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GEOSEISMIC INSTRUMENTATION OF A FRICTION PILE-BOX FOUNDATION

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ABSTRACT

In order to learn more about the behavior of friction pile-box type foundations in the very soft clayey deposits of Mexico City, an actual foundation which is part of the support system of a vehicular and pedestrian bridge was instrumented. Due to critical conditions imposed by seismic events on foundations in Mexico City, particular emphasis has been given to measure the involved state variables during the occurrence of an earthquake; among them, foundation accelerations, loads on piles, contact pressure in the soil-raft interface, pore water pressure in the subsoil beneath the foundation and the foundation movements. The bridge is under construction now and the digital accelerograph as well as the complete seismic/geotechnical data acquisition system has not been connected yet; however, a full description of foundation construction, sensors, placing procedures and initial measurements with manual read-out, during the construction stage, are presented and discussed in this paper.

KEYWORDS

Piled box foundation; geotechnical instrumentation; seismic instrumentation; friction piles; soft soils

INTRODUCTION

Foundation engineering in Mexico City faces up to very difficult conditions, not only due to the presence of soft and highly compressible clayey lacustrine deposits, but due to the occurrence of frequent and strong earthquakes and regional ground subsidence. Among the different types of foundation in the lacustrine clay sediments, a mixed system constituted by a hollow box and friction piles used to be the most common foundation for five to fifteen-floor buildings until 1985, when the Michoacan earthquake occurred. Due to this seismic event, 13% of these buildings, a figure truely high and unacceptable, suffered conspicuous settlements and tiltings, severe structural damages, and even a full overturning.

As a response to the above situation, more stringent regulations were imposed in the Building Code to this type of foundation, but a full evaluation of its behavior has not been done yet. Trying to fulfill this gap, a geotechnical and seismic instrumentation in a prototype of a friction pile-box foundation was set up, which is described and discussed in this paper. The selected prototype (15x22x3 m-foundation box with 77 friction piles down to 30 m) is the support of the central span of a bridge located in the Northeastern zone of Mexico City.

The bridge is under construction at the moment of writing this paper; this process is being monitored by manual digital recorders. The readout station has not been built yet and that is why the automatic digital recorders have not been connected to the seismic and geotechnical sensors.

GEOTECHNICAL CONDITIONS IN MEXICO CITY

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 settled to the bottom of a former lake. Transition Zone (II) is characterized by even abrupt stratigraphical changes from place to place between those of Zones I and III.

Friction pile foundations are frequently used wherever poor ground conditions exit; a conspicuous and extreme example is the Lake Zone of Mexico City, comprising the downtown area. Mexico City soft clayey lacustrine soils are characterized by very high compressibility, low shear strength, normally or lightly overconsolidated behavior, very low shear wave velocities, thick deposits and quite high water contents, void ratios and plasticity indexes; some average figures are as follow:

* After an artificial fill and dry crust of few meters, the thickness of the Upper Clayey Formation (UCF) in downtown area reaches 30 to 40 m, and increases up to 80 m toward the East. A "hard" silty sand stratum, 2 to 3 m-thick and locally known as the First Hard Layer (FHL), underlies these very soft deposits; it should be stressed that FHL soils may be even more compressible than London clay, for example. A Lower Clayey Formation is underlying, with lightly overconsolidated soft clayey soils of lower water contents than those of UCF soils.
* Usual water contents of UCF soils are 300 to 400% (most of the times close to the liquid limit), void ratios close to 10 and plasticity index of about 240.
* Coefficients of compressibility, a_v, are usually larger than 0.01 kPa^{-1} (1 cm^2/kg).
* Undrained shear strength varies between 10 and 40 kPa for UCF soils.
* Common shear waves velocities are as low as 30 m/s for UCF soils.

These poor soil conditions are worsened by *i)* the regional ground subsidence of the Valley of Mexico where the city is settled with typical sinking rates of 5 to 20 cm/year; this phenomenon is associated to the piezometric heads losses due to the exploitation of the underground deep aquifers; and *ii)* the frequent occurrence of distant but strong earthquakes; so, for example, the Michoacan earthquake (M_s=8.1) of September, 19, 1985 caused severe damage to Mexico City, even thought the city is more than 350 km far from the epicenter located in the subduction trench near the coast of the Pacific Ocean.

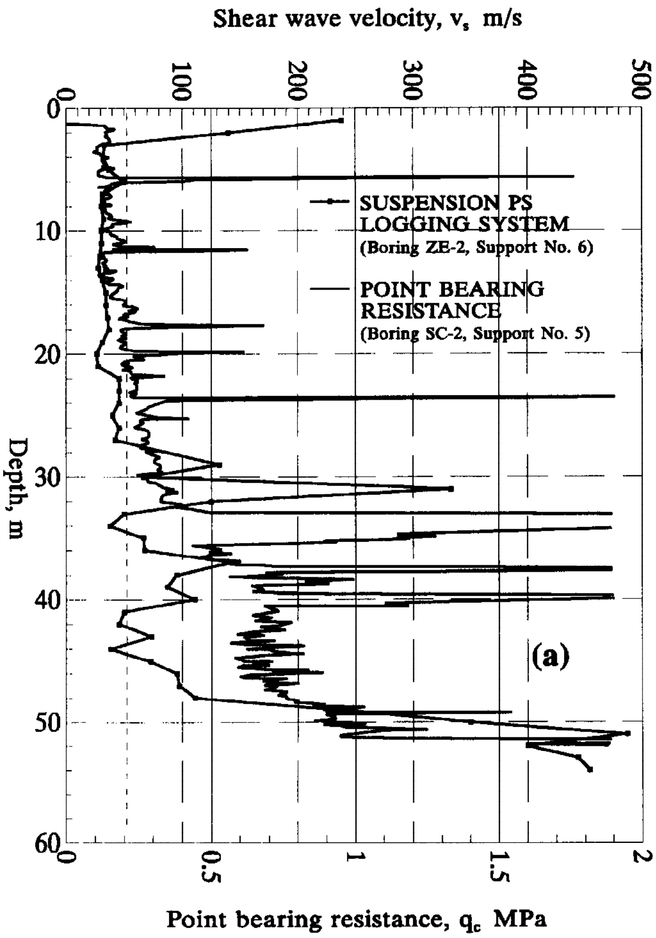
Foundations on point-bearing piles were used to support heavy buildings, making use of the FHL as the bearing stratum. However, they are rarely used now because regional subsidence induces an apparent emersion of the buildings resting on this type of foundation, provoking unattractive appearance, undesirable differential movements on neighboring buildings and problems on service lines. To overcome these problems in such a cases, the profession has made use of friction piles, or point-bearing piles with some kind of load controlling devices placed at their connection to foundation slab or beams, in order to fit the building displacements to the movements of the surrounding ground.

Design and construction of foundations in the Zone III faces very difficult engineering problems because of the geotechnical and seismic environment previously mentioned. Indeed, the most spectacular effect of local soil conditions ever seen in the world during an earthquake occurred during the Michoacan earthquake. While no damages or very few occurred in Zones I and II, a high dynamic amplification happened in Zone III with the consequent extensive damages; a discussion of the characteristics of the ground motion in different parts of the city, as well as of the local soil conditions effects has been presented by Romo and Seed (1987). Very large total and differential settlements, impressive tilting and even a full toppled building were evidences of unsatisfactory foundation performance (Mendoza and Auvinet, 1988; Mendoza, 1990). As was mentioned, the results of a survey made in the downtown area of all the 5 to 15-story buildings seriously damaged or collapsed, showed that in 13% of them, damage was attributed directly to the foundation; most of those buildings were founded on mixed foundations, which comprise a hollow reinforced concrete box and friction piles. So, a lot of uses, and abuses, of this kind of foundation have been made in Mexico City.

The state-of-the-art and practice in foundation engineering must be evidently improved in order to give better solutions to this type of foundation. There have been important efforts by Mexican engineers to overcome these difficult geotechnical problems, mainly through better analytical solutions, detailed subsoil exploration and careful laboratory testing; however, no foundation in the city had been instrumented to define and learn about its seismic performance; even, neither exist measurements in a prototype on static behavioral aspects such as load-sharing distribution between foundation slab and friction piles.

SOIL SITE CONDITIONS

After a relatively thin dry crust, which was almost eliminated by the excavation, the site is characterized by very soft clayey deposits. Average undrained shear strength of undisturbed samples taken from UCF and measured in quick triaxial tests was 12 kPa. The area of the site is a young normally consolidated deposit comprising the bottom materials of a shallow lake. This indeed low strength is corroborated by the very low cone bearing resistance, as well as by the profile of shear wave velocities, Fig 1a. P and S-wave velocities were measured at each meter (Gutiérrez, 1995) with the suspension P-S logging system (Uchiyama *et al.*, 1984) through an upward movement of the probe, which includes wave source and receivers.



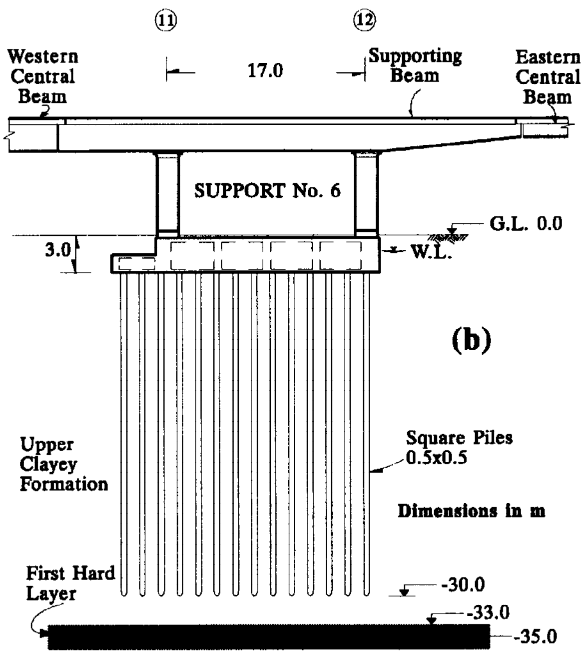


Fig 1. Geotechnical data of the site and lateral view of the instrumented friction pile-box foundation and bridge superstructure

INSTRUMENTED PROTOTYPE

The selected prototype is the support No. 6 of the central span of the vehicular and pedestrian bridge located in the Northeastern zone of Mexico City; it crosses a very wide avenue wherein the B-Line of the Metropolitan is under construction. The bridge is 689.0 m long and was solved with eight supports and lightweight approaching ramps; each support has two transversal lines of cast in place reinforced concrete columns. Directly on columns, Fig 1b, supporting beams with a cantiliver portion at each side of the columns isostatically carry central beams; these structural elements are prefabricated and prestressed concrete hollow "T" shape beams. The column to column Western span between supports Nos. 5 and 6 (central span) is 56.8 m, while the Eastern span is just 43.0 m; this difference defines a shorter length for the Western cantiliver than for the Eastern one.

Support No. 6 has 8 oblong columns in two lines, separated by 17.0 m; its foundation was solved with a rhomboidal in plan box due to the oblique crossing (104° 11') of avenues, Fig 2; its dimensions are 15 x 22 m

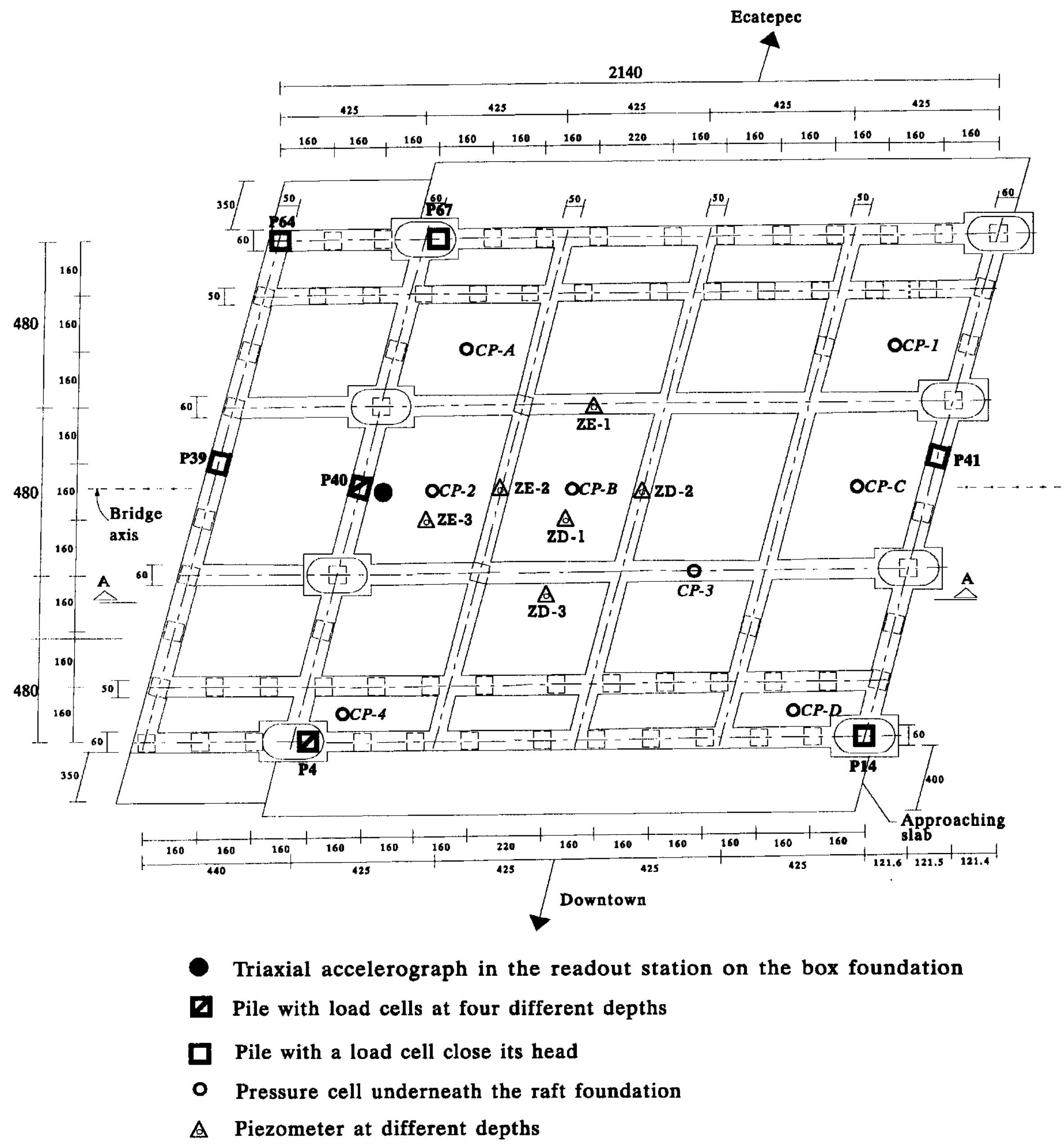


Fig 2. Plant view of the instrumented friction pile-box foundation

and the foundation grade level was at 3 m-depth. The hollow box is quite rigid because 2.7 m-height with 0.5 or 0.6 m-width foundation beams were designed in both directions, upstanding the foundation slab; they were monolithicly cast to the reinforced raft which has uniform thickness, 0.25 m. A second or cover slab, 0.22 m-thick, was cast on top of the beams, so forming a box foundation or cellular raft. The longitudinal dimension of the box is larger toward support No. 5, in order to eliminate the eccentricity due to larger loads of the central span. An impervious 4 mm-thick polyethilene geomembrane was installed underneath the raft foundation, in order to prevent the flow of soil water toward the hollow box.

Before the excavation for the box, 77 friction piles were driven in two segments 15 m each, down to 30 m-depth; keeping a gap between piles tips and FHL in order to accommodate future regional subsidence and foundation settlements. The piles coincide in plan with the foundation beams, and their design was oriented to include them as much as possible close to longitudinal edges, keeping in mind the overturning seismic moments in the transversal direction to the bridge. Reinforced concrete piles were precast in place, with a square cross section, 50 cm by side. An auger preboring, 50 cm ϕ, was made through the top 4 to 5 m; afterwards, a Delmag No. 33 piling hammer was used, although very few and controlled and reduced blows, an average value of 20, were in general necessary to apply; as a matter of fact, piles were tied up with steel cables for do not loose the design tip level. Each pile segment was finished with a 1.9 cm-thick steel plate covering the full cross section of the pile, to which the longitudinal reinforcement was welded; as soon as the first (tip) segment was driven maintaining its upper part 1 m above the ground surface, the second (head) segment was raised, matched with the first one and keeping it vertical, the steel plates were welded all around. Piling was finished when the pile's head reached a level about 0.5 m below the ground surface.

After all piling activities were over, the excavation was done taking care for not to damage the piles with the 1 cu. yard hydraulic shovel. Piles driving was carried out before excavation in order to reduce the heave at the excavation bottom, demanding by tension the upper part of the piles, and reducing the time the excavation was kept open. Piles head were demolished via pneumatic hammers, and once discovered the longitudinal steel, it was linked to the reinforcement of the foundation beams.

OBJECTIVES, TYPE AND PLACING OF SENSORS

Geoseismic instrumentation was designed to measure *a)* the contact pressure at some points of the soil-raft foundation interface, *b)* the applied loads on some friction piles, *c)* the piezometric heads in the subsoil water, *d)* the foundation accelerations, and *e)* the superficial movements of the foundation. The main objectives pursued with this instrumentation program are to learn more about the load-transfer mechanisms that develop among pile-soil and slab-soil. Thus, it is hoped that further light will be shed on the soil-pile-slab interaction problem, both in static and dynamic conditions.

The recording system is quite similar to those of accelerographic stations; in addition to the accelerometers signals, it will record those of geotechnical sensors both to predefined intervals, as well as to a certain threshold acceleration. To the authors' knowledge this instrumentation project is without precedents worldwide; previous studies in prototypes have been oriented for the static and long-term performance (Cook, 1981; Price and Wardle, 1983; Hansbo and Jendeby, 1983; Fredriksson and Rosén, 1985; Rickard *et al.*, 1985; Sommer *et al.*, 1985; Wood and Perrin, 1985; Yamashita and Kakurai, 1991).

The geotechnical (GEOKON) and seismic (TERRA) sensors installed were: 13 load cells, 8 earth pressure cells, 6 piezometers and a triaxial arrangement of accelerometers. Two independent and redundant instrumentation systems, one automatic and the other manual, were considered convenient for this project taking in account that monitoring time should be prolonged. The automatic system is oriented to record the response during a strong motion; the manual system is meant to be for measuring the performance of the foundation system under "static" conditions. Geotechnical sensors for the automatic system are of strain gage basis (SG), of convenient dynamic response, while vibrating wire basis transducers (VW) were considered more stable for long-term monitoring. Sensors of both systems are being monitored during the construction stage; in such a case, SG readings have been taken by means of a conventional strain indicator readout box, and those of VW instruments by frequency indicator readout box, both of manual type and operated with batteries. Full and accurate calibrations were done in the laboratory, before construction, of each sensor, making use precisely of the same manual and automatic recorders to be employed in the field, reproducing gains, gage factors, input voltages and placing conditions.

Load cells. Rugged waterproof construction 1.78 MN (400,000 lb) load cells were selected; their load element is a spool of high strength steel on which, in ten of them, electrical resistance strain gages are bounded in a full bridge configuration for temperature and for compensation of eccentric loads. These sensors were integrated to seven piles (Fig 2) close to their head (2.5 m beneath the raft foundation); in addition, three more load cells at different depths (0.41L, 0.7L and 0.93L, effective length) were included in two of these instrumented piles. These cells were carefully integrated to pile segments placing each one between two 25 mm-thick steel plates attached and lightly precompressed by 6 high strength galvanized steel bolts; the plates in turn were welded to the main reinforcement of the pile. Electric cables were routed through galvanized steel tubing within the piles concrete; for those cells of the first segment of the pile, it was necessary to take out the tubing and conduct it to the surface throughout the joint with the second segmental pile.

Earth pressure cells. The cells consist of two circular stainless steel thin plates welded together around their periphery and spaced apart by a narrow cavity filled with an stable degassed liquid, which is connected to the pressure transducer through a stainless steel tubing. External total pressure acting normal to the plane of the cell are balanced by an equal pressure induced in the internal fluid. Four of the cells have resistance strain gages and four more are of the vibrating wire type, all cells have a 173 kPa (25 psi) range. The cells were horizontally placed 10 cm beneath the raft foundation within a sand bed; their position (Fig 2) was defined to be representative of the total area of the soil-raft interface.

Piezometers. Six piezometers were placed at different depths in a small core beneath the box, Fig 2; the pressure transducer of three of them are of strain gages basis, and three more employs a sensitive diaphragm coupled to a vibrating wire that converts the subsoil water pressure into an equivalent frequency signal; in both cases with a 518 kPa (75 psi) range. The cylindrical body in stainless steel of the piezometer comprises the electric transducer and the porous element in the straight end. Each piezometer was carefully saturated in order to assure a quick response (Hartlén, 1985), and brought down in different boreholes to avoid possible interconnections between them; the boreholes were stabilized with water and two piezometers were located in clayey strata.

Accelerograph. A high resolution accelerograph including built-in triaxial servo accelerometers will be fixed to a concrete pedestal within the readout station which is now under construction on the box foundation. This Integrated Digital System (IDS) produces a data record in battery backed-up static CMOS RAM of seismic events; it continuously amplifies, filters and converts (1000 samples/second/channel) the sensor output to 16 bit digital form and sends the data, along with the synchronized internal time to a digital delay. After some tests in the site, the triggering criteria will be met, and the instrument will begin to record the data from the digital delay. By selecting the correct length of the digital delay, the entire event, including the first arrival, will be recorded.

Digital recorders. Digital recorders of the same type of the accelerograph will be used; instead accelerometers, they will have signal conditioned inputs for external geotechnical sensors. Fifteen geotechnical sensors will be monitored by five automatic digital recorders (three per instrument) which will be in a slave-master network with the triaxial accelerograph, under common timing for sample synchronization. The schematic arrangement of the seismic/geotechnical data acquisition system is depicted in Fig 3. Specific software operating on a Lap Top or PC provides a menu driven user interface for IDS and accelerograph set-up, trigger definition, data transfer analysis and system diagnosis; direct communication is via RS232C port, although in the future it is planned to make use of remote communication via modem.

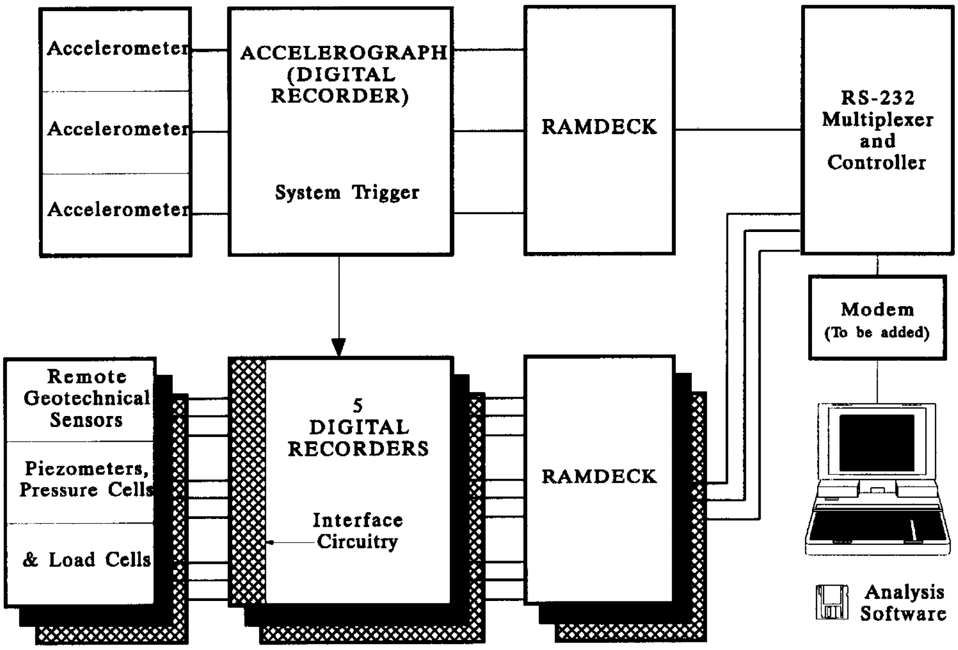


Fig 3. Seismic/geotechnical data acquisition system

A VIEW ON DESIGN APPROACHES

Two alternative foundation principles have been used in Mexico City for the design of piled box foundations (Auvinet and Mendoza, 1987; Mendoza, 1990). Type I is a conventional approach in terms of bearing capacity, for which the quantity and dimensions of the piles are selected to withstand by themselves the weight of the building, not only under static conditions but also under dynamic situations, and with a convenient safety factor.

Type II is oriented to carry the load not only through piles, but with direct contact pressure on the box; under this condition, the piles are used as a supplement to a box foundation, or viceversa. In this way, both bearing elements share the loads in order to diminish settlements, reaching an economic solution; however, it has been demonstrated (Mendoza, 1990) that inadequate behavior of buildings with friction piles during the 1985 earthquakes were closely related with an abuse (high average contact pressure) of this Type II-design. To this approach belongs the compensated friction-pile foundation advanced by Zeevaert (1957) in which the contact pressure must not exceed the overload removed by excavation; as well as the concept of creep piles (Hansbo and Jendeby, 1983; Hansbo, 1984; Jendeby, 1986) in which the piles are designed in such a way that the load in excess of the preconsolidation pressure of the clay is assumed to be carried by the piles, and the remainder by contact pressure.

The authors were not involved in the foundation design, but through simple comparisons between ultimate bearing capacity of piles and permanent loads of foundation and superstructure, it would seem that type I approach was adopted; so, piles should comply with the above design assumptions. Even though is quite early to draw conclusions because construction is not over, preliminar measurements, Fig 4, show the load sharing mechanism between piles and raft. These results seem to point out, at least during construction, that another alternative situation can be recognized between the present design procedures (Types I and II). The contact pressure on the raft is now about 70% of the overburden total pressure before excavation at that level, and it represents about 20% of the total applied load. Although the contribution of piles and raft varies during construction (Rickard *et al.*, 1985), upon completition it has been found (Cooke *et al.*, 1981) with much more stiffer clay, that the raft typically takes 25% to 40% of the total load, the remainder being supported by the piles

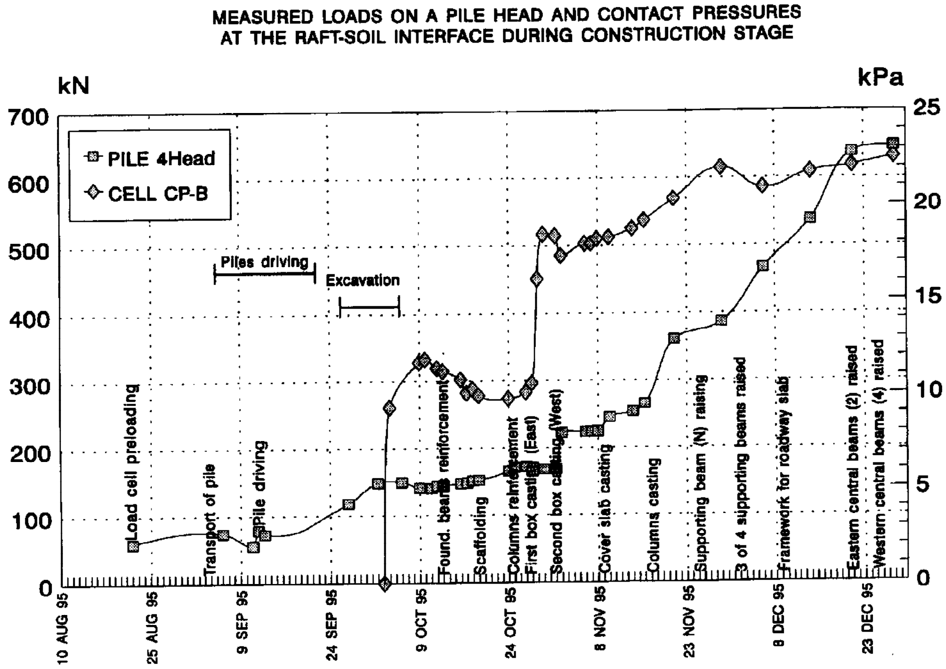


Fig 4. Measured loads on a pile and contact pressures at the soil-raft interface during construction stage

CONCLUSIONS

Multiple uncertainties in the performance and design of piled box foundations were detected after the Michoacan earthquakes in 1985; reduction of them is the aim of the instrumentation project which was described herein. The use of instrumentation within a prototype of this kind of foundation on the very soft clayey deposits of Mexico City has allowed, when about 80% of the structural load has been applied at the end of 1995, to learn about the load sharing mechanism between piles and raft foundation. These initial measurements would indicate that other design approaches could be implemented in the future where the load carrying capacity could be considered as being shared by piles and raft foundation; this evidently would be reflected on possible foundation cost savings.

A full analysis of the response of all geotechnical sensors will be made at the end of construction and will be reported in a subsequent publication, which it could contain information on seismic effects if an event of this type happens.

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