CYCLIC BEHAVIOR PARAMETERS OF REINFORCED SOIL WALLS

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Abstract: A total of twenty, 1m height, reduced scale shaking table tests were conducted on reinforced soil walls. Physical models were subjected to constant acceleration amplitude sinusoidal input motion, different parameters of the model such as reinforcement length, spacing, stiffness, and soil density were varied and changes in the seismic response of the wall was studied. Shear stiffness (G) and damping ratio (D), distribution along the wall height was studied and effect of confining pressure and shear strain on variation of G and D was found. According to the findings, drawing hysteretic loops of cyclic behaviour of reinforced soil at various heights of the wall revealed that G will increase from top to bottom of the wall with increasing confining pressure. In addition, at each level with constant confining pressure, G will decrease with increasing shear strain. However, considering precision of shear strain measurements, D doesn't change along the wall height and corresponds to the shear strain variation, and at large strains of about 10^{-3}, an average damping ratio about 20 % was observed.

Keywords: Seismic behavior, Cyclic load, Degradation, Stiffness, Physical modeling, Reinforced soil wall

INTRODUCTION

Study of seismic behavior of reinforced soil walls is one of the main issues in geotechnical and earthquake engineering. Non-uniform distribution of acceleration throughout the wall and non-linear behavior of reinforced soil seems to be important for analyzing reinforced soil walls. As it is well distinguished, shear stiffness (G) and damping ratio (D) for small to medium strains are the two key parameters affecting dynamic response of any soil body. In the case of reinforced soil walls this issue is not well addressed in the literature. Based on element tests carried out by Iwasaki et al. (1978) the cyclic stress-strain test results show that G is proportional $\sigma_v^{m(\gamma)}$ where $m(\gamma)$ is the power that is a function of shear strain amplitude, γ , increasing from about 0.5 at γ less than about 0.01 % and becoming about 1.0 as γ approaches 1 % .

If the dependency of seismic response parameters such as shear stiffness modulus (G) and damping ratio (D) through the wall is understood, researchers can illuminate how much their model would be affected by low confining pressure and extend the results to the prototype structures for more realistic quantitative predictions. Furthermore, the test results can be applied to develop and validate numerical codes that can be used instead of investigating wall response at prototype scale (Koseki et al. 2006).

The recent review of the literature by Koseki et al. (2006) has highlighted the need for more sophisticated models for both soil and the reinforcement to better predict the seismic response of GRS structures. They emphasized that hysteresis models based on massing functions and their variants and strain-softening models for the backfill soil are potentially areas of future researches.

A 1g reduced scale physical model tests using a shaking table is the most common approach to gain qualitative and quantitative insights into the seismic behavior of reinforced soil wall systems. A disadvantage of reduced scale tests is that the response of the model may be influenced by low confining pressure, far end boundary conditions of the shaking table box, and improperly scaled mechanical properties of the reinforcement. Nevertheless, qualitative insights are possible using this experimental approach. Furthermore, the models can be used to develop and validate numerical codes that can be used in turn to investigate wall response at prototype scale (Koseki et al. 2006).

SHAKING TABLE TESTS

This study is a part of physical model test program including a total number of 20 shaking table tests carried out on reinforced soil walls. Tests were undertaken with a shaking table with the following specification:

- deck dimension: 1.8 * 1.2 m^2
- hydraulic jack capacity : 60 kN
- hydraulic jack displacement course : 250 mm
- electronic card A/D, D/A speed : 100 kHz
- model box dimension: 0.80 * 1.23 * 1.82 m^3

The 0.8*1.23*1.82 m^3 container box was fabricated from rigid, transparent Plexiglas sheets to make wall deformations and behavior visible. At different tests various model parameters, such as: length, spacing and stiffness of reinforcements; soil density; amplitude, frequency and duration of input motion were changed to find the effect of the aforementioned parameters on the seismic response of the wall, with the emphasis on the amount and modes of deformation. A brief summary of the tests is presented in Table 1.

According to the research plan, a study of reinforcement stiffness effect on GRS wall response was proposed. Two different categories of reinforcement, consisting of very low stiffness (too extensible) and relative higher stiffness (extensible) material were selected. The tests can be divided into two series: "strong type" reinforced soil wall tests and "weak type" reinforced soil wall tests.

MODEL GEOMETRY

To be consistent with previous shaking table studies conducted by other researchers, all physical models were constructed 1.0 m high. With consideration to the height of traditional walls, between 3.0 and 7.0 metre with an average of 5.0 metre, a 1.0 metre high model with a scale factor equal to 5.0 is a good physical model to reflect seismic behavior of geosynthetic reinforced soil walls.

Figure 1 shows a schematic geometry of the model used for all the present research. Foundation soil thickness was a firm 15 cm thick layer for all models. All walls, except wall 11, were constructed in 10 layers of 10 cm thickness. Wall 11 was built in 5 layers of 20 cm thickness. Wall facing is selected as wrap-around type. This type of facing is used to avoid any conflict of reinforced soil wall interaction with a structural rigid facing. Walls with a wrap around facing expect to show more displacement than other facing types, because there is no structural “stiff facing” to withstand lateral displacements.

Table 1. Brief summary of tests

TEST No.	a_{max} (g)	Frequency (Hz)	L/H	Sv/H	Wall Type	Geosynthetic Type	J (N/m)	T_u (N/m)
TEST 01	0.2	5	0.7	0.1	Strong	yw-m	9400	1700
TEST 02	0.3	5	0.7	0.1	Strong	yw-m	9400	1700
TEST 03	0.3	8	0.7	0.1	Strong	yw-m	9600	1300
TEST 04	0.3	8	0.9	0.1	Strong	yw-m	9600	1300
TEST 05	0.3	8	0.5	0.1	Strong	yw-m	9600	1300
TEST 06	0.3	5	0.5	0.1	Strong	yw-m	9600	1300
TEST 07	0.15	10	0.7	0.1	Strong	yw-m	9600	1300
TEST 08	0.2	2	0.7	0.1	Strong	yw-m	9600	1300
TEST 09	0.1	10	0.5	0.1	Strong	yw-m	9600	1300
TEST 10	0.2	10	0.7	0.1	Strong	li-t	29000	600
TEST 11	0.1	2	0.7	0.2	Strong	yw-m	9600	1300
TEST 12	0.3	5	0.7	0.1	Strong	yw-m	9600	1300
TEST 13	0.1	10	0.9	0.1	Failed	pk-t	90	-
TEST 14	0.15	10	0.7	0.1	Weak	bu-t	115	200
TEST 15	0.1	10	0.7	0.1	Weak	wh-t	260	3920
TEST 16	0.1	10	0.7	0.1	Weak	bk-t	190	1000
TEST 17	0.15	5	0.7	0.1	Weak	bk-t	190	1000
TEST 18	0.15	5	0.7	0.1	Weak	wh-t	260	3920
TEST 19	0.1	2	0.7	0.1	Weak	wh-t	260	3920
TEST 20	0.15	5	0.5	0.1	Weak	wh-t	260	3920

MODEL CONSTRUCTION PROCEDURE

After construction of the foundation layer the first layer of reinforcement is laid on the foundation soil and soil filling by mechanical gravitational soil raining system, was carried out. During construction, a rigid frame is used as partial support at the face of the wall. Using a facing form was essential for achieving identical, good compaction and relative density at the facing. After partial backfilling and folding the reinforcement at the face, the remaining backfill was placed on the overlap length of the reinforcement, up to the next layer elevation. To retain backfill material at the wall face, a textile sheet is used at the face with 10 cm up and down tail and lateral flaps in each layer.

On reaching next layer elevation, the face form was removed and a coloured sand layer was placed at the box wall boundary to differentiate each layer and make deformations visible during shaking. During construction each sensor is laid at its predefined place within the soil mass.

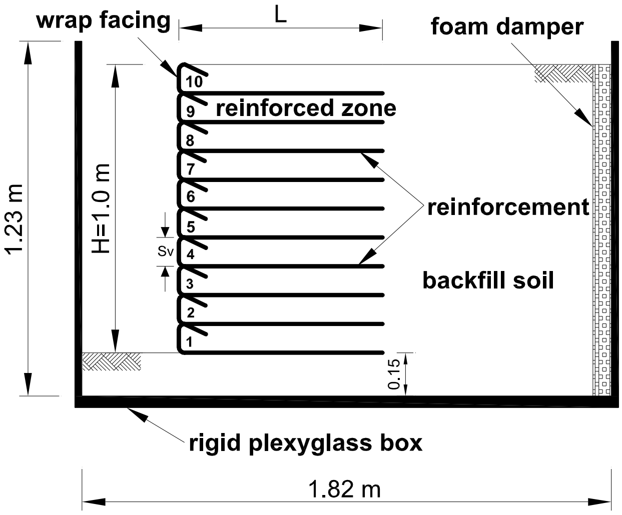


Figure 1. Schematic view of physical models

INSTRUMENTATION

Acceleration and deformations were measured using accelerometers and displacement transducers (LVDT sensors) respectively. Acceleration sensors were laid at predefined positions during incremental construction of the wall. One acceleration sensor was attached at the box base to measure base acceleration. Measurement of facing deformation was achieved using LVDTs attached to a rigid column connected to the box body with a stiff beam. Deformation of the facing was measured at 5 positions. Settlement of wall top at reinforced zone and backfill (in some tests) is measured with vertical LVDTs connected to the box frame with stiff beams. Various types of instrumentation were used for these series of tests, but the position of displacement sensors was identical in all tests. Typical instrument layout is shown in Figure 2.

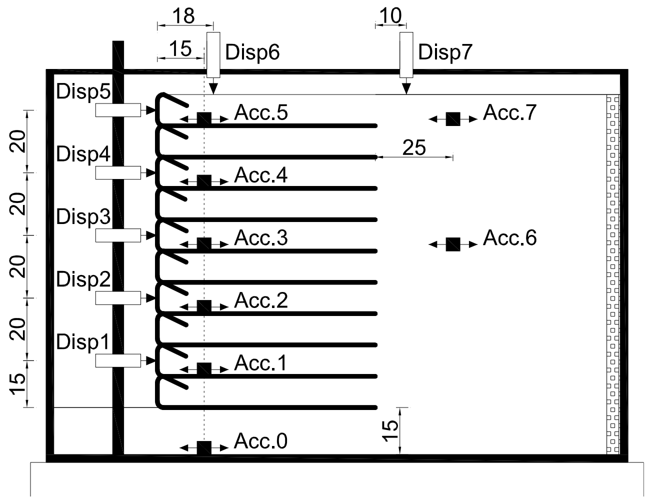


Figure 2. Typical instrumentation

CONSTRUCTION MATERIAL

Two main construction materials used for the physical models were soil and reinforcement.

Soil

Firuzkooh 161 sand is used for construction of reinforced soil wall and backfill soil. Firuzkooh sand’s gradation curve is similar to Toyora sand. Figure 3 and Table 2 illustrate some specifications of this standard soil type. The bulk unit weight was controlled to be constant at approx. 1.5 ton/m^3 at loose state and approx. 1.63 ton/m^3 at dense state. Relative density of the soil at loose state was approx. 47 %, and at denser state was approx. 84%. For these tests repeated experiences of soil pluviation helped to regulate the deposition and it was possible to achieve the target relative density. Considering scale factors between prototype and model, soil used for the model should behaved less stiff than field structures, so target relative density were chosen lower than field values which is consistent with other research.

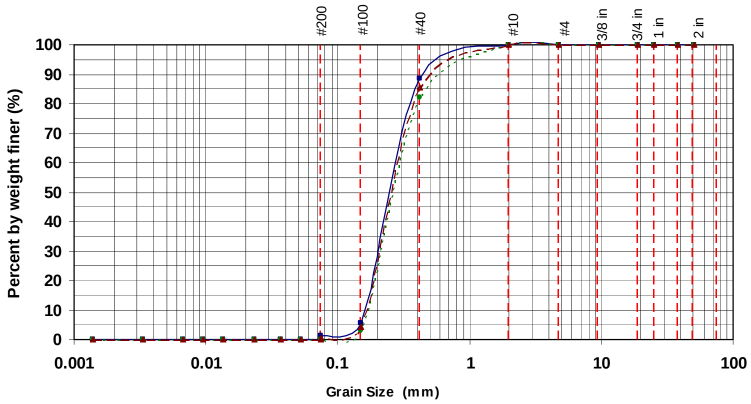


Figure 3. Gradation curve of firuzkooh sand

Table 2. Firuzkooh sand specifications

USCS Name	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	Passing #200 %	Sand %	Cu	Cc	ϕ degree	e_{max}	e_{min}
SP	0.16	0.21	0.27	0.3	1	99	1.87	0.88	40^0	0.874	0.548

Reinforcement

For soil reinforcement, 6 types of traditional textiles and plastic meshes were used. They can be categorized in two series either weak or strong according to their tensile stiffness at 2% strain. These two material categories were selected to observe if there is any detectable difference in wall response due to a large difference in reinforcement stiffness. Tensile strength and unit width stiffness was measured according to ASTM-D 4595 wide width tensile strength standard test method.

Descriptions of strong and weak geosynthetics used in the tests, tensile stiffness and ultimate tensile strength measurements are provided in Table 1. Weak and strong type categorization is based on tensile stiffness. Considering ultimate tensile strength, some weak type reinforcements (with low stiffness at 2%) have higher ultimate tensile strength when compared to some of strong type reinforcements (comparing wh-t type and bu-m type) and vice versa (comparing li-t type and bk-t type).

INPUT MOTIONS

For parametric study and simple interpretation of results and to enable a quantitative comparison, base excitation selected as sinusoidal with constant amplitude. At different tests, physical models are subjected to 2, 5, 8, and 10 Hz frequency input motion. Each model was subjected to sequential different excitations from weak (low amplitude) to strong (high amplitude) peak base acceleration. In total more than 77 harmonic time histories were applied to 20 models. Because of soil densification and deformation experienced by the model after the initial motion, only the results of first model excitation are used and interpreted in this paper.

TEST RESULTS

In this section data of test wall-06 is used and presented as an example for calculation of shear stiffness modulus (G) and damping ratio (D) of reinforced soil walls.

Cyclic behavior of reinforced soil wall

Stiffness & Damping

Cyclic behavior of reinforced soil walls considering shear stiffness (G) and damping ratio (D) is studied in this research. Having approximate values of stiffness and damping ratio for reinforced soil walls, design engineers can develop analytical models correctly and apply variation of these parameters along with variations in the height of the wall and between reinforced zone and backfill. The stiffness term might be considered to include two elements: the very small strain stiffness, which will in many situations control the dynamic response and propagation of waves through the model ground; and the nonlinear medium to large strain deformation properties of the soil (Wood, 2004). The small strain stiffness might be reckoned to be, principally, dependent on the effective stress level, σ, according to a relationship of the form:

$$G \ \alpha \ \sigma^{\alpha} \tag{1}$$

Experimental experience suggests that the exponent α might be of the order of 0.5 for sands and 1 for clays. Evidently a value α = 0 implies that the stiffness is independent of stress level. This research aims to indentify the range of α for reinforced soil walls. Calculation of G and D is done using a similar method of cyclic shear stress ratio calculation used for liquefaction problems. In this method the reinforced soil mass has been divided into horizontal slices, as shown in Figure 4. Inertial force is estimated using acceleration recorded within soil mass slices. Then, using free body equilibrium equations, the calculated sum of inertia forces of upper slices divide by the shear plane area will be equal to shear stress as noted in equations (2) and (3).

$$F_i = \sum m_i a_i = \sum \gamma_s h_i A_i \, a_i \tag{2}$$

$$\tau_{avg} = \frac{F_i}{A_i} = \sum \gamma_s h_i \, a_i \tag{3}$$

Calculation of shear strain is done using displacement records at the wall face. Considering Figure 4, displacement records consist of two components: residual displacement component (estimated as a baseline with 30 to 100 point averaging) and dynamic displacement component, which is elastic vibration around baseline residual component (estimated by subtracting total displacement record from residual component). As is known, the cyclic component is considered an elastic instantaneous response to base shaking, and the permanent component reflects progressive movement of the wall away from the backfill. Average shear strain for each slice is estimated by subtracting dynamic displacement at the top and bottom of each slice divided by the spacing, using equation (4).

$$\gamma_{avg} = \frac{(ddyn_i - ddyn_{i-1})}{h_i} \tag{4}$$

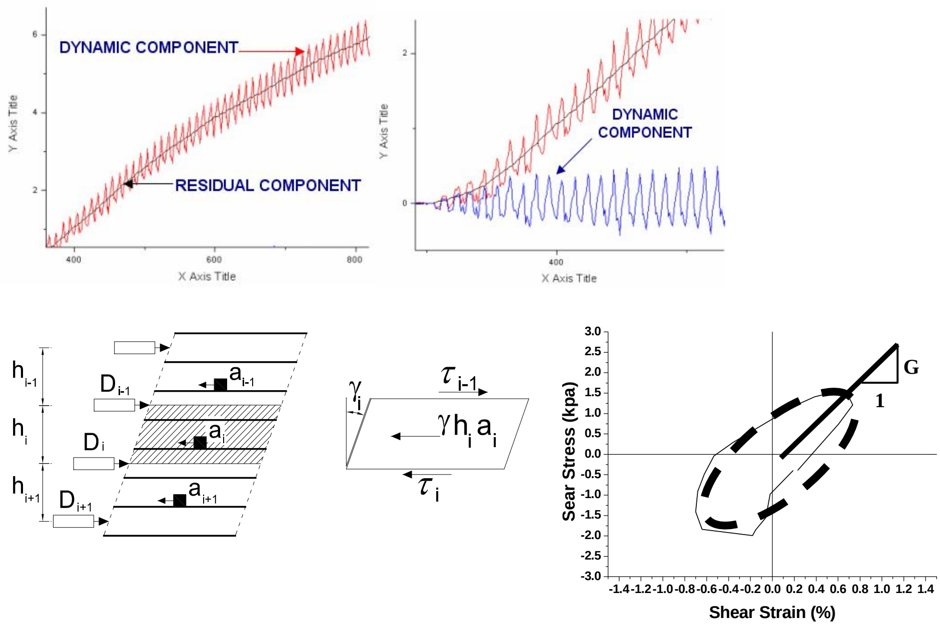


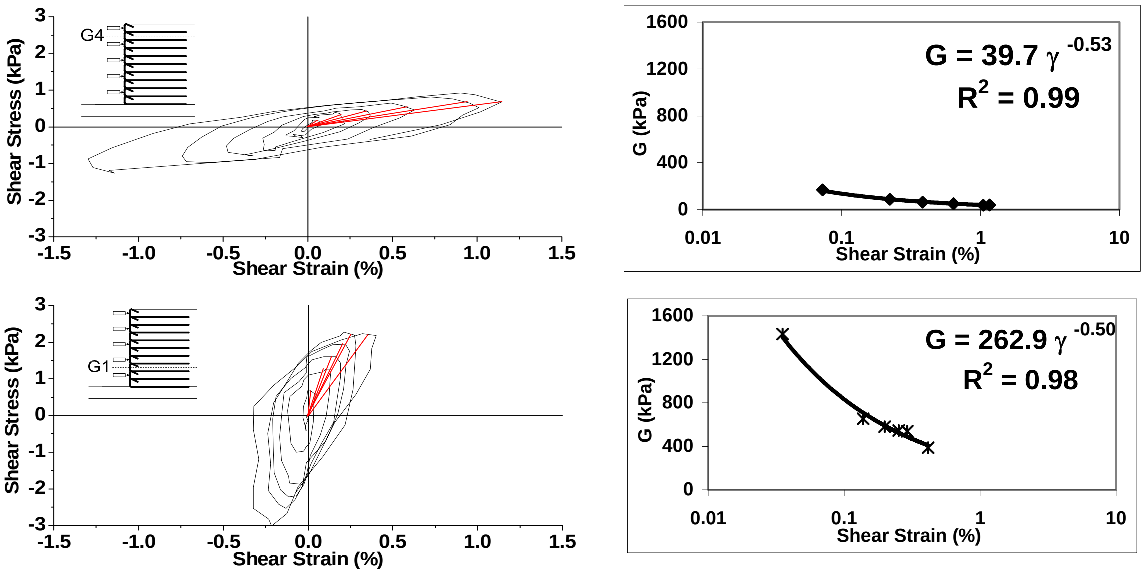
Figure 4. Calculation of stiffness and damping ratio

According to the aforementioned method, shear stress-strain hysteretic loop can be derived as depicted in Fig. 4. Secant shear stiffness modulus of reinforced soil is the inclination of the straight line from root to the point of maximum shear strain and shear stress. Maximum stored elastic energy (W) is equal to the area enclosed under the straight line. The energy loss per cycle (ΔW) is equal to the area enclosed by the hysteresis loop and the damping ratio is derived using W and ΔW as equation (5).

$$D = \frac{1}{4\pi} \left(\frac{\Delta W}{W}\right) \tag{5}$$

Figure 5 depicts the hysteresis loop for two sections of wall 06, (top and bottom). Data from wall 06 is selected for calculation of shear stiffness and damping ratio because this wall has the shortest reinforcement length and was subjected to a base excitation of 5 Hz frequency and 0.35g acceleration amplitude. Therefore this wall tolerated the largest displacements during first cycles of shaking. Therefore, qualitative and quantitative study of cyclic behavior of the wall is more convenient according to the high variation limits and larger shear strains consistent with accuracy of the calculation method. It appears, from Figure 5, secant shear modulus generally increases from the top to the bottom of the wall, because of higher confining stress at deeper parts of reinforced soil body. At each section (with constant confining pressure), shear modulus decreases cycle by cycle with increasing shear strain according to well-known stiffness degradation phenomenon. With available data it is possible to study effect of confining pressure and shear strain on cyclic behavior parameters of reinforced soil walls, simultaneously.

Considering regression equation a best-fit trend line for G-γ data, it is obvious that correlation coefficient is greater than 0.95 for the top and bottom levels of the wall with a power type equation. The power is about 0.5 in both top and bottom of the wall.



(a) **(b)**

Figure 5. Shear stress-strain hysteretic loop and parameters at different height of the wall 06; (a) Shear Stress-Strain loop; (b) variation of G and γ

From Figure 5, it appears that the highest value of shear stiffness belongs to the deeper part of the wall (with maximum confining pressure) at the first cycle (corresponding to the minimum shear strain), and the lowest value of G relates to the top of the wall (with the minimum confining pressure) at the sixth cycle (corresponding to the maximum shear strain).
To omit the effect of confining pressure, if the shear stress is normalized by confining pressure at each level, and shear strain versus normalized shear stiffness (G/σ_v) is plotted, the effect of decreasing shear strain magnitude with stiffness is revealed.

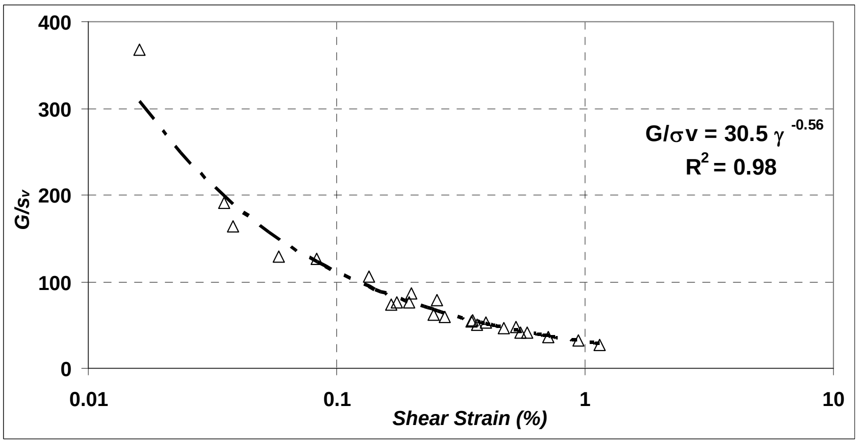


Figure 6. Variation of normalized shear modulus with shear strain

It can be observed from Figure 6 that combined normalized shear stress (G/σ_v) at four levels of wall-06 follow a power equation trend line with correlation coefficient factor equal to 0.98. It means that shear modulus is a function of confining pressure in this 1m height model wall. This sensitivity is expected to be more important at prototype walls with a height of

5m or more. To find the magnitude of σ_v influence on G with a variation of the wall height, relationship between G and γ is assumed as equation (6):

$$G/(\sigma_v)^{\alpha}=A\gamma^{\beta} \tag{6}$$

Then α changed and the best fit coefficient (R^2) in equation (6) calculated and plotted at Fig. 7 along with A and β coefficients.

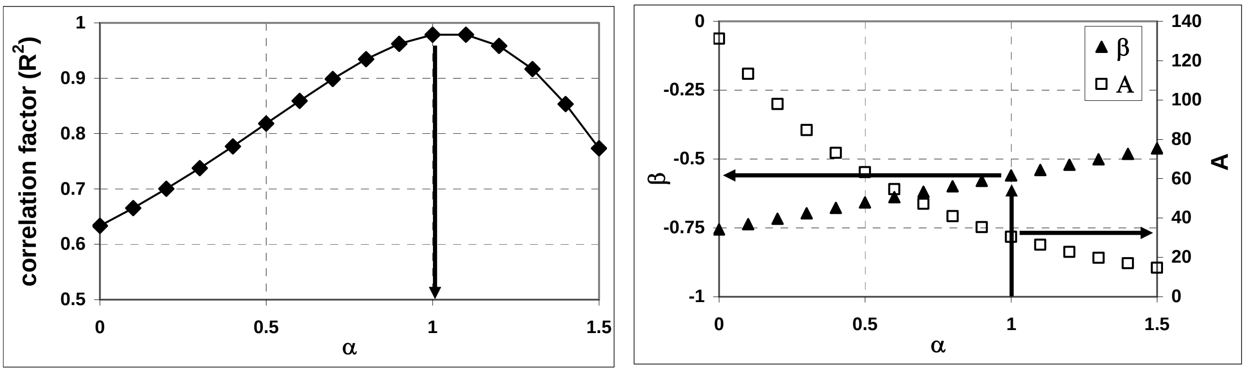


Figure 7. Parameters of exponential equation between G, σ_v and γ

It appears the correlation coefficient reaches its maximum value (0.98) when α equals 1. Hence the best fit with the power equation occurred between G and γ when power of σ is 1, and the parameters of the equation will be A= 30.53 and β=0.56. Therefore it can be concluded that G is sensitive to the confining pressure, not any power of it. Considering Figure 8, the damping ratio is calculated at four levels of the wall height at different cycles, corresponding to various shear strain. Based on instrument sensitivity, measurement method and calculation approximations, small shear strains less than 10^{-3} (0.1 %) are not reliable and should not be relied on qualitatively and specially quantitatively. In order to omit the importance of the low shear strain part of the chart, the x axis is drawn with decimal scale instead of logarithmic.

As observed in Figure 8, damping ratios at reliable shear strains (more than 0.1 %) do not appear to follow a detectable increasing or decreasing trend, changing shear strain and confining pressure. Small region of shear strain available data is the most important cause of this observation. So the main conclusion that can be made with available data is that the average damping ratio is about 20 % of whole wall height at a strain level of about 10^{-3}.

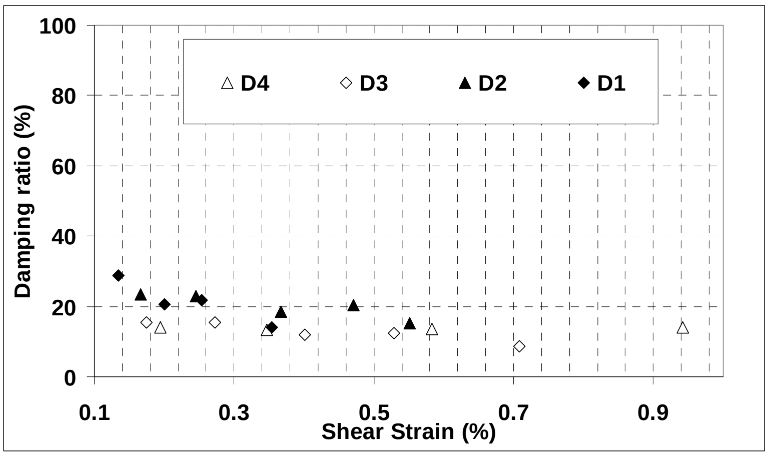


Figure 8. Variation of Damping Ratio with shear strain at different height of wall 06

CONCLUSIONS

Cyclic behaviour parameters of reinforced soil walls including shear stiffness modulus (G) and damping ratio (D) is studied in this paper. Based on the introduced method of calculation, which is approximate, these qualitative and quantitative conclusions are gained:

- Drawing hysteresis loops illustrated that, secant shear modulus generally becomes grater from top to bottom of the wall, because of higher confining stress at deeper parts of reinforced soil body.
- At each level of the wall (with constant confining pressure), shear modulus decreases cycle by cycle with increasing shear strain according to well-known stiffness degradation phenomenon.

- To find the magnitude of the σ_v influence on G variation at the height of the wall, a relationship between G and γ was assumed $G/(\sigma_v)^\alpha=A\gamma^\beta$. A best fit was obtained with a power equation between G and γ when power of σ is 1. This means that a sandy soil body reinforced with geosynthetics has behaved similarly as cohesive soil with corresponding $\alpha=1$.
- Base on the calculation method, damping ratio at reliable shear strains (more than 0.1 %) don't follow a detectable increasing or decreasing manner with shear strain change or confining pressure. The main conclusion reached with available data is estimation of average damping ratio about 20 % at whole wall height at strain level about 10^{-3}.

REFERENCES

Iwasaki,T., Tatsuoka,F. and Takagi,Y. 1978. Shear moduli of sands under cyclic torsional loading, Soils and Foundations, Vo1.18, No.1, pp.39-56.

Koseki,J., Bathurst,R.J., Guler, E., Kuwano,J. Maugeri, M. 2006. Seismic stability of reinforced soil walls. Keynote lecture, 8th IGS conference , Japan, Vol. 1, pp. 51-77 (Koseki 2006).

Wood D. M. 2004. Geotechnical modelling, Version 2.2, electronic copy, pp. 247.