

## MASONRY PERFORMANCE IN EARTHQUAKES

O. A. Glogau\*

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### ABSTRACT

Past and current methods describing the damage potential of an earthquake are reviewed and discussed in relation to actual earthquake damage. The performance of a number of structures subjected to intense shaking is described with particular reference to their detailing. Examples are given showing the effect of infilling walls on structural behaviour. Veneer on timber construction is discussed. The points made are illustrated with figures and photographs.

### 1. INTRODUCTION

Evaluation of the performance of a structure in an earthquake requires knowledge of those parameters in the structure and the earthquake's motion responsible for damage or failure. To predict future performance, in addition, an estimate of the likely variation in these parameters must be available. At the current state of the art this is difficult even for relatively simple structural systems in homogeneous materials. The problem is complicated for reinforced grouted masonry which is composed of four different materials and subject to greater quality variations than other materials commonly used in earthquake resistant construction. In addition the bulk of masonry structures are low, and as is common with buildings of this type, highly irregular. Caution is therefore indicated in evaluating masonry performance.

### 2. RELATING EARTHQUAKE EFFECTS TO SIZE AND CHARACTER OF EARTHQUAKE

#### 2.1 The Modified Mercalli Scale

Amongst the early attempts to describe earthquake effects is the well known MM scale of 12 steps which is useful for describing the effects of earthquakes in an area where the bulk of masonry buildings are unreinforced and of a known type and construction standard. Unfortunately, this scale is of no quantitative assistance to the designer of modern reinforced masonry.

#### 2.2 The Richter Scale

This scale, in which a step of magnitude rating represents a 30 fold increase of energy release at the source of an earthquake, has been shown to be quite unrelated to the intensity and character of shaking at a particular surface location. At best it gives an indication of the likely extent of the zone of intense shaking. It was of little comfort to the citizens of San Fernando and Managua surveying the destruction wrought by their "little"

earthquakes (M 6.6 and 6.5 respectively) to know that the energy release was only 1,000ths of that of the great Alaskan and Chilean earthquakes. Closeness of the event, geology and local soil conditions all have a tremendous effect on the intensity and quality of earthquake motions.

#### 2.3 Strong Motion Records

A new phase of engineering description of earthquakes began in 1940 at El Centro Imperial Valley, California, when, for the first time, the accelerograph record of intense ground shaking was obtained. This was a great step forward, but early optimism was soon somewhat dampened by uncertainty about interpretation of the damaging parameters of the motion. The peak accelerations of the locally highly destructive (MMX) El Centro earthquake were only about  $\frac{1}{2}$  g but so were those of the relatively minor (MM7) 1966 Gisborne earthquake. In 1966 a record was obtained on the San Andreas fault that contained peak accelerations of 0.5 g but "disappointingly" little damage was sustained by the few engineered structures in the area.

Gradually it was realised that besides peak accelerations at least two other parameters of an earthquake motion are highly significant. These are the length of the intense phase of the motion and its spectral content. Both of these are reflected in response spectra.

#### 2.4 Response Spectra

A response spectrum represents the response of single mass, damped resonators of various fundamental periods subjected to the earthquake motion. It is readily understood that duration of an event is an important parameter in any resonance situation of relatively low damping. Even more so if these motions predominantly contain a very narrow band of frequencies, i.e. if approximating harmonic motions they are extremely destructive to structures of fundamental periods matching these frequencies.

Such motions are more than just an engineer's nightmare; they exist, have been

\* Chief Structural Engineer, New Zealand Ministry of Works and Development.

recorded and have proved to be highly destructive to low rigid buildings. There were no modern masonry structures in the area affected by the Tokachi-oki earthquake but the R.C. structures, many of which had walls or deep membered frames with wide wall-like columns, were of a structural type common in reinforced masonry buildings. Valid conclusion can therefore be drawn by analogy as regards masonry. Of particular relevance are those of the Japanese investigating team reporting on the 1968 Tokachi-oki earthquake (11) which concluded from a statistical survey of structures damaged or left undamaged that a static coefficient of 1 g would need to be used in the design of shear failing systems if damage were to be avoided. This serves to confirm the theoretical investigations made by others (13) (14). While such high seismic coefficients are not practical, the requirements of DZ4203 for masonry wall systems, likely to fail in shear of 0.29 g (ultimate for seismic zone A Class III buildings), is seen to be reasonable.

It is noteworthy that the earthquake, which failed or damaged many 1 to 3 storey well built structures were conventionally designed to a level twice that of the current N.Z. Code (0.2 g working stress design), had its epicentre at a distance of some 250 km, and produced local ground accelerations of only 0.225 g (Figs. 1 and 2).

The complexity of the problem by no means rests here because earthquake motions are three dimensional. In addition to two horizontal components very large vertical accelerations have recently been recorded; two-thirds as large as the horizontal at San Fernando and comparable to the horizontal at Managua. Testing facilities in Japan and California are able to simulate, at least in part, these three-dimensional motions. Less is known about torsional ground motions - but there is substantial evidence that they exist and some indirect provision is made for them in the revised N.Z. Loading Code. When evaluating damage it is as well to bear in mind that all these motions are applied simultaneously to a structure.

New methods have been developed to give a better pictorial description of the damaging parameters of earthquakes. Starting with the well known accelerogram of an earthquake (peaks approximately  $\frac{1}{3}$  g) (Fig. 3) one proceeds to obtain response spectra (Fig. 4). Response spectra for velocity, acceleration and displacement can be related to each other by relatively simple equations although for non-harmonic motions such relationships are only approximate. Fig. 5 shows the now familiar tripartite representation of response spectra from which, for a given natural period and damping value acceleration, velocity and displacement response can be obtained. A novel representation of the effect of the duration of an earthquake is shown in Fig. 7 (2 i). This relative velocity response envelope spectrum (5% damping) (VRES) shows that the very intense shaking of the ground motion between 3 and 5 sec results in an immediate velocity response of 60 cm/sec for structures less than 1 sec period but a delayed, very severe effect on structures with over 2 sec period,

some 5 sec later (at 10 sec). A three-dimensional representation is shown in Fig. 8. The height of the hills and valleys (z axis) represents the velocity response of resonators of various periods (y axis) as it varies with the progress of the earthquake (x direction). The xy projection corresponds to that of Fig. 7 (but for the Managua South component). Very interesting too is the time duration spectrum shown in Fig. 6. In the south direction levels of 40 to 60 cm/sec lasted between 4 and 8 cycles for many structures with periods of between 0.3 and 0.6 sec and levels of 20 to 40 cm/sec were experienced for 8 to 16 cycles for structures in the range of 0.4 to 0.8 sec. Cyclic testing of most structural elements, and particularly of those in masonry indicates that number of cycles of high strain play a significant part in damage. Hence the relevance of the representation method in Fig. 6 as an index of damage.

As a final word of caution: Ground motions are known to vary significantly between locations a short distance apart and on apparently similar ground. One such instance being reported by Hudson concerns two instruments located only a few hundred yards apart on the grounds of the University of California.

It is seen that evaluating earthquake performance in quantitative terms is not an easy task. Bearing this in mind there is nevertheless a great deal that can be learned from earthquake damage, or the lack of it, particularly with regard to detailing.

### 3. MASONRY PERFORMANCE

#### 3.1 General

Very few modern reinforced masonry structures have been reported as having been subjected to intense shaking and of those that have been, many were of very low height and hence presumably well above code strength.

One such single storey building of semi-domestic construction with light corrugated roof located on the shores of the Laguna De Asososca housed the only instrument to provide a record of the 1972 Managua earthquake (Fig. 22). The building, reported to have been reinforced to the U.B.C. requirements and constructed to a high standard, was not damaged. Notwithstanding the recorded accelerations of approximately  $\frac{1}{3}$  g in all directions the isoseismal zone was only MM VI or VII compared to MM IX for other parts of the city (Fig. 9).

The performance of a building complex of 1 and 2 storey reinforced masonry structures located in the zone of intense shaking during the 1974 San Fernando Earthquake is of some interest because differing standards were followed for the detailing of the various blocks. All blocks appeared to comply with the Los Angeles City code loading requirements but only some with the O.A.C. (15) detailing, requirements. Qualitative conclusions can be drawn from their performance with regard to method of detailing and construction.

Figs. 11 and 12 show the poor performance of stack bonded reinforced masonry without close spaced horizontal reinforcement. The poor bond between the vertical surfaces of conventional units is well known. Mortar adhesion is frequently lost during laying as blocklayers make adjustments to obtain uniform joint widths. Fig. 10 too shows a rather unattractive mode of failure of a wall containing relatively light horizontal reinforcement. During its disintegration only small amounts of seismic energy would have been dissipated. Fig. 13 and Fig. 14 show the manner of failure of a 12" wall reinforced at 24" centres both ways. For 24" long units this is about the closest practical spacing for vertical bars. Although this wall also has little resistance left, a good deal more energy would have been dissipated in the process of its break up. A feature of all the failures that is very disturbing is the spalling of face shells which, if it had occurred some distance above the ground, would have resulted in a shower of deadly missiles. Use of an expansive grout additive may overcome this problem to a degree at least, and testing by M.W.D. is continuing to confirm this.

Fig. 14 also shows how reinforcement in the horizontal joint plane and near the surface is a contributing factor of early disintegration. The "open end bond beam" unit which is standard for all M.W.D. high lift grouting work has several features that reduce the potential for the type of failures seen in Figs. 10 to 14. The webs, lowered by 2½", allow placing horizontal reinforcement well below the level of the horizontal mortar joint plane. The open end of the units is not so much specified because of convenience in laying - in high lift construction the vertical reinforcement is generally introduced after laying of units - but because it restricts vertical mortar beds to the surfaces of the walls and ensures that grout is in contact with most of the web of adjacent units resulting in a more homogeneous wall.

### 3.2 Infilling Panels

These are defined by NZS.1900 Chapter 9.2 1964 as "any wall between beams, columns or floors which by virtue of its position and construction is subject to induced and or applied loadings." This definition perhaps should have concluded with the words "and in framed buildings usually is a menace." With the exception of strong, unperforated panels, located within a suitably reinforced frame, and in positions carefully considered by the designer, and remaining there during the lifetime of the structure, infilling panels have had a very poor record. The adverse effects of infilling panels may include:

- (a) Formation of accidental column hinge mechanism. Figs. 15 to 17 show how very weak hollow masonry partitions in the upper stories of the Macuto Sheraton resulted in lower storey column hinge mechanisms aggravated by high axial loads.
- (b) Attracting shear to parts of the structure not designed for it and resulting in column failures. Fig. 18 shows how the infilling panels at the ends of a building

designed as a frame caused them to become unintentionally, inadequate shear walls. The diagonal compression strut action of the infill panel split the beam column joint at the ground level in combination with sliding shear failure of the wall and high overturning compressions. There are many recorded cases where infilling panels that extended part height between some columns attracted high forces to those columns. In addition the resulting low H/D ratios for the columns increases the bias towards shear type failure. Examples: Annexe to Hotel Hilton, Acapulco, 1962 Earthquake (17); Lorenzo Arenas Market, Concepcion, 1960 Chile Earthquake (17).

(c) Becoming a danger to occupants in and around the buildings due to failure of the panels themselves. Fig. 19 is from Anchorage. The definition of "means of egress" does not include the words "from this world"! Falling masonry was also a hazard widely experienced in Caracas (4). Also see Fig. 23 (Managua Earthquake). If infilling panels are not to become just inadequate shear walls they must be separated more than in a merely token manner. That this applies also to infilling walls in shear wall buildings becomes self evident from Figs. 20 and 21. The exterior panels in the R.C. shear wall Elmendorf Hospital did not know that they were to be non-structural! Note again the spalling of face shells.

(d) Inducing building torsion. This effect is well known and covered adequately by the literature of earthquake damage (7).

An example of infilling panels that did not fail is shown in Figs. 24 and 25. The National Theatre was located on the south shore of Lake Managua in MM zone VII - VIII, north of the Palacio National. The surrounding concrete frames were very substantial and so was the thickness of the panels which was evidently sufficient to resist face loadings by arching and good tensile resistance.

### 3.3 Partition Walls

NZS.1900 Chapter 9.2 defines partition walls as walls which, by virtue of their position and construction, do not contribute to the strength or rigidity of a structure. Perhaps the definition might have added "but in most cases not for long", because the separation provisions of the current NZS Ch.8 are quite inadequate and even those proposed in DZ4203, (18) severe though they may appear to be, in ductile frames are adequate only for very moderately sized motions (½ El Centro N.S. for Class III buildings).

Rigid partition walls may thus produce the same adverse effects as those listed under infilling panels, particularly when located in framed buildings. The behaviour of unreinforced partition walls during the Anchorage earthquake was very poor and a combined team of consultants - California Institute of Technology, National Academy of Sciences, and U.S. Army Engineers - reported (16): "the extensive use of unreinforced 4" concrete masonry for non bearing walls contributed very extensively

to the total damage." "Had the shock occurred earlier when offices and industrial buildings were occupied or later when barracks bunks were occupied it is certain that loss of life and serious injury would have been tragically high." "Aside from the monetary aspect of damage to unreinforced masonry walls, it is readily apparent that in seismic areas due to the safety aspects, the use of unreinforced walls must be eliminated." The team recommended that "all masonry walls, both bearing and non-bearing, in seismic areas be reinforced both vertically and horizontally and be substantially anchored on both sides and the top edge."

Fig. 28 (Anchorage) shows that electric wiring is a "must" in buildings with 4" unreinforced partitions. Note remains of 4" partition overhanging doorway. Failure of the type shown in Fig. 29 was common and resulted in loss of life. Fig. 26 shows the danger of means of egress. Fig. 29 is proof that there is nothing peculiar about overseas earthquakes. The failure occurred in 1972, Te Aroha, New Zealand.

### 3.4 Detailing

Detailing has a profound effect on the reserve energy dissipating capacity of reinforced masonry, the mode of failure of walls and members built in to them.

The lesson from the failures shown in Figs. 30 and 31 is the importance of adequate amounts of horizontal reinforcement, suitably anchored and in particular the need for supplementary reinforcement in regions where higher local stresses may occur.

The need for close supervision of construction is evident from Fig. 33 (San Fernando) and Fig. 32 (N.Z. 1968) is shown to emphasize that "it can and does happen here too". Garden and boundary walls failed extensively at San Fernando (Fig. 34).

Figs. 35 to 37 from Anchorage show the disturbing modes of failure of walls containing little "basketing" reinforcing. Cracks follow no particular mathematically derived stress patterns but simply the line of least resistance into regions where there are no reinforcements to impede their progress. Whether or not the failure of these walls was secondary to that of the roof diaphragm is of little importance to the lesson to be learned.

### 3.5 Veneers

The most common form of masonry veneers in N.Z. is of the tied variety. Tied veneers can fail due to a number of causes including lack of adequate ties (Fig. 38, 1935 Montana). Poor performance has, however, also occurred where apparently substantial quantities of ties were used. Fig. 39 (Anchorage), Fig. 40, shows a veneer on timber failure from Inangahua, N.Z. 1968.

Tiled roofs are the cause of large seismic forces on the structure. To make matters worse the timber bearing walls of veneered, conventional construction are not

as stiff as those in a house sheathed with weatherboards. Additionally the lack of stiffness of the roof itself will introduce loadings to the top edges of the walls. The timber walls are more flexible than the veneer not only perpendicular to their plane but also in their plane and there is a tendency for the veneer to act as a kind of shear wall. The deformations of walls and partitions at right angles causes the veneer panels to be distorted, fail, and thus in turn, lose their capacity to act as shear walls.

Since veneer on timber construction is used in the first instance to reduce maintenance, its vulnerability to earthquake damage should be reduced. To achieve this, timber bearing walls must be significantly stiffened by sheathing with structural plywood or by other methods. Roof stiffness must also be improved and finally, the veneer ties themselves must be capable of withstanding very high loadings without sustaining deformations incompatible with the deformations of an unreinforced brick wall or tearing out of timber wall or veneer.

While the problem of damage to veneers on timber in low structures is largely a matter of economics, veneers on timber in high buildings may constitute a severe life hazard. Where there are no suitable alternatives to veneer on timber it is imperative that the veneers should not only be suitably supported against face loads but also be separated sufficiently from the structure to prevent the veneer from being subjected to induced in plane forces. This is much easier achieved in stiff shear wall buildings than in ductile frames. Whether the failure of the veneer is direct or induced is of little interest "to the man in the street".

Decorative screen walls in multi-storey buildings failed extensively during the 1962 Acapulco earthquake, Fig. 37 (17).

### 3.6 Conclusion

Notwithstanding significant progress, an adequate engineering description of the parameters of an earthquake damaging to masonry is not yet available. The irregularity of the average masonry structure and the non-homogeneous character of the material add to the problem.

Important lessons particularly as regards detailing may nevertheless be learned from the performance of masonry in earthquakes. Lightly, unidirectionally reinforced walls are seen to be poor energy dissipating members and to constitute a hazard to people in and around buildings. Infilling walls may significantly alter intended structural performance.

Careful attention must be paid to the anchorage of roof members in walls. The bracing of timber structures with tied masonry veneers must be improved and the adequacy of ties reviewed.

Used with common sense, within its limits as a material and constructed to a high standard, modern masonry may be

expected to perform adequately in a severe earthquake, but departure from proven sound seismic design and detailing principles is to court disaster.

#### 4. CREDITS

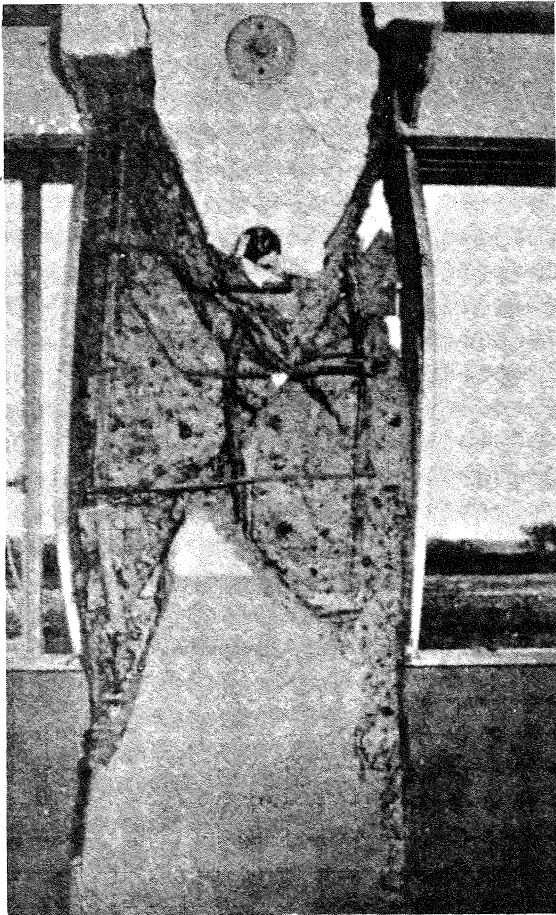
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FIGURE 1:  
HAKODATE COLLEGE  
TOKACHI OKI E.Q.  
1968 (1)



Heavily damaged short  
column ( $h_0/D=2$ )

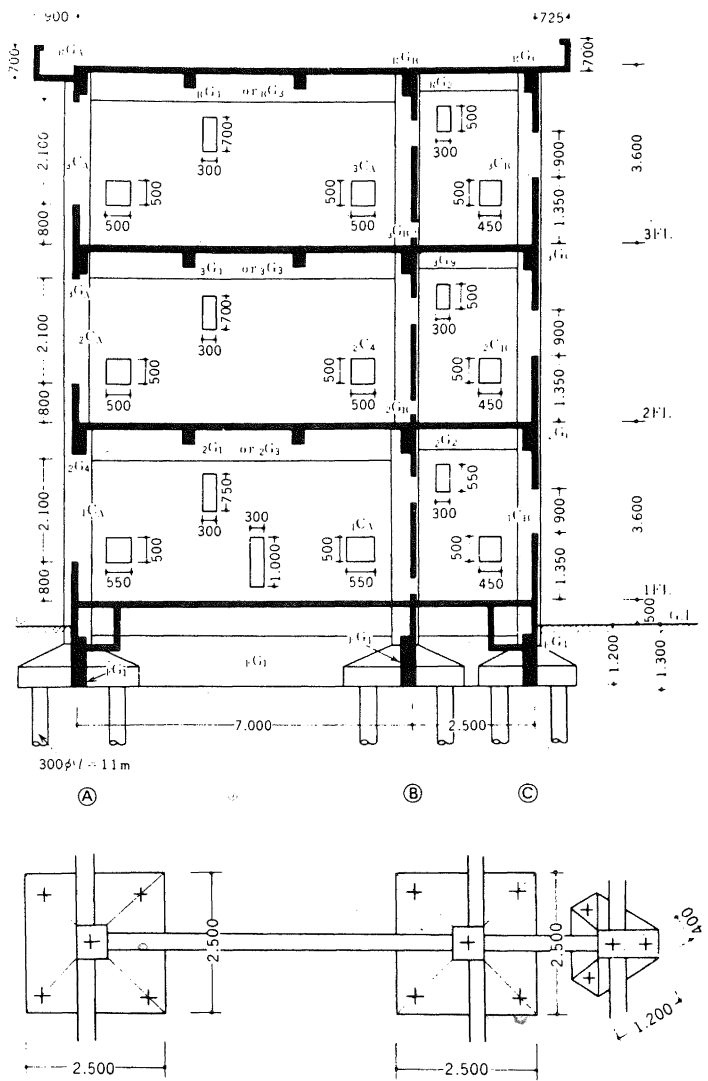


FIGURE 2: HACHINOHE TECHNICAL COLLEGE (1)

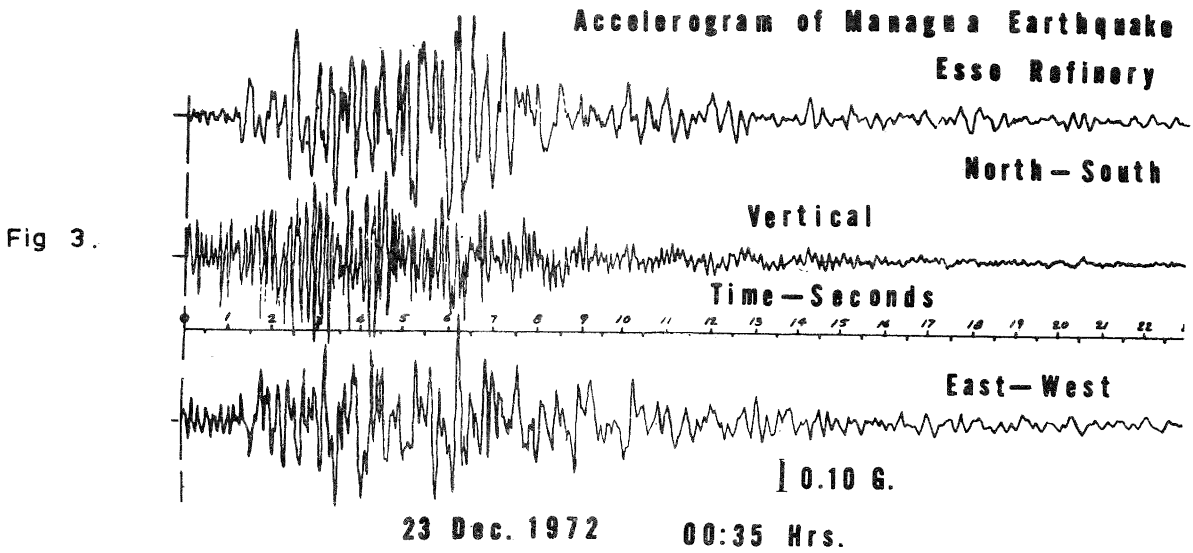


Fig 3.

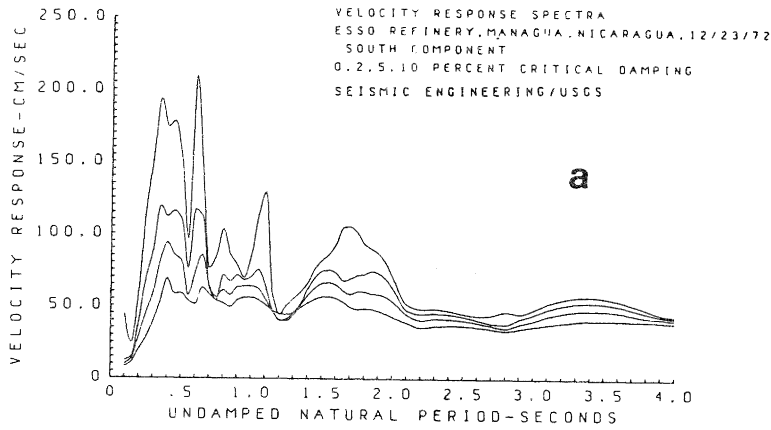


Fig 4.  
(2i)

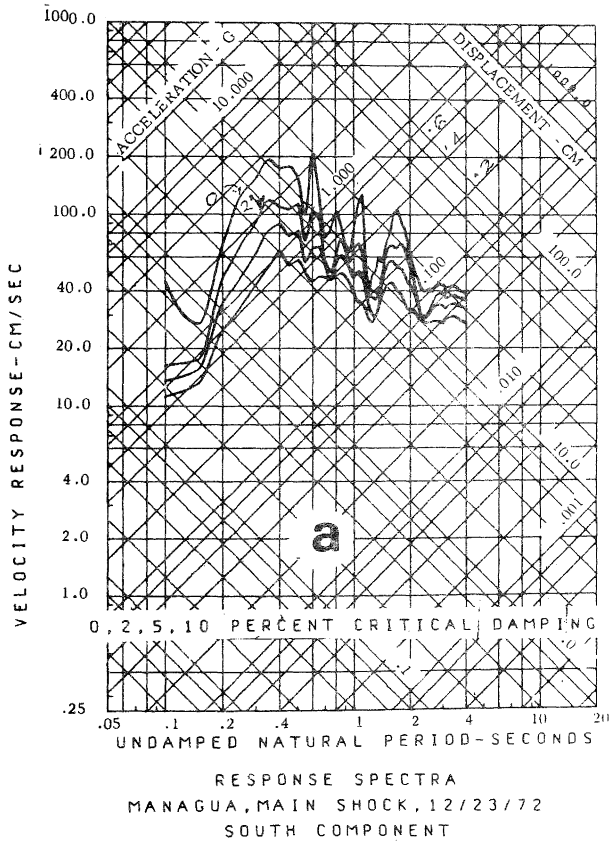


Fig 5 (2i)

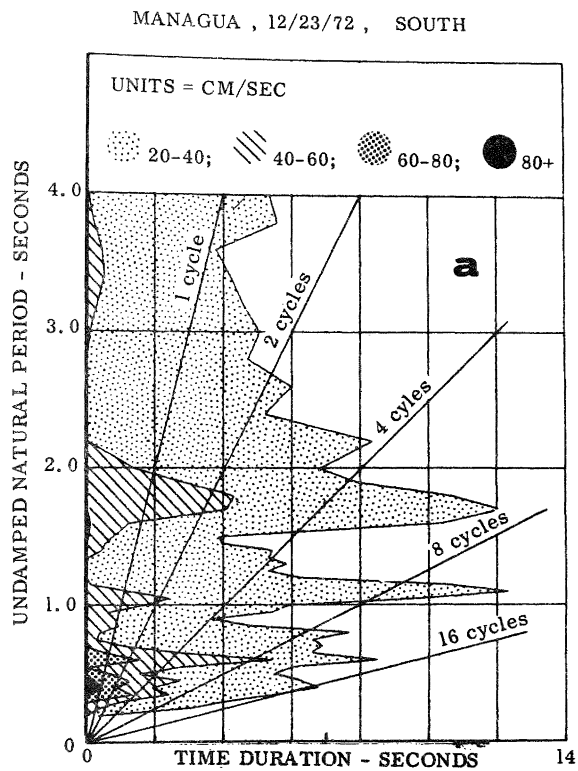


Fig 6 (2i)

Fig 7  
(2i)

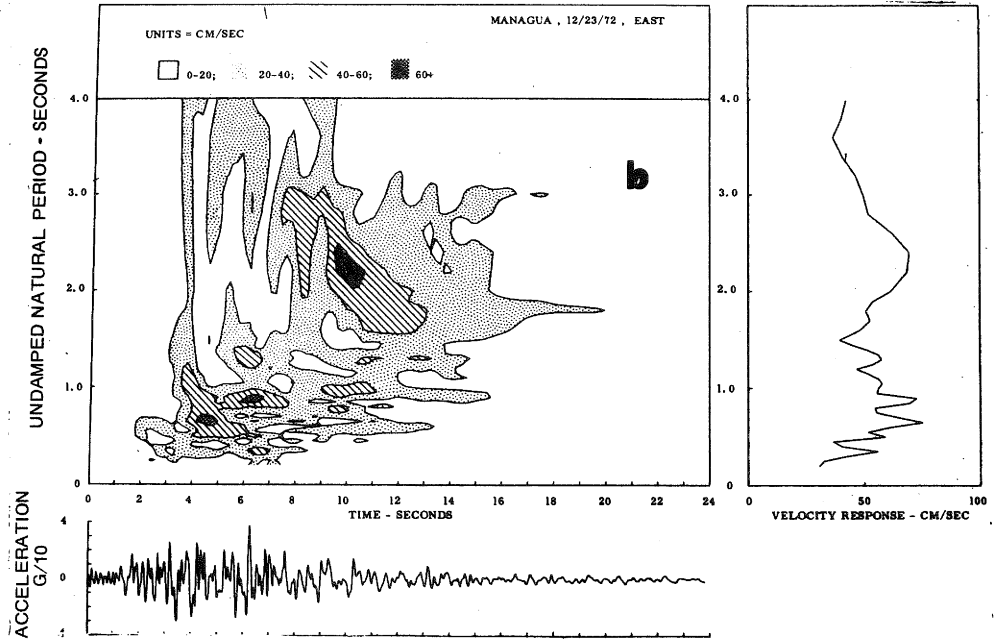


Fig 8  
(2i)

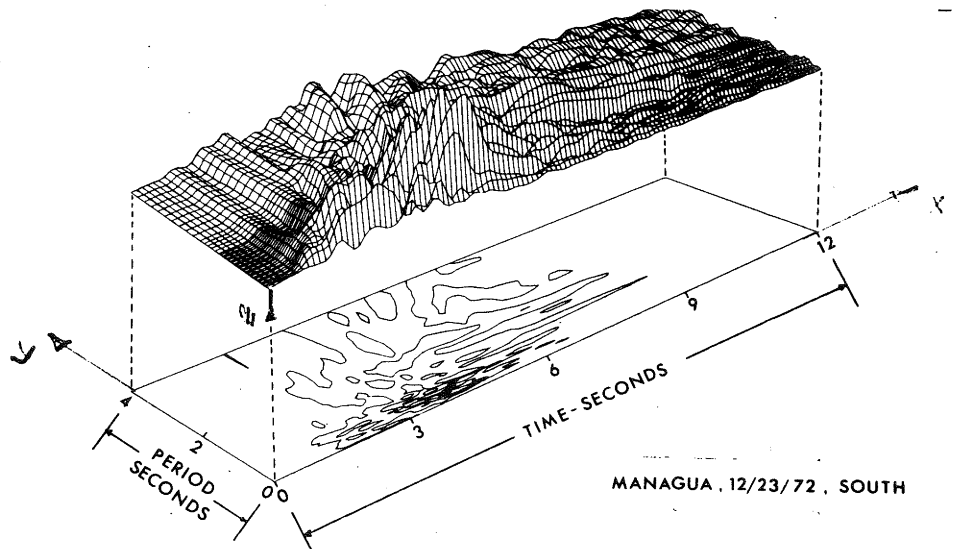
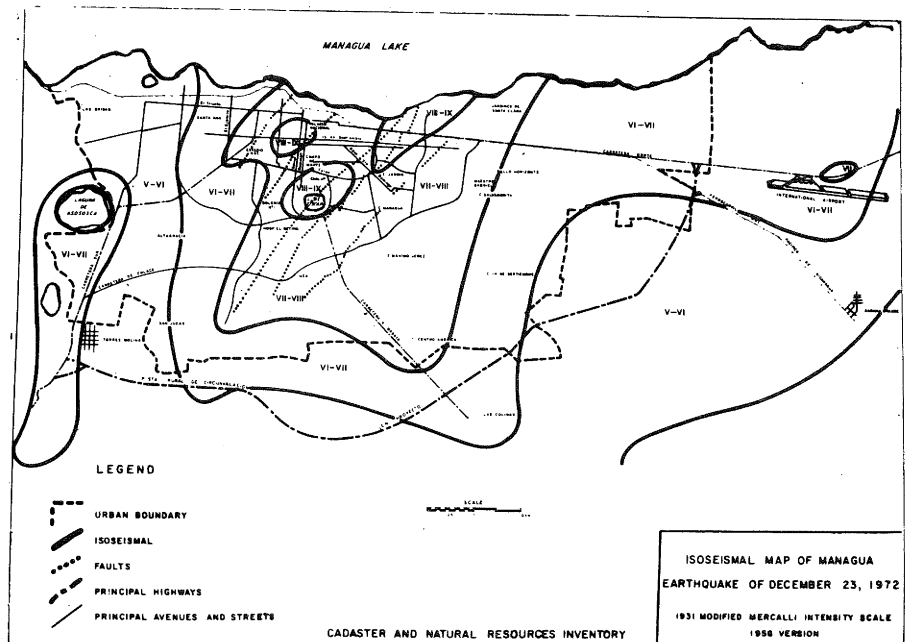


Fig 9  
(2i)



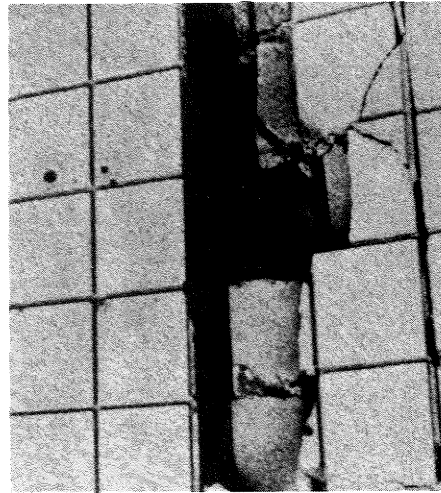
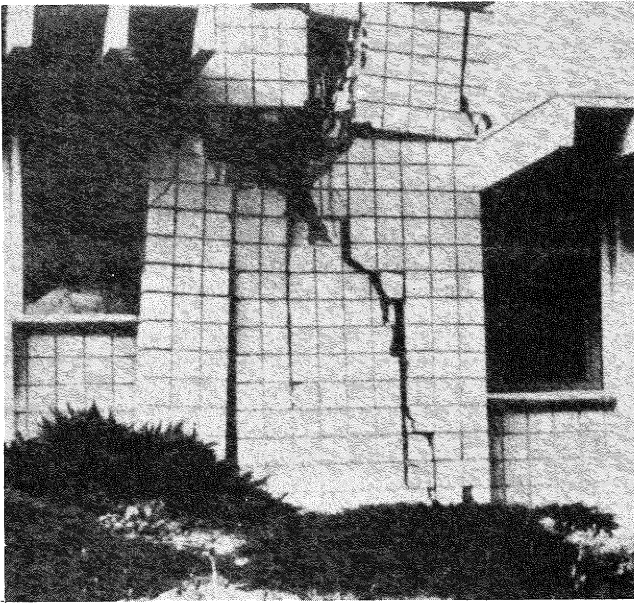


Fig 10 (6)

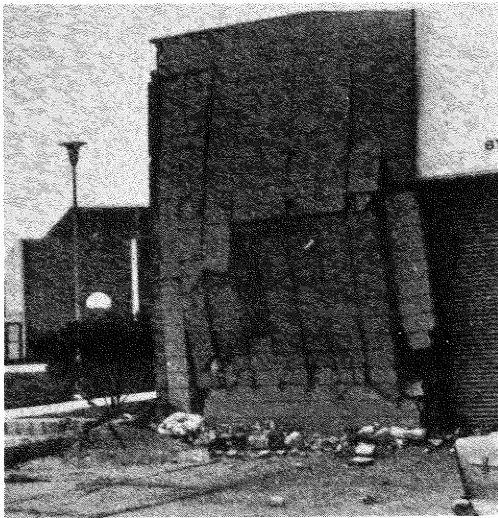


Fig 11 (6)

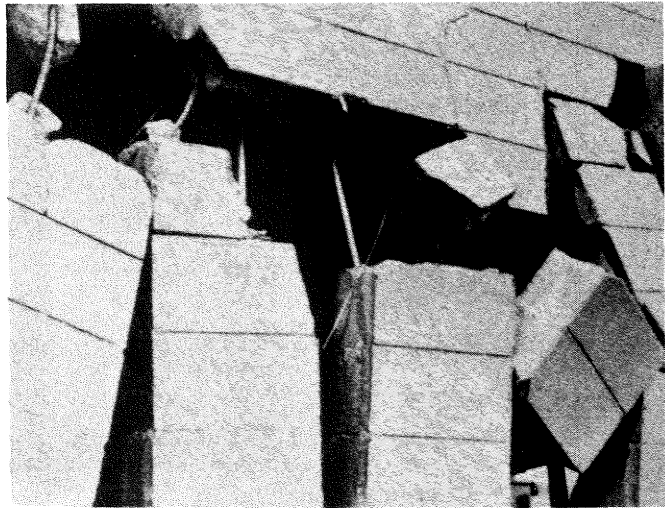


Fig 12 (6)

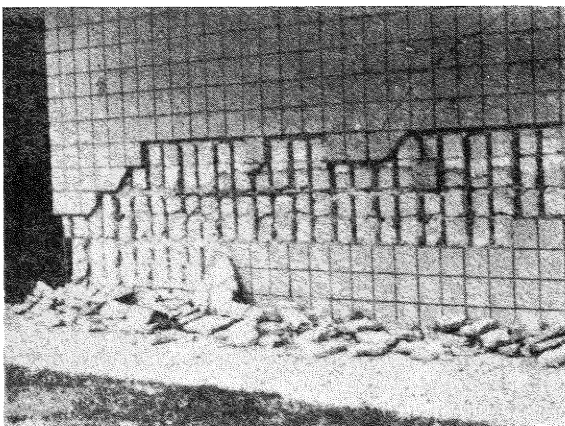


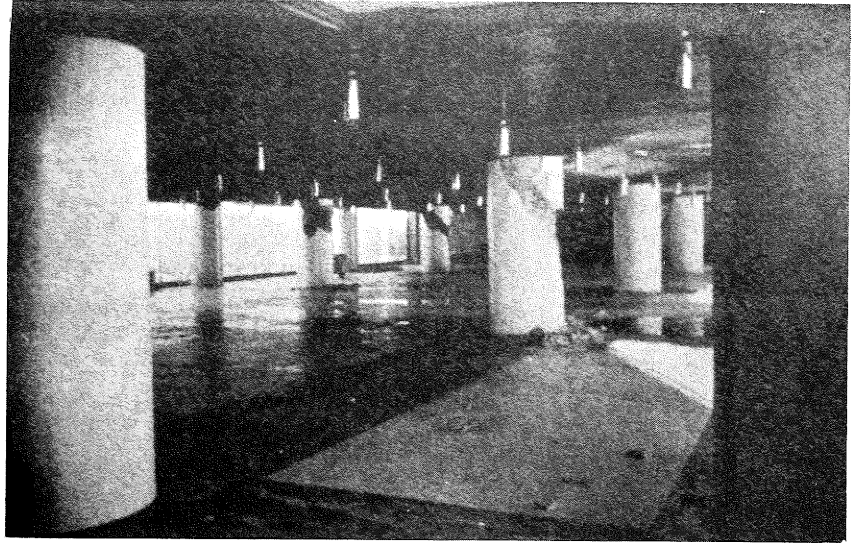
Fig 13 (6)



Fig 14 (6)

Mezzanine Lobby of  
Macuto Sheraton

Fig 15 (3)



Column Failure  
in Macuto Sheraton

Fig 16 (3)

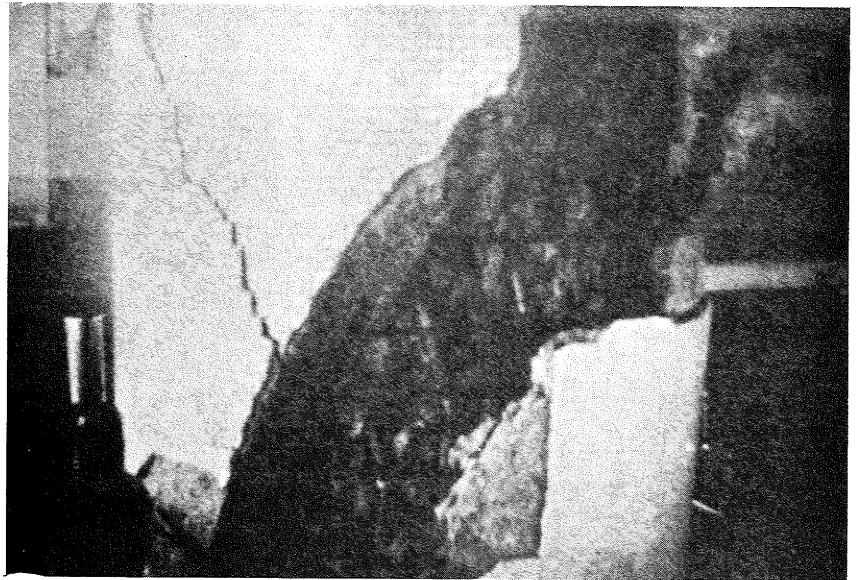
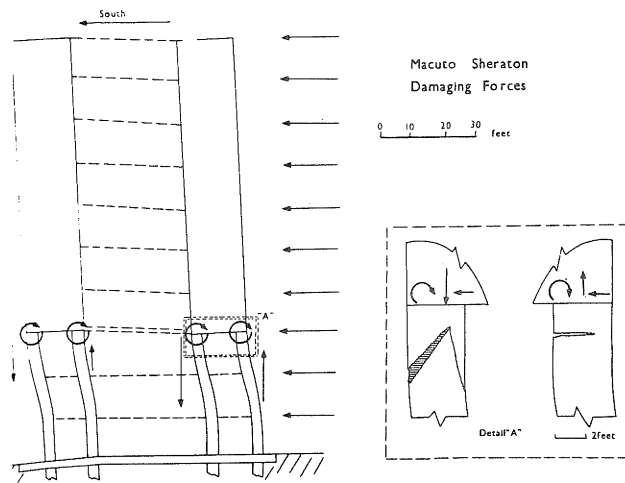


Fig 17 (4)



Macuto Sheraton, transverse section showing the column loadings that caused damage during the severe lurch towards the south.

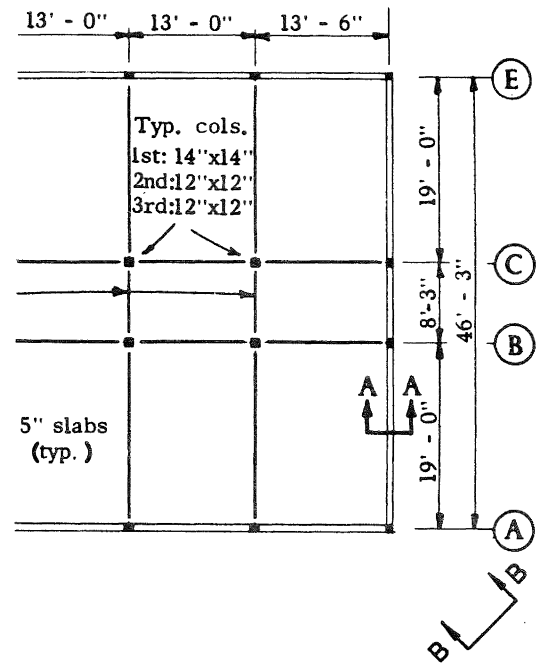
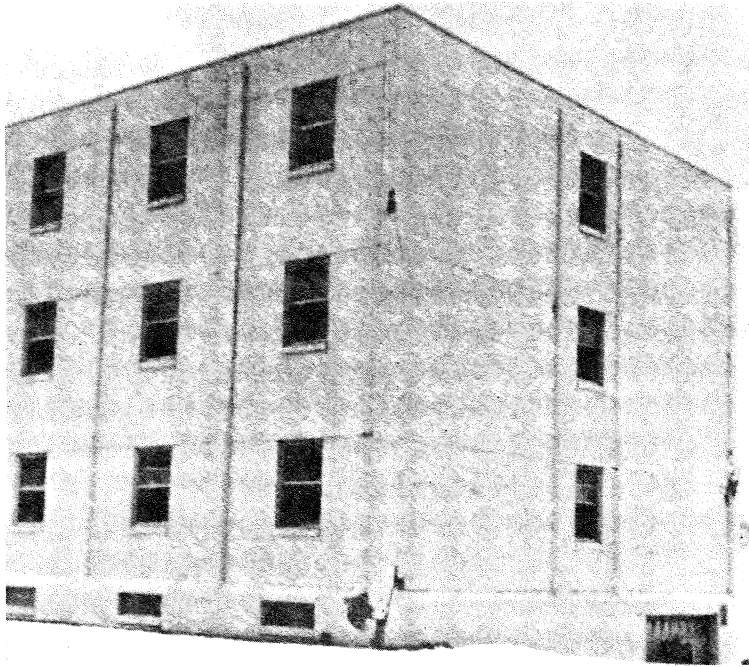
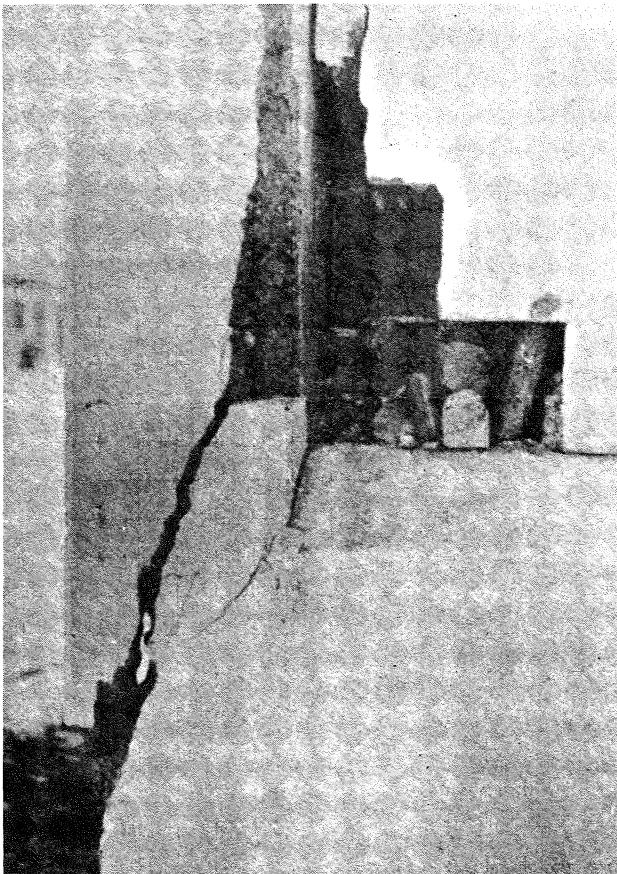


Fig 18 Barracks Building, Elmendorf Base (5i)

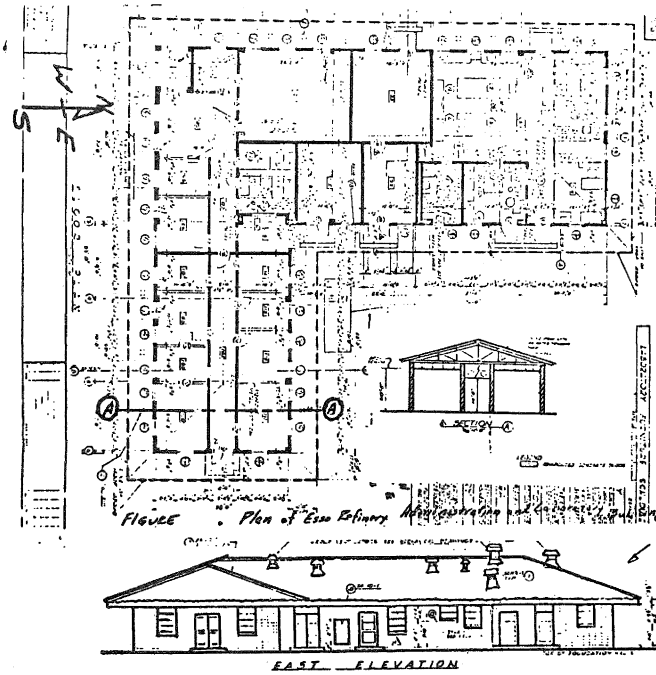


B - B



Fig 19 Hillside Manor Apartments. (5i)





levation of administration building and laboratory building

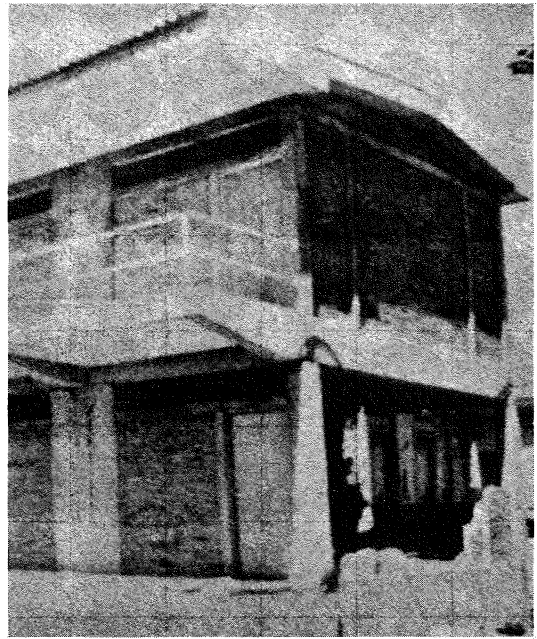
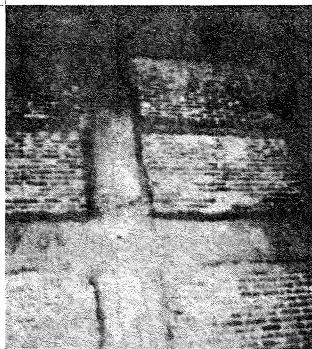


Fig 23 (2ii)

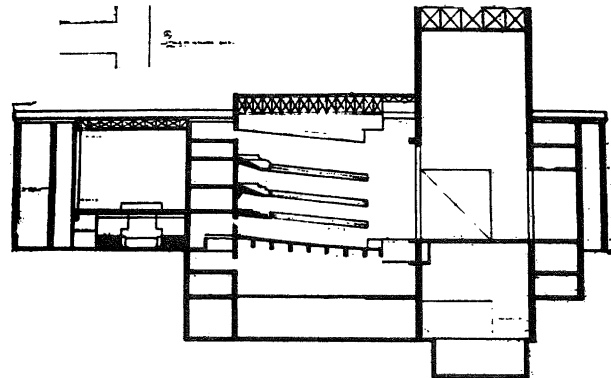
Fig 22 (2ii)



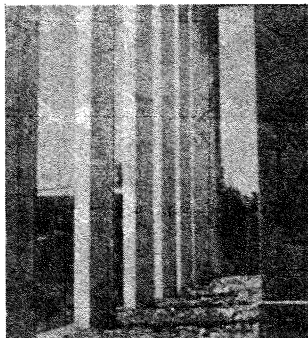
interior of masonry wall. Note tie member sub-dividing masonry infilled wall.



Spalled plaster off exterior of south wall.



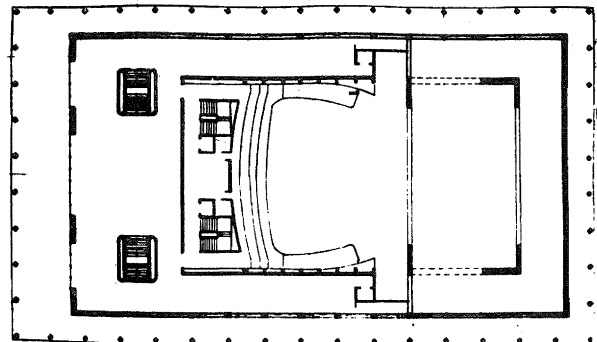
Longitudinal section



The marble veneer was not dislodged from the tall columns.



Slight ground settlement occurred around building.



Plan of Theater

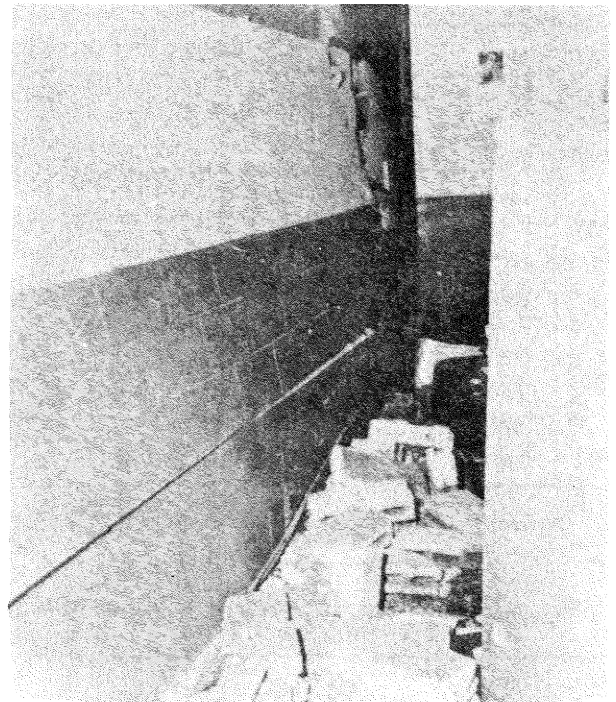
Fig 24 (2ii) National Theatre, Managua

Fig 25 (2ii)



-Stairs Littered with Debris, Banco Central.

Fig 26 (2 iii)



-Failed hollow concrete block partition in a barracks building. Elmendorf Air Force Base. U. S. Air Force photograph.

Fig 27 (5i)

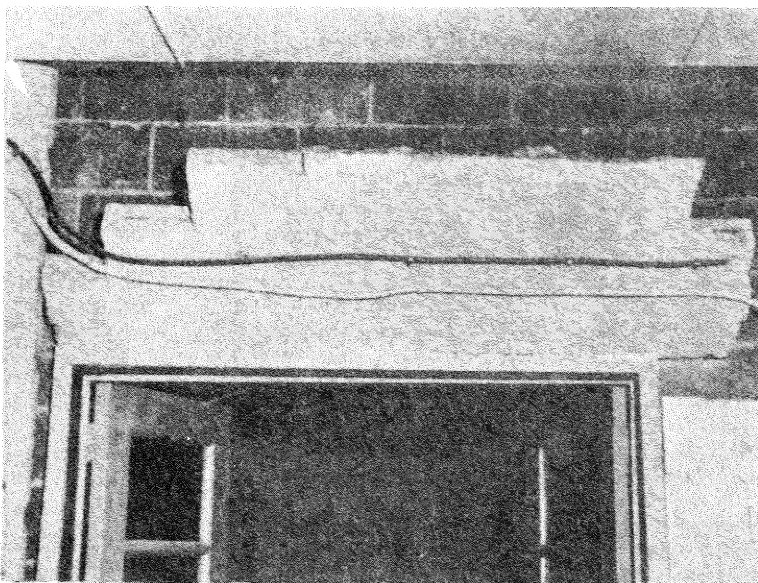
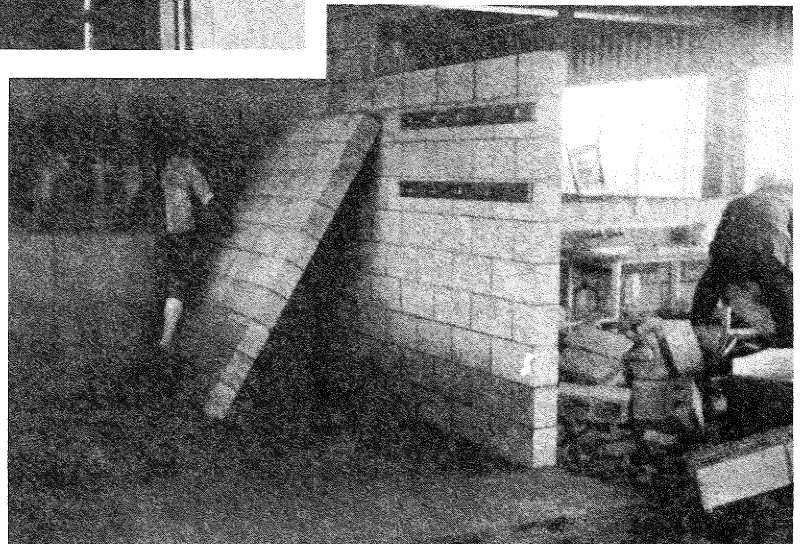
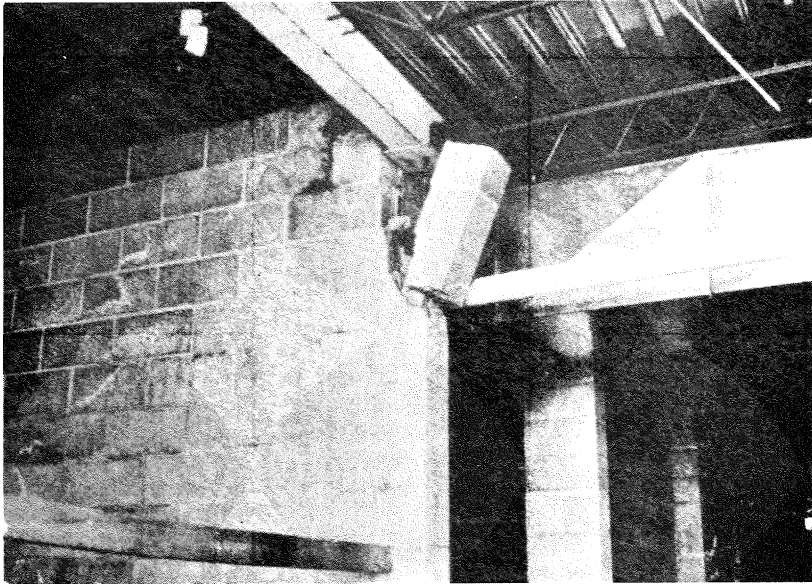


Fig 28 (8)

Fig 29 (5i)





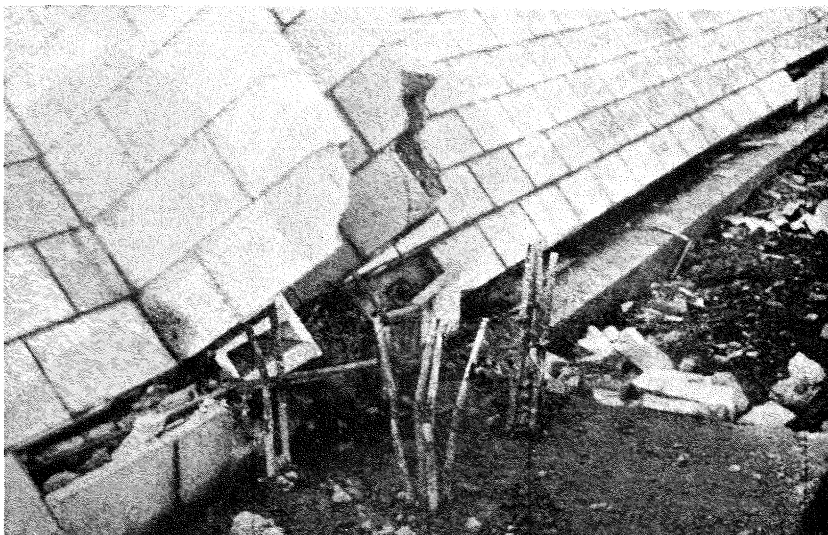
**Fig 30 (5ii)**

-Steel beam delivered load to end of concrete block wall. Romig Junior High School.

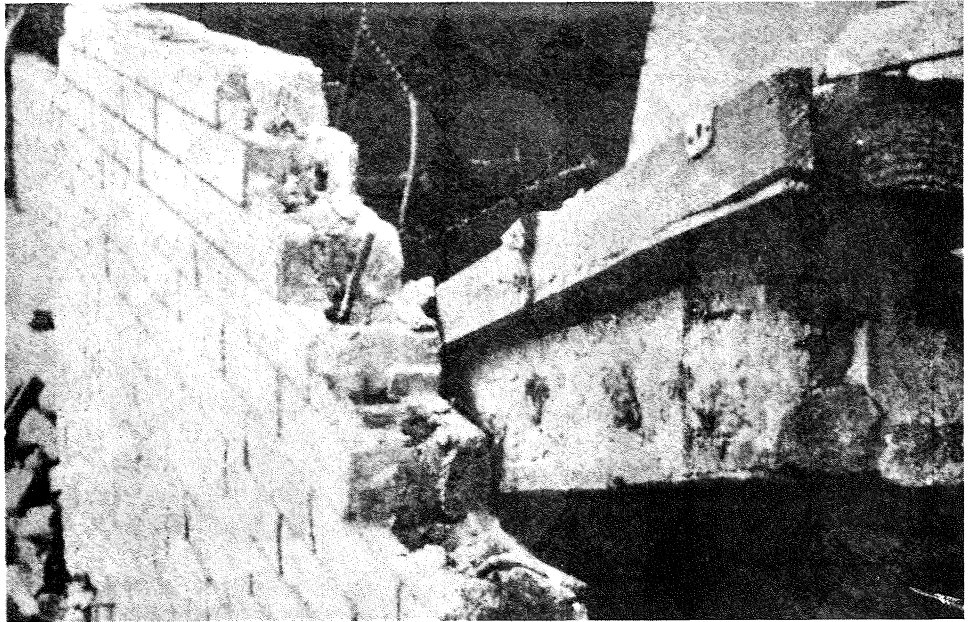


**Fig 31 (5ii)**

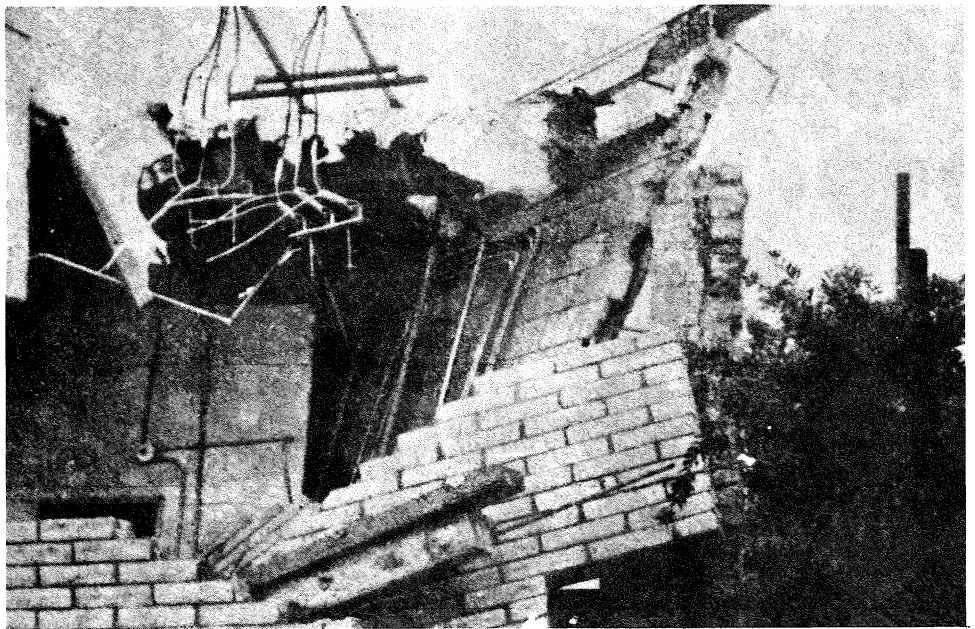
-View of west wall at north end of gymnasium. Note damage at beams. Romig Junior High School



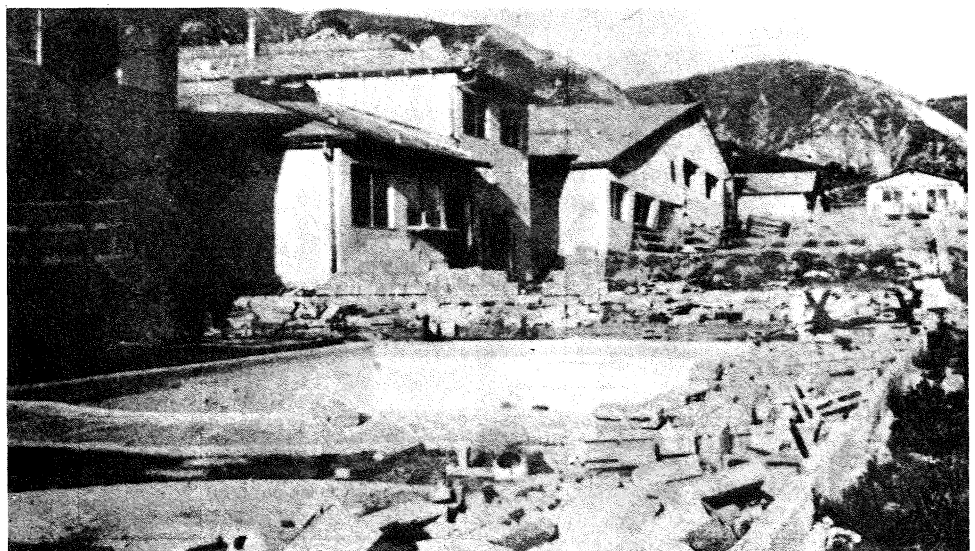
**Fig 32**  
**Storm Damage, N.Z. 1968**  
 Inadequate splice length  
 and excessive number of  
 bars in small cavity.

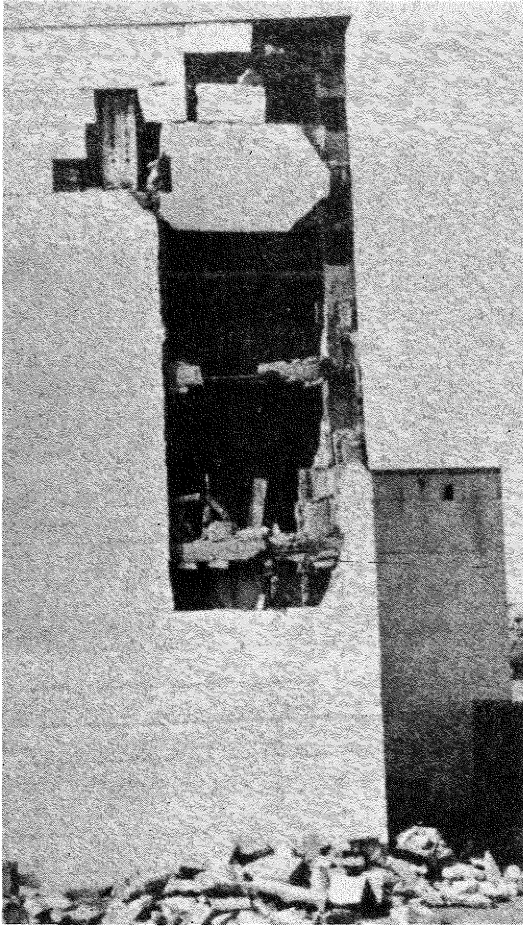


**Fig 33 (9)**  
**Liquor Store**  
**San Fernando .**  
**Discontinuous**  
**vertical reinf.**



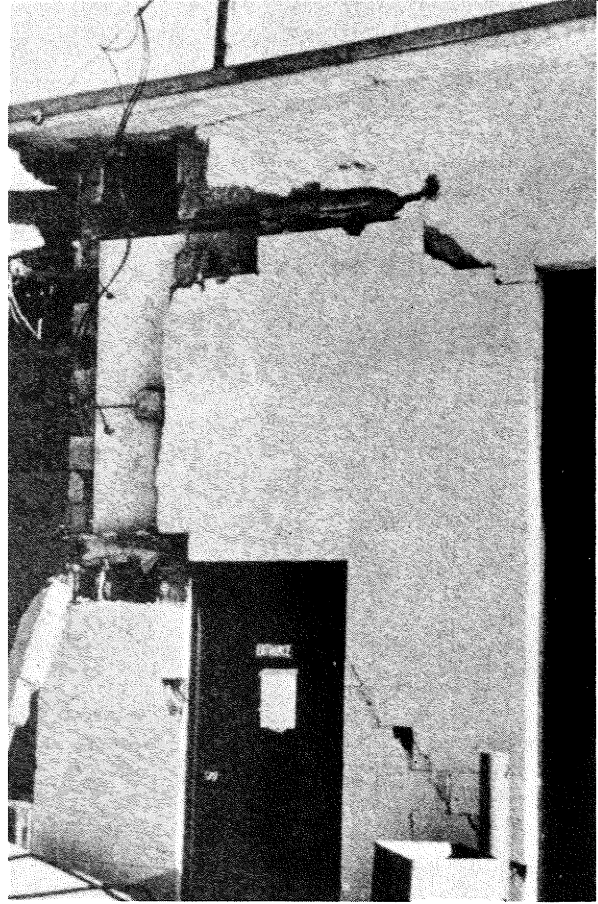
**Fig 34**  
**San Fernando**  
**Failure of**  
**boundary walls .**





Chrysler Center. *Frank McClure photograph.*

Fig 35 (5i)



—Damage at northeast corner of the Alaska Van and Storage Co. *John L. Cerruti photograph.*

Fig 36 (5i)

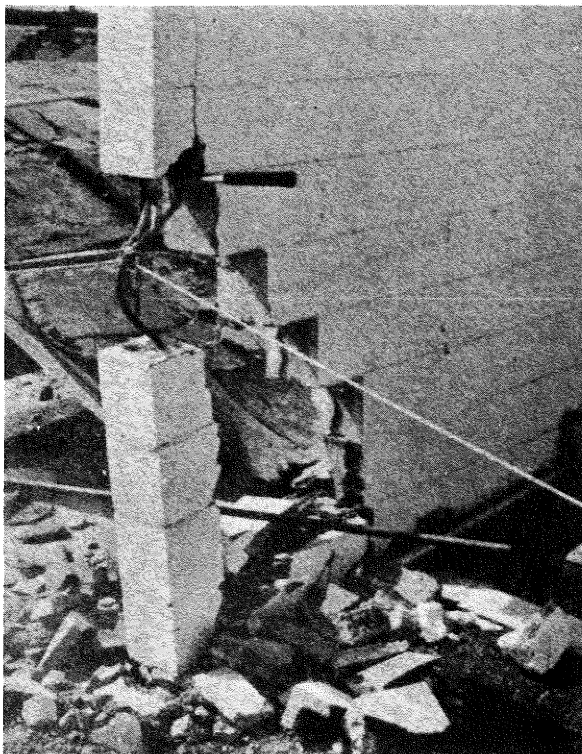
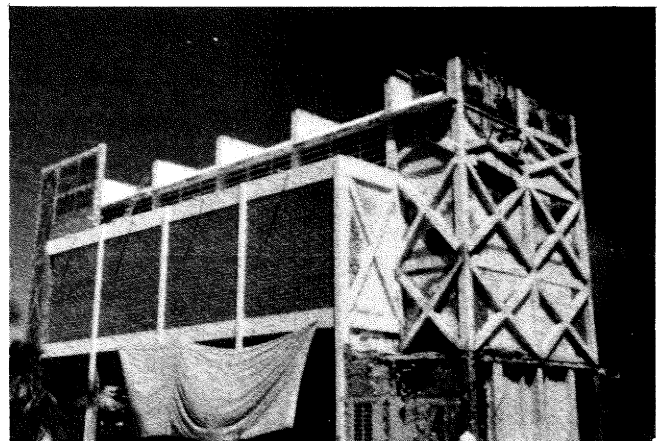


FIGURE 68.—Damage at base of the east fin suggests over turning stresses. *Frank McClure photograph.*

Fig 37 (5i)



Decorative masonry screen walls failed extensively during the 1962 Acapulco Earthquake (17)

Fig 38. (10)

-New Helena (Mont.) High School.  
lack of adequate ties. Helena, Mont., earthquakes  
1935. .

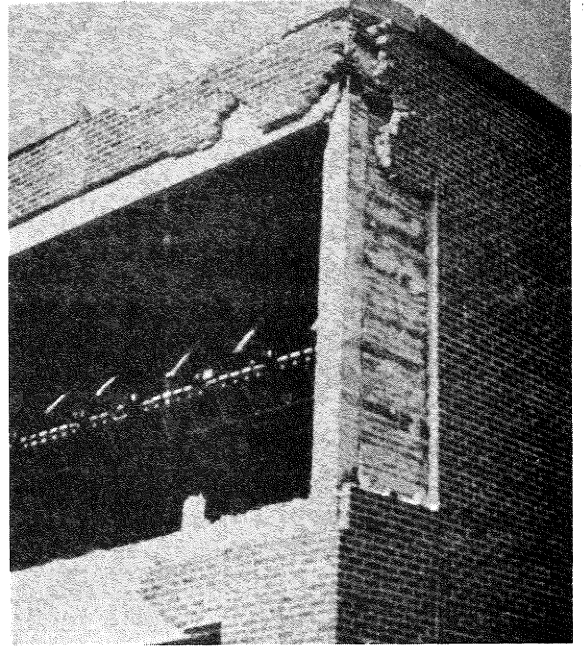
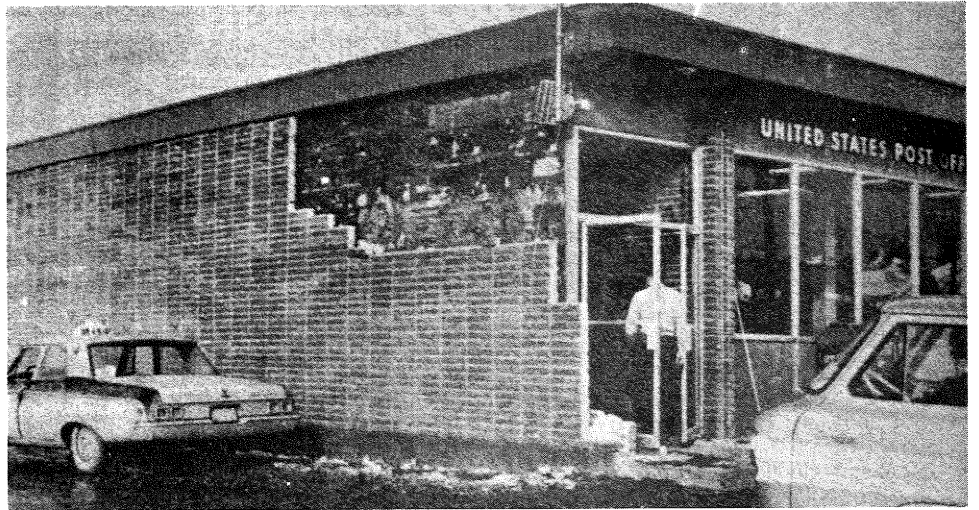


Fig 39 (5i)



-Damage to 4-inch brick veneer on a wood stud wall. Wall construction from inside to outside: Sheetrock on the interior face of the studs, 2- by 6-inch studs at 16 inches, 3/8-inch plywood on outside face of the studs, and 1 7/8-inch air gap between the plywood and the 4-inch brick veneer. Plans called for masonry veneer ties at 16 inches horizontally and 24 inches vertically.

Fig 40  
Inangahua  
E.Q. N.Z. 1968

