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An instrumented geogrid reinforced soil wall

Un mur de soutènement renforcé par géogrids

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SYNOPSIS: A large-scale geogrid reinforced soil wall was constructed and then surcharged until failure occurred. The wall was 3 m high and 2.4 m wide and retained a volume of soil in excess of 50 m^3. The structure was constructed with rigid articulated facing panels and a weak polymer grid reinforcement was used to ensure rupture of the grid layers under sustained uniform surcharging. Rupture of the reinforcement and collapse of the wall occurred at a surcharge pressure of 70 kPa. Qualitative features of the behaviour of this wall are presented and some important implications to the design and analysis of these structures are identified.

1 INTRODUCTION

The development of advanced polymeric materials has led to their use as extensible geosynthetic reinforcement in soil retaining wall structures and steep slopes. Between 1974 and 1987 it is estimated that about 200 polymerically reinforced soil walls and steep slopes have been constructed in North America. An excellent summary of documented projects in North America is given by Yako and Christopher (1987). However, as their summary reveals, there is relatively little data from instrumented structures that can be used to optimize current design methods. Most conventional methods of design are based on limiting equilibrium approaches where factors of safety are used to relate the stability of these composite structures at failure to working load conditions. Nevertheless, little (if any) data from closely monitored prototype scale walls is available to properly relate failure conditions to working conditions in these systems. The deficiencies in current methods of analysis and design were illustrated during an international prediction exercise held at the Royal Military College of Canada (RMC) in 1987. "Class A" predictions of the load-deformation behaviour of two trial geogrid reinforced soil walls were made by participants. The predictions varied widely but generally underestimated the stiffness of these composite structures (Jarrett and McGown 1987).

This paper is focused on a description of the RMC Retaining Wall Test Facility and the results of a large-scale test of a geogrid reinforced soil wall constructed with rigid incremental panel facings. The test is part of a long-term research project which is being carried out to further the understanding of the mechanics of reinforced soil wall behaviour and to provide experimental data that can be used to evaluate design methodologies. At the time of writing a total of 9 geosynthetic reinforced soil walls have been constructed in the RMC test facility. These tests have also included walls constructed with wrap-around facings and rigid propped facing panels (Bathurst et al. 1987, 1988a,b).

2 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 (Figure 1). The principal structural components of the facility are six rigid heavily reinforced concrete counterfort cantilever wall modules. These modules are arranged to laterally confine a block of soil up to 6.0 m long by 3.6 m high by about 2.4 m wide. Test walls are 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. The inside walls of the structure are faced with plywood/plexiglas/polyethylene sheeting

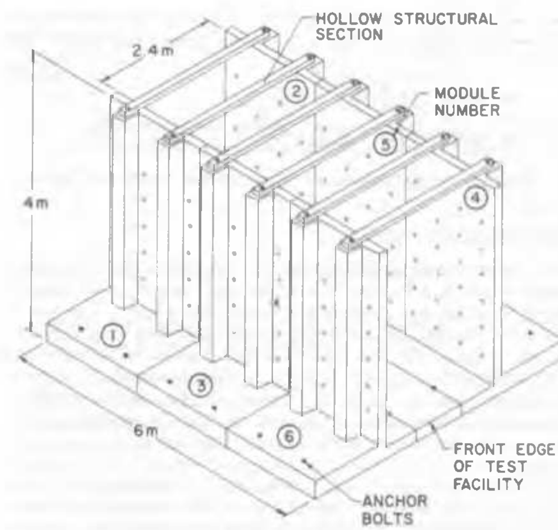

Figure 1 RMC Retaining Wall Test Facility

which acts to reduce sidewall friction. Shear box tests showed that the sand/sidewall interface had a fully-mobilized friction angle of 15°. Stability calculations show that the friction reducing sidewall construction is effective and reduces the contribution of the test facility boundaries to less than 15% of the total active earth force that would be resisted by the facings in a true plane-strain condition (Bathurst and Benjamin 1987).

3 TEST CONFIGURATION

The large-scale model reinforced wall was 3 m high and retained a sand backfill with a uniform surcharge. Figure 2 shows the general arrangement for the test which comprised 0.75 m high rigid articulated facing panels. In an earlier incremental wall test reported by Bathurst et al. (1987) a similar test geometry was used together with a Tensar SR2 geogrid. This test represented a typical working load condition rather than a failure condition. A preliminary test reported by Bathurst et al. (1988a) using a weaker and more extensible geogrid reinforcement failed under a surcharge of about 50 kPa. This earlier test was not heavily instrumented and the test reported here is essentially a repeat test that was carried out to recover a wide range of load-deformation data from the composite structure during construction and surcharging up to failure.

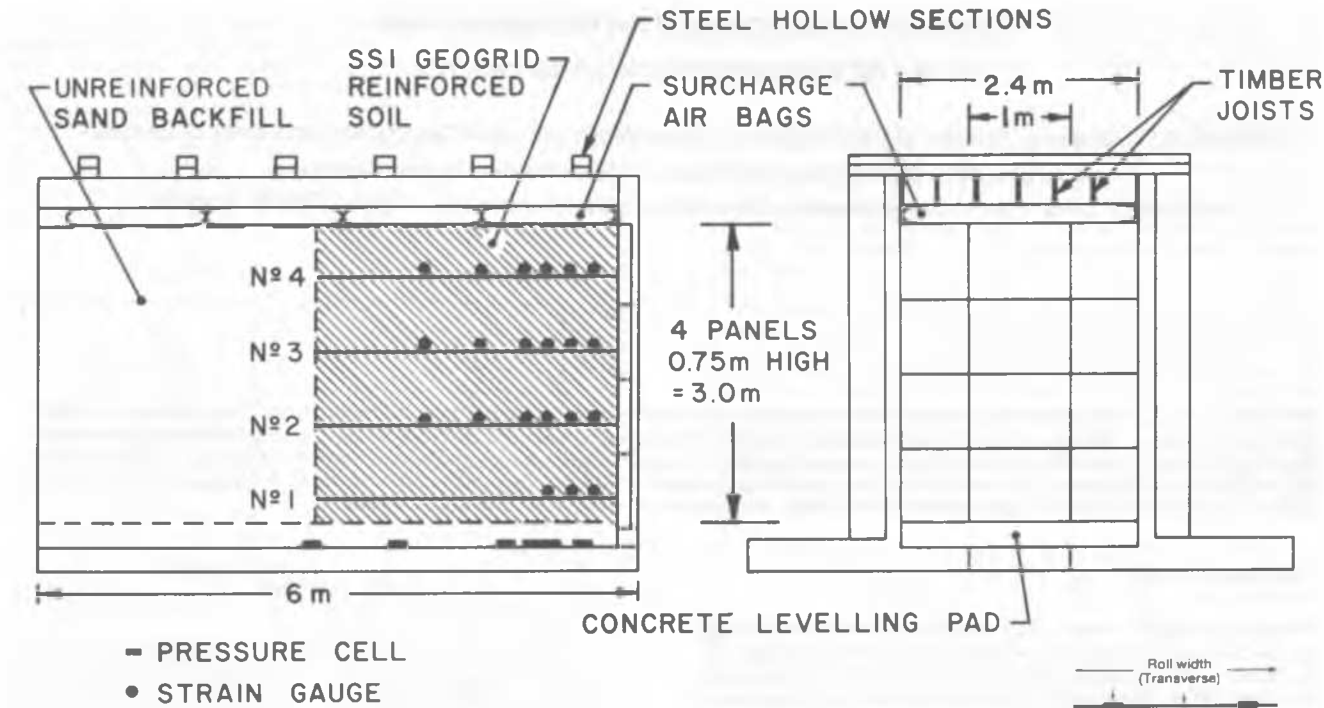

Figure 2 General arrangement for incremental panel wall test

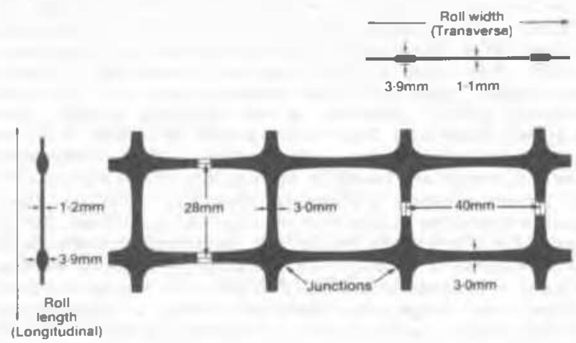

Figure 3 Typical dimensions for Tensar SS1 Geogrid (from Netlon Ltd. 1984)

3.1 Geogrid Reinforcement

The reinforcement used in the current investigation was a Tensar SS1 Geogrid which is a biaxially-oriented high density polypropylene geogrid with an approximate mass of 200 g/m^2. The reinforcement was attached to the facing panels with the longitudinal (or weak) direction of the biaxial grid in the plane-strain direction of the test facility. In practice, this particular geosynthetic is not used in retaining wall construction because it is relatively weak. It has been used in the most recent tests at RMC in order to amplify system deformations and to ensure that the trial walls can be surcharged to failure. The dimensions of Tensar SS1 Geogrid are shown in Figure 3 and Index mechanical properties are given in Table I. The reinforcement elevations in the current test were based on a standard 750 mm spacing that has been used for all tests carried out so far in the RMC test facility.

3.2 Granular Fill

The backfill material comprised a uniformly graded washed sand with some gravel. The average bulk density for the sand backfill is 1.80 Mg/m^3 with a moisture content of 1 to 3 % following placement and compaction. Results of large-scale direct shear box tests gave a peak (secant) friction angle that varied from $\phi = 53°$ at a normal confining stress of 12 kPa to $\phi = 40°$ at a normal confining stress of about 120 kPa. The high strength of this granular material at low confining stresses is considered to be a consequence of its high angularity.

Table I
Mechanical Properties of Tensar SS1 Geogrid Reinforcement*

SS1 Orientation	Stiffness (kN/m) (@ 2% strain)	Peak Load (kN/m)	Strain @ Peak Load (%)
transverse (strong)	330	20	13
longitudinal (weak)	170	14	17

(* after Yeo (1985), from *Index* In-isolation Tensile tests at 2% strain/minute and 20°C)

4 CONSTRUCTION AND SURCHARGING

The reinforced wall was constructed with a central instrumented section nominally 1 m wide and two 0.7 m wide edge sections. This construction was adopted to reduce edge-effects on the performance of the monitored central reinforcement strip and facing units. A foam rubber void filler was placed along all vertical panel edges in order to prevent the panels from binding during outward movements. In addition, a layer of foam rubber was placed at each horizontal panel interface to assist in panel levelling during construction of the wall and to minimize additional reinforcement loads near the facings due to soil settlement. The bottom row of panels was restrained in the vertical and horizontal directions by an array of load cells used to measure toe loads but were free to rotate about their bottom edge. Each row of panels was temporarily supported until the sand backfill had been placed and compacted to the top of each row. The purpose of this form of construction was to progressively mobilize the inherent tensile capacity of the geogrid reinforcement as the height of the composite structure was increased. Following construction, the surface of the sand backfill was uniformly surcharged. The surcharge was applied in the following increments: 12, 20, 30, 40, 50, 60 and 70 kPa. Each surcharge increment was sustained for at least 100 hrs in order to observe creep in the composite structure.

5 INSTRUMENTATION

The principal physical measurements taken over the course of the test were:

1) Horizontal and vertical movements of the facing panels.
2) Horizontal reinforcement displacements and strains.
3) Vertical earth pressures at the base of the sand backfill.
4) Vertical earth pressures immediately above and below selected grid layers.
5) Loads at the panel/geogrid connections.
6) Horizontal and vertical toe loads.

Details of the instrumentation employed and interpretation of grid strains are similar to that reported in earlier tests (Bathurst et al. 1987, 1988a,b).

6 SELECTED TEST RESULTS

Figure 4 shows horizontal displacements recorded along the length of grid layer 3 and at the front of the corresponding panel. Extensometer locations shown in the figure were at 0.16, 0.33, 0.60, 1.0, 2.1 and 3.0 m from the facing. Deformations have been plotted with respect to a fixed datum taken at the time of grid 3 installation. The figure shows that rapid changes in displacements corresponded to increased surcharge loading. As the magnitude of loading increased, time-dependent deformation in the grid (i.e. creep) became more pronounced. Relatively large deformation rates were recorded as soon as the 70 kPa surcharge was applied leading to failure of the reinforced soil mass about 93 hrs later. The results from internal instrumentation in this test and direct observation in a previous similar test lead us to believe that a well-defined shear plane through the reinforced soil mass had developed at this stage. The composite system remained essentially intact for a further 300 hrs at which point the grid in layer 2 ruptured followed by immediate tearing of grid layers 3 and 4 and collapse of the composite system. The data shows that in layer 3, the largest movements were recorded by extensometers 1, 2 and 3 indicating that significant grid-to-soil load transfer is restricted to the first 1 m of grid behind the facing. Similar qualitative effects were recorded in all four layers although the magnitude of displacements was relatively small in layer 1 due to the restrained toe of the wall. The maximum out-of-vertical alignment at the top of the wall was about 90 mm with respect to the toe of the wall at collapse.

Strain data from grid layers has been summarized in Figure 5. The distributions show that grid strains are about 1% or less at the end of construction and less than 2% at about half the surcharge capacity of the wall. These strain levels are well within the performance limit of the material (e.g. McGown et al. 1984). Just prior to soil failure the peak strains in layers 2,3 and 4 were about 6%. At grid rupture the peak strains were between 8 and 9%. At test completion, the wall was carefully excavated and a well-defined failure plane was observed that closely matched the locations of peak strain in layers 2 and 3 and the elevated strains at the 1 m location in the topmost layer. At all stages in the test program strains in the vicinity of the connections were significant despite the compressible void filler that was placed between panel edges.

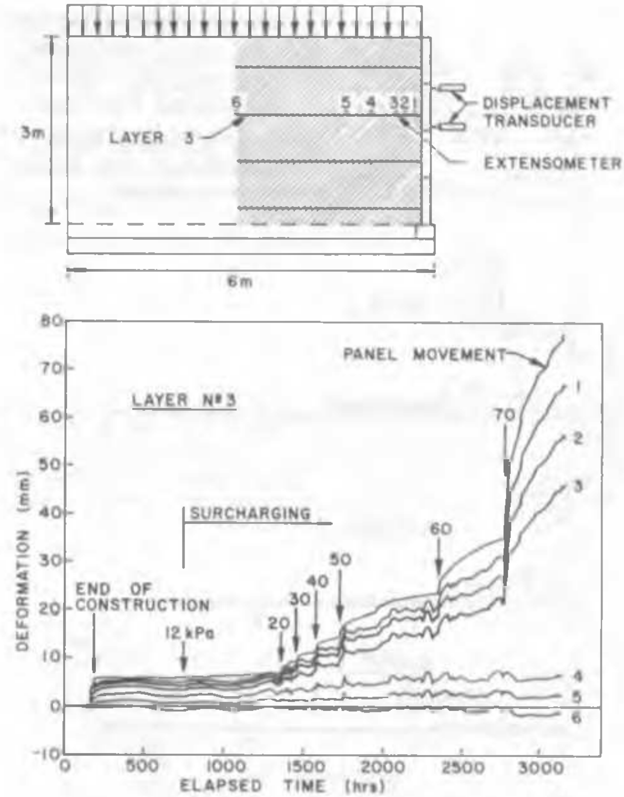

Figure 4 Layer 3 panel and grid displacements

7 CONCLUSIONS

Based on the results of the current investigation several general conclusions can be made related to qualitative features of the performance of a heavily-surcharged geogrid reinforced soil wall with an incremental panel construction:

1) Two distinct failure mechanisms were identified during the 70 kPa surcharging increment of the trial wall. The first failure mechanism was a soil-to-soil failure through the reinforced soil mass. This was followed by grid rupture about 300 hrs later. Both failures were preceded by significant levels of strain and increased strain-rate (creep) in the reinforcing layers.

2) Despite the use of a compressible void filler between panels, connection loads were significant at all stages during the test. Measured connection loads were comparable to those interpreted from grid strains at peak locations.

3) Localized peak strain locations along the length of grids 2,3 and 4 were coincident with a Rankine failure plane drawn from about the location of the grid/panel connection at layer 1 and using $\phi = 53°$ for the soil.

4) Grid lengths of 3 m appeared to be more than adequate for anchorage and stability of the composite structure at working load levels (say 40 kPa). In addition, the pullout capacity of the reinforcement layers was adequate since measureable strains were not observed to propagate more than 1.5 to 2 m from the facings even at failure. The extent of strain propagation into the soil mass was greatest in layer 4 which was the least confined layer in the composite structure.

5) The magnitude of grid strains in layers 2,3 and 4 at the location of the Rankine failure plane was essentially constant at both soil and grid failure. Grid layer 1 did not record significant strains due to proximity to the bottom of the test facility.

6) The results of toe forces showed that they were significant at all stages in the test. At collapse of the structure they were about 4 and 25 kN/m in the horizontal and vertical directions respectively.

8 CONCLUDING REMARKS

The results of this investigation and earlier tests emphasize the important influence that the reinforcement/facing connections have on the distribution of strains within the reinforcing layers. Despite vertical compliance of the facing units using compressible foam between panels and careful fill placement behind the facings the connection loads were significant. The membrane action that results when backfill soil is restrained vertically may generate the critical reinforcement loadings in actual walls. This effect is even more pronounced in propped wall construction where there

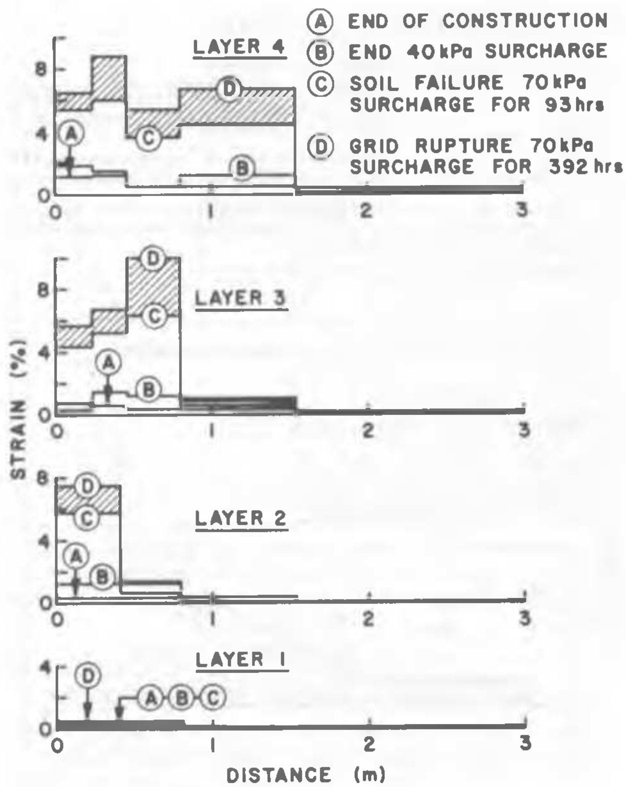

Figure 5 Summary of grid strains

is essentially no vertical degree of freedom in fixed connections (e.g. Bathurst et al. 1987, 1988a). A preliminary recommendation based on the results of RMC tests is that connection loads at wall failure are at least equivalent to peak loads occurring in the reinforcement at the location of the potential wedge through the reinforced soil mass.

The distributions of strain in the reinforcement at a surcharge load equivalent to about 1/2 of the ultimate capacity of the composite structure do not generally reflect the trends observed at failure. This observation highlights the difficulty in relating mechanisms at failure to wall behaviour at working load conditions. It may be concluded that a simple factor of safety approach to relate failure conditions to working load conditions is not appropriate for this type of structure.

The distribution of strains along the reinforcements showed that grid-to-soil load transfer occurred over a short grid length estimated to be less than 0.5 m beyond the peak strain locations associated with the internal soil failure plane.

The distribution of vertical earth pressures below the reinforced soil mass was essentially uniform except for the first 0.5 m behind the panel facing. At this location there was about a 30% reduction in earth pressure based on soil self-weight and uniform surcharge pressure. This loss appears to be equivalent to the vertical force transmitted through the toe of the facings. An implication to the analysis of these structures is that the assumption of uniform pressure along the base of the reinforced soil mass is appropriate if toe forces are considered.

Stability calculations were carried out at soil and grid failure using grid forces based on isochronous load-strain-time data for the reinforcement material. The calculations showed that if all boundary forces are considered (including toe forces and sidewall friction) then, horizontal and vertical equilibrium of the observed failure wedge through the reinforced soil mass was satisfied using a simple tie-back (Coulomb) wedge approach.

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REFERENCES

Bathurst, R.J., Wawrychuk, W. and Jarrett, P.M. (1987). Laboratory Investigation of Two Large-scale Geogrid Reinforced Soil Walls. The Application of Polymeric Reinforcement in Soil Retaining Structures. NATO Advanced Study Institutes Series, Kluwer Academic Publishers, 71-125. P.M. Jarrett and A. McGown (eds)

Bathurst, R.J. and Benjamin, D.J. (1987) Preliminary Assessment of Sidewall Friction on Large-Scale Models in the RMC Test Facility. The Application of Polymeric Reinforcement in Soil Retaining Structures. NATO Advanced Study Institutes Series, Kluwer Academic Publishers, 181-192. P.M. Jarrett and A. McGown (eds)

Bathurst, R.J., Benjamin, D.J. and Jarrett P.M. (1988a). Laboratory Study of Geogrid Reinforced Soil Walls. Geosynthetics for Soil Improvement. ASCE Special Publication 18: 178-192. R.D. Holtz (ed)

Bathurst, R.J., Jarrett, P.M. and Lescoutre, S.R. (1988b) An Instrumented Wrap-around Geogrid Reinforced Soil Wall. Proc. 3'rd Canadian Symposium on Geosynthetics, Waterloo, Ontario, October 4'th

Jarrett, P.M. and McGown, A. (eds) (1987). The Application of Polymeric Reinforcement in Soil Retaining Structures. NATO Advanced Study Institutes Series, Kluwer Academic Publishers

McGown, A., Andrawes, K., Yeo, K. and Dubois, D. (1984). The Load-Strain-Time Behaviour of Tensar Geogrids. Symposium on Polymer Grid Reinforcement in Civil Engineering, Paper No. 1.2, London

Netlon Ltd. (1984). Test Methods and Physical Properties of Tensar Geogrids

Yako, M.A. and Christopher, B.R. (1987). Polymerically Reinforced Retaining Walls and Slopes in North America. The Application of Polymeric Reinforcement in Soil Retaining Structures. NATO Advanced Study Institute Series, Kluwer Academic Publishers, 239-283, P.M. Jarrett and A. McGown (eds)

Yeo, K.C. (1985). The Behaviour of Polymeric Grids used for Soil Reinforcement. Ph.D Thesis. Department of Civil Engineering. University of Strathclyde. Glasgow, Scotland