17 Apr 2004, 10:30am - 12:30pm

Behavior of a Friction-Piled Box Foundation for an Urban Bridge in Mexico City Clay

Manuel J. Mendoza
National University of Mexico, Mexico City, Mexico

Miguel P. Romo
National University of Mexico, Mexico City, Mexico

Marcos Orozco
National University of Mexico, Mexico City, Mexico

BEHAVIOR OF A FRICTION-PILED BOX FOUNDATION FOR AN URBAN BRIDGE IN MEXICO CITY CLAY

Manuel J. Mendoza
Institute of Engineering
National University of Mexico
04510 Mexico City (Mexico)

Miguel P. Romo
Institute of Engineering
National University of Mexico
04510 Mexico City (Mexico)

Marcos Orozco
Institute of Engineering
National University of Mexico
04510 Mexico City (Mexico)

ABSTRACT

Severe seismic effects and regional ground subsidence are well recognized conditions that should be considered for the design of foundations in Mexico City. These phenomena are magnified by lacustrine clayey soft deposits with very high compressibility and very low shear strength, which are found at the extensive Lake Zone of the city. To clarify the soil-structure interaction phenomena, a prototype foundation was instrumented, selecting a friction-piled support of an urban bridge located in the Metropolitan area of Mexico City. Soil-raft contact pressures, loads on selected piles, and pore water pressures in the subsoil below the foundation have been monitored continuously, since the beginning of the foundation construction eight years ago. The long-term performance of the foundation is known in terms of these state variables. The response of the soil-foundation system has been recorded during the occurrence of ten mild to strong earthquakes. Time-records of those geotechnical variables were obtained at the very moment of the earthquakes, additionally to the accelerograms. This case history yielded valuable information about the foundation performance before, during and after seismic events, regarding the analysis, design and regulations for this kind of foundation.

INTRODUCTION

The design of safe and cost-efficient foundations for medium-to-high-rise buildings or urban bridges, located within the clayey zone of the Metropolitan area, is a geotechnical challenge (Zeevaert, 1972). Friction piles have been a common solution to the foundation of these structures in Mexico City. The piles are capped by a thick raft, or a structural box with stiff foundation beams of reinforced concrete, the latter with the purpose of compensating partially or fully the weight of the structure. Although this mixed type of foundation might be a convenient solution, is vulnerable to earthquake loads particularly when high static pressures are allowed at the raft-soil contact. In fact, in a geotechnical analysis carried out after the 1985 Michoacán strong earthquake (M=8.1) was concluded that this type of foundation had the worst seismic behavior from those used for different purposes in the city (Mendoza and Auvinet, 1988). This piece of information pointed out that the profession should improve the knowledge on pile-soil-box load-transfer mechanisms. So, the goal was to monitor the foundation state variables to better our knowledge on the performance of this type of foundation under static and dynamic loading.

Soil-raft contact pressures, loads on selected concrete piles, and pore water pressures in the subsoil below the foundation have been monitored ever since the beginning of the foundation construction until now, seven years after the bridge was opened to traffic. A peculiar characteristic of this study is the measurement of the internal geotechnical variables during ground shaking caused by mild and strong earthquakes, for which an automatic recording system was implemented. It is the first experience in Mexico with this objective, and possibly in the world, for a foundation prototype with friction piles. That system has a master-slave array controlled by any of the horizontal components of the strong motion instruments that are on the top slab of the foundation, and at free field, on the ground surface and in a deep hole. During bridge operation, the response of the soil-foundation system has been recorded during the occurrence of ten significant seismic events. Measuring the relevant internal variables of this case history, we have learned how the piles and the foundation raft share the imposed loads, through mechanisms of interaction among the foundation elements, both in a long-term basis and during seismic events. Valuable information is discussed in this paper about the foundation behavior before, during and after seismic events. Previous results have been published elsewhere (Mendoza and Romo, 1998; Mendoza et al., 2000).

SUBSOIL, FOUNDATION AND INSTRUMENTATION CHARACTERISTICS

Three geotechnical zones are recognized in Mexico City (Marsal and Mazari, 1969). The Hill Zone (I) is characterized by well cemented pumice-type tuffs and dense sandy soils in the western part of the city, and basaltic lava flows to the south. Lake Zone (III) comprises very soft clayey deposits interbedded with thin lenses of sand or volcanic glass; these soils correspond to materials found at the bottom of a former lake. Transition Zone (II) is sandwiched by the two above and is characterized by abrupt stratigraphical changes.

Site conditions

The site of interest is located in the north-eastern virgin portion of the Lake Zone, characterized by thick deposits of very soft lacustrine clay, interbedded with silty-sand lenses. After a shallow superficial dry crust that was removed by the excavation, young and normally consolidated clays, with a mean value of 346% for the natural water content show up. The mean undrained shear strength of the upper clay formation (UCF), for the top 30 m, was only 12.6 kPa, determined by UU triaxial tests and CPT, Fig. 1. Its reduced stiffness was verified by field determinations of shear wave velocities by means of the P-S logging system; a mean value of 35.5 m/s was measured. Underlying the UCF lays a layer of lightly cemented silty sand, 2 m-thick, locally known as the first hard layer (FHL). It is followed by the lower clay formation (LCF) and then, from 52 m-depth, a harder formation of cemented silty sand with gravels, called the deep deposits, is found.

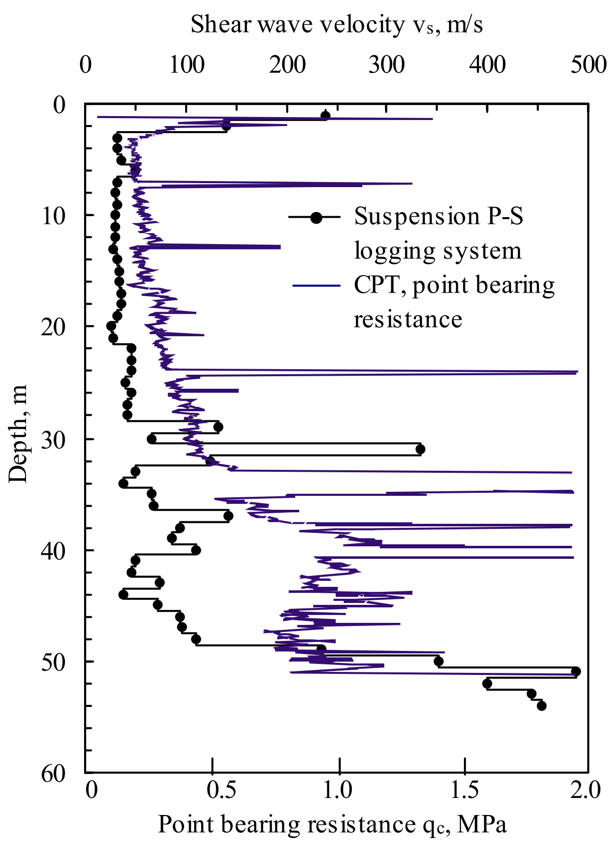

Fig. 1. CPT and shear wave velocity profiles.

Instrumented prototype

The profile and the plant of the instrumented foundation are depicted in Fig. 2. It consists of a reinforced concrete hollow box, 15 m wide, 22 m long and 3 m deep. It has a rhomboidal shape in plan, according with the oblique–angled intersection of the merging avenues. 77 precast square piles (0.5 m of side) were driven (no prebored) before the excavation for the box foundation, and coinciding with the foundation beams. Their tip reaches a depth of 27 m and they work primarily by friction; about 3 m clay cushion was kept to guarantee enough space for future foundation settlements, mainly due to ground subsidence induced by aquifer water withdrawal. This box supports eight columns in two axes that carry the deck-supporting box girders of the bridge; these elements in turn, carry the central prestressed concrete box girders in both sides of this Support No. 6, under freely supported condition.

Description of the instrumentation

The instrumentation consists of eight pressure cells distributed in the raft-soil interface, six piezometers laid down to different depths below the foundation, and thirteen load cells integrated to seven piles. The location of the instrumented piles within the foundation is depicted in Fig. 2. Load cells in piles were integrated to them near (2.0 m) the union with the foundation beams. In two of them three more cells were included to other depths. The objective in this case was to measure the axial force transmitted to these piles, and to know how they transfer the load to the surrounding soil through the pile shaft and tip base. The loading element in load cells is a thick-walled, high-strength steel cylinder instrumented with strain-gauges. Each cell was integrated to segments of the pile by placing them between steel plates, Fig. 3, that were fixed to each other by high-strength steel bolts, providing some prestress. The supporting plates were then welded to the reinforcement steel of the pile segments, assuring their structural continuity.

Portable readout apparatus were used during construction, and now under operation, to record the geotechnical variables under sustained loads. Vibrating wire and electric resistive-type strain gages sensors were involved. A digital triaxial accelerograph was attached to the box foundation. Additionally, two triaxial accelerographs were installed later. One is located on the ground surface in free field, 60 m far from the foundation. The other one lays down in a 60 m-deep well in the Deep Deposits, precisely in the vertical of the surface sensors. Fifteen of the resistive-type transducers, eight load cells, four pressure cells and three piezometers were continuously monitored, in order to record the effect of earthquakes. Five 16-bit digital recorders were implemented for this task, controlled by the accelerograph which triggers the system when an acceleration threshold is reached. The master-slaves array is shown in Fig. 4. A rate of 250 samples/second is used for recording each channel.

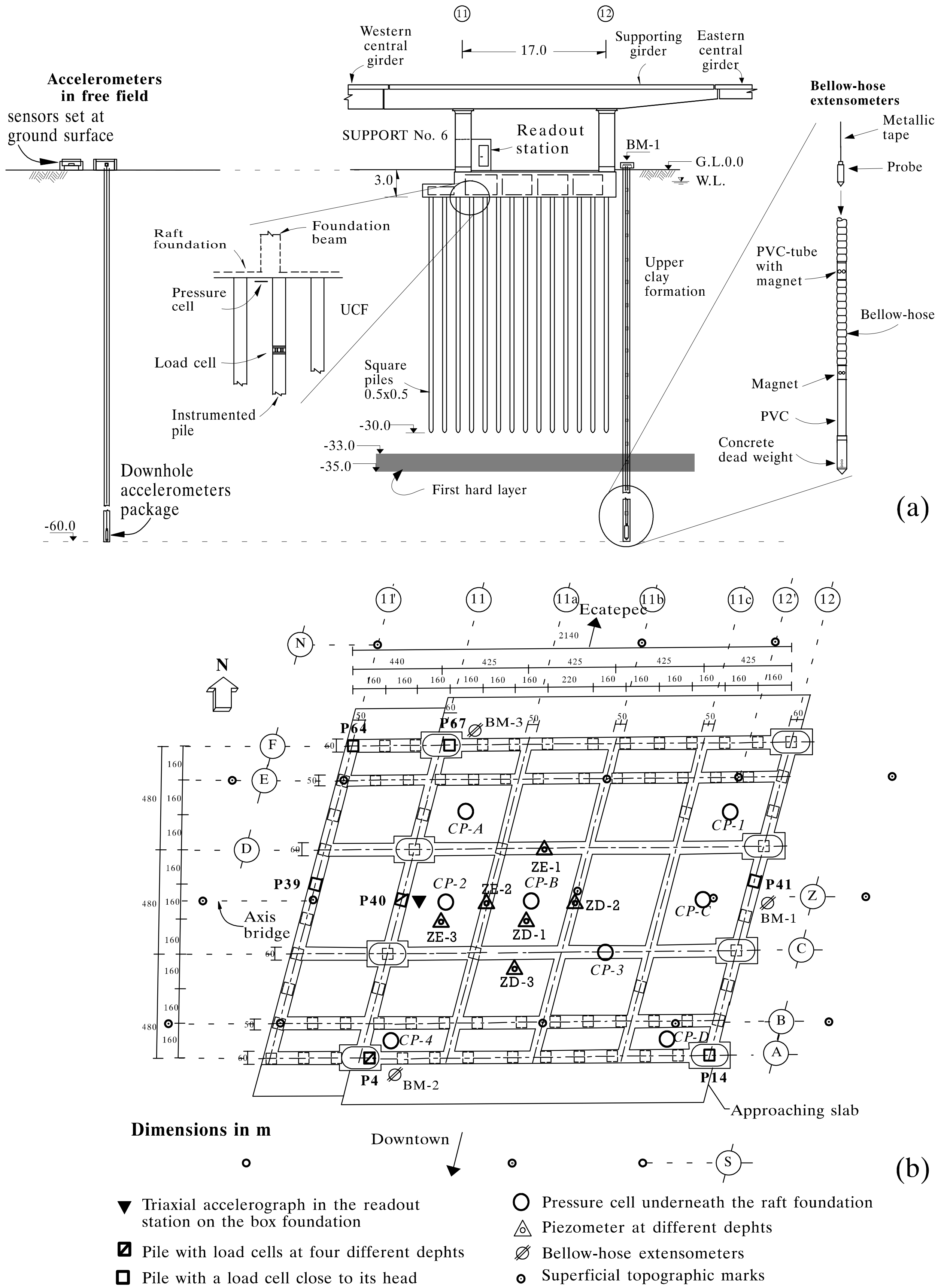

Fig. 2. Layout of the instrumented prototype foundation

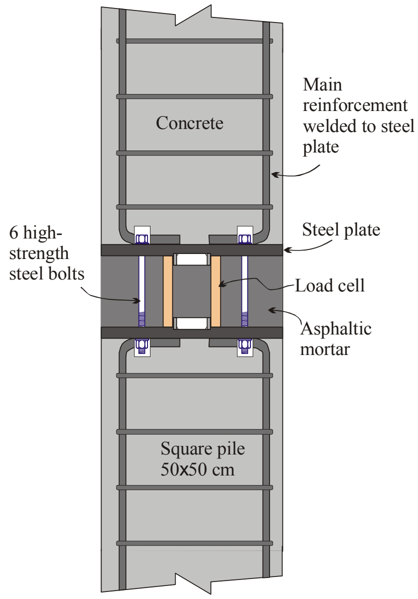

Fig. 3. Load cell in an instrumented pile

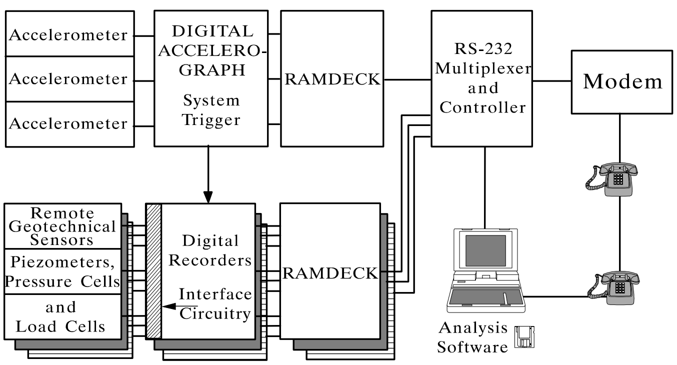

Fig. 4. Automatic data acquisition system for seismic events.

LONG-TERM BEHAVIOR OF THE FOUNDATION

Evolution of loads on piles

Loads have been measured near the head of seven piles, since the beginning of construction, as it is depicted in Fig. 5. Loads carried by the piles at the end of bridge construction were not evenly distributed. There was a maximum to minimum load ratio of 2.35 (Dec., 1995). However, six months later, when the bridge was opened to vehicle traffic, this ratio had decreased to 1.56. This non uniform distribution was mainly due to uneven or temporary eccentric loads on the foundation system, because construction operations like pile driving, ground excavation, and girders lifting.

Loads on piles kept on increasing, albeit at small rates, at apparently constant load after the end of bridge construction. This load gain was found to be caused by water flooding of the foundation-box-cells, as verified in the field. Also, the

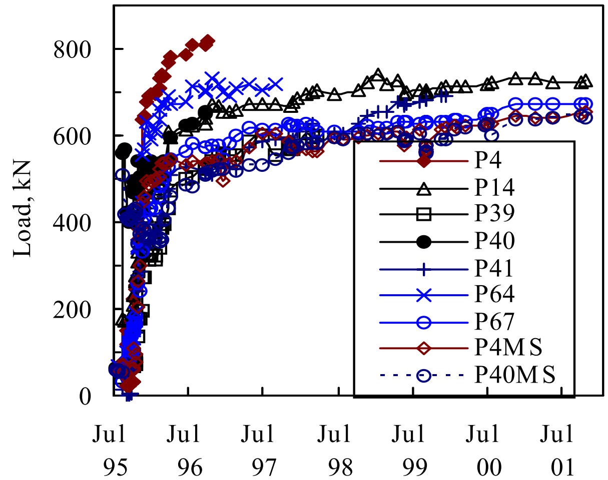

Fig. 5. Evolution of loads on piles.

increment of load on piles after the building completion has been reported (Yamashita & Kakurai, 1991) for a friction pile-raft foundation. The load-time patterns also show that the pile loads scatter continues decreasing with time. This suggests that once the box foundation is fully structured its high stiffness starts playing a load-uniformizing role, until the distribution of loads on piles is compatible with the foundation settlement pattern. Also, it is likely that any residual soil stresses that might have developed during pile driving were released through this process.

It is worthwhile to notice that corner piles and one at the long-edge carry heavier loads than do interior and short-edge piles. This agrees with which has been measured by other researchers (Hansbo, 1984; Sommer, 1985; Jendeby, 1986). This fact is also in agreement with the theoretical solution given by Poulos (1968) for a rigidly capped pile group.

Piles P4 and P40 were instrumented with four load cells along their length. The cells at pile P4 have responded consistently throughout the initial period of measurement, yielding valuable information regarding the load-transfer phenomenon at pile-soil interface. The load distribution-depth curves are included in Fig. 6. These results show that at all times positive skin friction developed along the full length of the pile, except for a short period, at the early construction stages, near the pile tip where small negative skin friction seemed to develop. It is interesting to observe that, for friction piles in a very soft clay deposit, the end-bearing load can be as high as 30% of the total reaction of the pile, and that the soil at the tip of the pile might be yielding.

Knowing the load-transfer mechanism (Fig. 6) the shear stresses along the soil-pile interface may be easily computed. At the end of the bridge construction the shear stress along the mid-third had reached the undrained shear strength of the adjacent soil; the upper and the lower thirds had developed smaller shear stresses than the soil strength.

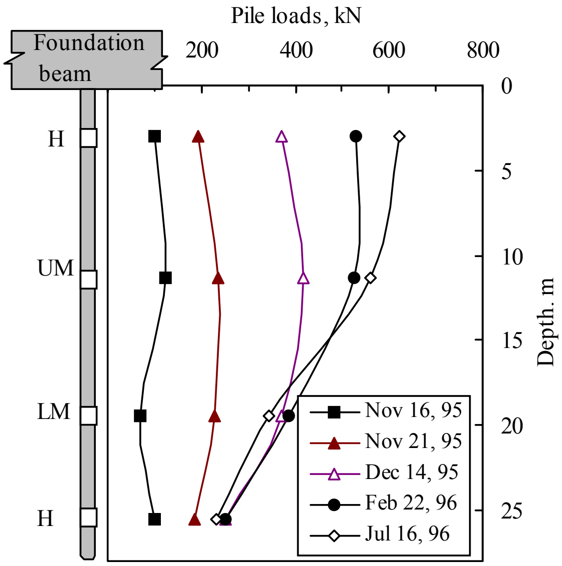

Fig. 6. Time-varying load-transfer curves for pile P4

Evolution of soil-slab contact pressures

Observed time variations of soil-slab contact pressures follow a similar pattern to that of pile loads. Fig. 7 shows how the pressures increase with loading during the construction stage and remain more or less stable after the bridge was opened to vehicle transit. The soil-slab pressures measured across the foundation vary spatially. The maximum to minimum pressure ratio of 2.42 at the end of bridge construction reduces to 1.93 after three-year period of bridge operation. Tendency to contact pressure homogenization, as in the case of piles, appears indicative of larger soil deformations at the areas where higher pressures existed initially. Total pressure fluctuations seem to be related to seasonal changes of ground water level.

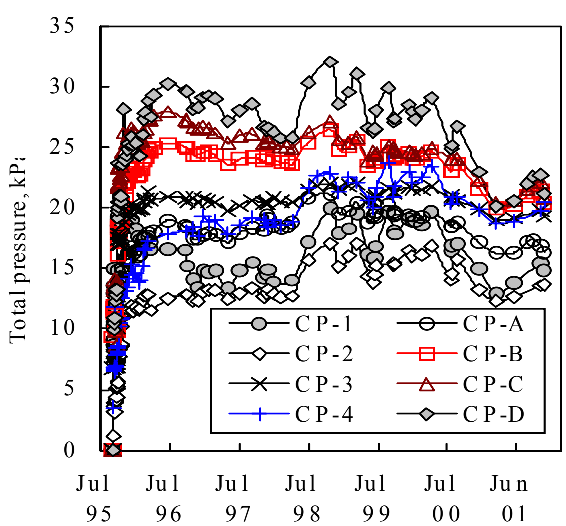

Fig. 7. Time-varying raft-soil contact pressures.

Following the approach given by Clancy & Randolph (1996), the group of piles resulted about five times stiffer than the slab foundation. The pile-slab transfer mechanism may be idealized as follows: applied loads are first taken by piles until they start yielding and from this moment on, the slab begins sharing the applied load. The percentage of the total amount of load that both piles and slab carry depends on their individual load-deformation characteristics and the total settlement the foundation undergoes. In this case history, the piles carry approximately 85% of the total applied load, and the remainder by the box foundation.

Movements of the foundation

Continuous levelling surveys of the foundation have been carried out since its construction up to date. Initial heave due to piling and subsequent settlements of the box foundation are depicted in Fig. 8. Maximum settlements are 45.2 cm at the axis 11 (see Fig. 2), and 40.8 cm at axis 12, five years after the bridge began its construction.

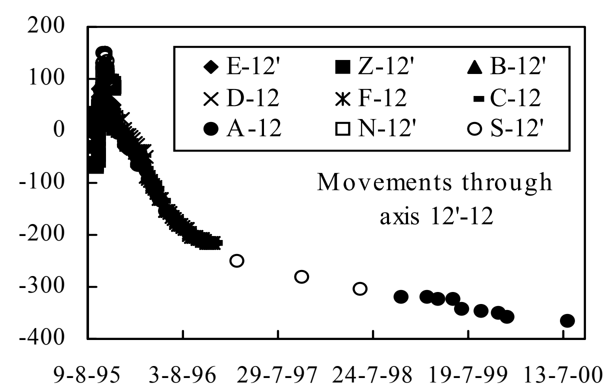

Fig. 8. Time-varying settlements curves

Evolution of soil water pressures

The piles were driven by hammer impacts without any preboring, except along the top dry crust. As a result, some 500 m^3 of soil were displaced causing compression and distortion of clayey soils. The highest pore water pressure induced by these effects was about 21% of the hydrostatic pressure near the pile tip elevation (piezometer ZD-3, at a depth of 27 m) as depicted in Fig. 9. Pore water pressure increments measured by the other piezometers were smaller. Time-pore pressure graphs show that the maximum pore water pressure was recorded by piezometer ZD-3. Afterwards it dissipated within a short period of time. This would seem rather surprising on account of the clay low permeability (10^{-8} to 10^{-10} m/sec). However, it has long been recognized that ground subsidence caused by aquifer water withdrawal has produced clay micro fissuring in many areas in Mexico City. Thus, it would be expected that pile driving operations, particularly when carried out with no preboring, activate them forming a net of fissures or cracks, allowing faster water

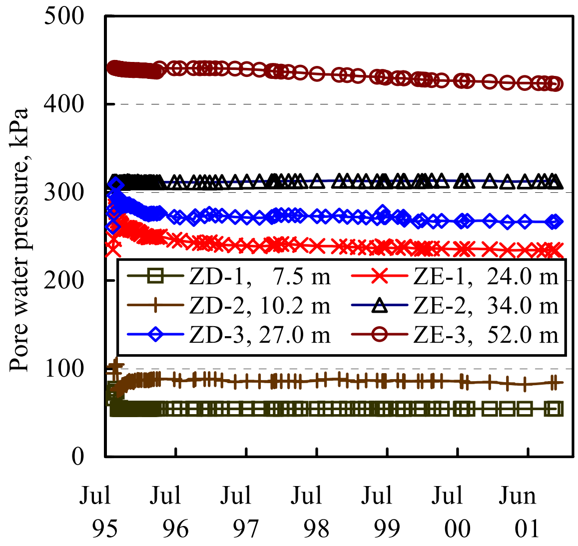

Fig. 9. Time-varying soil pore water pressures.

flow. It may be seen that the pore-pressure regime, established since the bridge was open to traffic, has not been modified within the clay layers. On the other hand, piezometer ZE-3, located at a depth of 52 m, shows a water level depletion of 4.5 m, which is attributed to aquifer water withdrawal. According to superficial surveys referred to a fixed point located at a relatively near rocky hill, this has already induced about 20 cm/year of ground subsidence at the foundation site.

SEISMIC RESPONSE OF THE FOUNDATION

Main characteristics of the recorded earthquakes

A number of earthquakes have shaken the foundation site since the bridge construction. All earthquakes with magnitudes larger than 5.9, have been recorded. The main characteristics of the ten recorded earthquakes are included in Table 1. All these earthquakes are associated to the subduction phenomena occurring along the Pacific coast.

Table 1. Main characteristics of the recorded earthquakes

Earthquake	Date	Magni -tude	Epicen -tral d. km	Depth, km
1. Michoacán Coast	110197	7.3	450	17
2. Oaxaca Coast	190797	6.3	400	10
3. Tehuacan,Puebla	150699	7.0	216	92
4. P Escondido, Oax	300999	7.4	452	33
5. Low Balsas River	281299	5.9	294	45
6. Puebla-Morelos	210700	5.9	155	47
7.Gue.-Mich. Coast	090800	7.0	441	35
8. Guerrero Coast	07100	6.1	303	10
9. Guerrero Coast	180402	6.3	409	15
10. Colima Coast	210103	7.8	543	33

Geotechnical variables during the January 11,1997 earthquake

Loads on piles. The history of loads on the pile head P41 during this seismic event is shown in the Fig. 10a. It is distinguished that the maximum amplitude of the dynamic load was of approximately 25 kN, which represents 4.6% of the sustained load right before the earthquake. The recorded load-time history shows no degradation or decrease of the pile load capacity. The seismic event is completely transitory and after the shaking no change in the pre-earthquake loading is appreciated, Figs. 10a and 10b.

The recorded signal at the pile P4 in its upper-medium cell, located 11.35 m below the foundation grade is depicted in the Fig. 11a. Larger changes are observed in the transient cyclic loads during the seismic event. Load amplitudes reach up to 88.3 kN. In this pile, some degradation on the load capacity is observed toward the end of the earthquake. This fact is related to the larger amplitudes of the cyclic loading regarding to the static capacity of the pile (Jaime et al., 1990). Observe that this pile is located in one of the long edges of the foundation plan, where maximum displacements caused by overturning moments occur; pile P41 in turn is in the longitudinal axis of the bridge, where lower rocking develops during the shaking.

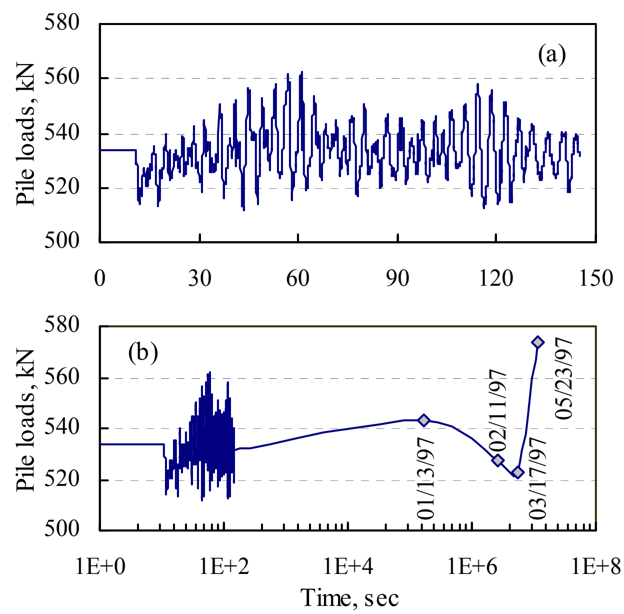

Fig. 10. History of loads on pile P41 during the January 11, 1997 earthquake

Now, regarding the load increase after the earthquake halted (Fig. 11b), it may be explained on the grounds that shear stresses along pile-soil interface dropped from the dynamic to the static loading condition leading, by equilibrium conditions, to an increase on the load measured by the cell. This stress drop most likely did not occur instantaneously, as indicated by the load measured on Jan. 13, 1997. This load (495 kN) that is similar to the average load measured after the first load

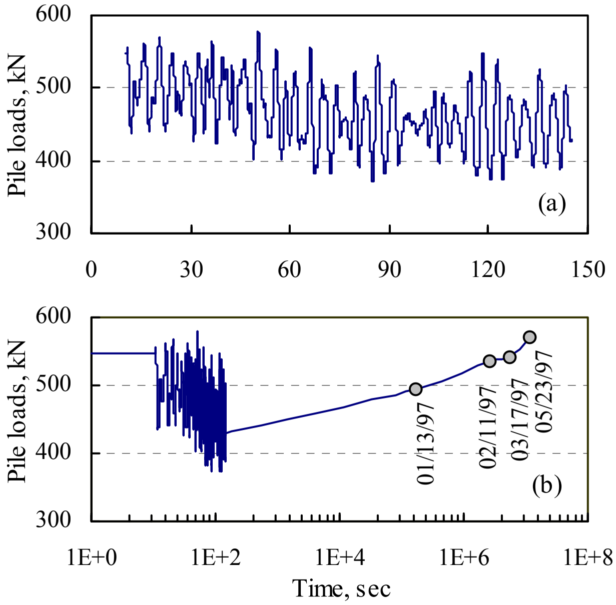

Fig 11. History of loads on pile P4 during the January 11, 1997 earthquake

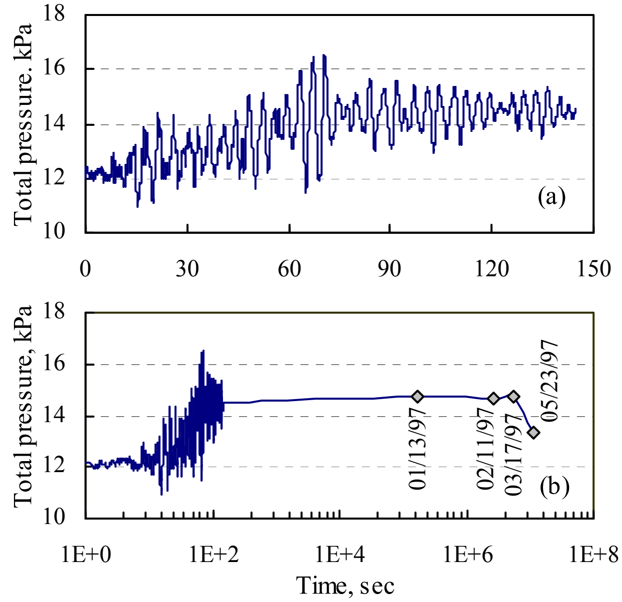

Fig. 12 History of vertical pressures at CP1 at the soil-slab contact during the January 11, 1997 earthquake.

pulse, keeps on increasing until the pre-earthquake load was reached on February 11, 1997. Afterwards, the load remains practically constant over a period of some 34 days, pointing out that seismic loading did not cause any appreciable fatigue damage to pile-soil interface materials. Then, the load increased again, perhaps due to water flooding of the box as contended previously and thixotropic effect.

<u>Vertical pressure at the slab-soil interface.</u> Pressure on the soil-slab contact exhibited transient cyclic low variations that attained larger amplitudes near to the long edges. Indeed, while the cell CP2 located in the axis of the bridge recorded an amplitude of 0.45 kPa, the cell CP1 placed near a corner reached a value of 2.5 kPa, as can be distinguished in Fig.12. The time-records in the four pressure cells reveal a gradual and cyclic increase in the total pressure as the earthquake proceeds, indicating that a phenomenon of transfer load from the piles to the soil-slab contact develops. This interpretation is consistent with the measured values with the portable digital readouts, under sustained loads. "Static" measurements of the geotechnical sensors were carried out on Dec. 19, 1996 (earlier to the 11 January, 1997 earthquake). Less than 48 hours after this seismic event, instrumental readings were repeated. While a decrease in the loads supported by the piles was measured, a small but clear increment on the foundation slab pressure was recorded. It is also convenient to highlight the recovery of the load supported by pile P4; from the load of 441 kN immediately after the earthquake, in less than two days the load increased to 494 kN. This could be explained by thixotropic effects of the clay around the shaft piles, and local consolidation near this same surface (Holmquist & Matlock, 1979). Afterwards, while load on piles increased, Fig. 11b, contact pressures decreased, Fig. 12b. This is a clear manifestation of the pile-slab transfer mechanism

<u>Pore water pressure in the subsoil</u>. This variable was recorded during the earthquake by means of the three quick-response piezometers, placed at different depths below the central portion of the foundation plan. The piezometer ZD1 laid down in a sandy stratum at the level –7.5 m showed a cyclic and sustained very small increase of pressure, with double-amplitudes of not more than 3.5 cm of water column. The

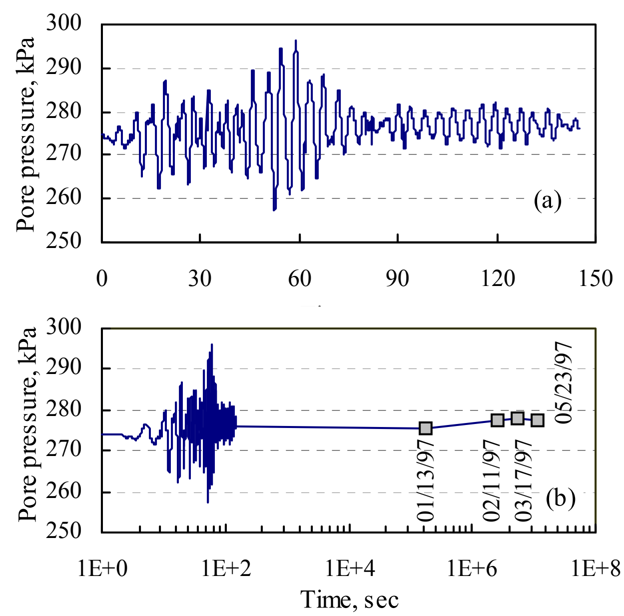

Fig. 13. History of pore water pressures in clayey strata to 27.0 m-deep during the January 11 earthquake, and later

piezometer ZD2 located ex professo in a clayey stratum to –10.2 m reached dynamic double-amplitudes of 27 cm of column of water. However, the piezometer ZD3 located in a clayey stratum, but near the tip of the piles where some stress concentration develops, it exhibited pulses of larger amplitude (Fig. 13a). The maximum recorded amplitude was 1.88 m of water column, which is 6.7% of the hydraulic pressure before the earthquake. When shaking finishes, a very small residual pore pressure was observed. This denotes the transitory nature of the phenomenon. These results seem to point out that consolidation processes by earthquake-induced pore pressures are not relevant in Mexico City clay, at least at some distance from the piles. These measurements are in agreement with laboratory results of different research works. They indicate that the dynamic pore water pressure induced by cyclic loading is small, even near the failure (Romo, 1995). If significant pore pressures develop in the pile-soil interface, their effect seems to dissipate at relatively short distance from the pile.

Frequency analysis of state-variable-time-histories. Several interesting aspects of the dynamic soil-foundation interaction phenomenon of the soil-foundation system may be drawn comparing Fourier spectra of the geotechnical state variables (Romo et al., 2000). In general, spectra for the horizontal directions follow similar patterns. Thus, only the longitudinal response is discussed for the Jan. 11, 1997 event, mainly.

Fourier spectra of the loads recorded during the seismic event on piles P4 and P41, as well as the acceleration Fourier spectrum are shown in Fig. 14a. In a broad frequency range, pile loads and acceleration spectra have peaks and valleys practically at same frequencies. This fact is related to the large stiffness of the box-foundation system. The maximum amplitude in all three spectra occurs with a 0.24 Hz frequency, which corresponds to the dominant frequency of the free field. Pile P4 develops larger amplitudes than pile P41. This may be likely due to their location within the foundation. Since pile P41 is located at the edge of the foundation and almost on its longitudinal axis, it would be expected that the load be mostly caused by the rocking with respect to the transverse axis. Alternatively, this confirms that when the foundation is appreciably stiffer than the supporting soil, the loads induced by the structure-foundation rocking may be estimated by simple procedures.

Fourier spectra of the pressures recorded during the seismic event on cells CP1 and CP2 are compared in Fig. 14b with the acceleration spectra. It is remarkable the similitude of the three curves, although the maximum spectral amplitude of the pressure spectra shows a slight shift to higher frequencies with respect to that of the acceleration spectra. This could be caused by the coupled effects of foundation rocking and the higher-frequency vertical vibration of the box-foundation system. The larger pressure amplitudes on cell CP1 are explained on the basis of its location. While cell CP2 is near the geometric centre of the foundation, cell CP1 is near one of the box edges.

Fourier spectra of the water pressures recorded by piezometer ZD3 for two seismic events are compared in Fig. 14c.

It is clear that all spectral curves are alike in their shape. The dominant frequencies of the system motions coincide with those of the geotechnical variables. This implies that these variables are in phase with the foundation movements.

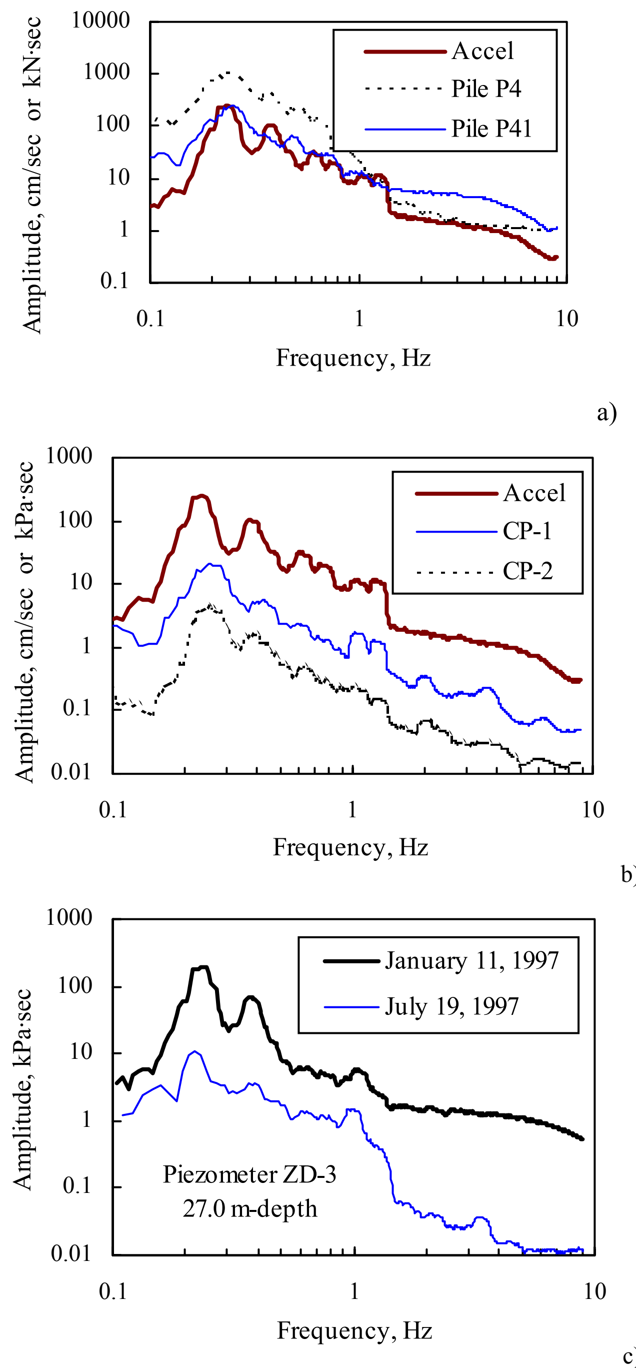

Fig. 14. Fourier spectra for longitudinal accelerations and a)loads on piles, b) pressures on foundation raft-soil interface and c) pore water pressures during two earthquakes

REMARKS ON DYNAMIC INTERACTION OF THE FOUNDATION-SOIL SYSTEM

The full seismic instrumentation, including the free field sensors, was already available for the Tehuacan earthquake. Its records have given valuable information regarding the foundation-soil dynamic interaction. Maximum accelerations recorded at the box foundation and free field, both at the surface and down to 60-m depth (Deep Deposits) are included in Table 2. The relative amplifications or ratios between the foundation and the surface free field motions (FoFs), and between foundation and Deep Deposits motions (FoFd) are also gathered in this table.

Table 2. Accelerographic data recorded at the foundation and near free field during the Tehuacan earthquake (150699).

Direction	Maximum accelerations at the foundation		FoFs
	gals	relative	
Longitudinal	28.10	1.000	0.96
Transversal	24.62	0.876	0.99
Vertical	2.15	0.077	0.14
	Maximum accelerations at the surface of free field		FoFd
Longitudinal	29.07	1.000	1.78
Transversal	24.66	0.848	2.26
Vertical	15.20	0.523	2.91
	Maximum accelerations down to 60-m deep in free field		Longitudinal direction of the bridge: N80°W
Longitudinal	16.31	1.000	
Transversal	10.91	0.669	
Vertical	5.22	0.322	

An interesting result that stands out is that the horizontal accelerations at the surface of free field and those recorded at the foundation are practically the same, which implies a very reduced soil-foundation interaction to lateral movements. Accordingly, friction piles follow the dynamic lateral movements of the surrounding soil. It is interesting to observe the reduced influence of the box foundation against lateral actions. However, it is worthwhile noticing the drastic reduction of the vertical accelerations recorded at the bridge foundation, in comparison with those recorded at the surface of the near free field. While the maximum horizontal accelerations in the foundation and the surface free field were similar, the vertical movement in the foundation was seven times lower than in the neighbour surface. Without doubt, the larger stiffness of the piles in the vertical direction in comparison with that one of the soil in the same direction plays the key role. This shows the important attenuation caused by the pile-soil interaction in the vertical ground movements, as opposed to the horizontal movements.

On the other hand, the maximum accelerations recorded at the surface of the free field, in comparison with those measured in the Deep Deposits, reveal the well known amplification of the seismic waves when they travel from rigid to soft soil deposits. Indeed, in this lacustrine deposits occurred the most conspicuous site effects ever recorded (Romo and Seed, 1988), during the Michoacan strong earthquake (M=8.1) in 1985.

A more detailed review of the dynamic interaction due to seismic events can be observed in terms of the transfer functions for all frequency domain. The transfer functions depicted in Fig. 15 were obtained as the ratio between the accelerations Fourier spectra of the foundation and the surface free field, for each component. A vertical reference line marks the fundamental frequency of the site (0.24 Hz). The transfer function for the vertical component shows an appreciable interaction effect in the entire frequencies interval, as indicated by the values of the relative amplitude always lower than one. On the contrary, lateral transfer functions make evident the low dynamic interaction; their values are around one throughout the frequency range.

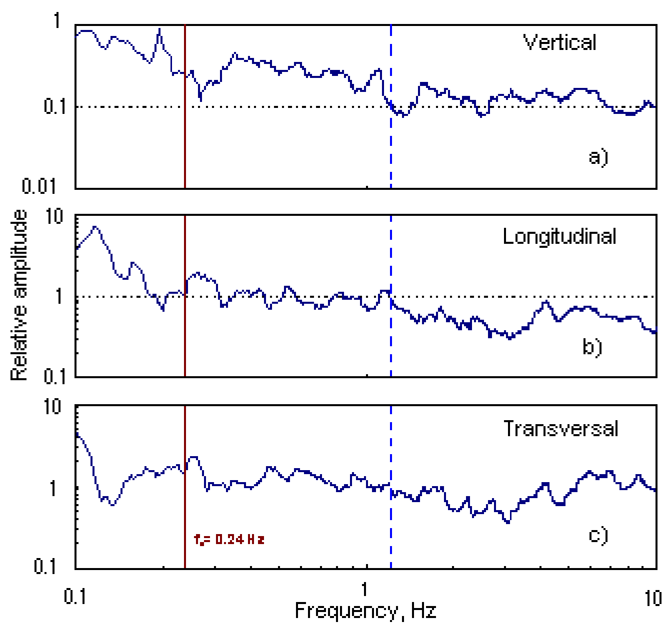

Fig.15. Transfer functions for the motions between the foundation and the surface free field. Tehuacan earthquake

CONCLUSIONS

It is clear that from just one prototype, is not possible to derive definitive and general conclusions. However, the following lessons were learnt:

1) The total bridge load is transmitted to the soil through the piles and the raft. The load-sharing is about 85% for the piles and 15% for the raft, at the time the bridge was opened to traffic. No significant changes were measured during the operation period of the bridge.

2) The pile tip capacity is near 30% of the total pile bearing capacity. Neglecting this contribution, as is usually done for friction piles, might lead to conservative designs.
3) Shear stresses along the pile shaft computed from the measurements indicate that the soil strength has been reached in the mid third of the pile.
4) Distribution of loads on piles and of raft contact pressures are not uniform mainly due to the construction sequence, which included asymmetric loading. This aspect deserves more consideration because it may lead to overloading of some pile elements, thus putting them in risky conditions, particularly for seismic conditions.
5) Pore water pressures increased during piling operations due to the large volume of soil displaced, but the effect of loads imposed by the bridge was negligible. Clay micro fissuring seems to have a considerable influence on the latter phenomenon.
6) The time-records indicate transitory cyclic processes in the axial loads on piles, in the total vertical pressure on the slab-soil interface, and in the pore water pressure in the subsoil, when a seismic event occurs. Dominant frequencies of these variations are in close relationship with those for the accelerations of the foundation box; they are in phase.
7) The force time-records on the friction piles and vertical pressure on the slab-soil contact, indicate a load transfer mechanism through which the reduction on the capacity of piles is taken by the foundation slab. Thus the raft slab of the box foundation provides an element of safety in this type of foundations, if the pre-earthquake contact pressure plus the induced dynamic pressure do not exceed the soil yielding stress.
8) During a mild but prolonged earthquake, while the pile P41 in the longitudinal axis of the foundation endured dynamic amplitudes up to 4.6% of the sustained load, the pile P4 located in the long edge underwent 16.2% of the static load. No degradation of the bearing capacity for the first pile was detected; nevertheless, a loss of 19.2% was measured for the second one to the end of the earthquake.
9) Two days after the earthquake, the piles reacted in general with larger load to that one recorded immediately after the seismic event. This recovery was as high as 12% for the pile P4. To this phenomenon likely contribute local consolidation, mainly in the short term, and then a thixotropic effect of the clay in contact with the piles.
10) The pore water pressure in the subsoil below the foundation suffers very small transient cyclic variations during the earthquake. The maximum transient increment reached 6.7% of the pre-earthquake pressure in a clayey stratum, close to the piles tip. The residual pore water pressure induced by the earthquake of January 11, 1997 did not exceed 0.8% of the static pressure. It was verified that such a pressure dissipated after few days.
11) Friction piles offer a clear reaction to the vertical movements due to seismic events. On the contrary, very low dynamic interaction develops for lateral seismic movements.
12) The instrumentation has exhibited a very satisfactory response, after a monitoring period of eight years.

REFERENCES

Clancy, P. & M. F. Randolph [1996]. "Simple design tools for piled raft foundations" Géotechnique. Vol. 46, No. 2, 313-328.

Hansbo, S. [1984]. "Foundations on friction creep piles in soft clays", *Proc. Int. Conf. on Case Histories in Geotech. Engrg, St. Louis MO*, Vol. II, 913-922.

Jaime, A., M. P. Romo & D. Reséndiz [1990]. "Behavior of friction piles in Mexico City clay", *J. of Geotech. Engrg. Div.*, Vol. 116, No. 6, 915-931.

Jendeby, L. [1986]. Friction piled foundations in soft clay. A study of load transfer and settlements, D. thesis, Chalmers Univ. of Technology, Gothenburg, Sweden.

Marsal, R. J. and M. Mazari [1969]. *The Subsoil of Mexico City*, National University of Mexico.

Mendoza, M. J. & G. Auvinet [1988]. "The Mexico earthquake of September 19, 1985 - Behavior of building foundations in Mexico City", Earthquake Spectra,4-3,835-853.

Mendoza, M. J. & M. P. Romo, [1998] "Performance of a friction pile-box foundation in Mexico City", Soils and Foundations, Vol. 38, No. 4, 239-249.

Mendoza, M. J., M. P. Romo, M. Orozco & L. Domínguez [2000], "Static and seismic behavior of a friction pile-box foundation in Mexico City clay", Soils and Foundations. Vol. 40, No. 4, 143-154.

Poulos, H.G. [1968]. "Analysis of the settlement of pile groups", Géotechnique, Vol. 18, 449-471.

Romo, M. P. & H. B. Seed [1986], "Analytical modelling of dynamic soil response in the Mexico earthquake of Sept. 19, 1985". *Proc. Int. Conf. on the Mexico Earthquake-1985*, ASCE, Mexico City: 148-162.

Romo, M. P. [1995]. "Clay behavior, ground response and soil-structure interaction studies in Mexico City", State-of-the-art paper. *Proc. 3rd. Int. Conf. on Recent Adv. in Geotech. Erthq. Engrg. & Soil Dyn.*, St. Louis MO, Vol 2, 1039-1051.

Romo, M. P., M. J. Mendoza and S. García [2000] "Geotechnical factors in seismic design of foundations: State-of-the-Art Report", *Proc.12th World Conference on Earthquake Engineering*, Auckland, New Zealand, No. 2832, 24 pp.

Sommer, H., P. Wittmann & P. Ripper [1985]. "Piled raft foundation of a tall building in Frankfurt clay", *Proc. 11th ICSMFE*, San Francisco. Vol. 4, 2253-2257.

Yamashita, K. & M. Kakurai [1991]. "Settlement behavior of the raft foundation with friction piles". *Proc. 4th Int. DFI Conference*, Balkema, 461-466.

Zeevaert, L. [1972] *Foundation Engineering for Difficult Subsoil Conditions*, Van Nostrand Reinhold, New York.