REVIEW OF THREE INSTRUMENTED GEOGRID REINFORCED SOIL RETAINING WALLS

by

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ABSTRACT

High quality data from well-instrumented geosynthetic-reinforced soil walls is required to guide the development of rational methods of analysis and design of these structures and to assess their serviceability performance. The paper describes the results of two instrumented walls in the field and some results from a program of carefully instrumented large-scale model tests carried out at the Royal Military College of Canada.

INTRODUCTION

The use of geosynthetics to reinforce soil retaining wall structures is becoming an accepted alternative to more traditional forms of retaining wall construction. For retaining wall structures over 3 metres in height the use of horizontal layers of polymeric reinforcement sheets including geotextiles and geogrids is often the most cost-effective solution (e.g. Simac et al. 1991, Crowe et al. 1989, Mitchell and Villet, 1987). The methods that are routinely used to analyze and design these structures are founded on concepts of classical Rankine earth pressure theory which have provided a framework that is familiar to the practicing geotechnical engineer. Nevertheless, the lack of quantitative data on the mechanical performance of these structures has led to misunderstanding of the mechanics of these complex soil/polymeric composites and great conservativeness in design methods. In order to develop rational design methods that lead to economical structures, carefully monitored prototype-scale testing of reinforced soil walls is required.

The following paper reports some experiences that have been gained from two field instrumentation projects in North America that used geogrid materials as the soil-reinforcing elements and carefully instrumented large-scale reinforced soil wall models at the Royal Military College of Canada (RMC). The measured data from these case studies has proved valuable in the development of a better understanding of the behaviour of geosynthetic-stabilized soil retaining wall systems.

HIGHBURY AVENUE INSTRUMENTATION PROJECT

The Highbury Avenue structure is a geogrid-reinforced soil retaining wall located in London Ontario that was constructed as part of Highbury Avenue reconstruction and widening. The wall was instrumented during construction and will be monitored for several years in order to evaluate the serviceability performance of the structure in terms of wall movements and reinforcement strains.

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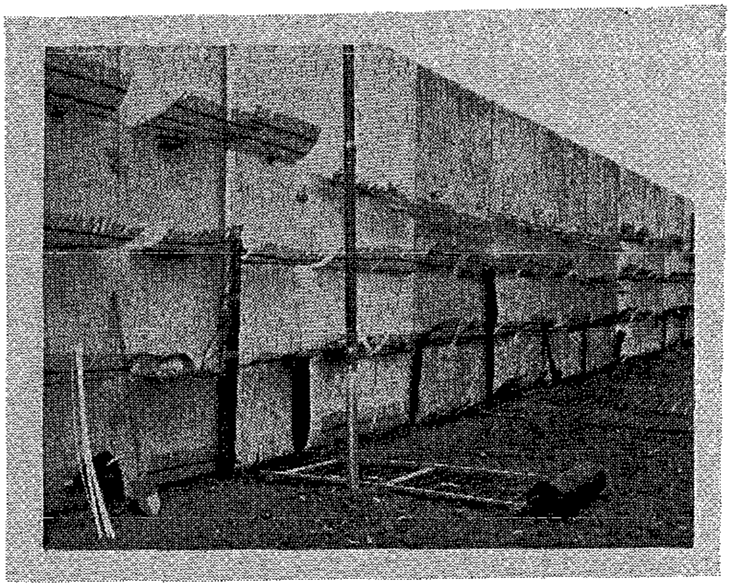

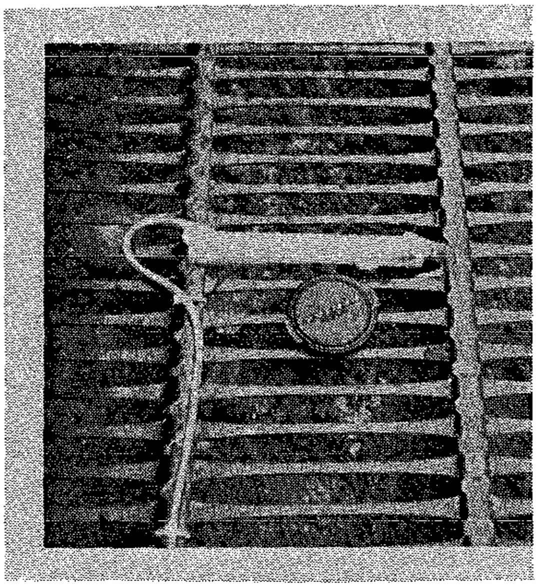

Figure 1 Highbury Avenue wall under construction **Figure 2** Strain gauge installation

The wall is 125 meters in length and was constructed using a series of full–height propped panels made of reinforced concrete and having a width of 2.4m and a height of 1.25m to 7.1m. The facing panels were constructed using reinforced concrete and were seated on concrete strip footings to provide vertical support and to maintain grade. The panels were braced externally during construction and released only after all fill placement and compaction was completed. The retained backfill comprised a coarse sand fill conforming to a Ministry of Transportation of Ontario Granular 'B'. The polymeric reinforcement was a Tensar uniaxial geogrid (UX1600) attached by polyethylene rods to short flaps of the same geogrid material cast into the concrete facing units (**Figure 1**).

INSTRUMENTATION

A total of three panel sections were instrumented. At each section two inclinometer casings were put down into the underlying dense sandy till to a depth of about 2m. One inclinometer was attached directly to the back of the facing and another was put down at a distance of 1.5m behind the wall at the same station (**Figure 1**). Selected layers of grid were strain–gauged at each of the three instrumented sections to monitor strain in the reinforcement during and after construction. The second inclinometer at a distance of 1.5m behind the panels was located so that it would intersect the potential Rankine failure wedge that is used to define the grid anchorage length in the conventional tie–back wedge method of analysis (e.g. Christopher et al. 1989, Task Force 27, 1990). Strains in the reinforcement were monitored by mounting high–elongation strain gauges (10% strain limit) directly to mid–rib locations on the uniaxial grid. The gauges were protected from moisture and mechanical damage during soil compaction by coating the grid, gauge and lead wires with an acrylic paint and then encapsulating the assembly within a silicon–infilled plastic sock (**Figure 2**).

Uniaxial Tensar grids are manufactured by a drawing process that gives them the geometry shown in **Figure 2** and results in a range of local tensile modulus that varies from relatively high at the mid–rib location to relatively low at the junctions between ribs and transverse members. Since design methods assume gross strain in reinforcing elements it is necessary to

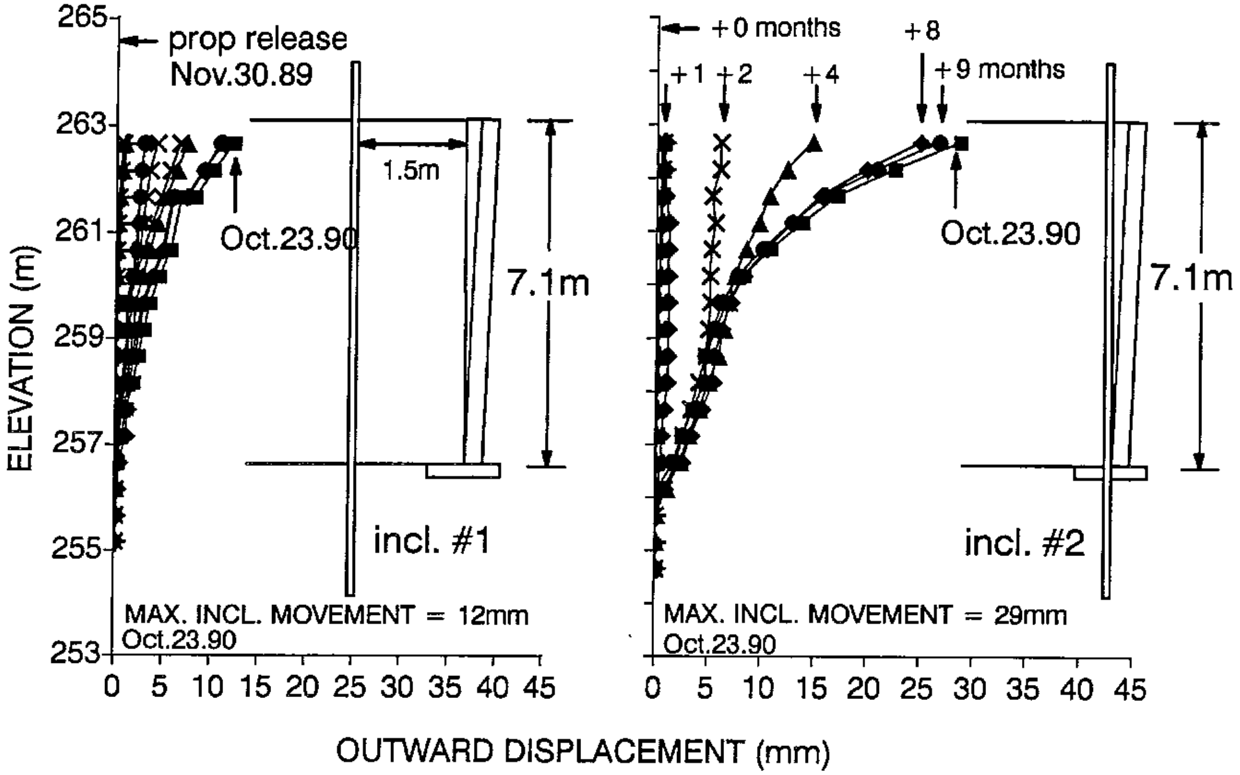

Figure 3 Highbury Ave Panel 39 inclinometer movements

compare the locally monitored strain to the gross strain measured over two or more transverse members in order to relate so-called *gauge strain* to gross reinforcement strain. This calibration was carried out by performing 2% strain/minute in–isolation tensile tests. During these tests vertical displacements across the entire height of the specimen were recorded together with gauge response. In the field, the gauges have been monitored using a manually operated strain gauge box.

RESULTS

Inclinometer results from Panel 39 are illustrated in **Figure 3**. The datum for the displacement profiles was taken just prior to prop release. The data shows that very little movement was recorded over the first month and this can be explained by the freezing of the contained soils that occurred very soon after the wall was completed. After a winter thaw more noticeable displacements were recorded but by the end of the summer wall movements had stabilized. The total out of alignment of the wall was about 29mm which corresponds to a normalized displacement of about $\Delta/H = 0.4\%$ where H is the height of the wall. Based on a review of the literature, the maximum wall movement at this panel was within limits typically expected from other types of geosynthetic reinforced structures that have performed satisfactorily (e.g. Christopher et al. 1989).

The results of strain gauge readings are illustrated in **Figure 4** which shows strains accumulated since prop release. The data shows that the maximum strain was about 3.5% which is well within the strain–limit of the reinforcing material. In other words, at this level of strain the creep in the reinforcement can be expected to diminish with time based on long–term

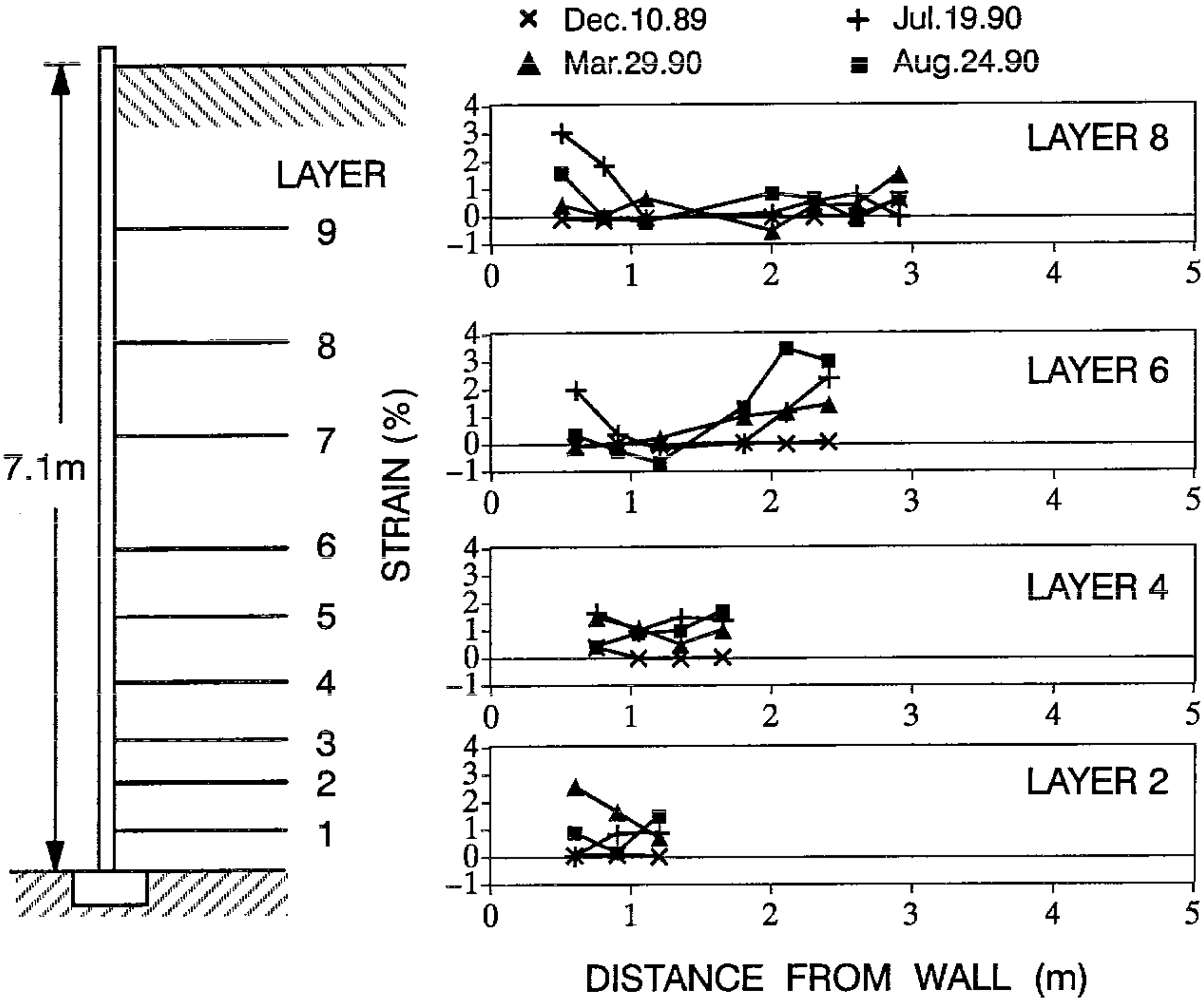

Figure 4 Strain gauge response at Highbury Ave wall Panel 39

creep testing of the product as reported by the manufacturer. The trend towards high connection strains apparent in **Figure 4** is a common phenomenon with rigid panel structures that are tied–back with horizontal reinforcing elements. The high connection strains are due to the relative downward movement of the retained soil with respect to the less compliant panel structure. As a result, the largest strains are likely at the connections rather than at deeper locations commonly associated with a potential linear Rankine failure plane propagating from the toe of the panel. Experience at RMC with full–height propped panel walls 3m in height that have been surcharged to failure have illustrated this mechanism clearly (e.g. Bathurst and Benjamin 1990, Bathurst et al. 1988). Strict quality control of fill placement and compaction is required at the connections to minimize loose or voided soil below the connections that can lead to over–stressing of the reinforcement at this location.

FHWA INSTRUMENTATION PROJECT

A research project sponsored by the Federal Highways Administration (FHWA) is currently underway at Algonquin, Illinois that involves the full–scale testing of several proprietary retaining wall systems constructed with extensible (polymeric) and in–extensible reinforcement and a variety of facing types. The wall that is discussed here had a total length of 15m and was

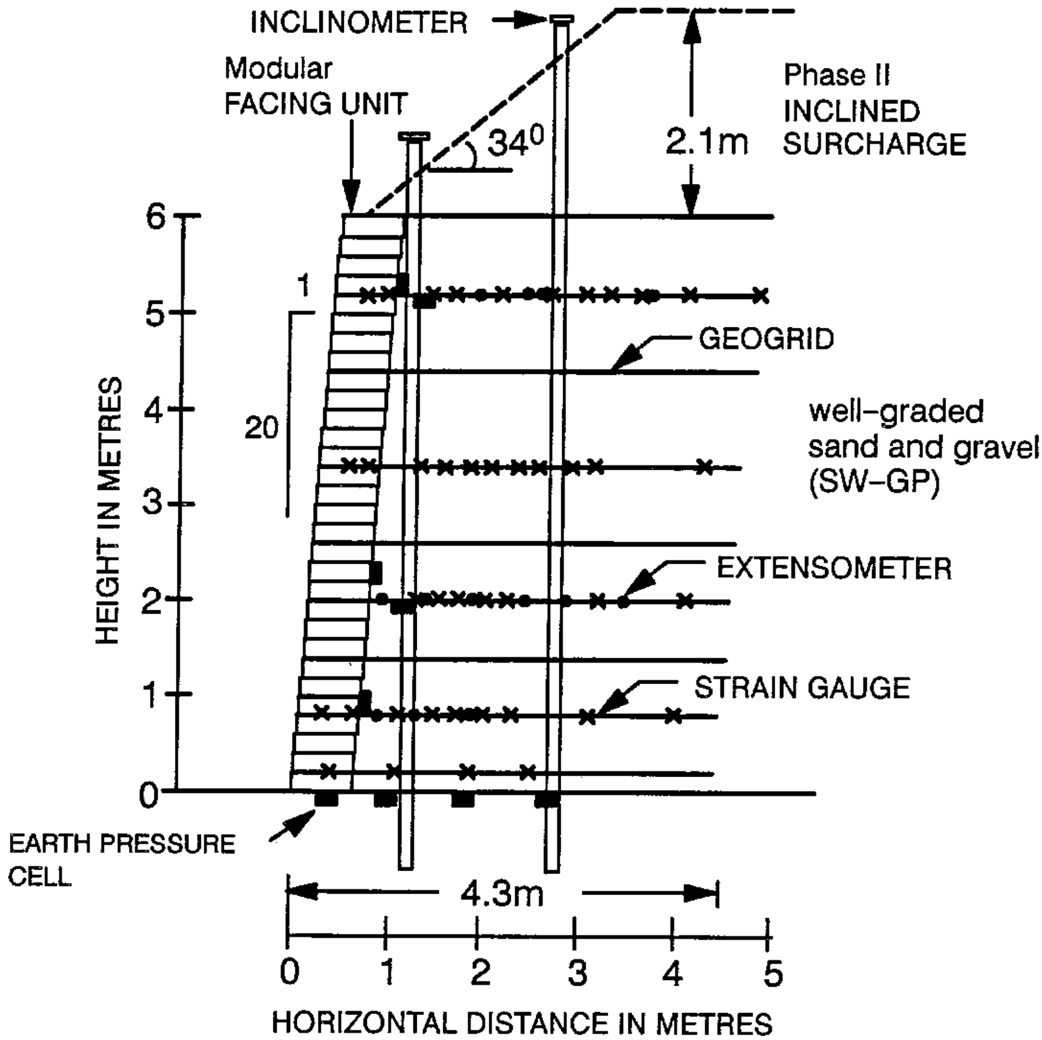

Figure 5 Instrumented geogrid reinforced soil wall with modular facing units (Simac et al. 1990)

constructed using a 45.7 kg hollow (soil infilled) modular concrete facing block anchored to a high–tenacity continuous filament polyester grid (Mirafi 5T). A cross–section of the instrumented section is illustrated in **Figure 5**. Preliminary results from the testing of one of these walls has been reported by Simac et al. (1990). The wall was designed using a computer program that implements a conventional limit–equilibrium approach together with Coulomb–type soils (Bathurst 1989). The number and vertical arrangement of the grid reinforcement was selected to give a Factor of Safety of unity with respect to over–stressing of the reinforcement beyond its serviceability limit load of 10% (based on in–isolation tensile testing of the grid material). With the surcharge in place the same factor of safety was reduced to less than unity. This was done purposely to ensure detectable trends in measured strains and wall movements and to examine conservatism in current methods of design.

INSTRUMENTATION

The instrumentation that was employed included strain gauges bonded directly to the polyester strands of the grid and extensometers attached to the grid and monitored at the wall

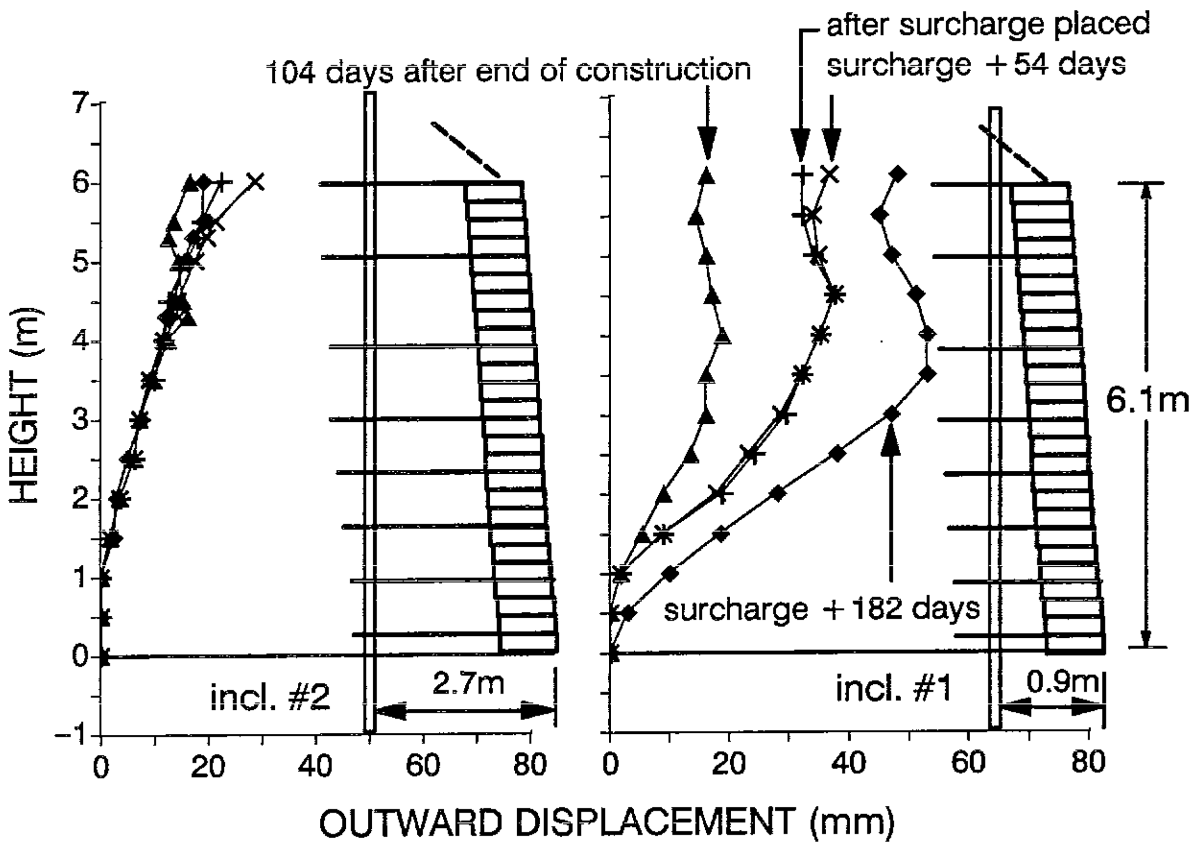

Figure 6 Modular facing wall inclinometer movements (datum is end of construction)

facing. Inclinometers were installed to determine mass soil movements within the reinforced zone. Gloetzl earth pressure cells were arranged to monitor lateral earth pressures behind the facing units and vertical pressures below the reinforced block of soil.

SELECTED RESULTS

Figure 6 shows the results of inclinometer readings at stages after construction including surcharging. The results of inclinometer movements during construction showed that the wall moved about 36mm at the top. However, this movement was partially compensated for during construction and represents about one tenth of the design facing batter. The wall experienced additional deformations after surcharging but the total distortion is not visually apparent and would not compromise the serviceability of the wall in a commercial application (i.e. $\Delta/H = 0.9\%$). **Figure 7** summarizes some of the strain gauge data. The data shows that strains in the reinforcement 104 days after surcharging were less than 1%. Despite having designed the wall at a serviceability strain limit the actual strains are significantly lower. Conventional design methods are clearly conservative when estimating grid forces under working load conditions. A qualitatively different response can be noted in the strain distribution of layer 6 in **Figure 7** and layer 6 in **Figure 4** (Highbury Ave wall). The high connection strains are not evident in the FHWA wall that was constructed with a modular facing despite larger outward post–construction wall movements. A possible explanation for this is that more uniform and greater compaction is achieved directly behind modular facing units since the soil surface and blocks present a flush surface for compaction equipment during incremental construction. In addition, relative downward movement of the retained soils may be less than in propped wall

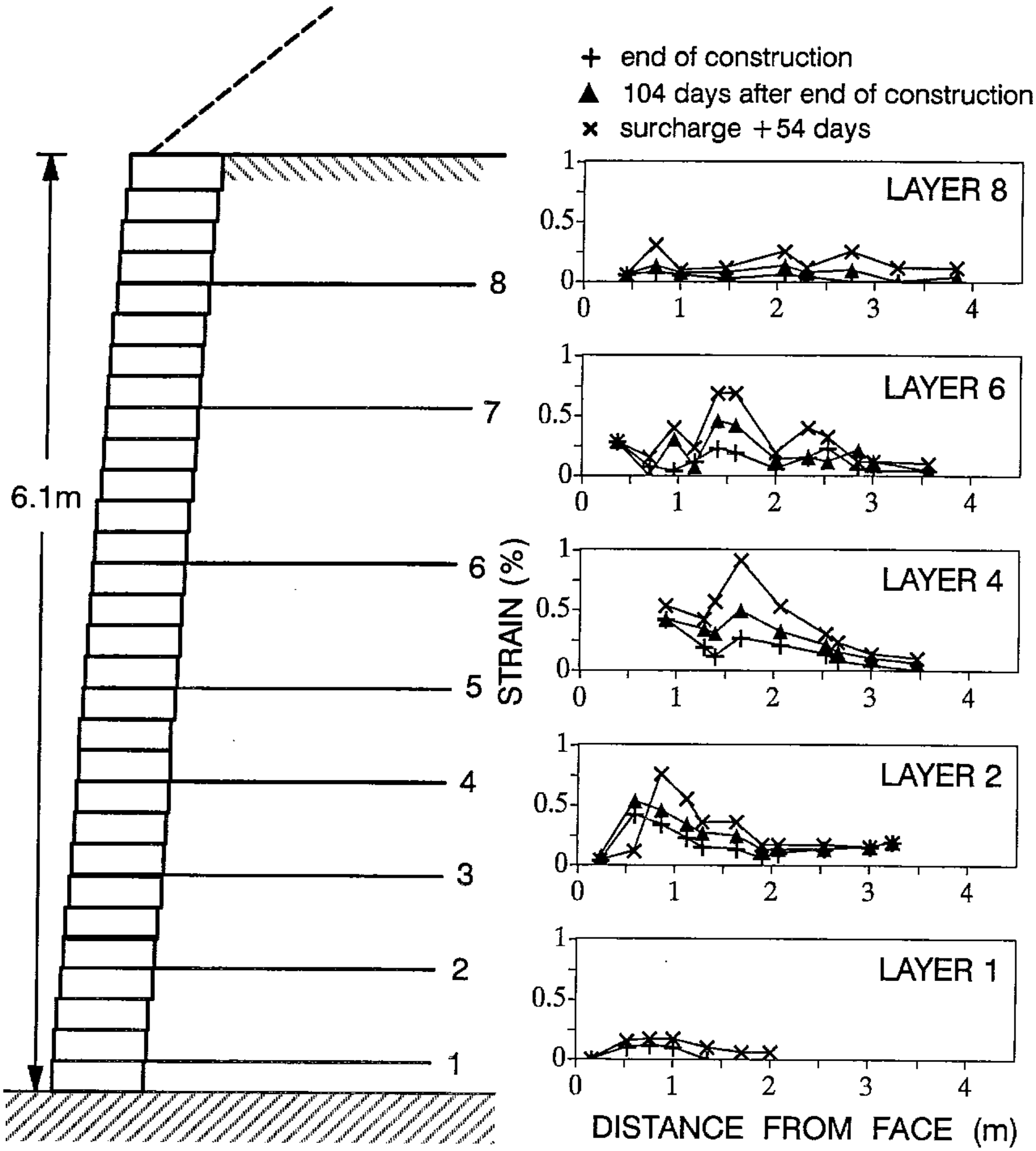

Figure 7 Strain data from modular facing wall test

systems since the former method allows the soil and wall to deform during construction. The results of vertical earth pressure measurements in the FHWA wall prior to surcharging showed that earth pressures were less than γH over the width of the reinforced soil zone. This observation conflicts with predicted high earth pressures in the vicinity of the wall toe (i.e. > γH) that results from assumptions of static equilibrium and a trapezoidal earth pressure distribution in conventional methods of analysis. However, the distribution of lateral earth pressures directly behind the wall did appear to support the linear distribution predicted using conventional Rankine earth pressure theory.

LARGE–SCALE TESTING AT RMC

A research program has been under way for several years at the Royal Military College of Canada (RMC) that is directed at acquiring detailed measurements of the mechanical behaviour of geosynthetic–reinforced soil retaining wall systems. The experimental program involves the construction and testing to failure of carefully monitored full–scale models of geogrid reinforced soil–walls constructed within the RMC Retaining Wall Test Facility. At the time of this paper a total of 13 walls have been tested comprising a variety of facing treatments including wrap–around, incremental and full–height propped panel wall types. Some of the results of these tests have been published elsewhere by the author and co–workers (e.g. Bathurst et al. 1988a,b, 1989, Bathurst and Benjamin 1990)

RMC RETAINING WALL TESTS

The RMC Retaining Wall Test Facility and a typical test configuration are illustrated in **Figure 8**. The test facility was constructed to provide a general purpose large–scale apparatus to test a variety of reinforced soil wall systems. In its current configuration the facility can confine a block of soil 6.0m long by 3.6m high by 3.4m wide. The facility sidewalls are comprised of a composite plywood/plexiglas/polyethylene sheeting that assists to reduce sidewall friction. In a typical test the soil surface is surcharged by inflating airbags at the top of the facility. The current surcharging arrangement allows a vertical pressure of up to 100 kPa to be applied to the upper soil surface. An incremental panel wall construction is illustrated in **Figure 8.** In this method each row of panels is placed sequentially as the height of the retained soil is increased and is temporarily supported until the soil behind the wall has reached the top of each row. The wall facings are constructed with 0.75m high articulated panels and each panel is connected to a separate strip of geogrid reinforcement extending 3m into the soil backfill. In some tests the panels have been bolted together or manufactured in one unit to create full–

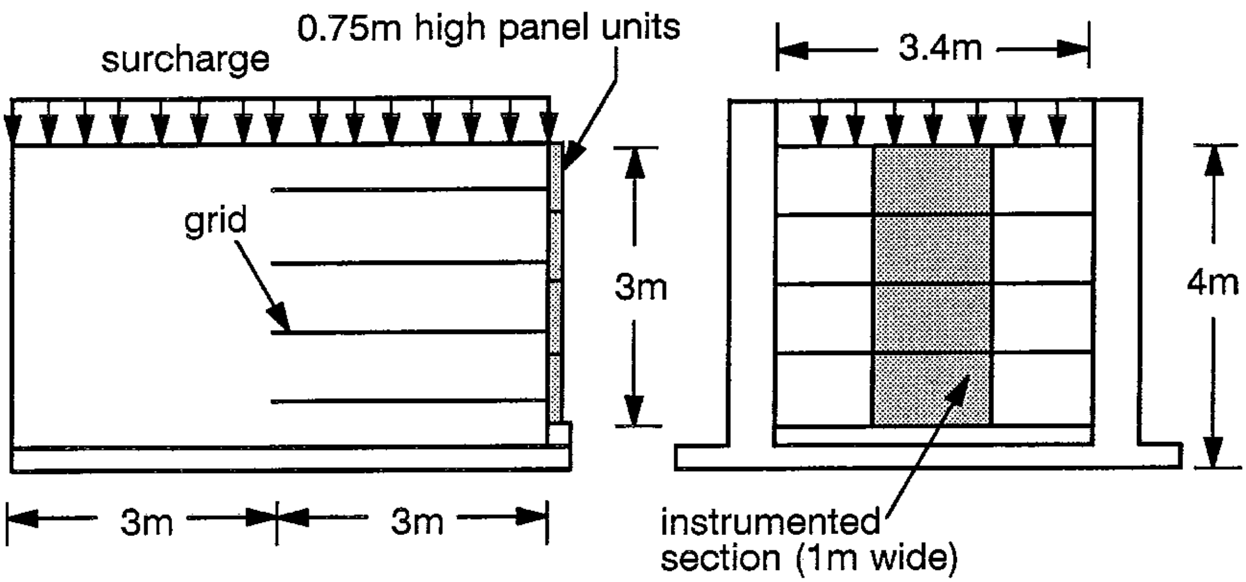

Figure 8 Typical arrangement for RMC incremental panel wall test

height (propped) panels similar to those used for the Highbury Ave structure. In all RMC tests a standard coarse–sized uniformly graded sand has been used as the soil.

Figure 8 shows how the facings are placed in three columns in order to decouple as much as possible the central instrumented panels from the influence of the test facility sidewalls. Four central panels, 0.75m x 1m x 400mm thick, were manufactured out of aluminum in order to mount a variety of instrumentation. All tests to date have used UX1500 (SR2) and BX1100 (SS1) Tensar Geogrids as the geosynthetic reinforcement. The choice of Tensar Geogrids has been largely dictated by the convenience of being able to mount strain gauges directly to the reinforcement ribs. However, initial experience with the relatively high strength/high modulus UX1500 material resulted in wall models that were very stiff and could not be failed with the surcharge capacity at hand (Bathurst et al. 1987). Consequently, a relatively weak and extensible Tensar Geogrid (BX1100) has been used in more recent RMC trial walls and these structures have exhibited excessive deformations and grid rupture leading to wall collapse. The standard procedure following construction is to stage load the test configuration by applying a series of uniform surcharge pressures up to a maximum of 100 kPa. The composite systems exhibit time–dependent deformations under constant surcharge loading which is largely the result of the properties of the constituent polymer in the geogrid. Consequently, each load is typically left on for 100 hours or more to observe time–dependent deformations in the wall and, in particular, creep in the grid reinforcement.

INSTRUMENTATION

Since the RMC test facility is indoors where construction quality can be more carefully controlled and protection of exposed instrumentation is possible, a wider range of instrumentation can be deployed than has been possible in the field projects described earlier. A typical instrumentation arrangement for a recent incremental panel wall test is illustrated in **Figure 9**.Up to 300 electronic devices have been routinely installed in RMC test walls. The data acquisition system was controlled by a micro–computer that was programmed to record the response of all instruments at a selected time–interval (typically 8 hours). However, several critically located devices were monitored continuously and were programmed to trigger full–channel acquisition if significant changes in device output were sensed. In this way significant events in the testing program were captured such as tertiary creep in the grids just prior to wall collapse. A detailed description of the instrumentation used in the RMC tests can be found in the papers by Bathurst (1990a,b).

SELECTED RESULTS

The pattern and magnitude of horizontal facing movements is a primary set of data since facing geometry is an important and obvious indicator of wall performance. **Figure 10** shows wall deformation profiles at selected times during a full–height propped panel wall test. In this particular test a failure occurred through the reinforced soil mass first (curve 9 in the figure). Nevertheless, even after this initial failure mechanism the grids and panels remained intact. Only some time later under continued surcharging did the grid rupture and then it was at the panel connections rather than at the location where internal soil failure along the Rankine plane was observed.

Displacements and strains in the reinforcement inclusions were measured because these parameters allow conclusions to be drawn concerning grid/soil load transfer mechanisms and

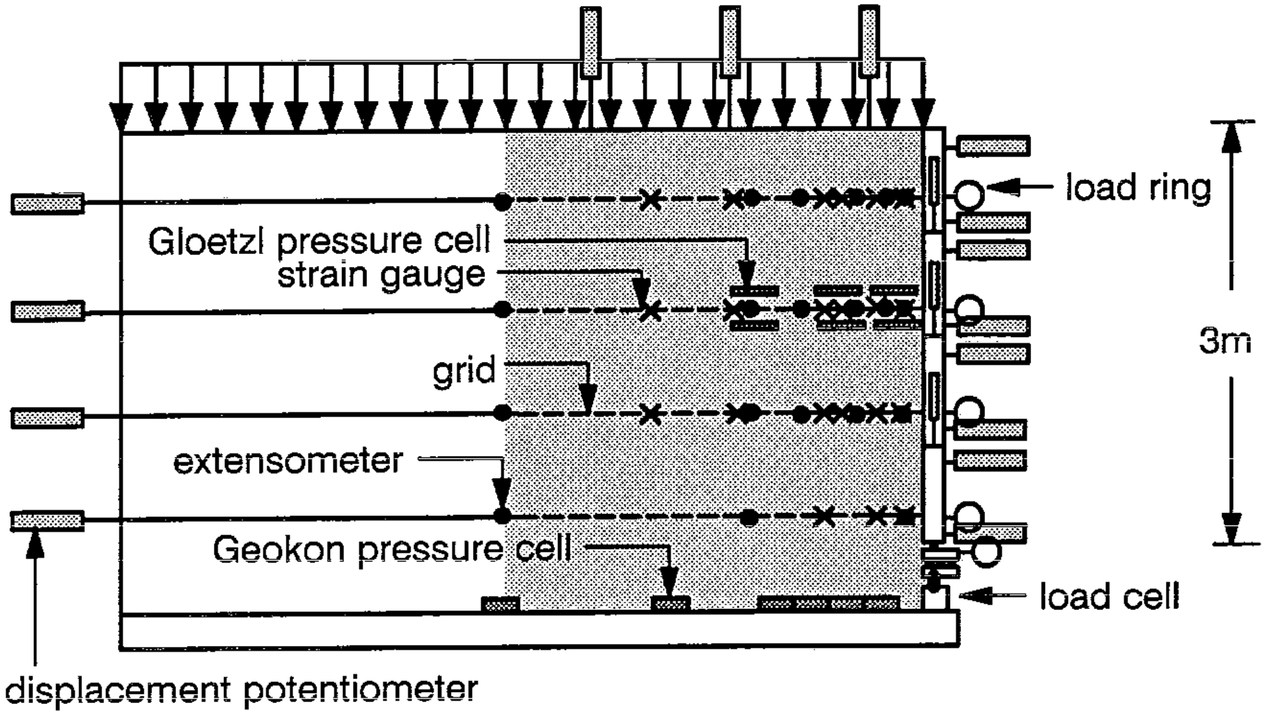

Figure 9 Typical instrumentation arrangement for RMC tests

creep behaviour in polymeric grids. In addition, if the mechanical properties of the grid material are known, grid forces can be estimated and the grid forces used to examine stability of the retaining walls at limiting equilibrium.

The horizontal displacement response of extensometers mounted on grid layer 3 of an incremental wall test is shown in **Figure 11.** The data shows that abrupt changes in grid displacement matched surcharge loading steps and that as the magnitude of surcharging increased there was increased creep deformation in the reinforcement layer. Similar qualitative features were observed in all layers. In this particular test there was a soil–to–soil failure through the reinforced mass of soil after the final load increment had been applied for 93 hours. Approximately 400 hours after soil failure, grid rupture occurred and the wall collapsed. The data in the figure suggests that all significant soil movements were restricted to within a short distance of the panels (i.e. between extensometers 3 and 4). Unlike the propped panel test referenced in **Figure 9**, the grid failure was close to the deep–seated failure surface predicted by the potential Rankine failure plane and not at the connections.

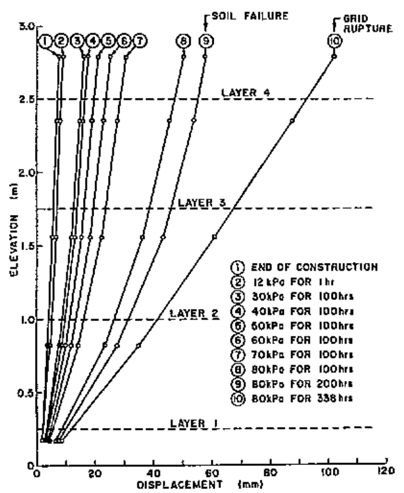

Figure 10 Wall deformation profiles from propped panel wall test

Strains in the grid reinforcement were measured by bonding high–deformation gauges directly to mid–

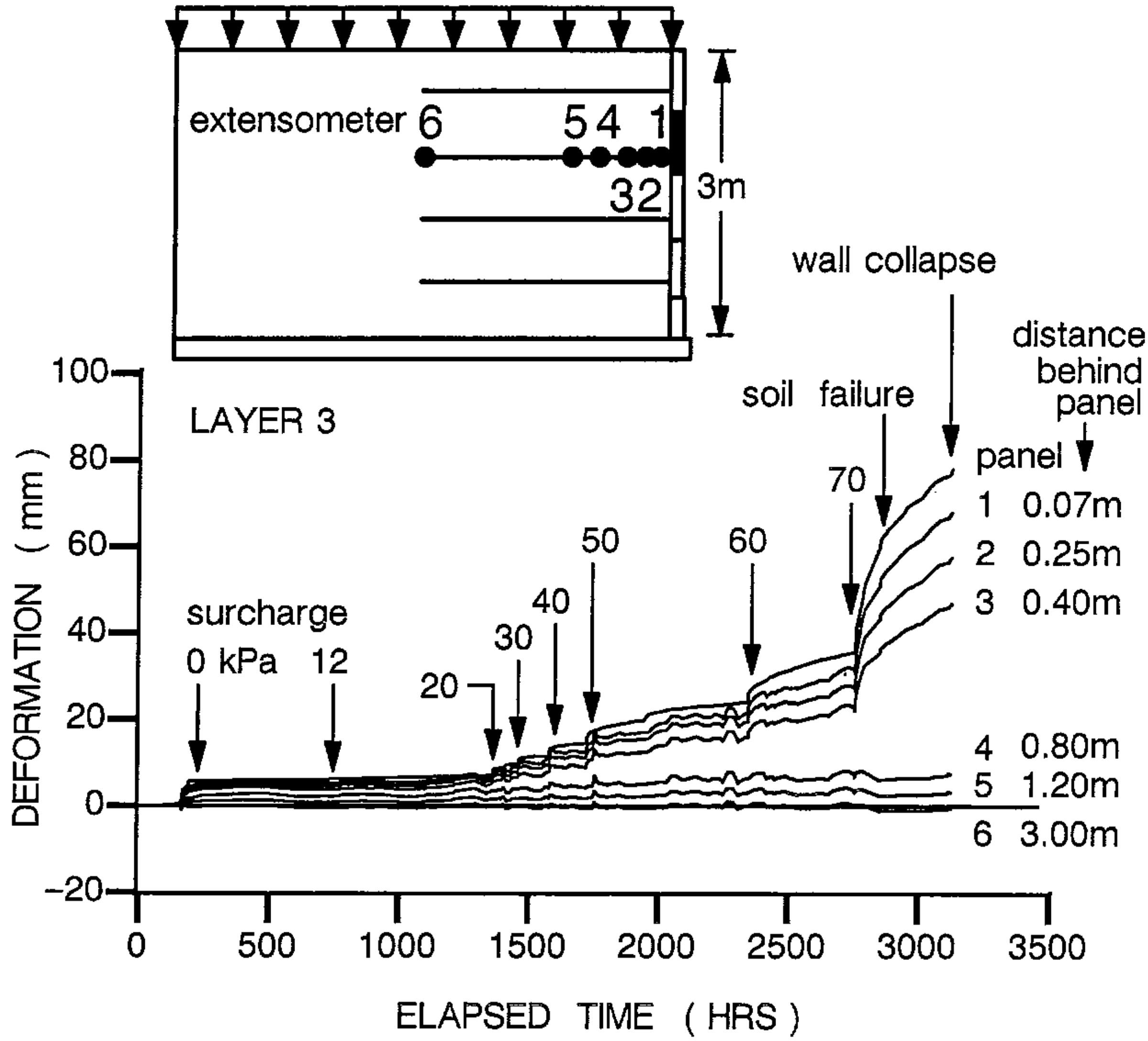

Figure 11 Horizontal grid displacements recorded by extensometers

rib locations on the reinforcement in a similar manner as that described for the Highbury Ave project and by extensometers. **Figure 12** presents grid strain profiles at different times during surcharging of an incremental panel wall constructed with a weak grid. The data in **Figure 12** shows that all significant grid strains were restricted to less than 1.5m behind the panel facings. An implication from these results is that the reinforcement lengths are unnecessarily long even though conventional limit equilibrium-based methods of design for these systems would typically result in grid lengths greater than 2m assuming a design surcharge of 50 kPa. The data on **Figure 13** from the same test shows that grid strains were largest at locations on the reinforcement layers corresponding to the internal failure wedge observed during excavation of the reinforced wall and during excavation of an unreinforced wall that was carried out for comparison purposes. Superimposed on the figure is an approximation to the failure line based on Rankine theory. It appears that at incipient collapse the volume of failed soil is reasonably well-represented by a Rankine failure wedge. A similar preliminary conclusion has been reported by Simac et al. (1990) based on the performance of the FHWA wall discussed earlier. However, as **Figure 12** demonstrates, the pattern of grid strain distribution at working load levels (say 50 kPa) does not reflect the trend observed at the end of the test at 100 kPa

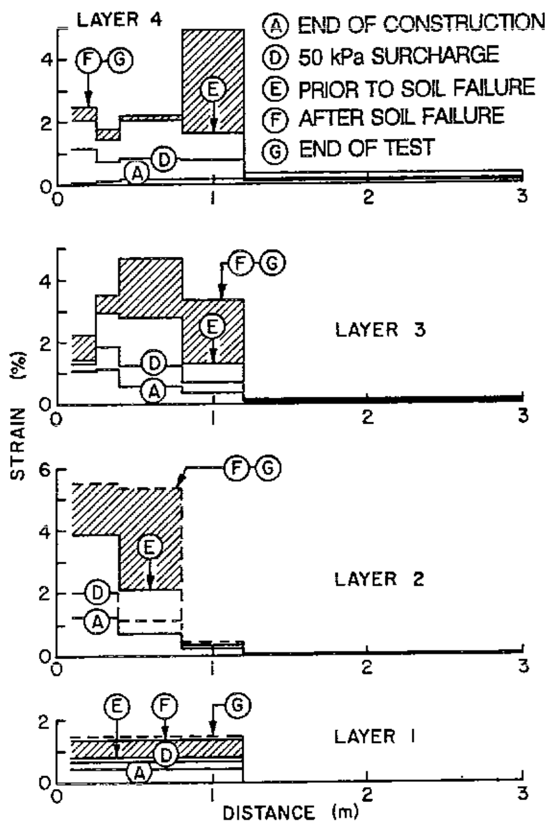

Figure 12 Strain distribution from incremental panel wall test

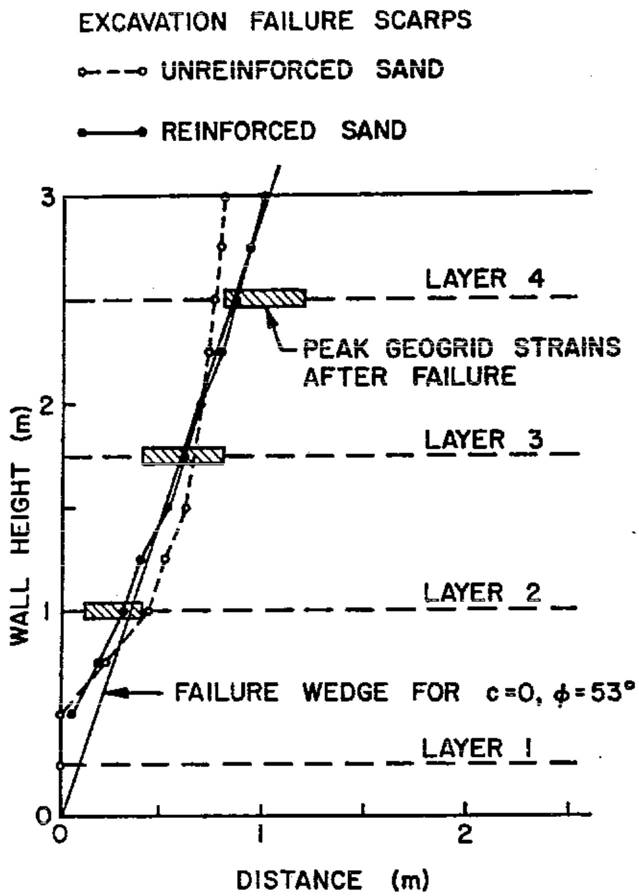

Figure 13 Excavated failure surfaces

surcharge when the wall was close to collapse. This discrepancy highlights the problem of using design methods that attempt to scale conditions at limiting equilibrium to working load conditions.

The calculation of grid forces at locations within the soil mass is difficult owing to the complex load–strain–time–temperature behaviour of the polymeric grids. The mechanical properties of grids in this context has been the topic of investigation by others (e.g. Yeo 1985, McGown et al. 1984). Based on this earlier work, in–isolation tensile testing was carried out on virgin samples of UX1500 and BX1100 grid taken from the same roles of material supplied by the manufacturer. Each sample was subjected to a constant load and temperature for periods up to 1000 hours. The results of this testing were used to estimate the tensile grid forces at selected times during the loading program based on the assumption that the cumulative strain during a surcharge load increment is equivalent to the strain that would have occurred had the surcharge load been applied in a single load step. The results of stage–loaded in–isolation tensile testing of Tensar Geogrids suggests that this is a reasonable assumption (Yeo 1985). The results of grid force calculations confirmed that the early trial wall tests with the relatively strong UX1500 were stable and that grid forces and strains were well below levels associated with long–term rupture.

Geosynthetic reinforced walls constructed from incremental or rigid facing units are built with a concrete footing that serves to support and maintain grade for the facing units. If the stability of the reinforced wall at limiting equilibrium is based on a tie–back wedge analysis then it possible to view the free–body diagram defining the failure wedge as having a restraining force acting at the wall toe as illustrated in **Figure 14**. The diagram also shows forces that must be included in the wedge analysis for the laboratory condition of sidewall friction (Bathurst and Benjamin 1988). The additional toe restraint is not considered in any current methods of analysis known to the authors. In order to monitor these forces a series of load cells were used to support the pin connection at the base of the facing units (**Figure 10**). The base of the wall was restrained in the horizontal direction by a series of proving rings. The results of horizontal force measurements showed that the magnitude of the horizontal toe force was roughly equivalent to the connection forces at wall failure. This observation has important implications to the design of these structures since this additional stabilizing force is not included in wall design but can, as our results have shown, contribute a resistance equal to 20% of the total active force measured at the facings. When all boundary forces are considered, as in **Figure 14**, the measured forces satisfy horizontal and vertical equilibrium at incipient collapse. If they are ignored, as in conventional methods of analysis because of the indeterminate nature of the problem, the actual surcharge pressure to cause collapse is typically twice the predicted value. In practical terms, for short wall heights subjected to uniform surcharge pressure, there is an additional factor of safety of 2 against collapse that is available to the designer but not included in the stability analysis.

The distribution of vertical earth pressures at the base of the RMC walls was measured using a total of six pressure cells. Details of the manufacture, installation and calibration of these devices and interpretation of results can be found in the papers by Bathurst (1990a, b).Typical data for a full–height propped panel wall test is summarized in **Figure 15**. In general, the vertical earth pressure distribution was observed to be roughly equivalent to the value predicted from the combined weight of the applied surcharge and soil self–weight with the exception of the cell closest to the wall. However, the results of vertical toe load measurements were consistent with the results of earth pressure measurements at the base of the reinforced soil mass. In other words, the integrated vertical earth pressure distribution plus the vertical component of wall force was equivalent to the soil self–weight and surcharge force. This data together with similar measurements from other tests indicates that under surcharging a significant portion of vertical soil pressures in the immediate vicinity of the facing panels is carried by the wall using rigid propped panels. The high strains observed in the vicinity of the connections in the Highbury Ave wall are due to the same mechanism.

In selected tests a number of pressure cells were placed above and below grid layer 3 (**Figure 10**). The results of vertical earth pressures measured in the vicinity of grid layer 3 were consistent with qualitative features in **Figure 15** indicating that there is a membrane effect whereby vertical stresses are relieved by the reinforcement inclusion in the area of the connection and transferred to the wall panels.

CONCLUSIONS

The results from the instrumented projects reviewed in this paper serve to illustrate important points in the design, analysis and performance of geogrid–reinforced soil walls. Some of the points made in the paper can be summarized as follows:

1. The type of facing treatment and construction method will influence the tensile strain and forces developed in the geosynthetic reinforcement. The strains in the reinforcement con-

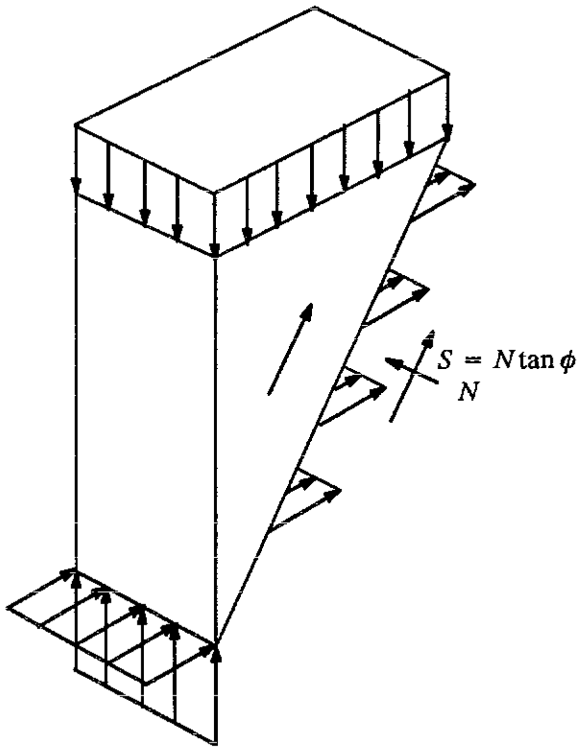

Figure 14 Three–dimensional wedge stability analysis for RMC trial walls

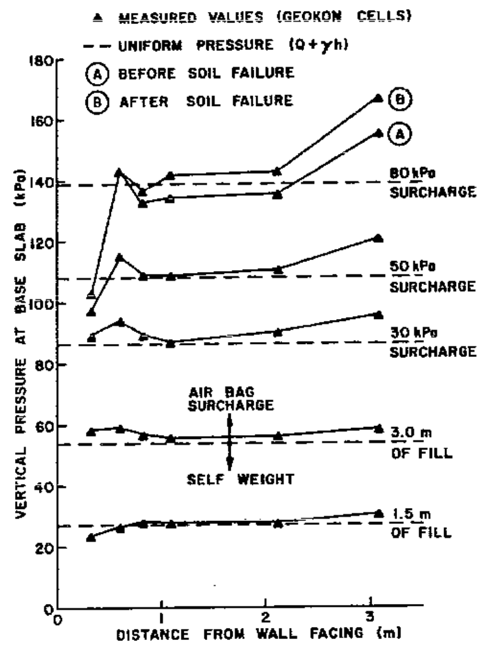

Figure 15 Vertical earth pressures at base of reinforced soil (full–height panel wall)

nections are likely the greatest strains for rigid panel wall systems. For more compliant wall systems constructed with incremental panels or modular facing units the greatest strains are at the the location of the internal Rankine failure plane.

2. Toe restraint due to the footing below the facing units may account for a significant additional system capacity that is not considered in conventional analysis.
3. The anchorage length in grid–reinforced soils is shorter than that required using conventional methods of analysis.
4. Vertical earth pressures below the reinforced soil zone in two of the cases presented were shown not to exceed that calculated from the combined influence of soil self weight and magnitude of uniform surcharge.
5. Conventional limit–equilibrium methods of analysis based on a tie–back wedge method of analysis are conservative when used to predict reinforcement forces and surcharge capacity of grid–reinforced soil wall systems.
6. Wall deformations for the two field constructed projects reviewed here were within tolerable amounts. However, the experience of the authors has been that wall deformations are controlled by the quality of construction.

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