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LOAD TESTS ON GEOGRID REINFORCED GRAVEL FILLS CONSTRUCTED ON PEAT SUBGRADES

ESSAIS DE CHARGEMENT DE REMBLAIS SUR TOURBE, RENFORCES AVEC UN «GEOGRID»

BELASTUNGSVERSUCHE VON MIT GEOGRIDS VERSTÄRKTEN KIESSCHÜTTUNGEN AUF TORF

Large scale, plane strain loading tests were conducted on gravel fills compacted on a 900 mm peat deposit. Fills with thicknesses varying from 150 mm to 450 mm were tested both with and without geogrid reinforcement. For the test, a 203 mm wide beam which spanned the full 2.4 m width of the test pit was pushed into the gravel under a series of incrementally increasing loads.

Measurements of the beam loads and beam displacements were taken together with tensile loads and vertical and horizontal movements of the geogrid reinforcement. From this information the action of the reinforcement as a "tensioned membrane" can be analysed. The significant reinforcement effect of the geogrid is attributable to its action as a tensioned membrane.

An einer auf 900 mm dickem Torfplanum verdichteten Schotterbettung wurde mit einem in großen Maßstab angelegten Versuchsmodell die ebene Spannung untersucht. Bettungen von 150 mm bis 450 mm Dicke wurden mit und ohne Verstärkung durch "Geogrid" getestet. Als Versuchsmodell wurde ein 203 mm breiter, die gesamte 2,4 m breite Versuchsgrube überspannender Träger bei inkremental zunehmender Belastung in den Schotter gestoßen.

Gemessen wurden Trägerbelastung und -verschiebung, Zugbeanspruchung sowie die senkrechten und waagerechten Bewegungen der "Geogrid"-Verstärkung. Mit den Meßergebnissen kann die Wirkung der Verstärkung als "zugbeanspruchte Membrane" errechnet werden. Der signifikante Verstärkungseffekt von "Geogrid" ist auf seine Wirkung als zugbeanspruchte Membrane zurückzuführen.

INTRODUCTION

The use of geotextiles and geogrids in the construction of access roads and earth embankments on soft, highly compressible organic soils is now widespread. The experimental program described in this paper has been carried out to develop a better understanding of the mechanisms by which the geosynthetic reinforcements help in the construction of access roads over peats. Specifically, the problem under study is the situation where the vehicle loads represent the major disturbing forces and the weight of the fill itself is relatively insignificant.

TESTING ARRANGEMENT

Large scale, plane strain loading tests have been made in the laboratory on a series of thin gravel fills compacted over a peat subgrade. The tests were carried out in a test pit 3.7 m by 2.4 m in plan and 2 m deep. The basic testing arrangement is shown schematically in Figure 1. A brief description of the testing arrangement follows but a more detailled description was provided by Jarrett (1).

A reproducible, finely fibrous, Sphagnum peat subgrade was consolidated to an average moisture content of 850%, a depth of approximately 0.9 m and with an average vane shear resistance of 4 kPa. Gravel fills were compacted on this subgrade using a well graded, 20 mm, crushed limestone aggregate. This is an excellent aggregate and would be defined in North America as a Granular A material that might be used as a Base Course in permanent road construction. Fills were compacted with thicknesses of 150 mm, 300 mm and 450 mm both with and without reinforcement. Where reinforcement was used it was placed at the peat to gravel interface and for these tests a Tensar Geogrid, Type SS2, was used.

For the tests, a beam spanning the full 2.4 m width of the pit and itself 0.203 m wide was pushed into the gravel fill under a series of incrementally increasing loads. The loads were applied with a computer controlled hydraulic actuator that could apply either static or cyclic loads. During testing beam loads and beam displacements were monitored. In addition vertical and horizontal movements were observed at a number of locations on the reinforcement. In two tests, 3 load cells were inserted in a 0.305 m wide strip of the geogrid. This strip was placed along the longitudinal centerline of the pit and allowed the tensile force mobilized in the plane of the reinforcement to be measured at positions, directly beneath the beam centerline and at lateral distances of 0.4 m and 0.8 m from the beam centerline.

TESTING PROCEDURES

Most tests followed a rather complex loading procedure that involved loading the beam incrementally until at least 0.2 m of beam displacement occurred, then cycling the maximum load on and off for 5 cycles. After the 5 cycles, the beam was removed and the surficial rut that had been formed was filled. The beam was then replaced and loading was taken to higher levels. Again this procedure is more fully described by Jarrett (1). However for this paper only the results during the first incremental loading phase of the tests will be addressed. In this initial incremental loading phase the beam loads remained constant until the rate of beam displacement became less than 0.02 mm/min at which time the beam load was increased. This phase of the test was terminated when the accumulated beam displacement exceeded 0.2 m.

The incremental load tests represent static loading conditions. To assess the effect of more dynamic, repetitive loading, tests were also run using sinusoidal loading pulses at a frequency of 0.5 Hz. In these tests the loads were again raised incrementally but at each intensity of load at least 10,000 cycles of that load were applied before the load was increased to the next level.

TEST RESULTS - STATIC LOADS

The basic results for the static incremental load tests are presented as a set in Figure 2. For the three thicknesses of fill tested the beam load is plotted against the beam displacement for the reinforced and unreinforced tests. Also drawn on each plot are the results of a test in which the beam was placed directly on the peat surface and then pushed into the peat under a series of incremental loads. A correction had to be applied to the beam displacements of the 450 mm reinforced test. The correction was necessary as the results of the vertical settlement measurements made at the peat to gravel interface indicated that the peat layer was still consolidating slightly during the beam loading program. In the unreinforced 450 mm test a longer period was left between compacting the fill and testing so that consolidation was more complete in this test prior to the beam loading.

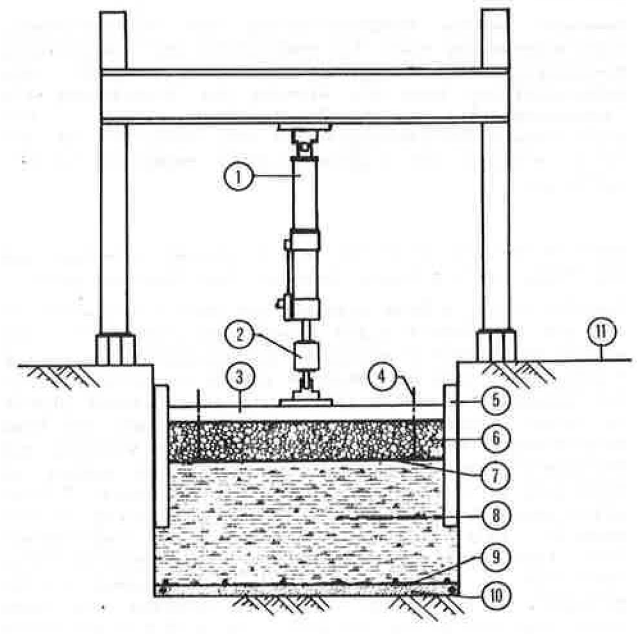

FIGURE 1 Test Apparatus

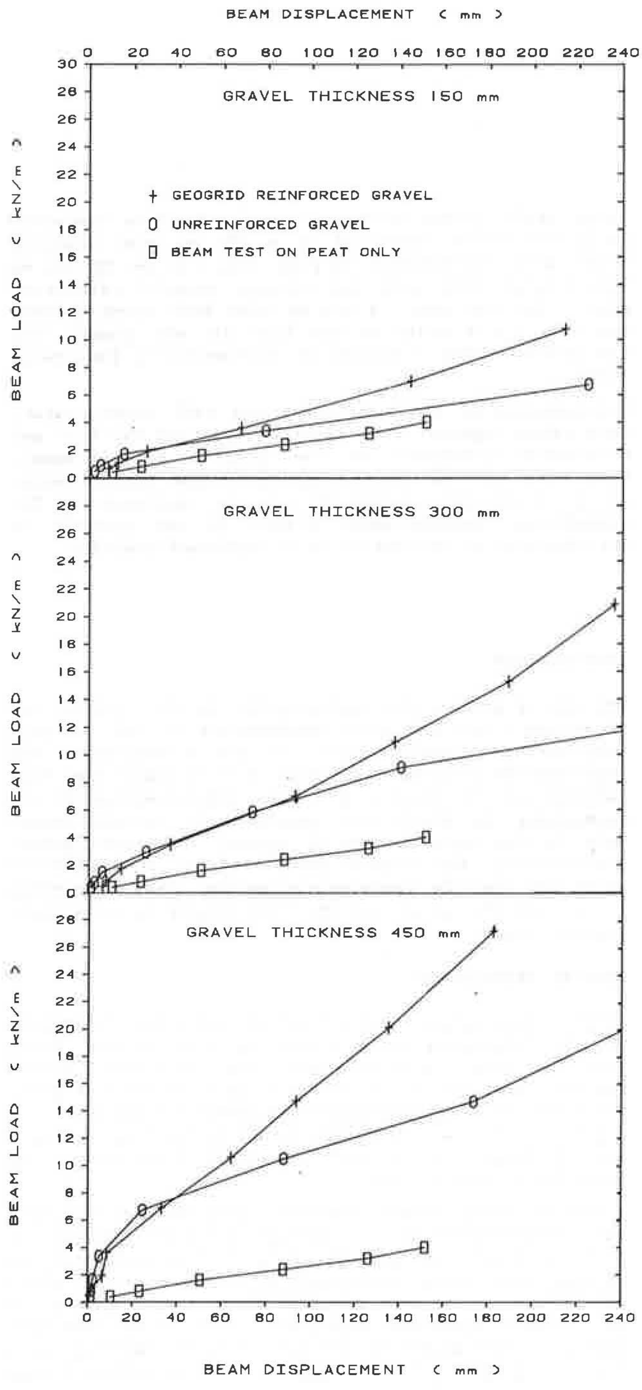

FIGURE 2 Basic Results for Static Loads

From these plots, one can observe for each thickness of gravel the increase in bearing capacity produced by the unreinforced gravel when compared at the same displacement to the peat only test and then the further increase in bearing capacity produced by the reinforcement. It will be noted that there is little difference between the reinforced and unreinforced gravel at small beam displacements. However, if the increments during which the beam displacement exceeded 200 mm are considered then in the 150 mm and 300 mm unreinforced tests a state close to failure was observed. In fact, in the 300 mm test, the rate of beam displacement did not drop to the limiting value during that increment and the load had to be removed to prevent a punching failure. Comparatively in the reinforced tests a strain hardening state is indicated as the beam displacement increases. It is at this stage of the test that the reinforcement is being most effectively mobilized as a tensioned membrane.

To better visualize the results at this stage of the tests it is advantageous to compare the beam loads required to produce some fixed beam displacement. As an example the beam loads required to produce 200 mm of displacement for the six basic tests and the peat only test are plotted in Figure 3. Small extrapolations from the results in Figure 2 were necessary for the peat only test and the 450 mm reinforced test. Examining the unreinforced results it is seen that the improvement in bearing capacity is non-linear with gravel thickness. This is believed to be due to the achievement of better compaction as the gravel thickness increases. In fact the 150 mm unreinforced gravel was very difficult to compact at all on the very compressible peat. In a similar vein the additional bearing capacity attributable to the reinforcement is also greater as the gravel thickness increases. For the thicknesses of gravel considered in the tests, the primary influence producing this effect is the greater effectiveness of anchoring the reinforcement as the gravel thickness increases. For the reinforcement to maintain a tension in the crucial load support areas beneath the beam, it must be anchored in some laterally removed zone. Anchorage for geogrids is brought about by interlocking and friction and thus as the gravel thickness increases, the anchorage capacity increases. This allows more tension to develop in the reinforcement and hence produce greater support. Jarrett and Bathurst (2) studied the friction at gravel-geogrid-peat interfaces using a shear box and various forms of pullout tests. For conditions modelling the anchorage zones in the beam tests it was found that as tension was applied to the geogrid, sliding occurred only between the geogrid and the peat. The gravel was firmly interlocked with the grid and moved with it. The angle of shearing resistance along this lower interface was found to be 34°.

One measure of the relative effectiveness of the anchorages is shown on Figure 4. In Figure 4, the vertical and horizontal movements of the reinforcement are plotted for each reinforced test for the load increment when the beam displacement was closest to 200 mm. The horizontal movements were observed directly by providing a small lined hole in the gravel through which a node on the geogrid could be viewed using a cathetometer (travelling microscope). The vertical movements were observed by taking level readings on rods connected to small plates placed on the geogrid. Looking at the horizontal movements measured at the point furthest from the beam centreline it is apparent that 150 mm of gravel provided poor anchorage as an inward slippage of 29.5 mm occurred, whereas the slippage reduced to 11.5 mm under 300 mm of fill and 2.8 mm under the 450 mm fill. If one considers the geometric similarity of the deformed geogrids in all three cases, it becomes apparent that more strain has to occur in the reinforcing in the case where less slippage occurs. The higher strain leads to higher tension and thus presumably greater support of the beam load. It is also interesting to note that the inward slippage was measured at distances of approximately 1.6 m from the load centreline. From the practical point of view in access road construction, this means that the outer wheel path of the road must be kept at least that distance from the shoulder if good anchorage is to be ensured unless greater thicknesses of gravel are employed to reduce the necessary length of anchorage zone.

The vertical movements shown in Figure 4 indicate the deformed profile of the geogrid under load. With the provision of lateral anchorages the reinforcement acts as a tensioned membrane. From the profile it is possible to gauge the geometry of the central "concave up" section of membrane. This section provides vertical support beneath the beam. The magnitude of the vertical support is a function of the tension in the membrane and its geometry. In addition the "concave down" section that follows after the inflection point represents the lateral zone into which the vertical support forces are spread to the subgrade soil by the tension in the membrane.

Having defined the geometry of the tensioned membrane, the values of tension in the plane of the membrane are therefore of importance. Tensile load cells were used in the reinforced tests with 150 mm and 300 mm of gravel. In Figure 5, the values of tensile load carried by the reinforcement and measured at the end of each beam load increment are plotted against that beam load. The tensile load was measured over a width of 0.305 m but this has been presented as a load per metre. The load cells functioned well and reasonable confidence is felt in the values obtained. As an initial comment on these results, it is observed that the maximum tension occurs directly beneath the load and decreases significantly as one moves laterally away from the centerline. Intuitively this is expected but it is not in accordance with the assumed tensile force

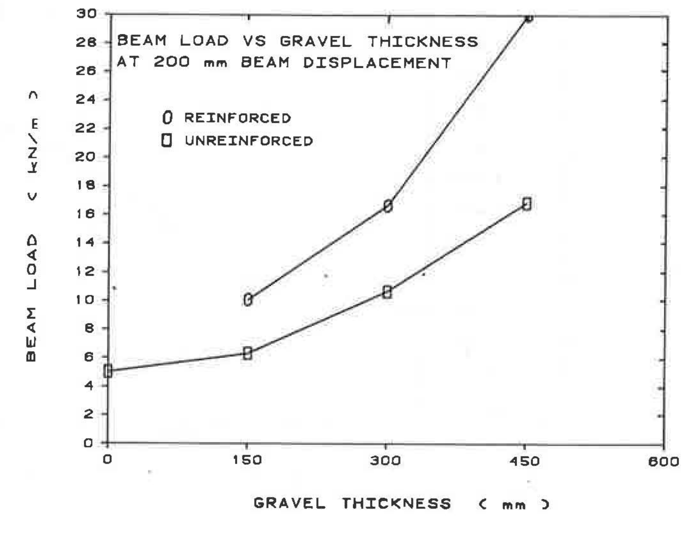

FIGURE 3 Beam Loads causing 200 mm Displacements

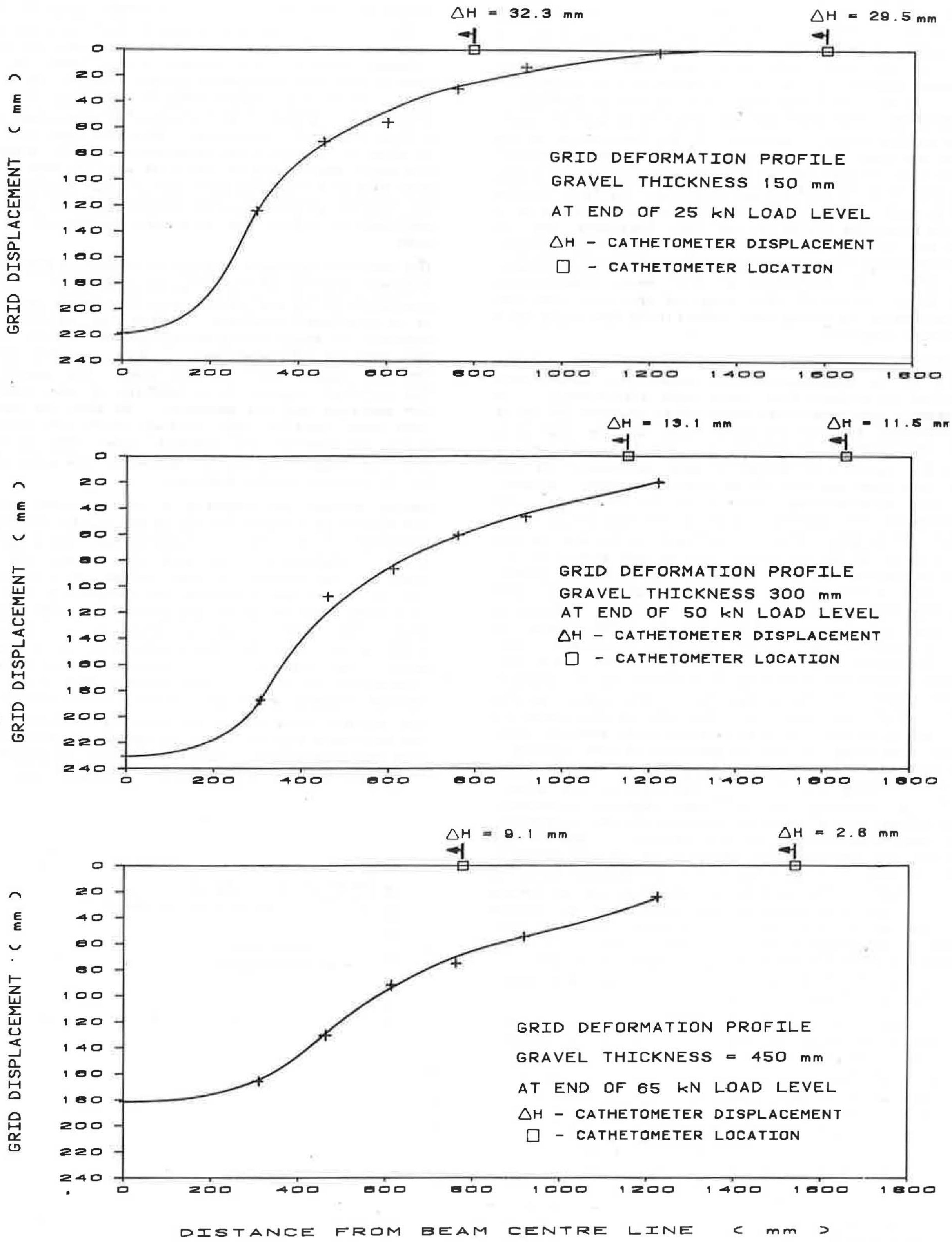

FIGURE 4 Reinforcement Geometry after Approximately 200 mm of Central Displacement

distribution used in a number of the simple tensioned membrane analyses that have been proposed as a means of calculating the magnitude of the vertical support. In addition, although large displacements had occurred at the maximum beam loads applied, it appears as though the tensile forces in the grid had not yet reached their maxima and thus further reinforcing action could be anticipated. If the anchorage zones are considered to start at 800 mm from the beam centerline and extend outwards from there then it is estimated from the pull out tests previously mentioned (2) that the ultimate tensile anchorage capacity under the 300 mm fill would be almost twice the 2.8 kN/m maximum value of tension measured in the load cell 800 mm from the centerline. To mobilize this extra tensile resistance however, lateral displacements of more than 50 mm would be required at the 800 mm location. This would obviously require a considerably greater and unacceptably large beam displacement unless for instance the rut were filled prior to any further loading.

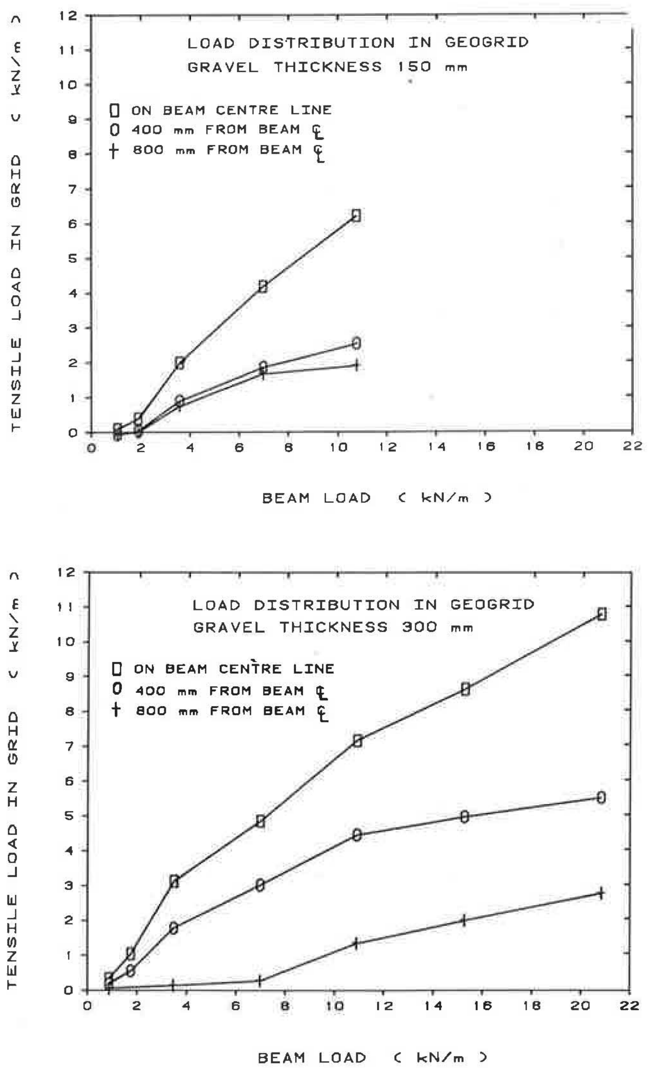

FIGURE 5 Tensile Loads Measured in Reinforcement

TEST RESULTS - CYCLIC LOADS

The basic purpose of this research program is to assist in the construction of access roads over peat subgrades. The plane strain static loading tests are used to give a basic comprehension of behaviour as they allow ease of analysis and ease of comparison between tests. Ultimately however the results must be related to the practical system in which repeated applications of dynamic loads are applied. As a step in making this correlation cyclic beam loading tests were made and a set of results is presented in Figure 6. On this figure the static beam load against beam displacement values for the 300 mm reinforced fill are repeated. Also shown are the beam displacements measured after 100, 1000 and 10,000 cycles of load at each load level on a similar 300 mm reinforced fill. At the 14.5 kN/m load level 116,000 cycles of load were applied and the displacement resulting is also indicated. In general it is noted that 10,000 applications of loads at 0.5 Hz frequency do not produce as large a displacement as the static loads. From the 14.5 kN/m load increment it appears as though approximately 30,000 load applications produces equality between the static and cyclic cases. From a practical point of view most access roads are designed for far fewer load applications than this. Two possibilities arise from this. The static load condition may be assessed as a worst cast scenario and used for design or the family of curves indicated for 100, 1000, 10,000 load cycles may be used to develop a trafficing correction to be applied to designs based on the static load analysis. This latter approach is similar to that of the US Corps of Engineers utilized in (3).

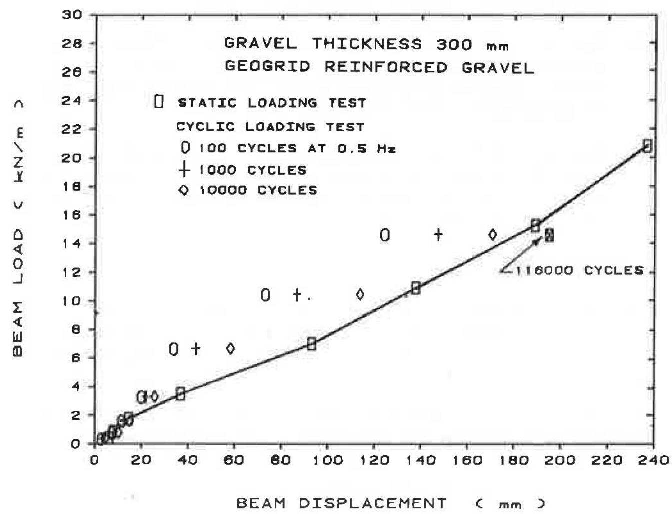

FIGURE 6 Cyclic Load Results

DISCUSSION

The basic results presented have defined the geometry and movements of a geogrid reinforcement under load, the tensile forces measured in the reinforcement, the stress-displacement properties of the subgrade peat, the interfacial friction coefficient between the reinforcement and the peat, and the benefits produced by the presence of the reinforcement. The complete set of results represent a means of verifying the accuracy of analytical methods that may be used to calculate the magnitude of tensioned membrane or other support.

At present a number of approximate methods for calculating membrane support have been proposed. These methods all follow a systematic approach suggested by Giroud and Noiray (3). In these methods assumptions are made either directly or implicitly of the tensile distribution in the reinforcement, of the strain in the reinforcement calculated by assuming fixity or perfect anchorage at some point and of the shape of the reinforcement under load (parabolic or circular in the central section). The test results presented indicate that there are shortcomings in the assumptions used in these methods. For instance the lateral slippage measured is far greater than the analytical methods allow for. The tensile distribution measured is far different from those assumed which in general are either that the tension is constant along the central section of the membrane or even that it is a minimum beneath the load.

Whilst indicating that the assumptions used are not accurate, it should also be indicated that the analyses do still provide reasonable estimates for the membrane support values. The ultimate analysis however should not just consider the central load bearing section of the membrane but should analyse the membrane in its entirety to balance out the support forces beneath the load against those spread laterally to the subgrade and to balance the membrane tensions against available friction and anchorage and lateral soil support.

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