New design philosophy of reinforced soil wall

New Horizons in Earth Reinforcement – Otani, Miyata & Mukunoki (eds)

Working stress design for geosynthetic reinforced soil walls

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ABSTRACT: The paper briefly reviews current practice with respect to working stress design (WSD) for geosynthetic reinforced soil walls with a focus on North America and Japan. A number of issues are identified which have implications to current and future WSD for geosynthetic reinforced soil walls.

1 INTRODUCTION

This paper is focused on major features of working stress design (WSD) methods for reinforced soil walls constructed with extensible polymer reinforcement. A common feature of working stress design is the use of a factor of safety applied to prescribed failure modes. This represents a classical geotechnical approach to the design of earth structures and, at least in North America and Japan, represents the current state of the practice. Within the brief space available a number of issues are identified which have implications to current and future WSD for geosynthetic reinforced soil walls and future methodologies based on a limit states design format.

2 BRIEF HISTORY

The first geosynthetic reinforced soil walls were built in France in 1970 and 1971 (Leflaive 1988, Leclercq et al. 1990, Puig et al. 1977). A review of early French experience can be found in the paper by Allen et al. (2002).

Geosynthetic reinforced walls have been in use in the United States since 1974. Bell and Steward (1977) describe some of these early applications, which were primarily geotextile wrapped-face walls supporting logging roads in the northwestern United States. The history of geosynthetic wall design in North America has been summarized by Allen and Holtz (1991) and Berg et al. (1998).

3 GUIDANCE DOCUMENTS

Limit states design methods have been developed in the UK (BS8006 – BSI 1995), Hong Kong (Geoguide 1 – Geo 1993), and Australia (RTA 2003). In North America the most recent issue of the AASHTO (2007) highway bridge design code uses a limit state design approach for these structures (called LRFD – load and resistance factor design). However, these methods have at their core the deterministic equations found in WSD. Furthermore, limit state design methods have been calibrated by fitting to WSD rather than a formal calibration using a rigorous reliability-based framework and measured load and resistance data.

In the USA, the most recent fully WSD-based AASHTO guidance document for reinforced soil walls is AASHTO (2002). In Canada, the CFEM (2007) guidance document published by the Canadian Geotechnical Society is available. The design manual by the National Concrete Masonry Association (NCMA 1997) is focused on the design of geosynthetic reinforced segmental (modular block) walls. At present, both these guidance documents adopt a WSD approach.

Based on examination of both WSD and limit-states design methods, it can be argued that the underlying general approach for the key deterministic equations for reinforcement load prediction resistance have not changed significantly over the last 30 years.

4 CURRENT WSD PRACTICE

In North American working stress design practice, factors of safety are assigned to failure modes that are broadly classified as external, internal or facing stability modes of failure (Figure 1). In this section the general approach is reviewed with some examples. However, the reader is advised that there are many variations in the details of method implementation between guidance documents.

External modes of failure treat the composite facing and reinforced soil zone as a monolithic block. Factor

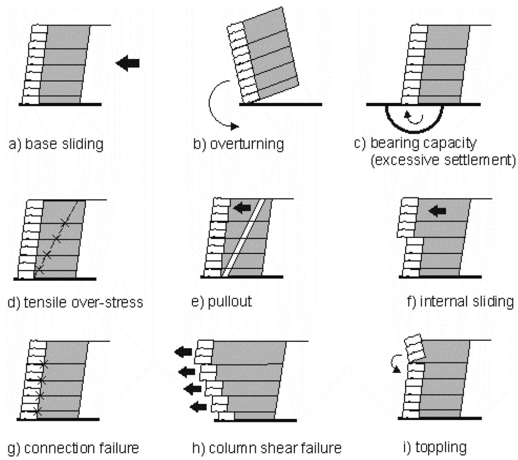


Figure 1. Modes of failure for geosynthetic reinforced soil walls: a), b), c) External; d), e), f) Internal; and g), h), i) Fac-ing (after CFEM 2007).

of safety expressions are limit equilibrium-type and are the same as those used for conventional gravity wall structures. The total active earth force is calculated using the horizontal component of Rankine or Coulomb earth force. Coulomb theory has the advantage of explicitly including the influence of wall batter. Interface friction, δ, between the wall facing and soil is often taken as zero, and all soil volumes above the wall crest are treated as an equivalent uniform surcharge. The zero interface friction assumption is consistent with US design codes and practice. However, it is recognized that this assumption is likely conservative for the calculation of active earth forces used in internal stability design. The geometry and forces associated with external modes of failure are summarized in Figure 2. The minimum length of the reinforced soil zone taken from the face of the structure (L) is typically prescribed as $L/H = 0.6$ to 0.7 regardless of the magnitude of factors of safety against overturning or sliding.

In North America, the Simplified Method (also known s the Tie Back Wedge Method) is used to compute reinforcement loads and to establish minimum anchorage lengths in internal stability design and to compute loads acting on the facing. This is a limit equilibrium approach, and contains two key assumptions for the calculation of reinforcement load:

1. The magnitude of tensile load in each reinforcement layer is proportional to the soil overburden stress. Hence, reinforcement load will increase linearly with increasing depth of soil below the crest of the wall.
2. Tensile load in the reinforcement is a direct indicator of the state of stress in the soil since the reinforcement layer is assumed to carry the full lateral active earth pressure in the soil in the vicinity of the layer (i.e. contributory area approach).

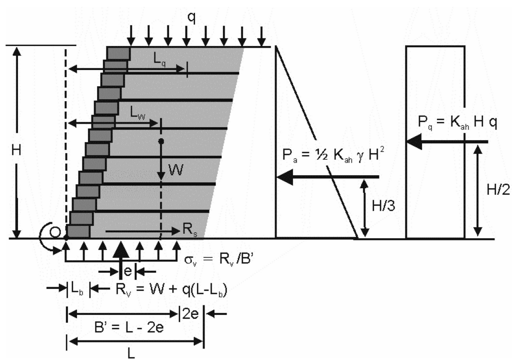


Figure 2. Free body diagram for external stability calculations (after CFEM 2007).

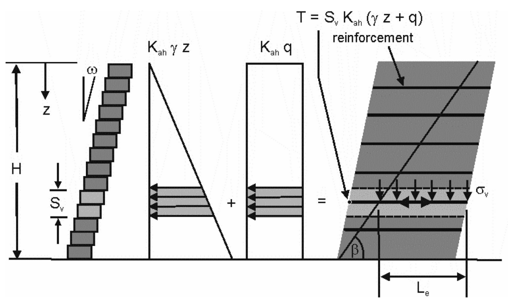


Figure 3. Free body diagram for internal stability calculations (after CFEM 2007).

The expression for the maximum reinforcement load is:

$$T_{max} = S_v K \sigma_v = S_v K(\gamma z + q) \quad (1)$$

where K is the lateral earth pressure coefficient computed using Rankine or Coulomb earth pressure theory, σ_v is the vertical pressure acting at the reinforcement layer located at depth z below the crest of the wall, S_v is the reinforcement spacing, γ is the unit weight of the soil, and q is a uniformly distributed surcharge pressure. It can be noted that BS8006 (1995) calculates the vertical stress σ_v using a Meyerhof approach. This increases the vertical stress at the level of each reinforcement layer and thus the tensile load assigned to the layer is larger compared to the AASHTO approach. In view of the comments made later in this paper, it may be argued that the Meyerhof approach adds to additional conservatism with respect to design against reinforcement overstressing.

The predicted maximum load level T_{max} in a reinforcement layer using Japanese practice PWRC (2000)

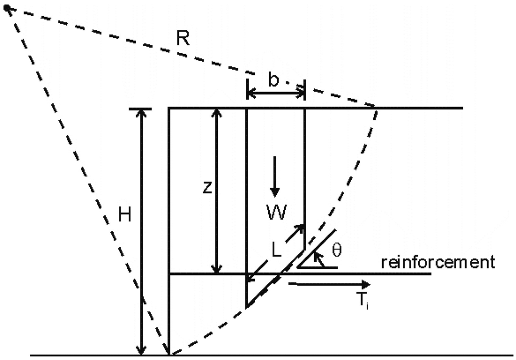


Figure 4. Circular slip analysis showing parameters used to compute required tensile load capacity of reinforcement layers.

is also calculated using Equation 1. However, K is computed as:

$$K = \frac{2 \ \Sigma T_{req}}{\gamma \ H^2} \tag{2}$$

Here the required minimum sum of tensile load capacity for the reinforcement (ΣT_{req}) intersecting a circular failure surface is calculated as:

$$\Sigma T_{req} = \max \frac{F_s \ \Sigma(W \sin\theta) - \Sigma(cL + W\cos\theta \tan\phi)}{\Sigma\left\{\frac{2}{H^2} z\, b \tan\theta\,(\cos\theta + \sin\theta \tan\phi)\right\}} \tag{3}$$

The other parameters used in the conventional circular slip analysis for an unreinforced soil mass (Bishop's Method of Slices) are shown in Figure 4. The recommended factor of safety is $F_s = 1.2$. Miyata and Bathurst (2007a) showed that setting $F_s = 1.0$, $q = 0$ and assuming a purely frictional soil gave the same the values for T_{max} as the AASHTO Simplified Method. Nevertheless, this factor should not be interpreted as being equivalent to the overall factor of safety FS used in Equation 4 (discussed below) in the context of the AASHTO method. In the Japanese approach, the factor of safety term F_s applies only to uncertainty related to the load side.

The allowable long-term tensile load of the reinforcement T_{allow} according to AASHTO (2002) and PWRC (2000) practice can be calculated as:

$$T_{allow} = \frac{T_{ult}}{FS \times RF_{design}} \tag{4}$$

where FS is the overall factor of safety and the term $RF_{design} = RF_D \times RF_{CR} \times RF_{ID}$ is the product of partial factors to account for installation damage ($RF_D \geq 1.1$), creep ($RF_{CR} \geq 1.2$ – typically) and environmental degradation ($RF_{ID} \geq 1.1$), respectively. For the AASHTO method, FS = 1.5 in Equation 4.

For current Japanese practice, the denominator in Equation 4 is calculated using FS = 1.0 and $RF_{design} = 1.67$ or 3.33 depending on the reinforcement. It can be argued that the factor (F = 1.12 or 1.21) used to reduce the average ultimate strength value from tensile tests, is qualitatively similar to the overall factor of safety term FS in the AASHTO method. However, it must be pointed out that FS in the AASHTO method accounts for uncertainty that is related to both the load and resistance side of internal stability design equations while in the Japanese approach, F is related only to the resistance term (reinforcement capacity).

With the exception of wrapped-face walls, loads are transmitted from the reinforcement layers to the facing (Figure 1g). Where these facings are structural (e.g. concrete panel or modular block), the connections must be designed to have adequate design capacity computed as:

$$T_{conc} = \frac{T_{ult} \times CR_u}{FS \times RF_{CR} \times RF_D} \tag{5}$$

where FS = 1.5 is the overall factor of safety. The other partial factors are for installation damage ($RF_D \geq 1.1$) and creep ($RF_{CR} \geq 1.1$). The reduction factor CR_u is determined from connection tests and is the ratio of connection strength to the index rupture strength of the intact reinforcement. For static loading cases using AASHTO design, the connection load is computed using the Simplified Method described earlier and without modification (i.e. connection loads are not increased or decreased from maximum internal tensile design loads). The value for RF_{CR} should be based on creep connection test (FHWA 2001). However, typically, this value is taken from creep-reduced strength values determined from in-isolation creep-rupture data.

5 SOME ISSUES WITH RESPECT TO DESIGN OF GEOSYNTHETIC REINFORCED SOIL WALLS

5.1 *Accuracy of load predictions*

A fundamental feature of current WSD approaches is the scaling of failure loads and resistance at limit-equilibrium to working stress conditions using one or more factors of safety or partial factors as described in the previous section. However, the stresses at incipient collapse cannot be simply scaled to working stress conditions in this manner.

Predicted versus measured T_{max} values are plotted with normalized depth below the crest of the wall in

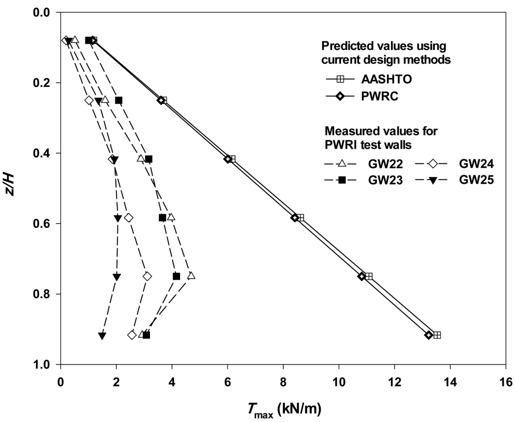


Figure 5. Predicted versus measured values of T_{max} using AASHTO (2002) and PWRC (2000) design methods and PWRI case histories (Miyata and Bathurst 2007a).

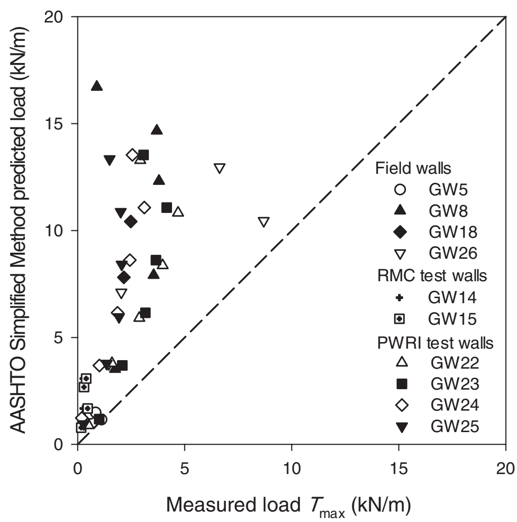


Figure 6. Predicted versus measured values of maximum reinforcement load using the AASHTO (2002) Simplified Method and structures with frictional backfill soils (Miyata and Bathurst 2007a).

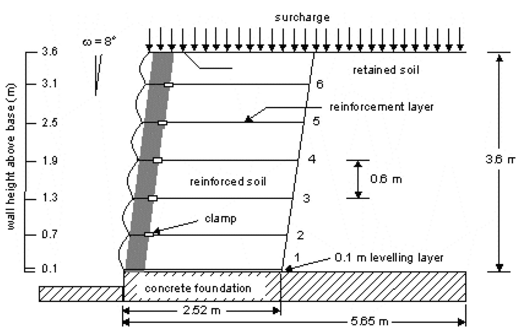


Figure 7. Cross-sections of nominal identical walls with flexi-ble wrapped-face and hard-faced modular block construction (Bathurst et al. 2006).

Figure 5 for a set of 6 m-high PWRI walls in Japan that varied only with respect to facing type. The figure shows that for practical purposes both AASHTO (2002) and PWRC (2000) design methods give the same load predictions but are conservative with respect to measured loads.

Allen et al. (2002) and Miyata and Bathurst (2007a) carefully estimated the loads in reinforced layers from a large database of instrumented and monitored full-scale field and laboratory walls that used a frictional backfill. The loads were compared to predicted values using the Simplified Method. The comparisons were made using an estimate of the peak plane strain friction angle for the soil in each case. This required increasing the friction angle from triaxial or direct shear tests using published equations. The data are plotted in Figure 6. The measured loads are taken within the soil backfill away from the connections which may be locally higher as discussed later in the paper. The data show that predicted loads are consistently greater than the measured loads. Miyata and Bathurst showed that the ratio of measured to predicted loads was on average was about one third. This means that current WSD design methods are conservative with respect to predicting measured loads under operational conditions by at least a factor of three. This ratio is even greater if peak triaxial or direct shear strengths are used to compute predicted reinforcement loads since these values are typically less than plane strain values.

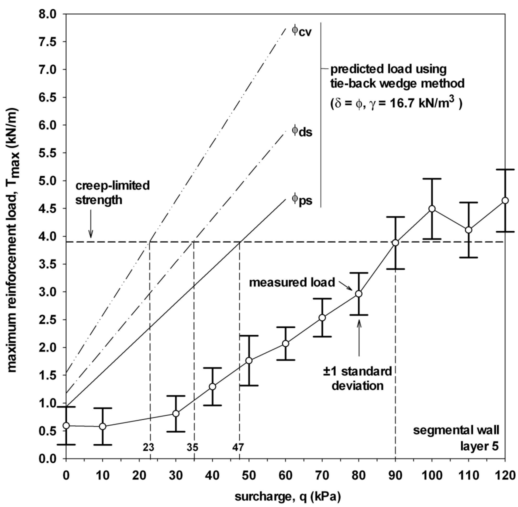
Bathurst et al. (2006) explored the accuracy of the Simplified Method with respect to influence of selection of soil friction angle to be used in reinforcement load computations and to identify those conditions when the general approach may be expected to give accurate estimates. They compared the reinforcement loads measured in 3.6-m high full-scale reinforced soil models constructed in the RMC Retaining Wall Test Facility with computed values using the Simplified Method. The walls were nominally identical except one was constructed with a column of dry stacked solid masonry concrete units and the other with a very flexible wrapped-face construction (Figure 7). They showed that for the most critical reinforcement layer in the wrapped-face wall, the measured and predicted reinforcement loads were in reasonable agreement provided that the peak plane strain angle was used. The discrepancy between predicted and measured loads increased in the order of peak direct shear friction angle and constant volume friction angle. For the companion wall with a hard (structural) facing, the Simplified Method grossly over-estimated the measured reinforcement loads even when the peak plane strain friction angle of the soil was used. The last observation is consistent with the database of results for a large number of field and laboratory walls that show that current limit-equilibrium methods are excessively conservative when reinforcement loads are computed for walls with structural facings. This is because the wall facing resists earth pressures, and load capacity is not due to just the mobilized shear strength of the soil and mobilized tensile capacity of the reinforcement. This is particularly true under operational conditions when the wall deformations are least and mobilized extensible reinforcement tensile capacity is low, as opposed to incipient (limit equilibrium) collapse conditions.

5.2 *Residual strength of geosynthetic reinforcement*

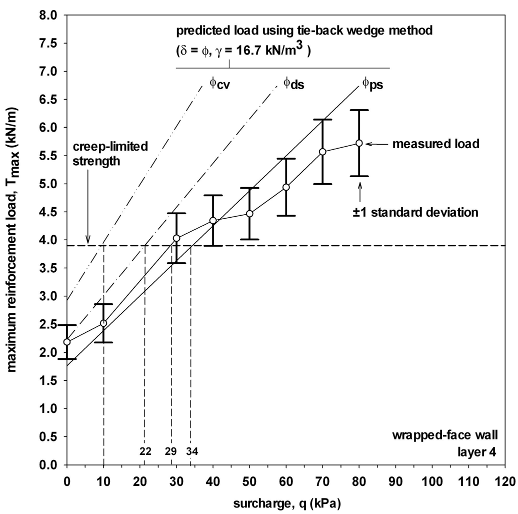
Conventional practice with regard to the selection of creep-limited strength values is to develop creep-rupture curves. Bernardi and Paulson (1997) and Greenwood (1997) summarized observations from the results of index tensile tests carried out on geosynthetic reinforcement materials after long-term creep loading. They concluded that the rupture strength reduction of PET and polyolefin reinforcement products does not vary linearly with logarithm of time. Rather, the residual index strength of polymeric reinforcement products is always greater than what is assumed based on conventional log-linear creep-rupture curves. Residual strength curves for materials with an index tensile strength, T_o, are illustrated in Figure 9. The residual strength curves are assumed to intersect the conventional creep-reduced strength curve at static and dynamic design strength values, T_{DS} and T_{DD}, respectively. In North American practice the design load under seismic loading can be increased by 33%. Hence, $T_{DD} > T_{DS}$ in this figure. Importantly, a reinforcement layer at a value of T_{DD} can be expected to have an available residual strength $T_{RDS} >> T_{DD}$. This additional strength is not considered in current limit-equilibrium methods of design and is a potential source of conservatism. An implication of observations reported in this section to seismic design is that the available strength and stiffness of geosynthetic reinforcement products under earthquake loading is not less than conventional estimates of available reinforcement strength in static load environments and may indeed be very much greater.

5.3 *Connections*

As noted earlier, current practice is to assume that connection loads at the facing of a wall are the same as those computed for internal stability design (i.e. for tensile over-stress and for pullout). However, there is evidence from monitored walls that connection loads are the highest loads in a layer of reinforcement. This



a) hard-faced wall



b) flexible wrapped-faced wall

Figure 8. Predicted and measured maximum reinforcement tensile loads at the end of construction and during surcharging for flexible wrapped-face wall and stiff-face segmental retaining wall (Bathurst et al. 2006).

can be attributed to relative downward movement of the soil backfill with respect to the relative vertically stiffer facing column or panel as the wall facing moves outward and as the soil is compacted and settles.

Figure 10 shows normalized peak strain values collected by the writer and colleagues from a total of 16 instrumented field walls. The range bars in the figure represent ± 1 standard deviation on the mean of data sets grouped according to distance intervals. The local peak at the free end of the reinforcement

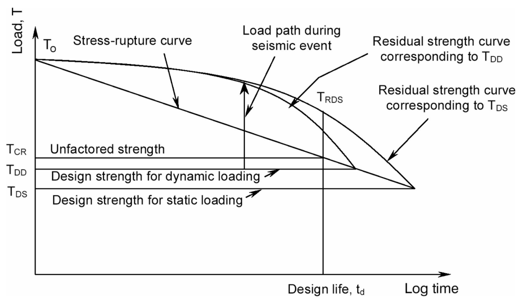


Figure 9. Concept of residual strength available to rein-force-ment layer under static and dynamic loading (Bathurst et al. 2002).

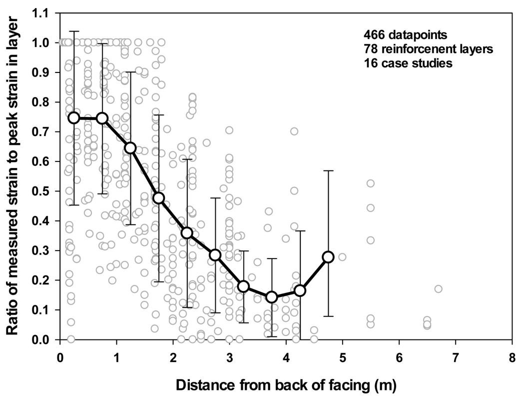


Figure 10. Normalized peak strain values for instrumented full-scale field walls constructed with a hard facing.

layers is likely an artifact of the limited data available at distances greater than about 4.5 m from the back of the facing column. The figure shows a clear visual trend in support of the hypothesis that connection loads are, on average, the highest loads in a reinforcement layer attached to a hard facing. Hence, if the estimate of maximum reinforcement is assumed to be correct, the same estimated load for the connection is non-conservative. Fortunately, internal tensile loads appear to be excessively over-designed and this may explain why connection failures are not systemic in these types of structures.

An example set of data for a creep connection test carried out in accordance with the recommendations found in FHWA (2001) is presented in Figure 11. The reinforcement material in this study was a biaxial coated woven polyester geogrid. It was connected to commercially available hollow masonry blocks with a discontinuous shear key. Hence the connection was a frictional-mechanical type. The connection-creep test protocol calls for a series of constant loads to be applied to the free end of the reinforcement and the

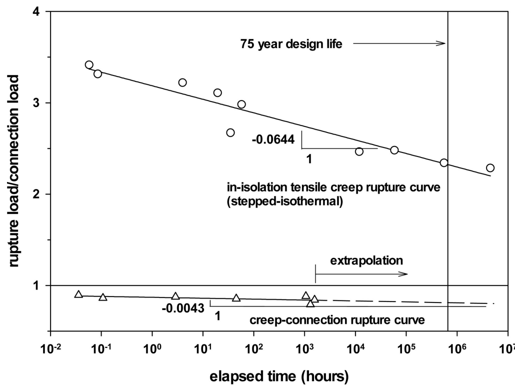


Figure 11. Normalized creep rupture curves for reference data and from creep-connection testing for woven polyester geogrid in combination with a typical hollow masonry facing unit.

time to rupture recorded for each test. The tensile loads are selected to generate a series of data points up to 1000 hours. The data presents as a straight line on a conventional semi-log plot. The curve can be extrapolated to a prescribed design life – in this example equal to 75 years. In this particular plot all tensile rupture loads have been normalized with respect to the index connection strength from a rapid constant rate of displacement test as described in the conventional ASTM D6638 standard. This has been done to focus on the qualitative features of the data rather than the actual block and geogrid materials used in the testing. Superimposed on the plot is the result of conventional creep-rupture data reported by the manufacturer for the same product. The important observations that can be made from this plot are that the index connection load is less than the short-term isolation strength. This is almost always the case for block-geogrid connections. However, the log-linear creep rate is much less for the connection test. This is, in the experience of this writer very typical. Hence, conventional practice which is to use the slope of the in-isolation creep data to reduce index connection test results for creep is conservative.

5.4 *Use of cohesive-frictional backfill soils*

In North America, the use of frictional well-draining backfill soils is preferred and recommended particle size distributions are presented in the guidance documents cited earlier. These granular materials are desirable because they have relatively high strength and stiffness, are easy to compact and are free-draining. However, in many cases and particularly on a worldwide basis, these good quality soils are not available or are cost-prohibitive. Nevertheless, many walls constructed with cohesive-frictional soils have performed well. Current practice is often to ignore the cohesive strength component of a backfill soil and to carry out load computations based only on the frictional component of soil strength. The argument offered is that the cohesive strength component may be attenuated due to moisture content increases and hence may not be available for the life of the structure. However, this may lead to conservative design for structures in which moisture content increases are not permitted to occur due to good drainage design and implementation.

Miyata and Bathurst (2007b) proposed a method to compute an equivalent secant friction angle from laboratory direct shear and triaxial c-ϕ Mohr-Coulomb shear strength data that can be used in Simplified Method calculations. Their approach removes a portion of the conservatism that occurs by simply setting $c = 0$ in load calculations. Nevertheless, similar to experience with walls constructed with purely frictional soil backfill, they showed that the Simplified Method still results in excessively conservative estimates of reinforcement loads based on measured load values from a database of monitored walls constructed with c-ϕ soils. For example, ratios of measured to predicted reinforcement loads were in the range of 1/3 to 1/5. Again, this may explain the good performance of many of these walls even for cases when they may have been poorly compacted and/or the soils have been wetted up due to poor soil surface drainage management.

6 CONCLUSIONS

This paper provides a brief review of some key aspects of current conventional working stress design (WSD) for geosynthetic reinforced soil retaining walls. This technology has been largely driven by economics since it has been shown that these systems can be constructed at up to 50% of the cost of conventional gravity wall systems (Koerner et al. 1998). Nevertheless, test methods and design methods have lagged. In particular, the distribution and magnitude of reinforcement loads predicted using conventional Tie Back Wedge Methods of analyses are likely excessively conservative. Fortunately, the writer and collaborators have collected over a number of years a database of wall performance data that can be used to formulate calibrated design methods that preserve features of conventional approaches but hold promise to better predict reinforcement loads. A companion paper by Bathurst et al. (2007) that appears in this conference proceedings describes recent developments in the K-stiffness Method which is a step in this direction.

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