Reprint: Full Scale Testing of Geosynthetic Reinforced Walls, Bathurst et al., *ASCE Special Publication, Proceedings of GeoDenver 2000,* 6-12 August 2000, Denver, Colorado.

Full Scale Testing of Geosynthetic Reinforced Walls

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Abstract

The paper is focussed on the preliminary results of a program of full scale tests of geosynthetic reinforced retaining walls carried out by the Geotechnical Research Group at the Royal Military College of Canada (RMCC). The long term research program involves the construction of 10 walls. The paper describes the test program, some instrumentation details and presents selected results from the first four walls completed to date. Three of the walls were constructed with a column of dry-stacked modular concrete units and one nominal identical wall was constructed with a wrapped-face. All of the structures were surcharge loaded to stress levels well in excess of working load conditions. The data gathered from this program has been useful to identify important performance features of the reinforced soil structures and to identify possible sources of conservatism in current methods of analysis for geosynthetic reinforced soil structures in North America. The high quality data will also prove useful for calibration of numerical models.

Introduction

The Geotechnical Research Group of the Civil Engineering Department at the Royal Military College of Canada (RMCC) is engaged in a research project related to design and performance of geosynthetic reinforced soil structures. A major component of this research has been the construction, surcharge loading and monitoring of instrumented full scale geosynthetic reinforced soil retaining walls. The principal objectives of the experimental work are:

- Develop a better understanding of the mechanical behavior of geosynthetic

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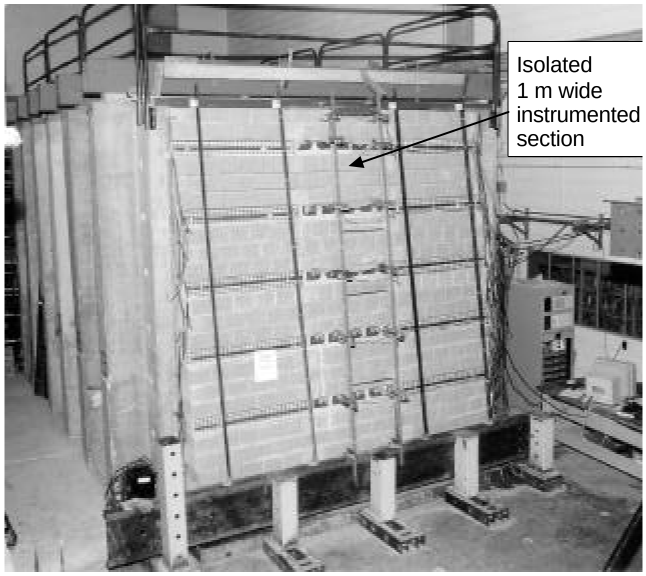


Figure 1. Geosynthetic reinforced segmental retaining wall (Wall 1)

reinforced soil walls during construction, at working load levels and under uniform surcharge loading approaching collapse of the structures;

- Create a database of results from carefully instrumented full scale walls that can be used to calibrate numerical models, and;
- Use the lessons learned from full scale tests to improve current design methodologies for reinforced soil retaining walls.

The long term research program involves the construction of 10 walls. At the time of writing four walls have been completed. Each of the walls was 3.6 m in height. Three of the walls were constructed with a column of dry-stacked modular concrete units (reinforced segmental retaining wall - see Simac et al. 1993; Bathurst and Simac 1994) and one nominal identical wall was constructed with a wrapped-face. Following construction, all of the structures were stage surcharge loaded to stress levels well in excess of working load conditions. An example reinforced segmental retaining wall structure from this test program is shown in Figure 1.

RMCC Retaining Wall Test Facility

The full scale walls have been constructed in the RMCC Retaining Wall Test Facility located within the Civil Engineering Department structures laboratory at RMCC. The test facility allows soil retaining wall structures to be constructed that are 3.6 m high by 3.4 m wide with backfill soil extending to a distance of 6 m from the front edge of the facility. The backfill soil and wall facing are seated on a rigid concrete foundation. The soil is laterally contained between two parallel reinforced concrete counterfort walls that are bolted to the structural laboratory floor. A series of hollow steel sections at the top of the facility are attached to bolts that extend through the counterfort columns to the laboratory floor. These sections act as the restraining system for a timber joist and plywood ceiling. The ceiling in turn confines a series of air bags that are used to apply a uniform surcharge to the entire soil surface. The air bags are commercially available products made of a paper shell that encapsulates a polyethylene bladder. They are used as void fillers in partially loaded shipping containers to prevent damage to cargo. Uniform surcharge pressures up to 115 kPa have been applied in recent experiments using these inexpensive and disposable air bags. The toe of full scale wall models is located at the front of the test facility. The back of the soil mass is restrained by a series of rigid reinforced concrete bulkheads at the opposite end of the test facility. The inside surfaces of the test facility (side walls) are comprised of Plexiglas and multiple layers of lubricated polyethylene sheeting to ensure that model performance approaches a plane strain condition. The test facility allows full scale walls to be constructed, surcharged, excavated and monitored in a controlled indoor laboratory environment.

Test Program

General

Table 1 summarises essential details and objectives of the four test walls completed to date.

Table 1. Full scale test wall program

Wall	Description	Objectives/Variables
1	Geosynthetic reinforced soil wall with modular block facing and a weak biaxial polypropylene (PP) geogrid placed at 0.6 m vertical spacing	Reference wall
2	Geosynthetic wall with modular block facing and a weak biaxial PP geogrid. Every other longitudinal rib removed to produce geogrid with one half the stiffness and strength of the control test reinforcement (0.6 m vertical spacing)	Influence of reinforcement stiffness and strength on reinforcement loads/strains/deformations and overall level of safety
3	Geosynthetic wall with modular block facing, using a weak biaxial PP geogrid (0.9 m vertical spacing)	Influence of large reinforcement spacing on reinforcement loads/strains/deformations and overall level of safety
4	Geosynthetic wall with geosynthetic wrapped-face using a weak biaxial PP geogrid (0.6 m vertical spacing)	Influence of wall toe restraint and facing stiffness on reinforcement loads/strains/deformations and overall level of safety

The modular facing units for Walls 1, 2 and 3 were a solid masonry block with a continuous concrete shear key. The blocks are 300 mm long (toe to heel), 150 mm high, 200 mm wide and have a mass of 20 kg. The wall facing units were built with a staggered (running joint) pattern matching the construction technique used in the field (Figure 1). The setback of each layer of blocks corresponds to a facing batter of 8 degrees from the vertical. The 3.4 m wide modular block facing was constructed in three discrete panels (columns) - two 1.2 m wide outside sections and a central 1 m wide panel. The central column section and its footing support were instrumented and isolated from the outside facing columns to further decouple the instrumented section from the test facility side walls. Only the 1 m wide reinforcement strips over the middle 1 m width of the test facility were instrumented.

Soil

A sand backfill was used in the tests. The material is a uniform size rounded beach sand with $\phi_{cv} = 35^o$ and a peak plane strain friction angle $\phi_{ps} = 44°$. The sand was compacted to a unit weight of 16.8 kN/m^3 (relative density 50%) using a light-weight gasoline driven plate tamper.

Reinforcement

An extruded biaxial polypropylene geogrid reinforcement product was used in all four test walls. Each layer of reinforcement had a total length of 2.52 m measured from the front of the facing column. The geogrid material used is relatively weak and extensible and was specifically selected to generate detectable (i.e. large) strains in the reinforcement and to encourage large wall deflections using the available surcharge capacity of the test facility. In-isolation constant load (creep) tests were carried out on specimens of each type of geogrid in order to develop isochronous load-strain data using the method described by McGown et al. (1984). The creep tests were carried out at the same temperature as the wall models. The isochronous data was subsequently used to infer tensile loads in the reinforcement directly from recorded strains and elapsed time measurements using the approach reported by Bathurst and Benjamin (1990) and Bathurst (1990).

Instrumentation

Up to 300 instruments were deployed in each test wall in order to record the following measurements:

1. Strain in the reinforcement layers (approximately 100 strain gauges).
2. Connection loads between the facing column and the reinforcement layers (Walls 1-3).
3. Wall facing deflections.
4. Horizontal and vertical toe loads (Walls 1-3).
5. Vertical earth pressures at the base of the soil mass and within the soil mass.
6. Vertical deformations within and at the surface of the soil mass.

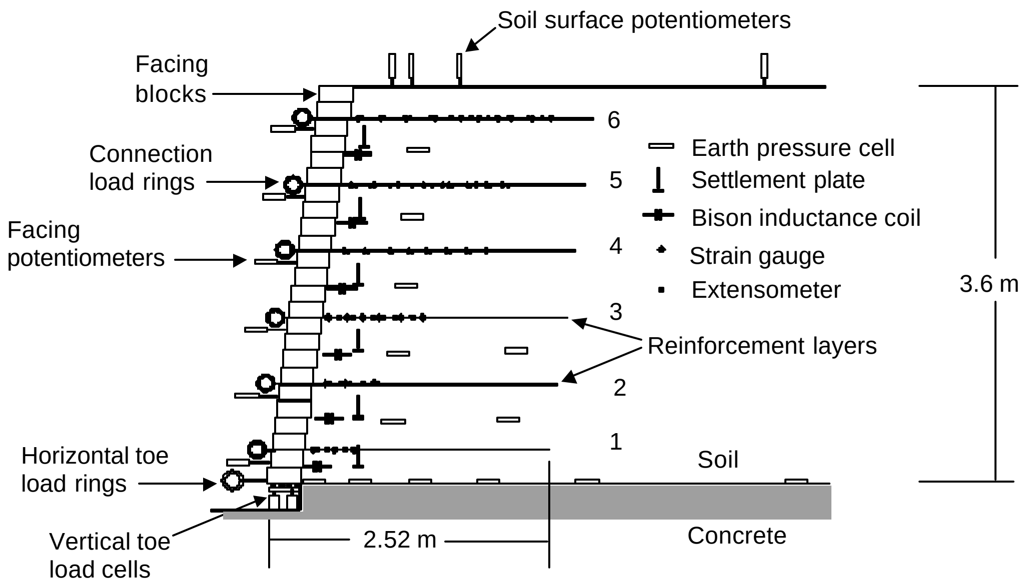


Figure 2. Typical instrumentation plan (Wall 1)

A typical instrumentation plan is illustrated in Figure 2. The extruded polypropylene geogrid used in the tests has the advantage that it can be easily instrumented using speciality foil strain gauges (rated to 10% strain) bonded directly to the surface of longitudinal members. The strain gauges were mounted in pairs at nominal identical distances from the front of the reinforcement for measurement redundancy. Experience with this technique has shown that the local strain recorded by the strain gauge may not be the same value as the average strain recorded over a gauge length that captures many geogrid apertures (e.g. Bathurst 1991). Hence, each combination of gauge type, bonding method and geogrid type must be individually calibrated in order to determine the relationship between local strain and "global" strain using in-isolation wide width strip tensile tests. This is particularly important since reinforcement tensile forces inferred from strain readings must be global values for back-analysis purposes (Bathurst 1993; Karpurapu and Bathurst 1995).

Wire-line extensometers were used to measure reinforcement displacements. Each device was comprised of a thin metal cable attached to a geogrid junction. The line was then passed through a stiff plastic tube (to isolate the cable from the surrounding soil) and attached at the opposite end to a potentiometer-type displacement transducer that was located at the back of the test facility. Displacement readings by selected pairs of extensometer devices were used to estimate "global" strains in the reinforcement when the foil strain gauges had failed due to debonding from the reinforcement after large strains.

Selected modular blocks in the facing column for Walls 1-3 were modified with specially manufactured load rings to measure connection loads during construction, staged surcharging and excavation. To simplify interpretation of test results, the connections were designed so that there was no slip of the reinforcement layers at the interface between concrete units. Hence, the connection strength was equal to the strength of the reinforcement in the soil.

The base of the facing column in Walls 1-3 was seated on a rigid steel base plate. The base plate was supported in turn by a roller plate arrangement. Two rows of load cells were located directly below the roller plate. A row of load rings connected to the base plate restricted horizontal movement at the base of the wall to less than 2 mm. Hence, the base of the facing column (footing) was essentially fully restrained with respect to vertical and horizontal degrees of freedom. The load cells and load rings at the base of each facing column allowed the decoupled vertical and horizontal footing load components to be measured independently for the duration of each experiment.

An automated data acquisition system was used to record all instruments at programmed intervals during construction, surcharging and excavation. The data was exported to prepared spreadsheets so that a complete record of wall performance was available to the writers within a few hours of data downloading. In this way current data was available to make rapid decisions regarding the surcharge loading program.

Test Configurations and Surcharging

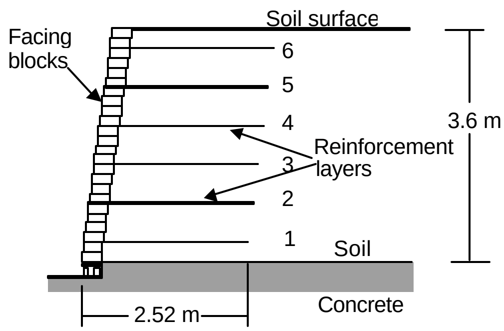
Test configurations for Walls 1 through 4 described in Table 1 are illustrated in Figure 3.

Wall 1 (Figure 3a) was constructed with a polypropylene geogrid with low strength and low stiffness properties in order to encourage large strains and large wall deformations under uniform surcharge loading. The wall was designed to satisfy current National Concrete Masonry Association (NCMA) guidelines (Simac et al. 1993) with the added constraint that the reinforcement layer spacing not exceed a distance equal to twice the modular block toe to heel dimension (AASHTO 1996). This wall is the control or reference case for the remaining wall structures in the current research program.

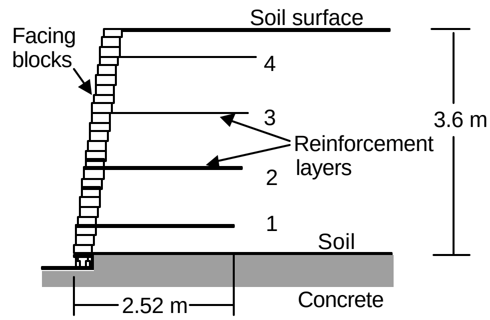
Wall 2 (Figure 3a) was constructed in an identical manner to Wall 1 except that the polypropylene geogrid was modified by removing every second longitudinal member. Hence the reinforcement in this wall had 50% of the strength and 50% of the stiffness of the reinforcement used in the control structure. This was done to isolate the influence of reduced reinforcement strength and stiffness on wall performance.

Wall 3 (Figure 3b) was constructed in an identical manner to Wall 1 except that four layers of reinforcement were used instead of six layers. This wall allows the influence of reinforcement spacing to be isolated.

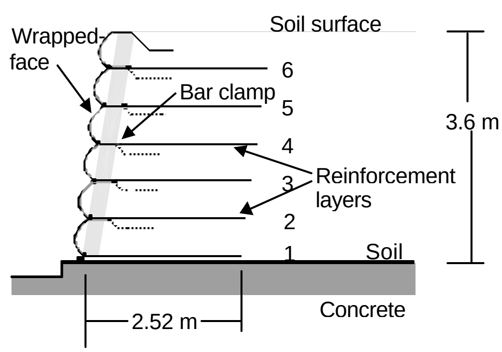
Wall 4 (Figure 3c) was constructed without a hard facing. The reinforcement was arranged to form a wrapped-face in order to isolate the influence of the facing



(a) Walls 1 and 2



(b) Wall 3



(c) Wall 4

Figure 3. Test configurations for Walls 1 to 4

type on wall performance (i.e. compare performance of Wall 4 to Wall 1(control)). The wall was constructed using a "moving form work" braced against the front of the test facility with a target facing batter of 8 degrees from the vertical. Two wrapped-face layers were supported simultaneously during construction. The shaded zone close to the face of the structure in Figure 3c corresponds to the cross-section occupied by the equivalent stacked modular block facing in Walls 1, 2 and 3. Each wrapped-face was attached to the reinforcement layer above using a metal bar clamp. With the exception of the top layer, each wrapped-face was not extended back into the reinforced soil zone corresponding to the dashed lines in the figure. Hence, the geometry and reinforcement attachment used in Wall 4 does not correspond to a typical wrapped-face construction. The construction technique used in this investigation was purposely adopted to facilitate quantitative comparison of walls built with a hard face and a nominal identical wall built with an idealised flexible face.

Following construction, each wall was subjected to staged uniform loading using the airbag system described earlier. An example surcharging history is illustrated in Figure 4. The surcharge load increment was kept constant for a typical minimum duration of 100 hours in all tests. The time datum in the figure is the beginning of construction. At the end of the surcharging program (return to zero surcharge load) the toe was released in Walls 1, 2 and 3 to examine the influence of the horizontally restrained toe on wall performance. Finally, each wall was carefully excavated in 300 mm deep layers while continuously monitoring strain gauges and extensometers attached to each instrumented reinforcement layer. In this way, the location of internal failure surfaces through the reinforced soil mass could be visually confirmed and stress relaxation in the reinforcement layers due to removal of overburden recorded.

Performance

A large amount of data has been gathered from the four test walls completed to date. Selected test results are reported here. A full report on the results of the long term test program is reserved for future publications.

Figure 5 shows the results of surveyed facing column profiles for Walls 1-3 at the end of construction. The dashed line in the figure is the target facing batter based on the geometry of the block units and the built-in concrete shear key location (i.e. this is the profile of the wall face if the blocks could be placed without backfill and each unit pushed forward against the shear key on the underlying block). The figure shows that the actual facing alignment is steeper than the target batter as a result of the incremental construction of the facing column. In addition, the amount of construction-induced rotational movement is greater for Walls 2 and 3 that were constructed with weaker reinforcement layers and a lesser number of reinforcement layers, respectively, compared to the control structure (Wall 1). The amount of construction-induced wall movement recorded at the crest of the facing column ranges from 2 to 4% of the height of the wall.

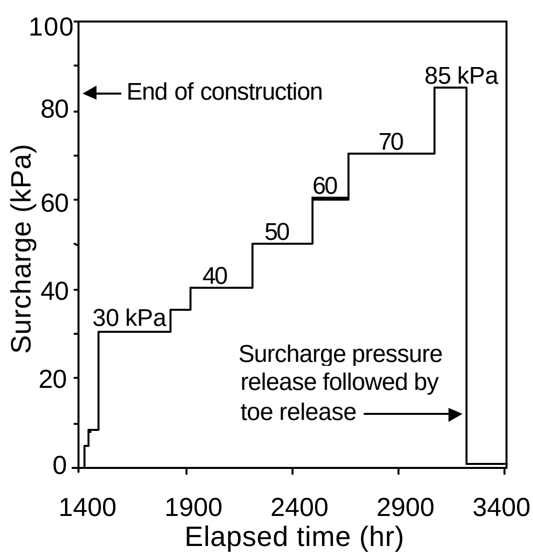


Figure 4. Surcharge history for Wall 2

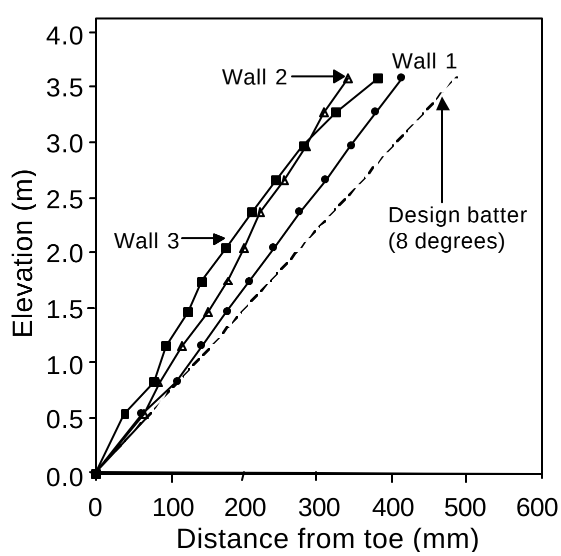


Figure 5. Facing column profiles at end of construction for Walls 1 to 3 (modular block facing)

Figure 6 compares the surveyed facing profile of each of the four walls at end of construction plotted to a common datum. Not unexpectedly, the relatively more flexible wrapped-face wall can be seen to have displaced by about 250 mm at the base of the wall. This movement was generated largely at the time the bottom form work was removed after construction of the two lowermost layers of reinforcement. Nevertheless, the target batter of 8 degrees was reasonably well achieved for the remaining reinforcement layers.

Figure 7 illustrates wall deflections recorded for Wall 1. The horizontal deflections were recorded at reinforcement elevations on the outside of the facing column. Each jump in a deflection curve corresponds to the application of a new surcharge load. Creep of the structure is clearly evident in the figure as a result of the heavy surcharge loads applied to the backfill soil.

Figure 8 shows facing profiles for Wall 2 taken with respect to end of construction. Bulging of the facing column during surcharging is evident in the figure. The maximum outward movement of approximately 70 mm corresponds to about 2% of the height of the wall. The deflection profile for the wall shows a bulge at about ¾ of the height of the wall. At the end of the test the surcharge load was removed, the horizontal toe restraint released and the base of the wall allowed to move outward by about 20 mm. The outward movement of the toe clearly demonstrates that soil pressures acting on the back of the facing column were transmitted to the footing in this experiment.

Figure 9 shows the history of reinforcement displacements recorded by extensometers attached to layer 4 of Wall 2. The time-dependent deformation of the

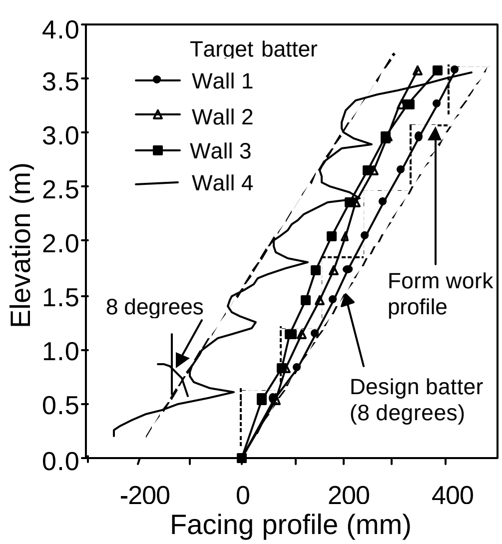


Figure 6. Facing profiles for Walls 1 to 4 at end of construction (from common datum)

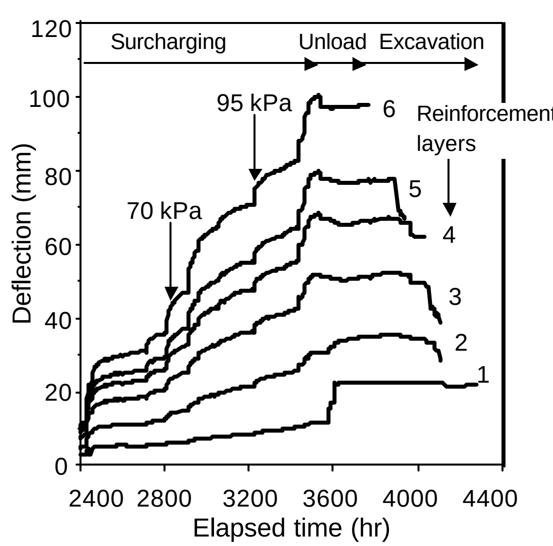


Figure 7. Horizontal deflections measu-red at face of Wall 1

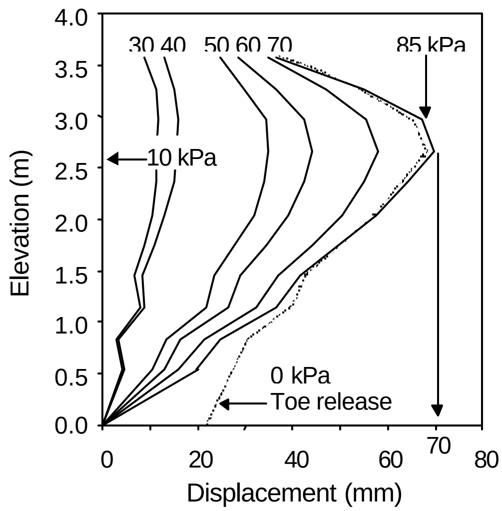


Figure 8. Facing profiles for Wall 2 taken with respect to end of construction

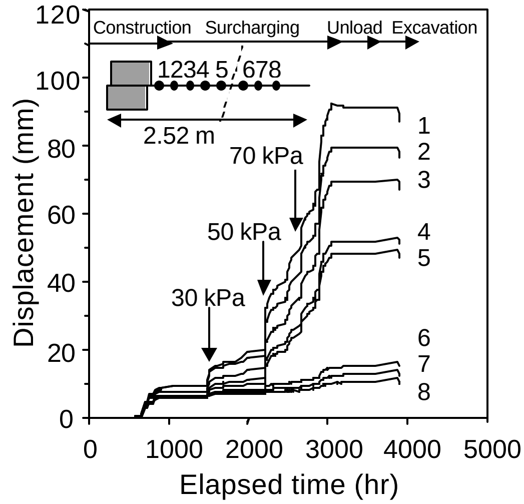


Figure 9. Extensometer displacements recorded for reinforcement layer 4 of Wall 2

reinforcement layer is clearly evident in the data. At the application of each surcharge load increment there was a corresponding jump in the extensometer movement followed by time-dependent deformations that increased in magnitude but at a decreasing rate until the application of the next load increment. As expected, the horizontal displacements in each reinforcement layer were largely irrecoverable after surcharge unloading. The plots in the figure also show that relatively small magnitudes of movement were recorded by the three extensometers located closest to the free end of the reinforcement layer. This behavior is consistent with the conventional notion that distinct active and anchorage soil zones develop at incipient collapse of a reinforced soil mass.

Figure 10 shows the distribution of strains in selected reinforcement layers at the end of construction of Wall 2. The plot shows that the strains are very low but that they are, nevertheless, largest at the connections. Figure 11 shows the distribution of strains in layer 5 of Wall 2 at different surcharge load levels. Only after the surcharge load reached 60 kPa did the peak reinforcement strain move from the connection to a location on the reinforcement corresponding to the internal failure plane in the reinforced soil zone.

Figure 12 shows the measured strain in the reinforcement at approximately the same elevation (layer 3) for Walls 1 to 4. At the end of construction (Figure 12a) the largest measured strains occurred close to the facing in all walls. The strains for Wall 4 were as great as four times the magnitude of the strains recorded for the comparable modular block structure (Wall 1) suggesting that the hard facing in combination with the restrained footing carries a significant portion of the lateral earth loads. The relatively high strains at the connections with respect to each

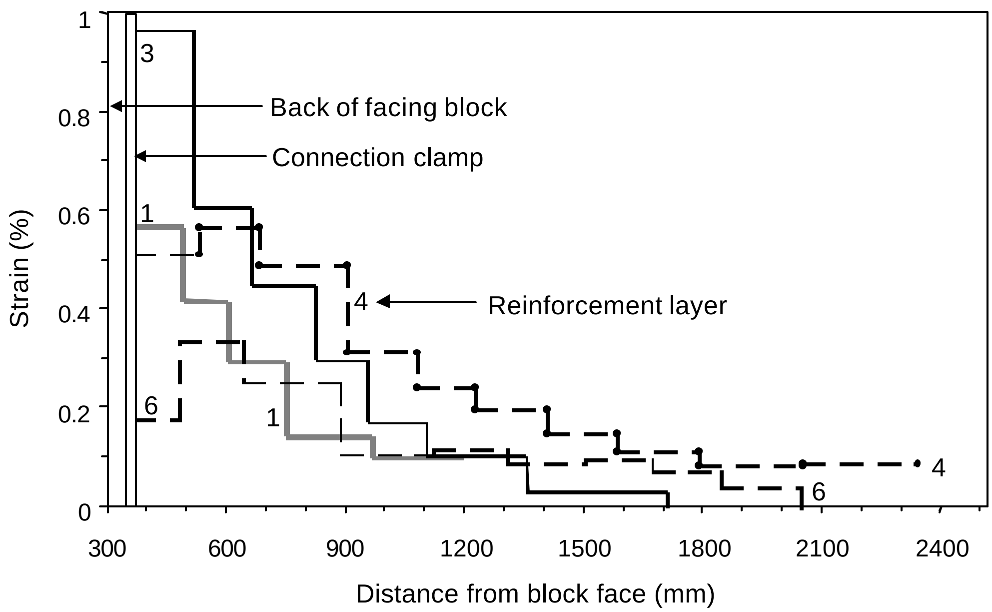


Figure 10. Distribution of strains at end of construction for Wall 2

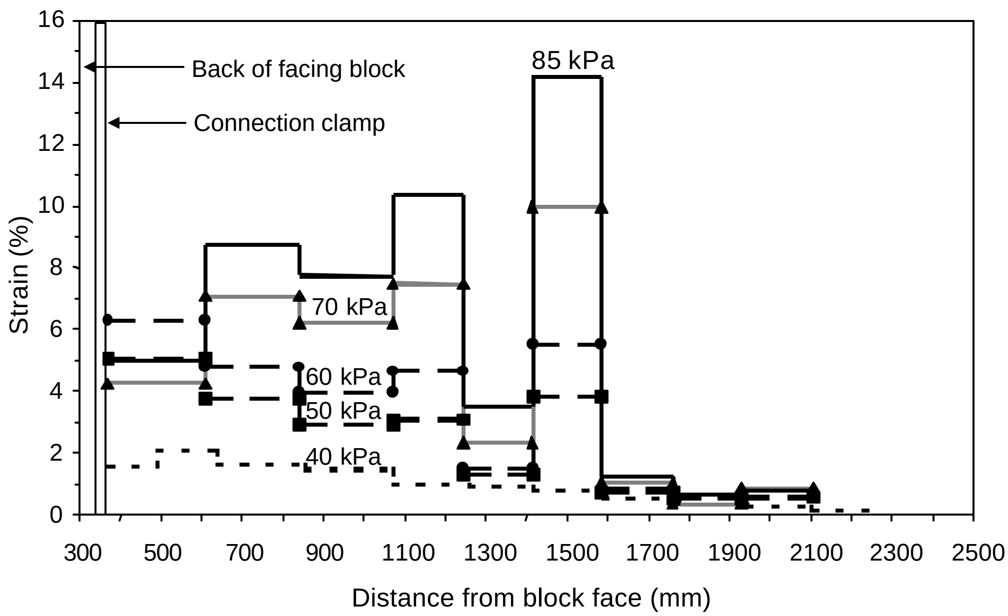
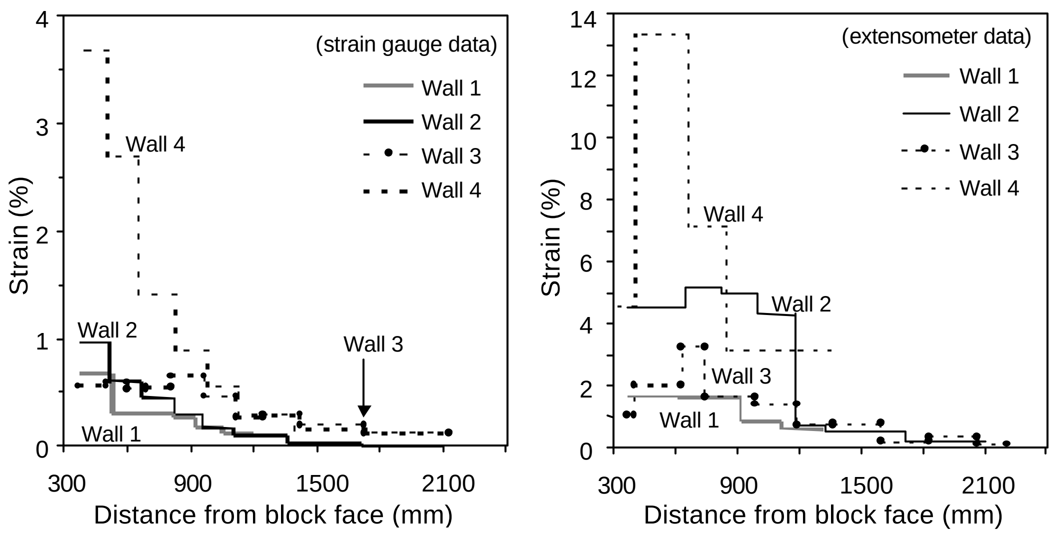


Figure 11. Strain in layer 5 for Wall 2 during surcharge loading

individual modular block wall can be attributed to the relative downward movement of the soil behind the facing. This movement occurs as a result of outward rotational movement of the facing column during construction (see Figure 5) and the settlement of the sand as a result of compaction during incremental construction. The high strains recorded at the same location in the wrapped-face wall are likely due to the downward sagging of the wrapped face (see Figure 6). A similar pattern of peak



(a) End of construction (b) 60 kPa surcharge load

Figure 12. Measured strain in reinforcement layer 3 for Walls 1 to 4

strains close to the face has been reported by Bathurst et al. (1988) for a wrapped-face wall at end of construction. This earlier wall was constructed using a similar reinforcement material but with 750 mm reinforcement spacing and no artificial clamping of the reinforcement layers as described earlier.

At the end of construction, the strains for Wall 3 (four layers of reinforcement) were generally larger than for Wall 1 (6 layers of reinforcement) and were observed to propagate deeper into the reinforced soil zone. Similarly, the peak strains for Wall 2 (less stiff reinforcement) were larger than the strains recorded for Wall 1 constructed with reinforcement having twice the stiffness.

In Figure 12b the magnitude of strains are larger for each wall as a result of the 60 kPa surcharge load. The relative magnitudes of strain identified at end of construction are amplified in this figure with the exception noted earlier that peak strains for the modular block walls occur within the reinforced soil zone rather than at the connections. In the same figure it can be seen that a 50% reduction in the reinforcement stiffness resulted in a more than doubling of the measured strain (Wall 2 compared with Wall 1).

The co-incidence of the location of peak reinforcement strain in reinforcement layers for Wall 1 at peak surcharge load and the internal soil failure surface exposed at excavation is illustrated in Figure 13. The triangle-shaped markers on the figure denote the locations of directly measured peak changes in aperture length measured after reinforcement exhumation. These measurements corroborate the locations of peak strain recorded by strain gauges and inferred from extensometer readings. The failure plane was observed to exactly fit a log-spiral geometry using a plane strain

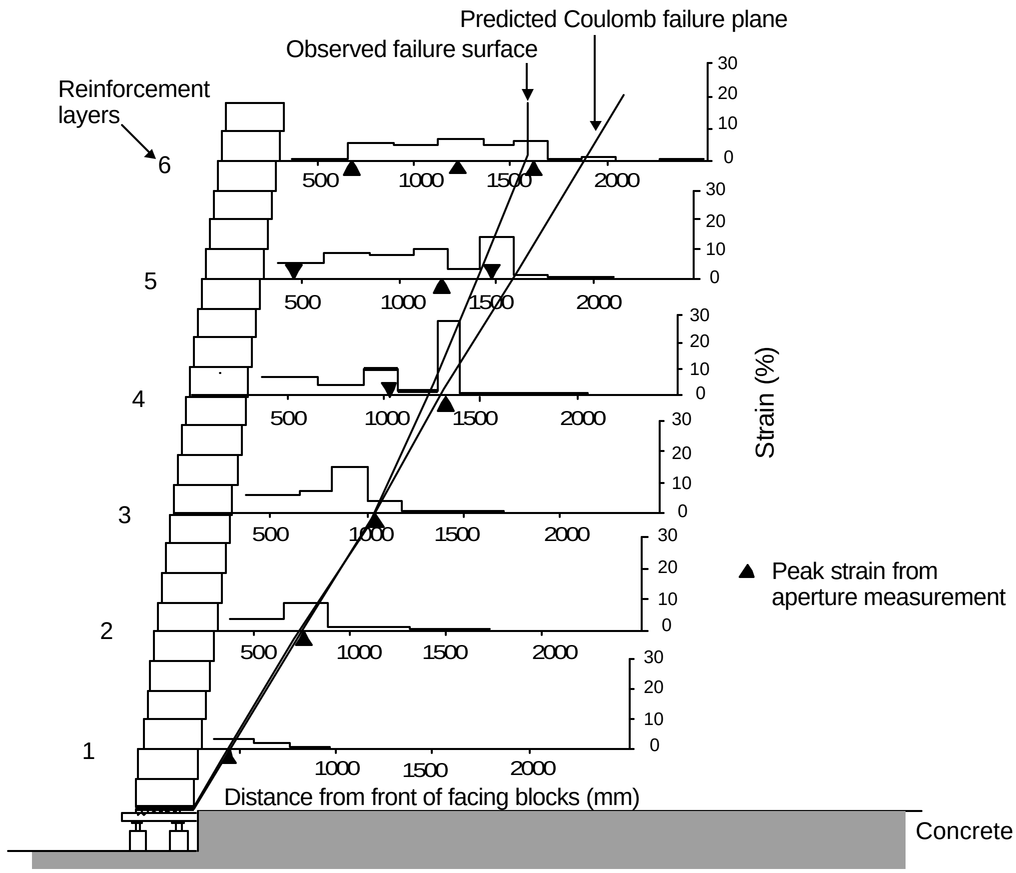


Figure 13. Location of peak reinforcement strain and internal failure surface for Wall 2

peak friction angle ϕ_{ps} = 44 °. However, from a practical point of view the predicted (and simpler) Coulomb failure plane using the same friction angle is reasonably accurate. A similar observation was made for Walls 2 and 3.

Figure 14 shows the history of horizontal toe load measurements recorded at the base of Wall 1 and the sum of connection loads recorded at each reinforcement layer. The figure shows that the restrained toe attracted a significant portion of the total horizontal earth force acting against the facing column. This is not surprising since the toe of the wall is very much stiffer than the reinforcement layers at end of construction. During surcharging, tensile load is mobilised in the reinforcement layers and proportionately more of the horizontal earth force exerted against the facing column is carried by the reinforcement layers. Nevertheless, the toe carries approximately 40% of the total horizontal earth force recorded at the facing column at the end of the surcharge loading program.

Figure 15 shows the history of vertical toe load forces recorded during construction of Wall 3. Each facing block was individually weighed and hence the self-weight of the facing column during construction can be plotted as the linear line

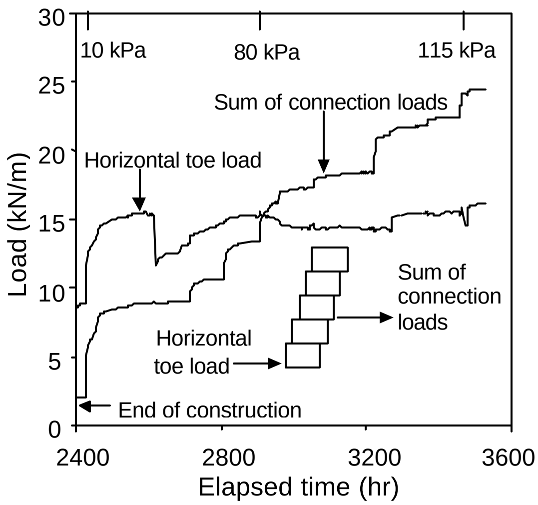


Figure 14. Horizontal toe load at the base of Wall 1 during surcharge loading

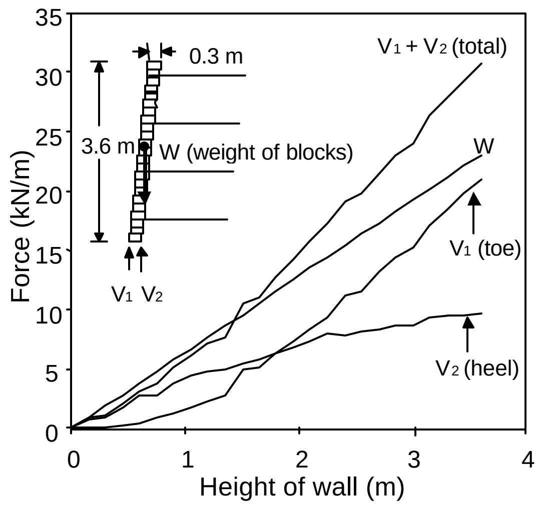
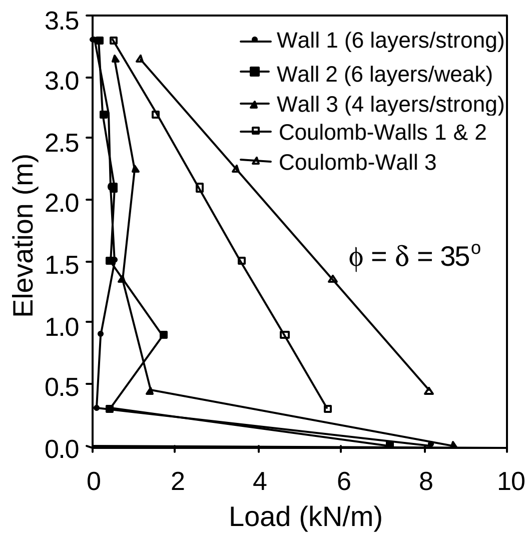


Figure 15. Vertical toe load forces for Wall 3 during construction

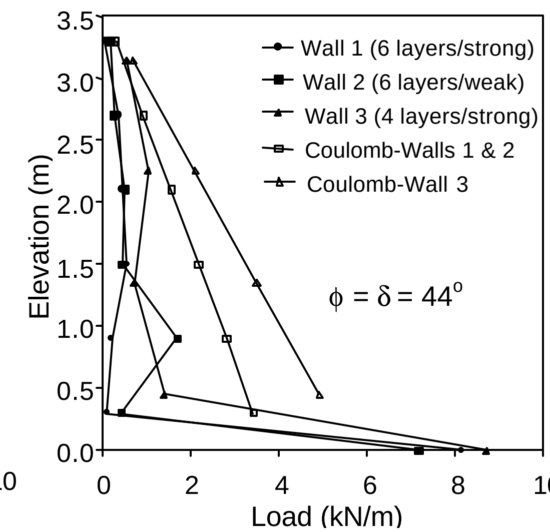
in the figure. Superimposed on the figure is the net vertical footing load and individual loads recorded by two parallel rows of load cells located at the toe and heel of the base plate directly below the facing column (see Figure 3). The sum of the vertical loads is greater than the self-weight of the facing column. This observation is attributed to the vertical downdrag force developed at the connections due to relative downward movement of the sand fill directly behind the facing column. This downward movement is a result of compaction of the soil and outward rotational movement of the facing column. While not shown here, the distribution of vertical earth pressures recorded by earth pressure cells located below the reinforced soil mass was also consistent with the development of vertical load transfer from the soil to the facing column (i.e. vertical earth pressures measured directly behind the facing column at the base of the soil mass were less than values predicted from soil self-weight and surcharge loading).

As the wall was built higher there was a shift of vertical load to the toe of the wall consistent with the notion of wall rotation about the toe of the facing column. However, the heel of each block unit was not unloaded indicating that the batter of the wall was sufficient to keep each block-to-block interface in compression. The hinge height (Simac et al. 1993; Bathurst et al. 1993) for this structure based on a target batter of 8 degrees is 2.1 m An important implication of these measurements to design of the modular block structures in the current study is that the hinge height calculation is conservative for design.

Figure 16 shows the measured connection loads versus the predicted loads using Coulomb lateral earth pressure theory for the end-of-construction condition. In contrast to the triangular distribution of the predicted loads, the measured connection loads are almost uniform with depth. The magnitude and pattern of measured



(a) Constant volume friction angle



(b) Peak plane-strain friction angle

Figure 16. Measured versus predicted connection loads at the end of construction

connection loads is a direct consequence of the rigid toe attracting a significant portion of the horizontal earth forces acting on the facing column, the low stiffness of the geogrid reinforcement layers and, possible redistribution of reinforcement load during construction-induced outward movement of the facing column. Clearly, a shortcoming of conventional earth pressure theories applied to geosynthetic reinforced soil walls with a structural facing is their inability to account for the load that is carried by the restrained toe at the base of the facing column.

For the two reinforcement stiffness cases investigated, there was a negligible effect of reinforcement stiffness on magnitude of measured connection loads. There is a noticeable difference in connection load for Walls 1 and 2 at an elevation of 0.9 m that may be the result of local residual compaction stresses generated during the construction of Wall 2 (i.e. soil at this elevation may have been subjected to a higher degree of compaction). The larger reinforcement spacing used in Wall 3 results in a larger contributory facing area and hence larger predicted reinforcement load. This trend is confirmed by the measured loads shown in the figure.

Figure 16 also shows the influence of the magnitude of friction angle on predicted reinforcement loads. Figure 16a predictions are based on a constant volume friction angle, ϕ_{cv}, and Figure 16b shows predicted reinforcement loads using the peak plane strain friction angle, ϕ_{ps}. The selection of ϕ_{cv} produced an excessively conservative estimate of the reinforcement loads. Using the peak plane strain friction angle resulted in a conservative but more reasonable prediction of the connection loads particularly in the upper elevations of Walls 1 to 3.

Conclusions

A large amount of data from the first four walls in this test program is currently being analysed and the results compared for the four different configurations. Some preliminary observations can be made:

- Connection loads for the structures with a modular block facing construction are the largest loads in the reinforcement at the end-of-construction condition.
- The toe of the wall in these experiments carried a significant portion of the horizontal earth forces acting on the hard facing column. This load capacity is not accounted for in current methods of analysis and design that use conventional earth pressure theories to predict reinforcement loads and hence is one source of conservatism in current design practice.
- The selection of the friction angle for the backfill material is another source of conservatism. Peak plane strain friction angles should be selected to reduce the conservatism in the analysis and design of geosynthetic reinforced soil structures constructed with a hard facing.
- A hard facing column is a structural element that acts to reduce the magnitude of strains that would otherwise develop in a wall with a flexible facing.
- The vertical normal load acting at the toe of the facing column is greater than the sum of the block weights due to soil down drag forces acting at the back of the facing column. This has important implications to connection design and confirms that for the wall batter used in these experiments the current NCMA method to calculate normal forces at the block interfaces is excessively safe.

Future Work

At the time of writing, six more reinforced soil walls are planned. These structures will isolate the influence of other material properties, geometry and facing type on the response of walls that are variations of Wall 1 (control) described in this paper. For example, walls will be constructed with polyester and steel mesh reinforcement materials, with different facing batters, smaller reinforcement spacing and with a full height propped panel configuration. Concurrently, the experimental results from this program and measurements from field-instrumented walls reported in the literature are being used to calibrate numerical models. In turn, numerical models will be used to extend the database of test configurations reported here to include higher walls, different soil types, different facing materials and a greater range of reinforcement spacing and properties.

Acknowledgements

Funding for the research program described here has been provided by the Washington State Department of Transportation, the AASHTO MSE Pooled Fund, the National Concrete Masonry Association, Natural Sciences and Engineering Research Council of Canada, Academic Research Program of the Department of

National Defense (Canada) and grants from the Department of Infrastructure and Environment (DND Canada). The authors would like to acknowledge the contribution of Research Assistants (J. Escobar, M. Domingo, M. Leinhard, R. Madrigal and J. Lacharite) who helped to construct the walls described in the paper and Risi Stone Systems and Terrafix Inc. for provision of the facing units and reinforcement materials, respectively.

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Keywords:

Geosynthetics, Reinforced soil walls, Segmental, Wrapped-face, Full scale testing.