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RESPONSE PREDICTION OF FOUR PILE-SUPPORTED STRUCTURES DURING THE MEXICO EARTHQUAKE OF SEPTEMBER 19, 1985

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SUMMARY

In order to validate a methodology for the seismic pile-structure interaction analysis developed for the response prediction of pile-supported critical structures four buildings that suffered quantifiable moderate damage to either structure, foundation or both during the September 19, 1985 Mexico Earthquake were selected and analyzed. The analyses included the prediction of site specific ground motions, calculation of foundation impedances including soil and pile nonlinear effects, inelastic response calculations of the structures and the comparison of the predicted damage to the observed damage. In general the predictions were very satisfactory and encouraging, particularly in a global sense.

INTRODUCTION

A methodology for seismic pile-structure analysis was developed for use in the response prediction of pile-supported critical structures (Ref. 1). Although the state-of-the-art of nonlinear pile response is discussed extensively in the technical literature, the verification of these methodologies when applied to actual structures is difficult to achieve. Relatively large single piles in-situ or extremely small pile group models for centrifuge testing are the feasible bounds to test the adequacy of these analytical methodologies. The preferred test data against which to verify these methodologies are those provided by strong earthquakes. The September 19, 1985 Mexico Earthquake provided such a realistic data base. Because of the very soft foundation clays structures in Mexico City are generally founded on piles, and there was a spectrum of damaged buildings/foundations such that a positive verification of the proposed methodology could be made.

SELECTION OF TEST STRUCTURES

A detailed screening of more than 1000 buildings was carried out based on several criteria. Buildings preferred for analysis included low-rise buildings with large plan dimensions that would maximize soil-pile-structure interaction; buildings with adequate structural and pile data; and quantifiable moderate damage to either or both foundation and superstructure (totally collapsed buildings were not considered).

In order to severely test the adequacy of the proposed methodology four categories of damage states were defined as shown in Fig. 1. Any methodology to prove itself must be able to predict the response of buildings that sustained quantifiable Structure and Foundation Damage (Type A); Foundation Only Damage (Type B); Structure Only Damage (Type C); and No Damage to Either Structure or Foundation (Type D).

Fig. 2 shows the location of the four buildings in the Roma area of Mexico City that were selected for detailed analysis. Table 1 gives a summary of the structural characteristics of the four selected buildings.

Table 1 SUMMARY OF BUILDING CHARACTERISTICS

Building ID	308	301	68	78
Damage Type	A	B	C	D
Construc. Date	1970	1943	1981	1960
Plan Dimensions	9.0 x 28.5	Irregular	7.65 x 37.3	9.5 x 15
Load Resisting System	R/C Frame with shear walls	R/C Frame	R/C Frame	R/C Frame
Slab Type	Grid	Solid	Block	Solid
No. of Stories	7	7	9	8
Basement	No	Yes	Yes	Yes
Pile Type	Precast Conc. Friction	Timber Friction	Precast Conc. Compensated	Precast Conc. Compensated
No. of Piles/ Length/Diameter	26/24/0.5	168/27/0.3	28/24/0.5	16/16/0.45
Fixed Base First Mode Frequencies	0.62(X)/0.99(Y)	0.72(Y)/0.77(X)	0.57(Y)/1.19(X)	1.00(Y)/1.14(X)
Coupled Struct./ Found. Freq.	0.61(X)/0.90(Y)	0.69(Y)/0.72(X)	0.53(Y)/1.06(X)	0.87(Y)/0.99(X)
Excitation Direc. (Model)	X (2-D)	X, Y (3-D)	Y (2-D)	X, Y (3-D)

RESPONSE PREDICTION PROCEDURE

The methodology used to analyze and evaluate the selected four buildings consists of the following steps: generation of site specific motions; calculation of foundation impedances including nonlinear effects; modeling of the structure and obtaining its inelastic response; and assessment of predicted damage to that reported from the field.

Site Specific Motions The nearest Lake-zone recorded motion was obtained at the Secretaria de Communicaciones y Transportes (SCT). Although initially attempts were made to use the motions recorded at Tacubaya (TACY) as rock outcrop motions, the final results given here are based on the SCT motions as the starting point.

Soil profiles for the development of the final ground motions are based on the stratigraphy and soil parameters obtained as a result of field and laboratory soil tests performed for this study at the SCT and at the selected building locations. The site investigations included soil borings and cone penetrometer soundings, crosshole shear wave velocity tests, and selected laboratory soil tests on undisturbed samples. Fig. 3 shows the best estimate soil profiles and shear wave velocities as obtained by the geophysical methods listed above.

The objective was to develop a motion at the base (at 80m depth) of the SCT soil profile and then to generate site specific motions at each of the building sites. It was determined that the Profundo must be included in the model. The fact that the base motion will be used to amplify motions through several other profiles than the one used in obtaining it, mandates that its characteristics be as reliable as possible. Therefore, the commonly used methods for soil amplification/deamplification studies where the amplitude and frequency characterization is conducted in one pass had to be modified. For this study a two-step solution was adopted. Specifying a uniform response spectrum at the base of a one-dimensional soil column to a depth of 80m at SCT the frequency characterization of the soil profile was determined by comparison with the SCT recorded data. Recorded ground motions are a function of the recording instrument orientation. Dissimilar motions in two orthogonal directions are not consistent with the basic one-dimensional soil model. And therefore, the ground motion predictions are based on the use of the most correlated set at SCT (Ref. 2). Spectra from three instrument orientations are shown in Fig. 4.

Studies of the response of the soil profile at SCT demonstrated that the spectrum peak at 0.495 Hz (2.02 sec) is dominated by the response of the upper clay layer. In order to match this frequency, it was necessary to modify the shear wave velocity of the upper clay only to 45% of the actual geophysically measured values. These modified values (about 60-70 m/sec) are in general agreement with other published data for the Valley of Mexico. This approach in effect uses the actual earthquake to obtain a strain-dependent shear wave velocity for the upper clay layer. Similarly, the spectrum peak at 0.375 Hz (2.67 sec) was found to be dominated by the properties of the Profundo. In this case, a good match with the recorded frequency was achieved using the low-strain dynamic properties of the Profundo without any modification. The calculated response spectra for the four sites are shown in Fig. 5. The differences among the motions obtained as a result of variations of layer depths and shear wave velocities in the upper clay are to be noted. Of particular note are (compared to SCT): a) relatively broader frequency spectrum, b) higher peaks at about 0.59 Hz (1.7 sec) and 0.65 Hz (1.5 sec), and c) significant amplification of the peak at about 1.5 Hz.

Foundation Impedances Structural response beyond an initial low-level response stage may result in moderate to large levels of strain in the soil surrounding the piles and possible pile stiffness degradation due to concrete cracking. An approximate method for considering these effects was developed for this study (Ref. 3). In essence the method considers the soil nonlinear effects on static, single-pile response in terms of p-y curves. Concrete stiffness degradation is considered iteratively by checking the bending moment along the pile with the ultimate capacity of the corresponding pile cross-section. The extent of static nonlinear soil response at the estimated load level is then used as a guide to modify the soil properties in computing the pile group impedance functions. Soil nonlinearity can be considered by means of a weakened cylindrical zone around the pile. The geometry of the weak zone and modified soil properties are selected based on the amount of soil nonlinearity observed from the static analysis described above. The impedance functions so calculated are not only frequency-dependent, but also loading-dependent. For subsequent response calculations impedances at system frequencies are determined iteratively.

Structural-Foundation Response When dealing with buildings in Mexico City consideration of structure and pile nonlinear response become essential to the understanding of their performance during this or other significant earthquakes. Mexico City earthquakes are characterized by a relatively narrow frequency band and hence it becomes imperative that the structure-pile system eigenparameters be defined in an accurate fashion. Bechtel's version of DRAIN 2D (Ref. 4) is used. Since BDRAIN is applicable to two-dimensional (2-D) structures a method was developed to simplify a three-dimensional (3-D) structure to a 2-D model (Ref. 3) when the elastic analysis indicates considerable overstress in structural members. Otherwise the 3-D linear elastic analysis is used to assess the damage.

The most critical direction of seismic loading is determined from a detailed 3-D model and the specified free field ground motion. The 2-D nonlinear model in the critical direction is then constructed by compressing the out-of-plane direction. Each member of the 2-D frame possesses the cumulative properties of all the members it represents. A static analysis including dead load and estimated equivalent seismic loads is performed on the 3-D model. The results of this analysis are then used to determine the expected loading on the columns. These loadings are used, along with the appropriate moment-interaction diagrams, to calculate the moment capacity for each of the columns in the 3-D model. The moment capacities of the columns in the 2-D model are then calculated by simply adding all the capacities of the appropriate individual columns from the 3-D model.

Damage Assessment Structural damage is assessed by identifying first the beams and columns which experienced yielding during the earthquake according to the nonlinear analysis. Then, the damage is quantified by calculating the required maximum member ductility for each of those beams and columns. The required member ductilities thus calculated are compared with the available ductilities before collapse. The analytically predicted damage is then compared with the reported actual damage during the earthquake.

RESULTS FROM FOUR STRUCTURES

Building 78 (Damage Type D) Only a 3-D elastic analysis was carried out since the results did not indicate the need for a 2-D inelastic analysis. Observed minor damage was consistent with the damage prediction. All facilities (electric, hydraulic and sanitary) and water and gas tanks were operational postearthquake.

Building 68 (Damage Type C) A 2-D inelastic analysis was required for this building. Observed damage to this building included extensive cracking in some beams and columns which is in general agreement with the prediction. No foundation failure was observed consistent with the prediction.

Building 301 (Damage Type B) The linear elastic analysis predicted some limited structural distress in a few beams and columns. The structure experienced some noticeable cracking in the ground floor beams. A postanalysis inspection revealed the existence of a ramp in the basement that was not shown on the drawings. This postdesign modification would particularly modify the damage prediction of some of the basement beams. More importantly though the analysis failed to predict the tilting of the structure during the earthquake. The building has a history of continuous tilting that was incremented noticeably during this earthquake. The long-term settlement could be the result of the negative skin friction that had developed during 44 years of upper clay subsidence in Mexico City. The consideration of this negative skin friction would have predicted the observed tilting during this earthquake.

Building 308 (Damage Type A) A 2-D inelastic analysis was required for this building. the damage to beams and columns in general follow the prediction except the first floor front east side columns.. The structure tilted towards the West 0.6m. Although a large tilt was predicted, an actual estimate of the tilt is not possible without a detailed evaluation of the relative subsidence of the soils beneath the structure.

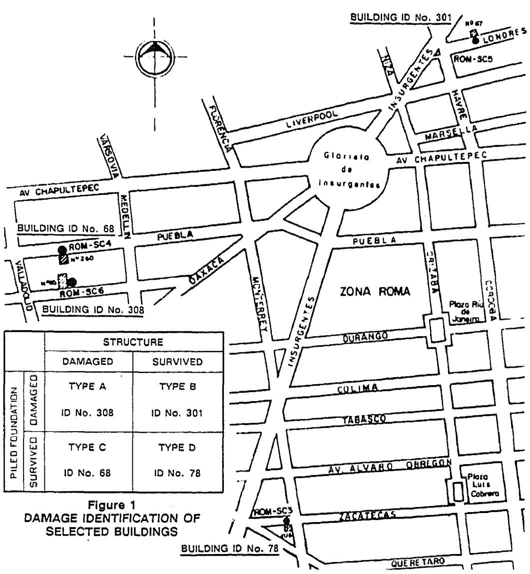
CONCLUSIONS

Except for the failure to predict the tilt of Building 301, which appears to be due to negative skin friction, the predictions of both structural and foundation behavior and extent of damage have been very satisfactory and encouraging, particularly in a global sense. Given the fact that the nonlinear analysis is performed on an aggregated 2-D model, an element-by-element comparison of the predicted with the actual damage is not possible. Furthermore, structural modifications are not properly documented which could well affect the prediction of localized behavior.

It is interesting to mark the system frequencies listed in Table 1 on Fig. 5. These are indicated by vertical lines and identified by X or Y axes of the buildings. The essentially no damage in Building 78 is clearly corroborated by the location of the building frequencies. Building 301 is beginning to enter the strong shaking range, which could explain the tilt increment during this earthquake. Building 68 is a candidate for damage which would help steer the structure out of the danger zone. And finally, Building 308 is definitely poised for intense shaking. This is the building that suffered both foundation and structural damage. Thus, the elements for a successful response prediction are detailed ground motion definition and structure-foundation system frequencies considering nonlinear structure and foundation characteristics.

REFERENCES

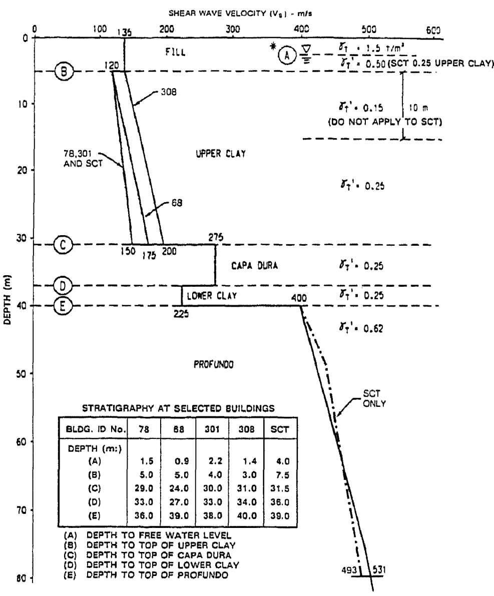
1. Hadjian, A. H., Principal Investigator, "A Verification of Seismic Pile Design Methodology Through the Mexico Earthquake Experience," Bechtel Final Report (18499) for the Tokyo Electric Power Co., Inc., (1987).
2. Hadjian, A. H., "On the Correlation of the Components of Strong Ground Motion - Part 2," Bulletin of the Seismological Society of America, pp. 1323-1331 (1981).
3. Hasegawa, N., Oshima, Y., Hadjian, A. H., Lau, L., and Fallgren, R. B., "Modeling for Nonlinear Response Prediction of Pile-Supported Structures," Proceedings 9WCEE, Tokyo-Kyoto (1988).
4. Porter, F. L. and Powell, G. H., "Static and Dynamic Analysis of Inelastic Frame Structures," Report No. EERC 71-3, Earthquake Engineering Research Center, Univer. of California, Berkeley, (1971).



		STRUCTURE	
		DAMAGED	SURVIVED
PILED FOUNDATION	DAMAGED	TYPE A ID No. 308	TYPE B ID No. 301
	SURVIVED	TYPE C ID No. 68	TYPE D ID No. 78

Figure 1
DAMAGE IDENTIFICATION OF SELECTED BUILDINGS

Figure 2
ROMA AREA BUILDING LOCATIONS



STRATIGRAPHY AT SELECTED BUILDINGS

BLDG. ID No.	78	68	301	308	SCT
DEPTH (m:)					
(A)	1.5	0.9	2.2	1.4	4.0
(B)	5.0	5.0	4.0	3.0	7.5
(C)	29.0	24.0	30.0	31.0	31.5
(D)	33.0	27.0	33.0	34.0	36.0
(E)	36.0	39.0	38.0	40.0	39.0

(A) DEPTH TO FREE WATER LEVEL
(B) DEPTH TO TOP OF UPPER CLAY
(C) DEPTH TO TOP OF CAPA DURA
(D) DEPTH TO TOP OF LOWER CLAY
(E) DEPTH TO TOP OF PROFUNDO

Figure 3
BEST ESTIMATE SHEAR WAVE VELOCITIES FOR ROMA AREA BUILDINGS AND SCT

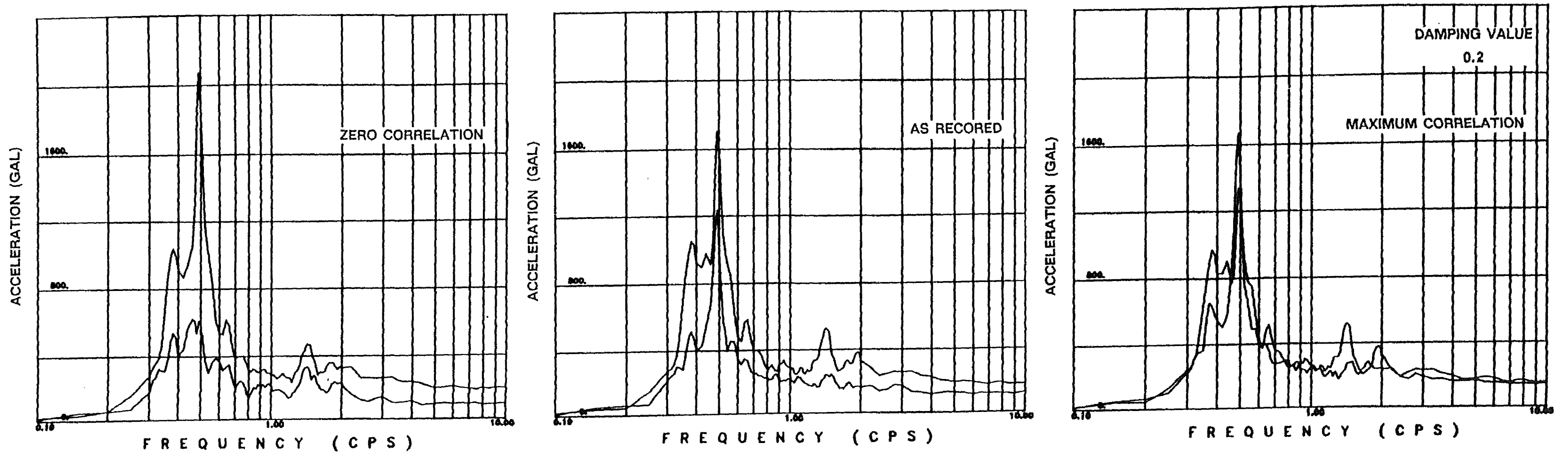


Figure 4
ACCELERATION SPECTRA FOR SCT1

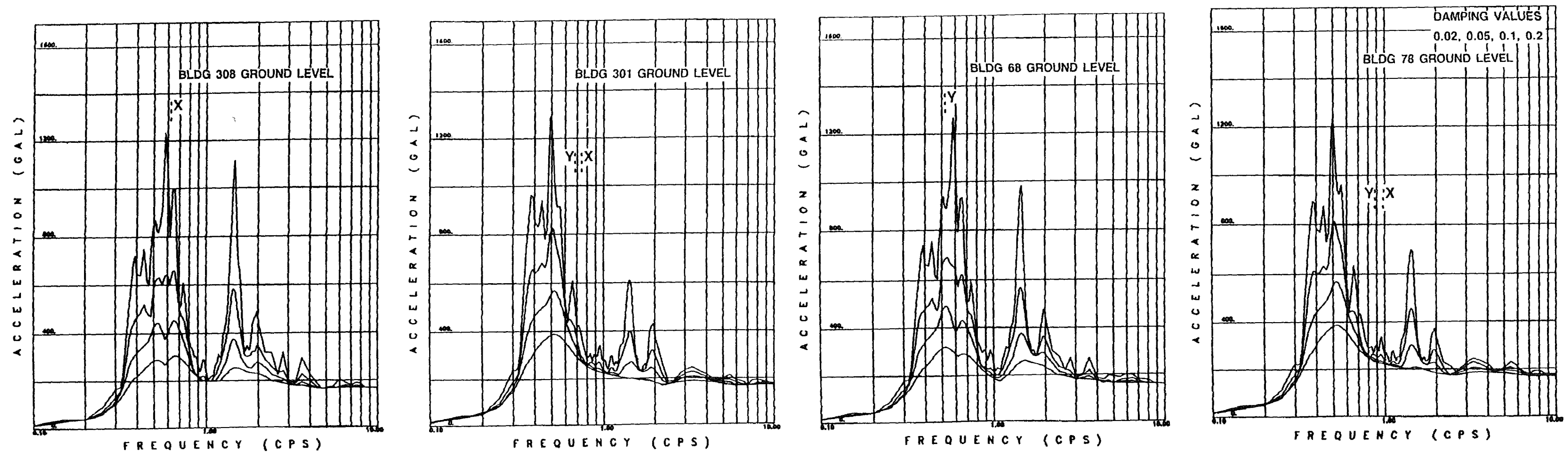


Figure 5
ACCELERATION SPECTRA FOR 4 BUILDINGS