Geosynthetic Reinforced Soil Structures in New Zealand

A. K. Murashev
M.E. (Hons), Ph.D.
Senior Geotechnical Engineer, Beca Carter Hollings & Ferner Ltd, New Zealand

Summary Because of their cost-effectiveness, the application of geosynthetic-reinforced soils (GRS) to permanent important structures carrying roads (such as highway embankments, bridge abutments and retaining walls) in New Zealand is rapidly increasing. Therefore, in 1997 Transfund New Zealand commissioned a research project to review the current state of the design practice of GRS in New Zealand and overseas. As part of this research project Beca Carter Hollings & Ferner Ltd has collected information on recent New Zealand GRS case studies and design methods currently used. A summary of these case studies is given. New Zealand geotechnical engineers currently use several different overseas standards and design guidelines to design GRS structures. This results in the fact that GRS structures are designed to different standards, and therefore have different levels of static and seismic resistance. It is also shown that there is disagreement between different researcher's points of view and different standards on how to take into account seismic loads in GRS design. Several methods currently used for static and seismic design of GRS structures in New Zealand are outlined.

1. INTRODUCTION

Geosynthetic reinforced soils (GRS) have been found to have cost benefits relative to traditionally used retaining structures in specific situations. As a consequence the application of GRS to structures carrying roads is rapidly increasing. Applications of GRS structures on highways in New Zealand include bridge abutments, reinforced embankments supporting highways, retaining walls and repair of slope failures. While the family of geosynthetics includes a wide range of products, geogrids are the most common product used for soil reinforcement purposes in New Zealand.

New Zealand geotechnical engineers currently use several different overseas standards and design guidelines to design GRS structures. Although static behaviour of GRS structures is comparatively well understood and design practice is well established, there are still variations between sizes of GRS block and amounts of reinforcement required by different design methods. There is also a lack of clear guidelines for the seismic design of GRS structures. Moreover, there is disagreement between different researchers' points of view and different standards on how to take into account seismic loads in GRS design. The amount of laboratory and field testing specified and types of soil strength parameters used by different standards and design guidelines also differ. All this results in the fact that GRS structures in New Zealand are designed to different standards and, therefore, have different levels of static and seismic resistance, and different risks of failure under seismic loads.

Given the fact that the use of GRS structures in New Zealand is increasing, Transfund New Zealand commissioned a research project to review the current state of design practice of GRS in New Zealand and behaviour of such structures under static and seismic conditions.

2. STATIC BEHAVIOUR OF GRS STRUCTURES

Over 200 of published GRS case histories have been reviewed as part of this research. While the analyses of every case study is beyond the scope of this article, the information collected on the case histories is given by Murashev, (1998).

Beca Carter Hollings & Ferner Ltd has collected information on 61 recent GRS case studies in New Zealand. Part of this information is summarised in Table 1. The typical geometry of the GRS structures is shown on Figure 1. The information was obtained through a review of published case histories and interviews with the New Zealand traffic controlling authorities, representatives of manufacturers and design engineers. General comments were received that performance had been satisfactory and no advice was received on any significant problems. Although designed by different methods, most of the GRS structures listed in Table 1 performed satisfactorily. However, there has been some outward movement in structure 5 due to lack of drainage.

Table 1. Summary of New Zealand Case Studies

Nº	Year	Location	Function	Wall Length (m)	Geometry						Reinforcement make and grade	Design Method Used
					H, m	Z, m	β, Deg	W, Deg	l, m	S, m		
1	1993	SH45 Herekawe	Road embankment	40	8.4		75	0	5.5	0.6	Tensar SR55	Win Slope
2	1992	Wanganui/SH 52 Weber Slip Reinforcement	Slope Stabilisation and road widening		7	0	60	0	5.5	0.4, 0.6, 0.8, 1.0	Tensar SR55, Tensar SS2	Win Slope, Win Wall/Bautechnik Method
3	1993	Wanganui/Flood damage Ore Dropout SH4	Road Reinstatement	35	6 to 7	0	75	0	4.6	0.6, 0.7 0.9, 1.0	Tensar SR55, Tensar SS1	Win Slope, Win Wall/Bautechnik Method
4	1993	Wanganui/SH4 Flood damage Burrells Road	Reinforced Road embankment	50	10	0	60	0	9, 10	0.4, 0.5 0.7, 1.0	Tensar SR55, Tensar SS1	Win Slope, Win Wall/Bautechnik Method
5	1994	Wanganui/Manawapou Reconstruction/SH3	Road realignment	160	9.3	0	75	0	6	0.4, 0.8	Tensar SR55, Tensar SR80, SS1	Win Slope, Win Wall/Bautechnik Method
6	1994	Hot Water Beach Rd	Support of the road	70	8	0	75	0	2.8, 3.8, 4.8	0.3, 0.6	Tensar SR80, Tensar SR55, SS20	Win Slope, Win Wall/Bautechnik Method
7	1994	Huntly/Renown Mine Haul Road overbridge Abutment	Bridge support		5.9	0	90	0	4.5	0.4, 0.6	Tensar SR80, Tensar SR110	Tensar wall and slope design program/Bautechnik Method
8	1998	SH2 562/3.3	Slip Repair	46	11	0	70		7	0.75	Tensar RE	Win Slope, Win Wall
9	1993	Hawkes Bay Tararua	Slip Repair	140	4 to 8	5	60	10		0.4 to 0.8	Fortrac 35	HA68/94
10	1955	Auckland Waokauri Bridge	Bridge Abutment	60	3 to 7	2	75	15	4 to 8	0.5	Fortrac 35, 55	Geosolve
11	1997	Auckland	Road Embankment	60	5	0	70	0	6	0.3 to 0.6	Fortrac 35	Leshchinsky Method, ReSlope
12	1997	Northland	Slip Repair	56	7	0.6	65	15	3.1	0.5 to 0.6	Fortrac 35	Leshchinsky Method, ReSlope
13	1998	Northland	Bridge Abutment	25	3.7	0	60	0	6.5	0.3 to 0.5	Fortrac 35, 80	Leshchinsky Method ReSlope

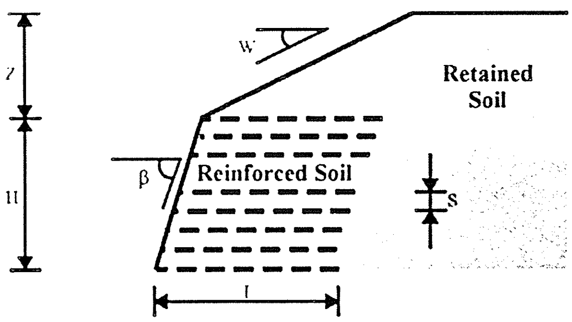

Figure 1. Typical geometry of a GRS structure.

In spite of the fact that information about failures of GRS is not always published, a review of published case histories undertaken by Beca Carter Hollings & Ferner Ltd indicates that there have been cases of unsatisfactory performance of GRS structures under static loads. However, most of the reviewed case histories prove that generally GRS structures perform well under static conditions.

3. SEISMIC BEHAVIOUR OF GRS STRUCTURES

To the authors knowledge none of the GRS structures constructed in New Zealand has experienced a strong earthquake, therefore, information on the seismic behaviours of GRS structures was obtained from overseas.

The 1995 Kobe earthquake (Japan) had its epicentre on the northern tip of Awaji Island. The earthquake caused severe damage of gravity type retaining walls. The Japanese Geogrid Research Board and the Public Work Research Institute inspected sixteen GRS structures near the hypocentre where a seismic intensity of MMVII was recorded (Nishimara et al., 1996). The inspected structures were geogrid reinforced walls supporting highways, railway lines and parking areas. No signs of damage were observed during the inspections.

For ten of the inspected sixteen GRS structures a detailed verification of their seismic resistance was undertaken. The committee on Earthquake Observation and Research in Kansai reported peak ground accelerations between 0.4 g and 0.8 g for the area where the ten structures were located. Nishimara et al. (1996) suspect that some of the structures had not been designed to withstand any seismic loads. Nevertheless all of the structures demonstrated high seismic resistance and stayed in serviceable condition after the earthquake.

For example, the detailed analysis of the seismic stability for one of the walls, assuming the horizontal ground acceleration as low as 0.1 g, indicated that the structure's factor of safety was less than 1.0. However, the wall stayed in serviceable condition despite having undergone ground accelerations between 0.5 g and 0.7 g and despite the ground rupture that resulted in a 0.3 m wide crack on the ground surface in front of the wall.

The predicted critical horizontal seismic coefficients (in other words, seismic coefficient which, according to the current design methods, would be expected to initiate permanent movement of the structure) for all analysed walls appear to be significantly lower than the factual recorded seismic coefficients. On the basis of the analysis, Nishimura et al. suggest that the existing seismic design methods for GRS structures are conservative. Similar observations and conclusions were reported by Tatsuoka et al. (1996) and Collin et al. (1992).

The satisfactory dynamic performance of the GRS structures is normally explained by the following:

- The actual tensile strength of geosynthetic materials under seismic conditions is higher than that for static conditions. For seismic design, tensile strength tests conducted at higher than standard rates of strain would be more appropriate. These tests would often result in an allowable tensile resistance for seismic conditions appreciably higher than that for static conditions.

- The confined (in-soil) tensile strength of geosynthetic materials is very often higher than their unconfined (in-air) strength. In-soil ultimate tensile strength as high as four times the in-air strength has been reported (Claybourn et al., 1993).

- The static design procedures are conservative.

- The existing seismic design methods do not take into account the ductility of GRS structures. For example, Tatsuoka (1996) suggests that the ductility of GRS structures could be reflected in the design by using a design seismic acceleration lower than the expected peak ground acceleration.

4. STATIC DESIGN OF GRS STRUCTURES

Different New Zealand consultants use different empirical design methods. The various methods yield varying results. All empirical methods analyse external and internal stability of GRS structures.

In the external stability analysis the structure itself is considered to be a rigid body and the stability of the structure with respect to sliding, overturning, bearing capacity or global failure (beneath or behind the structure) is checked by common methods of soil mechanics and foundation engineering. The internal stability analysis addresses the structural integrity of GRS structures. A number of different methods for the internal stability assessment have been developed. Most of them are based on limiting equilibrium analyses of destabilising horizontal forces resulting from earth pressures and stabilising horizontal forces due to reinforcement. Internal failure of a GRS structure occurs in two different ways, either failure by excessive deformation (or breakage) of geosynthetic reinforcement or failure by pullout of geosynthetic reinforcement out of the soil mass. The design process comprises determining maximum tension forces in the geosynthetic reinforcement and comparing them with the available pullout capacity and the tensile strength. Almost all empirical methods assume that an active zone and a resistant zone are present within a GRS structure. These zones are separated by an imagined maximum tensile force line. Most of the methods do not assess deformations of GRS structures, caused by the flexibility of the body of the structure itself. Several most frequently used design methods for GRS structures with vertical and near vertical ($\beta \geq 70°$) face are outlined below. Different design methods use different factors of safety (total, partial or both), assume either peak (ϕ'_p) or constant volume (ϕ'_{cv}) angles of internal friction, and recommend different angles of friction at the back of the structure (δ). The assumptions and the factors of safety used by the methods are summarised in Table 2.

- Forest Services Method (USA)

This design approach was first proposed by Steward, Williamson and Mohney (1983) of the US Forest Service and has been subsequently incorporated (with some amendments) in a number of US design guidelines for GRS structures. (AASHTO, 1992; Christopher et al., 1990). To determine the geosynthetic layer separation distances, earth pressures are assumed to be linearly distributed with depth based on active (K_a) conditions for the soil backfill and at-rest (K_o) conditions for the surcharge.

Table 2. Summary of Design Methods

Method	Amount of reinforcement relative to that required by Forest Service method (%)	Assumed failure surface	Foundation bearing pressures	Soil pressures	Soil strength	Soil pressures and friction on back of wall	Minimum length of reinforcement	Soil/reinforcement interaction friction coefficient	Factors of safety
Forest Service	100	Rankine	Meyerhof pressure distribution	Ka-conditions	ϕ'_p	Ka-conditions, no friction	70% of wall height or 2.4 m minimum	From pullout and direct shear tests or from empirical relationships	Total Factors: Sliding 1.5 , Overturning 2.0 Bearing Capacity 2.5, Pullout resistance 1.5 , Rupture1.2-1.78
Leshchynsky	170	Log-spiral or planar	Not analysed	-	ϕ'_p	Ka-conditions, $\delta = (2/3)\phi'$ to ϕ'	-	From pullout and direct shear tests or 0.7-0.8	Total Factors: Sliding 1.1-1.3 , Pullout resistance1.5 Partial Factors: Rupture 1.3-1.5, Soil Strength 1.3
Schmertmann	55	By-linear	Not analysed	-	ϕ'_p for small strains; ϕ'_{cv} for large strains	Ka-conditions; $\tan\delta = \frac{\tan\phi'}{1.5}$	-	0.9	Partial Factors: Soil strength 1.5
Bonaparte	60	Rankine	Meyerhof pressure distribution	Ka-conditions, (vertical stresses induced by thrust of retained fill are calculated using Meyerhof)	ϕ'_p	Ka-conditions, no friction	-	From pullout and shear tests or from empirical data	Total Factors: Should be used but no specific guidance given. Partial Factors: Soil strength 1.5
DIBt	70	Bi-linear	Meyerhof pressure distribution	Not analysed	ϕ'_{cv}	Ka-conditions $\delta = (2/3)\phi'$	-	0.8	Total Factors: Sliding 1.5, Bearing capacity 2.0, Pullout 2.0 , Rupture 1.5
BS8006 (walls)	75	Rankine	No specific guidance to assess ultimate pressures	As for Bonaparte method	ϕ'_p	Ka-conditions, no friction	70% of wall height or 3m minimum for vertical wall	From pullout and direct shear tests or from empirical data	Partial factors: Soil unit mass 1.0-1.5 External dead loads 1.0-1.5 External live loads 1.5 Soil strength 1.0-1.6 Spoil/reinforcement interaction 1.0-1.3 Bearing capacity 1.35 Sliding 1.3-1.3 Economic ramifications of failure 1.0-1.1
BE 3/78	110	Rankine	Trapezoidal pressure distribution	Ka-conditions	ϕ'_p	Ka-conditions, no friction	80% of wall height or 5m minimum	From pullout and direct shear tests	Total Factors: Sliding 2.0 Bearing capacity 2.0 Pullout 2.0
HA 68/94	70	Bi-linear	Not analysed	-	ϕ'_{cv}	Ka-conditions no friction	-	From pullout and direct shear tests	-

However, it was shown later that K_a rather than K_o conditions should be used for surcharge. The method is based on substantial amount of test data obtained as part of major research projects.

- Leshchinsky Method (USA)

Leshchinsky et al. (1995) proposed a design method based on the variational limit equilibrium approach. Using variational extremisation Leshchinsky showed that there are two possible failure modes in homogeneous soil: rotational or translational. The slip surface geometry corresponding to the first mode is log - spiral and to the second mode - planar. The failure mechanism with the lowest factor of safety is likely to develop. A closed-form solution for each failure mode based on variational approach was used to develop detailed design charts. The charts account for a transition from a translational failure surface (for near vertical slopes) to a rotational failure surface (for flatter slopes). A computer program ReSlope based on Leshchinsky method has been developed and is used by some New Zealand consultants.

- Schmertmann Method (Design Charts) (USA)

Schmertmann et al. (1987) developed design charts for GRS slopes with angles varying from 30º to 80º for the Tensar Corporation. Schmertmann's method is a further development and improvement of the design charts published earlier by Ingold (1982), Murray (1984), Jewell et al., (1984), Leshchinsky and Reinschmidt (1985), Schneider and Holtz (1986) and Ruegger (1986). All these methods are based on limit equilibrium models to investigate the amount of reinforcement required to maintain slope stability.

- Bonaparte Method (USA)

Bonaparte et al. (1985) suggested a design method which is very similar to all other tieback wedge methods. The only difference is in the definition of the lateral pressures. As opposed to other tieback wedge methods, in the Bonaparte method earth pressures within the reinforced soil mass include a vertical component of the earth pressure thrust from the retained fill. The maximum vertical stresses induced by gravity, uniform vertical surcharges and the active thrust from the retained fill at any depth are calculated assuming Meyerhof's internal vertical stress distribution beneath and within the reinforced fill.

- Deutsches Institute für Bautechnik (DIBt) Method (Germany)

The DIBt design method is described in the Deutsches Institut für Bautechnik (DIBt) Certificate. The method is based on appropriate German DIN standards. Internal stability calculations are based on a two part wedge analysis of the reinforced soil block. The design strength of the geosynthetic reinforcement is specified in the DIBt Certificate. The method has been incorporated into the Winwall computer programme developed by Netlon Limited and is widely used in New Zealand.

- HA68/94 Method (UK)

The details of the method are given in the UK's Department of Transport advice note HA68/94. The method deals with steep slopes (up to 70º) on stable foundations and assumes a two part wedge mechanism. Surcharges are presented as uniformly distributed loads which can be modelled as an extra height of the GRS structure. The method is based on a two part wedge mechanism of failure which, whilst being a simplified approach, enables a conservative solution to be obtained. In essence, this method is very similar to the Schmertmann method described above. The only difference between HA68/94 and the Schmertmann method are that interwedge friction in HA68/94 is ignored, when in the Schmertmann method the interwedge friction angle is assumed to be equal to the factored soil friction angle. HA68/94 gives design charts similar to the Schmertmann method.

- BE 3/78 Method (UK)

The details of the method are given in the UK's Department of Transport technical memorandum BE3/78 which was first published in 1978 and updated in 1987. Reinforcement is designed at working loads with built in safety factors to produce permissible stresses. Lateral earth pressure distribution is based on the active pressure coefficient. The depth of the reinforced soil block and hence the length of reinforcement is determined from the external stability analysis while the quantity and layout of reinforcement is determined from the internal stability analysis. In the analysis of the external stability the effect of friction on the back of the wall is ignored. The minimum length of reinforcement is specified as the greater of 5 m or 80% of the retained height.

- BS8006 Method for GRS Walls (UK)

BS8006 has been developed as guide for good current practice in the United Kingdom. The document has been written in the limit state format, using partial factors of safety. The BS8006 method for GRS walls is a tie-back wedge method similar to the Bonaparte method described above in that the earth pressures within the reinforced soil mass include a vertical component of the active thrust from the retained fill and that the Meyerhof internal vertical stress distribution is used to calculate the vertical stresses induced by gravity, vertical surcharges and the active thrust. BS8006 is probably the only guidelines which provide recommendations on the acceptable post-construction strain limits for the reinforcement. According to BS8006 the post construction strains should be considered separately from the external and the internal stability analyses.

Table 2 also gives typical geosynthetic quantities, required by different design methods relative to the quantity required by the Forest Service method for a 4m high GRS wall with vertical face and foundation, reinforced fill and backfill comprising granular material. The comparison indicates that there is a discrepancy in amounts of geosynthetic required by different design methods.

5. SEISMIC DESIGN OF GRS STRUCTURES

Currently there are two different groups of seismic design methods. The first group is based on pseudo-static approach. These methods assume that the failure occurs along a certain slip surface. The internal stability, i.e. the ability of the reinforced soil mass to behave as a coherent mass and be self-supporting under the action of its own weight, reinforcement tension force, surcharges and inertia forces (caused by the seismic excitation of the reinforced fill) is checked. These methods address the seismic stability of GRS structures but do not address their deformation or displacement.

The second group of methods is based on the consideration of permanent displacement of GRS structures. These methods allow GRS structures to be designed for a critical acceleration less than the anticipated maximum seismic acceleration. The permanent displacement is assessed on the basis of the Newmark type analysis. Several methods commonly used in the current design practice are briefly outlined below.

- Public Works Research Institute (PWRI) Method, Japan (pseudo-static).

The method assumes that tensile forces in geosynthetic reinforcement under static conditions increase linearly with depth. It is also assumed that tensile forces in reinforcement caused by seismic loads are distributed uniformly. The maximum total tensile force in the reinforcement is then obtained by the equilibrium analysis using circular slip surfaces where a pseudo-static force equal to the seismic coefficient times the weight is added at the centroid of each slice. The factor of safety of 1.2 is required for static conditions and the factor of safety of 1.0 is considered sufficient for seismic conditions. The tensile forces in each reinforcement layer are then calculated (using the assumed tensile force distribution), and geosynthetic type, length and spacing are determined. Finally, the overall external stability under seismic conditions is checked assuming that GRS block behaves as a pseudo retaining wall. Nishimura et al. (1996) suggest that the method is conservative.

- Geogrid Research Board (GRB) Method, Japan (pseudo-static)

The Japanese Geogrid Research Board (GRB) developed and published design and construction manual for GRS structures (Nishimura et al., 1996). A two part wedge failure mechanism is considered to obtain the maximum tensile force in reinforcement. The required reinforcement length is then assessed from the reinforcement pullout analysis. The external stability analysis is done in a manner similar to that for the PWRI method. The final reinforcement length is the maximum of those obtained from the two part wedge stability analysis and the external stability analysis (including sliding and overturning failure modes).

- Bonaparte Method, USA (pseudo-static)

Bonaparte et al (1986) used a simple pseudo-static, rigid-body analytical model to develop design charts for the seismic design of GRS slopes. The total required reinforcement force is determined through a limit equilibrium analysis assuming a Coulomb failure wedge. The increase in the required length of reinforcement due to the horizontal inertia force is evaluated based on two criteria: reinforcement pullout behind the critical wedge and sliding of reinforcement soil mass over a layer of reinforcement at the elevation of the toe of the slope. The Bonaparte method suggests the use of 85% of the peak ground acceleration, because the horizontal inertia force associated with the peak ground acceleration is applied for only a very short period of time. It is also suggested to use 90% of the allowable reinforcement tensile strength and to apply factors of safety 1.1 to 1.5 to the constant volume soil shear strength.

- AASHTO Method, USA (pseudo-static)

The AASHTO method is based on a modified pseudo-static design approach. A dynamic horizontal thrust (P_{AE}) exerted by the retained fill is evaluated by the pseudo-static Monanabe-Okabe analysis. The horizontal inertia force acting in the reinforced soil mass (P_{IR}) is calculated assuming that only a portion of the total reinforced block with a base width of 0.5H is affected. The maximum acceleration coefficient at the centroid of the reinforced mass is assumed to be: $A_m = (1.45 - A)\,A$, where A is the maximum ground acceleration coefficient. In the external stability analysis the equilibrium of the reinforced block under static forces (lateral thrust due to static soil pressures and surcharges), 50% of the seismic thrust (P_{AE}) and the full internal force (P_{IR}) is checked. The reduced P_{AE} is used because

P_{AE} and P_{IR} are unlikely to peak simultaneously. The internal stability is checked with respect to the elongation (or breakage) of the reinforcement and the pullout failure. The design for internal stability, therefore, consists of determining the maximum developed tension forces, their location along a critical slip surface and the resistance provided by the reinforcement both in pullout and tensile strength. The inertia force affecting the active wedge is assumed to be distributed in the reinforcement layers in proportion to the resistant length of each reinforcement layer. The total tensile force in the layers of geosynthetic reinforcement is calculated as a sum of the maximum static tension and the dynamic increment.

- Ling Method, USA (pseudo-static)

The detailed description of this method is given by Ling et al. (1997). The design procedure is an extension of the static design method proposed by Leshchinsky. The seismic inertia force is considered to be acting at the centre of gravity of the potential failure soil mass. Ling proposed to use maximum earthquake to describe seismic loading, because amplification in GRS structures is still not well understood. The design internal friction angle of soil is obtained by reducing the available internal friction angle with an appropriate factor of safety. In the internal stability analysis the required strength for each layer of geosynthetic is calculated by tieback failure analysis. The equilibrium of a potential failure soil mass, extending from layer n (or from the top of structure for the first step) to layer n-1 is considered and all maximum geosynthetic forces required to stabilise the soil mass are determined. The outermost log-spiral surface obtained from this analysis defines the active soil mass. Compound stability analysis (considering potential slip surfaces emerging beyond the outermost log-spiral failure surface either outside or within the effective anchorage length) is then undertaken to determine the required anchored length of geosynthetic. If necessary, the minimal lengths calculated from the internal stability analysis are increased to achieve a required factor of safety. The length required to resist direct sliding of a reinforced soil block is then determined on the basis of the two-part wedge mechanism. A uniform design layout of the geosynthetic reinforcement is recommended based on the longest of the lengths required to resist compound or direct sliding.

- Ling Method, USA (permanent displacement)

The proposed procedure is an extension of a Newmark type sliding block analysis. In direct sliding analysis the reinforced soil zone is treated as a rigid-plastic block in which displacement is induced only when the critical acceleration is exceeded. The method also assumes that the critical acceleration in the reverse direction is large enough not to cause any permanent displacement. The critical acceleration against sliding is obtained by considering the equilibrium. The horizontal permanent displacement is obtained by double-integrating the equation of motion ignoring the damping and stiffness terms. The double integrating is conducted numerically for a random earthquake motion representative of site conditions. It is critical for the method that a representative strong ground motion and displacement chart is selected for the design. Ling's comparison of predicted permanent displacements with factual monitored displacement (based on a representative ground motion chart) for two GRS structures indicated satisfactory agreement. (Ling et al., 1997).

The seismic design procedures described above differ in many ways. However, in our view, the main difference is the difference in the methods recommendations on what magnitude of seismic acceleration should be used in the design. According to such recommendations, the methods can be divided into three categories: (A) - methods which require the maximum magnitude of seismic acceleration to be used for the whole GRS structure including backfill; (B) - methods which recommend the use of reduced magnitudes of seismic accelerations but assume that permanent displacement of a GRS structure will not take place; (C) - methods which allow permanent displacement of GRS structures to be assessed. (GRS structures, designed by these methods, can have critical seismic accelerations significantly lower than expected maximum seismic accelerations).

A comparison of three methods (one from each category) has been undertaken (Murashev, 1998) for 5 m high GRS structure with 20 equally spaced geogrid layers, vertical face and backfill soil with $\varnothing' = 30^{\circ}$ using charts developed by Ling et al. (1997). For the Category A method the seismic accelerations for both GRS block and backfill of 0.3 g was assumed. For Category B and C methods seismic accelerations for GRS block and backfill of 0.15 g and 0.3 g respectively were assumed. The comparison indicates that the reinforcement length required by Method B to resist direct sliding is 0.62H instead of 1.2H required by Method A. The required reinforcement length to resist sliding can be reduced to 0.5H and 0.4H if Method C is used and permanent displacements of 15 mm and 60 mm respectively are assumed to be acceptable. In summary, for the GRS structure with parameters described above, the required reinforcement length varies from 0.4H for methods of Category C assuming a permanent displacement of 60 mm to 1.2H for methods of Category A assuming no permanent displacement. In some instances, described by Ling et al. (1997) the upper limit of the required reinforcement length for Method A was as high as 14H. It is, therefore, obvious that in many cases methods of Categories A and B require excessively long geosynthetics. Very often space constraints and high cost involved make installation of long geosynthetics impractical. As currently there are no commonly accepted methods of Category C, designers very often rely on the conservatism of the static design procedures in these cases.

6. CONCLUSIONS

Different New Zealand consultants use different GRS design methods. Existing guidelines on static and seismic design of GRS structures differ. There is a discrepancy in the amounts of geosynthetic reinforcement required by different design methods. This results in the fact that GRS structures constructed to date have different levels of static and seismic resistance and different risks of failure under seismic load. Although designed by different design methods, most GRS structures in New Zealand performed satisfactorily. There is clearly a need for design guidelines for GRS structures in New Zealand.

7. ACKNOWLEDGMENTS

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