# Influence of foundation compressibility on reinforced soil retaining wall behaviour



Fawzy Ezzein GeoEngineering Centre at Queen's-RMC, Civil Engineering Department, Royal Military College of Canada, Kingston, ON, Canada Richard J. Bathurst GeoEngineering Centre at Queen's-RMC, Civil Engineering Department, Royal Military College of Canada, Kingston, ON, Canada

# ABSTRACT

The influence of compressible foundations on the mechanical behaviour of geosynthetic reinforced soil walls is very complex. This paper presents the results of two 1/6-scale reinforced soil wall tests that were carried out to isolate the influence of vertical foundation compressibility on wall behaviour. These tests are a continuation of a research program that initially investigated the influence of horizontal and vertical toe compliance on wall performance. The results of these physical model tests have been published in the proceedings of the two previous CGS conferences (Ezzein and Bathurst 2006, 2007). A control wall (Wall 16) was constructed using a high quality sand backfill, a rigid foundation and a rigid horizontal support at the toe of the facing. A second wall (Wall 17) was nominally the same but was constructed over compressible layers of rubber and foam. The paper presents measured results for wall deformations, reinforcement strains and soil settlement at end of construction and during staged uniform surcharge loading. These results have important implications to current design of reinforced soil walls that do not consider the influence of foundation compliance on the magnitude of reinforcement loads and their distribution.

# RÉSUMÉ

L'influence des fondations compressibles sur le comportement mécanique des massifs de sols renforcés de géosynthétiques, est très complexe. Cet article présente les résultats de deux essais à une échelle de 1/6 sur massifs renforcés, réalisés pour isoler l'influence de la compression verticale de la fondation sur le comportement du massif. Ces essais sont une continuation d'un programme de recherche qui initiallement étudiait l'influence de la souplesse verticale et horizontale en pied de mur sur le comportement du massif. Les résultats de ces essais sur modèles physiques ont été publiés dans les compte-rendus de deux congrès antérieurs de la SCG (Ezzein et Bathurst 2006, 2007). Un mur contrôle (Mur 16) a été construit avec un remblais de sable de haute qualité, une fondation rigide et un soutient horizontal rigide au pie du mur. Un second mur (Mur 17) était essentiellement identique sauf qu'il a été construit sur des couches compressibles de caoutchouc-mousse. L'article présente les résultats mesurés pour les déformations du mur, les déformations des renforcements et le tassement du sol après construction et pendant l'application par palier de surcharges uniformes. Ces résultats ont des implications pour le design actuel des massifs de sols renforcés, qui ignore l'influence de la souplesse de la fondation sur l'intensité et la distribution des efforts dans les renforcements.

# 1 INTRODUCTION

Current design methods for reinforced soil retaining walls simply assume that the structures will be constructed over rigid or very stiff foundations. Hence, they ignore the influence of foundation stiffness (soil compliance) on the magnitude and distribution of reinforcement loads under serviceability and ultimate (collapse) conditions. Direct or indirect evidence of the influence of foundation support condition on wall behaviour can be found in the literature and some examples are reported here.

Murray and Farrar (1990) reported the results of an instrumented incremental concrete panel wall 4 to 8 m in height constructed with steel strip soil reinforcement. The wall was built over poor ground. Settlements at the wall footing as great as 90 mm were recorded at the end of construction. Settlements below the reinforced soil zone were not recorded. There was an approximate 40%

reduction in vertical pressure at the foundation surface directly behind the facing column compared to measurements recorded at 1 m or more beyond the back of the wall. This was ascribed to interface friction between the back of the concrete facing units and the soil. The peak tension in the reinforcement on a per unit area of facing panel was measured as  $65 \text{ kN/m}^2$  at 1.5 - 2 m back from the facing at the bottom reinforcement layer. Nevertheless, there was no visual evidence of wall distress due to possible differential settlements in the direction normal to the running length of the wall. Horizontal movements at the base of the wall were not recorded. However, wall out-of-alignment was small (about 20 mm) and was due to local panel rotations.

Rowe and Skinner (2001) carried out a numerical investigation based on a full-scale test wall constructed by the Public Works Research Institute in Japan (Tsukada et al. 1998). They used a finite element model (FEM) code to quantify foundation stiffness effects on an 8-m high wall constructed using a modular block (segmental) construction. They reported that the soft compressible foundation case resulted in wall deformations that were 150% greater than the same wall built on a rigid foundation. They also reported that the facing rotated about the toe when the wall rested on a rigid foundation and around the crest when the wall rested on a soft foundation. In the same study, they showed that reinforcement strains increased between 80% and 350% for the wall built on a soft foundation. Similar trends in wall response using small-scale physical models have been reported by Palmeira and Monte (1997).

The authors and co-workers at the Royal Military Collage of Canada (RMC) are carrying-out a long-term research project on reinforced soil retaining walls. The purpose of this research is to investigate the mechanical behaviour of these structures and to generate a highquality data base that can be used to calibrate numerical models and refine current design methods. A total of 13 full-scale reinforced soil retaining walls have been constructed and tested at RMC (Bathurst et al. 2000, 2006; Saunders et al. 2001). All of these walls were constructed on a rigid foundation and the toe was restrained horizontally and vertically, but was free to rotate.

This paper reports results from a subset of the larger research program at RMC that investigates the effects of toe support and foundation stiffness on reinforced soil retaining wall behaviour using reduced-scale physical model tests. The testing program was comprised of a series of reinforced soil retaining walls with different boundary conditions. Their response was compared to a control wall case with rigid toe and foundation conditions. Results that are focused on the influence of horizontal and vertical toe restraint on wall response are reported by Ezzein and Bathurst (2006, 2007).

In this paper the control wall (Wall 16) and Wall 17 are described. The control wall was constructed using a high quality sand backfill, a rigid foundation and a rigid horizontal support at the toe of the facing. Wall 17 was nominally the same but was constructed over compressible layers of rubber and foam. This paper presents quantitative measurements that isolate the influence of foundation stiffness on the behaviour of these two walls.

#### 2 EXPERIMENTAL APPROACH

# 2.1 Test Facility and General Arrangement of Wall Models

The test facility for reduced-scale retaining walls is located in the Dolphin Structures Laboratory of the Civil Engineering Department at RMC. The facility has been described by Esfehani and Bathurst (2001).

The facility was designed as a strong box seated on a rigid base. The inside dimensions of the facility are 1.5 m high by 1.57 wide. The sand backfill extends 2.3 m behind the wall facing column. The base of the facility is anchored to the laboratory strong floor. The sidewalls of the test facility are made of 18 mm-thick transparent



Figure 1. View of test facility strong box and model wall

Plexiglas sheets braced by an arrangement of steel beams. Three layers of clear lubricated polypropylene sheets were placed over the Plexiglas sidewalls to reduce sidewall friction. The combination of friction reduction, stiff external bracing and a model width to height ratio of 1.3 resulted in boundary conditions approaching an idealized plane strain condition. A pair of air bags was used to apply a uniform surcharge to the entire top surface of the wall backfill. On top of the airbags, two layers of plywood and a set of hollow structural steel sections were anchored to the strong floor using threaded rods to provide a reaction to the air bags. The surcharge arrangement allows pressures up to 65 kPa to be applied to the backfill surface. An overview of the test facility is shown in the photograph of Figure 1.

The facing column was constructed from 32 vertical stacked and interlocking rectangular hollow steel sections with cross section dimensions of 76 mm by 38 mm and 4.8 mm thick. The sections were connected together by shear pins to transfer shear forces and to prevent lateral movement between section layers. In this series of tests the sand backfill was reinforced with six layers of a modified polyester (PET) geogrid as shown in Figure 2.

2.2 Materials

## 2.2.1 Soil Backfill

A clean uniform particle size rounded beach sand was selected as the backfill material in these tests. Figure 3 provides the particle size distribution curve. The material is classified as a poorly graded sand using the Unified Soil Classification System with coefficient of curvature  $C_c = 1.23$ , and coefficient of uniformity  $C_u = 1.83$ . The same material has been used in the full-scale testing program at RMC and in the earlier related work reported by Ezzein and Bathurst (2006, 2007). This material was selected because it has a flat compaction curve and hence the same relative density after compaction was easily achieved for all tests. The plane strain friction angle of the sand interpreted from laboratory plane strain tests is 44 degrees (Hatami and Bathurst 2005). The sand was placed in 190 mm lifts then compacted to bulk density of



Figure 2. Cross-section view of model wall and instrumentation (Wall 17 with compressible foundation)

 $1680 \text{ kg/m}^3$  using a hand plate tamper.

#### 2.2.2 Reinforcement

A commercially available knitted and coated polyester (PET) geogrid was used as the geosynthetic reinforcement material in this investigation and earlier related tests. This product was selected because it was the PET geogrid with lowest available stiffness at the time of this research program (Esfehani and Bathurst 2001). Furthermore, PET products are less sensitive to creep effects than polyolefin products with similar strength and stiffness.

The PET geogrid was modified by removing 2/3 of the longitudinal members in order to achieve reasonable scaled stiffness. According to the scaling laws proposed by Iai (1989) the equivalent stiffness at prototype-scale can be calculated as  $J_p = (\lambda^2) \times J_m$  where  $J_p =$  stiffness of a typical prototype-scale geogrid,  $J_m =$  stiffness of the model geogrid, and  $\lambda = 6$  is the geometrically determined scaling factor. Hence, the model reinforcement stiffness  $J_m = 26$  kN/m is equivalent to  $J_p = 936$  kN/m at prototype scale.

The basic properties of the original and modified

geogrid reinforcement used in the wall models are given in Table 1.

#### 2.2.3 Toe and Foundation Condition

The objective of the current study was to investigate the influence of foundation compliance (stiffness) below both the facing and backfill on reinforced soil retaining wall performance. Two walls were constructed and tested that were identical except for the foundation boundary condition. Wall 16 was the control wall with the facing and backfill constructed on a rigid base (steel plate under facing and concrete base under the backfill). The toe of the wall was restrained horizontally and vertically. For the second wall (Wall 17), the entire model (facing and backfill) was constructed on compressible rubber and foam layers. The toe of Wall 17 was allowed to rotate and move vertically but was restrained in the horizontal direction by using a special hinge mechanism (Figure 4).

The rubber and foam materials, number of layers and their arrangement were selected based on numerical investigation (using FLAC models), standard correlations between subgrade modulus and soil type, and laboratory compression tests on candidate materials. The subgrade



Figure 3. Particle size distribution

modulus stiffness value for Wall 17 is  $k_s = 1,840 \text{ kN/m}^3$ . For Wall 16 with a rigid toe, the value of  $k_s$  is assumed as infinity. Table 2 summarizes the properties and number of compressible layers used for Wall 17. The vertical foundation stiffness in Wall 17 is equivalent to loose sand or very soft clay (Das 1990). To distinguish this wall from the control wall, the foundation for Wall 17 is called compressible in the following text.

#### 2.2.4 Instrumentation

The wall models were heavily instrumented with a total of 133 to 138 instruments. Figure 2 shows the general instrumentation scheme for Wall 17. The vertical toe load was measured by six button load cells placed directly under the steel plate supporting the facing. Two button load cells installed in the horizontal direction at the base of the steel plate were used to measure the horizontal toe reaction. The vertical earth pressure at the base of the test facility was monitored by three earth pressure cells embedded in the concrete foundation. The connection loads between the facing column and the geogrid reinforcement layers were recorded by three load rings at each layer. The horizontal facing displacements were measured by a single column of displacement transducers (potentiometers) mounted against the centreline of the wall face at the reinforcement elevations and at the top of the test facility. A single displacement potentiometer was used to check for possible displacement at the wall toe. The surface settlement of the backfill soil was monitored by a set of displacement potentiometers attached to aluminium plates located at a distance of 50, 200, 350, 500, 650, 800 and 950 mm behind the facing. The foundation settlement of Wall 17 was measured by five displacement potentiometers installed in the concrete base at 122, 310, 730, 900 and 1260 mm distance directly behind the centre of the facing column. The reinforcement layer displacements were recorded by wire-line extensometers attached to the geogrid and monitored by displacement potentiometers mounted at the back of the test facility. Finally, local strains in the reinforcement were measured directly by

Table 1. Reinforcement properties

Property	Original	Modified
Aperture size (mm x mm)	27 x 22	81 x 22
Number of longitudinal members per metre width	36	12
Ultimate strength (kN/m)	17.5	5.6
Strength at 5% strain (kN/m)	4.4	1.3
Stiffness at 5% strain (kN/m)	88	26

Note: strength and stiffness properties from single strand tensile tests carried out at 10% strain per minute.

Table 2. Foundation details for Wall 17

Material type	Pad Floor Foam, and SBR Durometer black rubber gum
Foundation arrangement	Four layers of foam and one layer of black rubber
Modulus of subgrade reaction k <sub>s</sub> (kN/m <sup>3</sup> )	1,840

strain gauges bonded to selected longitudinal members of each reinforcement layer.

The data from all instruments were recorded continuously using an HP 3497A data acquisition system controlled by a PC computer running HP VEE software.

#### 2.2.5 Wall Construction

The facing panels with shear keys were placed from bottom-up to the elevation of each reinforcement layer. The sand was placed, compacted using a hand-held plate tamper and then levelled. Next, the geogrid layer was laid on the top of sand layer and the layer instruments connected to the data acquisition system. This process was repeated until the full height of the wall was achieved. During construction the facing column was braced externally. However, the external props were arranged so that there was no transfer of vertical force from the wall facing to the props that could complicate the interpretation of measured footing loads during construction. After the top layer of sand was levelled, the airbags and the surcharge system were installed. The external props were released (end of construction) and the wall model left unsurcharged for 24 hours. Next, surcharge loads were applied in stages with each increment maintained for 24 hours until the maximum surcharge of 65 kPa. After 96 hours, the wall was unloaded in 5 steps. Finally, the wall was carefully excavated in horizontal lifts to examine the reinforcement and to survey vertical settlements at each reinforcement layer.



Figure 4. Toe configuration for Wall 17 (compressible foundation)

## 3 TEST RESULTS

For brevity, only selected test results are presented here. Vertical toe settlements versus vertical foundation pressure are plotted in Figure 5. For Wall 17 (compressible foundation), vertical toe settlements generally increased with increasing vertical pressure. However, when the external props were released, an immediate jump in settlement was recorded. This is due to rotation of the facing about the toe. At the end of surcharging, the measured toe settlement was 38 mm.

Figure 6 shows foundation settlement profiles for Wall 17 during construction and prior to prop release. As the depth of fill increases the magnitude of foundation compression (settlement) increases. However, the settlement is not uniform between the base of the facing and below the compacted backfill soil during placement of the reinforcement layers. It is believed that there is initially little vertical load transfer to the facing through the connections and due to soil-facing shear transfer. Only after the full height of the compacted backfill is in place is there enough vertical load transfer to the facing to increase footing settlement to values similar to those recorded below the reinforced soil mass. At all stages prior to prop release the maximum settlement was recorded at 122 mm from the back of facing column. Once the props were released the largest settlement was recorded under the facing column (combination of vertical facing settlement and toe rotation) as shown in Figure 7. During staged surcharge loading the least settlement was recorded directly behind the facing column. This is believed to be due to load transfer to the facing column that leads to soil arching behind the heel of the facing. As

distance from the facing increases, foundation settlements become more uniform, particularly beyond the reinforced soil zone (Figure 7).

Figure 8 shows the evolution of vertical toe load during the construction stage. Also shown is the total facing weight. The data show that for the wall constructed over a rigid foundation (Wall 16), the vertical toe load was about two times greater than the self-weight of facing column. We attribute this to soil down-drag forces that are generated at the back of the wall due to the soil hanging up on the reinforcement layers at the connections with the vertically rigid wall, and possibly soil-facing interface shear. However, the effect of the down-drag forces was eliminated due to the ability of the wall facing to settle for the case of Wall 17 as described above. This can be verified by noting that the recorded vertical toe loads are close to the facing column self-weight for the wall constructed over the compressible foundation.

The data in Figure 9 show the total vertical load measured at the toe during surcharge loading of the two walls. The vertical toe load recorded for the control wall was higher than for the wall with the compressible foundation (Wall 17). At prop release the vertical toe load for Wall 17 was about 40% less. Only after the maximum surcharge pressure increment was applied did the toe load recorded for the more compressible foundation wall approach the value for the rigid foundation case. However, this may be due to the greater toe rotation and the influence of wall facing geometry (larger outward deformations) since the pressures applied at the toe and below the soil mass were within the linear elastic range of



Figure 5. Vertical toe settlement versus vertical pressure (soil self-weight plus surcharge)



Figure 6. Vertical foundation settlement during construction



Figure 7. Vertical foundation settlement during surcharge



Figure 8. Vertical toe load versus wall height during construction



Figure 9. Vertical toe load versus surcharge pressure



Figure 10. Horizontal toe load versus surcharge pressure

the compressible foundation material.

Horizontal toe load versus surcharge pressure is plotted in Figure 10. The horizontal toe load in Wall 17 was comparable to that of Wall 16 at the end of construction but increased during surcharging. At a surcharge pressure of 30 kPa the ratio of toe load values was about 1.5. At higher surcharge loads there is a drop in horizontal toe load for the compressible foundation wall that is due to overloading of the mechanical hinge. Where the data for this wall is not distorted by mechanical failure, the greater horizontal toe load in Wall 17 is thought to be due to reduced shear stiffness within the foundation support and at the soil-foundation interface. In other words, load transfer to the rough rigid concrete foundation bottom of Wall 16 is greater than for Wall 17 which was built with layers of deformable rubber and foam materials. This boundary condition requires that the toe of Wall 17 carry more load.

Figure 11 shows the facing profile for the two walls at 65 kPa surcharge increment based on manual surveys. The datum for these measurements is the vertical orientation of the facing column just prior to prop release. It was observed that the control wall recorded less displacement than the wall with a compressible foundation particularly over the bottom half. The horizontal displacement at the base of Wall 17 is consistent with comments made above regarding the reduced shear stiffness of the foundation constructed with deformable materials. The data in Figure 11 show that foundation stiffness had no significant effect on the magnitude of the *maximum* horizontal facing movement, but the location of maximum deformation shifted from 0.75H for the wall with a rigid foundation to 0.5H for wall with the compressible foundation where H is height of wall.

Figure 12 shows the backfill settlement profiles recorded at the surface of the soil backfill at the end of the maximum surcharge increment (65 kPa). As expected, the settlements behind Wall 17 are greater than for the control wall. The greater settlement in Wall 17 is the result of greater facing deformation and larger foundation settlement as shown in earlier figures. The *maximum* backfill settlement for Wall 17 is 40% greater than for Wall 16 and settlement propagates beyond the reinforced soil zone for the compressible foundation model. However, for both walls there is evidence of a local reduction in settlement immediately behind the facing which is consistent with comments made earlier regarding soil-facing shear transfer and soil hanging up on the connections.

Figure 13 shows the strain magnitude and distribution in the six layers of the reinforcement at surcharge load level of 40 kPa for the model walls. The measured results were taken from strain gauge readings. Note that some readings are missing because of gauge failure due to overloading. For those gauges that survived, the strains in both walls are similar except for the two upper layers where strains are lower at locations close to the back of the facing for the compressible foundation wall. This can be ascribed to the observation that the horizontal toe load was greater for this wall and hence less total tensile load was mobilized in the reinforcement layers for Wall 17.



Figure 11. Wall displacement profile



Figure 12. Backfill settlement profiles from settlement plate measurements at end of 65 kPa surcharge stage

#### 4 CONCLUSIONS

This paper presents the results of two 1/6 reduced-scale geosynthetic reinforced soil wall models that were constructed to a height of 1.2 m using a sand backfill and a propped panel facing and then uniform surcharged in stages. The control wall was constructed over a rigid foundation and the second wall was built over a compressible foundation. The test results demonstrate that the quantitative behaviour of these reinforced soil retaining walls was significantly affected by foundation stiffness. The data show that for the wall with greater compressibility below the toe and soil backfill: a) wall facing deformations and backfill settlements at end of construction and during surcharging were larger; b) downdrag forces at the connections and vertical toe loads were less, and; c) horizontal earth loads were redistributed to the toe and less load was carried by the reinforcement lavers. This is attributed to less shear stiffness within the compressible foundation and at the soil-foundation interface in this investigation. These results have





Figure 13. Distribution of reinforcement strains at end of 40 kPa surcharge loading

important implications to current design of reinforced soil walls that do not consider the influence of foundation compliance on the magnitude of reinforcement loads and their distribution.

#### ACKNOWLEDGEMENTS

Financial support for this study was provided by the Natural Sciences and Engineering Research Council (NSERC) of Canada and grants from the Department of National Defence (Canada). The first author was supported by a scholarship awarded by the Government of Libya.

#### REFERENCES

- Bathurst, R.J., Walters, D., Vlachopoulos, N., Burgess, P. G. and Allen, T.M. 2000. Full scale testing of geosynthetic reinforced walls, keynote paper, ASCE Special Publication No. 103, Advances in Transportation and Geoenvironmental Systems using Geosynthetics, Proceedings of Geo-Denver 2000, 5-8 August 2000, Denver, Colorado, pp. 201-217.
- Bathurst, R.J., Vlachopoulos, N., Walters, D.L., Burgess, P.G. and Allen, T.M. 2006. The influence of facing rigidity on the performance of two geosynthetic reinforced soil retaining walls, *Canadian Geotechnical Journal*, 43(12): 1225-1237.
- Das, B.M. 1990. *Principles of foundation engineering*, second edition, PWS Publishing, Boston, Mass., USA.
- Esfehani, M. and Bathurst, R.J. 2001. Reduced-scale testing of reinforced soil retaining walls with non-select fills, In *CD proceedings of 54th Canadian Geotechnical Conference*, 2001, Alberta, 7 p.
- Ezzein, F. and Bathurst, R.J. 2006. Influence of horizontal toe restraint on reinforced soil retaining wall behaviour, In *CD proceedings of 59th Canadian Geotechnical Conference*, 2006, Vancouver, BC, 8 p.
- Ezzein, F. and Bathurst, R.J. 2007. Influence of vertical toe condition on reinforced soil retaining wall behaviour, In *CD proceedings of 60th Canadian Geotechnical Conference*, 2007, Ottawa, ON, 8 p.
- Hatami, K. and Bathurst, R.J. 2005. Development and verification of a numerical model for the analysis of geosynthetic reinforced soil segmental walls under working stress conditions, *Canadian Geotechnical Journal*, 42(4): 1066-1085.
- Iai, S. 1989. Similitude for shaking table tests on soilstructure-fluid models in 1g gravitational fields, *Soils* and Foundations, 29(1): 105-118.
- Murray, R.T. and Farrar, D.M. 1990. Reinforced earth wall on the M25 motorway at Waltham Cross, *Proceedings* of the Institution of Civil Engineers, Part 1(88): 261-282.
- Palmeira, E.M. and Monte, L.M. 1997. The behaviour of model reinforced walls on soft soils, In *proceedings of Geosynthetics'97*, Long Beach, California, pp. 73–84.
- Rowe, R.K. and Skinner, G.D. 2001. Numerical analysis of geosynthetic reinforced retaining wall constructed on a layered soil foundation, *Geotextiles and Geomembranes*, 19(7): 387-412.
- Tsukada, Y., Ochiai, Y., Miyataka, H. and Tajiri, N. 1998. Field performance of a geosynthetic-reinforced soil wall with rigid facing, In *proceedings of 6th International Conference on Geosynthetics*, Atlanta, USA, pp. 577–580.
- Saunders, D.D., Walters, D.L., Bathurst, R.J. and Allen, T.M. 2001. The influence of reinforcement stiffness on the performance of full-scale reinforced soil segmental retaining walls, In *proceedings of 54th Canadian Geotechnical Conference*, 2001, Calgary, Alberta, Canada, 8 p.