IMPLICATIONS OF STRONG MOTION DATA FOR DESIGN OF REINFORCED CONCRETE BEARING WALL BUILDINGS

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ABSTRACT

A study is made of the performance of two reinforced concrete bearing wall buildings subjected to recent California earthquakes. The buildings are analyzed to verify modeling techniques. The buildings are compared with similar buildings in Chile, and based on the comparison, conclusions are drawn from the Chilean practice regarding likely performance in strong US earthquakes. Design recommendations are made.

INTRODUCTION

Observations of building performances following previous earthquakes have revealed comparatively good performances for reinforced concrete shear wall buildings. These observations extend not only to the combined frame-wall system that has been a popular form of construction in recent years, but also to the bearing wall system in which the walls act as both the vertical and lateral load resisting system. Despite the good performance record of reinforced concrete shear wall buildings, current codes [eg., the Uniform Building Code (UBC) (1)] effectively penalize such buildings in design. A study of the performance of bearing wall buildings during recent earthquakes and of their inherent response characteristics has been undertaken so that more consistent recommendations can be developed.

The study is founded largely on the measured responses and observed performances of shear wall buildings during recent earthquakes. The measured responses have been obtained from two multistory bearing wall buildings located in California that were subjected to low to moderate intensity ground motions. The measured data are used to calibrate analytical models of bearing wall buildings. Having developed confidence in the modeling procedure, responses of several similar buildings subjected to the 1985 Chile earthquake are studied and compared with observed performances. Comparison is made between the Chilean and US bearing wall buildings, and between the Chilean and expected US earthquakes. Conclusions are drawn regarding the expected performance of US bearing wall buildings in the US. Design recommendations consistent with the expected performance are presented.

BEHAVIOR OF TWO BEARING WALL BUILDINGS DURING RECENT CALIFORNIA EARTHQUAKES

Description of the Buildings

The two buildings under study are designated in this paper as Building 1 and Building 2. (These are identified in the CSMIP as CSMIP

Building SN 356 and CSMIP Building SN 385, respectively.) The buildings are each ten stories tall. Plan views of the two buildings are in Fig. 1. The vertical and lateral force resisting system for both buildings consists of reinforced concrete bearing walls coupled by thin slabs. The walls in Building 2 are precast with hollow mandrels into which concrete and reinforcement are cast in the field, apparently achieving an effectively monolithic construction. Slabs in Building 1 are post-tensioned and in Building 2 are precast panels with topping. All walls are continuous over height except within Building 1 for which the two interior corridor walls are discontinued at the sixth floor.

Building 1 was constructed in 1971/72. Building 2 was constructed in 1974. Details appear to be consistent with those in common practice at the time of construction. Materials in Building 1 are: 3000-psi NWC in the walls; 4000-psi LWC in the slabs; Grade 60 reinforcement for all bars larger than No 5, otherwise, Grade 40 reinforcement. Materials in Building 2 are: 5000-psi precast walls with 4000-psi mandrels (4600 psi assumed average); LWC slabs; Grade 60 reinforcement throughout.

The total weight of Building 1 was calculated to be 24,000 kips. The total weight of Building 2 was calculated to be 23,000 kips.

<u>Computed Building Strengths</u>

Base shear strengths of the two buildings were calculated in each of two principal directions considering (a) flexural mechanisms extending over the height, and (b) wall shear strength computed according to the UBC. Values are listed in Table 1.

<u>Dynamic Analytical Model</u>

Linear-elastic properties of the buildings were modeled using conventional software. Lacking specific information on soil conditions, all analytical models were given fixed bases at the foundation model.

Given the symmetry in the floor plan of Building 1, a simple 2-dimensional model was prepared for each direction. The model of the transverse direction considered contributions only of walls alined in that direction. The model in the longitudinal direction considered two frames (one to model the exterior column lines and another to model the corridor walls). In both models, gross-section properties were assumed for all elements. Effective widths of coupling slabs were computed using the results of Qadeer, et al. [2] (the resulting slab width was typically equal to wall width plus six slab depths). A rigid floor diaphragm was assumed. Responses of the 2-D models were computed using the program SAP-80.

Because Building 2 had some assymmetry in plan, a complete 3-D model was prepared using the program ETABS. Member modeling assumptions were essentially the same as those assumed for Building 1.

Computed periods for the models are in Table 2. The fundamental periods are approximately N/20, where N is the number of stories in the building. For comparison with the values in Table 2, Eq. 12-3 of the UBC gives a fundamental period of 0.58 sec for each building.

Ground Motions at Building Sites

Response records for Building 1 were obtained for the Morgan Hill and Mt. Lewis earthquakes. Only the former is considered here. Response records for Building 2 were obtained for the Whittier-Narrows earthqukae and aftershocks. Only the former is considered in this paper. Instrument and processing details can be obtained from the CSMIP. Ground acceleration histories (obtained from instruments in the ground floor of the buildings) are plotted in Fig. 2.

Measured and Computed Responses

Responses to the measured horizontal ground accelerations were computed for each of the buildings using the gross-section models described previously. For the 3-D model of Building 2, the two horizontal components were considered to act simultaneously. Viscous damping equal to five percent of critical was assumed for all calculations.

Computed and measured roof relative displacements are compared in Fig. 3. (Measured relative roof displacement is computed as the difference between measured absolute displacements at the roof and the base. For relative transverse displacements of Building 1, as described by Mahin, the error margin is one-third of the measured displacement. Thus, because it is not possible to accurately gage responses for the transverse direction for this building, no transverse displacement plots are shown.)

From the data in Fig. 3, it is apparent that the computed building periods are too short. Some error is likely to arise because effects of soil-structure interaction have not been considered. For buildings of the proportions considered, soil-structure interaction effects are expected to result in approximately ten percent increase in the computed period. The measured discrepancies thus appear to be beyond what can be attributed to soil-structure interaction.

Another likely source of error lies in the assumption that element stiffnesses could be based on gross-section quantities. At the present time, there is no well-defined technique by which to gage the degree of cracking that occurs in a structure over time. In the absence of load, some initial cracking can be expected due to shrinkage and temperature effects. A modest reduction in stiffness would be expected due to this effect. Under high lateral loads, load-induced cracking will result in more significant reduction of stiffness. Analyses of the effective stiffness of typical walls in the study buildings under the action of applied moments and axial loads indicates a fully-cracked stiffness of approximately 40 percent of the gross-section stiffness. Such a reduction could be expected for Building 2, which was subjected to moderate earthquake excitation. The stiffness reduction for Building 1 might be less, as the degree of shaking was less. Lacking a well-defined reduction, a stiffness of approximately 70 percent of gross section might be reasonable. (Clearly more study of this problem is warranted, and possible using the CSMIP data.)

Figure 4 compares measured and computed roof displacements for the two buildings with stiffness taken as 70 percent of gross section for Building 1 and 40 percent of gross section for Building 2. The degree of correlation is greatly improved. Periods and response amplitudes are closely matched in both directions.

It is noted that the computed and measured periods in Fig. 4 compare reasonably throughout the entire duration of response. The implication is that, cracking aside, the response of the buildings during these earthquakes was basically elastic. This conclusion is consistent with the relatively high base-shear strengths of the buildings (Table 1). Likely performance during more severe motions is gaged by comparison with experiences with bearing wall buildings in Chile.

PERFORMANCE OF BEARING WALL BUILDINGS IN CHILE

Given the frequency of strong earthquakes, the Chileans have developed a formula for building design that is remarkably different from that in the US. The typical building in Chile is a reinforced concrete bearing wall building. The structures are designed for lateral forces similar to those prescribed in the US. However, in sharp contrast with US practice, the Chilean engineers have learned through experience that this form of construction can survive strong earthquakes without ductile details and without inspection during construction.

The success of the Chilean "formula" for design was apparent following the 1985 Chile earthquake [3]. The city of Vina del Mar, with over 400 reinforced concrete buildings over 5 stories, was subjected to a ground motion having peak acceleration of 0.36 g and duration of strong shaking in excess of one minute. The success of the Chilean construction is apparent in the cost of repair, which averaged 35 cents per square foot. A cost this low is unusual for an event of this magnitude.

As part of the study of bearing wall building performance, a detailed study of the Chilean buildings has been undertaken. Details of the study are reported elsewhere [4].

Five-percent damped response spectra for the 1985 Chile earthquake and for representative US design earthquake motions are compared in Fig. 5. It is noted in relation to that figure that the Chile motion has response ordinates representative of those that might be expected in the US. Thus, it has been concluded [4] that the Chilean earthquake provided a representative test of bearing wall construction for the US.

Figure 6 compares computed strengths and periods of several Chilean buildings and the two California bearing wall buildings with inelastic spectra for the ATC soil type 1, which is similar to the spectrum of the Chilean earthquake. For these buildings, it is estimated that the maximum displacement ductility demand of approximately three is likely to have resulted in the Chilean earthquake. The comparison of spectra in Fig. 5 suggests that similar results would be expected in a California earthquake. For a displacement ductility of three, results reported by Paulay [5] indicate a curvature ductility of eight or less for the usual bearing wall.

Moment-curvature relations of typical reinforced concrete bearing walls have been computed using standard techniques (Fig. 7). For reinforcement ratios less than or equal to one percent, the available curvature ductilities in the absence of confinement reinforcement exceeds the required ductility by a safe margin.

CONCLUSIONS

Based on a study of California bearing wall buildings and of similar buildings in Chile, the following have been observed:

1. Periods and responses of reinforced concrete bearing wall buildings cannot be adequately gaged on the basis of uncracked section properties, even when the earthquake loading is not severe (as in the Morgan Hill earthquake).

2. By modifying the periods of the two study buildings in the range between the cracked and uncracked periods, good correlation with measured behavior was obtained.

3. Simlarities in strength of the two study buildings and of similar Chilean buildings were noted. Also, the similarity between design US motions and the 1985 Chilean earthquake was observed. By observation and by analysis, it is reasonable to conclude that US bearing wall buildings would perform in a strong US earthquake with similar success as the Chilean buildings, even in the absence of the ductile details required in the US. Reductions in detailing may be appropriate.

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3. "The 1985 Chile Earthquake," EERI Spectra, February, 1986.

4. J. W. Wallace, "The 1985 Chile Earthquake: Implications for Design of Bearing Wall Buildings in the US," PhD Dissertation submitted to the Graduate College, University of California at Berkeley, January 1989.

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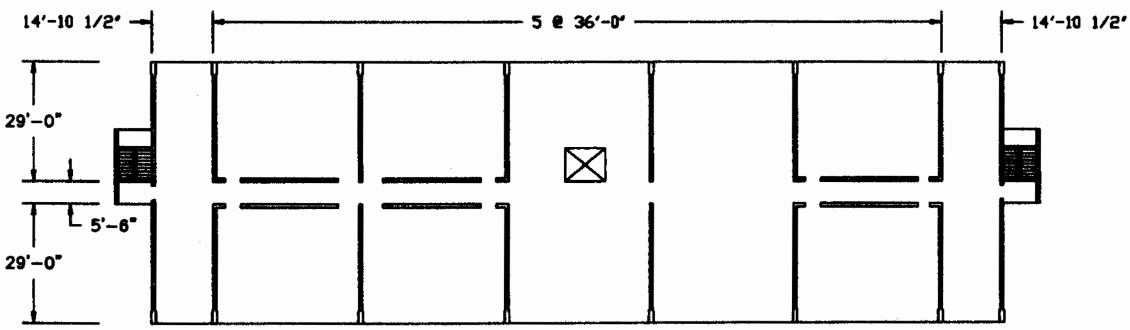
Table 1 - Computed Base Shear Strengths

	Shear Strength		Flexural Strength	
	Trans.	Long.	Trans.	Long.
Building 1	0.20W	0.47W	0.32W	0.22W
Building 2	0.41W	0.37W	0.32W	0.29W

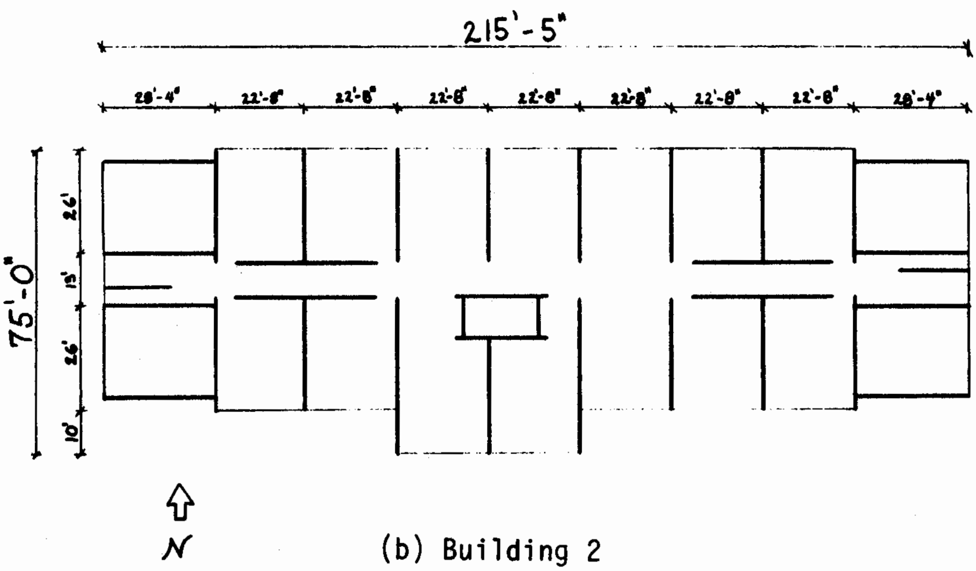
Note: W = 24,000 kips for Building 1,
23,000 kips for Building 2.

Table 2 - Computed Periods, sec

	Trans.	Long.	Tors.
Building 1	0.32	0.50	---
Building 2	0.31	0.34	0.28

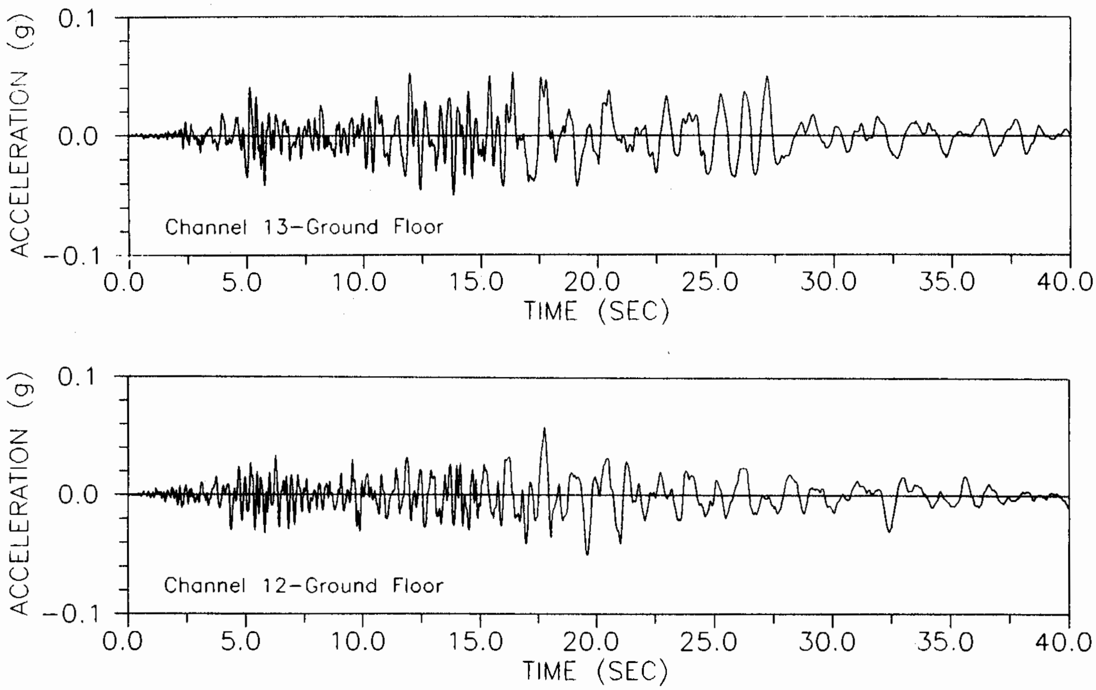


(a) Building 1

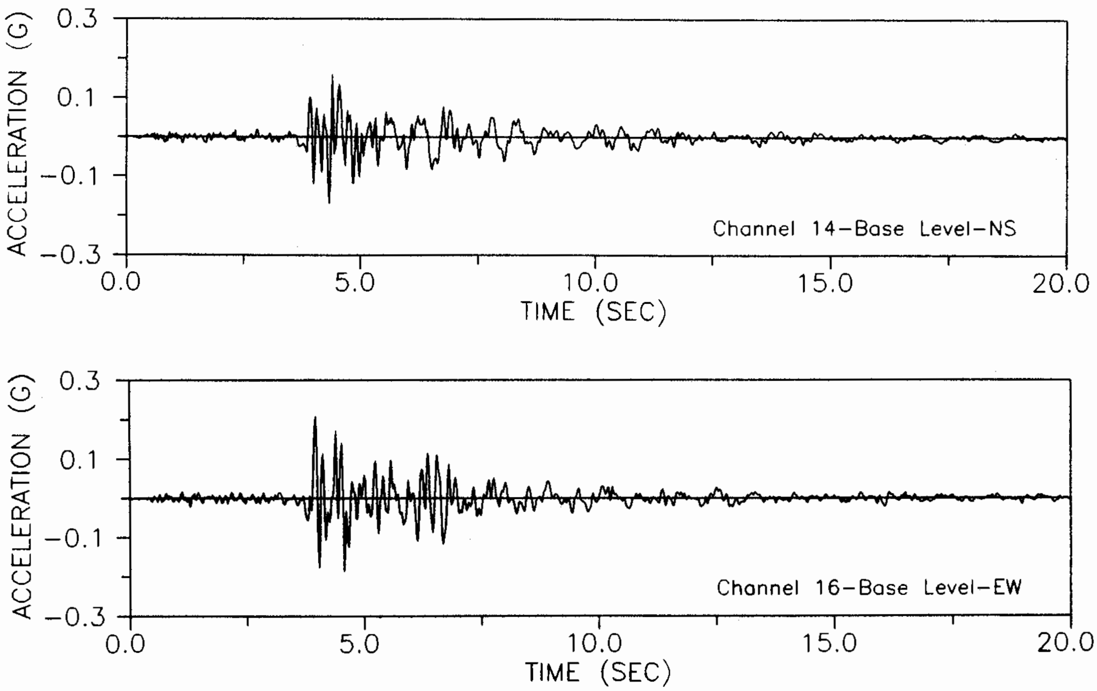


(b) Building 2

Fig. 1 Building Plans

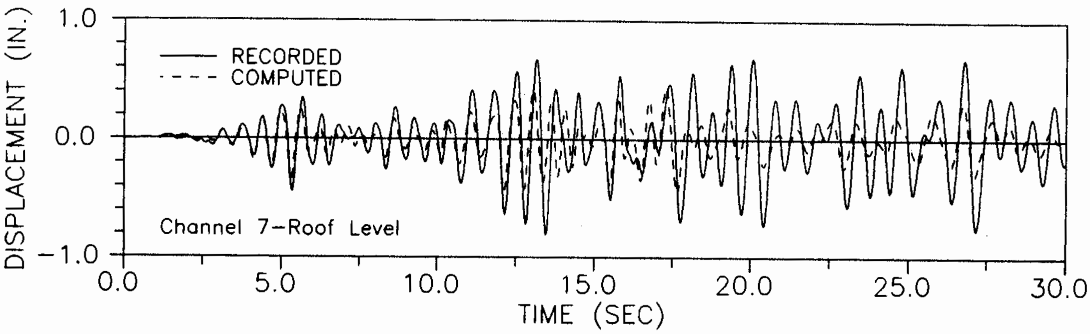


(a) Building 1

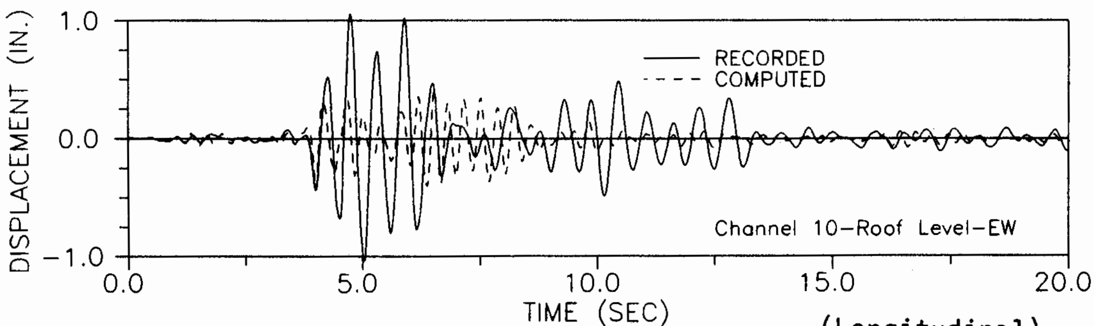


(b) Building 2

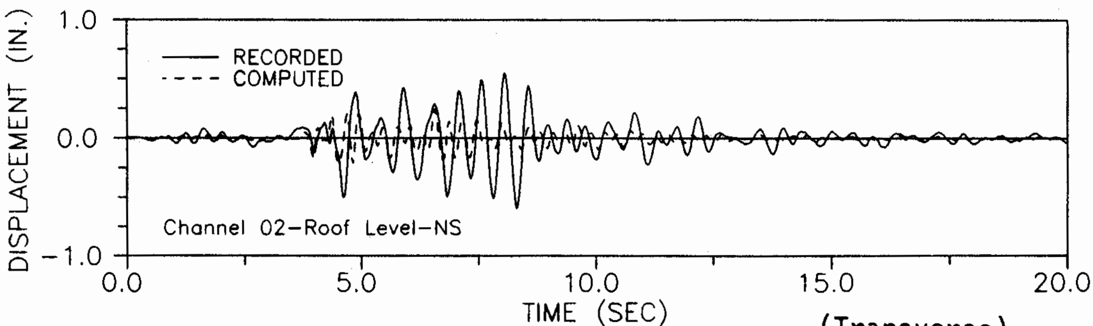
Fig. 2 Measured Base Accelerations



(a) Building 1, Longitudinal



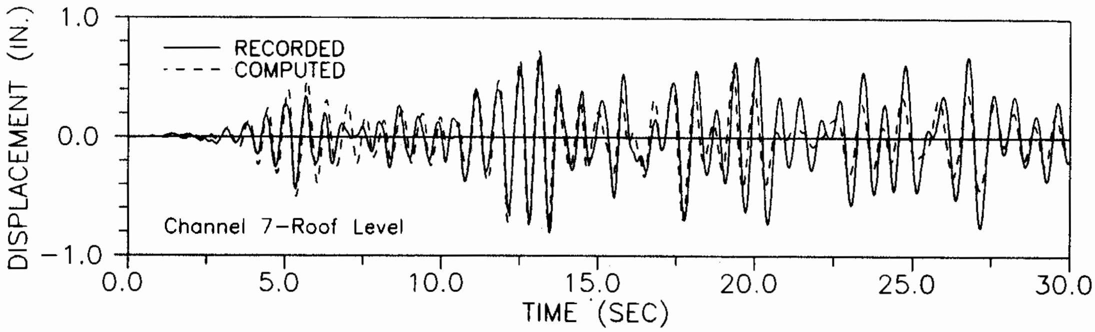
(Longitudinal)



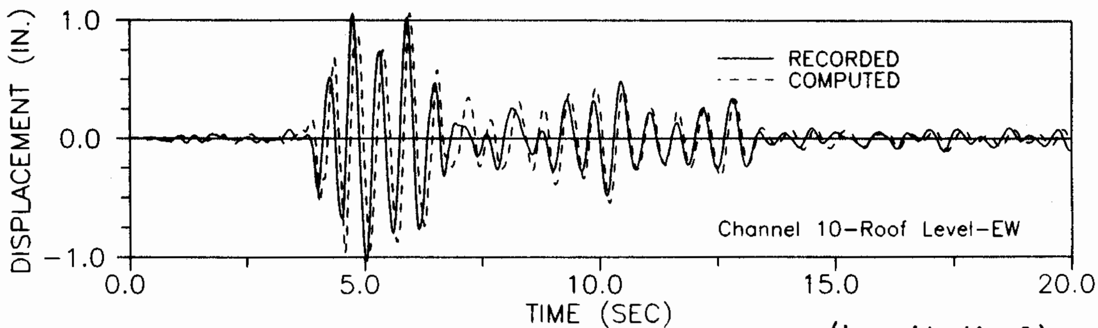
(Transverse)

(b) Building 2

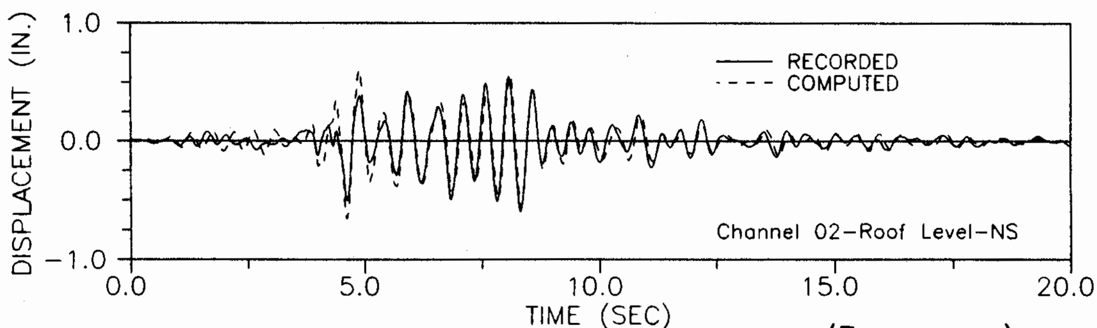
Fig. 3 Measured and Computed Roof Responses, Gross-Section Model



(a) Building 1, Longitudinal



(Longitudinal)



(Transverse)

(b) Building 2

Fig 4 Measured and Computed Roof Responses, Cracked Model

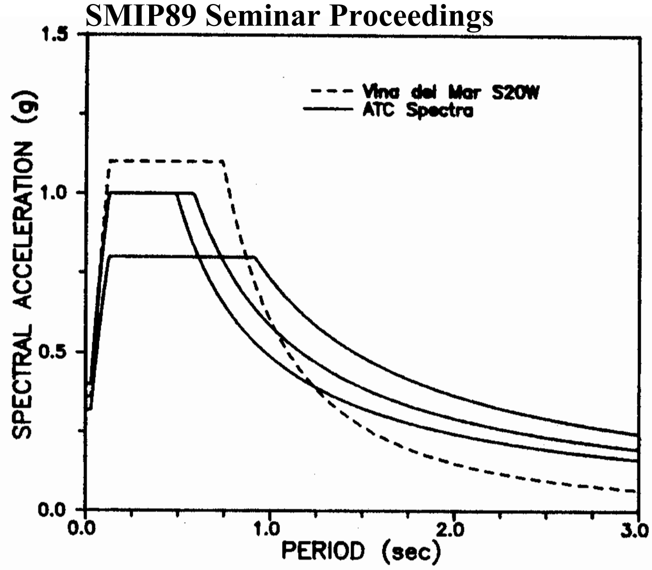


Fig. 5 Comparison of Chile and ATC Spectra

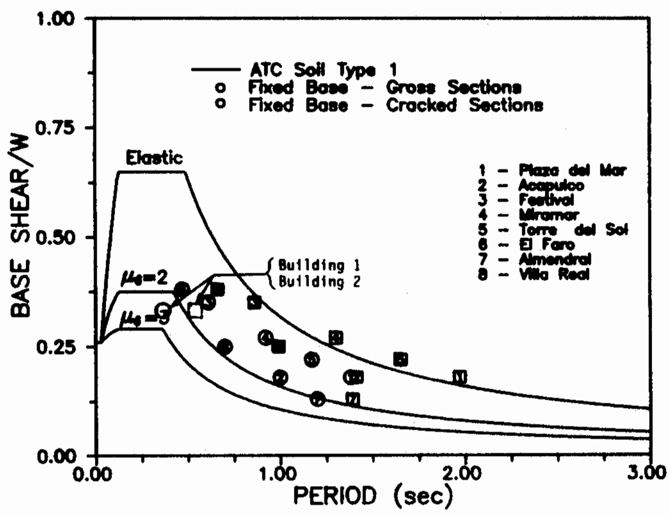


Fig. 6 Comparison of Ductility Demands

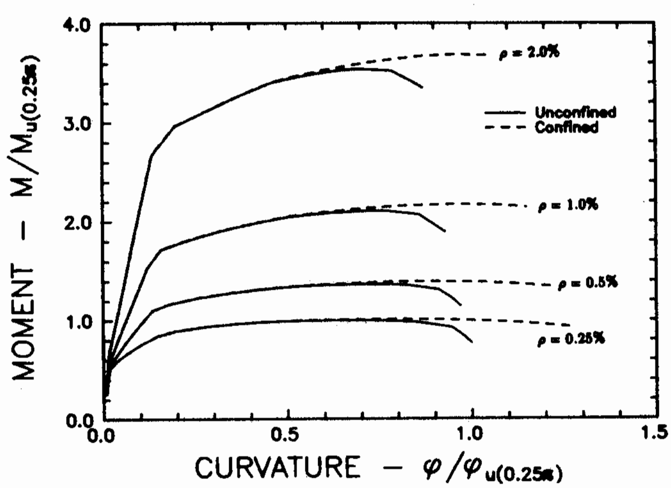


Fig. 7 Computed Moment-Curvature Relations for Bearing Walls