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RETAINING WALLS UNDER SEISMIC ACTIONS: SHAKING TABLE TESTING AND NUMERICAL APPROACHES

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SUMMARY

The results of shaking table tests on a small earth retaining wall are presented, with the aim of performing a comparison between the experimental results, and those obtained by numerical methods based on a sliding block model [Newmark, 1965]. The tests were performed at the Earthquake Engineering Research Centre (EERC) of the University of Bristol, as part of an EU funded programme of joint research between the Department of Civil Engineering of Bristol and the Dipartimento di Ingegneria Geotecnica of the Università di Napoli Federico II, studying various aspects of soil structure interaction. The retaining wall model and the soil deposit were tested within a 4.8 m long, 1.2 m high and 1 m wide flexible shear stack that has been developed by the EERC for testing geotechnical structures. The concrete gravity wall was 90 cm high, and retained a dry Leighton Buzzard sand deposit. The input motions used were typical sinedwell time-histories, with a basic frequency of 5 Hz. Experimental data obtained during two different testing phases are compared with analytical results calculated using the original Newmark model and the Zarrabi-Kashani model [1979]. The comparison confirms the effectiveness of the rigid block model, which reproduces the exact kinematics of the sliding phenomenon. In addition the experimental displacements and those obtained from Zarrabi and Kashani approach show that the analytical method is reliable but still slightly conservative.

INTRODUCTION

The evaluation of the deformations and displacements suffered by earth retaining structures under seismic conditions is a topic of considerable interest with the increasing development of performance based design criteria. In particular, since the original block model sliding on a plane surface was proposed by Newmark in 1965, several methods have been defined for predicting displacements of retaining walls. Such methods evaluate the permanent displacements and/or rotations induced in the structure by means of the numerical integration of the excitation acceleration time-histories [Nadim and Whitman, 1993; Prakash et al., 1994]. Despite the increasing availability of such methods, parallel experimental research on the same topic has not been adequately developed to validate the results obtained from these theoretical models. Nevertheless, advanced and large earthquake facilities have recently allowed effective experimental investigations on even complex geotechnical models [Iai and Sugano, 1999].

This paper outlines an experimental study performed on a prototype earth retaining wall of reduced dimensions, by means of a large earthquake simulator. This research is part of a joint project between the Department of Civil

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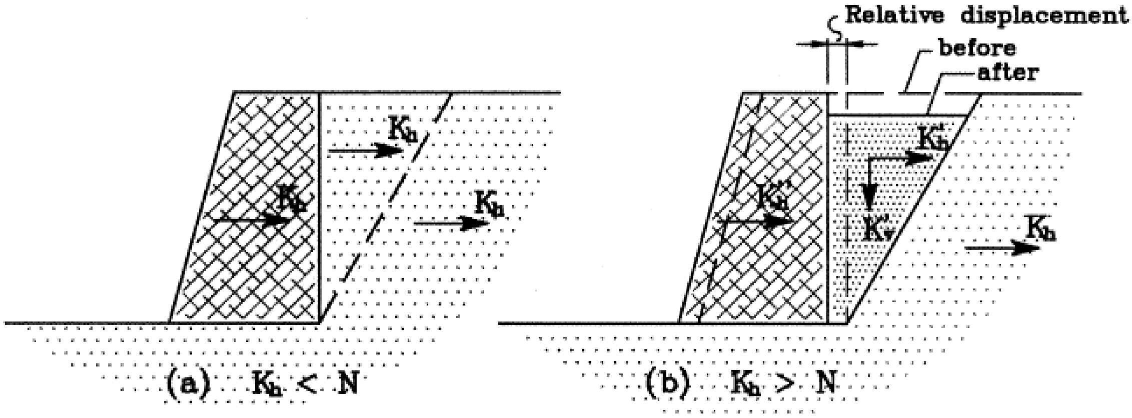

Figure 1: Zarrabi model

Engineering in the University of Bristol and the Dipartimento di Ingegneria Geotecnica of the Università di Napoli Federico II. The project is supported by the European Consortium on Earthquake Shaking Table (ECOEST). One of the main objectives of the research is the experimental validation of the effectiveness of the theoretical approaches previously mentioned, with the final aim of outlining useful recommendations for regulations and codes.

ANALYTICAL MODEL

The displacements of the wall and the ground were computed by the numerical integration of the acceleration excitation time-histories. The behaviour of the wall-soil system was analysed according to the model proposed by Zarrabi in 1979, which is based on the well-known theory of a rigid block sliding on a rough plane surface [Newmark, 1965]. The Newmark theory was first extended to cover the analysis of retaining walls by Richards and Elms [1979], who considered the wall and the soil wedge as a whole rigid sliding block. The peculiarity of the Zarrabi model, which is based on Richard and Elms's work, is that the actual vertical acceleration of the soil wedge is taken into account, even if the vertical component of the ground motion is zero (see Figure 1). The components of the wall, the wedge and the soil accelerations are then computed, assuming that there is no relative horizontal displacement between the wall and the wedge of the soil. Then relative vertical movements between the wall and the wedge are allowed. As a consequence, the active thrust on the wall and hence the limit acceleration value $N \cdot g$ (g is gravity) varies as the wall displacement takes place. It is also worth noting that the Zarrabi model does not allow upward movements of the soil wedge so the direction of any applied acceleration record becomes important with regard to the final wall displacement.

EXPERIMENTAL APPARATUS

The experimental activity was performed in the EERC Laboratory at the University of Bristol. The shaking table at the EERC is a servo-hydraulic actuated device, which operates by means of eight actuators (four vertical and four horizontal) each having a capability of 5 t. The system has six degrees of freedom. The table consists of a cast aluminium seismic platform (3 m x 3 m), capable of carrying a maximum payload of about 15 t. A special attribute of the EERC earthquake simulator is the complete control of the six degrees of freedom of the system by means of the simultaneous action of the eight actuators. The maximum allowable displacement of the actuators, which limits the intensity of the dynamic action applicable at the table at low frequencies, is 30 cm peak to peak. The table has an operating frequency range of 0-200 Hz.

In order to accurately model a soil deposit on the shaking table a large container for the soil-structure system, named a shear stack, has been developed. The shear stack is a flexible rectangular box, 4.80 m long, 1.2 m high and 1 m wide. Its walls consist of spaced rigid rectangular aluminium rings, which are connected by rubber elements on the end walls, while remaining separated along the side walls. The shear stack has been designed to reproduce the field boundary conditions which exist for a soil element during an earthquake [Dar, 1993; Crewe et al., 1995].

PHYSICAL MODEL SET-UP

The physical model was composed of a prototype concrete gravity wall and a backfill of uniform siliceous sand. The wall and backfill were placed inside the shear stack with the wall standing quite close to one of the end walls allowing adequate space for the development of failure planes in the soil deposit. The base of the shear stack, whose length is greater than the side of the table, was connected to the table surface by means of a rigid steel structure. In order to perform plane deformation tests it was necessary to prevent any transverse deformation of the shear stack. To achieve this, two rigid triangular steel frames with sets of linear bearings were placed along the length of the shear stack, down the longer walls, to eliminate any lateral displacement over the whole height of the shear stack. The retaining wall had a trapezoidal shape, and was 40 cm wide by 90 cm high. It was designed to have the sliding safety coefficient value lower than the tilting coefficient. The wall was placed on the base of the shear stack, on the surface of which a thin layer of sand had previously been glued. The measured friction angle between the wall and the base of the shear stack (φb) was 32°. The back face of the wall was greased and covered with a layer of thin rubber in order to avoid friction between the structure and the backfill.

The backfill material was Leighton Buzzard Sand, which is a fine to medium, dry and cohesionless sand. The physical and mechanical properties of this sand have been widely studied by Stroud [1971] and Budhu [1979] at Cambridge University. It is a siliceous and uniform sand; whose specific gravity, and maximum and minimum void ratio values, according to Stroud and Budhu, are reported in Table 1. The sand was placed in 10 cm thick layers, using a deposition procedure that produced a uniform porosity of backfill. The measurements performed both layer by layer, and on the whole deposit, gave an average value of the void ratio e=0.741, which corresponds to a relative density (DR) of 17.5%. The main physical and mechanical characteristics of the sand backfill are listed in Table 1.

Table 1: Main characteristics of backfill sand (Leighton Buzzard Sand)

Specific gravity	2.66
Maximum void ratio emax	0.79
Minimum void ratio emin	0.51
Backfill void ratio e	0.741
Backfill relative density DR	17.5%
Backfill dry unit weight γD	15.3 t/m3
Backfill internal friction angle φ'	43°

A complex network of instruments was used during the testing, with the aim of controlling the actual motion of the table, and continuously and automatically monitoring the behaviour of the shear stack and the wall-soil system (Figure 2).

EXPERIMENTAL TESTING PROGRAMME

At present two experimental testing programmes have been carried out. The main objectives of the testing activity have been the study of the kinematics of the wall, the evaluation of the limit acceleration of the wall-soil system and the measurements of their relative displacements. One of the final aims of the experimental research is the validation of the numerical methods for the evaluation of permanent displacements [Newmark, 1965; Zarrabi-Kashani, 1979]. Regarding the kinematics, unidirectional horizontal excitations along the longitudinal axis of the shear stack have been applied, to produce plane deformation and failure phenomena. The wall was designed to simulate a sliding kinematic model, which generally seems to prevail over tilting movements in the case-histories that have been examined. With regard to the detection of the limit acceleration of the wall-soil system (i.e. the limit value over which the relative movement starts), the table was excited with series of typical input motions with different maximum acceleration values. In order to study the behaviour of the wall, the movements of the base and top of the wall were monitored using two sets of accelerometers and displacement transducers, placed along the horizontal and vertical directions of the shear stack. On the backfill surface two sets of displacement transducers were placed, one quite close to the wall and the other at some distance from it, to monitor the local sand behaviour near the wall and the free field behaviour of the sand.

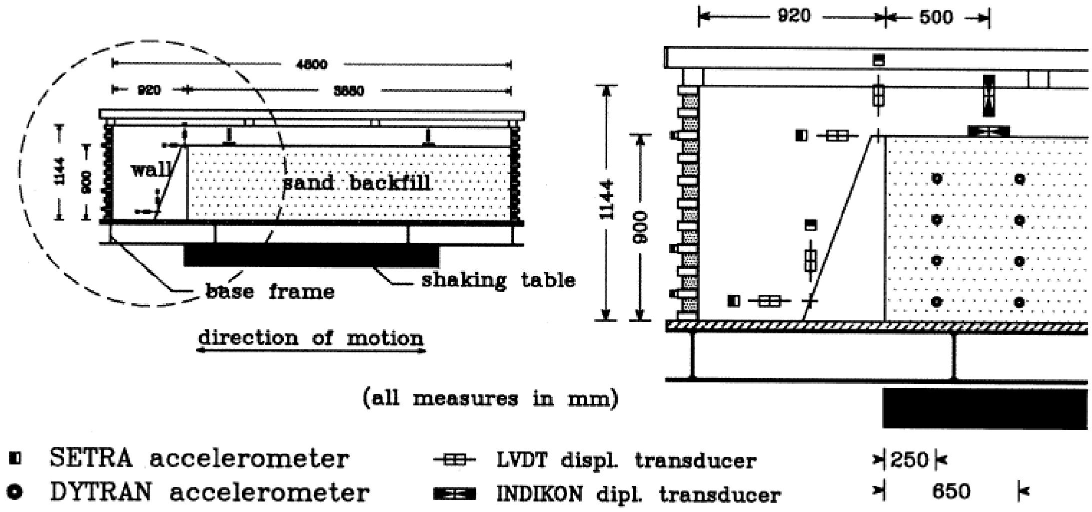

Figure 2: Elevation of the shear stack showing the location of the retaining wall and the associated instrumentation

Three different kinds of unidirectional horizontal motion were chosen for both the experimental testing programmes: Sinedwell functions, the Tolmezzo accelerogram (NS component), and the Calitri accelerogram (WE component).

Sinedwell functions are quasi-harmonic functions, with finite length, constant frequency and variable amplitude. In these tests a 5 Hz signal was adopted, with amplitude increasing from zero to the maximum value over the first ten cycles. The signal then remained at constant amplitude over the subsequent twenty cycles and the amplitude was reduced to zero over the last ten cycles. This kind of input motion allowed clear investigation of the kinematics of the wall-soil system and its behaviour at acceleration values close to the limit value. The Tolmezzo and Calitri acceleration time-histories were also chosen since they are representative of two major Italian earthquakes (Friuli 1976 and Calitri 1980). The natural frequency of the sand deposit without the retaining wall was about 30 Hz, therefore the three motions chosen for these tests did not cause resonance in the soil deposit at low levels of table acceleration.

The tests and main results of the first experimental activity, relative to Sinedwell and Calitri input motions, have already been discussed [Crewe et al. 1998]. As regards the second phase, the results obtained utilising Tolmezzo accelerogram have been illustrated by Carafa et al. [1998]. This paper refers in particular on the results of the Sinedwell tests. Nine runs were performed, with maximum acceleration values of the time-histories increasing from 0.175 g to 0.411 g (see Table 2).

Table 2: Sinedwell tests: main characteristics of wall kinematics

Run	Amax [g]	Dbase [mm]	Dtop [mm]	Δ* [mm]	β** [degree]
1	0.254	0.10	0.71	0.61	0.039
2	0.175	0.00	0.00	0.00	0.000
3	0.192	0.00	0.00	0.00	0.000
4	0.222	0.00	0.16	0.16	0.010
5	0.278	0.50	1.19	0.69	0.044
6	0.394	0.80	1.26	0.46	0.029
7	0.307	1.00	1.41	0.41	0.026
8	0.347	5.00	5.41	0.41	0.026
9	0.411	22.00	22.00	0.00	0.000

* Δ= Dtop-Dbase; ** β=Δ/H; H=height of wall

TEST RESULTS AND DISCUSSION

All the instrumentation responses recorded during the nine runs of Sinedwell series have been accurately analysed. Particular attention was devoted to the displacement and acceleration responses of the wall in order to study effectively the kinematics of the wall-soil system. The permanent displacements measured at the top (Dtop) and the base (Dbase) of the wall, the difference between them (Δ) and the rotation of the wall (β) are reported in Table 2, together with the maximum acceleration value of each run.

The first input motion (Run 1) was larger than expected. The maximum acceleration was 0.254 g, a little higher than the value of the limit acceleration computed by a pseudo-static back-analysis (0.234 g). This value assumes that the friction between the wall and the backfill was equal to zero, the wall-base friction φb was equal to 32° (measured), and the backfill friction angle φ' was equal to 43° (estimated). Nevertheless the recorded behaviour of the wall (Figure 3) showed that the sliding phenomenon did not occur and that the actual limit acceleration of the wall-soil system has not yet been exceeded. In fact the displacement of the base of wall is negligible (see Table 2), while the top displacement (Dtop=0.71 mm) is slightly lower than that required for the backfill failure (approximately 0.1% of the wall height). This evidence was clearly confirmed by the subsequent tests (although the data does show some noise that has a maximum amplitude of about 1 mm peak to peak, at frequencies between 30 and 50 Hz). Run 2 to Run 4 input motions, with maximum acceleration value increasing from 0.175 g to 0.222 g, did not cause any significant movement of the wall. On the other hand, Run 5 input motion, with Amax=0.278 g (slightly higher than Run 1) gave small but significant displacements (Dbase=0.50 mm and Dtop=1.19 mm), due to the sliding of the wall (see Figure 3). In conclusion Run 1 and Run 5 indicate that the actual limit acceleration value lies between their maximum accelerations. In particular N appears to be very close to 0.254 g. The difference between the observed and the computed limit accelerations is quite small ($\approx$ 0.02 g). In our opinion this is mainly due to the slight approximations of the parameter values assumed in the back-analysis; for instance, this difference could be due to some friction (δ) between the wall and the backfill (e.g. assuming δ=6°, N becomes equal to 0.254).

As regards the kinematics, starting from Run 5, the ratio between top and base displacement difference (Δ) and the average displacement of the wall decreases, confirming the prevailing of the sliding kinematics; no significant rotations of the wall were recorded. The comparison between the experimental and the theoretical displacement results is illustrated in Table 3 and Figure 4.

Table 3: Sinedwell tests: Comparison between measured displacements and numerical analysis

Run	Amax [g]	Dbase [mm]	Dtop [mm]	Dave/H %	DNewmark [mm]	DZarrabi [mm]
1	0.254	0.10	0.71	0.04	0.00	0.00
2	0.175	0.00	0.00	0.00	0.00	0.00
3	0.192	0.00	0.00	0.00	0.00	0.00
4	0.222	0.00	0.16	0.01	0.00	0.00
5	0.278	0.50	1.19	0.09	0.45	0.40
6	0.394	0.80	1.26	0.11	1.78	1.64
7	0.307	1.00	1.41	0.14	3.39	2.98
8	0.347	5.00	5.41	0.58	11.59	10.28
9	0.411	22.00	22.00	2.44	30.64	27.55

Dave = (Dbase+Dtop)/2; H=height of wall

In Table 3 the displacements measured at the base and at the top of the wall are reported together with the ratio between the average displacement and the height (H) of the wall. Finally, in the last two columns the displacements computed by the numerical integration of the acceleration time-histories, on the basis of the Newmark and Zarrabi model, are also listed. To be consistent with the experimental data, the limit acceleration factor N=0.254 was adopted in the analyses.

In Figure 4, the displacement time histories relative to the larger input motions (Run 8 and Run 9) are illustrated. The wall moves perfectly in phase with table. The displacements of top of wall contain a Sinedwell component that is significantly larger than that at the bottom, due to the rocking of the wall over the rigid base. Subtracting this 5 Hz component, the agreement between the shapes of measured signals and those computed by Zarrabi model is quite good, the main difference being the size of displacements.

As regards the computed permanent displacements, the results obtained using the Zarrabi method show better agreement with the measured ones; this is certainly due to the peculiarity of this model, which takes into account the effect of vertical displacements of the soil wedge, as shown by Crewe et al. [1998]. In any case, the analytical predictions are slightly conservative, as already confirmed by previous experimental activities [Simonelli et al., 1997; Carafa et al., 1998].

This difference is probably due both to the slight underestimation of the actual limit acceleration, and to the partial densification of the backfill sand caused by the repeated shakings. It could be worthwhile in the future to reduce the number of test motions, or even to arrange an experimental apparatus to reliably assess the density inside the backfill before each test (at present a global density of the sand is estimated on the basis of measured backfill displacements).

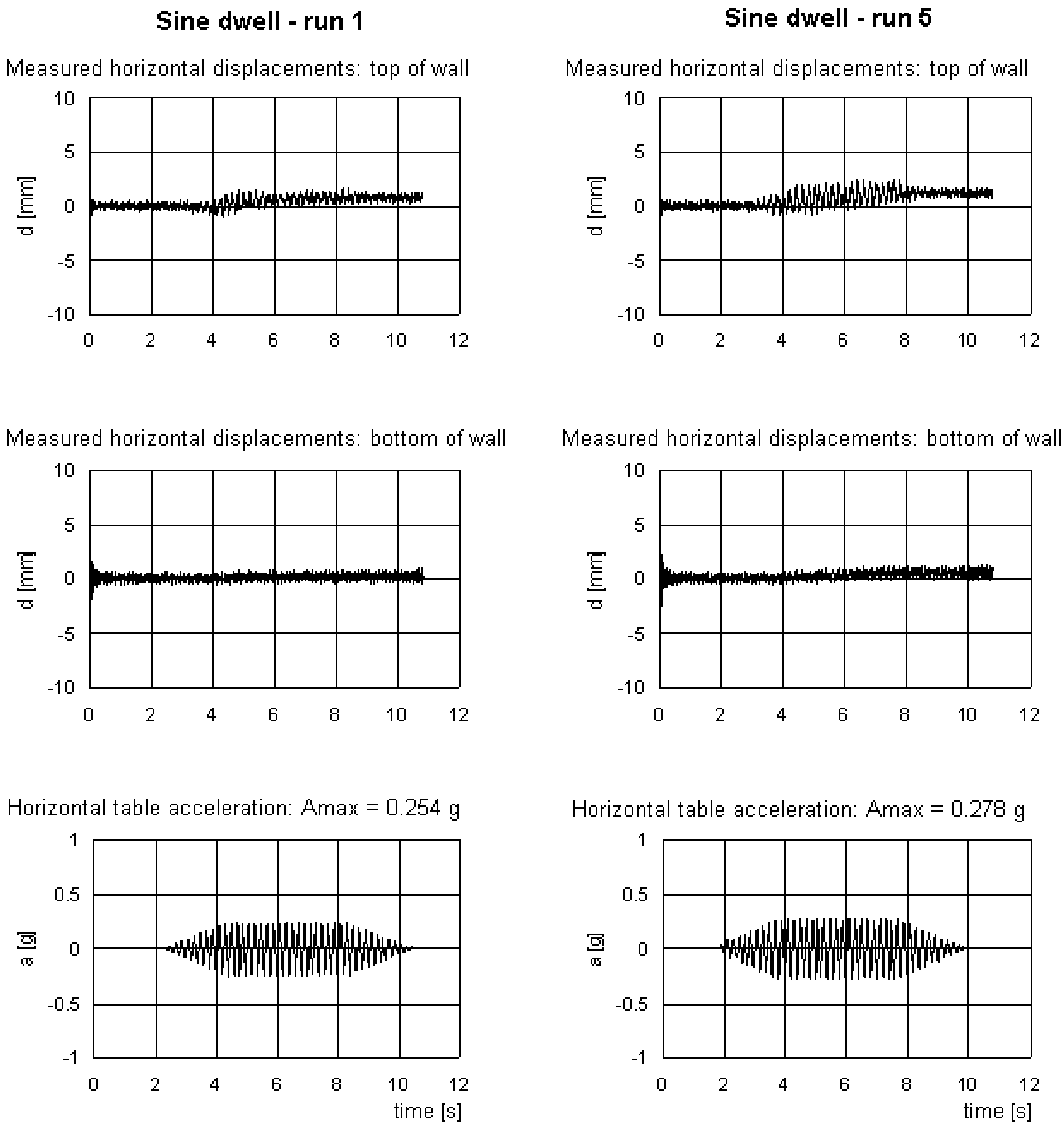

Figure 3: Displacements for table accelerations close to the limit acceleration of the wall-soil system

CONCLUSIONS

An overall comparison between the measured and calculated permanent displacements (Zarrabi method) is shown in Figure 5, in which wall displacements are plotted against the ratio between the maximum acceleration in the test and the computed wall-soil limit acceleration. The agreement is good; in particular when Amax/Ng greater than about 1.3, the difference between the computed and the measured displacements is roughly constant (≈ 5 mm); as a consequence, this difference becomes less significant in percentage in the field of higher displacements, which is of major interest for engineering design (Dave greater than about 0.5% H).

In conclusion, the numerical approaches derived by Newmark model can be considered reliable predicting tools, and could be effectively be utilised inside new performance based design methods.

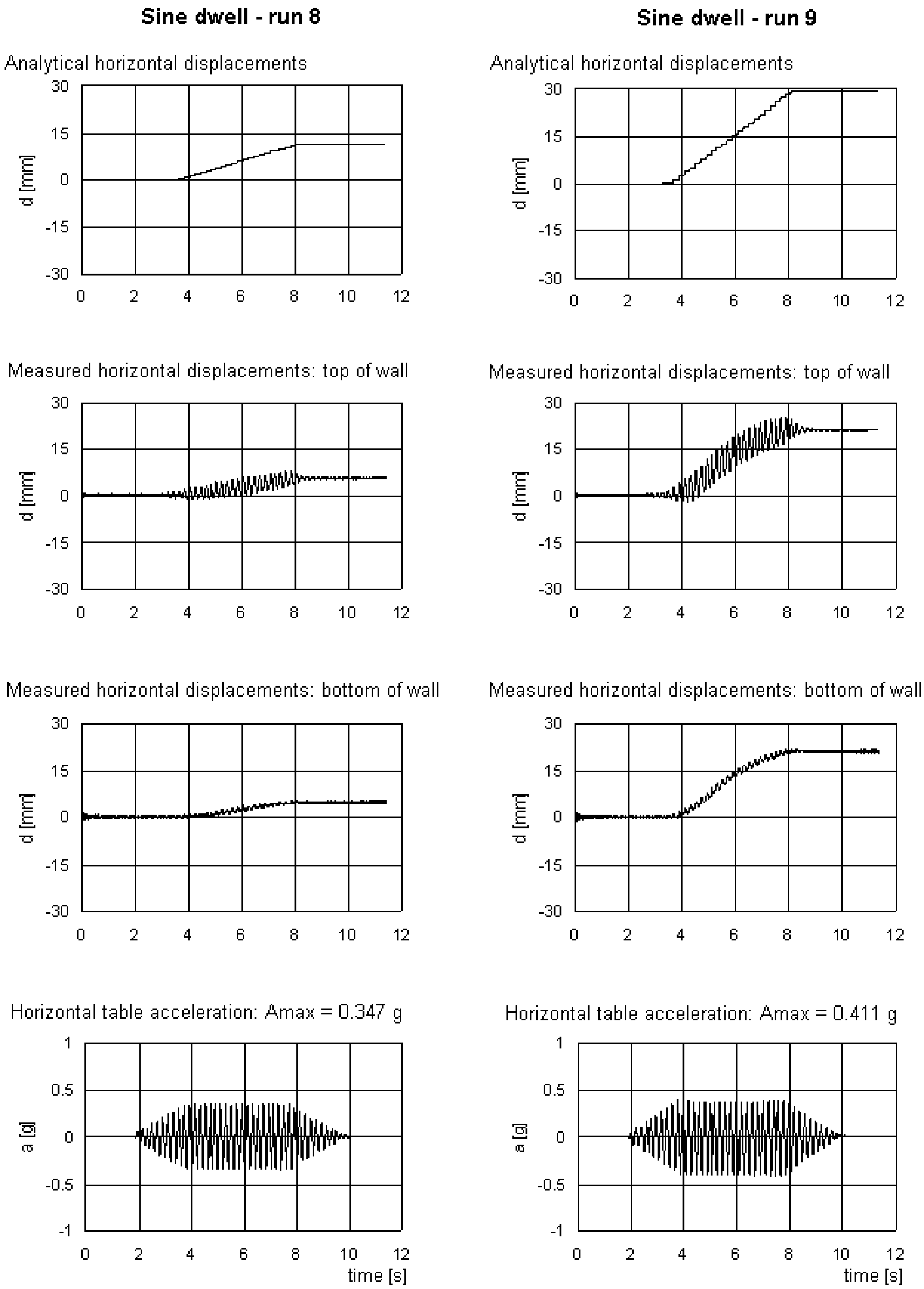

Figure 4: Comparison between the responses generated by the analytical model and the experimental tests

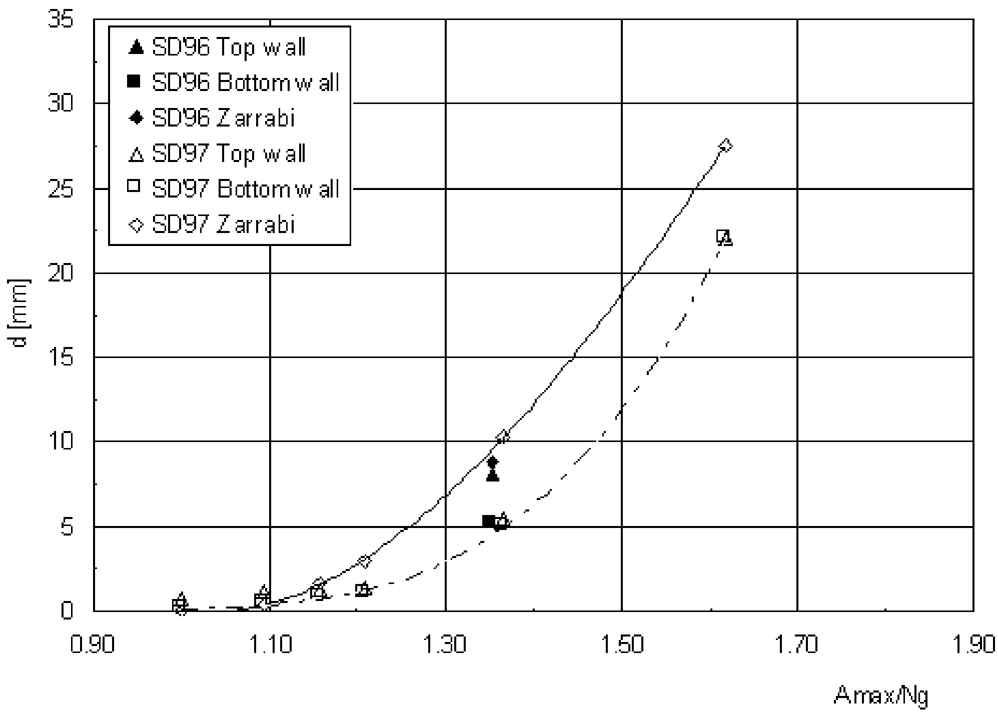

Figure 5: Comparison between measured and Zarrabi computed displacements

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