Instrumentation of Geogrid-Reinforced Soil Walls

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An experimental program involving the construction and monitoring of large-scale geosynthetic-reinforced soil walls has been under way at the Royal Military College (RMC) of Canada for several years. Several 3-m-high model walls have been built within the RMC Retaining Wall Test Facility and these walls taken to failure under uniform surcharge loading. The instrumentation, calibration of equipment, data acquisition, and monitoring strategies that have been developed over the course of this program are described. Examples of qualitative features of model wall behavior during construction, under working load conditions, and at incipient collapse are given. These examples highlight the success of the instrumentation program to date.

A research program conducted by the Civil Engineering Department at the Royal Military College (RMC) of Canada has been under way for several years. The program is directed at acquiring detailed measurements of the mechanical behavior of soil retaining walls constructed with polymeric reinforcement (i.e., geosynthetics). 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. The data collected can then guide the development of physically correct models that are needed to design geosynthetic reinforced retaining walls and predict their performance in the field.

A total of 10 reinforced soil walls constructed with a variety of facing treatments have been tested to date. The results of these tests have been reported by the author and coworkers in other publications (*1-5*).

This paper is focused on a description of the instrumentation that has been developed and employed in recent wall tests to allow researchers to measure the load-deformation response of these composite structures during construction, under working load conditions, and at collapse of the structures during surcharging.

RMC RETAINING WALL TEST FACILITY

The RMC Retaining Wall Test Facility was constructed to provide a general purpose, large-scale apparatus to test a variety of reinforced soil wall systems (Figures 1 and 2). The principal structural components of the facility are six rigid heavily reinforced concrete counterfort cantilever wall modules that are used to confine a block of soil 6.0 m long by 3.6 m high by 2.4 m wide. The facility sidewalls are composed of a composite plywood-plexiglas-polyethylene sheeting that assists to reduce sidewall friction. Shear box tests have shown that the sand-sidewall interface has a fully mobilized friction angle of 15 degrees. Three dimensional stability analyses have shown that the friction-reducing sidewall construction reduces the contribution of the test facility boundaries to less than 15 percent of the total active earth force that would be resisted by the facings in a true plane strain condition (*6*).

In a typical test the soil surface is surcharged by inflating airbags that are confined between the concrete modules and structural steel sections 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.

Typical Test Configurations

Typical test configurations are shown in Figure 3. Figure 3(a) represents an incremental wall construction technique in which rows of panels were placed sequentially as the height of the retained soil was increased. Each row was temporarily supported until the soil behind the wall had reached the top of the panels. In this way a portion of the load-carrying capacity of the grid reinforcement layers was mobilized as construction proceeded. The wall facings were constructed with 0.75 m high articulated panels. Each panel was connected to a separate strip of geogrid reinforcement extending 3 m into the soil backfill. In some tests the facings comprised single full-height (propped) panels (*1,2,5*) and in others a wraparound fascia (*3*). For the propped wall construction illustrated in Figure 3(b), single full-height panels were used and the external support to these panels was only released after the retained soil had been placed and compacted to the full height of the model wall.

The panels in the incremental and propped wall tests were constructed 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, 0.75 m by 1 m by 400 mm thick, were manufactured out of aluminum and designed to support a variety of instrumentation. The panels were mounted independently to form an articulated incremental wall or bolted together to form a full-height (propped) panel wall. The base of each panel column was supported by a pinned connection. The pinned connection was in turn supported by an instrumented levelling pad, which corresponds to the footing that provides support and alignment for similar fascia in the field.

All tests carried out to date have used SR2 and SS1 Tensar geogrids as the geosynthetic reinforcement (Figure 4 and Table 1). The choice of Tensar geogrids has been largely dictated by the convenience of being able to mount strain gauges directly

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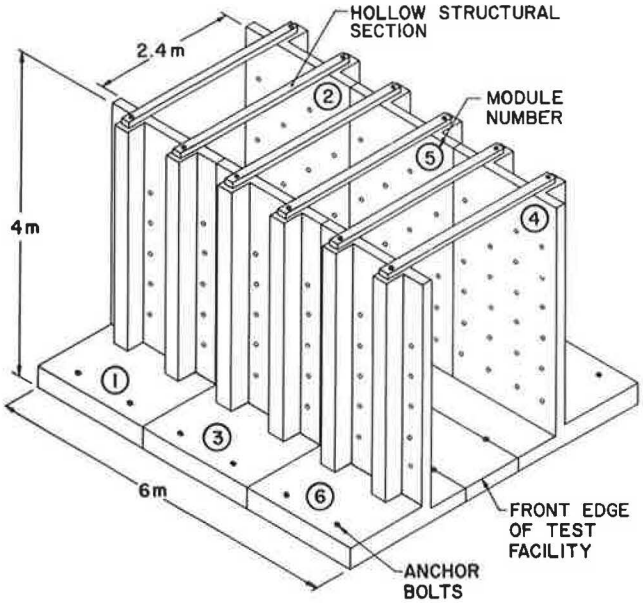


FIGURE 1 RMC Retaining Wall Test Facility.

to the reinforcement ribs. However, initial experience with the relatively high-strength, high-modulus SR2 material resulted in wall models that were very stiff and could not be failed with the surcharge capacity at hand (*1,2*). Consequently, a relatively weak and extensible Tensar geogrid (SS1) has been used in more recent RMC trial walls, and these structures have exhibited excessive deformations and grid rupture leading to wall collapse. Finally, it should be noted that a coarse sand material has been used in all RMC model tests carried out to date.

General Test Procedure

The performance of test walls was carefully monitored, commencing at construction and ending at failure under uniform surcharging. The standard procedure following construction was to stage load the test configuration by applying a series of uniform surcharge pressures up to a maximum of 100 kPa. The composite systems exhibited time-dependent deformations under constant surcharge loading, which is largely the result of the properties of the constituent polymer in the geo-

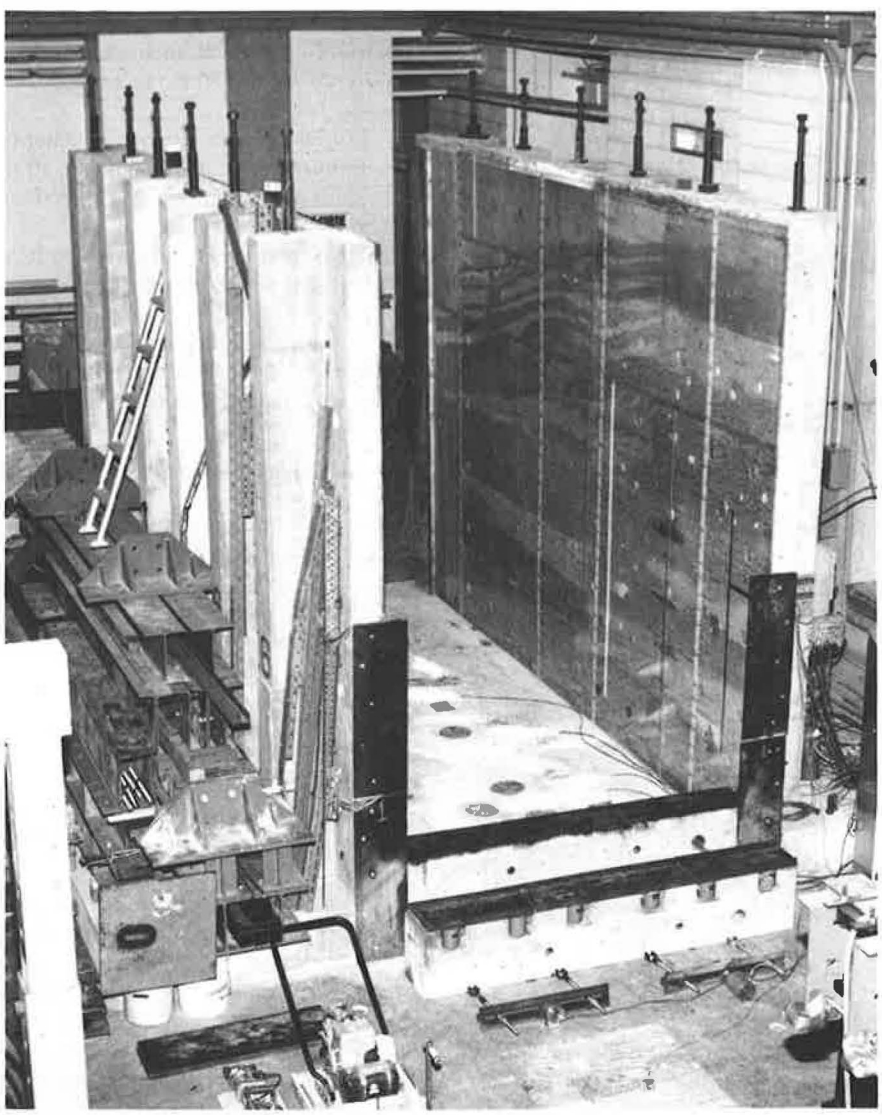


FIGURE 2 Overview of RMC Retaining Wall Test Facility.

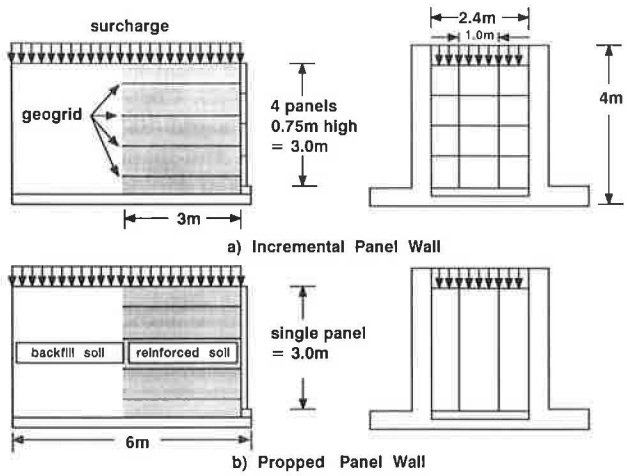


FIGURE 3 Typical test configurations: (a) incremental panel wall; (b) propped panel wall.

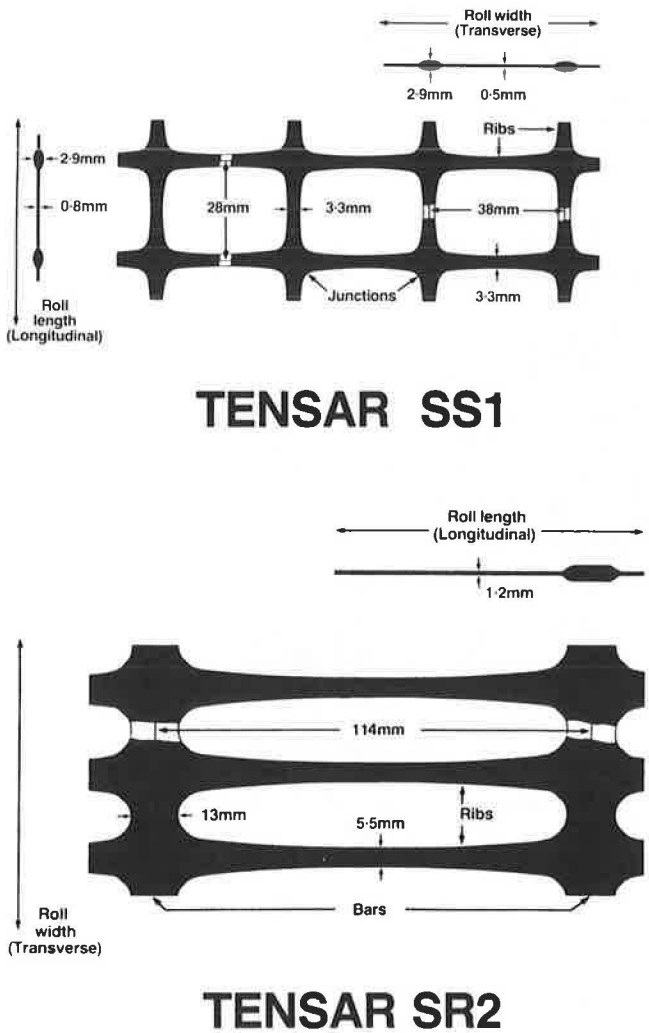


FIGURE 4 Tensar geogrids: (*bottom*) Tensar SR2; (*top*) Tensar SS1.

TABLE 1 MECHANICAL PROPERTIES OF GEOGRID REINFORCEMENT MATERIALS (ASTM D4595 WIDE WIDTH STRIP TENSILE/ELONGATION TEST)

Material	Stiffness @ 2% strain (kN/m)	Peak Load (kN/m)	Strain @ Peak Load (%)
Tensar SR2[a]	1,096	79	17
Tensar SS1[b]			
transverse (strong)	292	20	14
longitudinal (weak)[c]	204	12	14

[a]High-density polyethylene uniaxial grid.
[b]Polypropylene biaxial grid.
[c]SS1 oriented in weak direction for RMC trial walls.

SOURCE: Manufacturer's literature.

grid. Consequently, each load was left on for a minimum of 100 hours to observe time-dependent deformations in the wall and, in particular, creep in the grid reinforcement.

INSTRUMENTATION

General

The following measurements were considered to be of primary importance in the RMC wall tests:

1. horizontal and vertical movements of the facing units;
2. reinforcement displacements, strains, and forces;
3. loads at the panel-grid connections;
4. vertical earth pressures;
5. horizontal and vertical toe loads; and
6. internal soil displacements.

The instrumentation used to make these measurements is described in the following sections, and an instrumentation layout used in a typical test is shown in Figure 5.

Horizontal and Vertical Movements of Facing Panels

The pattern and magnitude of horizontal facing movements is a primary set of data, because facing geometry is an important and obvious indicator of wall performance. Spring-loaded hybrid track rectilinear (HTR) potentiometer displacement devices manufactured by Penny and Giles Potentiometers Limited were used to measure wall deformations. These devices operate on a potentiometer principal and were powered by 10 volt DC excitation. The devices gave an accuracy of ± 0.5 mm and were relatively cheap and extremely robust. In some instances, devices that were in pieces due to wall collapse were reconstructed and used again with no performance deterioration. Figures 6(a) and 6(b) show typical panel deformation profiles recorded at the end of construction, at the end of the 50 kPa surcharge increment, and just prior to wall failure in two recent tests that used a very extensible reinforcement (Tensar SS1). Figure 6(b) shows that the largest panel movements in the incremental wall occurred in Panel 3. However, subsequent tests have shown that the relative movement in incremental panel movement profiles is critically dependent on the details of panel placement and alignment during the construction phase.

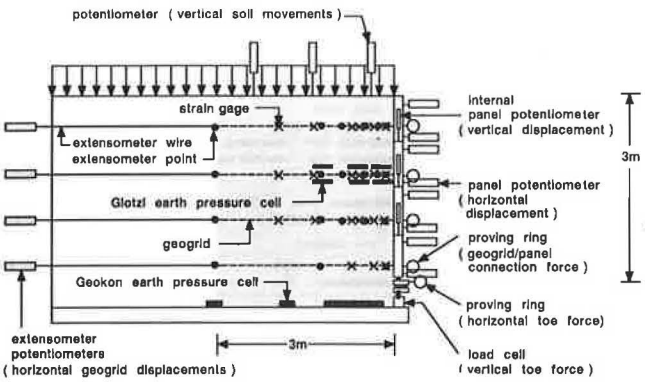


FIGURE 5 Typical layout of principal instrumentation for incremental panel wall test.

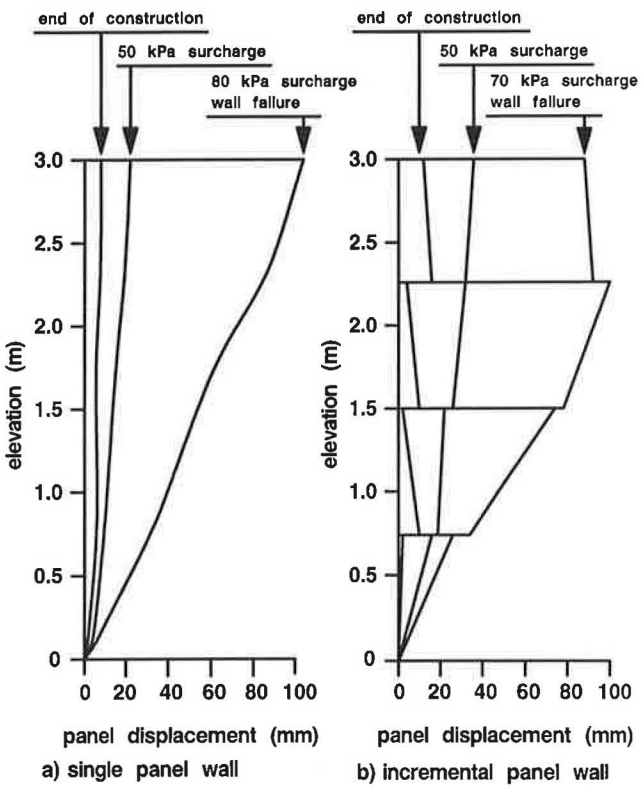


FIGURE 6 Panel displacements using SS1 geogrid: (a) single panel wall; (b) incremental panel wall.

Reinforcement Displacements, Strains, and Forces

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

Total horizontal displacements at selected grid locations were monitored by extensometers attached to grid junctions. These devices were constructed in-house and comprised a thin galvanized steel line attached to a miniature bolt and eye attachment passing through the grid junction. The extensometer wire was protected and isolated from the soil by passing it through a stiff plastic brakeline tubing. The wires were attached to HTR potentiometers mounted on a rack at the back of the test facility (Figure 5). The horizontal displacement response of extensometers mounted on grid layer 3 of an incremental wall test is shown in Figure 7. The data show 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. Figure 8 plots displacement profiles in grid layers as recorded by the extensometer devices from several tests at wall failure. The data shows that all significant grid deformations were restricted to less than 1.5 m 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 2-m, assuming a design surcharge of 50 kPa.

Strains in the grid reinforcement up to 2 or 3 percent strain were measured by bonding high-deformation gauges directly to mid-rib locations on the reinforcement. The combination of grid type, grid surface preparation technique, and type of glue was developed at RMC after much experimentation (*7*). A high-strain foil-type gauge manufactured by Showa Measuring Instruments Co. Ltd. (Type Y11-FA-5-120) has proved successful with both polypropylene and high-density polyethylene Tensar geogrids. Grid surface preparation involved abrading the surface of the rib with a fine grit sand paper, surface cleaning, surface neutralization followed by bonding of the gauge to the grid using a RTC two-part epoxy resin cement. The gauges and lead wires were protected by a waterproof bubble of silicon and the silicon wrapped with flexible plastic tubing. The strain gauging technique has proved very reliable, and a 100 percent success rate following placement in soil is routine. The same gauge and bonding technique has

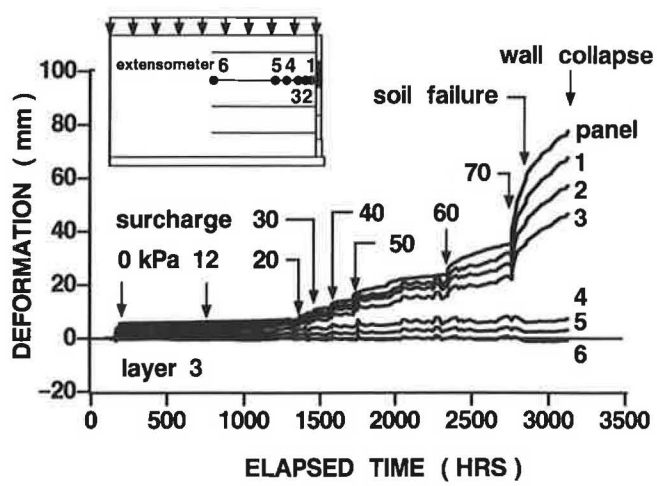


FIGURE 7 Horizontal grid deformations measured by extensometers (incremental panel wall test).

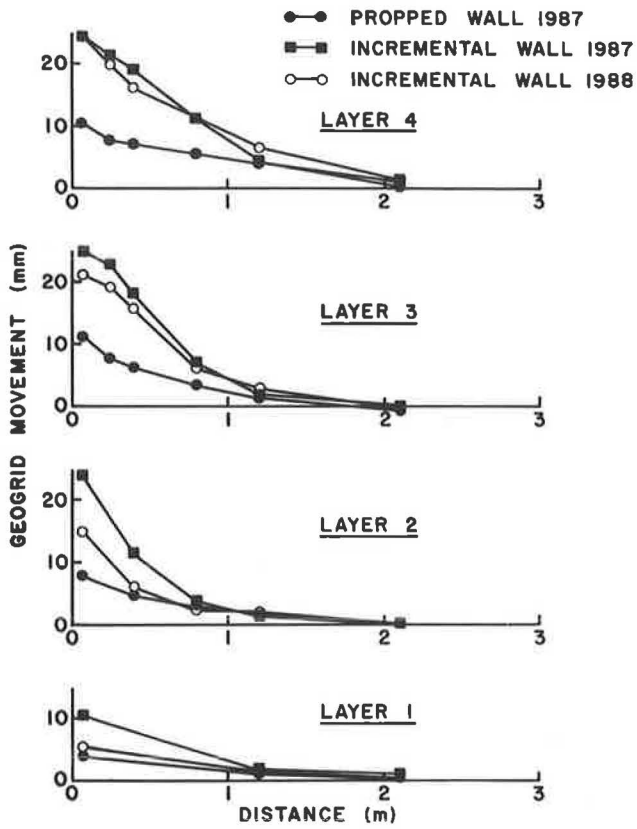


FIGURE 8 Distribution of grid displacements (incremental and propped panel wall tests).

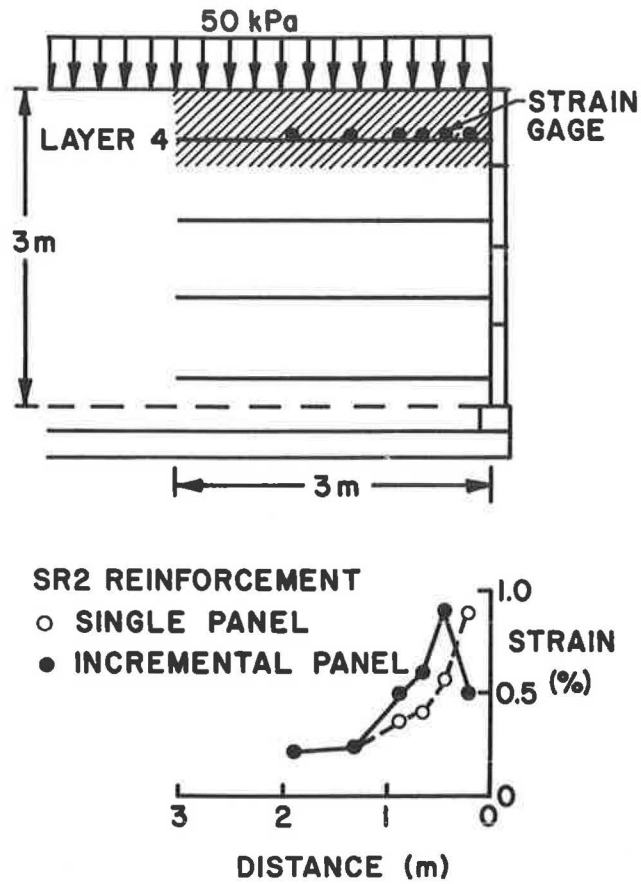


FIGURE 9 Influence of wall type on grid strains.

been successful in more aggressive environments, including strain-gauged grids placed at the interface between granular bases and peat subgrades (*8*) and in hot mix asphalt pavements (*9*).

Experience with full-scale geogrid-reinforced soil walls constructed with Tensar SR2 geogrid showed that the strains in the grid did not exceed 1 percent strain even under surcharging to 50 kPa (*1*). At this level of strain the gauges were adequate. However, when trial walls with a very weak grid were constructed, these grids exhibited prerupture strains as high as 10 percent, by which time the strain gauges had debonded (*2,4,5*). Nevertheless, at large strains the extensometer movements were such that, after about 3 percent strain in the grid, extensometer movements were great enough that large-strain measurements could be calculated with confidence from the array of extensometers attached to each grid layer. An example of strains measured along the length of the topmost grid layer in incremental and propped walls reinforced with Tensar SR2 is shown in Figure 9. The profiles indicate that significant grid strains only extend to about 1 m into the soil when a 50 kPa surcharge pressure was applied to the models. The maximum strain is less than about 1 percent strain.

Also of importance is the difference in the pattern of strain between these two tests. The propped panel wall showed maximum strain close to the connection, while the grid with the incremental panel showed a peak strain at working load conditions that was located back from the wall. The trend toward peak strains in the vicinity of the connections in the propped wall case is thought to be due to the relative downward movement of the retained soil with respect to the panels, which are restrained in the vertical direction. The incremental panels, on the other hand, were constructed with a compressible foam layer between panels that reduced the magnitude of relative movement. An important implication to the design of propped wall systems is that, at working load levels, the largest grid strains are likely to occur at the connections rather than at locations within the reinforced soil mass as predicted by tie-back wedge methods of analysis. This is particularly true in the field, where voids in the soil directly below the connections are inevitable. Consequently, tensile loads due to a membrane effect can be anticipated for these grids immediately behind the wall, in addition to the tensile loading associated with grid anchorage. Details of connection arrangements to minimize connection strains have been reported by Jones (*10*).

Figure 10 shows grid strain profiles at different times during surcharging of an incremental panel wall constructed with a weak grid (*4*). The data for Figure 11 show 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 (*6*). Superimposed on the figure is an approximation of 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. However, as Figure

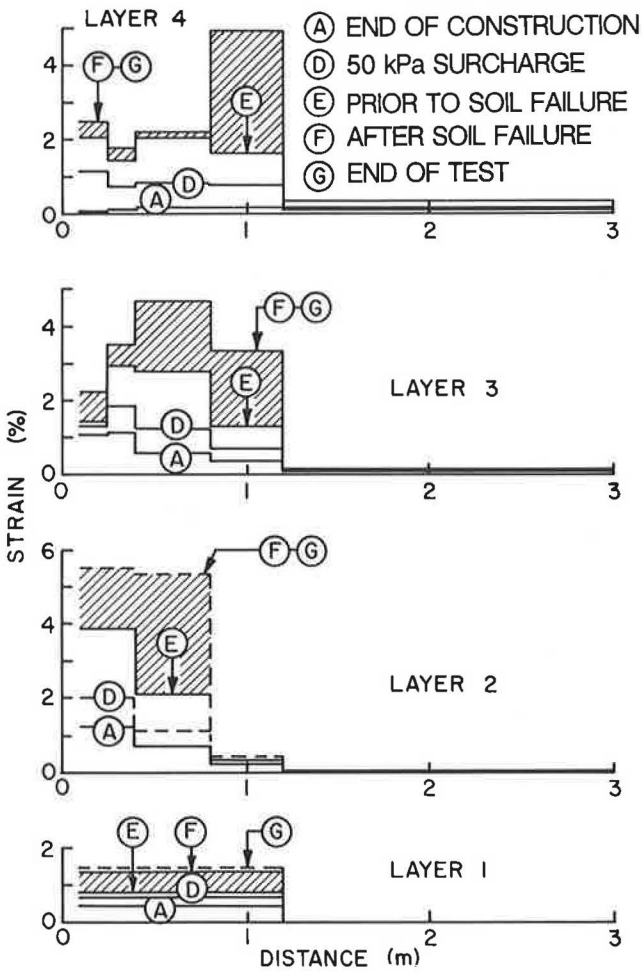


FIGURE 10 Distribution of grid strains (incremental panel wall test).

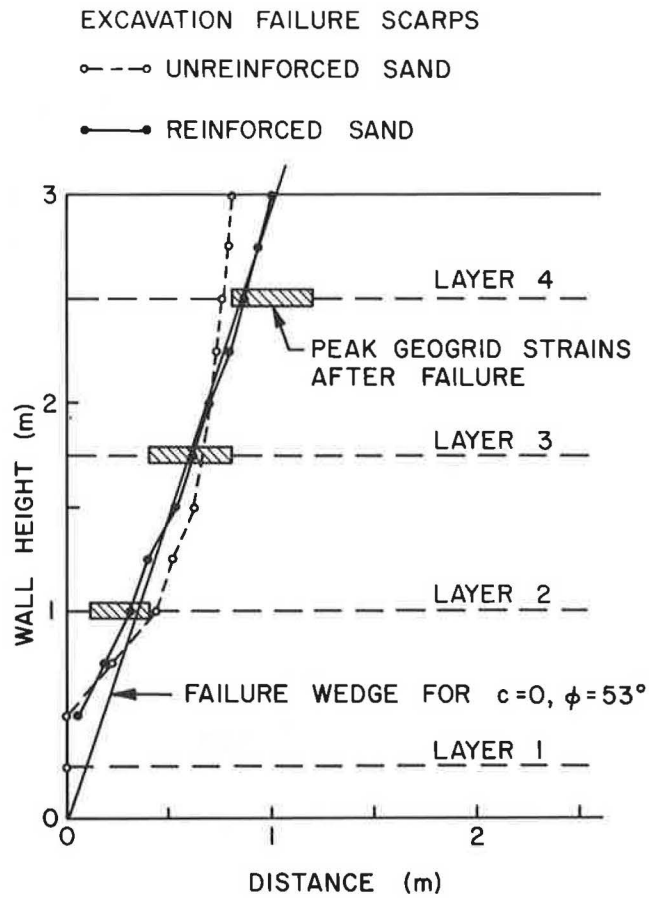


FIGURE 11 Excavated failure surfaces.

10 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 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 results of in-isolation calibration tests with uniaxial SR2 and biaxial SS1 grids loaded in the longitudinal (weak) direction has shown that gauges mounted at mid-rib location record strains that are sensibly equivalent to the gross strain in the sample measured over several grid apertures. However, this is not necessarily true of all grid materials. The location of the gauge, the geometry of the rib, and the modulus of the highly oriented polymer all influence gauge registration.

The strain gauges in the RMC walls were placed in rows such that there were three gauges at nominally identical locations from the back of the facings. This procedure ensured a representative average strain at nominally identical locations, since the strain gauge response can be influenced by small variations in the location of the gauges on the rib and the local effects of soil in contact with the gauge-grid assembly. In addition, it has been noted that grid tensile strains may not be attenuated uniformly along rib lengths, owing to (1) inherent warps during attachment and laying out of the grid, and (2) skewness in the grid as manufactured.

The calculation of grid forces at locations within the soil mass is difficult, owing to the complex load-strain-time-temperature behavior of the polymeric grids. The mechanical properties of grids in this context have been the topic of investigation by others (*11,12*). Based on this earlier work, in-isolation tensile testing was carried out on virgin samples of grid taken from the same rolls of material supplied by the manufacturer. Each sample was subjected to a constant load and temperature for periods up to 1,000 hours. The in-isolation testing temperature of 20°C corresponds to the ambient temperature of the RMC test facility. The results of testing of this type on Tensar SR2 are presented in Figure 12 in the form of isochronous load-strain curves and a Sherby-Dorn plot (*1*). These data were used to estimate the tensile grid forces at any time 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 suggest that this is a reasonable assumption (*11*). A similar tensile testing program was carried out with samples of SS1 Tensar geogrid and the data used to estimate grid forces in the RMC trial walls constructed with a weak reinforcement. The results of grid force calculations confirmed that the early trial wall tests with SR2 were stable and that grid forces and

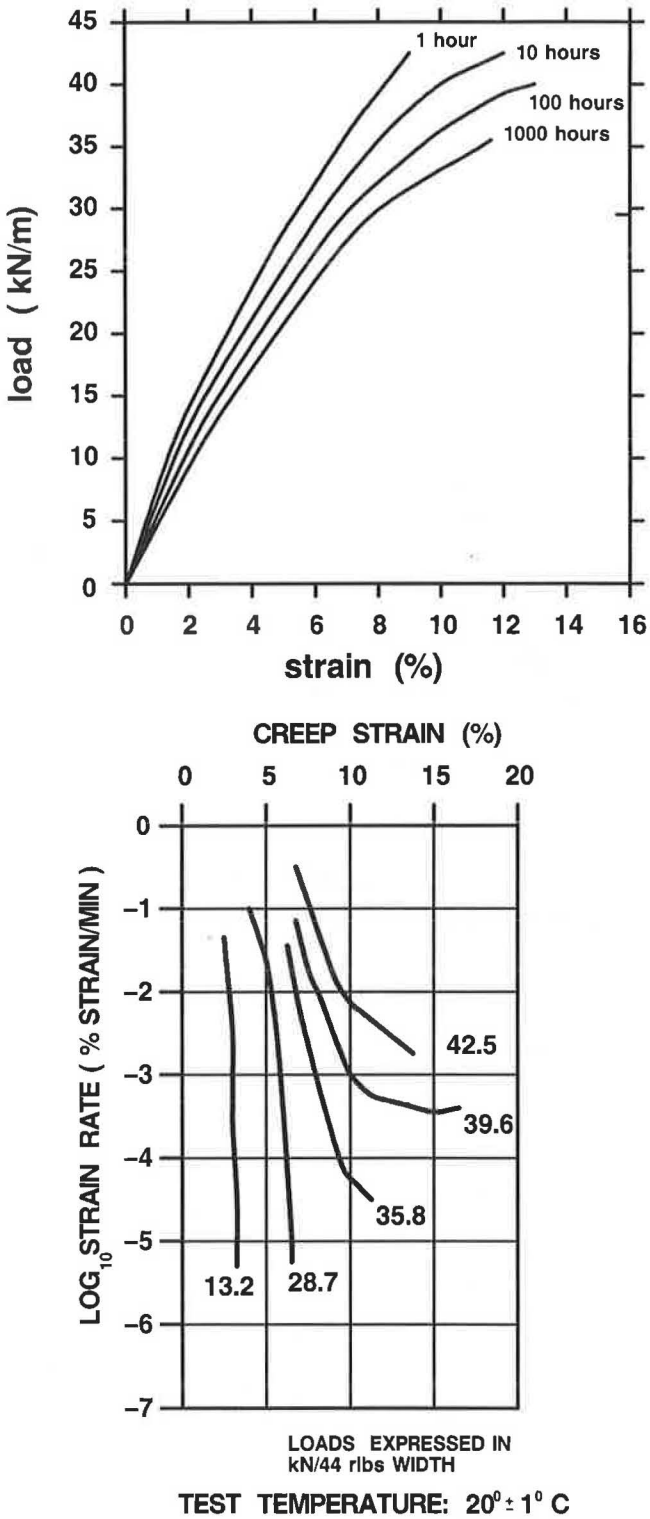


FIGURE 12 Load-strain-time properties of Tensar SR2 geogrid: (top) isochronous load-strain curves; (bottom) Sherby-Dorn plot.

strains were well below levels associated with long-term rupture. Similarly, walls that have failed using SS1 grid material showed that inferred tensile forces were consistent with the restraining forces required to maintain a wedge-shaped zone of reinforced soil at the point of incipient failure (*4*) and that measured connection forces were consistent with inferred grid tensile forces in the vicinity of the connections when incremental facings were used.

Connection Forces

Loads generated at the connection between the panel and the grid reinforcement can be used to estimate the distribution of lateral earth pressure acting at the facings. The calculation of lateral earth pressures is a routine step in many current methods of reinforced soil wall design.

Connection loads have been measured using a series of proving rings connected to the grid layers. The essential features of these devices are shown in Figure 13. Five proving rings per layer were used. The grids were clamped to a plate using a bolt and angle arrangement, and the clamp was connected in turn to the proving rings by high-strength stainless steel rods passing through a series of bushings to the proving rings.

An example of the response measured in a recent reinforced wall taken to failure is illustrated in Figure 14. The forces measured at the connections show that they are sensitive to surcharge load level and that there is time-dependent load shedding to the facing units. An interesting feature is that prior to the final 70 kPa load increment there was a nonuniform distribution of connection loads. However, at incipient failure the connection loads appear to have become uniform, suggesting that as the collapse condition is approached there is a tendency of load redistribution in the vicinity of the connections.

Vertical Earth Pressures

The distribution of vertical earth pressures at the base of the wall was measured using a total of six Geokon EP-3500 pressure cells. Each cell is constructed from two circular stainless steel plates welded together along their perimeter to create a narrow cavity, which is filled with an incompressible fluid (Figure 15). The cells are 230 mm in diameter and have an aspect ratio of 18 (diameter/thickness ratio). A length of stainless steel tubing connects the pressurized cavity to a housing that contains a semiconductor strain gauge pressure transducer. The pressure cells were modified by the manufacturer with low-pressure transducers to ensure adequate sensitivity to the relatively low vertical earth pressures anticipated for these walls (i.e., maximum of 160 kPa).

Two problems routinely present themselves when using earth pressure cells for the purpose described here: the difficulties associated with seating the instrument so that the pressures at the point of measurement are not altered by the installation; and accurate calibration of the device. The installation problem was overcome by placing the cells in a plaster of paris layer so that the face of the cell was flush with the surface of the plaster. The plaster of paris was extended over a wide area so that arching of the soil in the vicinity of the active face was prevented, and the connecting tube and housing were also rigidly seated.

A number of calibration techniques were considered. Calibration in air was not attempted, since general experience with a number of earlier cells has shown that calibration in air does not necessarily give the in situ response despite nominally identical pressures. Instead, as a first attempt, the cells

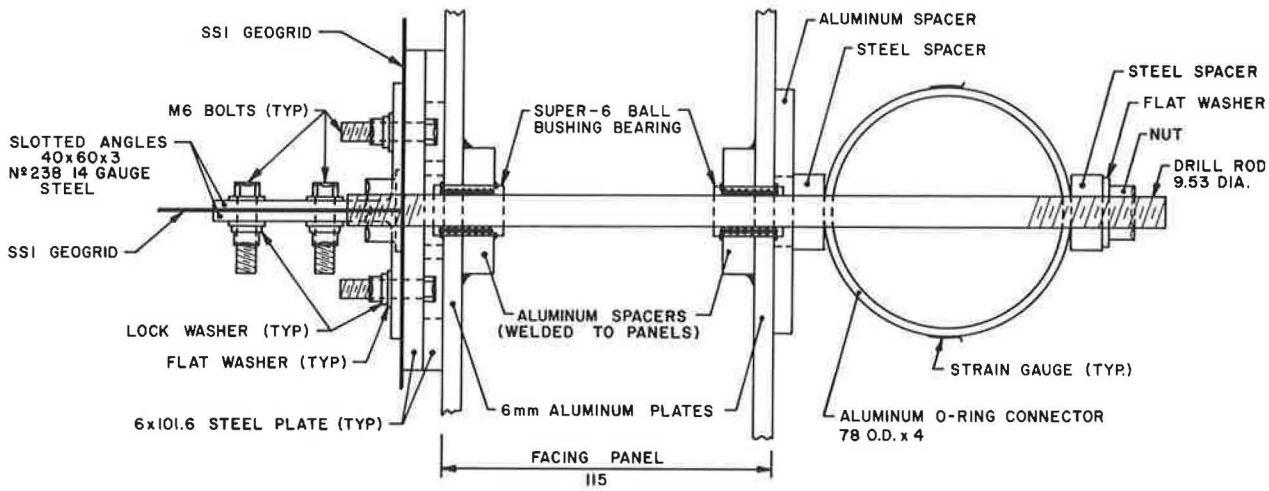
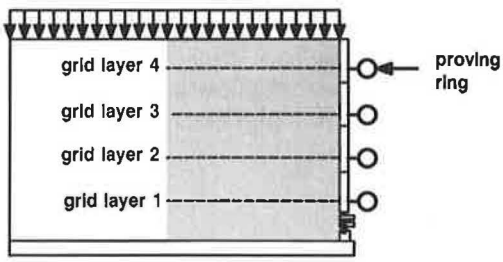


FIGURE 13 Proving ring arrangement for connection load measurement (all dimensions in mm).



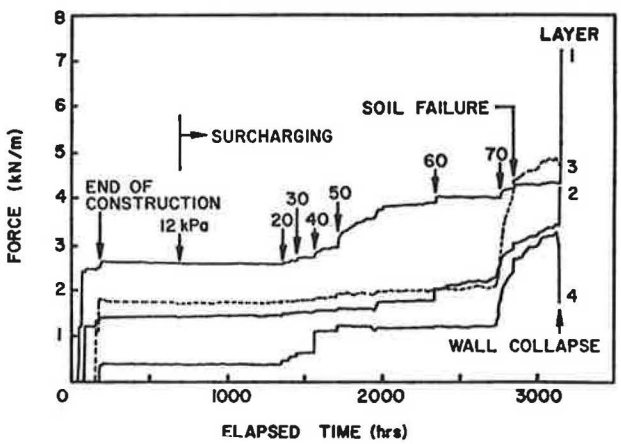


FIGURE 14 Connection forces (incremental panel wall with SS1 geogrid reinforcement).

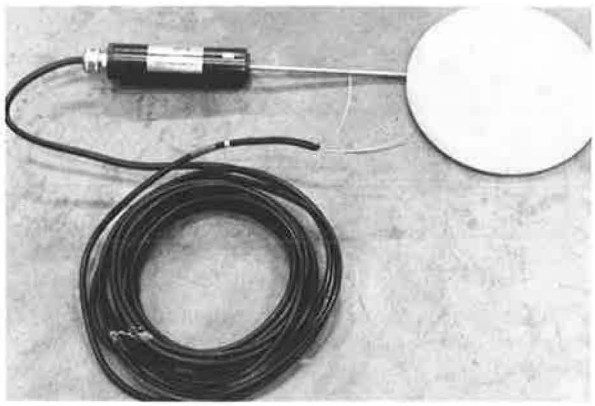


FIGURE 15 Geokon EP 3500 earth pressure cell (diameter = 230 mm).

were placed in a modified 250 mm diameter Rowe oedometer. However, the response of the cells was found to be sensitive to details of placement and compaction of the sand soil and oedometer edge effects. The most successful method was to calibrate the Geokon pressure cells in situ. In this approach the cell response was determined based on the unit weight of the soil placed to a depth of 1 m over the cell during the initial stages of wall construction. Once this calibration was established, it was used to determine the response of the cell at all subsequent stages in the loading program. This method avoided the difficult problem of replicating in situ placement conditions within a calibrating device.

The Geokon cells proved very successful in identifying the distribution of earth pressures at the base of the reinforced soil mass. Typical data for an incremental panel wall test are summarized in Figure 16. 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. These data, together with similar measurements from other tests, indicate that under surcharging a significant portion of vertical soil pressures in the immediate vicinity of the facing panels is carried by the wall.

Horizontal and Vertical Toe Forces

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 is possible to view the free-body diagram defining the failure wedge as having a restraining force acting at the wall toe. This additional restraint is not considered in any current methods of analysis known to the author. Of particular interest has been the magnitude

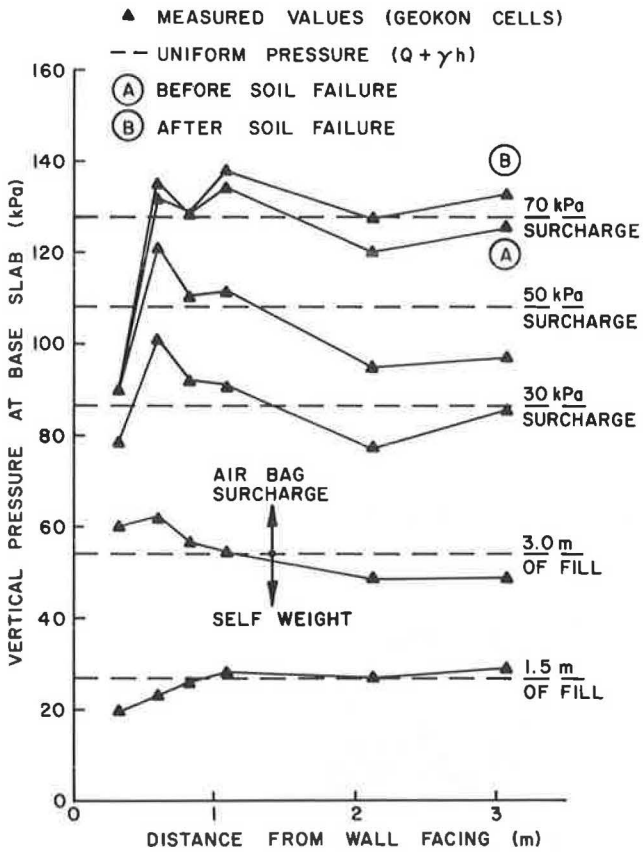


FIGURE 16 Distribution of vertical earth pressures at base of reinforced soil (incremental panel wall with SS1 geogrid reinforcement).

of the vertical and horizontal forces carried by the levelling pad in walls having a generic construction similar to the RMC trial walls.

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 5). The base of the wall was restrained in the horizontal direction by a series of proving rings manufactured in-house. The results of horizontal 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. The results of horizontal force measurements showed that the magnitude of vertical toe force was roughly equivalent to the connection forces at wall failure. This observation has important implications for the design of these structures, since this additional stabilizing force is not routinely included in reinforced wall design but can, as these results have shown, contribute a resistance equal to 20 percent of the total active force measured at the facings.

Additional Instrumentation

An array of instrumentation of secondary importance was deployed in recent RMC trial wall tests and is described briefly here for the purpose of completeness.

A number of 30 mm Bison inductance coils were placed in coaxial pairs at selected locations within the reinforced soil mass to record horizontal soil movements. These devices have been used by other investigators to measure "strains" in soil (*13*). Experience with these devices showed them to be difficult to install and align, and soil "strains" could not be inferred from the response. Nevertheless, the devices did prove useful in confirming the location of soil volumes that were disturbed and the exact times at which events such as soil failure occurred.

HTR potentiometers were mounted inside the panels to record the compression of foam layers between incremental wall panels and to record vertical soil movements at the surface of the reinforced soil mass (Figure 5).

In selected tests a number of Glotzl cells (*13*), 200 mm by 300 mm in plan with electrical pressure transducer readouts, were placed above and below grid layer 3 (Figure 5). These cells were also calibrated in situ using the same approach as that described for the Geokon cells. The results of vertical earth pressures measured in the vicinity of grid layer 3 were consistent with qualitative features in Figure 16, 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.

Data Acquisition

Up to 300 electronic devices have been installed in the RMC test walls. Necessarily, the devices can only be effectively monitored using automatic data acquisition. The primary piece of equipment to meet this need was a 300-channel Hewlett Packard HP 3497A/3498A data acquisition system. All electrical devices gave an analog DC voltage signal or were connected to external electrical circuitry that could convert output signals to a DC voltage that could be read by the data acquisition system (e.g., Bison inductance coils).

The data acquisition system was controlled by a PC-DOS microcomputer with its own 20 minute power backup supply. The data acquisition system was programmed to record the response of all instruments at a selected time interval (typically 8 hours). However, several of the potentiometers on the panel facing units and air pressure supply transducers 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.

At least as important as data acquisition was data post-processing. A tremendous amount of data was routinely generated because some tests lasted as long as 7 months. Commercially available software was used to write post-processing packages for the accumulated data after conversion to LOTUS format. The raw data from the PC controller was analyzed for changes in signal output and the optimized data set converted to useful units and then plotted. The post-processing software allowed a full history of all test instrumentation to be available within minutes. This ability to digest a large amount of data rapidly is important when quick decisions have to be made concerning the magnitude and duration of surcharge loading to be applied as wall failure is approached.

CONCLUDING REMARKS

The development of the RMC Retaining Wall Test Facility and ancillary instrumentation has provided the author and coworkers with the capability to carry out carefully monitored tests of full-scale geosynthetic-reinforced walls. The comprehensive monitoring of model walls and the quality of the data has allowed the author to identify important mechanisms in the behavior of these complex systems during construction, under working load conditions, and at failure. The data are also proving useful in the development and calibration of analytical models, which are proceeding concurrently with the experimental program.

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