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GROUND EFFECTS CAUSED BY THE ALTO MAYO EARTHQUAKES IN PERU

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ABSTRACT: On May 29, 1990 and April 4, 1991, two moderate earthquakes occurred in the north-eastern region of Peru. Despite their relatively low magnitudes, the severity of the damage was high because of the existing type of construction and soil conditions in the populated areas. The region is located in the peruvian upper jungle, with high precipitation.

Sedimentary rocks from the Jurassic to Cretaceous Periods are found in the nearby mountains and Quaternary materials in the Alto Mayo river valley. Quaternary deposits are composed of alluvial, colluvial, fluvial and residual soils. Moyobamba and Rioja are the most important cities in the area. The region is crossed by the Mayo river, whose banks are composed of liquefiable sand deposits. The following earthquake ground effects have been reported: soil liquefaction, instability and soil erosion in the slopes, differential settlements, soil amplification and landslides within the epicentral area. A geotechnical exploration program was undertaken in Moyobamba, Rioja and Soritor cities. The earthquake ground effects are described and a brief summary of the geotechnical characteristics of selected areas in the principal cities is presented.

1. INTRODUCTION

The Alto Mayo region in northeastern Peru was recently struck by moderate earthquakes. On May 29, 1990 at 9.34 pm (local time) an earthquake with a magnitude of mb=6.0 occurred to the southwest of the city of Rioja. This quake caused 70 deaths and damaged 6000 houses of a total of 20000 existing in the epicentral area. Most of the houses were constructed with adobe and tapial. A maximum intensity of VII MMI was observed in Soritor (Alva-Hurtado et al 1990; Huaco et al 1990; Torres et al 1990).

Beginning April 4, 1991, a series of shocks was triggered in the same region. The largest event occurred at 11.30 pm (local time) with a magnitude of mb=6.5. Its epicenter was 30 Km northeast of Moyobamba, near Angaisa mountain. The death toll was 40. Heavy damage was caused to property in the provinces of Moyobamba and Rioja. Intensities of VII MMI were observed in Moyobamba, Yantalo and Nuevo Cajamarca. Several persons saved their lives because they spent the night in their back yards or "tambos", due to the alarm produced by the precursor shocks that occurred earlier in the day (Cuadra & Chang 1991).

The Japan-Peru Center for Earthquake Engineering Research and Disaster Mitigation (CISMID) undertook a series of studies in the affected area. Several reports on field reconnaissance, damage evaluation and seismic microzonation of the cities of Moyobamba, Rioja and Soritor have been produced. In this paper a summary of the earthquake ground effects and the geotechnical characteristics of selected areas of the main cities are presented (Chariarse et al 1991; Cuadra et al 1991; Fukumoto et al 1991; Lara 1992).

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2. SEISMIC HISTORY OF THE AFFECTED REGION

The seismic history of the area is rather scarse, due mainly to the isolation of the populated areas and lack of good communications. Silgado (1978) has published the seismic history of the peruvian territory from the 16th century to the present time. The most important earthquakes that affected the region under study are described (Alva-Hurtado et al 1984):

- November 26, 1877. The citiy of Chachapoyas suffered a strong ground motion. Intensity of VI MMI.

- September 28, 1906. Seismic intensity of VII MMI was assigned to Chachapoyas. The magnitude of the earthquake was Ms=7.5.

- May 14, 1928. A noticeable seismic event occurred in northeastern Peru. Chachapoyas suffered almost completed destruction. A landslide in Pinpincos (Chamaya Valley) caused 25 deaths. The magnitude of the earthquake was Ms=7.3. Maximum intensity of X MMI in the epicentral area.

- August 6, 1945. A strong earthquake affected the departments of San Martin and Amazonas. A seismic intensity of VI MMI was reported for Moyobamba. The epicenter was located to the east of Moyobamba. Soil liquefaction occurred in Shango, Tahuisco and Azungue ravines.

- June 19, 1968. A strong earthquake struck the northern part of the department of San Martin. causing the death of 15 people. The magnitude assigned to the earthquake was Ms=6.9 and mb=6.4. Heavy damage in the cities of Moyobamba and Yantalo. The epicenter was located northwest of Moyobamba. Soil liquefaction along the Mayo river banks and around Moyobamba. Maximum intensity of IX MMI.

3. GEOLOGICAL SETTING

The region belongs to the subandean zone of northern Peru. The rocks outcropping around the Alto Mayo valley are sedimentary continental and marine rocks from the Jurassic to Cretaceous Periods and continental rocks of the Tertiary. These rocks have tectonic folding and averthrusting. The valley has quaternary deposits of fluvio-lacustral origen, overlying the rocks. Test boreholes in the valley have indicated the existence of peat deposits of 20 meters depth.

The type of faulting in the area corresponds to folds and high angle thrust faults that form imbricated systems. These faults have less dip with depth, producing a thrust and fold belt structure. Several of these faults have visible traces and evidence recent activity. Valley scarps can be to the west of the Alto Mayo, as well as longitudinal valleys and displaced morphological units, typical of active transcurrent faults. Also, to the north and south of Moyobamba, rectilinear scarps can be seen that could correspond to active normal faults (Martinez & Machare 1991).

The surface geology, as well as the fault traces and epicenters of the 1990, 1991 and past earthquake are shown in Figure 1.

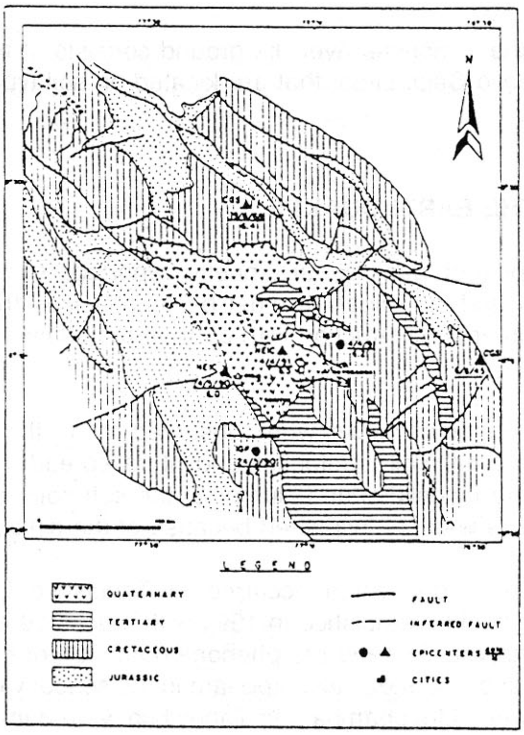

Figure 1. Geological and seismotectonic map of the Alto Mayo region.

4.0 GEOTECHNICAL CHARACTERISTICS

With respect to the geotechnical characteristics of the area investigated, four very distinct types can be distinguished (Kuroiwa & Deza 1968; Martinez-Vargas 1969):

Type I Rocky ground.- The rocks are of sedimentary type: sandstones and siltstones. These rocks aurround the Alto Mayo valley. They are indented because of the dentritic drainage existing in the area. The rocks are susceptible, because of their nature and the environmental conditions, to mass movements, such as rocks falls, landslides and slumps.

Type II Residual soils.- They are located over the parent rock and are the product of weathering. In general they have high to medium density (coarse soils) and high to medium stiffness (fine soils). These soils exist in the elevated parts of the cities of Moyobamba and Rioja. The residual soils have medium bearing capacity. There are slope stability problems when the slopes are high, with a steep slope angle and low clay content.

Type III Dense quaternary soils.- They consist of colluvial and alluvial soils with good drainage. Some colluvial soils and terraces contain blocks and gravels in a sandy matrix. These soils are located in the afluent valleys of the Mayo river, in the mountainous areas and in the superficial layers of low gradient terrain. In general, they have good bearing capacities.

Type IV Soft quaternary.- These are mainly saturated fine sands located on the banks of the main rivers and ravines. They are also the interlayered clays, peats and sands of lacustral origen that form the extense plateau of the Alto Mayo valley. In general, this type of soil has poor geotechnical behavior.

The city of Moyobamba was originally built on a stable plateau constituted by residual soils type II. The slopes around the city have erosion problems. The lowlands in Moyobamba, such as Tahuishco, Shango and Azungue have ground type IV. A similar situation exists in the city of Rioja where highlands are residual soils and lowlands are soft soils. However, in Rioja the slopes are more stable. Soritor is located on the right bank of the Tonchima river. Its ground consists of type III soils. There are other cities in the area, such as Nuevo Cajamarca, that are located on soil type IV.

5. GROUND EFFECTS OF THE EARTHQUAKES

The geotechnical types of damage are briefly reported, such as: ground cracking, soil liquefaction, soil amplification and landslides. It must be emphasized that most of the damage had structural origin, i.e. the design and construction with earthen materials. However, those damages will not be included in this paper.

Ground Cracking.- Tension cracks were observed in 1) the crest of the slopes of the Moyobamba plateau, associated with soil liquefaction and lateral spreading, 2) the highways, as tension zones that could develop future landslides and slump, 3) the soft soils in the Mayo river banks. The epicentral area in the mountains was not inspected because of the difficulty in getting there.

Soil Liquefaction.- Soil liquefaction occurred in Tahuishco Port in Moyobamba. Lateral spreading developed in the school in Tahuishco in 1991 with cracks 10 cm in width and 50 cm deep. One classroom floor was destroyed. In 1990 the phenomenom did not reach the school building, but did occur in the school yard; sand volcanoes also appeard in the school yard. During both earthquakes, segments of the highway between Moyobamba and Tahuishco were damaged. In Azungue, located in the lowlands of Moyobamba, ground cracks and lateral spreading developed. Cracks 100 m long and 40 cm wide with depths of 1 m were reported. Most of the houses on the slope collapsed. The sewage pumping station and sewage disposal pipes failed. All tapial houses and some masonry houses on soft ground collapsed. In Shango, tapial houses collapsed. Cracks 80 m long and 20 cm scarps were observed. In Miraflores street the cracks were 30 m long and 30 cm deep. During the 1990 earthquake soil liquefaction was reported in El Chorro and Molino Valencia in Rioja, also in Segunda Jerusalen-Azunguillo, Negro river and La Conquista.

Soil Amplification.- Considering the geotechnical characteristics of the soft soils consisting of clays, peat and silts in the lacustral and fluvial deposits of the Alto Mayo valley, it is suggested that local soil conditions have played a major role in the damage of structures as well as the ground cracking. Soil amplification could have occurred in Nuevo Cajamarca, Naranjos, Segunda Jerusalen, San Fernando, Yuracyacu, etc.

Landslides.- Several types of mass movements were produced during the 1990 and 1991 earthquakes.

Rockfalls.- They occurred on steep slopes having elongate prints, with heights of 50 m and widths of 30 to 40 m in fractured sandstones, limestones and residual soils. Mainly in unpopulated areas.

Translational slides.- These were present in the borders of the Moyobamba plateau, in sandy materials without fines and steep slope. This type was reported to the north and east of Moyobamba, in Punta San Juan, Punta Tahuishco, and Cococho district. The heights were as much as 30 m.

Rotational Slides.- These were observed at Km 500 of the Marginal highway for both earthquakes. The slides is related to a colluvial deposit resting on soft ground. The damage to the highway was 70 cm long.

Soil block slide.- In Punta Tahuishco this type of slide was observed. The block consists of silty sands 60 meters wide. The crack was 5 cm maximum width and 7 cm scarp.

Lateral spread slides.- They developed in areas of low gradient, where the subsoil consists of sand and silt and soil liquefaction was evident. Tahuishco and Azunque (Moyobamba) reported such slides.

The ground effects previously described, that were caused by the Rioja (1990) and Moyobamba (1991) earthquakes in the Alto Mayo region, are depicted in Figure 2, which is an update of the figure presented in Monge (1990). In figure 3 are presented the earthquake ground effects in the city of Moyobamba. The subsoil in the lower parts of the city, such as Tahuishco, Azungue and Shango consists of fine sands and silty sands with low relative densities and high water level. The soil in the slopes is constituted mainly by clayey and silty sands with medium densities and relatively low water table, whereas the ground in the elevated part of the city (plateau) consists of clays and clayey sands of medium to low bearing capacities and deep water table. Seismic intensities in the lower part where two degrees higher than in the elevated part of the city of Moyobamba.

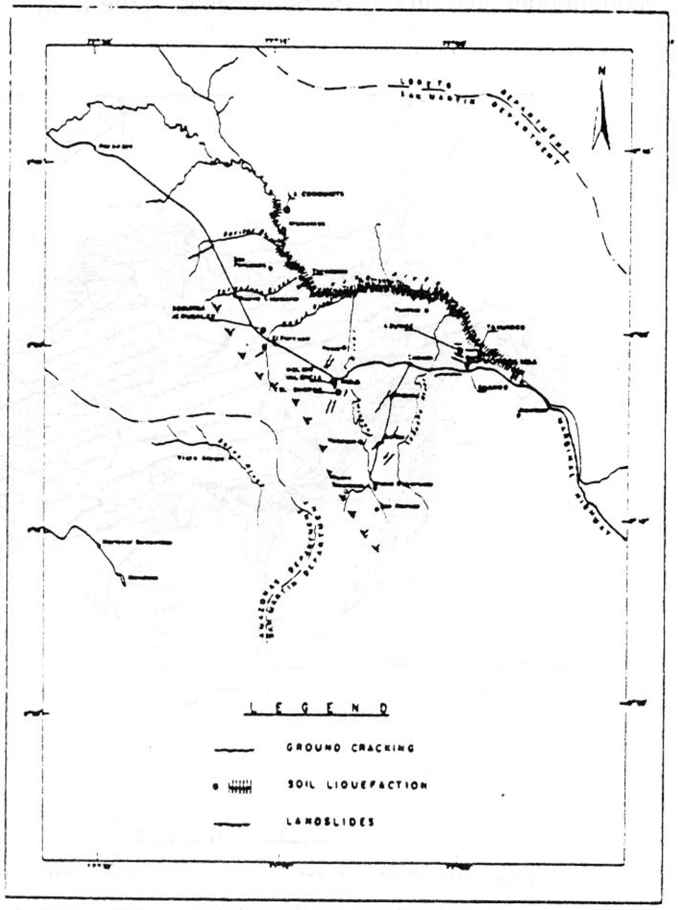

Figure 2. Earthquake ground effects in the Alto Mayo region.

6. CONCLUDING REMARKS

1. The seismic history of the Alto Mayo region indicates that the seismicity is very high. It is necessary to undertake detailed studies of the active faults existing in the area.

2. The earthen type of construction prevalent in the affected zone, adobe and tapial, is the reason for the heavy damage; however, the ground characteristics also contributed to the damage. There is a direct relationship between damage level and soil conditions.

3. In the lowlands of Moyobamba and Rioja, with loose sands and high water table, soil liquefaction occurred and contributed to the building damage.

4. In the lacustral area, with soft soils, such as Nuevo Cajamarca, the phenomenom of soil amplification developed.

5. Slope inestability was generated during the 1990 and 1991 earthquakes in the Alto Mayo region. In Moyobamba and Rioja soil slumps were produced, in connection with problem of erosion. Landslides in the high mountains did not affect the population. Rotational slides damaged highway and canals.

6. During the earthquakes no strong motion records were obtained; there were no nearby seismographs. It is recommended instruments to be installed in the area.

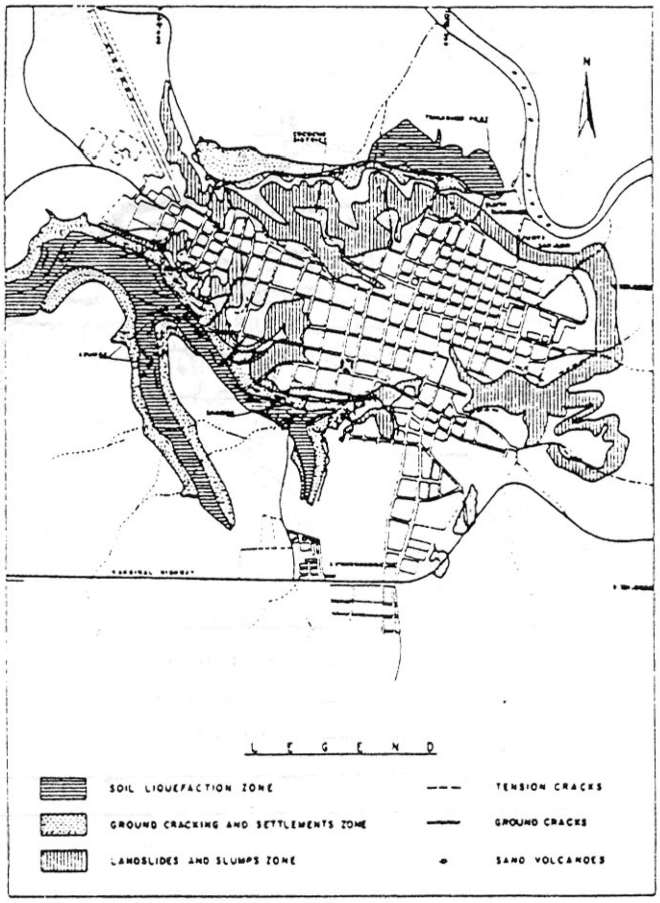

Figure 3. Earthquake ground effects in Moyobamba city

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PSEUDO DYNAMIC TESTS OF CONFINED MASONRY BUILDINGS

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ABSTRACT: Pseudo dynamic tests were carried out to investigate the behavior of confined masonry structures. Two-story one-bay specimens were used for the tests here reported. One full scale specimen and one half scale model-were built following current practices. The half scale model was identical to two other specimens previously tested statically and in a shaking table. This paper summarizes results from the pseudo dynamic tests and compares them with those obtained using different experimental techniques. There is reasonable agreement between test results; however, pseudo dynamic tests tend to give lower strength values due to stress relaxation.

1. INTRODUCTION

Most housing units in urban areas of Latin America are masonry structures. With few exceptions, four to five story masonry buildings have been used for all large housing projects of the past decades. Since this trend is expected to continue, a substantial part of the work at research institutions in the region deals with masonry.

This paper summarizes results from tests of confined masonry structures made at the Japan Peru Center for Earthquake Research and Disaster Mitigation (CISMID) of the National University of Engineering, in Lima, as part of a joint research program with the Pontificia Universidad Católica del Perú (PUCP). The term confined masonry is used here to denote a building system with clay brick load bearing walls confined by reinforced concrete elements.

The main objetive of the research program was to compare different testing techniques to simulate earthquake loads.

2. TEST PROGRAM AND PROCEDURES

2.1 Specimens

Two-story one-bay specimens, with two parallel walls connected by stiff horizontal slabs, were used for the tests. Design was based on Peruvian standards for confined masonry. One full scale specimen (tested at CISMID) and 3 approximately half scale models (two of them tested at PUCP and one at CISMID) were built.

Figure 1 and Table 1 show the dimensions of the specimens. Slab dimensions were not in the ratio 1:2. This was corrected by adding masses to the full scale specimen, in order to have the same vertical stress. Scale factors were 2 for displacement, 1 for acceleration as for strain, angular distortion, stress and elastic moduli, $\sqrt{2}$ for time, 4 for mass and force.

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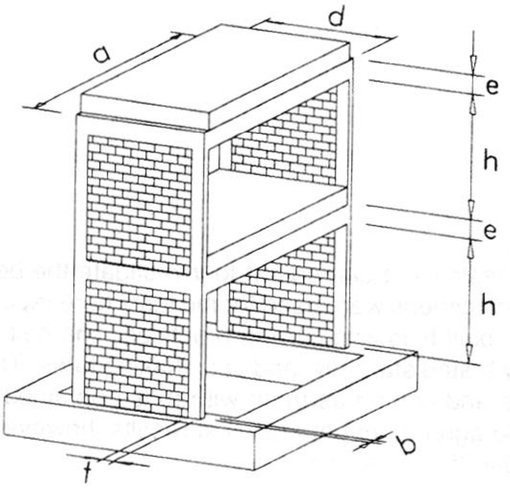

Figure 1. Test specimen.

Table 1. Specimen dimensions (cm)

	a	d	h	e	b	t
Full scale	247	214	210	20	25	14
Half scale	247	107	98	15	10	7

The mass of the full scale specimen was 15,26t (including added masses of 2,54t on each slab), with a ratio of 4 with respect to the original half scale model. However, operational limitations of the shaking table used in one of the tests made it necessary to decrease the fundamental frequency of the half scale models. Additional masses of 1,42 t were added for that purpose at each floor level. The same masses were added to the other half scale models tested with different techniques.

2.2 Materials

Nominal dimensions of the clay brick units used in the full scale specimen were 140 mm x 140 mm x 280 mm, with a net area of 61%. They were laid with a 1:4 (cement:sand) mortar, with an average joint thickness of 10 mm.

The units used for half scale models were cut from 60 mm x 120 mm x 250 mm bricks using a mechanical saw. Final dimensions of the cut units were 60 mm x 70 mm x 120 mm, with a 66% net area. The average joint thickness was 5 mm. The mean prism strength was 10,8 MPa.

The concrete used for footings, columns and slabs had a nominal strength fc of 20 MPa.

Columns were reinforced with 4#3 longitudinal bars and stirrups #2 at 14 cm, except near the joints, were a 10 cm spacing was used. The yielding stress of the steel was 410 MPa. In the half scale models the columns were reinforced with 4#2 bars; stirrups had a net area of 12 mm and a yielding stress of 200 MPa.

2.3 Test Program

The joint CISMID-PUCP research program included:

1. Static test of half scale model under monotonic loading, with equal horizontal forces at both floor levels.

2. Shaking table test of half scale model, with harmonic base acceleration of varying amplitude.

3. Pseudo dynamic (PD) tests of one full scale specimen and one half scale model.

This paper deals mainly with the PD tests conducted at CISMID; results from preliminary forced vibration tests are also presented. Results from the static and shaking table test conducted at PUCP have been reported by San Bartolomé et al. (1991); some of their findings are quoted here for purposes of comparison.

3. FORCED VIBRATION TESTS

3.1 Equipment and Instrumentation

Steady-state resonance tests were performed using a small rotating eccentric weight exciter. The exciter was set at the center on the top slab of the specimen being tested, producing a horizontal sinusoidal force, parallel to the walls.

Four accelerometers were located on the slabs, near the center of each wall, with their sensitive axis horizontal, parallel to the walls. The analog signals were filtered with 100 Hz low-pass filters and measured with a digital storage oscilloscope.

3.2 Natural Periods, Damping and Modal Shapes

Figure 2 shows typical resonance curves, obtained for the full scale model. Damping was estimated by considering the specimen as a one degree of freedom system and using the bandwidth (half power) method.

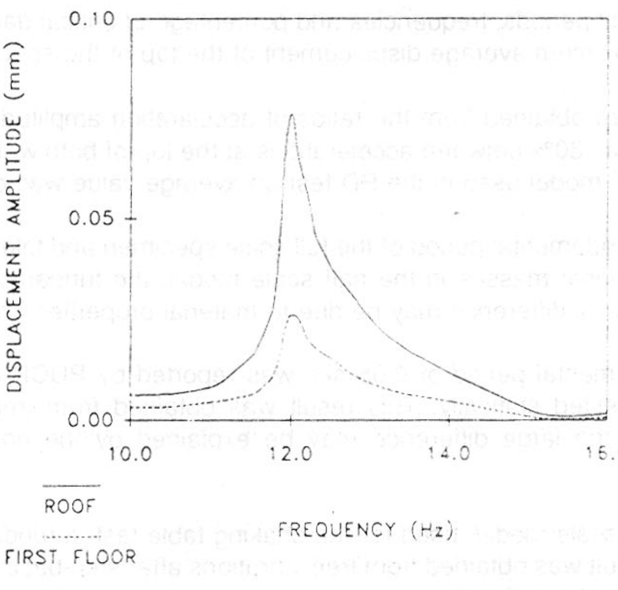

Figure 2. Resonance curves for full scale specimen

Table 2. Natural periods, frequencies and damping

	f (Hz)	T (sec)	(%)	u max (mm)
Full scale specimen				
First mode	12,5	0,080	1,5	0,038
Second mode	28,3	0,035		0,021
Half scale model				
First mode	13,5	0,074	1,5	0,097
Second mode	18,5	0,054		0,018

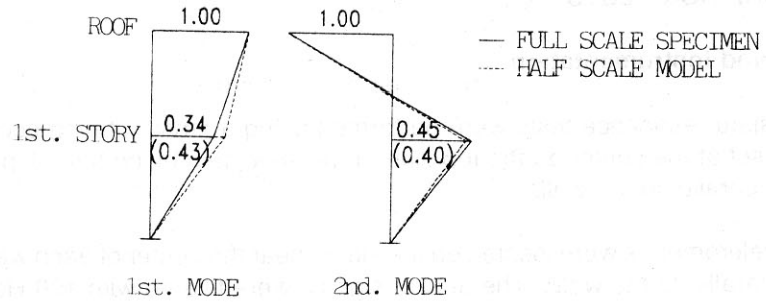

Figure 3. Modal shapes.

Table 2 lists natural periods, frequencies and porcentage of critical damping of the specimens tested at CISMID. The maximum average displacement at the top of the specimen is also tabulated.

Modal shapes were obtained from the ratios of accelaration amplitudes at resonance. There were differences as large as 30% between accelerations at the top of both walls; to be consistent with the hypothesis of a 2 DOF model used in the PD test an average value was considered.

Considering the fundamental period of the full scale specimen and taking into account the time scale factor and the additional masses in the half scale model, the fundamental period of the latter should be 0,077 sec. The 4% difference may be due to material properties and workmanship.

An average fundamental period of 0,05 sec was reported by PUCP researchers for the half scale model which was tested statically. This result was obtained from free vibration tests at the microtremor level; hence the large difference may be explained by the nonlinear behavior of the specimens.

For the other half scale model, used for the shaking table test, a fundamental period of 0,062 sec was reported. This result was obtained from free vibrations after step base motions, with maximum base accelerations of the order of 0,1 g.

4. PSEUDO DYNAMIC TESTS

4.1 Implementation at CISMID

Pseudo dynamic tests combine analytical and experimental techniques. They can be considered as a numerical integration of the equilibrium differential equations, except that restoring forces are directly measured from the specimen rather than computed using a mathematical model.

Different systems interact during PD tests. At CISMID two separate micro computers control the integration of the equations and the data acquisition. Displacements or loads are applied to the specimen by means of servo hydraulic actuators. Load cells, LVDTs and strain gages are used as transducers. The interaction of computers and mechanical equipment is through 12 bit A/D and 16 bit D/A converters, connected by GPIB.

The software allows for test with an arbitrary number of degrees of freedom. A summed form of the central difference method is used; an analysis of this procedure may be found in Shing et al. (1983).

The tests are usually controlled by "external" LVDTs attached to the specimen. Since the actuator displacement ("internal" LVDTs) may be different from that the specimen, the software provides for a correction, imposing target displacements slightly different from those obtained from numerical integration. To avoid instabilities, the correction is numerically damped by averaging the difference between internal and external LVDT readings with the correction considered in the previous iteration.

There is good agreement between theoretical and PD test results. For example, figure 4 compares time histories for the top displacement in the half scale model resulting from a 0,1 sec pulse of 100 gal. The fundamental period from the PD test was 0,075 sec, almost the same obtained with forced vibration tests. The theoretical time history was computed for a 2 DOF system, with the same lumped mass matrix used for the PD test. Viscous damping was assumed as 2,5% of critical. The stiffness matrix was obtained from previous measurements of flexibility coefficients.

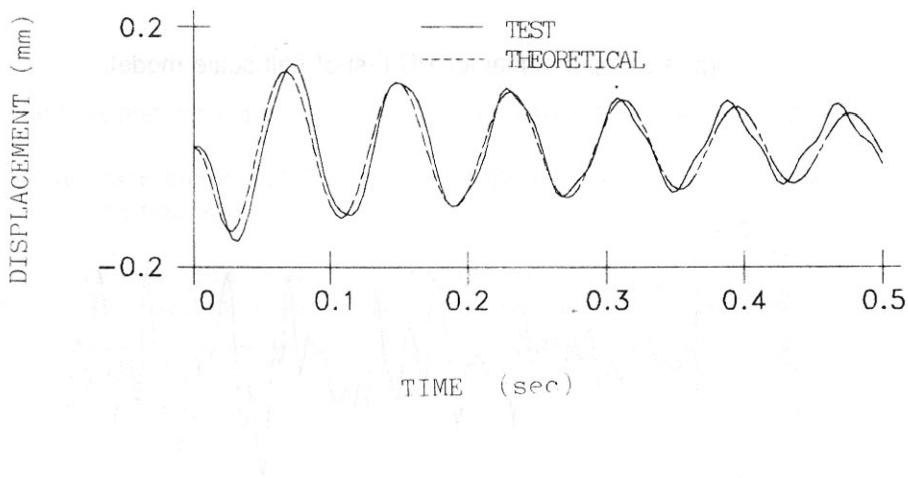

Figure 4. Experimental vs theoretical results.

However, it should be pointed out that when running a PD test with an earthquake record large relative errors occur at the beginning of the test, while displacements are small, sometimes of the same order of magnitude of the digitizing error.

4.2 PD test conditions

The specimens were modelled as 2 DOF systems with lumped masses. Perfect symmetry was assumed, hence the degrees of freedom were associated to the horizontal displacements at the center of each slab.

Preliminary tests were conducted to obtain flexibility matrices of the specimens. For this purpose a small load, of the order of 1 kN, was applied at one of the floor levels while the other floor level was free. The fundamental periods computed from these results were 0,086 sec for the full scale specimen and 0,067 sec for the half scale model.

Viscous damping was introduced in the form of a matrix proportional to initial stiffness, with 1,5% of critical damping for the first mode, and remained unchanged during the test.

For the PD test of the half scale model the input signal was the same used for the shaking table test. It consisted of a series of 5 Hz sine waves with different amplitudes (figure 5). During the first stage of the test, while the specimen (with a 13,5 Hz fundamental frequency) had little damage, this 5 Hz base motion was equivalent to a static loading. The maximum accelaration in the input signal is 1,3 g, although the specimen failed during the stage with maximum acceleration of 1,06 g. The integration time interval was 0,004 sec. The total duration of the test was about 16 hours.

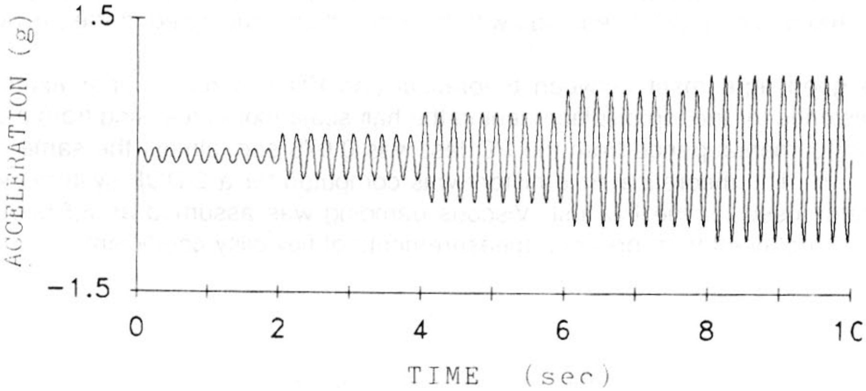

Figure 5. Input signal for PD test of half scale model.

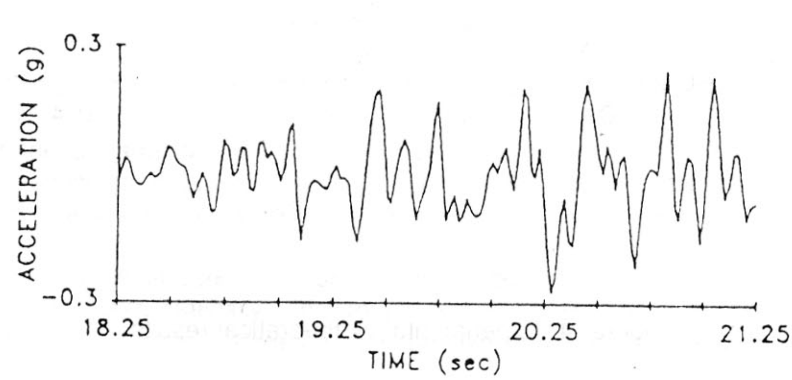

Figure 6. Input signal for PD test of full scale model.

The full scale specimen was tested with a ground acceleration corresponding to the NO8E component of the Lima earthquake of October 10 1966. A portion of the record, from 18,25 to 21,25 seconds, which contains the peak acceleration, was selected (figure 6). The test was repeated four times, scaling the record as required to have a maximun ground acceleration of 293,6 gal (original record), 400, 800 and 1200 gal. The integration time interval was 0,004 sec. The average duration of these tests was 12 hours.

4.3 Test Results for half Scale Model

Thin cracks were observed at the base of the walls from the beginning of the test. Diagonal cracks developed during the second stage and became increasingly important after 4 seconds. A large strain increment in the longitudinal steel reinforcement of the columns occurred at the same time. The failure mode was by shear, involving both masonry units and mortar joints. Diagonal cracks were also observed in the second level, in contrast with what was reported for the static and shaking table tests. The higher damage in the second level was clearly related with construction defects in one of the walls.

A plot of base shear versus first story displacement is shown in figure 7. The behavior was almost linear during the first two stages of test, while the story drift angle was less than 1/1000. Stiffness degradation and hysteresis were important from the third stage.

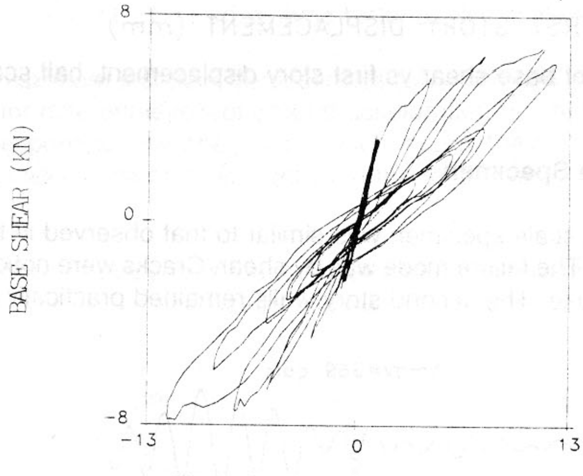

Figure 7. Base shear vs first story displacement, pseudo dynamic test of half scale model.

The maximun base shear was 78,3 kN, corresponding to a 0,0118 angular distortion. The maximun average shear stress was 0,52 MPa.

The relationship between critical damping ratio and displacement amplitude for each cycle is illustrated in figure 8. As expected, hysteretic damping increases with displacement.

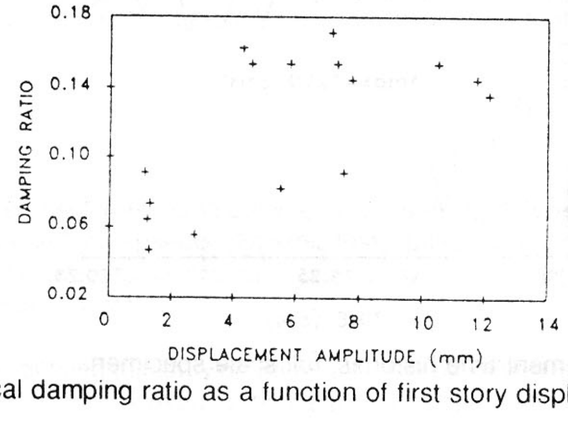

Figure 8. Critical damping ratio as a function of first story displacement amplitude.

Figure 9 compares envelope curves from static and dynamic tests, reported by San Bartolomé et al.(1991), with those from the PD test. Good agreement was found between results of static and shaking table tests. Lower values obtained in the PD test; this may be due to strain rate.

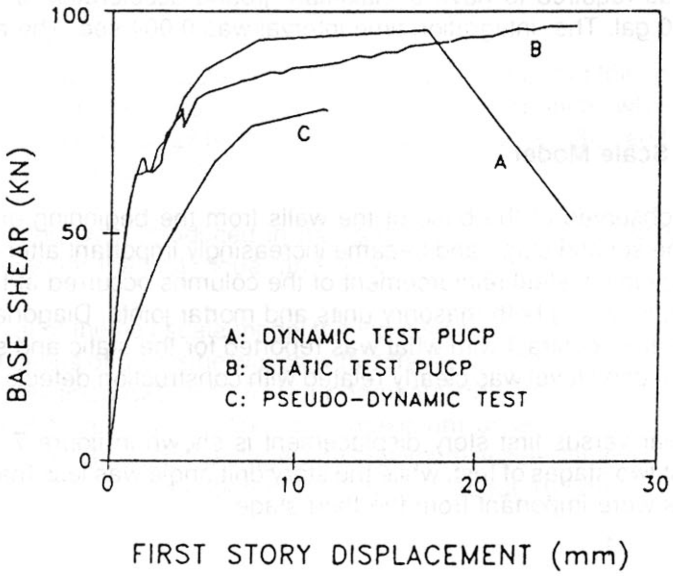

Figure 9. Envelopes of base shear vs first story displacement, half scale models.

4.4 Test Results for full Scale Specimen

The behavior of the full scale specimen was similar to that observed in the static and shaking table tests of half scale models. The failure mode was by shear. Cracks were noticeable in the first story walls after the 400 gal earthquake. The second story walls remained practically undamaged.

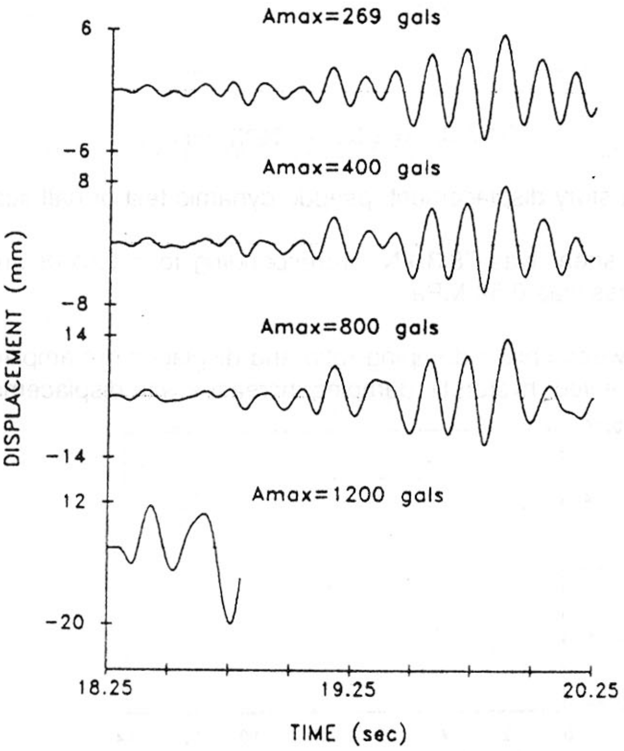

Figure 10. First story displacement time histories, full scale specimen.

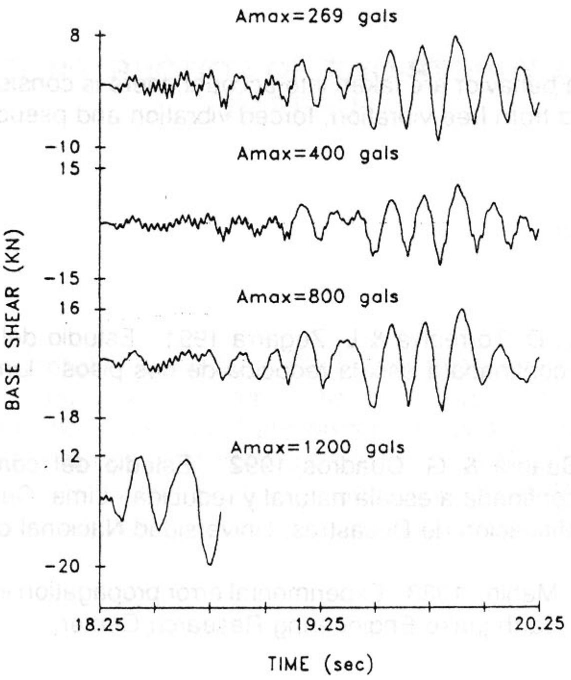

Figure 11. Base shear time histories, full scale specimen.

Table 3 lists maximum first floor displacement (u), maximum base shear (V), and predominant response period (T), for different levels of ground acceleration (a). Although maximum displacements and base shears correspond to only one point of each record, their relative magnitudes and the period elongation reflect the importance of nonlinearities in the response.

Table 3. First floor displacement, base shear and predominant period as a function of maximum ground acceleration.

a max (gal)	u max (mm)	V max (kN)	T (sec)
269	2,5	89	0,14
400	3,6	104	0,15
800	10,8	157	0,19
1200		192	0,28

The specimen failed at an average shear stress in the first level of 0,32 MPa, considerably lower than that reached by the half scale model. The allowable design stress in the current Peruvian code is 0,16 MPa. Part of the difference between experimental results may be explained by the vertical stress (0,23 MPa in the full scale specimen, 0,35 MPa in the half scale model). Other possible cause is the different frequency content of the input signals. Further research is needed to clarify this point.

5. CONCLUSIONS

Reasonable agreement was found between results using different testing techniques, although a lower strenght was obtained from pseudo dynamic tests, possibly because of stress relaxation effects. Additional research is required to incorporate proper corrections while integrating the equilibrium equations in pseudo dynamic tests.

The behavior of confined masonry structures is markedly nonlinear, hence seismic analysis and design procedures should consider material properties consistent with the expected strain level.

If nonlinearities in the behavior are taken into account, there is consistency between results for fundamental periods obtained from free vibration, forced vibration and pseudo dynamic tests.

6. REFERENCES

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