oversimplifications of the very complex real EQRD problem. **Modern building codes, which try to reflect great advances in knowledge and understanding in a very simple way, are not transparent about the expected level of performance of the whole building system (soil-foundation-superstructure and nonstructural components). The expected level of performance has become an implicit, rather than an explicit, part of the codes through a series of empirical factors and detailing requirements which obscure the true nature of the EQRD problem: building performance.**

Current seismic codes lack transparency in their provisions regarding the reliable establishment of critical EQGMs for the desired performance of the whole building system at its different limit states throughout its service life, and also in their provisions regarding the real expected response of the resulting designed and constructed building system to real critical ground motions (not just to code-specified motions). Although there have not been enough moderate and severe EQGMs in urban areas to permit analyses and judgements of the performance of building systems designed according to current seismic codes, the observed behavior of some modern buildings in recent EQs, particularly in Mexico City during the 1985 Michoacan EQ and in the Bay Area during the 1989 Loma Prieta EQ, indicates the need for refinement of the current U.S. code approach.

To define clearly what is needed to improve the present code approach, it is convenient first to summarize the lessons learned from recent EQs and research, and then to assess their implications regarding the adequacy of current U.S. seismic design codes and the possibility of improving the EQRD of structures. Before doing so, it should be noted that many failures during EQs have been consequences of lack of proper enforcement of the codes.

Adoption and Enforcement of Current Design Codes. The adoption and enforcement of current design requirements at state and local levels varies widely throughout the U.S. A 1977 survey by the National Conference of States on Building Codes and Standards indicated that about one quarter of the states had statewide building codes. Post-EQ investigations often reveal substandard construction practices which capable supervision or inspection during construction would have eliminated. Informal observations also suggest that enforcement of building codes may be lax except in the western states of the U.S.

LESSONS LEARNED FROM RECENT CATASTROPHIC EARTHQUAKES

INTRODUCTORY REMARKS. Strictly speaking, there have been few new lessons learned from the 1985 Chile, 1985 Mexico, 1986 San Salvador, 1987 Whittier Narrows, 1988 Armenia, and 1989 Loma Prieta EQs. Each of these EQs has some important features (characteristics or signatures) which not only differentiate each of them but also remind us that we have forgotten one or more important lessons learned from previous EQs. In what follows, summaries are given for each of the above EQs of only those characteristics that are considered important for improving EQRD of new structures and proper upgrading of existing hazardous structures.

THE 3 MARCH 1985 CHILEAN EARTHQUAKE. This EQ, with an epicenter located in the Pacific Ocean near the coastal center of Chile, was reported to be of Richter magnitude 7.8. The EQGMs were measured by strong-motion instruments (accelerographs) at least 35 stations. From evaluation of these records, Saragoni and his associates [21] concluded that this event consisted of two successive shocks: the first of Richter magnitude 5.3, with a duration of strong motion of 10 seconds, and the second, which occurred 10 seconds later, of Richter magnitude 7.8, with a strong motion duration of about 30 seconds. The duration of recorded motion was in some cases 120 seconds.

Recorded Ground Motions. Significant ground motions were recorded in at least 35 stations. Among the most important of the recorded EQGMs are the ones obtained at the Llolleo, Melipilla and Viña del Mar stations.

(a) Llolleo Record. This recording station was the closest to what has been considered as the center of the energy release of the EQ. The three recorded components of the acceleration are given in Fig. 9. The maximum accelerations (peak ground accelerations, PGA) were: 0.85 vertical component; 0.67g and 43g, respectively for the N10E and S80E components. The very large vertical component (27% higher than the horizontal PGA) of the ground acceleration deserves special study with respect to the response of structures, particularly buildings, and points out the need for EQRD code recommendations to consider the vertical component of the design EQ. The record of the horizontal component in the N10E direction is considered to have the greatest damage potential of all EQGMs recorded on firm soil for buildings with **fundamental period (T)** of vibration between 0.3 and 0.6 seconds, as has been shown by Uang and Bertero [11]. The **input energy** (E_I) and the **hysteretic energy** (E_H) for $0.3 \leq T(sec) \leq 0.6$ for this recorded EQGM are significantly larger than for any other recorded EQGM on firm soil. This is mainly due to the long duration of strong motion, nearly 50 seconds (see Fig. 9). It is of interest to compare the 5% damped linear elastic response spectrum (LERS) resulting from the recorded N10E component of the ground motion measured

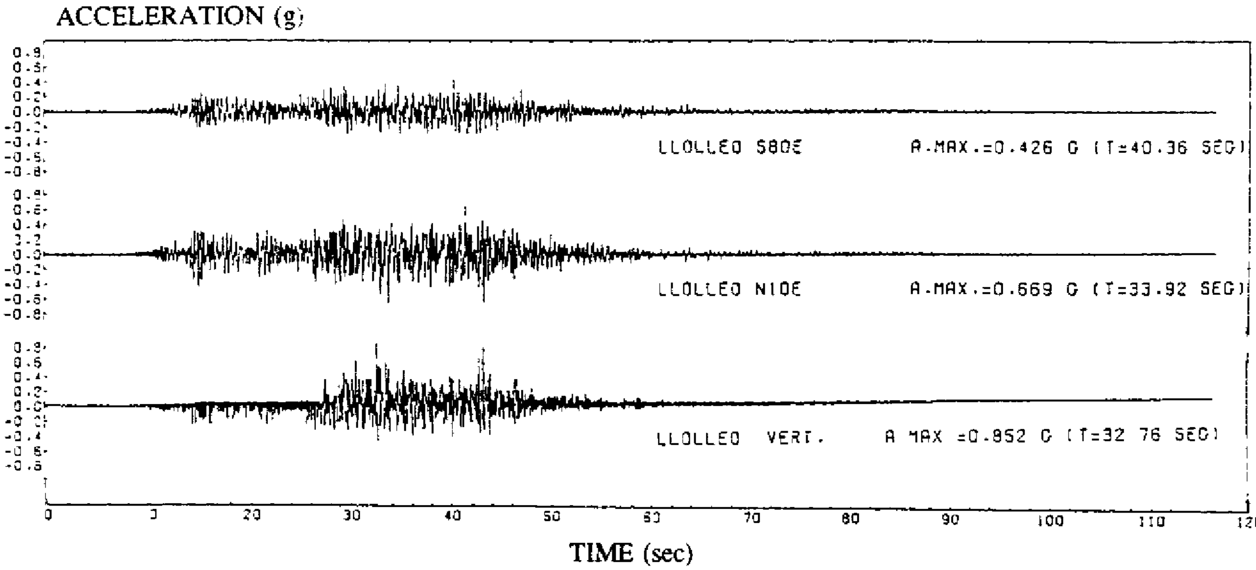

Fig. 9 **1985 CHILE EQ: ACCELEROGRAMS RECORDED AT THE LLOLLEO STATION [21].**

at Llolleo with those corresponding to the ATC-recommended seismic force coefficients (C_{sp}) as well as the 1985 SEAOC and 1988 UBC-recommended ZC (equivalent to C_{sp}) values for different soil types. These values of C_{sp} are based on assumed 5% LERS. Such comparisons are illustrated in Fig. 10. Even for the region of highest seismic risk in the U.S. (ATC map area No. 7 and SEAOC seismic zone 4), the recommended spectrum for C_{sp} for soil type S_1 (rock and stiff soil) and for periods less than 1.8 seconds is significantly smaller than the values corresponding to the similar spectrum obtained from the N10E EQGM recorded at the Llolleo station. Therefore, the EQGM of the 1985 Chilean EQ should be carefully evaluated regarding present seismic design spectra as recommended in U.S. codes. The damage potential of the recorded EQGM at Llolleo is significantly greater than any previously recorded or considered by any code for rigid buildings located on stiff soil sites.

Analysis of the recorded EQGMs and the computed LERS reveals that the spectrum amplification factors are somewhat higher than present U.S.-recommended values [22] For a level of probability of One Sigma (1 σ) and for 5% critical damping, the amplification factor suggested for the acceleration is 2.71, while the maximum recorded amplification factor was 3.6. Analysis of the 5% damped spectrum for this component reveals that if an EQGM similar to this component were to occur in the U.S., the maximum value of EPA could be significantly exceeded for periods between 0.1 and 0.8 seconds.

(b) Melipilla Record. This station was located about 42 km from the Llolleo station and about 76 km from the center of maximum energy release. In spite of this distance, the horizontal accelerations were very severe: 0.67g in the NS direction and 0.67g in the EW direction. They were the largest horizontal PGAs that were recorded. These records clearly show that, owing to local site conditions (soil profile and topography), the intensity of EQGMs can be very large even at large epicentral distances. The damage in the town of Milipilla was very severe, particularly in the 1-story adobe and unreinforced masonry houses.

(c) Viña del Mar Ground Motion. The maximum accelerations recorded by the instruments located in Viña del Mar were 0.23g in the N70W direction, 0.36g in the S20W direction, and 0.17g in the vertical direction Figure 11 shows the records obtained and Fig. 12 shows the LERS of the s20w component for a 5% damping coefficient. Analysis of the acceleration spectra shows that it presents two peaks, one at a period T=0.20 seconds and the other at T=0.70 seconds. Comparison of the recorded S20W components of the acceleration with those of the 1940 El Centro NS component shows that they are similar. Furthermore, the response spectra of the absolute acceleration of these two records are also similar up to a T of about 0.6 seconds (Fig. 13). For 0.6<T(sec)<0.9, the acceleration spectra for the Valparaiso EQ are significantly higher than those of El Centro. Duration of strong motion for Viña del Mar is considerably higher that for that of El Centro, about 45 seconds versus 24 seconds. The relative velocity spectra for the Viña del Mar record are nearly 50% higher than those for El Centro for 0.5<T(secs)<1.7 (Fig. 14). As a consequence of these differences, the elastic input energy, E_I, and hence also the damage potential for the Viña del Mar, is about twice that of El Centro.

Another important difference between the Viña del Mar and El Centro records is that, while the

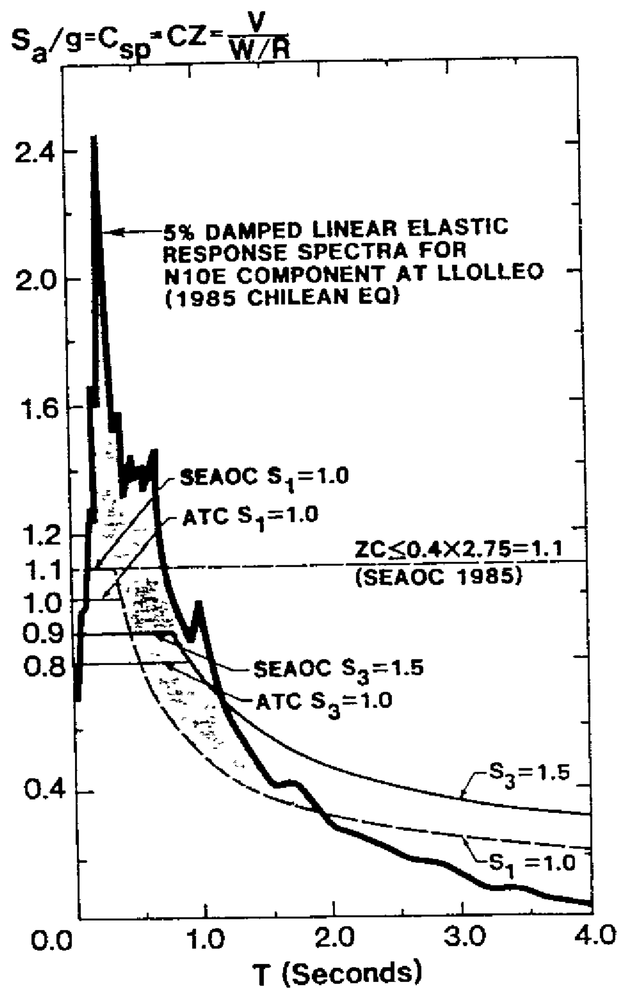

Fig. 10 COMPARISON OF LINEAR ELASTIC RESPONSE SPECTRUM FOR N10E COMPONENT RECORDED AT LLOLLEO DURING 3 MARCH 1985 CHILEAN EQ WITH 5% DAMPED LEDRS RECOMMENDED BY ATC-3-06 AND SEAOC (1985).

displacement spectra for El Centro increase as T increases from T=0 sec to T=3.0 secs, for the Viña del Mar the displacement spectra remain approximately constant for 0.8<T(sec)<3.0 (Fig. 15). The important implication of this type of spectra is that for buildings having period near T=3.0 secs, it is not possible to reduce the story drift by just increasing lateral stiffness, i.e., decreasing the T, except if T is decreased to a value below 0.3 seconds. The maximum horizontal displacement of the EQGMs was 5.56 cm.

The observed performance of the buildings in the housing subdivision of Canal Beagle indicates clearly that the most severe damage occurred in the buildings located on the steep-sloped ridges or next to a canyon, indicating that there may have been substantial amplification due to the general topography of the subdivision. This was confirmed by motions recorded during the aftershocks. This lesson points out the need for reliable microzonation of our urban areas.

Geotechnical Effects. Important geotechnical failures were observed in many areas. The most spectacular failures were the liquefaction failures of portions of a seawall in San Antonio (Fig. 16) and in Valparaiso. Large settlements were observed in buildings in downtown San Antonio. Slope instability was observed on the hill side of Renaca.

Several bridges sustained major damage as a result of movement of supporting piers. Figure 17 shows a photo of the bridge over the Maipo River between Llolleo and Rocas de Santo Domingo. Four spans fell down.

Performance of Modern Building Structures. Viña del Mar, where the record described above, which is comparable to the 1940 El Centro record, was obtained, is a city with numerous modern multistory RC building structures. There are over 300 modern RC buildings of between five and 23 stories. Although two of these modern buildings suffered severe damage (one has to be demolished), none collapsed and most of them suffered very little

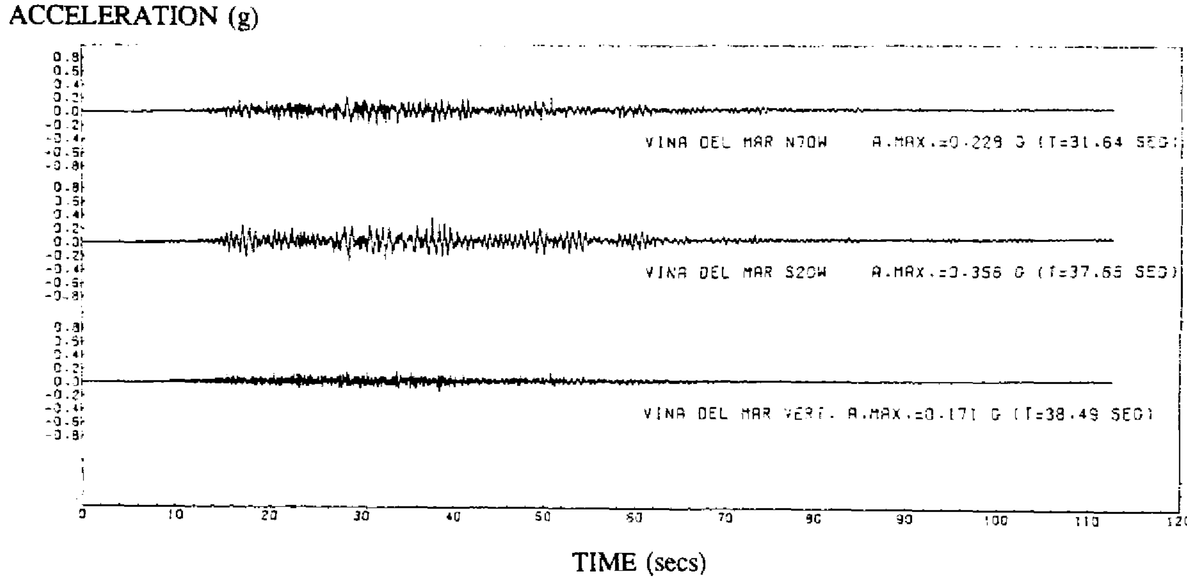

Fig. 11 1985 CHILE EQ: ACCELEROGRAMS RECORDED AT VIÑA DEL MAR [21]

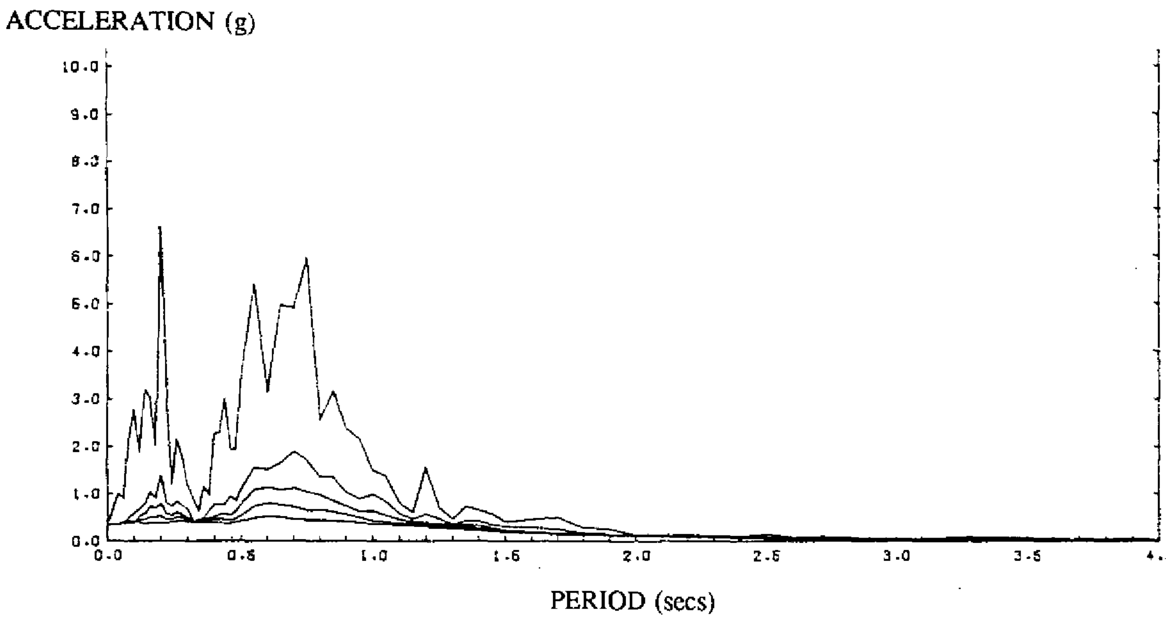

Fig. 12 ABSOLUTE ACCELERATION RESPONSE SPECTRA FOR THE S20W COMPONENT OF THE VIÑA DEL MAR RECORD [21]

damage. The main reason for this very good performance is the type of structural layout and structural system used in Viña del Mar for these multistory RC buildings. Most are apartment buildings, which allow the use of a dense array of shear walls (Figs. 18 and 19). The ratio of shear wall area to floor area for the tall apartment buildings in Viña del Mar has a mean value of 3.5% in each direction, i.e., a total wall area equal to around 7% of floor area. In the U.S., Arnold and Reitherman [23] have reported that for a typical 10- to 20-story MRSF steel or concrete building, the ratio between the area of all the columns at ground level and the plan area is 1% or less. If the structural system consists of a combination of MRSF and shear walls, the above ratio is about 2%. Even in the case of multistory office buildings having a structural system of shear walls alone, the above ratio does not exceed 3% in the U.S.

There is no doubt that the use of large numbers of shear walls (i.e., large redundancy) resulted in buildings in which the shear stress developed in the shear walls was relatively very low. Furthermore, these walls resulted

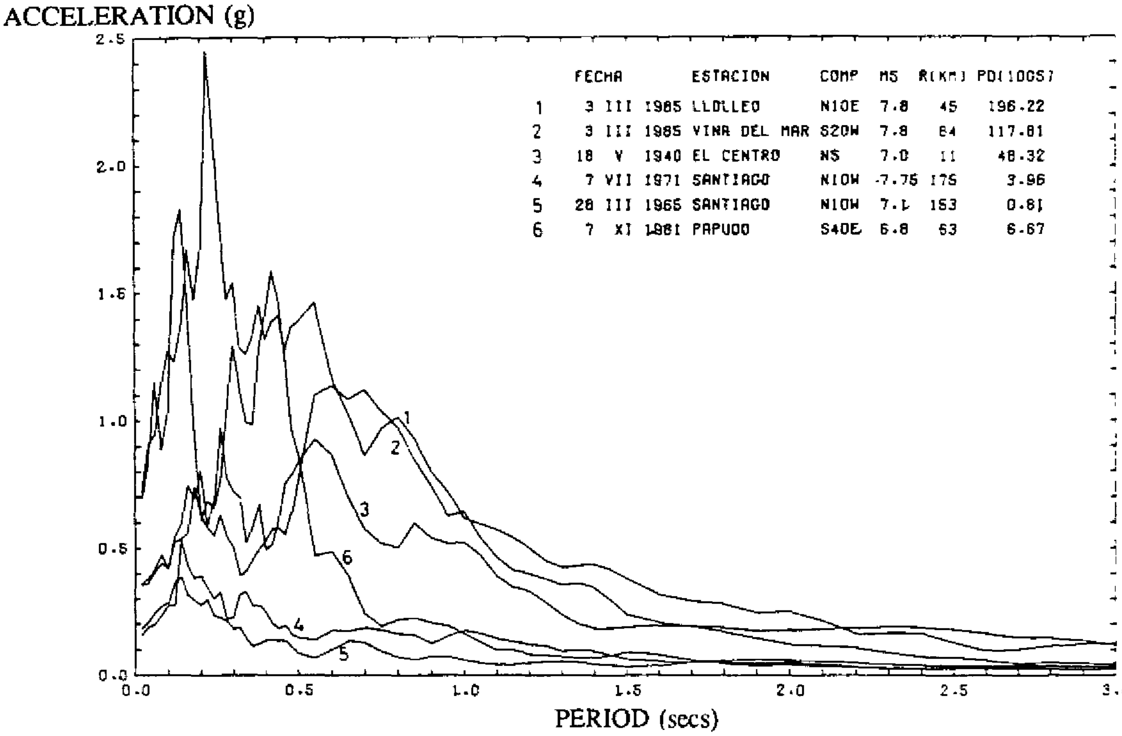

Fig. 13 COMPARISON OF ABSOLUTE ACCELERATION RESPONSE SPECTRA OF 1985 CHILE EQGMs WITH THOSE OF OTHER RECORDED EQGMs [21]

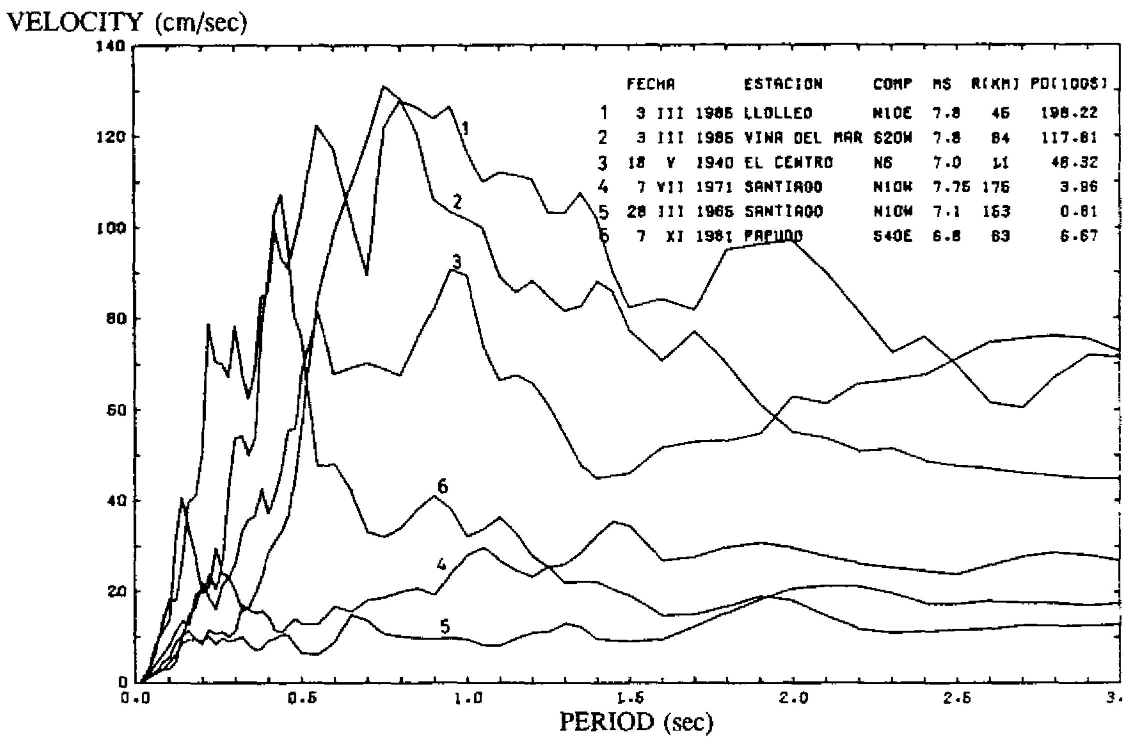

Fig. 14 COMPARISON OF RELATIVE VELOCITY RESPONSE SPECTRA OF 1985 CHILE EQGMs WITH THOSE OF OTHER RECORDED EQGMs [21]

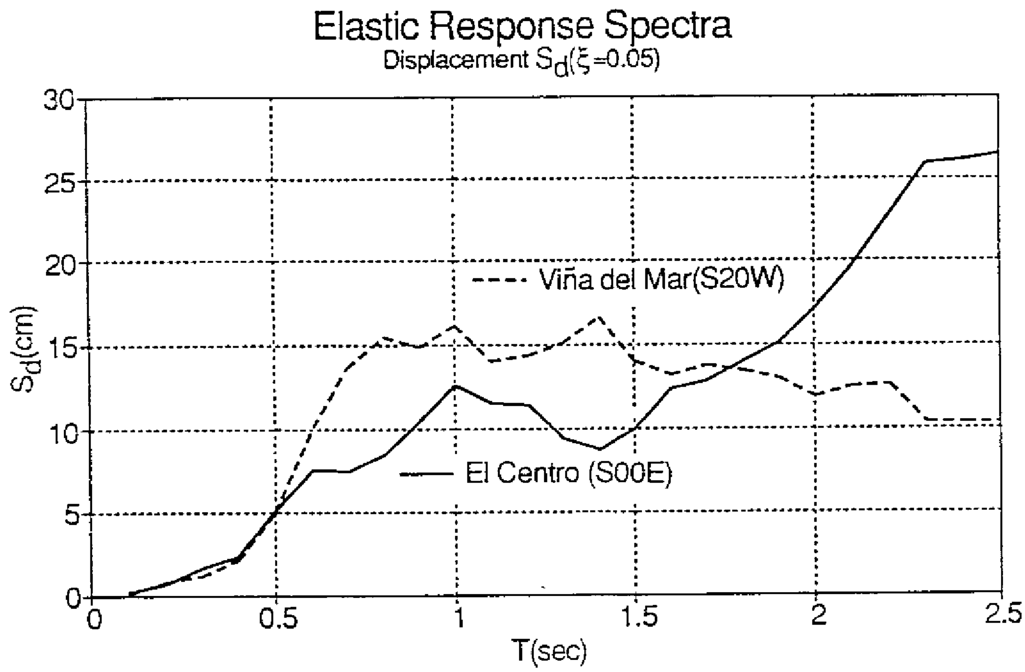

Fig. 15 COMPARISON OF DISPLACEMENT SPECTRA OF THE 1985 VIÑA DEL MAR EQGM WITH THAT OF THE 1940 EL CENTRO EQGM

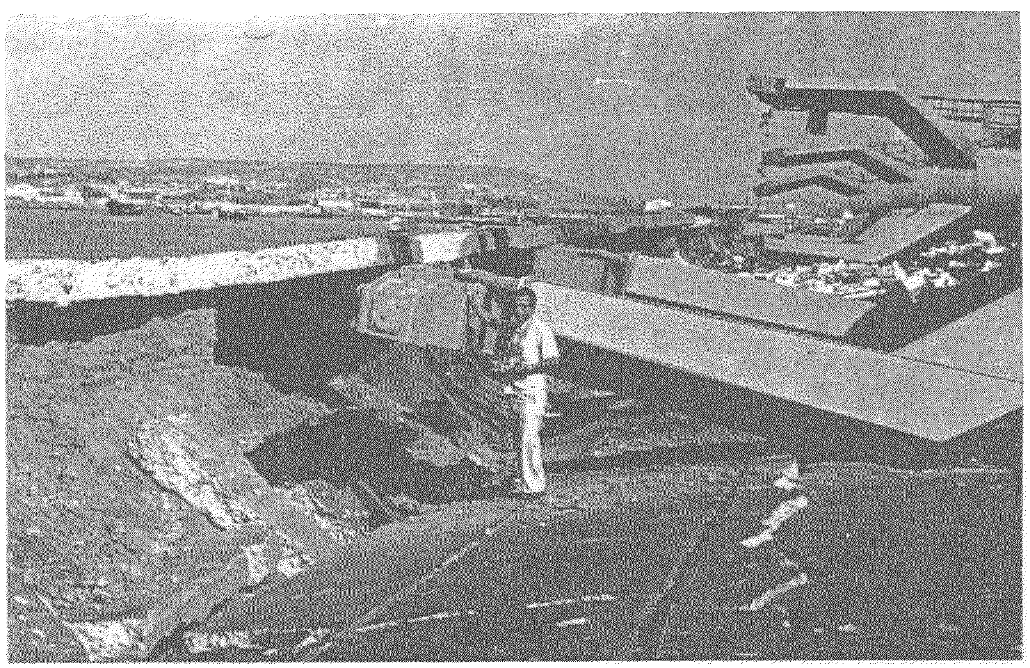

Fig. 16 COLLAPSE OF SHEAR WALL IN SAN ANTONIO HARBOR [21]

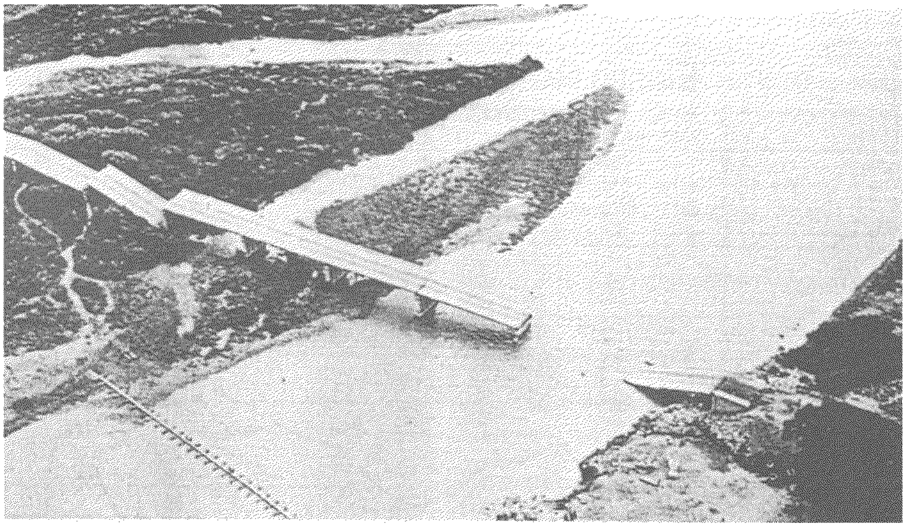

Fig. 17 COLLAPSE OF THE LO-GALLARDO BRIDGE DURING THE 1985 CHILE EQ [21]

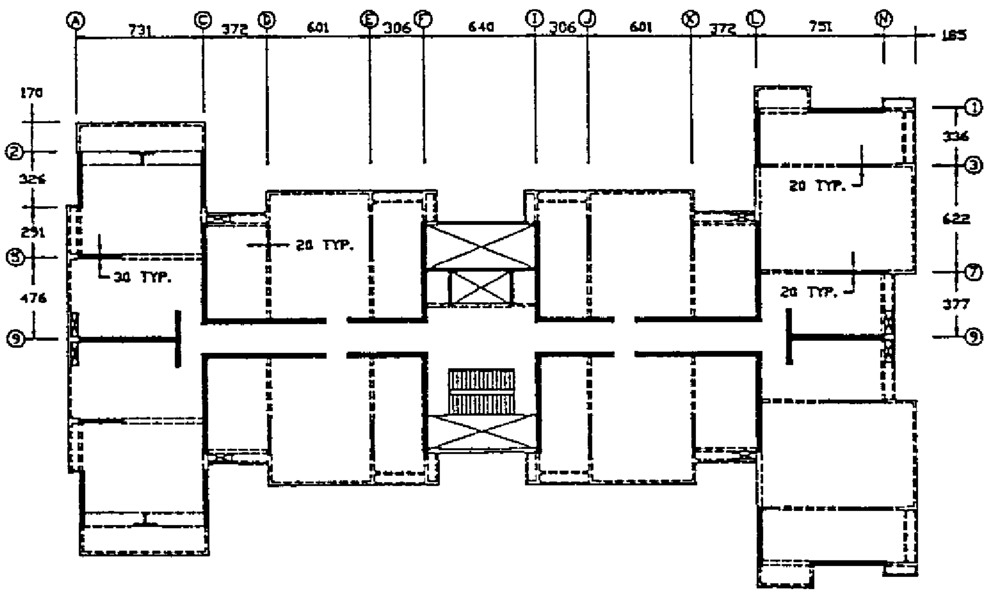

Fig. 18 TYPICAL FLOOR PLAN OF A 14-STORY RC BUILDING IN VIÑA DEL MAR (*UCB/EERC-89/05*, Wallace and Moehle, 1989)

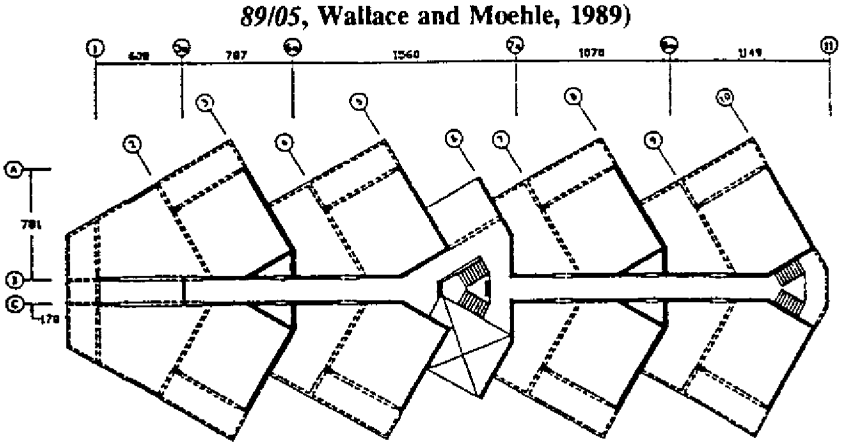

Fig. 19 TYPICAL FLOOR PLAN OF A 23-STORY RC BUILDING IN VIÑA DEL MAR (*UCB/EERC-89/05*, Wallace and Moehle, 1989)