THE JULY 29, 1967 VENEZUELA EARTHQUAKE
LESSONS FOR THE STRUCTURAL ENGINEER

by

Henry J. Degenkolb[I] and Robert D. Hanson[II]

ABSTRACT

On July 29, 1967, a 6.5 Richter magnitude earthquake occurred in the Caribbean Sea. Although this moderate earthquake was 50 kilometers (30 miles) northwest of Caracas, Venezuela, many modern multistory buildings were seriously damaged and five collapsed. This is significant because these buildings were designed and constructed using a building code with seismic considerations similar to those in use today in the United States.

This paper reviews several important lessons for the structural engineer resulting from the observation of damage caused by this earthquake.

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At 8:00 p.m. local time (23H59m 58.75 GMT) on Saturday night, July 29, 1967, a 6.5 Richter magnitude earthquake occurred in the Caribbean Sea with epicenter coordinates of 10.56 degrees north latitude and 67.26 degrees west longitude plus or minus 20 kilometers. The focal depth was estimated to be about 15 kilometers. Although this was a moderate earthquake as compared to other important earthquakes, it killed about 266 people in North Central Venezuela. The epicenter was approximately 50 kilometers northwest of the capital city, Caracas.

Because this earthquake affected about 1000 buildings in Caracas that are 10 stories or more in height, causing five major collapses and serious damage to dozens of large buildings built in practical conformance to modern building codes and to modern construction practices, its importance to the structural engineer is much greater than the magnitude rating would indicate.

It is impossible in a paper of this short length to present in detail the damage patterns, the details of geology, the construction and the damage details for each of the many buildings affected in Venezuela. That has been and will be done elsewhere by other authors, by the Venezuela Presidential Commission which is preparing a comprehensive report of both the earthquake and their subsequent investigations, and partially by the present authors in another more detailed report. In this paper, it is hoped to summarize only certain specific lessons that may be of concern to the structural engineer who designs buildings to resist earthquakes.

Some of the types of damage found in previous earthquakes were surprisingly scarce in Caracas, so these will not be mentioned here. Other types of serious damage were either noted here for the first time, or the number of examples found were much greater than observed previously. It is this latter group of observations that will be more fully discussed here.

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A. Influence of Non-Structural Elements: This earthquake illustrated to an unprecedented degree a factor that many structural engineers have realized for many years: Non-structural elements can seriously alter the anticipated performance of the designed structure. The additional rigidity of non-structural walls can "attract" forces to elements not designed to resist them.

An excellent example of this effect is the performance of the Mene Grande Building, Figure 1. The design assumptions for this building were the same as used in many areas of the world, including Venezuela. All lateral forces were designed to be resisted by frame action, distributed in proportion to the calculated stiffnesses of the various frames. The tile walls were not assumed to carry lateral load nor to affect the stiffness of the bents in any way. Under this assumption the bents on the ends of the wings (A2-A4, for example) as shown in Figure 2 are approximately half as stiff as the interior bents (B2-B4, for example). However, tile walls solidly infilled these exterior bents at the ends of the building, greatly increasing their stiffness. The four exterior bents together with the core-connecting walls, actually took most of the lateral loads while the interior bents took very little. As a result, the corner columns were forced to take the major part of the overturning stresses, greatly overstressing them in vertical compression. Seven of the eight corner columns failed as shown in Figure 3.

The same influence of the non-structural infilling of walls caused column failure in many other buildings including Mobil Oil, Cypress Gardens, Covent Gardens, and San Bosco buildings.

Since these architectural, non-structural walls can be placed haphazardly in the building, from a structural point of view, they can introduce torsion where none was expected, and thereby additionally overstressing portions of the structure.

B. Overturning Forces: Aside from the column failures attributable to unwanted stiffness due to non-structural elements, this earthquake has indicated overturning forces greatly in excess of anything anticipated in previous studies, research or observations. Figure 4 shows the tension cracks that developed in the lower two floors of the columns in the 20-story Petunia II building. Other buildings, such as the 10-story Amalfi, showed similar tension cracking. The 19-story Caromay building, Figure 5, is curved in plan and had enough tile walls that it must have acted as a unit rather than as a series of individual bents. Four columns, as shown in Figure 6, suffered complete failure and two others were cracked and spalled. As can be seen from Figure 7, the failures were at the approximate mid-height of the column in the lowest story and have a symmetrical pattern that suggests failure from pure axial compression. Rough calculations indicate that the column stresses at vertical load are about 700 to 800 p.s.i. and that overturning stresses in the columns for the MOP Normas required loads or the 1967 Uniform Building Code Zone 2 loads are from 110 to 220 lbs. per square inch as shown in the following tabulation.

APPROXIMATE P/A STRESSES(1) IN CERTAIN CAROMAY COLUMNS					
		P/A DUE TO OVERTURNING EFFECT(4)			
COLUMN	Vertical Load P/A (2)	By MOP Normas		By '67 UBC-Zone 2(3)	
		By Bent(5)	Unit(6)	Bent Action(5)	Unit(6)
D4	800	150	130	140	120
D7	800	220	120	200	110
B9 and B1	700	150	180	130	170

(1) All P/A stresses based on gross section of column neglecting vertical reinforcing steel (p.s.i.).

(2) Vertical load includes 20 lbs. per square foot reduced live load that was probably not present.

(3) The 1967 UBC-Zone 2 forces were not reduced by the "J" factor.

(4) Base shear by MOP Normas = 1.32%. Base shear by Zone 2, 1967 UBC KC = .67 x 2.12 = 1.42%.

(5) Taking lateral by bents, whereby bents on lines 2 to 8 are about twice as stiff as bents on lines 1 and 9. This corresponds to usual design methods.

(6) Taking lateral as unit about longitudinal axis.

Since the actual tested cylinder strengths of the concrete were over 4000 p.s.i., it could be seen that current lateral force codes must greatly underestimate the required overturning forces. Assuming that the most highly stressed column (D7) failed at 4000 p.s.i. and taking the most liberal assumption of overturning stress, it can be seen that the base shear would be at least $\frac{4000 - 800}{220} \times 1.32\% = 19.2\%$ G. For engineers familiar with the Uniform Building Code Zone 3 requirements, it is obvious that the change from Zone 2 to Zone 3 requirements would have had little effect on the design sizes of the members that failed.

From details that are given elsewhere, it should be noted that the worst damaged columns in the Macuto Sheraton Hotel are interior columns with the greatest loads. These failures have been widely publicized. While bending and shear stresses were present to a very large degree, it seems evident that direct compression due to overturning of the shear walls above was the predominate primary cause of failure.

The preliminary indications are that the present building code requirements for overturning may have to be drastically increased.

The overturning force problem is related to another feature of column design that must be recognized immediately. It is customary practice to design columns for maximum axial load plus bending as required. A much more critical condition, as regards tension steel in concrete columns or splice plate connections in steel columns, exists under the condition of bending with little axial compression load, or even tension. This very critical design condition may be further accentuated by the vertical accelerations due to the earthquake motion - a force usually neglected in design computations or codes.

C. Continuity of Beam Reinforcing: This earthquake has again demonstrated the well known fact that actual earthquake forces are many times as great as the code specified forces. While experienced structural engineers familiar with aseismic design know this, and the Structural Engineers Association of California and the 1967 edition of the Uniform Code tacitly recognize this fact, no other major United States code gives even the slightest hint that this may be true and even the requirements of these above named codes permit most construction to ignore that fact.

For most buildings, under the provisions of these codes, it is still possible to analyze and combine stresses caused by the dead load, live load and code required lateral forces and design for the result. A superficial review of this procedure might indicate that this combination of design forces would give a reasonable factor of safety. However, in considering reinforced concrete beams and girders, there is one item where the factor of safety might be only a little more than unity. This item is the distance of extension of the top reinforcing bars beyond the columns.

Figure 8 shows a typical concrete beam in a frame designed to resist earthquake forces, and the moment diagram that may be used to determine the required extension of the top reinforcing steel. In the upper moment diagram, the dashed line is the design vertical load moment diagram. If there were no lateral load, the top reinforcing steel in the beam could end slightly to the right of "A". When the code required lateral forces are added, the design moments of vertical and lateral are added, and the bars must be extended to slightly beyond Point "B". Many engineers, especially those trained to design for specific forces and with maximum economy, end all top bars at this point. The lower set of moment curves illustrates the fallacy of this attitude. If the lateral force moments are several times greater - shown in the diagram as about three times greater than code requirements - the combined moment curve indicates that top reinforcing steel is required up to Point "C". Since there is no required reinforcing steel at this point, the beam would collapse. This design is in full conformance to all current building codes. In the United States, there is often, but not always, a reserve strength. Our spans are quite long, our slabs are fairly thick, and we have appreciable shrinkage and temperature stresses, so there is usually some slab reinforcing steel parallel and adjacent to the girder or beam. This helps to reinforce the top of the beam and give it at least some minimal strength. However, in Caracas, using similar codes to ours, the joists are usually about 16-inches apart, the temperature is mild and there is usually little, if any, reinforcing steel parallel to the joists. Joists are calculated to resist lateral forces in frame action but have no top steel, similar to some designs in the United States. The beams usually have stirrups, therefore, there is some minimum top reinforcing steel all the way through the length of the beam.

In the authors' opinion, this absence of top bars in the joists, together with lack of slab reinforcing parallel to the joists, was probably the primary cause of at least two of the major collapses in Caracas. And this practice would conform to every current building code or specification in the United States today for the buildings that collapsed. The fact that most structural engineers in California go further in their practice and do provide the continuous steel - they know better than the code - is no excuse for the condition to exist. The very fact that the authors have to use the word "most" rather than "all" indicates an almost criminal neglect on the part of the profession as well as the code and specification writing bodies. The only code requiring some percentage of the reinforcing steel to go completely through the top and bottom of the beam is that of the "ductile moment resisting" space frame requirements in the 1967 edition of the Uniform Building Code and the SEAOC Code. These are only mandatory for buildings over 160-feet in height.

D. Column Ties: The longitudinal bars in concrete columns must be supported by adequate ties to prevent their buckling. At least one and usually many illustrations of the importance of this detail can be found in any important American earthquake. Caracas, with code requirements for column ties that are even more severe than ACI-318 in this regard, had many, many, illustrations of the importance of column ties.

Every one of the many scores of major column failures showed buckling of the longitudinal steel, resulting from failure of the ties. Only in the failures in the Macuto Sheraton did the ties fail by tension necking down and parting. These columns had much more tie steel than our codes require. In most other cases the hooks pulled or the ties loosened in some manner. Figures 9 and 10 show column failures in the Marco Aurelio and San Bosco buildings and are indicative of many failures in many buildings. The importance of ties is being diminished or overlooked in recent United States codes. The latest ACI Code has reduced the requirements for column ties in that some cases not all bars have to be tied.

In an actual column that has been in service in a building for some years, the longitudinal column steel is at a much higher stress than determined in the design calculations. This is due to the creep in the concrete and the resulting transfer of load to the column steel which does not creep appreciably. Then, if the column is subject to sudden overload, as in an earthquake, the steel tries to take its proportional part of the sudden increase in load.

The net result is that the bars tend to buckle. The confinement necessary to restrain the bars from inelastic buckling is much greater than when they are acting elastically. If this restraining force is not adequately supplied through ample ties, the bars will buckle outward, spalling off the portion of the concrete outside the longitudinal steel. This action takes place suddenly and when it occurs the remainder of the column, without benefit of the area of the outer 1-1/2-inch or 2-inch shell of concrete, is trying to support a load that was previously carried by the entire area of the column plus the reinforcing steel. Only disaster can result - and in many buildings in Caracas it did.

E. Column-Beam Connections: The majority of the energy absorption capacity of a structure exists in the members. The utilization of this energy absorption capacity requires that the connections be, at a minimum, as strong as the members themselves. This could mean that the connection should remain elastic while the members were free to deform in the post-elastic range. Numerous examples of connection failure were seen in this earthquake. Although the details for these buildings are the same as those specified in current United States codes and are in common use these connections are not adequate for earthquake resistant design.

Quite a few of the connections shattered as shown in Figure 11, a connection in the fourteen story Covent Gardens. This, and other connections are designed in a very similar manner to those in most of the United States although they would not qualify under the new "ductile moment resisting frame" requirements of the limited Structural Engineers Association of California code. Possibly, in this specific connection, there was insufficient shear area to transfer the moments from beam to column. Certainly, from the smooth appearance of the underside of the column and the offset of the column, there may not be adequate shear transfer from the column to the top surface of the beam. The failure in this figure might indicate that the slab concrete is weaker than the column concrete. If so, it is an example of why engineers must be fully aware of the requirements for the transfer of column stresses through the beam or floor slab, especially at corner columns where confinement of the concrete is lacking on two faces.

Column bars must be tied against buckling throughout the depth of the beam. Although current codes do not exempt ties in this area, the almost universal practice in the United States as in Venezuela, is to omit column ties in the beam depth. The failure of the joints in the airport tower in Anchorage, Alaska vividly illustrated the error of this practice. The joint failure in the Mene Grande Building, shown in Figure 12, reemphasizes this lesson which has not been learned and corrected in our codes almost five years after the 1964 Alaska earthquake.

F. Shear in Wide Flat Beams or Slabs: Certain slab failures in the Alaska 1964 earthquake gave a hint that our methods of calculating shear where a slab joins a column could stand improvement, especially where accidental or calculated moments must be transferred. Numerous cracks and shear failures in the wide flat beams, which act similarly to a slab of the Caracas buildings substantiate this hint of trouble to come. Figures 13 and 14 show beam shear failures in the San Bosco and Coral buildings. Most of these cracks were overlooked in the early stages of the investigations because of the sensational evidence of numerous column failures. After finishes were stripped and tile walls removed, it became obvious that the numerous cracks in the shallow beams near the columns were as potentially important as the column failures. The cracks adjacent to the columns in the Covent Gardens building, shown in Figure 15, while not spectacular, were alarming. While the reasons for the failure of the San Jose building may never be known, the architectural recesses at the front wall columns (Line 2), as shown on the typical floor plan in Figure 16, certainly did not help a shear situation that proved weak on other buildings with a greater area of contact of beam to column. Those of us in the United States that must furnish our architectural clients with the prominent, accented vertical line of the column in a tall building can sympathize with engineers all over the world faced with similar situations. Possibly it is time to exercise some restraint in the permissive use of this type of feature that greatly affects the performance of our structures in damaging earthquakes.

G. Sudden Change of Stiffness in the Framing: It must always be remembered that earthquake forces are inertia forces originating in all parts of a structure determined by the product of a moving mass and its acceleration. Most seismic codes, however, use pseudo static forces for analysis purposes and the dynamic effect is often unknown, overlooked or neglected by the designing engineer. Consideration must be given to the energy absorption capacity of the structure as it undergoes cyclic motion and distortion.

The structure must be able to transmit these seismic forces all the way from their point of origin at the mass positions to the underlying foundations.(III) If there is a single zone of weakness in this path of force transmission, or if there is a sudden change of stiffness, there is a zone of danger. Since this is such an important concept, and one not readily apparent to the engineers using the pseudo-static forces of a building code, it may be well to state it in another way.

If it can be assumed that our code required lateral forces are based on the performance of an older style "typical" structure where there are no sudden changes of stiffness, then the absorption of the earthquake energy through the foundations is distributed throughout the structure, either uniformly or in some regular continuous pattern. If we now consider a structure with a flexible portion under a rigid portion most of the energy absorption is concentrated in the flexible portion and very little is absorbed in the more rigid portion above. This would affect the magnitude of the shear force in the flexible portion quite drastically. Many examples of the results of this energy concentration were noted in Venezuela. Most of the taller apartments had many tile partitions and tile exterior walls that tended to act as shear walls at least until the tile failed. The ground floor, however, was often devoted to commercial

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III These statements conform to the usual convenient but incorrect assumption that the forces originate in the structure and are transmitted to the foundation. The authors are fully cognizant space or automobile parking so the tile walls were not continued to the ground. This concentrated the energy absorption in the first story with the consequent damage at that point. It is perhaps significant that all four of the collapsed buildings in Los Palos Grandes had this characteristic. The only discernible difference between the front and rear portions of the Palace Corvin was the amount of tile walls in the first story. This may hace been a major factor in the column failures of the Macuto Sheraton where the failures occurred just below the shear walls above the mezzanine. The San Bosco, Maria Luiza, Amalfi, Alta Mira, Sucre, Blue Palace, and many other buildings had these characteristics to a greater or lesser degree.

There is a strong architectural tendency throughout the world to have an open first floor - to place the building on "stilts" as it were. As one California structural engineer aptly put it - "Architects like to build their buildings with no visible means of support." It cannot be emphasized too strongly that current earthquake code requirements are <u>not</u> based on this type of dynamic stiffness distribution, and potentially there is a great probability of damage where these buildings are built to minimum code requirements in areas subject to great earthquake shocks. The experience in Caracas gave ample warning by way of damage to many buildings as to what may happen on the West Coast of the United States.

The authors note that in the Midwest there is a resurgence of the "flexible first story" concept of design so much debated in the 1920's. A thorough study of the basic concepts of this type of design together with all of the resulting implications should be made before it is attempted on a major structure that may be subjected to moderate to great earthquakes.

H. <u>Diaphragm Stresses</u>: In providing a continuous stress path from load to support, it is necessary to connect the floor or roof slab (diaphragm) to its resisting element. If the resisting element is a shear wall or cantilever pylon, there is often an accumulation of stress in the slab that requires special reinforcing to tie the slab to the shear wall. In the empirical method by which many engineers design, and in the absence of specific detailed requirements in all current codes and specifications, this critical area of design is more often overlooked than considered.

The central elevator and stair core of the Palace Corvin, shown in Figure 17, was seemingly designed to resist lateral loads as a vertical pylon. Yet, there were paper separators in the construction joints where the two wings joined the core, as shown in Figure 18, so there was no way for the transfer of the diaphragm forces to this resisting element.

III (continued)
of the fact that the motion and consequently the forces originate in the foundation and are transmitted upward. However, for our purposes here, the convenient inverted assumption is adequate and is in closer conformity to many engineers' visualization of the problem.

Figure 19 shows the typical floor plan of the thirteen story Bahia Del Mar, with the core shear wall extended to the fifth floor. Figure 20 shows the "tear" in the slab near Line 5 where the longitudinal forces generated in the right half of the building pulled the slab away from the shear wall. This type of failure has been observed before, but seems to have been overlooked by many engineers.

I. Pounding Damage: Previous earthquakes have repeatedly illustrated the damaging effects of the pounding of one structure against another when both are in motion during an earthquake. Although there were some examples in Venezuela, this type of damage was relatively rare because most multistory buildings in the areas reviewed were well separated. Figure 21 shows the canopy at the front of the Macuto Sheraton and the effects of the pounding of the concrete roof slab against the tile walls. After the wall and roof slab were removed, it was found that this pounding also shattered the concrete columns, as shown in Figure 22.

An extreme example of interstructure damage is shown in Figure 23 where the penthouse of the collapsed Mijagual building fell against the Nobel building, damaging the corner of the Nobel building.

J. Foundation or Geological Effects: Strictly speaking foundation effects are not as yet lessons learned from the Venezuela earthquake. Potentially, however, this could be the most important single item of new knowledge to result from this earthquake. Why were so many failures of high rise construction concentrated in the Los Palos Grande area when low buildings and walls were undamaged? Why were there so many one story buildings in the northwest part of the city demolished when the high rise buildings were relatively untouched? Why was the Caraballeda area hit so hard that both small and large structures were badly damaged or demolished? Undoubtedly there is some geological or subsoil reason, but a review of the damage as related to depth of alluvium does not indicate a simple relationship. It is understood that the Venezuelan Presidential Commission is undertaking an extensive investigation and analysis program. If this is successful, a major step in our knowledge of earthquake response will have been made. The pattern now is unclear, but it is definite that there is a pattern. It is greatly to be hoped that the Commission will be able to find the pattern and the reasons for it.

CONCLUSIONS

Although the Venezuela 1967 earthquake was only moderate in magnitude, the fact that it affected many hundred tall structures of better than average design and construction indicates that there may be several lessons to be learned by structural engineers. Some of these lessons are new and some of them are simply restatements of observations of past earthquakes which engineers and code writing bodies have not acted upon. Among these observations are the following:

1. Non-structural elements may drastically effect the assumed relative rigidities of resisting elements and consequently introduce damaging stresses.

2. Overturning forces are much greater than previous information would lead engineers to suspect.

3. Codes must be changed to require a certain amount of top and bottom reinforcing steel to extend the full length of beams and girders.

4. The quantity and size of column ties required for earthquake loadings is greater than for static loadings, and greater than present code requirements.

5. Column-beam connections failed for several possible reasons. Since these are critical for the ductility required for earthquake loading conditions, more attention must be given to their design.

6. Certain shear failures in wide flat beams may indicate caution in using present day design methods for shear in slabs.

7. Sudden changes in stiffness throughout the height of a structure accentuate strength and/or ductility demands in certain areas, similar to "notch effects" in structural members.

8. Floor and roof diaphragms must be positively and adequately connected to the resisting elements.

9. Interstructure pounding damage can be severe during an earthquake.

10. Possibly the greatest potential lesson to be learned from this earthquake, soil and geological effects, is still in doubt since there is ample evidence of a pattern but the relationship of the pattern to the cause is not clear.

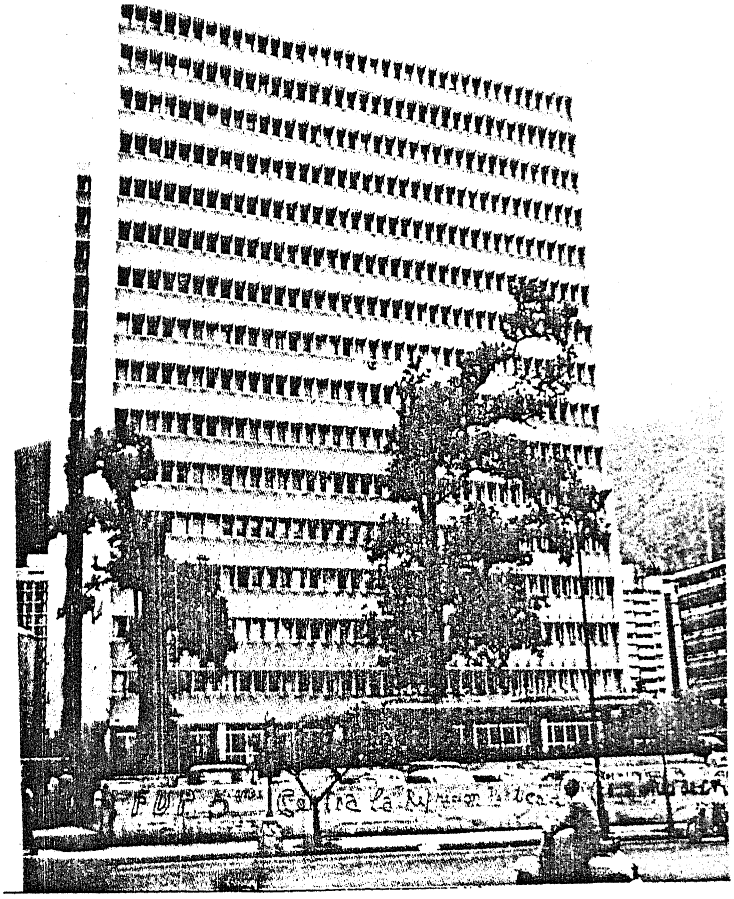


Fig. 1 - Mene Grande Building

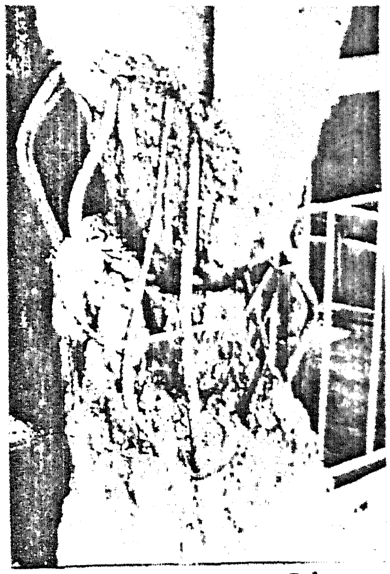


Fig. 3 - Failure of Corner Column, Mene Grande Building

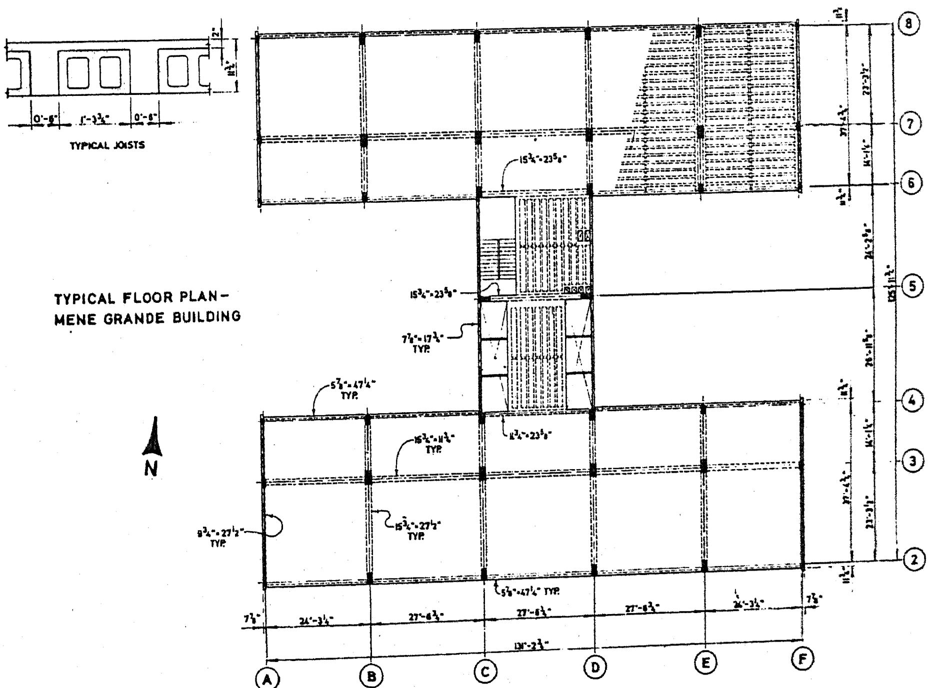


Fig. 2 - Plan of Typical Floor, Mene Grande Building

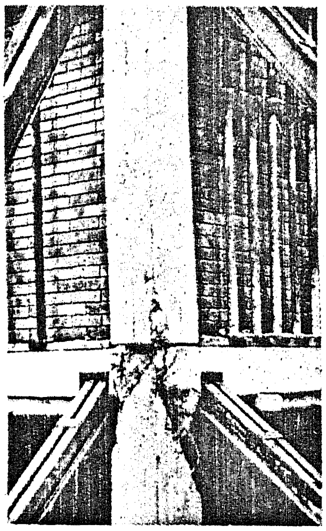


Fig. 4 - Failure of Column, Tension Cracks in Petunia II

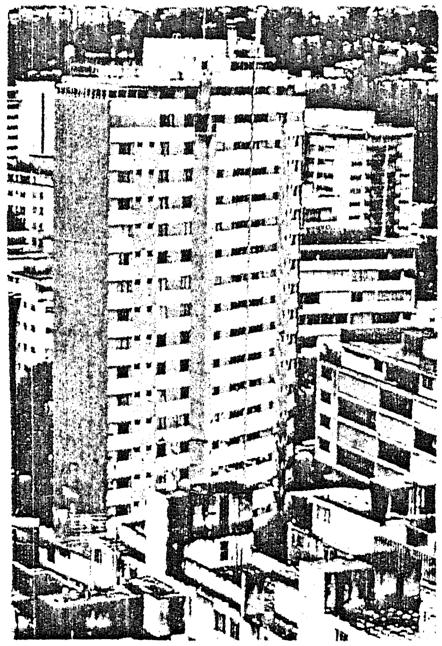


Fig. 5 - Caromay Building

Fig. 7 - Column Failure, Caromay Building

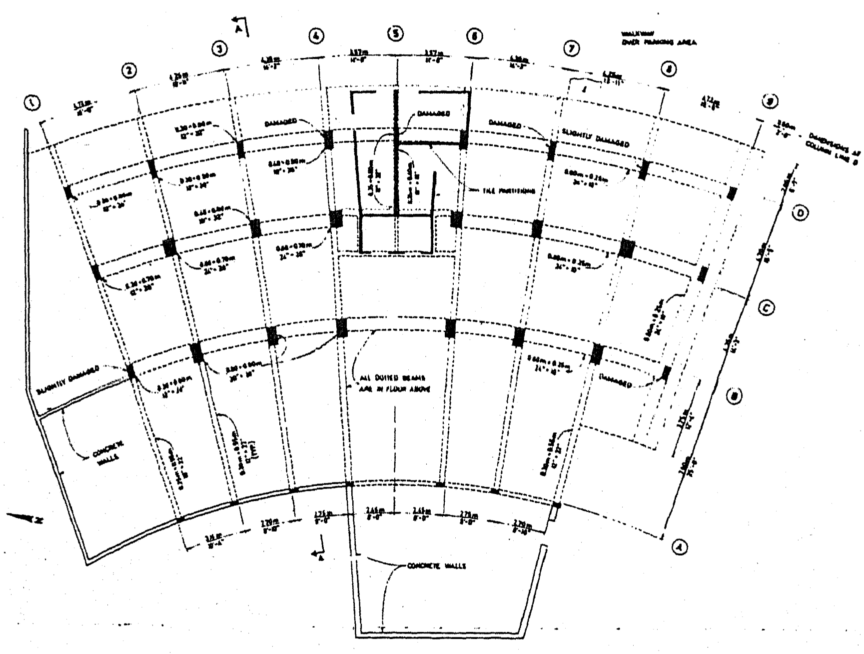


Fig. 6.- Basement Floor Plan, Caromay Building

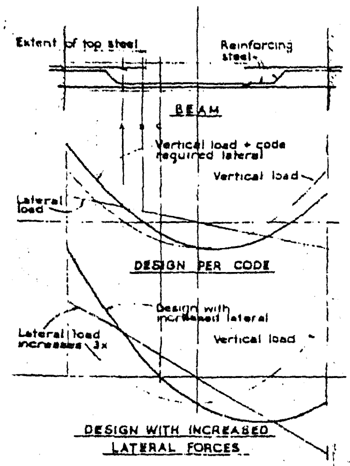


Fig. 8 - Moment Diagram

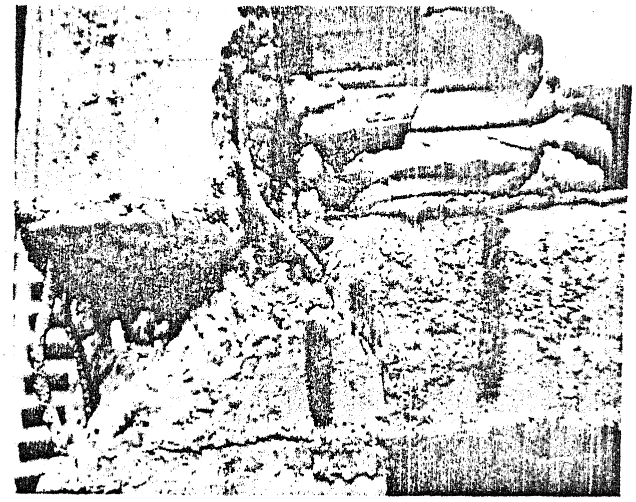


Fig. 11 - Connection Failure, Covent Gardens

Fig. 9 - Column Failure, Marco Aurelio Building

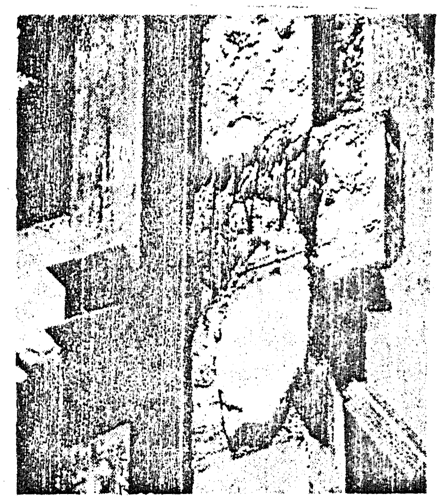


Fig. 12 - Connection Failure, Mene Grande Building

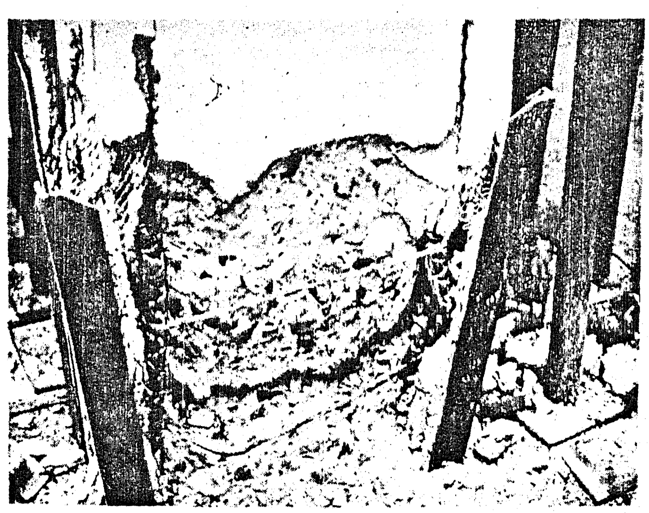


Fig. 10 - Column Failure, San Bosco Building

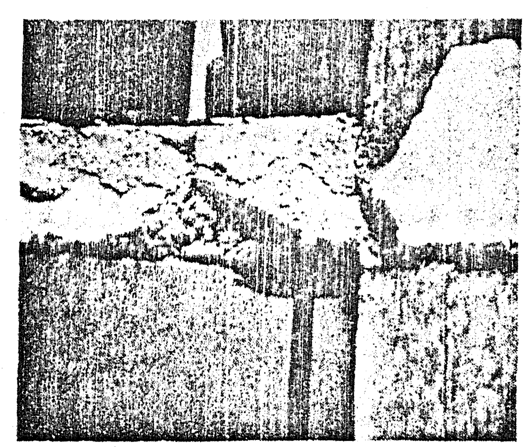


Fig. 13 - Shear Cracks in Beam, San Bosco Building

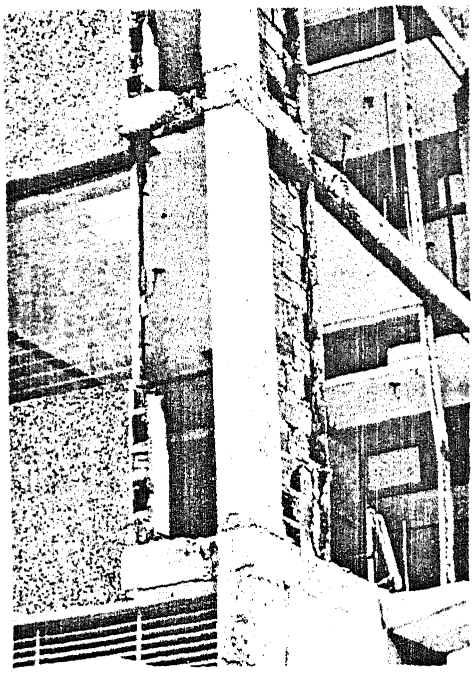


Fig. 14 - Shear Cracks in Beams, Coral Building

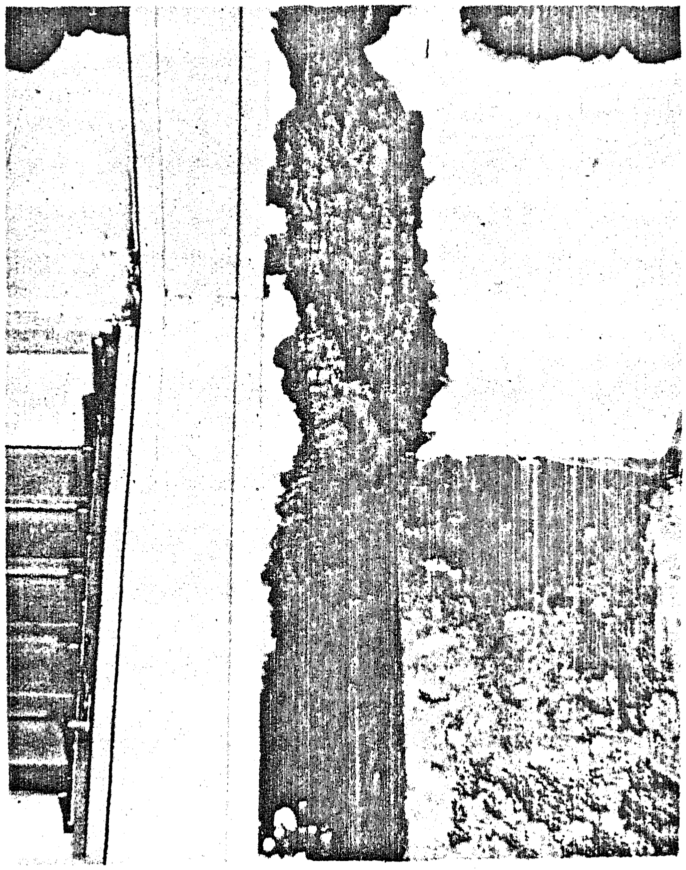
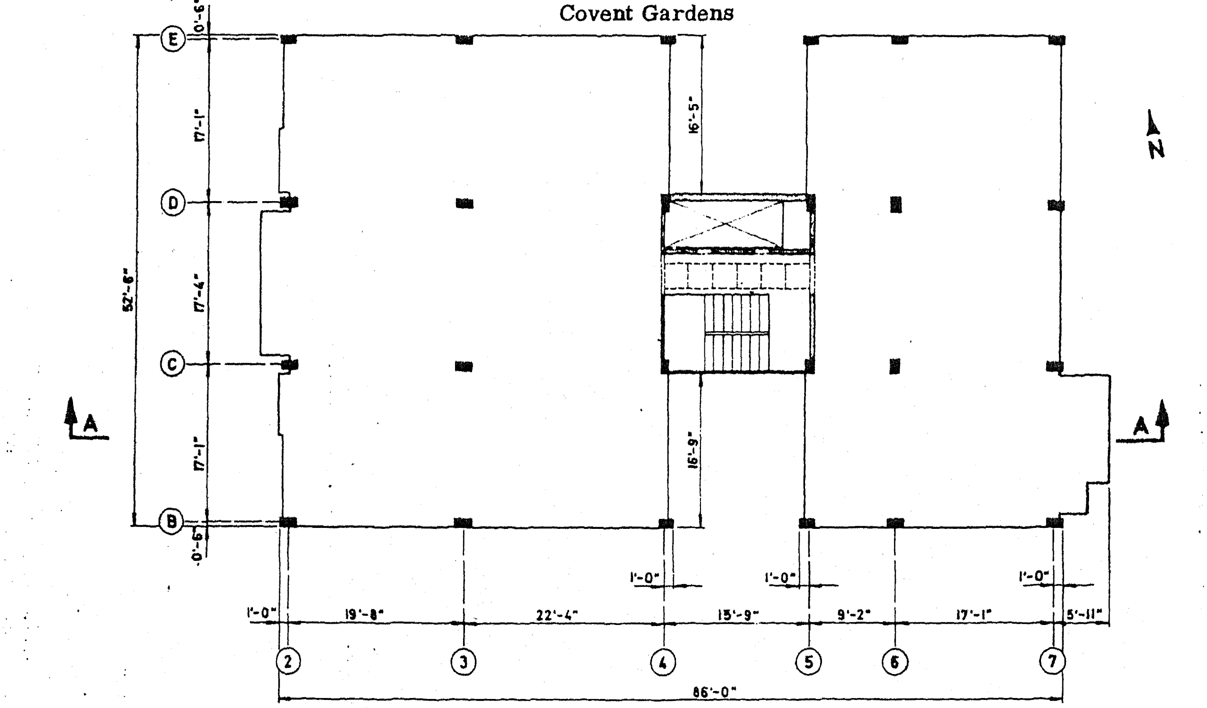


Fig. 15 - Cracks in Beam, Covent Gardens



TYPICAL FLOOR PLAN - SAN JOSE BUILDING

Fig. 17 - Palace Corvin Building

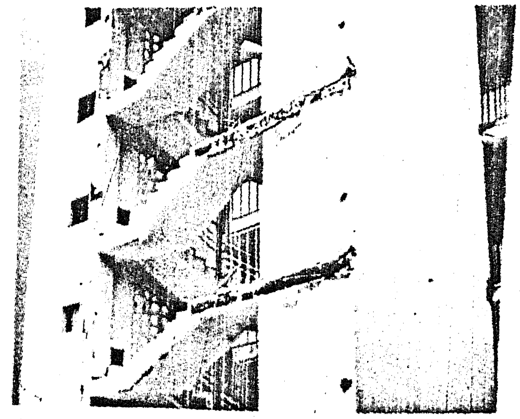


Fig. 18 - Paper Separators in Joint, Palace Corvin Building

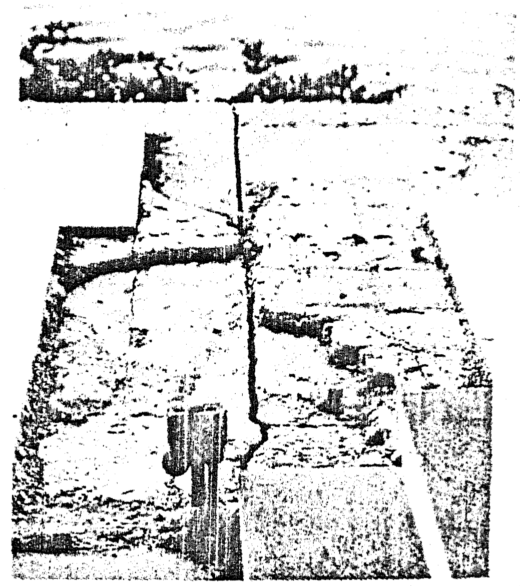


Fig. 20 - Crack in Floor, Bahia Del Mar

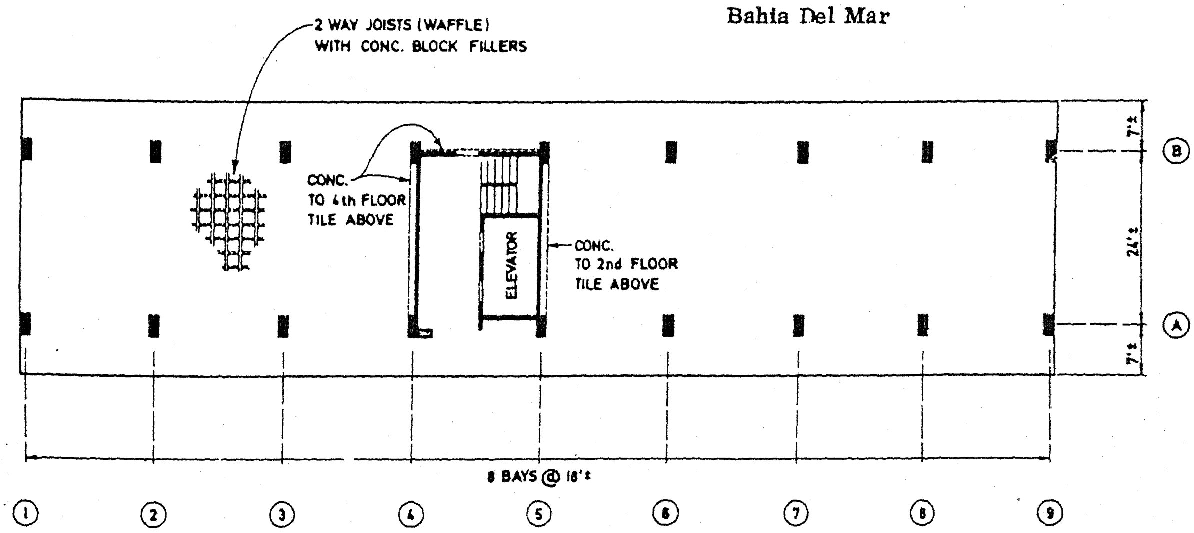


Fig. 19 - Typical Floor Plan, Bahia Del Mar

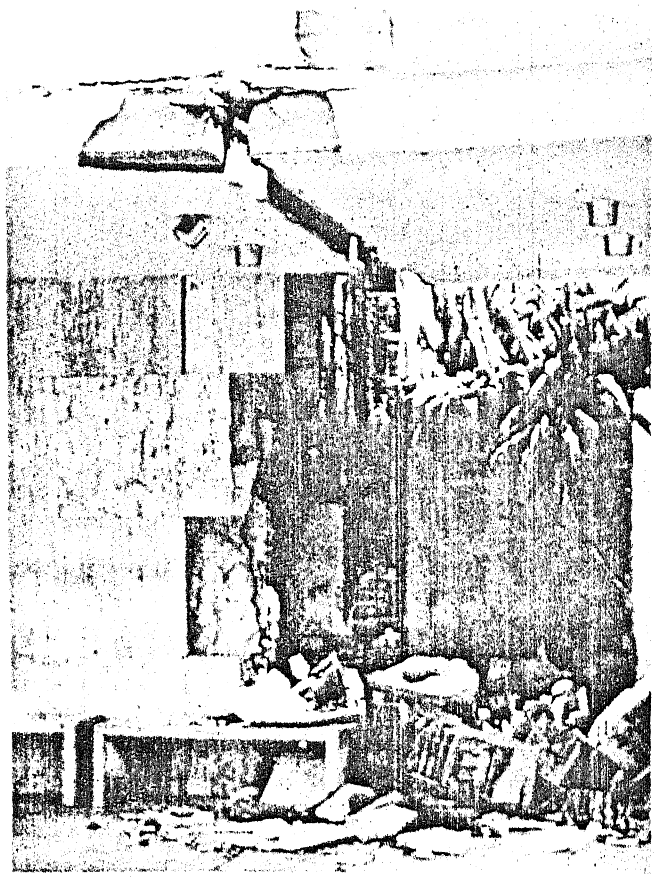


Fig. 21 - Front Canopy, Macuto Sheraton

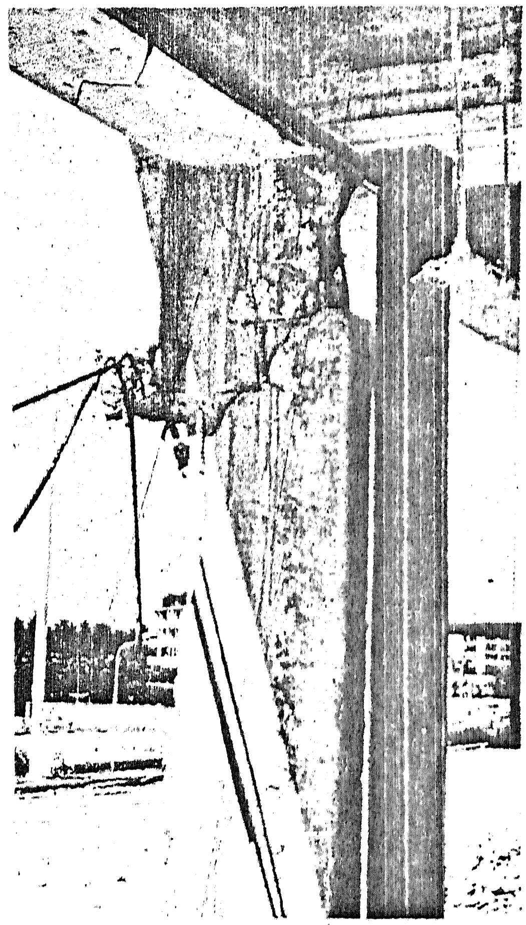


Fig. 22 - Pounding Damage at Column, Macuto Sheraton


Fig. 23 - Collapse of Mijagual on Nobel Building