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Prediction of Lateral Displacements Induced by Liquefaction in the Port of Manzanillo, Mexico During the Earthquake of October 9, 1995

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ABSTRACT

This paper presents the prediction of the lateral displacements due to liquefaction in the yards of the Container Terminal at San Pedrito in the Mexican port of Manzanillo during the earthquake of October 9, 1995 (M_S=7.3, M_W=8.0). It describes briefly the results of field and laboratory tests carried out after the earthquake in order to obtain the required parameters to compute lateral displacements using the Newmark model and a modified Newmark block analysis which takes account of dilatant behavior; the models were calibrated with results of centrifuge experiments. Field data and simplified liquefaction analysis showed that the liquefied fill had a thickness of 13 m and a gentle slope of one degree. From the laboratory tests a yielding shear stress of 3.3 kPa was estimated, and from seismic risk analyses the input used for the prediction of the lateral displacements was a sinusoidal wave of twenty two cycles of constants amplitude of 3.25 m/s^2 at a frequency of 1.4 Hz. Since in the case of Manzanillo the laboratory results did not show evidence of dilatant behavior, the original Newmark sliding block analysis was used and predicted a lateral displacements of 1.95 m, which is in good agreement with the actual movements (slightly higher than 2 m) observed after the earthquake.

INTRODUCTION

The permanent lateral displacement prediction has been a really difficult problem to deal with in geotechnical engineering. Damages caused by lateral displacement due to liquefaction have been important in the last years, therefore, research in this matter has been increasing. Existing methods for predicting lateral displacements can be divided into three categories, empirical, analytical and centrifuge modeling. Regarding the first category, Hamada et al. (1986), carried out a hard compilation of geothechnical and topographical data about observed displacements in uniform sands after the earthquakes of Niigata (1964), San Fernando (1971) and Nihonkai-Chubu (1983). Bartlett and Youd (1995), developed an empirical model to predict the magnitude of lateral displacements induced by liquefaction; however, in some cases these two did not give reasonable results. Analytical methods have been the most used for the prediction of lateral displacements, mainly the Newmark's block analysis (Newmark, 1965); nevertheless, limitations in the last model have come across, for that sake, in this paper it is proposed a modified mathematical model which takes account of dilatant behavior observed in field evidences (Zeghal and Elgamal, 1994), laboratory (Arulmoli et al. 1992), and centrifuge tests (Taboada and Dobry, 1998). To give versatility to the Newmark and modified models, both were included in a computational program, which was calibrated with results of centrifuge experiments carried out by Taboada (1995) at Rensselaer Polytechnic Institute (RPI). The integration of the field evidences, centrifuge modeling, cyclic laboratory testing, and calibration of a model used as a computational tools have allowed us to come closer to the reality, therefore, to complement this work a real case of lateral displacement was analyzed, presented in the Mexican port of Manzanillo during the earthquake of October 9, 1995 with M_w=8.0. After this event, a seismic (Ordaz, 1996), and geotechnical (Ovando et al. 1997) studies were carried out. Field and laboratory tests complemented the data necessary to improve this work. The prediction of lateral displacements for the Manzanillo case was very close to the observed ones. So it was possible to corroborate the applicability of proposed models.

RESULTS OF CENTRIFUGE MODELING

A total of 11 tests in centrifuge were carried out, two of them correspond to models having a horizontal surface and nine on inclined models (Taboada, 1995). The category 1 (α=0°) focused on a horizontal layer of loose sand subjected to a base excitation. Category 2 (α>0°) focused in slopes of loose sand with a light inclination. The soil used in these tests was Nevada sand with a relative density of 40-45% and water was used as saturation fluid. This sand was clean, uniform and fine, with D_{50}= 0.15mm and permeability of k=0.0021 cm/s (Arulmoli et al. 1992). The specific gravity of Nevada sand was 2.68 and maximum and minimum dry weights were 17.33 and 13.87 kN/m^3. The corresponding minimum and maximum void ratios were e_{min}=0.516 and e_{max}=0.894. The centrifuge category 2 represented a prototype thickness of 10 m. The container of the model was a laminar box with an inclination angle at prototype scale of 1.3°, 4.8°, and 10°. The experiments simulate the lateral displacements of a homogeneous, clean, coarse sand layer during earthquakes. The tests were conducted at 50g of centrifuge acceleration. The input excitation was generated during the process by a hydraulic actuator applying a maximum acceleration in prototype units of 0.24g to 0.46g with frequencies of 1 to 2 Hz acting parallel to the laminar box. Instruments were installed to measure excess of pore water pressure, normal and horizontal accelerations and displacements during the excitation. In these tests, four principal parameters were used, to carry out a parametric analysis: (1) field inclination angle α_{field}; (2) maximum acceleration of the input excitation in the base, a_{max}; (3) frequency of the base excitation, f; (4) ground surface permanent lateral displacements at the end of shaking, DH. The results permitted to draw curves showing the relationship between DH and the other three parameters. Herein these curves were used for calibrating the modified analytical sliding block model. From the parametric study some of the following trends were observed:

a)As the slope angle α_{field} increases the pore water pressure and the thickness of the liquefied soil decrease or stay constant; the soil acceleration increases and becomes asymmetric in the liquefied soil; the settlement decreases; and the permanent lateral displacement and shear strains increase.

b)As the input maximum acceleration a_{max} increases the permanent shear strain and the settlement stay constant or increase and the pore pressure, thickness of the liquefied soil, soil acceleration, and permanent lateral displacement definitely increase.

c)As the input frequency f increases the pore pressure, thickness of liquefied soil, soil acceleration, permanent lateral displacement and shear strain, and settlement all decrease. The ground surface lateral displacement DH recorded at a frequency of 1 Hz is 40% larger than at 2 Hz (fig. 7). It is interesting to note that for this case, a Newmark sliding block analysis would predict that DH at a frequency of 1 Hz would be four times higher that at 2 Hz.

In the sloping tests, at large cyclic strains of the order of 1-2%, the soil skeleton tries to dilate and induces an instantaneous reduction in pore pressure and a corresponding increase in soil shear strength. This dilative behavior is similar to that observed in cyclic undrained triaxial loading of small specimens. It shows up typically in the transducer raw data of the centrifuge tests as upslope spikes in the acceleration records and simultaneous drops in the piezometric records; i. e., as the ground moves downslope, the displacement is arrested by dilatancy, which causes a sudden drop of pore pressure accompanied by deacceleration , which temporally arrests the movement of the mass. This dilative response, which becomes stronger as the input acceleration increases and the input frequency decreases and as the slope increases, tends to limit the downslope strain accumulation and thus reduces the permanent lateral displacement DH. Similar acceleration spikes and dilative behavior have been observed in a case history of liquefaction and lateral spreading in the field (Zeghal and Elgamal, 1994).

MODIFIED SLIDING BLOCK MODEL

With all these evidences, we may conclude that Newmark's sliding block analysis shall not be used to evaluate lateral displacements due to liquefaction if the behavior of the soil deposit indicates evidence of dilation. Therefore, a modification to the Newmark sliding block analysis is necessary to account for the increase in soil strength at large strains (due to dilation). Abdoun (1994) presented an engineering method to evaluate lateral displacements based in the sliding block analysis which take into account the soil dilation behavior. The proposed modified block analysis was further modified herein to incorporate the dilative response making use of the simplified shear stress-strain relation shown in Fig. 1

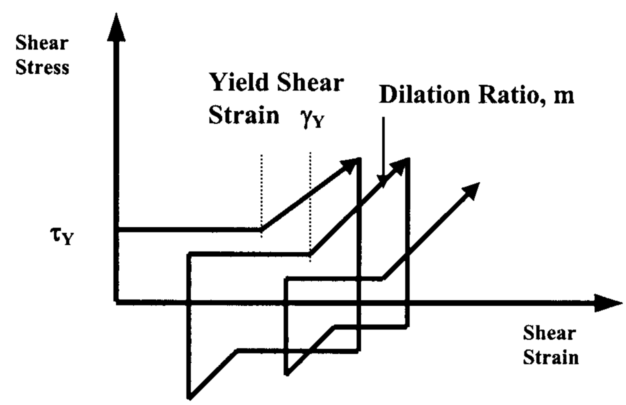

Fig. 1. Simplified shear stress-strain relation used in the modified sliding block model.

Thus, it is proposed an increment of the yielding shear stress, τ_Y, presented during the liquefaction phenomenon of sandy soils when the shearing strain levels exceed a threshold strain, γ_Y, in each excitation cycle, which is defined as the shear strain required in each cycle to trigger the dilative phenomenon; this increment of resistance is idealized by the slope of the shear stress-strain relation schematically indicated in Fig. 2. The dilation ratio (m) and the threshold strain (γ_Y), can be obtained carrying out cyclic undrained tests. If a dilative response is not observed, it would be recommended the use of Newmark's block analysis. To consider the resistance increment due to the dilative phenomenon, the analysis is carried out as follows: when the acceleration of the seismic event exceeds the soil yielding uphill or downhill acceleration, a relative movement occur between the block and the plane of sliding. These relative displacements produce shear strains in each cycle, when the shear strain exceeds the threshold strain, γ_Y, the dilative response is triggered, producing an increment in the yielding shear stress as shown in Fig. 2. From this figure it is defined:

The increment in the yielding shear stress, $\Delta\tau$, produces an increment in the yielding acceleration, Δa, and it is computed with the following expression:

$$\Delta\tau = \rho * \Delta a * Z) \qquad (2)$$

Where ρ is the mass density of the soil. Solving for Δa from Equation (2) and substituting Equation (1), we get:

$$\Delta a = [\Delta\tau/(Z*\rho)] = [m*\Delta\gamma/(Z*\rho)] \qquad (3)$$

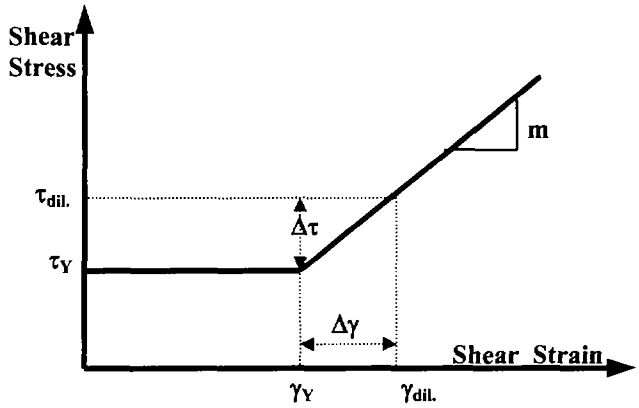

Fig. 2. Concept of the increase in soil strength at large strains (due to dilation).

Yielding acceleration

The condition considered corresponds to the case of a block completely submerged. If this slope is accelerated in the uphill direction with a yielding acceleration a_Y, this will produce an inertia force in the opposite direction of the acceleration, the block will start to move in the downhill direction; the opposit happens when the acceleration is applied downhill. If we carry out a dynamic equilibrium of the block (Fig.3), the next results are obtained:

$$a_Y = [1/(\rho*Z)]*[\tau_Y \pm \gamma' \operatorname{Sin}\alpha] \qquad (4)$$

Where γ' is the submerged unit weight in kN/m^3. If the threshold strain (γ_Y) is exceeded, the increase of the yielding acceleration caused by the dilative response is obtained as:

$$a_{Ydialt.} = \{[1/(\rho*Z)]*[\tau_Y \pm \gamma' \operatorname{Sin}\alpha]\} + [m*\Delta\gamma/(Z*\rho)] \qquad (5)$$

Where, a_{Ydilat} is the yielding acceleration during the dilative stage in m/s^2, τ_Y is the residual resistance in kPa, α is the inclination angle of the slope in degrees (see Fig. 3), m is the dilation ratio in kPa/% and $\Delta\gamma$ is the strain increment after the yield strain γ_Y.

CALIBRATION OF THE MODIFIED MODEL

The modified sliding block model was calibrated with the centrifuge test results. The predictions of the model were computed considering the following necessary input data backfigured from the centrifuge results themselves: liquefied soil layer of 3.5m thickness, yielding shear stress of τ_Y =1.5 kPa (for α=4.8°), threshold shear strain of γ_Y =0.5%, dilation ratio of 20.0 kPa/%. The input shaking applied to the base of the models consists of 22 cycles of acceleration of a single frequency but variable amplitude. In prototype units, the frequency was f = 2 Hz in nine of the tests and f = 1 Hz in two of the tests. The prototype peak accelerations ranged from a_{max} = 0.17g to a_{max} = 0.46g.

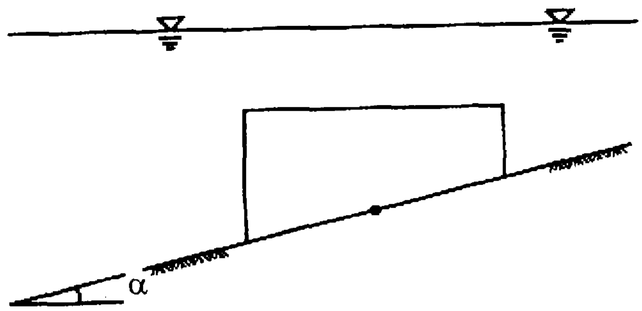

Fig. 3. Model of a submerged sliding block.

The variation of the measured permanent ground displacement DH, with prototype slope angle α_{field} =α, is presented in Fig. 6. In the centrifuge experiments α_{field} was increased from 0° to 10°, while keeping the frequency and maximum amplitude of the input acceleration constant at 2 Hz and about 0.23g, respectively.

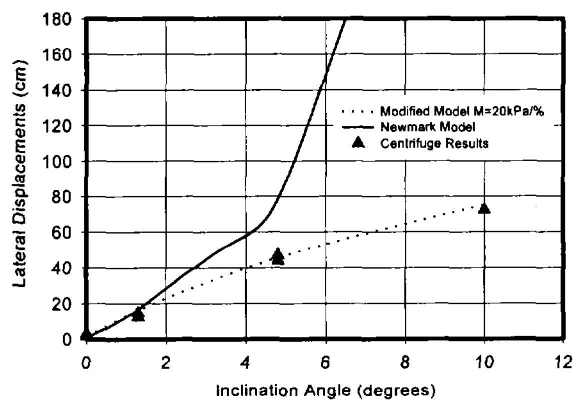

Fig. 6. Influence of the inclination angle(α_{field} =α) in DH.

Figure 7 shows the variation of permanent ground displacement at the end of shaking, DH, with the input frequency for the case where $a_{max} \cong 0.20g$, $\alpha_{field} \cong 5^0$, and number of cycles N=22, as observed in the three centrifuge experiments.

Figure 8 presents three centrifuge tests conducted using a prototype input acceleration frequency of 2 Hz and N=22 cycles, where the slope angle α_{field} *was about* 5^0, and the input peak acceleration was increased from $a_{max} \cong 0.23g$ to $a_{max} \cong 0.46g$.

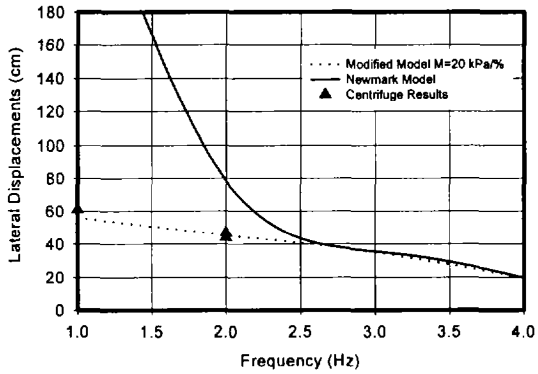

Fig. 7. Influence of the frequency in DH.

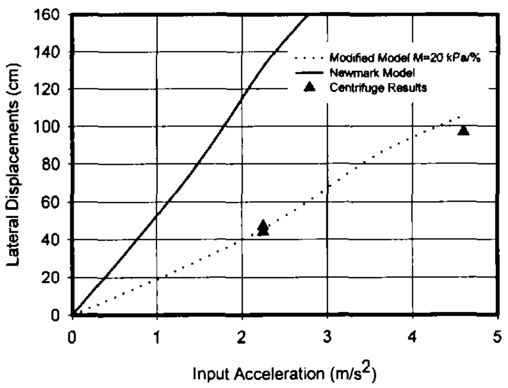

Fig. 8. Influence of the peak acceleration in DH.

The nine centrifuge tests revealed that the variation of DH with slope angle, as well as shaking acceleration and frequency are well predicted by the proposed modified sliding block model. On the other hand, the original sliding block model without dilation significantly overpredicts DH for step slopes, large accelerations and low frequencies.

EARTHQUAKE OF OCTOBER 9, 1995 IN THE MEXICAN PORT OF MANZANILLO.

Manzanillo, the second most important port in Mexico, located in the Pacific Coast, 510 km north of Acapulco and 200 km south of Puerto Vallarta was hit by a large earthquake on October 9, 1995 (M_s=7.3; M_w=8.0). Accelerations produced by the earthquake were recorded in several cities of the state of Jalisco including Puerto Vallarta, Ciudad Guzmán and Guadalajara, where maximum accelerations were 133, 50 and 24 gals. There were no accelerographic records in Manzanillo itself or at the San Pedrito harbor. However a network of several three-component accelerographs existed in the thermoelectric power house 5 km away from the port. One of them was placed in the free field. It recorded a maximum ground horizontal acceleration of 380 gals. The duration of the intense portion of the accelerogram was 50 seconds. Fourier and response spectra show that site effects influenced ground motion at this recording station.

San Pedrito, the main harbor was built on reclaimed land, which liquefied extensively during the earthquake. Materials used for land reclamation were dredged from the neighboring lagoons, typically their depth reaches 10 to 15 m. Settlements in yards produced the rupture of water supply and electricity lines; fissuring, cracking and dislocation of pavements were extensive; some buildings settled differentially and tilted. Differential settlements in the yards of the Container Terminal exceeded 80 cm and lateral displacements in unconfined slopes were more than 2 m. Many of the piles that support the docks suffered structural damage, especially those having short unconfined lengths. In one of docks, nearly 50% of the existing piles were damaged.

STRATIGRAPHICAL MODEL OF THE CONTAINER TERMINAL YARD

Field tests were performed after the earthquake to characterize the sands that underlie the yards of the Container Terminal at San Pedrito. The following stratigraphical model was obtained from the analysis of eight CPT sounding, four SPT tests, seven non standard dynamic penetretation tests and three seismic cone tests (Ovando et al. 2000), they are describe briefly:

Surficial crust. These were formed with materials taken from nearby banks and their thickness varies between 2.5 and 3.5 m. Its upper layer was compacted after placement and has been subjected to further densification by heavy traffic and movement of containers within the yards. The materials there are silty sands (SP or SP-SM) in which the finer particles seldom exceed 7%.

Hydraulic fills. These are the sandy materials that were originally deposited in the lagoon and were then dredged to form the fills; they lie below the surficial crust and reach the elevation -12 m in the container yards, -6.5 in the north side of the terminal and -8.0 m in its south end.

Lake bed muds. These are compressible, low strength, organic clays, CL-OH, that are some times mixed with fine sands. The thicknesses of these deposits vary between 1 and 6 m.

Preconsolidated clays. They form deposits of about 2 m thick and contain small amounts of sand. They were subjected to solar drying during a regression of the sea level, hence their preconsolidated state. Penetration resistance in these clays, CPT soundings, is about 0.7 MPa which can be taken as an index of their relatively high compressibility and low shear strength.

Fluviolacustrine soils. These soils are found at elevations that vary between -11 to -16 m and are formed by a sequence of lenses of compact sands, gravel and shell fragments within a matrix of clayey materials (10 to 15 %). Their CPT strength is more than 10 MPa and SPT resistance is usually higher than 50 blows.

Analysis of liquefaction potential using the field tests indicated that the granular fills would liquefied again if the maximum ground acceleration exceed 0.15 g.

LABORATORY TESTS

Samples from the fills in the Container Terminal were tested at the Instituto de Ingeniería, UNAM, using a computer-controlled triaxial cell. Static undrained triaxial tests performed on reconstituted samples formed by the wet tamping method were used to study their behavior at large strains (Ovando-Shelley et al, 1997).

Materials tested. They are fine to very fine sands in which the fine particles vary between 4 and 12% and occasionally contain small proportions of shell fragments, some of which can be relatively large (2 to 4 mm in diameter). The greater percentage of fragments is formed by quartz but basalt particles, transported from two volcanoes about 150 km west of Manzanillo, are also abundant; laminar micaceous fragments, 10 to 15% in weight, can be seen with the naked eye. Samples retrieved from different depths and locations during the geotechnical exploration program were sieved and their average grain size distribution curve was used to select the materials for the laboratory test specimens. These materials contained 8% of fine particles (liquid limit = 54.4%, plastic limit = 32.6%); those larger than 2 mm, mainly shell fragments, were removed.

Conditions for Triaxial tests. Samples (diameter = 36 mm; height = 81 mm) having a range of initial densities were consolidated isotropically or anisotropically against a backpressure of 250 kPa before shearing them in undrained conditions, in extension or compression; mean effective stresses after consolidation varied between 50 and 300 kPa. The wet tamping method was used to form the samples and saturation was achieved by circulating CO_2 and then water through them; all of the test specimens were provided with lubricated end platens.

Experimental results and their interpretation. The intergranular void ratio, e_g, or intergranular specific volume $V_g = 1 + e_g$, were used as parameters for analyzing the experimental results (Mitchell, 1976). Figure 9 shows the residual strength from the results of compression and extension triaxial tests normalized by the mean effective consolidation stress, and modified by a parameter *B* related with the consolidation stress ratio (Ovando and Pérez, 1997); $B = \sigma'_3/\sigma'_1$ for compression tests, $B=\sigma'_1/\sigma'_3$ for extension tests.

PREDICTION OF LATERAL DISPLACEMENTS

Lateral displacements in the container terminal yards, were calculated using the LASPRED 1D program (Villegas-Rodríguez, 2000), which uses the Newmark model as well as the explained modified model. The input data for the analysis are the following: soil total unit weight of 13 kN/m^3, thickness of the liquefied stratum of 13 m (average height of superficial crust plus hydraulic fills), the transition between the hydraulic fill and the lake bed mud allowed a failure plane with a slope angle of the block of 1°, yielding shear stress of 3.3 kPa, which was obtained from Fig. 9, for a relative density, $D_r \approx 40\%$, $V_g=1.78$, $B x q_{SS}/P'_O \approx 0.016$, $B=\sigma'_3/\sigma'_1 \approx 0.6$, and mean effective stress, p'_0, of 124 kPa. It is important to mention that the model used is the Newmark sliding block because the dilative phenomenon was not presented in laboratory. The applied excitation was a sinusoidal acceleration of 22 cycles (considering a $M_W=8.0$, according to Seed et al. 1975) with a constant amplitude of 3.25 m/s^2, and frequency of 1.4 Hz. The results that the program gave, are shown in Fig. 10. where the predicted lateral displacement was 1.95 m.

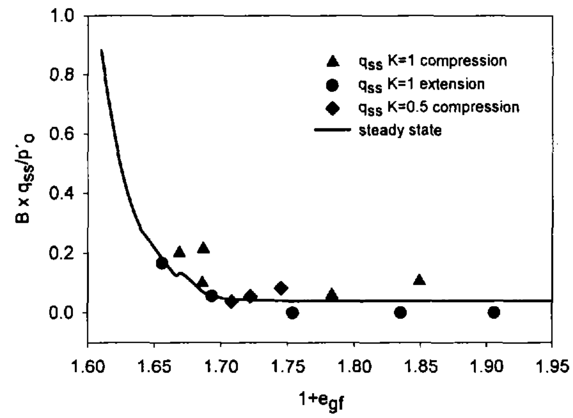

Fig. 9. Residual strength normalized by the mean effective consolidation stress, p'_0 (Ovando et al. 1997).

CONCLUSIONS

The calibration of the models with centrifuge test results allows us to limit the capability of each model. Lateral displacements produced in the "Container Terminal" located in the Mexican port of Manzanillo, due to the earthquake of October 9, 1995, were in excess of 2 m, while the predicted with the Newmark model was 1.95 m, this model was used because the dilative phenomenon was not observed in the laboratory tests. However, should a dilative response would had been observed in the laboratory, assuming a yield strain of 0.4% and a dilation ratio of 10 kPa/%, the predicted displacement would have been 96.5 cm. The results included in this paper clearly show the significant effect that soil dilatancy has on the lateral displacements. Thus, in modeling this phenomenon with a Newmark type approach this aspect should be taken into account to avoid overly conservative estimations of DH values.

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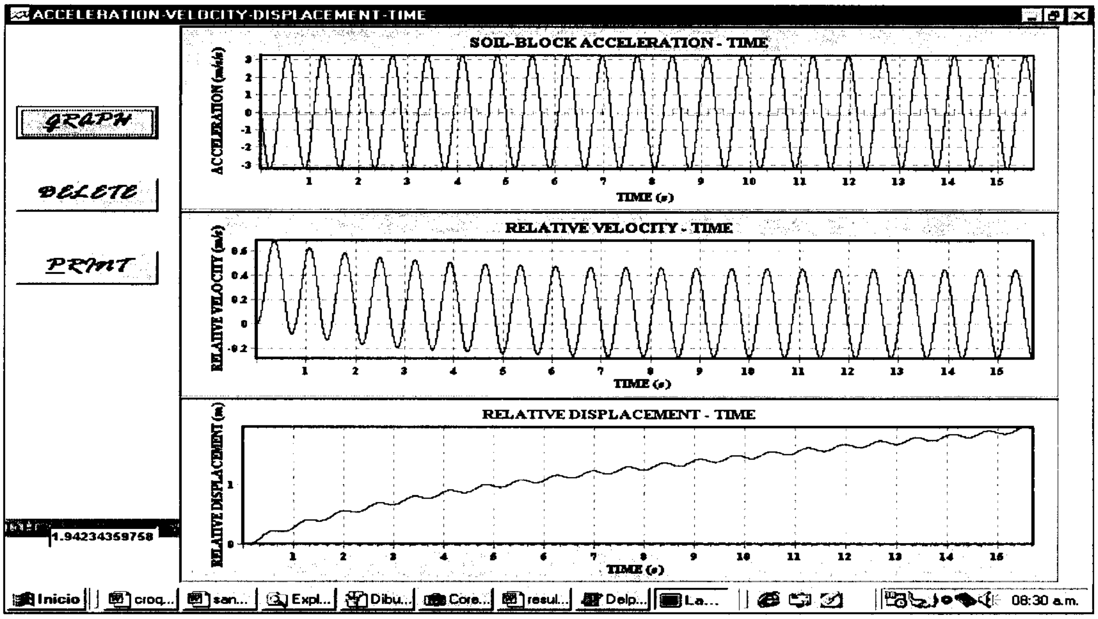

Fig. 8. Results window of the LaSpred 1D program for the Manzanillo case using the Newmark model.

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