DISCUSSION / DISCUSSION

Reply to the discussions on “The influence of facing stiffness on the performance of two geosynthetic reinforced soil retaining walls”[1]

Richard J. Bathurst, Nicholas P. Vlachopoulos, Dave L. Walters, Peter G. Burgess, and Tony M. Allen

The authors appreciate the comments raised by the discussers and welcome the opportunity to clarify a number of points related to the details of the test program described in the referring paper and the implications to current practice.

Response to points raised by Leshchinsky

The discusser raises concerns regarding the applicability of the restrained toe boundary condition used at the base of the toe.

In our segmental retaining wall, the facing column was constructed on a steel plate seated on roller bearings (Fig. 3). For brevity, details of this arrangement were not repeated from the very detailed descriptions that can be found in the cited companion papers by Hatami and Bathurst (2005, 2006). This plate (footing) was restrained by a series of compliant horizontal load rings, which were very stiff compared to the case of no horizontal toe restraint for the wrapped-face wall. Without the horizontal restraint provided by the load rings at the base of the block-faced wall, it was impossible to place and compact soil behind the facing. In other words, the blocks would have translated horizontally as soon as any lateral earth force was applied. Clearly, the toe of the wall provides lateral restraint. In the field, this restraint is provided by embedment and shear resistance at the interface between the bottom block and the leveling pad. The actual amount of horizontal compliance that exists in the field is less than the idealized case of a fully restrained (zero displacement) boundary.

In fact, there was a small horizontal compliance at the toe of the wall due to a row of horizontal load rings and beam support at the base of the wall (Fig. 3) whose magnitude is quantified and described in the companion paper by Hatami and Bathurst (2005) and cited in this referring paper. The horizontal resistance was estimated to be equal to a stiffness of 4 MN/m/m, which is 10 times less than the value of 40 MN/m/m, back-calculated from physical block–block shear tests. Hence, with respect to the rest of the column, the least stiff portion of the column was the toe. From the point of view of the two walls described in the referring paper, the block-faced wall was clearly restrained at the toe compared to the wrapped-face wall.

It follows that the toe restraint used in our model cannot be rejected out of hand as being improbable. If retaining walls could be built instantaneously (as is often assumed in numerical modeling), then a laterally unrestrained footing is likely adequate. However, as a result of the way these systems are constructed in compacted lifts, load is mobilized at the toe of any block-faced wall.

In the literature, the hard facing is often described as a formwork to ensure the target facing geometry of the wall is achieved. A formwork cannot perform its function unless it is capable of developing resistance to soil load during construction, which includes compaction. It is not possible to develop interface shear resistance between blocks over the lower sections of the wall without resistance at the toe.

The authors disagree that the bottom boundary is unrealistic with respect to the field. For example, it is common for propped panel walls to be pinned at the base on poured concrete footings (Bathurst 1991). Block-faced walls are typically placed on a gravel leveling pad and are buried. A hard-faced wall without lateral resistance at the base is, in the experience of the authors, a very unusual case. Furthermore, there is evidence that there is translational sliding at the block interfaces. Hence, wall-deformation profiles may appear to be rotational over the height of the wall, but these deformation profiles are mainly due to the relative sliding of the blocks. This is because the blocks have a small shear compliance at the interfaces. In all RMC block-wall tests, there was no evidence of the back of the

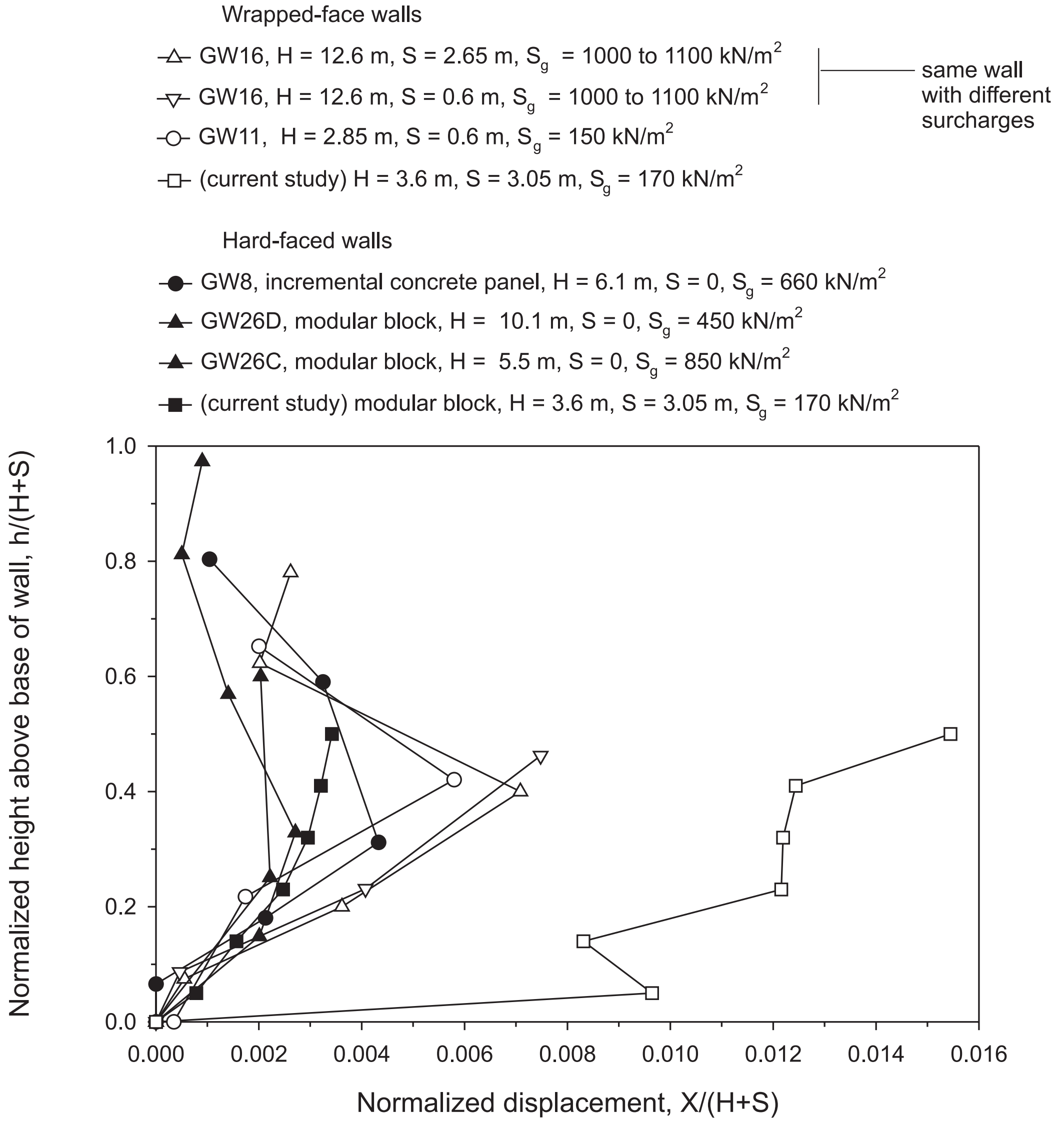
Received 18 April 2007. Accepted 30 September 2007. Published on the NRC Research Press Web site at cgj.nrc.ca on 4 February 2008.

R.J. Bathurst,[2] N.P. Vlachopoulos, and P.G. Burgess. GeoEngineering Centre at Queen’s-RMC, Department of Civil Engineering, Royal Military College of Canada, Kingston, ON K7K 7B4, Canada.
D.L. Walters. Golder Associates Ltd., Mississauga, ON L5N 5Z7, Canada.
T.M. Allen. Washington State Department of Transportation, State Materials Laboratory, Olympia, WA 98504-7365, USA.

[1]Appears in Canadian Geotechnical Journal, **44**: 1479–1482 and **44**: 1483.
[2]Corresponding author (e-mail: Bathurst-r@rmc.ca).

 doi:10.1139/T07-102

Fig. D1. Measured relative displacements for selected full-scale hard-faced and wrapped-face walls. Measurements are taken with respect to installation of each instrumentation point. H = height of wall, h = height above toe, S = equivalent surcharge height, X = displacement, and S_g = global stiffness. Wall designations are consistent with the ones in Allen and Bathurst (2002), Allen et al. (2003), and Miyata and Bathurst (2007*a*, 2007*b*).



bottom row of blocks lifting off the footing, based on the measured loads from parallel rows of the vertical load cells. Example toe- and heel-load histories were reported by Bathurst et al. (2000) and cited in the referring paper. The reference paper serves as a background paper to the larger study. The same paper, as well as the papers by Hatami and Bathurst (2005, 2006), reports connection loads and hence there was no attempt to report selective data (as suggested by the discusser) other than to focus the scope of the paper.

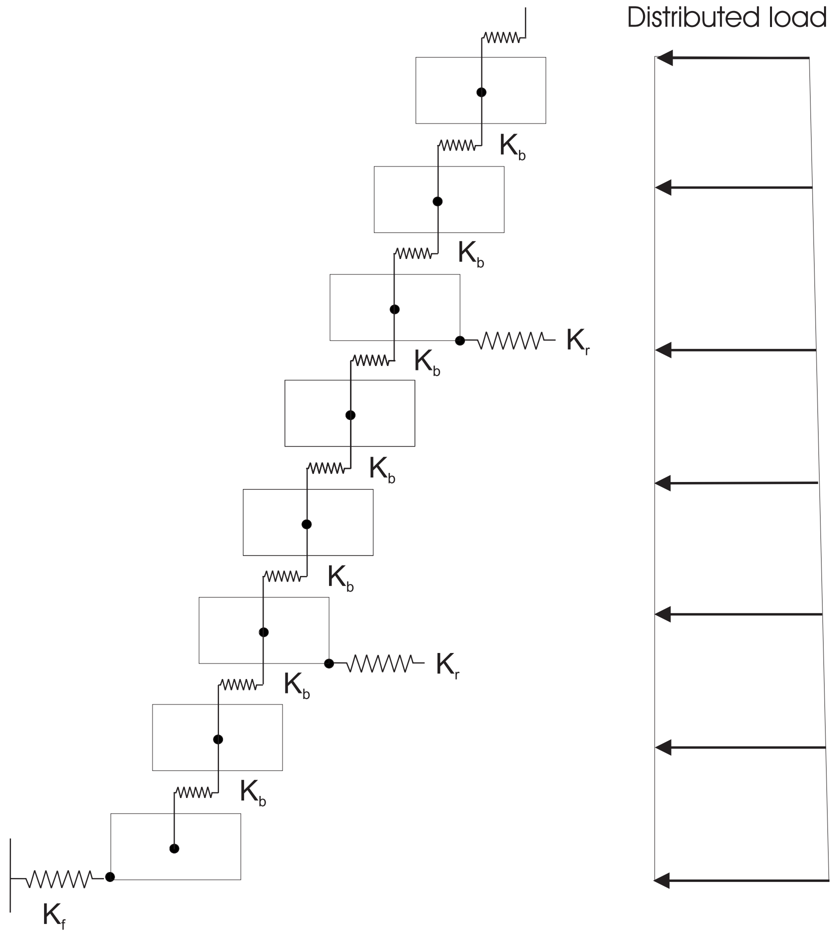
The discusser comments that common practice is to ignore wall facing embedment during design.

This is a design decision, not a performance feature. This design decision was made to simplify calculations with the expectation that this was conservative for design and thus was safe. The conservativeness of this decision was noted by the authors of the original NCMA design manual (Bathurst et al. 1993*a*). Walls are almost always embedded or should be according to AASHTO and NCMA. Hence, the ability of a wall facing to transmit load to the foundation is possible. In some cases, walls are built on buried concrete footings that are very stiff and clearly offer lateral restraint.

The discusser challenges the use of the term 'nominally identical' with respect to the two walls under investigation.

The term was used in the paper to convey to the reader that the two walls are, for practical purposes, the same with the exception of the facing. They have the same number of layers, reinforcement type, and compaction method. The difference was the facing type and facing construction. The authors stated clearly in the paper that the wrapped-face construction was not the same as what would be constructed in the field and pointed out the consequences of this devia-

Fig. D2. Illustration of analog free-body wall facing subjected to a distributed lateral load and with variable internal interface shear stiffness and boundary stiffness restraint.



tion. Rather, the facing was purposely constructed in the manner described in the paper to generate a condition that is as close as possible to the idealized conditions implicit in current tie-back wedge methods of analysis.

The discusser raises questions regarding the special construction of the wrapped-face wall and the consequences of this construction on wall displacement. Similar concerns are expressed regarding the effect on displacements due to the toe restraint on the block-faced wall.

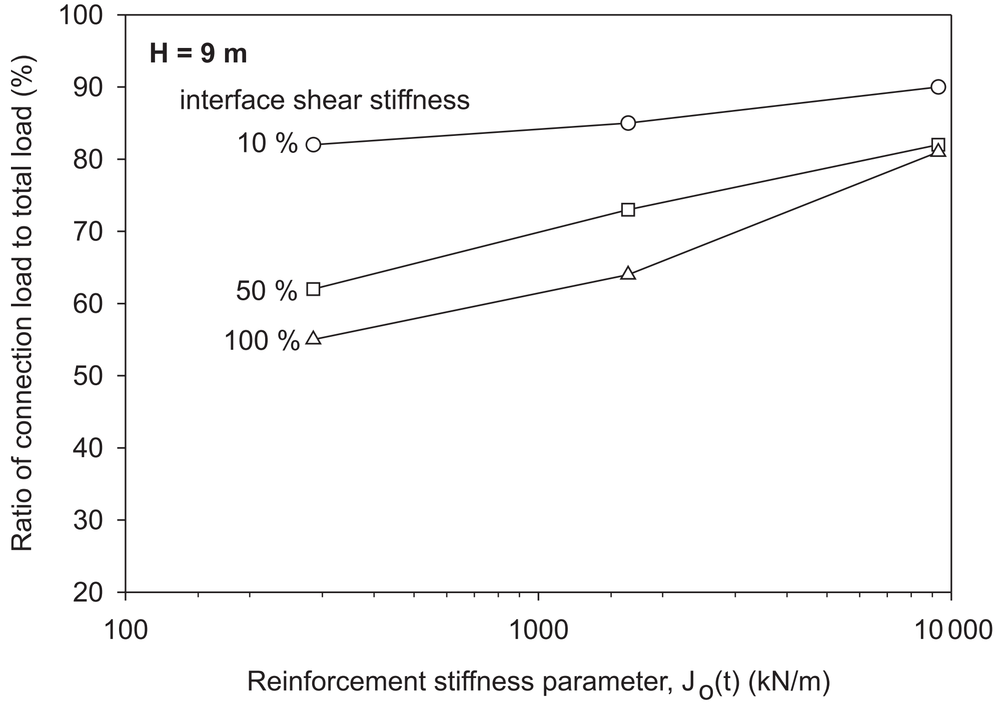
Figure D1 shows the face deflections of a full-scale wrapped-face field wall (GW16) and an older RMC wrapped-face wall (GW11). Both these walls were constructed with reinforcement tails in the wrapped facing, which effectively doubled the number of reinforcement layers and thus reduced the load in the primary reinforcement layers. The relative deformations of these walls are very much less than the ones for the RMC "perfectly flexible" wall in the current study. The flexible-faced wall in the current study was purposely designed to eliminate the complexity of the reinforcement tails on wrapped-face wall performance. Also, the deflection profile for the RMC block-faced wall in the current study is in the same range as the ones for the much taller field walls. Similarly, older wrapped-face RMC wall (GW11) is generally consistent with the much taller full-scale wrapped-face field wall (GW16). This shows that the RMC wall displacement behavior is reasonably consistent with and thus scaleable to taller field walls. Some of the displacement differences between walls may be due to differences in the global stiffness of the reinforcement defined as the sum of creep-reduced reinforcement stiffness values at 2% strain divided by the height of the wall (e.g., Allen and Bathurst 2002). In general, the RMC walls have a lower global stiffness than the field walls. However, this is part of the experimental design to encourage large wall deformations so that performance features are detectable. One interesting difference between the RMC block wall and the field block wall is the shape of the facing displacement. The RMC block-faced wall, as well as the perfectly flexible face RMC wall, appears to have significantly higher displacement near the wall top than the field walls. This is likely due to the heavy surcharge.

The discusser claims that the authors were cavalier in defining what facing stiffness is meant.

The discusser is correct that the authors did not introduce a formal definition of stiffness in the paper but relied on a notional concept. In our opinion, it is self-evident that a column of modular blocks of the type we used is clearly "stiffer" than the companion wrapped-face wall. Nevertheless, there are two ways to quantify facing stiffness for modular block walls.

Figure D2 illustrates the first approach, which can be understood as an analog to a continuously supported

Fig. D3. Influence of reinforcement stiffness and block interface stiffness on ratio of reinforcement (connection) load to total load on 9 m high wall (Huang et al. 2007, reproduced with permission of the Canadian Geotechnical Society).



"beam." The distributed load can have a wide range of distributions and is shown here as a concept only. The wall stiffness is related to the value of the interface shear stiffness between blocks (K_b). A similar linear spring approach was used by Leshchinsky and Vulova (2001) and Hatami and Bathurst (2005, 2006) to model load transfer between blocks using FLAC codes. Since there is always some constraint at the base of a modular block facing due to embedment, base shear or both, then $K_f > 0$. From our physical tests, this value has been back-calculated to be 4 MN/m/m and thus 10 times less than the measured interface shear stiffness value of the blocks (40 MN/m/m). Hence, it can be argued that the facing toe restraint is not perfectly rigid and indeed the horizontal stiffness is less than the stiffness between the blocks. The reinforcement layers provide reactions to the free-body stack of blocks. In this analog, they have a stiffness K_r. The relative amount of load, carried by the reinforcement layers, is a function of the relative stiffness of the reinforcement layers (K_r), block interface shear stiffness (K_b), toe stiffness (K_f), and the height (or number of block units and number of reinforcement layers). Our physical experiment with the wrapped-face wall was an attempt to achieve $K_b = K_f = 0$. When $K_b = 0$, any discussion of the effect of toe condition and stiffness K_f is irrelevant. As $K_f >>> K_r$ for the same number of reinforcement layers, the fraction of load carried by the toe increases. In the opinion of the authors, $K_f > 0$ is the typical case.

The second approach was mentioned in the paper under discussion. This is the use of the *K*-stiffness method, which explicitly introduces a facing stiffness factor as an analog to an elastic cantilever. This factor can be used to quantify the influence of the facing on reinforcement loads (Allen et al. 2003; Bathurst et al. 2005; Miyata and Bathurst 2007*a*, 2007*b*). The facing stiffness factor, computed for the two walls in our paper, predicts that the reinforcement loads for the wrapped-face wall at end-of-construction conditions are about 3 times higher than for the stiffer block wall. This is indeed the case and is consistent with the measurements from field walls, as mentioned at the end of the paper.

The discusser takes issue with the possible influence of wall height, and facing stiffness on magnitude and distribution of reinforcement loads and toe loads.

The effect of facing stiffness, as the discusser pointed out, diminishes with wall height. However, numerical modeling by the authors has shown that the stiffness (or strength) at the base (or top of bottommost block) must be reduced to a very small number for the effect of the constrained toe to diminish entirely (Huang et al. 2007). For example, Fig. D3 shows the results of parametric analyses carried out on the RMC block walls taken to 9 m, using a FLAC model that has been verified against several of the RMC block walls (Hatami and Bathurst 2005, 2006). The simulations were carried out with the same base horizontal stiffness of 4 MN/m/m, as noted earlier, with interface stiffness values at 10%, 50%, or 100% of the reference interface block stiffness value of 40 MN/m/m. The data show that, for the same reinforcement stiffness, as block interface shear stiffness decreases, the proportion of load carried by the reinforcement goes up. However, even for a very soft interface condition, there is significant toe boundary capacity. Recall that the blocks used in both the physical and numerical tests are commercially available units. All commercially available modular block units can transmit shear through concrete shear keys, pins, and friction of one type or another. Hence, all are capable of delivering earth forces to the toe, where horizontal load capacity is generated. If the interface shear stiffness value could be taken to 0, then all the facing load would be carried by the reinforcement layers—a perfectly flexible wall case.

The analog described in Fig. D2 is also very instructive to explain that, during construction, load is generated against the wall face through soil self-weight and compaction and transmitted to the toe through the facing column. Even when the reinforcement layers are placed at higher elevations, some load is carried by the facing because of the stiff toe restraint. The values of K_b and K_f are larger than the reinforcement stiffness during construction and most likely larger under operational conditions for typical polymeric reinforcement products. It is only after large deformations are permitted that reinforcement tensile load capacity can be mobilized, which was demonstrated in our physical block wall test and in our numerical model. As the relative stiffness of the toe spring increases, more load is transmitted to the toe. However, toe load is not mobilized if the wall facing blocks cannot transmit interface shear. Load was carried by the facing at all stages in our block-faced wall.

Leshchinksy et al. (1995), using the limit-equilibrium approach, showed quantitatively that, as block–block interface shear capacity was increased from 0° to 15°, the load carried by the reinforcement layers for a vertical wall decreased by 25%. They admit that this is likely a conservative reduction estimate because they did not consider the self-weight of the blocks. Clearly, the block facing in their model was a load carrying element or at least a load attenuation mechanism. Leshchinsky et al. (1995) stated that interblock friction "… may explain the consistent conservatism of the measured values of tensile forces in reinforcement layers when compared with predictions from commonly used limit equilibrium analyses." In one example, they showed that ignoring fascia effects for a 6 m high geosynthetic reinforced soil modular block wall, reported by Bathurst et al. (1993*b*), led to reinforcement forces that were 4–5 times higher than measured or calculated values. It is difficult to understand why the discusser believes that the stiff wall in our study is not representative of an actual field wall and that a stiff modular block facing is not responsible for reinforcement load attenuation.

The discusser raises a number of questions related to the applicability of the selection of soil friction angle values used to back-calculate the loads in the reinforcement layers.

The intent of the paper was not to engage in any discussions regarding the details and theoretical validity of the limit-equilibrium-based tie-back wedge method. We simply carried out a series of calculations, using the only three possible values of soil shear strength that would be considered by an experienced design engineer and showed the consequences. The authors and co-workers demonstrated through a series of papers that design methods, based on conventional limit-equilibrium tie-back wedge approaches, were typically excessively conservative (e.g., Bathurst et al. 2005; Miyata and Bathurst 2007*a*, 2007*b*). The excessive conservatism that results in the selection of geosynthetic reinforcement type and layout has been noted by many others, including Leshchinsky and Vulova (2001). The level of conservatism, quantified for the block wall in our physical tests, is of similar magnitude to that computed for a large number of other laboratory walls and instrumented field walls published in the literature. Our work demonstrates the conditions under which it is possible to get a reasonably accurate estimate of reinforcement load. As noted in our paper, and by the discussers, it requires a very flexible wall construction that would not be constructed in practice. The discusser did not seem to acknowledge that the wrapped-face wall was purposely constructed to give a perfectly flexible facing and thus allow us to achieve a condition that most closely matches conditions for which the current tie-back wedge method is applicable.

The discusser mentions that connection loads for block-faced walls are in his experience less than the maximum reinforcement loads further back in soil.

This was not the experience of the authors. Figure D4 shows normalized strains plotted against distance from the back of hard-faced walls. These data were from instrumented field walls and were taken from a large database of case studies reported by the authors and co-workers (Allen et al. 2003; Miyata and Bathurst 2007*a*, 2007*b*). The data show that there is a tendency for the peak strains to occur close to the back of the facing. Hence, it is unsafe to attenuate loads at the connections, as proposed by the discusser, if you look at all available data. The reason for the large connection loads is that the soil is compacted and compressed during construction and settles as a result of outward facing deformations. There is relative downward movement of the soil with respect to the hard face that puts parasitic loads on the reinforcement at the connections. These loads cannot be predicted within the framework of current tie-back wedge methods of analysis. The authors have a large amount of forensic experience with failed modular block walls. In many cases, the failure was manifest as connection failure due to overstressing, which results from the loading mechanism described here. A fundamental shortcoming of the current tie-back wedge method is that it cannot account for the down-drag forces that are generated at the connections because the mechanism is the result of relative movements.

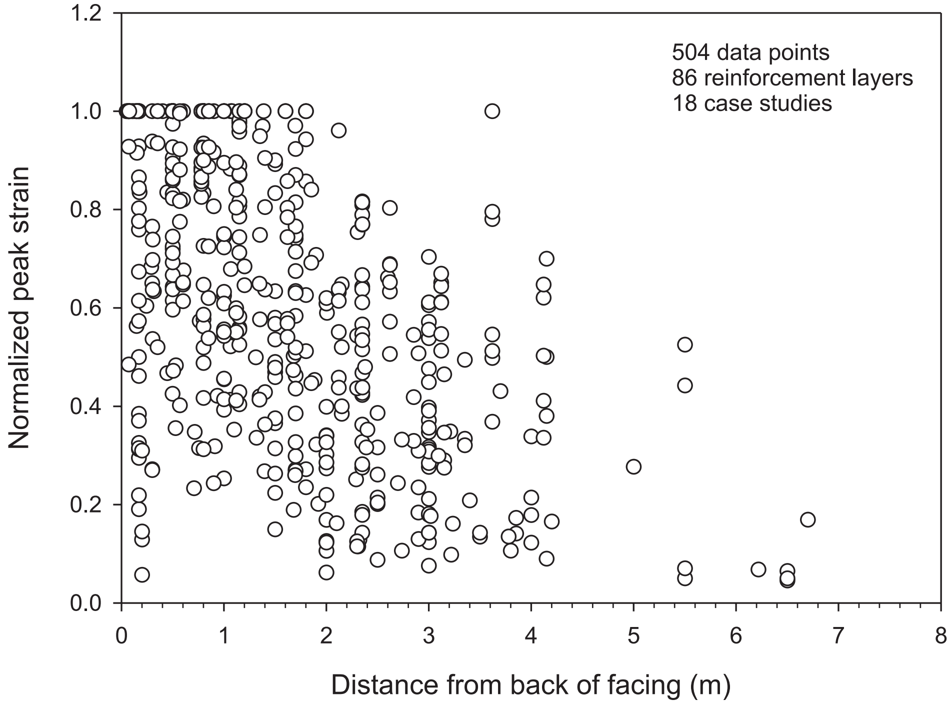
The discusser points out that the strains in the reinforcement in the wrapped face are less at the connections.

This is true for the data shown at a 80 kPa surcharge. However, under operational conditions (i.e., end of construction), the largest strains are at the location of the connections due to the sagging of the wrapped-face units. The drift of the peak strain location deeper into the soil during the 80 kPa surcharge level is consistent with the conditions approaching collapse (limit equilibrium in classical terms). However, it can be argued that the former distribution is more relevant since it captures the strain distributions under conditions for which the wall operates, not the near-collapse condition. The difference in strain or load profiles between end-of-construction and surcharge conditions highlights the difficulty of scaling incipient collapse performance (or limit-equilibrium analysis results) to operational conditions. This is a fundamental shortcoming of the current practice.

The discusser comments that the surcharge was unrealistically high and cannot be considered as equivalent to tall walls.

There is no claim in the paper that we were attempting to substitute surcharge height for wall height. It is important to note that the tie-back wedge method used to compute rein-

Fig. D4. Reinforcement strains normalized against peak strain in reinforcement layers for walls with a hard facing.



forcement loads is a general model that does not carry any restrictions on its applicability in current design codes. A robust model must be able to predict load for a wide range of scenarios, including structures with very simple boundary conditions and uniformly distributed surcharge loads of any magnitude. Our work shows that the general approach is conservative for the simple block-faced wall case presented in the paper, but not unreasonable for the idealized perfectly flexible wall case for which the tie-back method actually applies.

The discusser comments that for reinforcement spacing less than 600 mm (the spacing used in our walls), removing the facing does not destabilize the soil. In other words, the facing is not required for facing stability.

However, small reinforcement spacing does not prevent raveling of granular soils in the long term. Furthermore, in practice, designers tend to use as large a spacing as codes permit. This is largely to expedite construction (i.e., reduce construction time) and reduce cost.

The discusser asks why we are revisiting the current AASHTO and NCMA methods of design since they were developed based on 'extensive experimental work.'

The discusser is incorrect. Both R.J.B. and T.M.A. were involved in the development of both design codes. The computation of reinforcement loads in the NCMA code was developed to be consistent with the AASHTO method. The AASHTO method for geosynthetic reinforced walls was derived from the "tie-back wedge method" for steel-reinforced soil walls—not geosynthetic-reinforced soil structures. Since the development of these codes instrumented and monitored full-scale structures have demonstrated that the general approach is conservative. Furthermore, Allen and Bathurst (2002) and Allen et al. (2003) demonstrated that the current AASHTO method for geosynthetic-reinforced soil walls provides a very inconsistent prediction of reinforcement loads since the coefficient of variation (COV) of the ratio of measured to predicted load is about 90%. Statistically, this very high COV value demonstrates there is really no correlation between actual peak reinforcement loads within the reinforced soil zone and predicted values, using the tie-back wedge method.

While it is attractive to use the inherent conservatism in the current tie-back wedge method to protect the design from poor construction, it is unsound practice to bury in any design methodology an unquantified additional safety factor to account for possible poor construction. The correct approach is to develop a design model that is accurate for typical good construction and then to adjust the outcome for possible poor construction. Our strategy for the larger research work under way at RMC is to quantify sources of conservatism under typical good construction conditions. Once we can quantify the true margins of safety, then designers can make qualified decisions, regarding how much additional reinforcement capacity they need to feel comfortable as a result of anticipated level of construction quality on a project-specific basis. Without the baseline performance data presented in our larger research program, it is impossible for practicing engineers and researchers to know what the true margin of safety against poor performance actually is, or to provide quality data for verification of numerical models that can be used to extend our database of physical test results.

Response to points raised by Barrett

The discusser raises concerns that the wrapped-face wall is not representative of actual wrapped-face walls in the field.

The discusser has misinterpreted the intent of the wrapped-face wall geometry and construction used in our physical wall. We stated clearly in the paper that the experimental design was purposely selected to generate an idealized perfectly flexible facing. The intent was not to duplicate typical wrapped-face construction.

The discusser is of the opinion that had the blocks at the facing been removed, nothing untoward would have happened.

The authors disagree. The soil behind the facing would have sloughed out. In fact, this was observed during the exhumation of several of these types of walls at RMC with the same sand backfill. This has also been observed in the field by the authors.

The discusser comments on the 200 mm movement recorded for the facing profile of the wrapped-face wall.

It is important to note that the 200 mm offset was generated when the lower wrapped-face formwork was removed. The wrapped-face sagged-down generating the profile shown in Fig. 7 of the referring paper. Thereafter, the wall was constructed at about the target batter. It is true that the soil "failed" in the wraps. However, the same is true of any moving formwork construction regardless of spacing when the wraps are filled with a cohesionless soil. The wall did not translate as a body, which may be the impression given by the figure if the moving formwork construction is not appreciated. The authors disagree that a primary reinforcement layer is not placed at the base of the reinforced soil zone in a field structure. Without this layer, it is impossible to create the lowermost wrap.

The discusser observed that removing the facing blocks or burning the connections didn't lead to wall facing instability.

It is possible that, for very small reinforcement spacing, the wall backfill stays in place. However, this is likely a meta-stable condition due to apparent cohesion or suction in a sand soil. The longevity of such an exposed backfill is questionable. Geosynthetic reinforced modular block walls are constructed with the intention of the facing remaining in place for the design life of the structure. Because the facing acts as a formwork during soil placement and compaction, there are earth forces generated against the facing. As the wall moves outward, the backfill typically moves down with respect to the facing. This puts loads on the connections. We agree that these connection loads disappear by removing the facing blocks, regardless of reinforcement spacing. However, the authors observed that, when the connections are removed in the field by over-stressing (connection failure), the facing units fall from the facing; this is identified as wall failure and led to legal action. In many cases, connection failure results in the collapse of a volume of soil through the failed wall face as well. The authors agree that using a large number of reinforcement layers (e.g., smaller spacing at 200 mm) results in attenuation of reinforcement loads at the connections. However, as mentioned in the response to the previous discusser, this is usually not an economical solution. It is more efficient to keep the reinforcement spacing at a large a spacing as possible (e.g., 600 mm in our case to be compliant with AASHTO and NCMA recommendations) and use the stack of facing units to carry a portion of the earth pressure. However, we agree that encouraging smaller reinforcement spacing to improve redundancy in the reinforced system and reduce reinforcement loads is desirable, provided the structure is economical.

References

Allen, T.M., and Bathurst, R.J. 2002. Soil reinforcement loads in geosynthetic walls at working stress conditions. Geosynthetics International, **9**: 525–566.

Bathurst, R.J. 1991. Case study of a monitored propped panel wall. *In* Proceedings of the International Symposium on Geosynthetic-Reinforced Soil Retaining Walls, Denver Colo., 8–9 August 1991. *Edited by* J.Y.H. Wu. A.A. Balkema, Rotterdam, the Netherlands. pp. 159–166.

Bathurst, R.J., Simac, M.R., and Berg, R.R. 1993*a*. Review of the NCMA segmental retaining wall design manual for geosynthetic-reinforced structures. Transportation Research Record, **1414**: 16–25.

Bathurst, R.J., Simac, M.R., Christopher, B.R., and Bonczkiewicz, C. 1993*b*. A database of results from a geosynthetic reinforced modular block soil retaining wall. *In* Proceedings of Soil Reinforcement: Full Scale Experiments of the 80's, ISSMFE/ENPC, Paris, France, November 1993. pp. 341–365.

Huang, B., Bathurst, R.J., and Hatami, K. 2007. Numerical study of the influence of block interface stiffness on reinforced soil segmental walls of variable height [CD-ROM]. *In* Proceedings of 60th Canadian Geotechnical Conference, Ottawa, Ont., 21–24 October 2007. pp. 2167–2174.

Leshchinsky, D., and Vulova, C. 2001. Numerical investigation of the effects of geosynthetic spacing on failure mechanisms in MSE block walls. Geosynthetics International, **8**: 343–365.

Leshchinsky, D., Ling, H., and Hanks, G. 1995. Unified design approach to geosynthetic reinforced slopes and segmental walls. Geosynthetics International, **2**: 845–881.

Miyata, Y., and Bathurst, R.J. 2007*a*. Development of the K-stiffness method for geosynthetic reinforced soil walls constructed with c–ϕ soils. Canadian Geotechnical Journal, **44**: 1391–1416. doi:10.1139/T07-058.

Miyata, Y., and Bathurst, R.J. 2007*b*. Evaluation of K-stiffness method for vertical geosynthetic reinforced granular soil walls in Japan. Soils and Foundations, **47**: 319–335.