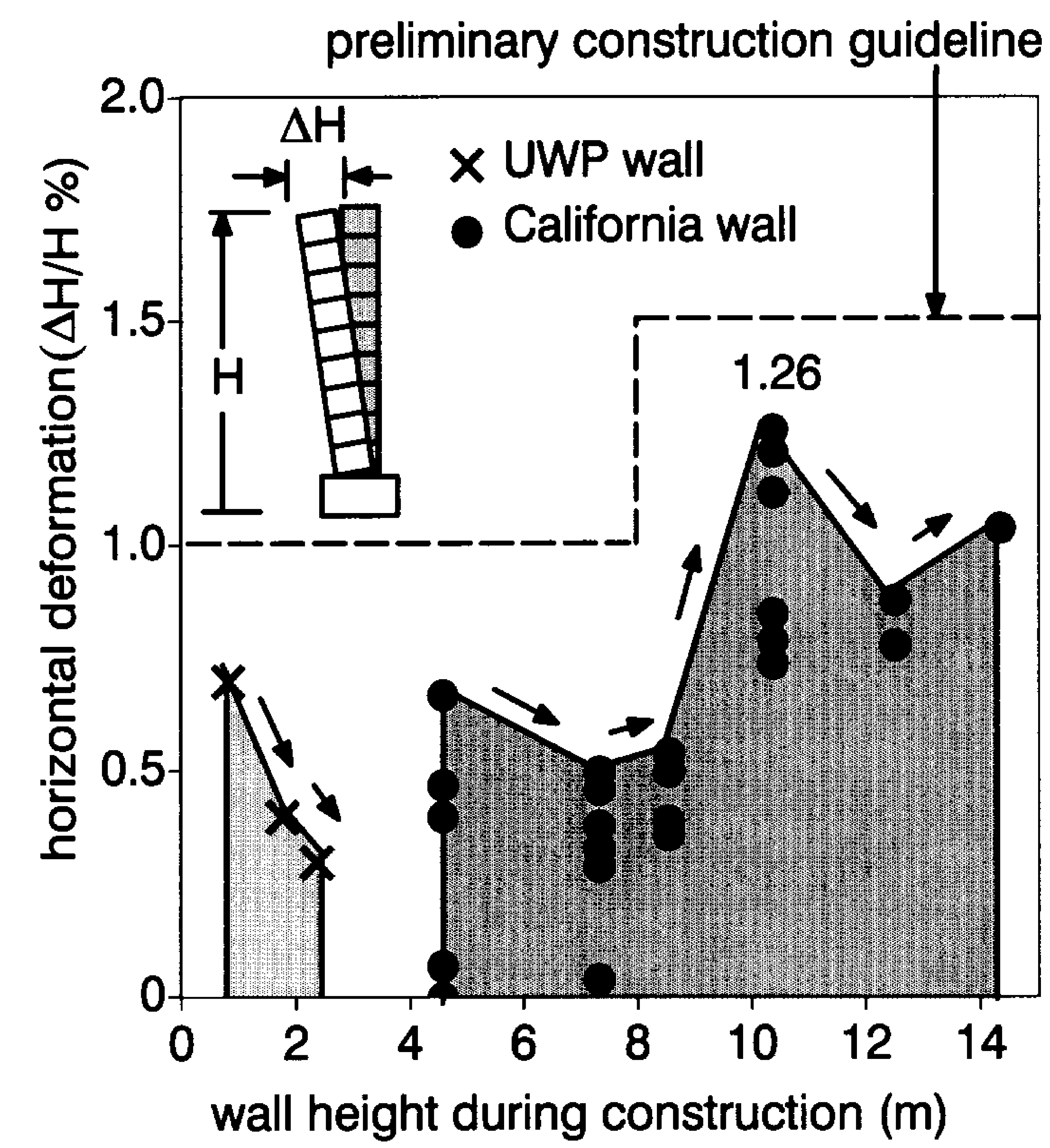


Fig. 27 Maximum normalized horizontal wall movement with increasing wall height *during* construction.

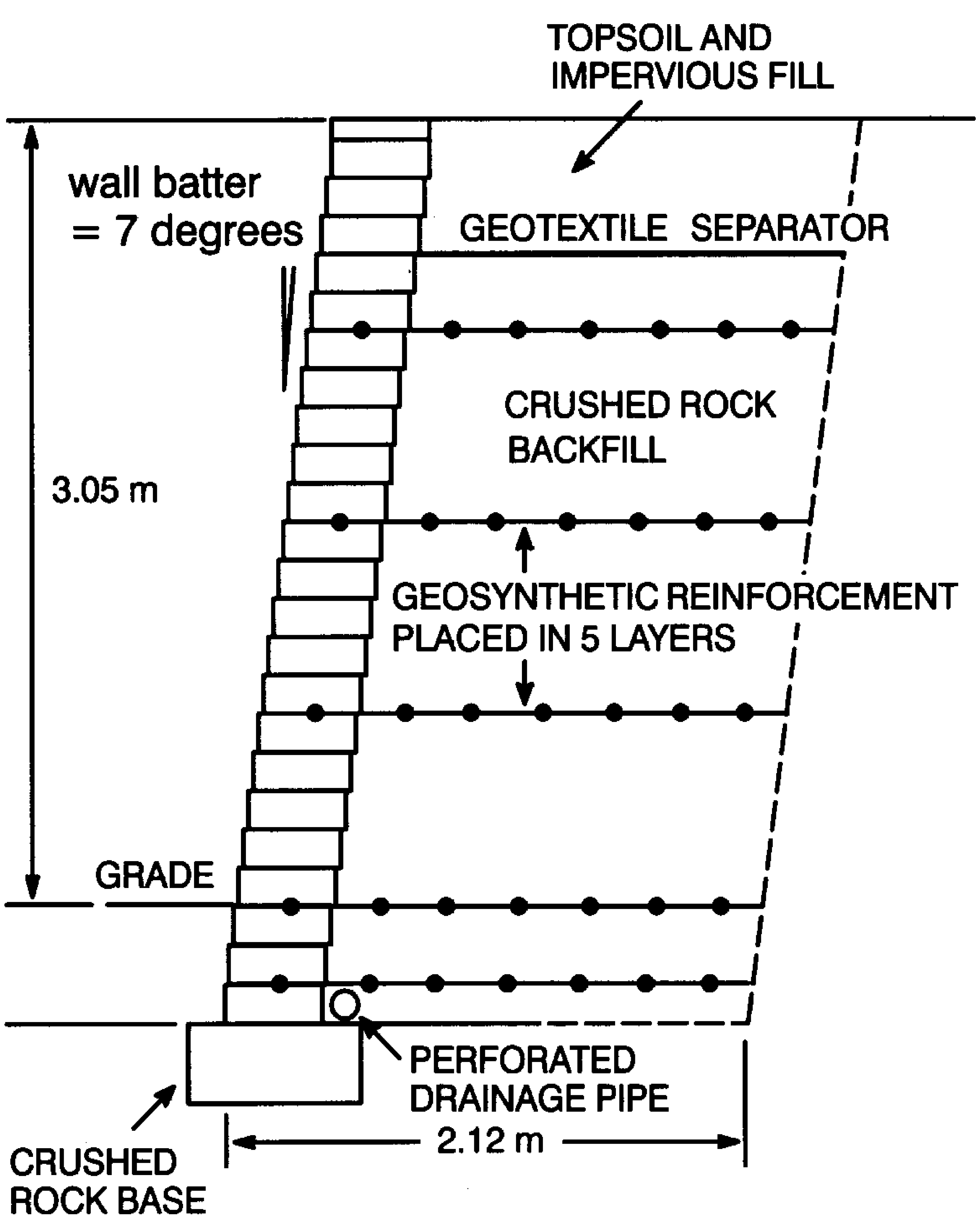


Fig. 28 University of Wisconsin-Platteville segmental retaining wall (after Wetzel et al. 1995).

above the point of measurement; predicted load based on the hinge height concept; and column weight plus down drag forces using Coulomb earth pressure theory (Figure 29). The plot shows that interface normal loads during construction and at the end of construction are significantly underestimated by neglecting facing-soil interface shear. However, with time there was a reduction in normal load and post-construction normal forces are seen to approach values predicted by neglecting down drag forces. In fact, the lowermost curve in Figure 29 (hinge height approach) provides a reasonable lower bound on all measured values reported by Wetzel et al. (1995). The initially high wall friction forces inferred from the field data may be due to relative downward movement of the reinforced soil mass behind the facing units. The maximum vertical settlement of the foundation since start of construction was 22mm. This large movement compared to the Algonquin wall, for which down drag forces appeared to be non-existent, may explain why wall friction was mobilized in the UWP wall. The influence of relative downward movements of the backfill on connection loads for rigid facing systems has been clearly demonstrated by Bathurst (1991, 1993) and Bathurst and Benjamin (1990). In rigid panel facing systems the maximum load in the reinforcement was observed

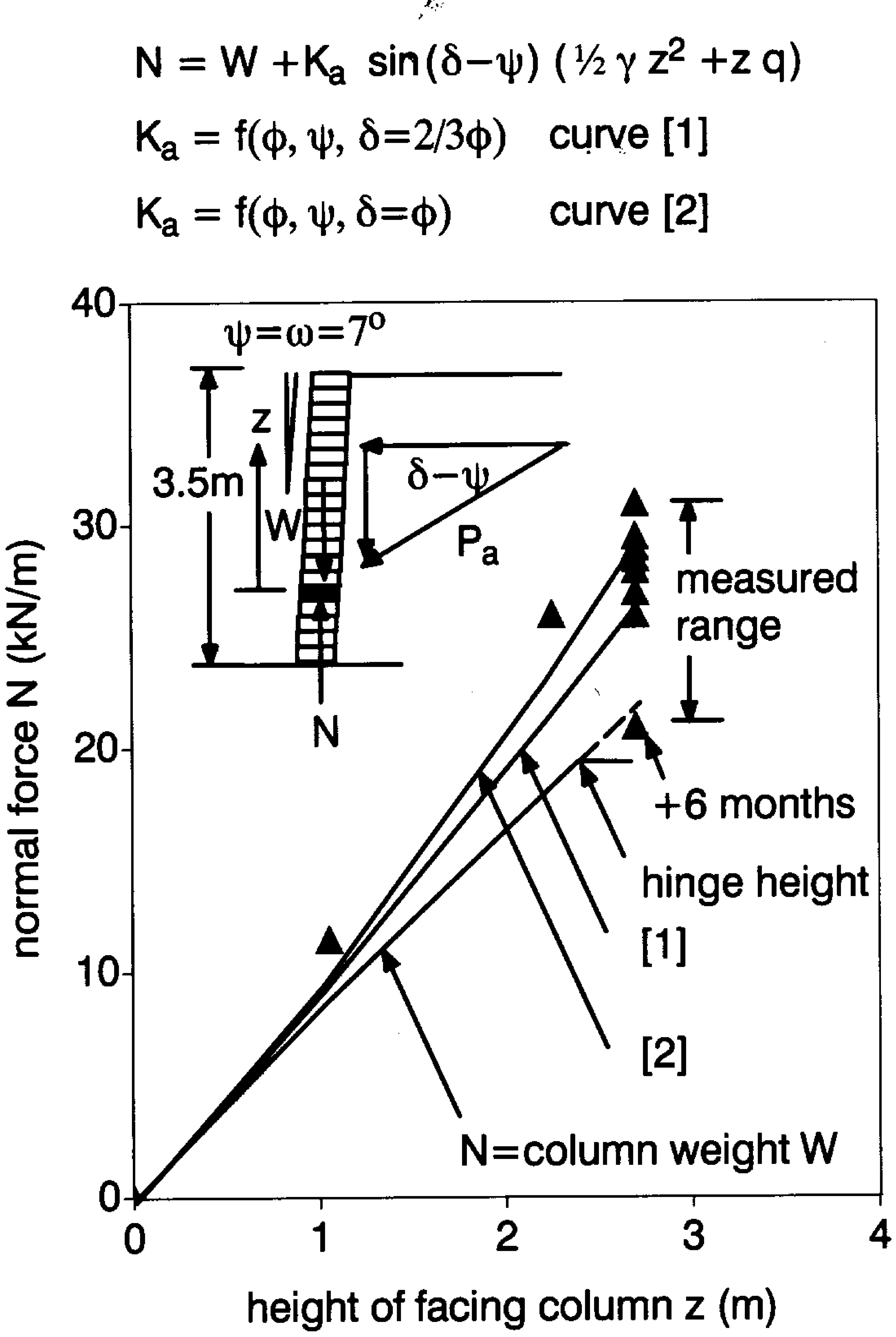


Fig. 29 Comparison of normal force measured within column facing with predicted values (UWP wall).

to occur at the connections under working load conditions. The data reported here indicates that an advantage of the relative flexibility of modular concrete facing systems over rigid panel systems is that modular column faces will relieve high connection stresses with time. Taken together, the data from the Algonquin wall and UWP wall support the conservative NCMA approach which is to neglect any down drag force in the calculation of interface shear and connection capacity for structures built on competent stiff foundations.

11 SEISMIC DESIGN

11.1 Performance

The discrete nature of the facing column in segmental retaining wall design has raised questions with respect to stability under seismic loading (Allen 1993). Quantitative data on seismic resistance is lacking although visual inspection of a 5.5m high segmental retaining wall after the Loma Prieta earthquake in California in 1989 did not reveal any signs of structural damage to the facing column nor was there visual evidence of movement (Eliahu and Watt 1991, Collin et al. 1992). Based on historical data for the California west coast and the location of the site with respect to the epicenter of the earthquake a reasonable upper limit on horizontal and vertical ground acceleration at the above site is 0.1g and .07g respectively. A more recent survey by Sandri (1994) of reinforced segmental retaining walls greater than 5m in height in the Los Angeles area after the recent Northridge earthquake in 1993 showed no evidence of visual damage to 9 of 11 structures located within 23 to 113km of the earthquake epicenter. Two structures showed tension cracks behind the reinforced soil mass that were clearly attributable to the results of seismic loading. Based on seismic data for the Los Angeles area the mean expected ground accelerations for the closest structure (7m high) was 0.2g and 0.13g in the horizontal and vertical directions respectively. However, no information is reported as to the direction of horizontal ground acceleration with respect to wall alignment.

11.2 Design

At the present time there are no specific guidelines in North America for design of segmental retaining walls subject to seismic loadings. However, current limit-equilibrium based (Coulomb) methods are easily extended to incorporate the additional inertial forces assumed to act during ground acceleration (i.e. Mononobe-Okabe approach) (Okabe 1926, Seed and Whitman 1970). This general approach has been adopted in a modified form in the FHWA guidelines (Chris-

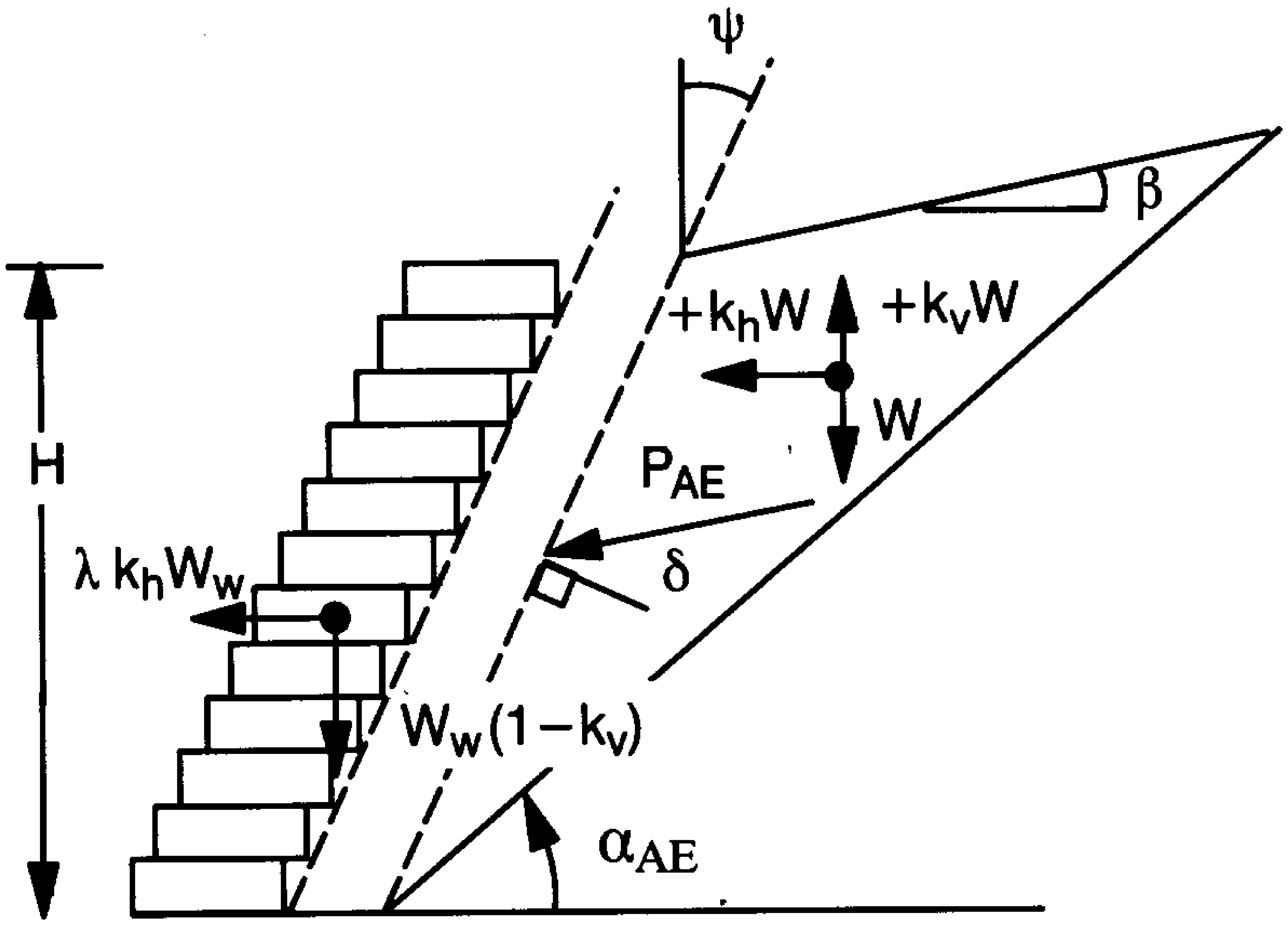


Fig. 30 Forces and geometry used in pseudo-static seismic analysis.

topher et al. 1989). Potential failure mechanisms for segmental retaining walls under seismic loading are the same as those identified in Figure 5. However, owing to the short duration of ground excitation and the expectation that peak horizontal and vertical ground accelerations for a gravity structure and retained soil zone will be out-of-phase during a seismic event, factors of safety employed in seismic design are typically taken as 75% of the values used in static analyses (Christopher et al. 1989). In addition, recent work on the cyclic response of HDPE and PET geogrids shows that these materials are stiffer at rates of loading anticipated under seismic loading (Bathurst and Cai 1994a). The implications of pseudo-static limit-equilibrium analyses to the stability of reinforced segmental retaining walls is currently under study at RMC and a number of analytical results from this work are described below.

11.3 Earth forces under seismic loading

The Mononobe-Okabe (M–O) method to calculate active earth forces acting on a planar surface that is inclined into the retained soil mass can be referenced to Figure 30. Quantities k_h and and k_v are horizontal and vertical ground acceleration ratios expressed as fractions of the gravitational constant. The calculation of dynamic earth thrust P_{AE} for a homogeneous soil is calculated as:

$$P_{AE} = \frac{1}{2}(1 \pm k_v)K_{AE}\gamma H^2 \qquad (13)$$

where γ is the unit weight of the retained soil and H is the height of the wall. The dynamic earth pressure coefficient K_{AE} can be calculated as follows:

$$K_{AE} = \frac{\cos^2(\phi + \psi - \theta)/\cos(\theta)\cos^2(\psi)\cos(\delta - \psi + \theta)}{\left[1 + \sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta-\theta)}{\cos(\delta-\psi+\theta)\cos(\psi+\beta)}}\right]^2} \qquad (14)$$

where ϕ is the angle of internal friction of the soil; ψ is the total wall inclination (from vertical); δ is the mobilized interface friction angle assumed to act at the back of the wall; β is the backslope angle (from horizontal); and θ is the seismic inertia angle given by:

$$\theta = \tan^{-1}\left(\frac{k_h}{1 \pm k_v}\right) \qquad (15)$$

The seismic inertia angle represents the angle through which the vectorial resultant of the gravity force and the inertial forces (both horizontal and vertical) is rotated from vertical. Equations (13) through (15) are an exact analytical solution to the classical Coulomb wedge problem that is modified to include the inertial forces k_hW and k_vW.

A distinguishing feature of the geometry of segmental retaining wall surfaces is that the surface against which active forces are assumed to act is oriented at $\psi > 0$ from the vertical. Hence wall surfaces are rotated in the opposite direction to that of most conventional gravity wall structures. Typically, this angle varies from 3 to 15 degrees depending on the setback of the stacked modular units. The influence of horizontal acceleration k_h, and wall inclination angle ψ on dynamic active earth forces is illustrated in Figure 31. The curves illustrate that the effect of positive wall batter (according to our terminology) is to reduce dynamic earth pressures to levels less than those developed against conventional gravity wall structures of the same height and retaining the same frictional soil. The example data in the figure shows that by increasing the wall batter from 0 to 15 degrees the dynamic pressure is reduced by 25% during a 0.2g event.

11.4 Results of stability analyses

The active earth force calculated according to equation (13) replaces the active earth pressure in the external and internal stability calculations described earlier for static analyses. However, interesting limits emerge when the pseudo-static approach is extended to factor of safety calculations for (local) interface shear and base sliding. Figure 32 shows the ratio of dynamic factor of safety to static factor of safety for these failure modes. The figures demonstrate that the reduction in factor of safety against translational modes of failure is relatively insensitive to the magnitude of vertical acceleration. However, the effect of even a modest horizontal ground acceleration is dramatic. For example, assuming that the total horizontal resistance to local sliding (R) is reasonably constant for the range of k_v values considered and the static factor of safety against shear failure is FOS_{static} =1.5 (as recommended by NCMA and FHWA) then the maximum ground acceleration to just cause local shear failure cannot exceed about 0.1 to 0.15g (Figure 32a). Consequently, the use of modular block systems with high shear capacity in seismic areas cannot be over-emphasized. A similar analysis can be carried out for base sliding as described in the FHWA guidelines (Christopher et al. 1989). The results are illustrated in Figure 32b. The figure shows that only a very modest ground acceleration is required to cause sliding failure of the reinforced soil mass (k_h = 0.1g). The prediction of modest ground accelerations to initiate mass sliding is consistent with the performance of the two walls reported to have developed tension cracks at the back reinforced zone during the recent Northridge earthquake. It should be noted that the type of block sliding analysis re-

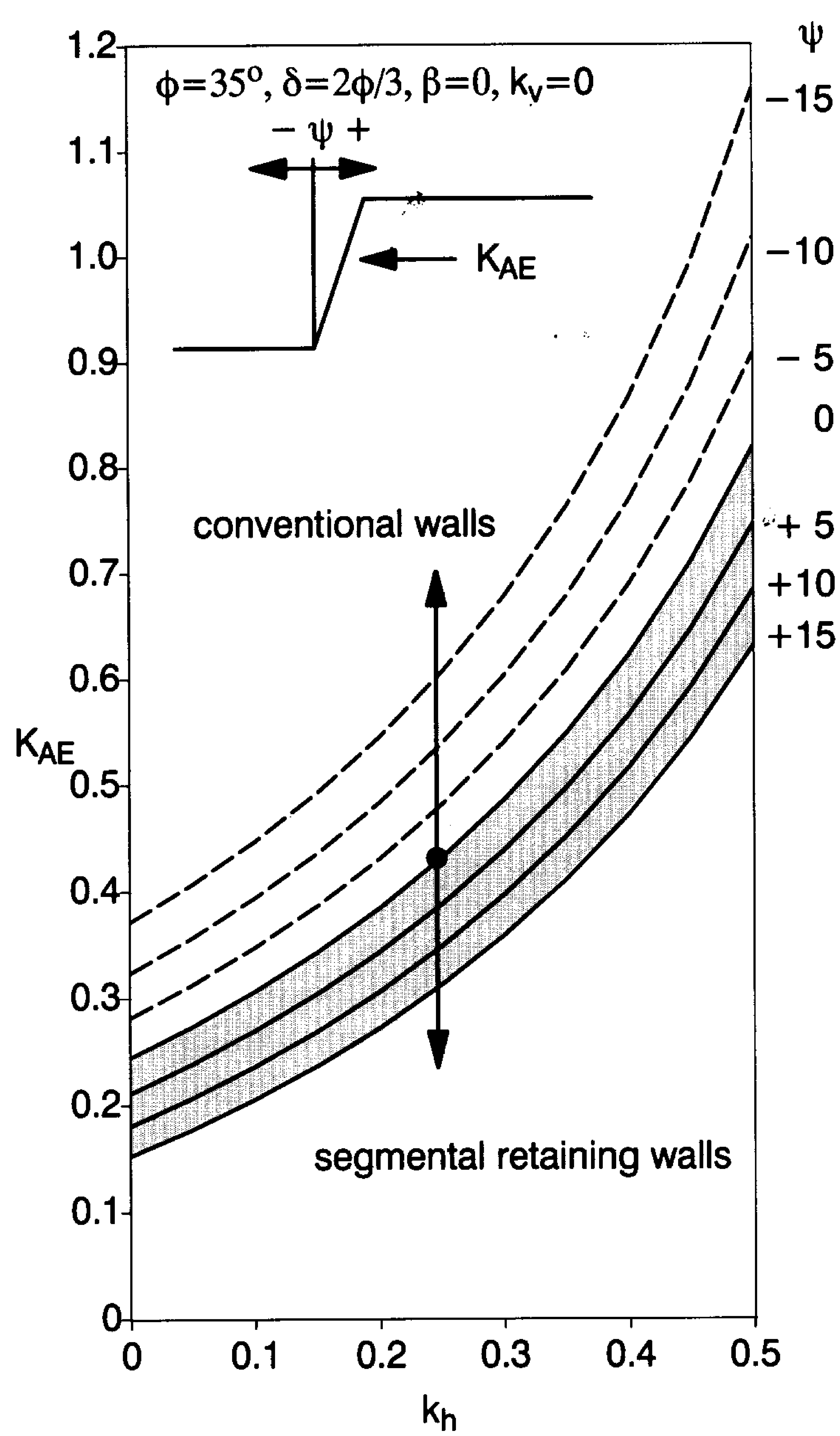
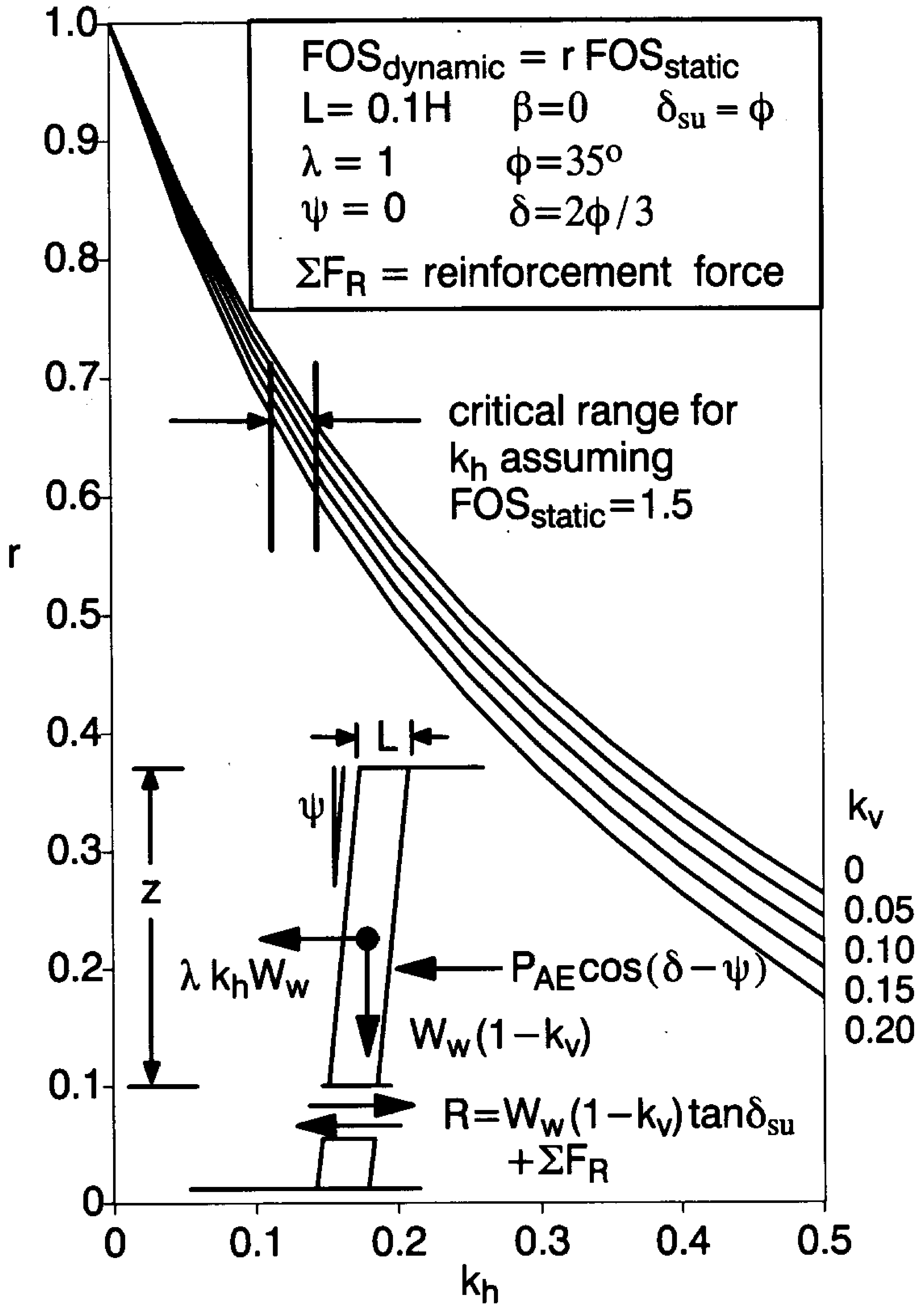
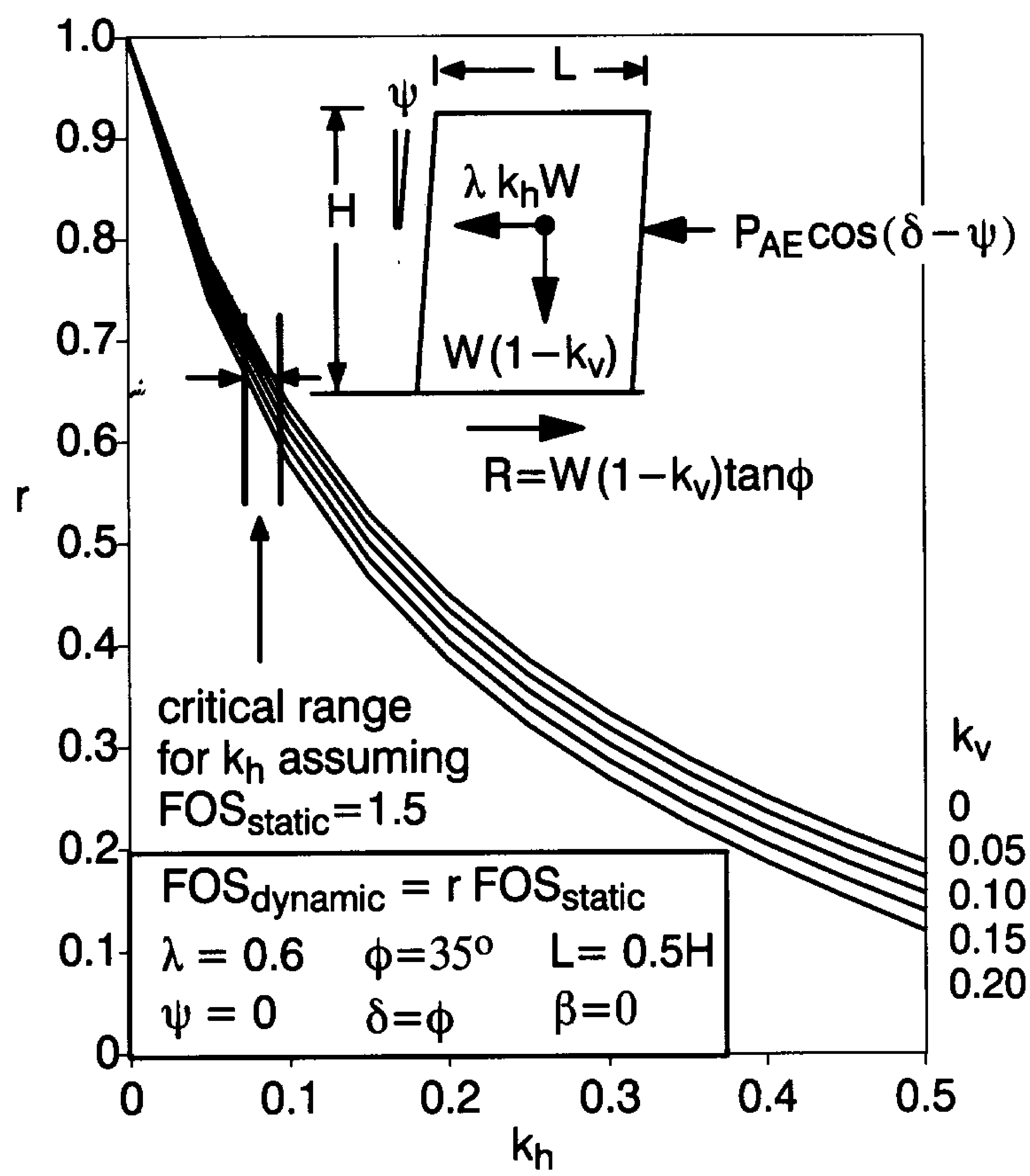


Fig. 31 Influence of horizontal ground acceleration and wall inclination on dynamic earth pressure coefficient.



a) influence of ground acceleration on interface shear



b) influence of ground acceleration on base sliding

Fig. 32 Influence of ground acceleration on interface sliding and base sliding.

ported here is independent of the type of gravity wall structure.

The very low values of ground acceleration required to initiate base sliding was originally pointed out by Richards and Elms (1979) who noted that there is very little margin of safety against base sliding of gravity structures designed to meet conventional static factors of safety. In the same paper, Richard and Elms have proposed a "limited displacement" method as an alternative to pseudo-static approaches of the type proposed in the FHWA guidelines. Nevertheless, translational body sliding as predicted in Figure 32b may not be catastrophic as reported by Sandri. The horizontal stability of segmental retaining wall structures may be improved by embedding the facing column and using facing units with high interface shear capacity or simply designing for a higher static factor of safety against sliding (i.e. increasing the length of the reinforced soil mass).

A closed-form solution for the orientation of the critical planar surface α_{AE} has been reported by Zarrabi (1979). The influence of horizontal ground acceleration on inclination angle is shown in Figure 33. The implication to internal stability design is that the length of reinforced layers, particularly near the top of the reinforced soil zone will have to be extended in order to provide a sufficient anchorage length under seismic loading (curves $\delta=2\phi/3$). The implication to external stability analysis is that the depth of the failure zone behind the reinforced soil zone increases with increasing horizontal ground acceleration (curves $\delta=\phi$). The use of modified M–O analyses for the design and analysis of segmental retaining wall structures is explored in detail in a forthcoming paper by Bathurst and Cai (1994b). Research is currently underway at RMC to investigate the validity of modified M-O methods for stability analysis of geosynthetic-reinforced soil walls using 1/6 scale models.

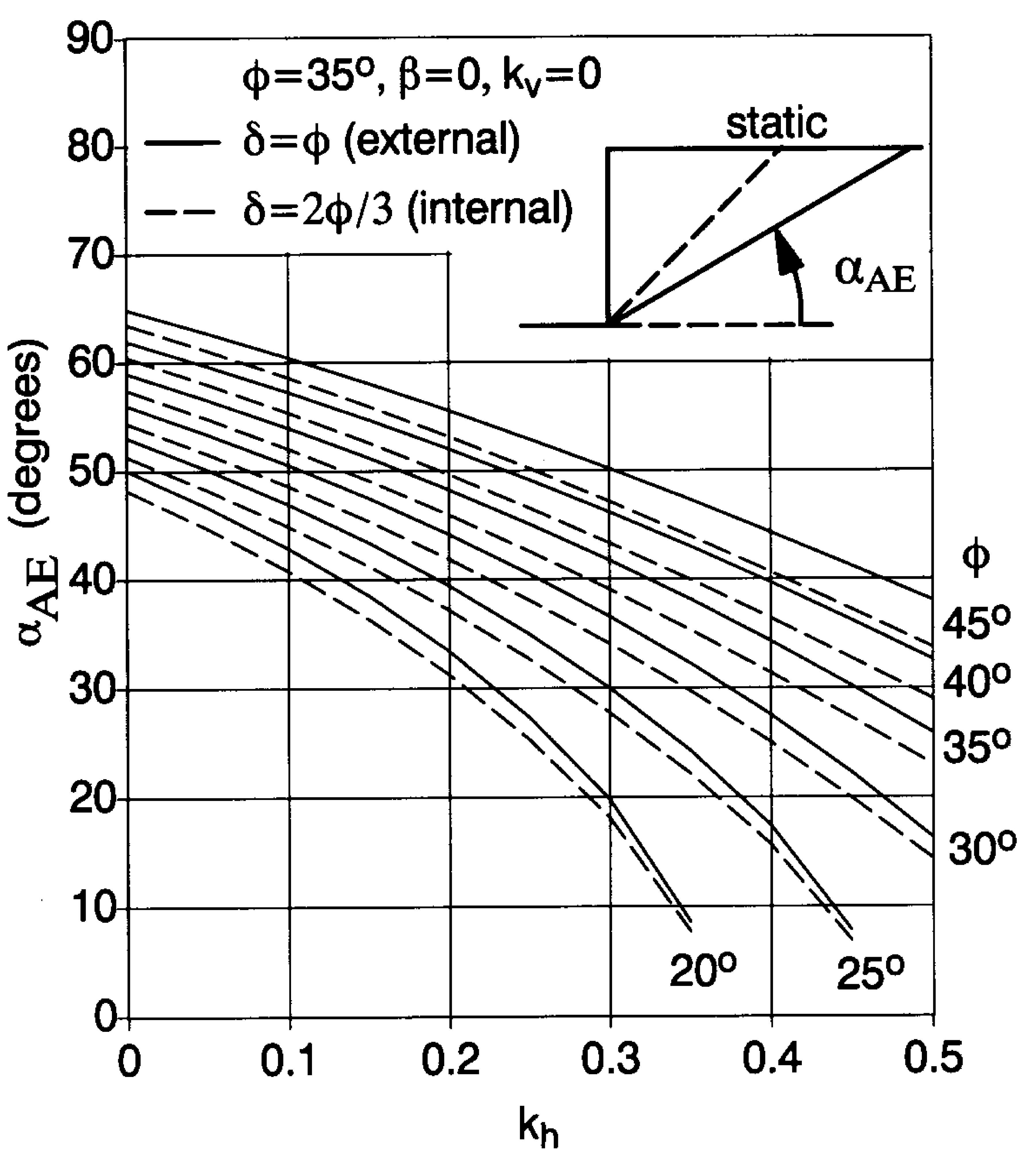


Fig. 33 Influence of horizontal ground acceleration and interface friction on orientation of internal failure plane.

12 CONSTRUCTION AND DURABILITY ISSUES

12.1 Drainage

Segmental retaining walls should be constructed with provision for good drainage. A drainage layer comprising a coarse single size stone is routinely recommended behind the modular units (Figure 3). Because of the shape of the modular units and small gaps between adjoining units a geotextile is recommended to prevent loss of materials through the facing. In addition, a geotextile may be required to act as a separator between the reinforced soil zone and the drainage column behind the facing units. The drainage column should be integrated with a drainage layer at the base of the structure. A gravity flow geotextile-wrapped pipe connected to outlets that direct water away from the foundation should be used. The use of drainage swales behind the wall and appropriate surface grading behind the wall crest should always be used to ensure that surface water is prevented from infiltrating the reinforced soil mass.

12.2 Segmental units

Modular concrete units have been observed to crack in segmental retaining wall structures. Some potential sources of cracking are illustrated in Figure 34.Vertical cracking may occur due to tensile failure of the concrete or diagonal cracks may be generated due to shear failure, particularly at the corners of modular units. The source of cracking may be due to differential settlement of the foundation and/or construction-induced cracking as described by Anderson (1993). Potential sources of construction-induced cracking are: horizontal surface irregularities generated during casting; inadequate cleaning of the top surface of the units; and overfilling of hollow units. These problems are overcome by strict attention to quality control during casting of the units and field construction. The importance of achieving a level foundation pad cannot be over-emphasized since any irregularities in the base course of modular block units will be carried up the height of the wall. For large wet-cast units, the use of a concrete footing may be desirable to ensure that differential settlements are minimized and the potential for large cracks avoided. Misalignment of modular units resulting in point contacts also has implications to available interface shear and connection capacity particularly for systems in which resistance to interblock sliding and connection capacity is developed through interface friction. Pinching of geosynthetic layers at point contacts and block edges can also be expected to reduce connection capacity. The authors experience has been that polyester geogrid materials have the advantage of being better able to conform to the interface geometry between modular unit courses. However, stiffer and generally thicker HDPE geogrids may be less susceptible to micro-damage caused by abrasion of the reinforcement between rough concrete surfaces. A strategy investigated by the authors to reduce abrasion has been the use of layers of nonwoven geotextile to cushion the geogrid reinforcement inclusion. In one project the use of a geonet to perform the same function has been reported (Anderson et al. 1991). However, the presence of a cushion material can be expected to influence interface sliding capacity and connection strength and therefore the cushion material must be present in the large-scale laboratory testing protocols described earlier in the paper.

In many cases the hairline cracks caused by uneven loading are not easily visible because of the broken textured appearance of the exposed face of many of the masonry units available on the market. While the appearance of any cracking may be undesirable from an aesthetic point of view, cracking may be considered desirable from a structural point of view since they are the result of stress redistribution through the facing column. Long-term monitoring of foundation settlement-induced cracks in the masonry units of a structure reported by Anderson (1993) showed that initial cracks did not grow in width with time even though differential settlements generated at the foundation elevation were as great as 300 mm over the 158 m length of wall. In another project, a segmental wall constructed with hollow granular infilled units recorded differential settlements of 200 mm over a wall length of 10 m with only minor cracking (Walls 1994). The distortional settlements in this last project were about 2%. Based on this experience, a preliminary upper limit on distortional settlements for segmental retaining walls with 300 mm high by 300 mm wide units (face dimensions) is 2% of the wall length. However, it is important to note that structures that have performed satisfactorily with respect to cracking after large differential settlements were constructed with units that have relatively thick concrete shells or were solid units. There are a number of hollow units on the market with relatively thin concrete shells. Cracking of these units has resulted in loss of the granular infill.

Durability of masonry concrete units under freeze-thaw cycles may be a concern in some locations. The potential for masonry concrete degradation may be increased in the presence of deicing chemicals from snow clearing operations for walls below or otherwise in proximity to roadways. Waterproof coatings have been proposed for masonry units in these environments. ASTM has recently published a standard for freeze-thaw testing of masonry concrete units (ASTM C 1262). Units may be tested either in water or in a 3% saline solution depending on the intended use of the units in actual service. Specimens are tested after each 8 to 12 freeze-thaw cycles and the procedure repeated until the loss in weight exceeds 10% of the initial saturated weight of the specimens. The NCMA is currently developing specifications for masonry concrete blocks in segmental retaining wall applications that are based on the results of the ASTM C 1262 method of test.

Specifications for the coping required at the top of concrete retaining wall structures to prevent water penetration can be found in AASHTO guidelines (1992)

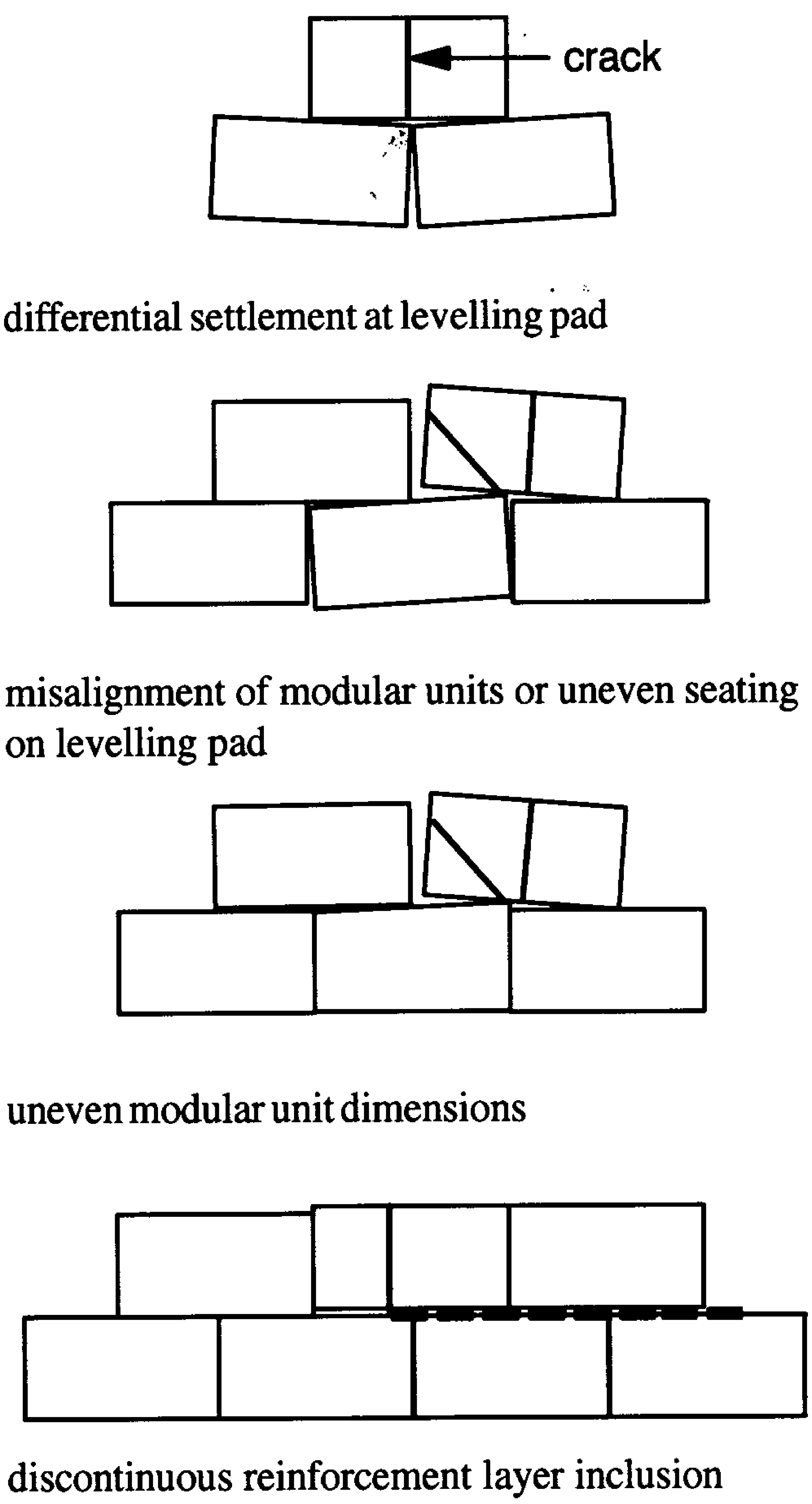


Fig. 34 Sources of modular block cracking.

13 CONCLUDING REMARKS

The paper has focused on recent developments in the design, construction and performance 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 vertical 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 authors 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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