SEISMIC RESISTANT RETAINING WALLS OF REINFORCED SOIL

Ramiro A SOFRONIE[1], Colin A TAYLOR[2] And Paul G GREENING[3]

SUMMARY

The paper presents the experimental results of a comparative study carried out on the Bristol shaking table. The behaviour under seismic loads of two similar models of retaining walls has been investigated. One of them modelled a reinforced soil wall, while the other was a confined wall. The test programme used three types of inputs: sinedwell, Calitri and EC8. Since testing objectives were the settlements, horizontal displacements and induced accelerations at five different levels appropriate recording devices were provided. A total of 50 quakes with gradually increasing intensities were performed: 26 for the gravity model and 24 for the confined one. The tilting phenomenon experienced by prototype retaining walls in the Kobe earthquake also occurred in the shaking table tests. However, the two models behaved essentially differently. While the induced accelerations were much amplified by the gravity model, the confined model showed a strong attenuation. Both conceptual design and further research of practical interest for developing advanced numerical models are supported by the results.

INTRODUCTION

Nowadays retaining walls of reinforced soil are widely spread. They are well defined by national codes like BS 8006-95 and DIN 4017. Design methods are based on comprehensive and reliable computer programs. The construction technologies are also easily adaptable to any site. The number of geogrids used as reinforcement increases with the wall height but remains under 2 daN/m^3 for vertical walls up to 10m, which makes these structures very cost effective. They are self-retaining soil structures [Jones, 1996].

Typical reinforced soil walls are massive structures. Indeed, the ratio between their base lengths *L* and heights *H*, the so-called aspect ratio *L/H*, is usually equal to or higher than 0.6. That means the behaviour of these retaining structures is governed by gravity. Under any combination of loading the resultant of all forces acting on them is always eccentric to their bases. Sometimes, when lateral loads increase and are dominant, the eccentricities exceed the middle third of the base and unequal settlements could occur. Reports from the Kobe earthquake have shown that none of the existing reinforced soil walls collapsed. Most of them developed horizontal displacements, and their facings slightly tilted [Tatsuoka, 1998]. One solution to improve the behaviour of soil structures applied in Japan was to pre-load and pre-stress them without changing their aspect ratios [Ukimura, 1998].

An alternative constructive solution is to confine the reinforced soil retaining walls. The procedure of confining consists of inducing, with the aid of steel cables or membranes of high tensile resistance, forces opposite to the active ones. A typical confining force is applied at or near the top of the reinforced soil structure. It consists of two components: one horizontal, acting as an anchoring force and another one vertical, parallel to structure facing, acting in the gravity direction as an additional compressing force. Its magnitude is chosen such as to counter the eccentricity derived from the equilibrium equation of all forces acting on structure base. There is a unique value of the confining force for a given case of loading. Usually, it takes around 20% of the structure weight, which is not great. When loading fluctuates, non-zero eccentricities accordingly occur. Generally, they

[1] *UNESCO Chair ECOLAND, University of Bucharest, Romania Email:ecoland@ecoland.ro*
[2] *Reader, Earthquake Engineering Reserach Centre, University of Bristol, UK Email:colin.taylor@bristol.ac.uk*
[3] *Research Associate, Earthquake Engineering Research Centre, University of Bristol. UK Email:paul.greening@bristol.ac.uk*

remain small. However, if necessary other more appropriate values for the confining force could be similarly derived. (Sofronie and Feodorov, 1998).

By confining, the aspect ratios of reinforced soil walls was reduced to 0.4. From such rather slender soil structures a better behaviour to seismic actions is expected. The results of analytical models are encouraging and therefore they should be practically validated. In order to draw out realistic conclusions, a comparative test programme was developed. It was adapted to the available facilities of the Earthquake Engineering Research Centre at Bristol University.

MODEL SET-UP

For the two models, an existing shear stack was used. It is a flexible parallelepiped box, composed of aluminium rings alternately connected with rubber elements, specially designed to reproduce the field boundary conditions experienced by soil structures during earthquakes. In order to reduce frictional effects, the inner sides of the stack were lined with a thin polyethylene sheet [Crewe, 1998]. The space used for models was 3880x1000 mm in plan and 900 mm in height. For the facing, facing a wooden solid plate with 20 mm thickness was chosen. It remained free at the bottom and the sides. As reinforcement, Tensar geogrids SR55, with quality control strength of 55 kN/m and specific weight of 0.5 daN/m^2 , were used. The grids were fixed to the facing by nails at five levels equally spaced at 150 mm. For the infill, andesite gravel, with a continuous grading between 2 and 12 mm, specific weight 16.57 kN/m^3 and internal friction 36°, was used. The backfill sand had a specific weight of 15.4 kN/m^3 and friction angle of 45°. Both materials were cohesionless. For the gravity model, the grids were carefully cut at L = 54o mm corresponding to the required aspect ratio of 0.6 (Fig. 1).

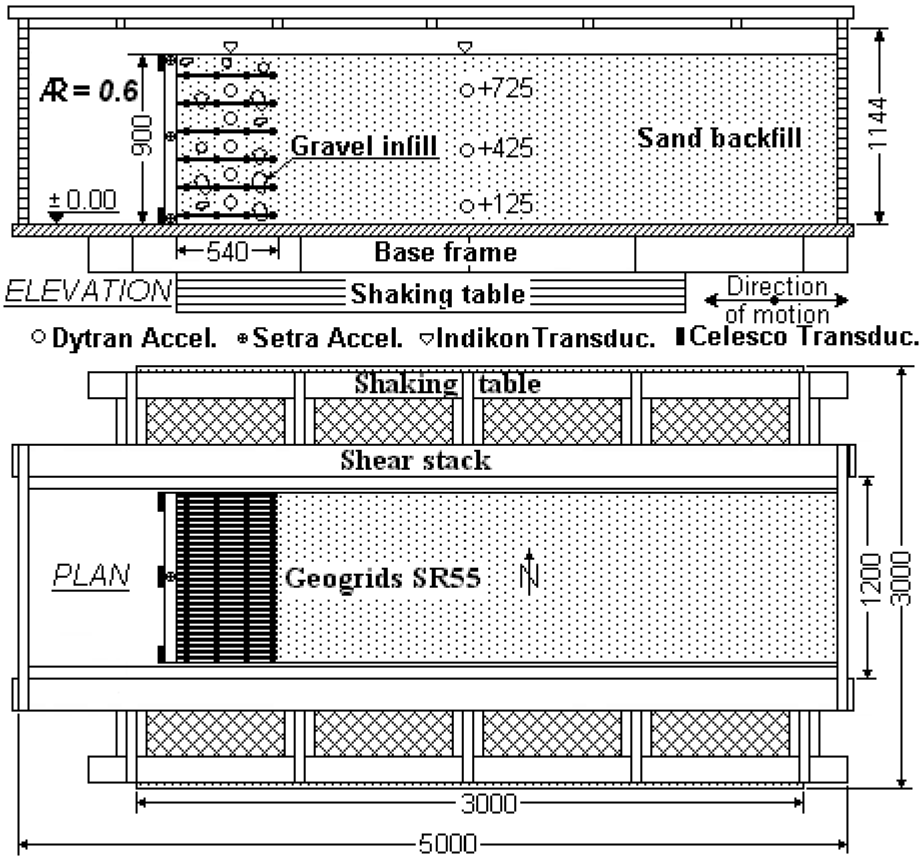


Figure 1: Elevation and plan of gravity model

Since the first model was designed to resist to all the applied quakes, it was accordingly adapted for the second model of the confined wall. After concluding first part of the test programme, the grids were cut to the length corresponding to the aspect ratio of 0.4, namely to L= 360 mm. The confining force was then induced with the aid of two steel cables Φ = 6 mm, each tensioned at 36 daN, and resulting confining degree was 8.9% (Fig. 2).

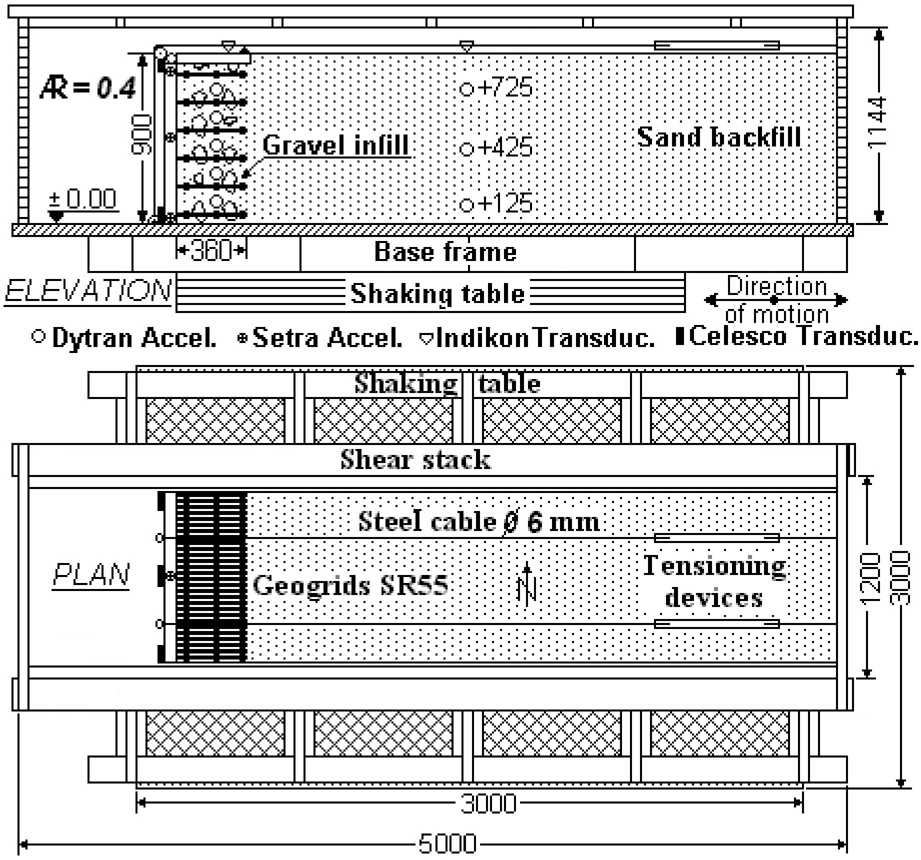


Figure 2: Elevation and plan of the confined model

RECORDING DEVICES

The actual motion of the shaking table and the behaviour of the model incorporated in the shear stack was monitored by 22 channels of instrumentation The settlements of in-fill and back-fill, horizontal displacements of the walls at different positions and the induced accelerations at five typical levels were recorded. For the horizontal and vertical displacements of the gravel and sand deposit surface, 4 INDIKON non-contact magnetic transducers were provided. They did not require a physical connection between the surface to be measured and the instrument itself. For horizontal displacements of the table and facing another 4 CELESCO transducers were installed: one at the top centre and two on the bottom at the north and south extremes. They also enabled measurement of the tilting and swinging of the wall facing. For measuring shaking table accelerations in the three principal directions, 3 unidirectional capacitance based SETRA accelerometers were used. Another 3 similar accelerometers were fixed at the bottom, middle and top of the shear stack. For the accelerations induced in gravel infill and sand backfill, 8 unidirectional piezoelectric accelerometers were placed at different levels: 5 in the infill between the geogrid reinforcements, and 3 in the backfill (Figs. 1 and 2). These miniature accelerometers, with 0.7 mm in diameter and 4.5 g in weight, were used so as to reduce the disturbance to the material behaviour. They were mounted inside small, light perspex boxes and positioned during gravel and sand deposition. A thin layer of sand was glued to each side of the boxes to guarantee a perfect good contact with the adjacent granular material. All accelerometers were connected by thin flexible coaxial cables. The cables were given a special protection during the experimental tests to prevent potential damage induced by the gravel or sand movements.

TESTING PROGRAMME

The six degrees of freedom shaking table at Bristol University was opened in 1987. It has a cast aluminium platform measuring 3m by 3m weighing 30 kN, and has a maximum payload of 150 kN. The platform is driven by eight 50 kN servo-hydraulic actuators, four acting horizontally around the perimeter and four acting vertically at the corners. The platform has peak displacements of ±150 mm in all translation axes, peak velocities of 0.5 m/s and peak bare platform accelerations of 4.8g horizontally and over 7.5g vertically. A PC-based real-time digital control system allows a wide range of recorded and synthetic earthquake and other motions to be reproduced by the shaking table. Up to 64 data acquisition channels are available [Taylor, 1997].

For fulfilling the three objectives of the comparative test programme, a single degree of freedom shake was applied in the longitudinal direction of the models. Three types of input excitations were successively induced: a sinedwell function with a frequency of 5 Hz (Fig. 3), the west-east component of the Calitri acceleration (Fig. 4) and a Eurocode 8 compatible artificial acceleration (Fig. 5). Since the dominant input frequencies were much

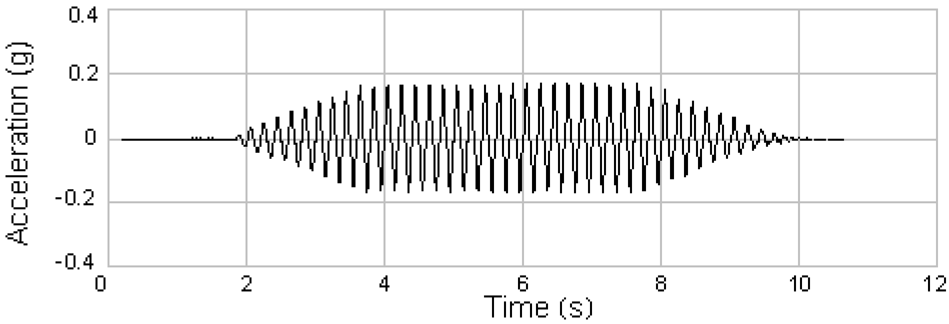


Figure 3: Sinedwell acceleration time history with a frequency of 5Hz

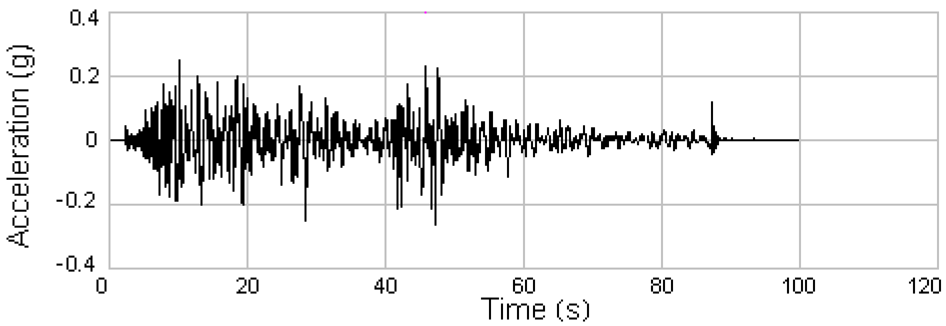


Figure 4: Calitri, Irpinia 1980 acceleration time history

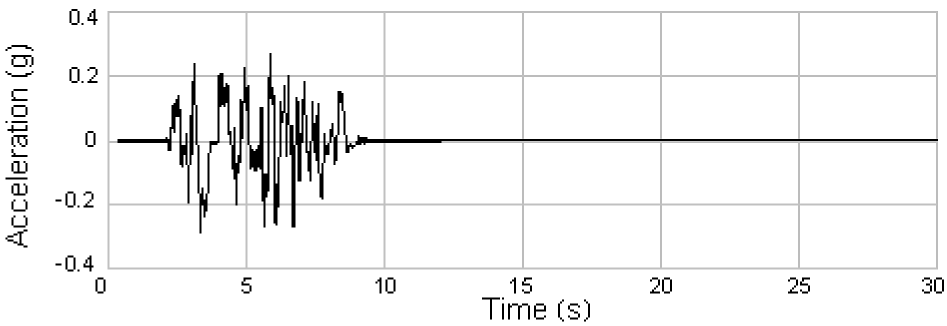


Figure 5: Eurocode 8 acceleration time history

lower than model natural frequency of 30 Hz, there was no danger of resonance occuring in the soil deposit at low levels of table acceleration. The sinedwell quasi-harmonic function has a constant length with increasing amplitude from zero to the maximum value over the first ten cycles, constant amplitude over the subsequent twenty cycles, with the amplitude decreasing down to zero over the last ten cycles. This kind of input motion allows clear investigation of the kinematics of soil structure models and their behaviour at acceleration values close to the limit ones. It was induced with gradual increasing intensities 8 times in the gravity model and 7 times in the confined one. Similarly, the other two types of inputs were applied. The first model was shaken with 26 quakes, while the second one with 24. The end of testing strong sinedwell induced inputs near to 2g but the confined wall did not yield at all. From those 50 tests performed 1100 recordings in real time were obtained.

SETTLEMENTS

At the top of both models, in two identical points located on the longitudinal axis of symmetry at the level +900 mm, the settlements of infill and backfill were considered. They were measured from one test to the subsequent one in a cumulative manner. For each model the different behaviour was observed of the infill and backfill under the three types of inputs with increasing intensities, and then the settlements of the two models were compared.

In the case of the gravity wall, during the first four quakes no vertical displacements occurred. At the test no. 5, when excitation intensity reached 273 gals, a slight settlement less than 2 mm was recorded in backfill. During the test no. 7 and 8, under 384 and 431 gals, critical settlements of 9 mm in the infill and 13 mm in the backfill respectively were recorded. The phenomenon remained rather stabilised until the end of test programme, when the maximum values of 16.3 mm for the infill and 20.3 mm for the backfill showed a satisfactory behaviour for a 780 gals input (Fig. 6).

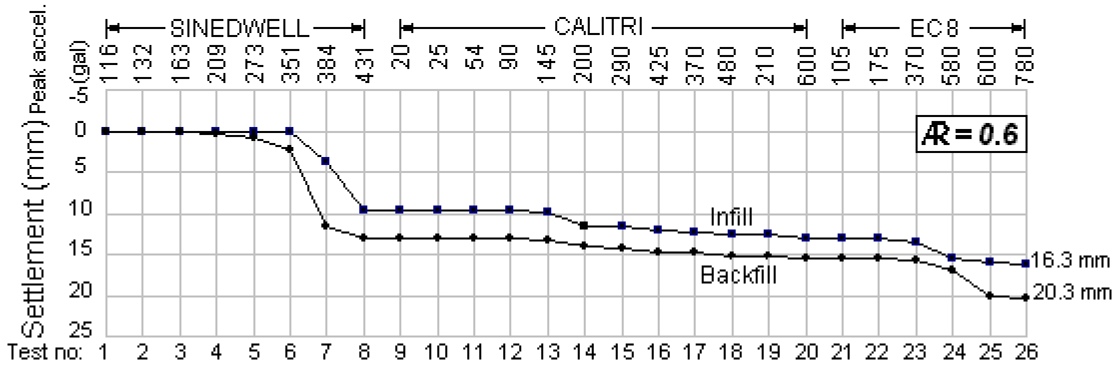


Figure 6: Settlements of the gravity wall

The behaviour of the confined wall in the vertical direction under seismic loads was different. After 19 quakes, with intensities reaching up to 600 gals, the infill settled with only 2 mm. A critical settlement occurred in the infill during test nos. 21 and 22, at 700 and 825 gals, but immediately after, during the next two tests, at 1500 and 1800 gals respectively, the phenomenon became stable and the final settlement remained at the constant value of 22.1 mm. The backfill developed a slight settlement beginning with 6 mm at test no. 7 at 460 gals. At the end of testing it had increased to 11.5 mm (Fig. 7).

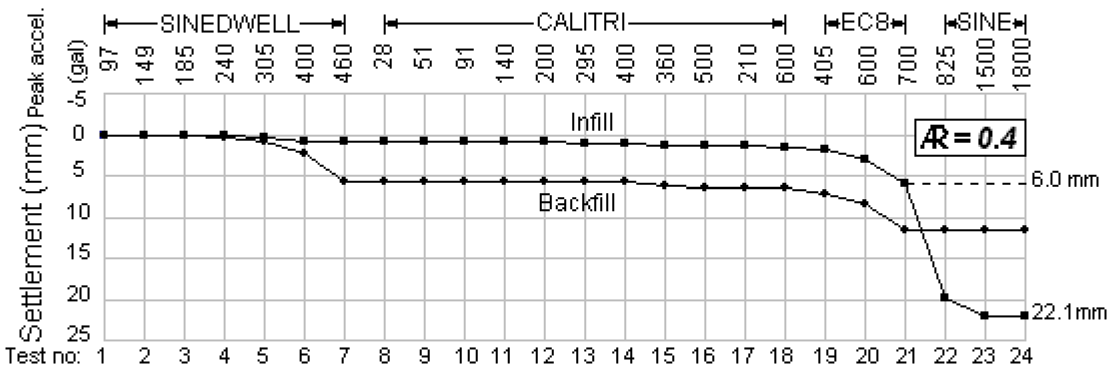


Figure 7: Settlements of the confining wall

Both models behaved well, showing again that retaining walls of reinforced soil submitted to seismic actions are safe structures. The interlocking effect of grid reinforcements explains the better behaviour of infill than that of backfill. The effect of confininment was higher than expected. Stress transfer from gravel to synthetic grids steadily continued under many repeating quakes with increasing intensities. The confining wall settled smoothly and uniformly. The final settlement values were smaller, with 37% for infill and 57.5% for backfill, than those developed by the gravity wall. However, during the last three tests under severe sinedwell excitation the interlocking effect disappeared and the infill gravel ceased to respond like a reinforced soil. Then somewhat larger settlements occurred and only the anchoring components of the confining force prevented the collapse.

HORIZONTAL DISPLACEMENTS

All measured displacements are in the downstream direction of the models, and show the positions taken by the facings after the quakes. In the case of the gravity wall, the top of its facing started to move noticeably during tests no. 7 and 8, under sinedwell excitations of 384 and 431 gals, when the settlements developed also. However, after that, during 16 quakes no additional displacements developed. This suggests a stabilisation phenomenon. Critical displacements occurred during test no. 24 and 25, at 580 and 600 gals, but during the next one, at a higher intensity of 780 gals, the phenomenon stabilised itself at 46.8 mm (Fig. 8).

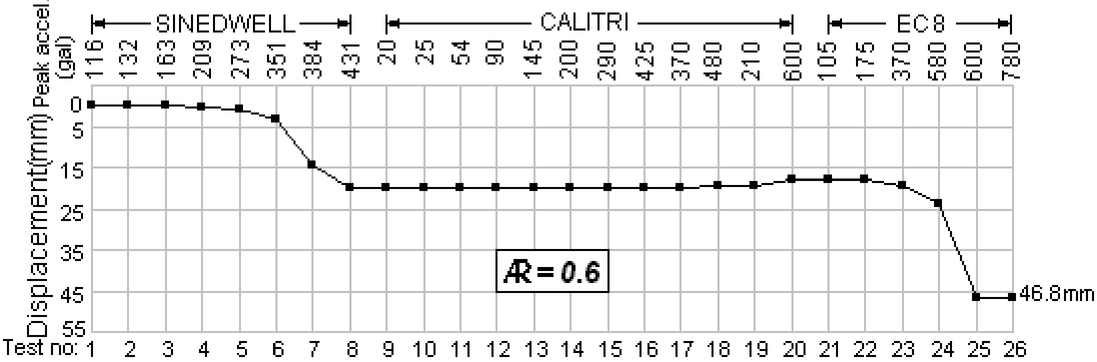


Figure 8: Horizontal displacements at the facing top of gravity wall

The facing of the confined wall remained standing during all successive 20 tests. After test no. 21, at an EC8 excitation of 700 gals, a displacement of 10.5 mm occurred. However, it was only 22.4% of the corresponding displacement of the gravity wall. Only during last the three quakes, in the same conditions, did critical horizontal displacements develop (Fig. 9).

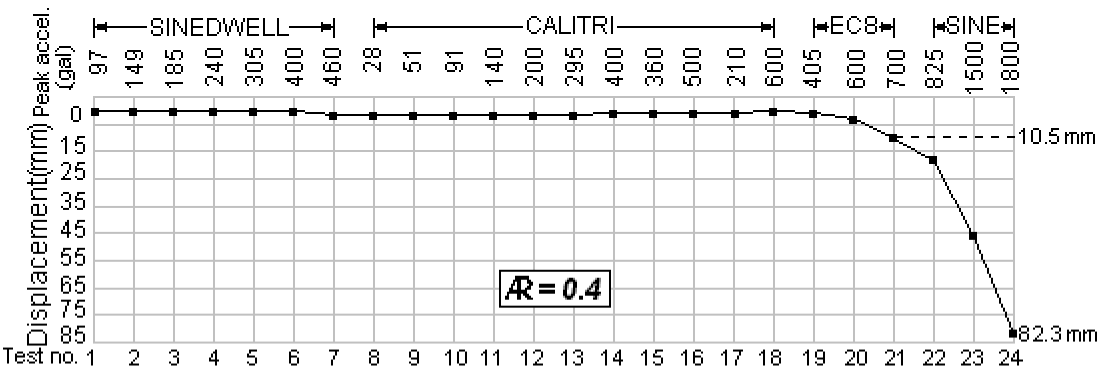
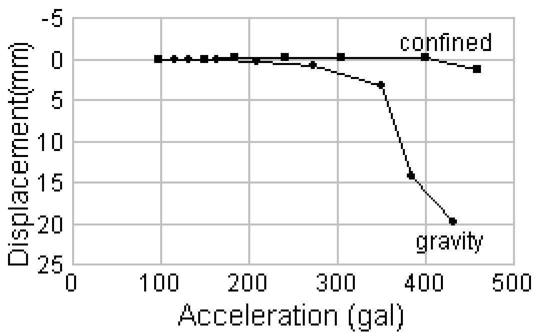


Figure 9: Horizontal displacements at the facing top of confined wall

The difference between the two walls appears clearer when the horizontal displacements are represented in terms of induced acceleration intensity (Fig. 10). The phenomenon of tilting was also distinctly identified (Fig. 11).



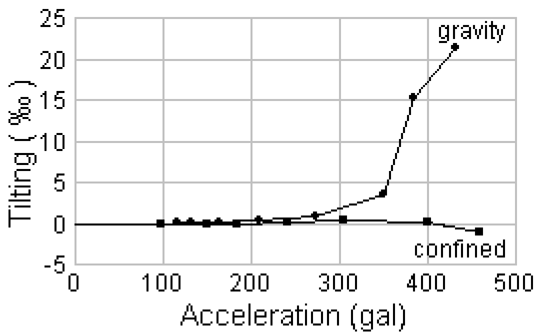


Figure 10: Facing top displacement **Figure 11: Facing tilting**

RESPONSE ACCELERATIONS

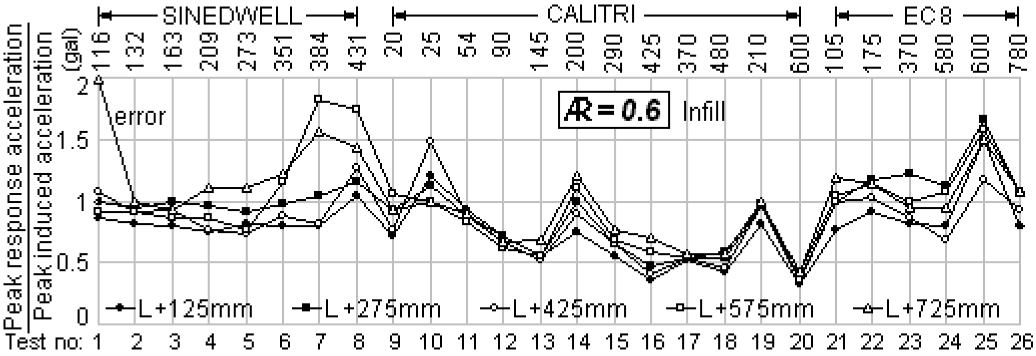


Figure 12: Dynamic behaviour of gravity wall infill

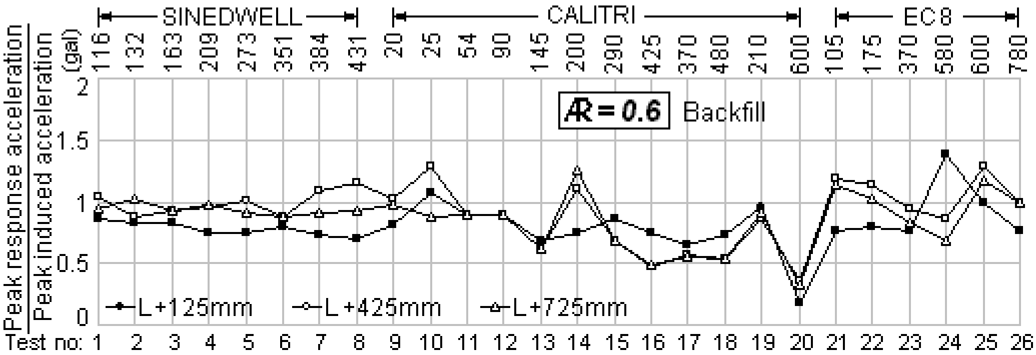


Figure 13: Dynamic behaviour of gravity wall backfill

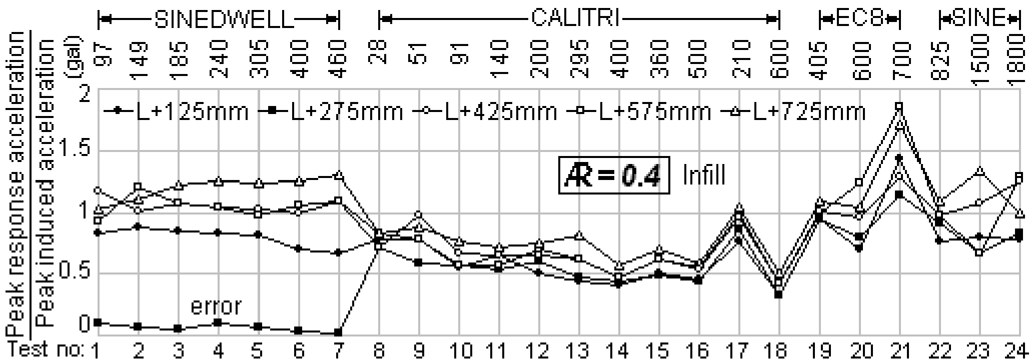


Figure 14: Dynamic behaviour of confined wall infill

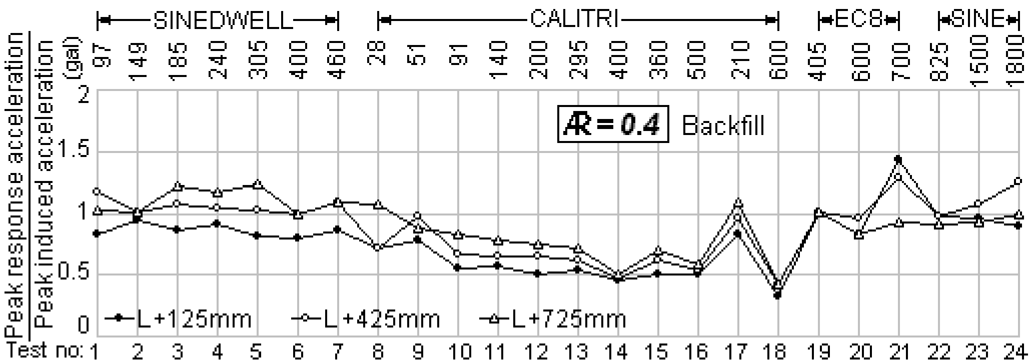


Figure 15: Dynamic behaviour of confined wall backfill

For the dynamic behaviour of the two walls, the ratio between response and induced accelerations was considered. The capacity of retaining soil structures to amplify or attenuate the induced motions was deduced. For both models, the response of the infills was stronger than that of backfill. For usual excitations, the behaviour of confined wall was smoother than that of the gravity wall. The alternation between amplification and attenuation from one test to another was higher for the gravity wall and lower for the confined one, but in both cases showed the role of developing inertial forces (Fig. 12 –15).

CONCLUSIONS

There is little experience in physical modelling of reinforced soil structures. The existing shear stack was of great help in designing the two models. Both were set-up as small full scale models or prototypes with accordingly adapted dimensions. To avoid breakage of reinforcement during testing, the quantity of geogrids was deliberately increased. In this way, from the two limit states of internal stability, only the pullout failure was allowed. However it never occurred, even under the most severe excitations. The two walls were accurately modelled. Each has shown that reinforced soil structures submitted to seismic actions are reliable and safe. The three objectives of the comparative test programme were fulfilled. There is no imminent danger of loss of the external stability of the two types of retaining walls by settling. The model of the gravity wall developed rather large and unequal horizontal displacements, with tilting of the facing as observed during the Kobe earthquake. However, the phenomenon of lateral stability remained under control. The dynamic behaviour of the two models emphasised the capacity of soil structures to amplify or attenuate the induced accelerations. The results are of special interest for improving the existing methods of seismic design. The comparative test study demonstrated the great advantages of confinement. Beside reduction of aspect ratio from 0.6 to 0.4, the confinement created a triaxial state of stress, allowing better behaviour of the granular materials. Under all seismic actions, the confined wall behaved much better than the gravity one. However, the practical techniques of confining should be improved mainly in the vertical direction.

ACKNOWLEDGEMENTS

The financial support of the European Commission DGXII for Science, Research & Development through the Project INCO IC15-CT96-0203 EUROQUAKE is gratefully acknowledged. Thanks are also due to the Tensar International Ltd, based in the UK, for supplying the polymer grids used in the testing programme.

REFERENCES

Crewe, A.J. et al. (1998), “Shaking table tests of scale models of gravity retaining walls”, Proceedings of the Sixth SECED International Conference, Oxford, pp. 187-194.

Jones, C.J.F.P. (1996), Earth reinforcement and soil structures, Thomas Telford, London.

Sofronie, R.A. and Feodorov , V. (1998), “Confining degree of reinforced soil structures”, Proceedings of the XIth Danube-European Conference on Soil Mechanics and Geotechnical Engineering, Poreč, Croatia, pp. 279-282.

Tatsuoka, F. (1998), “Seismic behaviour of geosynthetic-reinforced soil retaining walls”, Geosynthetics Asia’97, Bangalore, India, A.A. Balkema, pp. 137-144.

Taylor, C.A. (1997), Large scale shaking tests of geotechnical structures, ECOEST/PREC8, Report No. 3.

Uchimura, T. et al. (1998), “Preloaded-prestressed geogrid-reinforcementsoil bridge pier”, Proceedings of the Sixth International Conference on Geosynthetics, Atlanta, Georgia, USA, pp. 565-572.