Performance of Buildings With Shear Walls in Earthquakes of the Last Thirty Years

The author describes observations of shear wall performance in severe earthquakes in which modern reinforced concrete buildings stood the test of violent shaking, starting with the Chilean earthquake of May 1960 through most of the subsequent strong earthquakes, up until the Armenian earthquake of December 1988. Despite the excellent behavior of shear wall-type concrete structures as compared to concrete frame-type structures, building codes continued, up until the last decade, to give preference to concrete ductile frame structures (which are subject to higher distortions) while placing a substantial penalty on the use of shear walls. This code approach was due to the lack of experimental and analytical background information on shear wall behavior. While a large body of information on shear walls accumulated during the 1980s, still more experimental and analytical studies are needed to create a solid basis for a rational seismic design approach. The availability of such information should encourage a wider use of shear walls for earthquake resistance.

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The evolution of the modern approach to earthquake engineering of buildings started in the 1950s, at a time of intense construction activity following the conclusion of World War II. Early attempts to provide earthquake resistance in buildings were based on rather crude assumptions about structural behavior and were handicapped by a lack of proper analytical tools and earthquake records. Observations of the behavior of structures subjected to actual earthquakes, analytical studies, laboratory testing of structural elements and subassemblies, and accumulation of earthquake records over the last four decades have all contributed to rationalizing the subject of earthquake resistant structural design.

Initially, the ductile moment resistant frame evolved in the 1950s out of

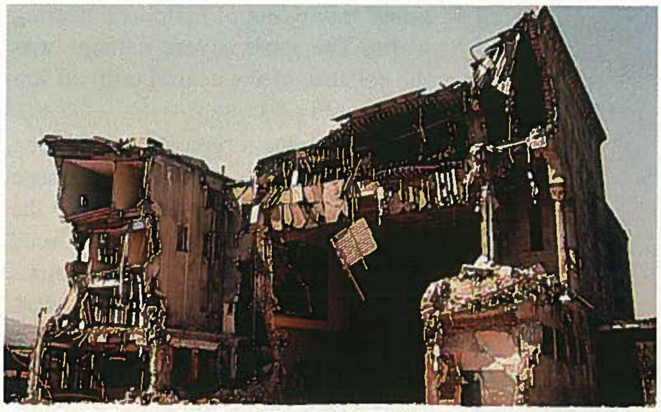

Fig. 1. Collapsed railroad station, Skopje, Macedonia.

Fig. 2. Building on Central Street, Skopje, Macedonia.

Fig. 3. Chunk of shear wall fallen out, Skopje, Macedonia.

Fig. 4. Party Headquarters, Skopje, Macedonia.

the moment resistant frame that, at the time, was the only system for multistory buildings constructed of both steel and concrete. By adding ductility to the available system, a convenient solution was created for the problem of earthquake resistance. This concept of the ductile moment resistant frame remained unchanged until the late 1970s.

In the meantime, more efficient structural systems for multistory structures (both steel and concrete) were developed for wind resistance, incorporating shear walls or trusses for concrete or steel structures, respectively. Pure frames for high rise buildings have almost disappeared because they are technically less efficient and not economically viable.

During the 1960s, 1970s, and 1980s, a large amount of significant analytical and experimental research, carried out throughout the world, accumulated a wealth of sophisticated information on the earthquake response of structural systems, including those containing shear walls. Also, starting from the mid 1950s, a substantial body of information was assembled on the performance of buildings in actual earthquakes.

Most of the analytical research in the 1950s and 1960s on the response of structures to earthquakes emphasized the importance of a ductile moment resistant frame to reduce the seismic forces. Assuming higher seismic forces in more rigid structures and assuming brittle response of shear walls to in-plane lateral forces, it was concluded that severe damage can be expected in shear wall buildings.

Based on this erroneous thinking, shear walls were considered undesirable for earthquake resistance and buildings were built primarily with moment resistant frames. While, in some countries, a degree of ductility was built into those frames as required by codes, in the majority of countries, particularly in those less economically advanced, the frames were brittle and incapable of withstanding severe earthquake shaking without severe damage. Consequently, many people in seismic regions of the world live in unsafe buildings, as has been documented in many of the earthquake reports of the last four decades.

This paper highlights some of the observations made by the author on the behavior of buildings containing shear walls in the earthquakes of the last 30 years.

The author was fortunate to have had the opportunity to visit most of the earthquake sites in which modern concrete structures were involved, starting in 1963, and to report on them in the professional literature.[1-15] For this paper, the author selected only those earthquakes that have important impli-

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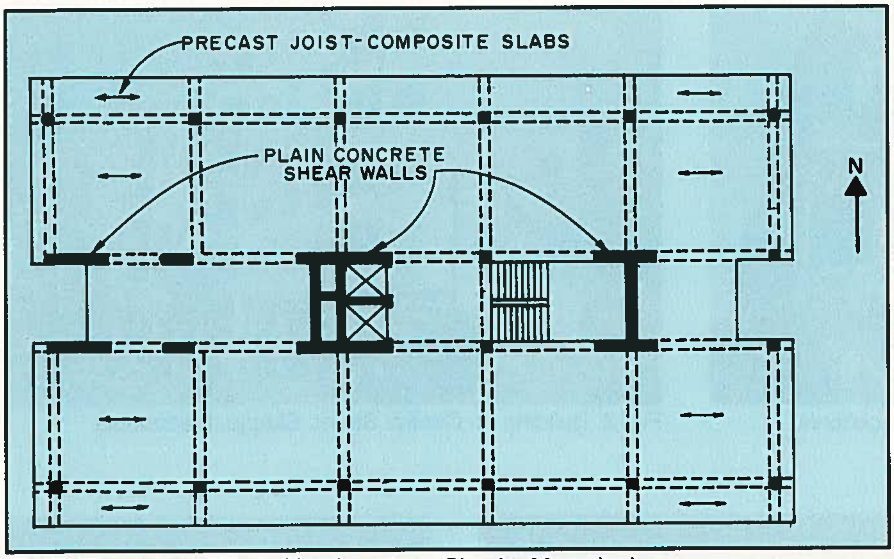

Fig. 5. Floor plan of Party Headquarters, Skopje, Macedonia.

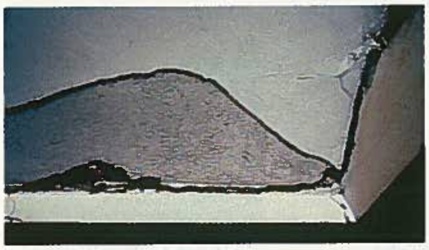

Fig. 6. Monolithic stairs.

tions on the design and construction of concrete structures and particularly those containing shear walls.

CHILE (MAY 1960)

Among the first reported observations concerning shear walls were those from the Chilean earthquake of 1960, as contained in Advanced Engineering Bulletin No. 6, issued by the Portland Cement Association (PCA). The report states:

"...the Chilean experience confirms the efficiency of concrete shear walls in controlling structural and nonstructural damage in severe earthquakes. There were instances of cracking of shear walls, but this did not affect the overall performance of the buildings. In all cases observed, the reinforcement held the walls together in good alignment, even though the amounts of steel exposed after spalling were, as a rule, less than specified by code. In essence, the walls continued to function after damage had occurred...."

SKOPJE, MACEDONIA (JULY 1963)

The first opportunity the author had to visit an earthquake site was the city of Skopje, Macedonia, in the summer of 1963. Skopje has historically experienced catastrophic earthquakes about every 500 years.

The earthquake had a magnitude of 6.2 on the Richter scale, its shallow epicenter located directly under the city. Skopje is built on several hundred feet of alluvial deposits, which were saturated prior to the earthquake. It is historically known that high earthquake damages are observed on drenched alluvial deposits, which amplify seismic shaking of the ground.

The new types of construction were mostly concrete skeleton buildings, infilled with clay masonry. On the central city streets, in a typical fashion of European cities, partitions on the ground floor were omitted to create open spaces for stores or offices; the upper four or five stories of those buildings were for residential occupancies.

More than 1100 people lost their lives, mostly in the native types of construction. The damage to more recent modern construction ranged from occasional, mild cosmetic cracking of plaster at the junction between masonry infill and the skeleton, to more severe cracking of masonry or, sometimes, falling out of peripheral bearing walls. The most severe damage was the collapse of the central railroad station (see Fig. 1).

Fig. 2 shows a typical city building in Skopje. The lateral bracing in those buildings was usually provided by the frame supplemented by unreinforced concrete cores into which flue pipes and drainage pipes were embedded. The brittleness of these unreinforced cores caused severe cracking and, sometimes, entire chunks of concrete were falling out (see Fig. 3). However, they succeeded in protecting the structure from collapse and also limited the amount of interstory distortions.

The 14-story Party Headquarters Building (see Fig. 4) had a structural system for lateral resistance consisting of a shear wall frame interaction. The three unreinforced concrete cores interacted with the frame of two-way slabs resting on beams (see Fig. 5). The building swayed considerably during the earthquake; desks were thrown from one end of rooms to the other. However, the building itself withstood the earthquake without damage, not even broken windows. The elevators had jammed but in several days they were repaired and put back into operation.

Typical Distress Patterns

Monolithic stairs — Fig. 6 shows the inclined platform carrying the stairs from one story to the next. Those inclined platforms, together with the walls around the staircase, form a vertical truss. Because the truss is much more rigid than the remaining frame of the building, it attracts the bulk of the lateral forces. If the lateral seismic forces exceed the capacity of the truss, distress or failure occurs, usually at the junction of the inclined surfaces to the horizontal surfaces. Only after this failure occurs can the frame take over the resistance of the lateral forces, if any of these forces are left at that point in time.

This characteristic failure of monolithic stairs is seen in many earthquakes. In some cases, observation of the stairwell showed evidence that an outwardly undamaged building was subjected to shaking. Because of its

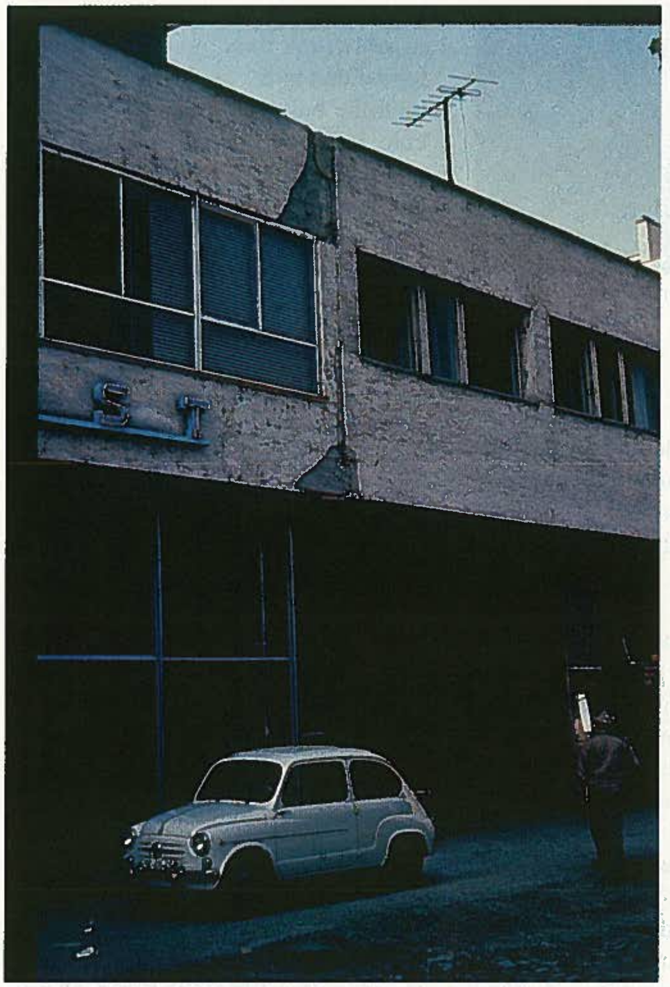
Fig. 7. Hammering of adjacent structures, Skopje, Macedonia.


Fig. 8. Collapsed portion of Palace Corbin, Caracas, Venezuela.

high rigidity, the vertical stairwell truss cracked at high lateral forces, thus protecting the frame by limiting the interstory distortions.

Hammering of adjacent buildings — Fig. 7 shows spalling and damage at the joint between two adjacent buildings that are not separated by a seismic joint. Each of the adjoining buildings sways during an earthquake according to its dynamic properties and needs sufficient space to deform without hammering and damaging its neighbor. Most modern seismic codes specify the width of seismic joints as a function of building height and seismic intensity.

CARACAS, VENEZUELA (JULY 1967)

Caracas is located in the northern part of South America and, similar to the west coast of the United States, its seismicity is affected by the Pacific Rim, which is the location of the world's most intense and frequent earthquake activity.

The moderate earthquake of 1967 measured 7 on the Richter scale. The epicenter was located in the Caribbean Sea off the coast of Venezuela. Caracas is located in a valley and is founded on alluvial deposits up to 1000 ft (300 m) deep. There was a large amount of modern construction after World War II. Caracas has experienced mild earthquakes many times in its recent history.

There were two areas of severe distress in this earthquake: one in the city of Caracas and the other on the Caribbean shore of Carabolida. In the city of Caracas, the earthquake caused the collapse of four 10-story buildings, killing several hundred people.

Fig. 8 shows the collapsed part of the Palace Corbin, which consisted of two 10-story wings interconnected by a core housing the stairwell and elevators. The core was not connected to the two buildings. The northern 10-story wing remained standing while the southern 10-story wing collapsed.

Regarding the failure of the four 10-story buildings, it seems plausible, in hindsight, that all four 10-story structures may have failed in resonance with the amplified motion of the soil in these particular locations in Caracas. At that time, the failure could not be conclusively correlated to the soil shaking because the quantitative record of the earthquake intensity in Caracas was not known.

Typical Construction

Fig. 9 shows the structural configuration of a typical multistory building design used at the time in Caracas and most of Central and South America. This is the skeleton of an 18-story apartment building under construction. There are several notable characteris-

Fig. 9. Typical building skeleton, Caracas, Venezuela.

Fig. 10. Damaged corner joint, Caracas, Venezuela.

tics. First, the columns are very slender for an 18-story building, considering the fact that the concrete strength is only about 2500 psi (17.23 MPa).

The one-way joist slabs are 8 in. (203 mm) thick. The joists are formed by embedment of hollow clay tiles. On the column lines in both directions are beams within the 8 in. (203 mm) slab thickness. This combination of very shallow beams and relatively slender columns created very flexible skeletons. There were no shear walls incorporated into the skeletons to increase or supplement the resistance of the frame to lateral forces.

Fig. 10 shows the details of a corner column and of the joint of a 15-story building. The size of the column appears only moderate. The vertical reinforcement of the column and the column ties are inadequate by American standards. The importance of the joint between the horizontal beams and the columns cannot be overemphasized because the joints are actually the heart of the resistance to lateral forces of a frame. Unless there is a proper transfer of forces within the joint between the beams and the columns, there is no resistance to lateral forces.

These skeletons were filled in with hollow clay tiles as partitions and as exterior walls. When an earthquake strikes a skeleton of such relatively high flexibility, the skeleton distorts as a result of the ground shaking and, consequently, the partitions are often cracked or damaged due to their brittleness and limited strength. Occasionally, the failure of partitions induces additional explosive forces into the skeleton that further aggravate the response to earthquake forces.

Figs. 11 and 12 show infill partition damages due to frame flexibility, as seen in many buildings in Caracas.

The other area of damage was at the Carabolida on the Caribbean shore, where two dramatic failures occurred. Fig. 13 shows an eight-story building in which the three upper stories collapsed during the earthquake. Seen behind the building is the 10-story Macuto Sheraton, a well designed, well constructed structure. Six typical floors of the Macuto Sheraton had concrete walls separating rooms. To create an open space for the restaurants, lobbies and other public spaces at the mezzanine floor, those wall stacks were omitted and each replaced with two free-standing columns.

The resulting drastic change in the rigidity between the shear wall-type upper six stories and the flexible, columns-only, mezzanine floor caused a large interstory distortion within the mezzanine floor. These short columns failed in shear (see Fig. 14) despite the fact that they were very well reinforced and of good quality concrete. The conclusion drawn is that most short columns or short stout beams (reinforced for high moment capacity) are more likely to fail in shear, regardless how well they are reinforced.

Plaza One

There was only one building in Caracas of a different structural configuration. The 16-story Plaza One Building (see Figs. 15 and 16) was a shear wall building, using bearing walls instead of columns in each of the two orthogonal directions. The American owners initially had planned a 20-story building; however, the Uniform Building Code (which was the code being followed at that time) did not permit more than 16 stories in concrete and this is why the decision for 16 stories was made.

Fig. 11. Damaged partitions, Caracas, Venezuela.

Fig. 12. Damaged partitions, Caracas, Venezuela.

Fig. 13. Buildings in Carabolida, Caracas, Venezuela.

Fig. 14. Column failed in shear at the Macuto-Sheraton, Caracas, Venezuela.

Fig. 15. Plaza One Building, Caracas, Venezuela. Remains of a 10-story collapsed building are seen in front.

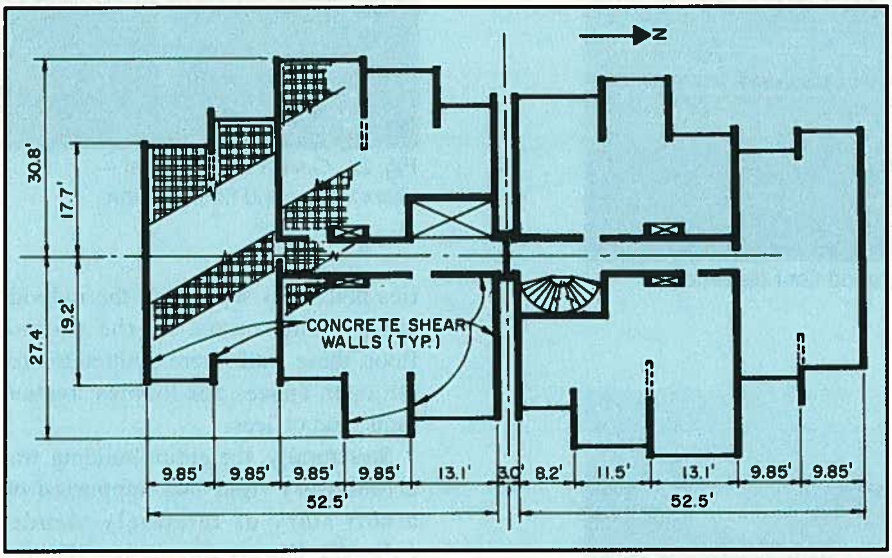

Fig. 16. Floor plan of Plaza One Building, Caracas, Venezuela.

The building was located in an area of heavy damage. A 10-story building in front of Plaza One collapsed and most of its neighboring structures suffered heavy non-structural damage. Plaza One went through the earthquake without any damage. The presence of the walls provided the carrying capacity for all the gravity loads and, at the same time, provided stiffness for lateral resistance sufficient to limit any interstory distortions.

Lessons Learned from the Caracas Earthquake

- Buildings containing shear walls performed considerably better than buildings with flexible frames.
- Incorporating low strength brittle partitions into flexible frames leads to costly damage to partitions.
- Short columns and short beams (low span-to-depth ratio) fail mostly in shear, no matter how well they are reinforced.

SAN FERNANDO, CALIFORNIA (FEBRUARY 1971)

San Fernando, a northern suburb of Los Angeles, lies on a branch of the San Andreas fault that was inactive for the last 100 years. The earthquake in 1971 had a magnitude of 6.8 on the Richter scale. Historically, this area has had a number of earthquakes ranging between 4 and 5 on the Richter scale.

The earthquake caused extensive damage in the San Fernando valley. Many buildings and bridges sustained damage or collapsed. There were a number of failed bridges in the San Fernando area. However, they represented a very small percentage of the overall number of bridges within the area of severe damage.

Several hospitals suffered damage, some of which are discussed below. The Veterans Administration Hospital (constructed in 1922) was a four-story building of unreinforced masonry (see Fig. 17). A collapsed wing of the hospital killed 46 patients. Other buildings at the site of the Veterans Administration Hospital fared quite well.

Olive View Hospital

The Olive View Hospital (see Fig. 18) was opened only 3 months before the earthquake. In plan, the five-story building was a hollow square. The inner court in the center was surrounded on all four sides with patient rooms in the upper four stories. At the four ends were free-standing stairwells not tied to the main structure; they were separated by earthquake joints. These stairwells stood on two-story high stilts. The four typical sto-

Fig. 17. Veterans Administration Hospital, San Fernando, California.

Fig. 18. Olive View Hospital, San Fernando, California.

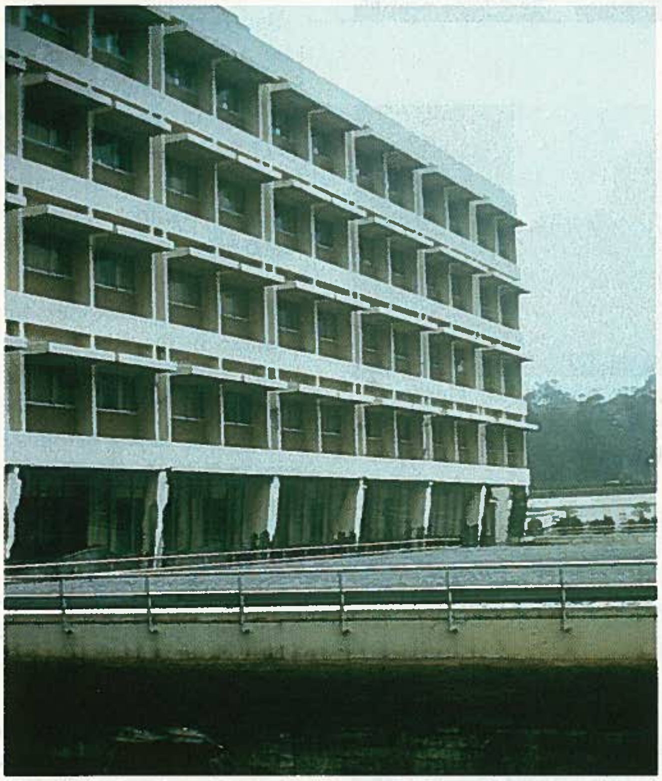

Fig. 19. Olive View Hospital — ground floor distortion.

Fig. 20. Olive View Hospital — overturned stair tower.

Fig. 21. Olive View Hospital — distorted ground floor column.

ries had walls separating the individual patient rooms. At the ground floor, these walls were omitted to create open spaces for lobbies, restaurants, and offices.

Structurally, the entire building was a four-story rigid box supported on a soft story of relatively slender columns. During the earthquake, the entire four-story rigid box moved extensively with a large distortion of the ground floor (see Fig. 19). As a result of the large movements, all four stair towers were pushed over and ended up flat on their backs (see Fig. 20). Within the ground floor, there was a very large permanent distortion of

Fig. 22. Indian Hill Medical Center, San Fernando, California.

Fig. 23. Holy Cross Hospital, San Fernando, California.

Fig. 24. Holy Cross Hospital — failed column.

Fig. 26. Banco Central.

Fig. 25. Banco Central and Banco de America, Managua, Nicaragua.

21 in. (533 mm) from the tops to the bottoms of the columns of the ground floor (see Fig. 21).

The square columns of the ground floor had circular spiral reinforcement. They had the capacity to distort 21 in. (533 mm) and still retain their load capacity to carry the four stories above it and, thus, prevent collapse of the building and save the lives of the patients. As expected, during such large distortions the shells spalled off. Only the L-shaped corner columns failed because they could not be spirally reinforced due to their shape. They could not withstand large distortions.

Indian Hill Medical Center and the Holy Cross Hospital

The six-story Indian Hill Medical Center (see Fig. 22) was located in an area of severe damage. Its structural system contained shear walls around the periphery and also a reinforced concrete core in the center of the structure. The building withstood the earthquake with only minor cracks in the shear walls and was back in operation within a week of the earthquake.

The six-story neighboring Holy Cross Hospital (see Fig. 23), which had a concrete frame structural system, suffered severe structural damage (see Fig. 24) and had to be torn down.

Lessons Learned from the San Fernando Earthquake

- Spirally reinforced columns designed by recent codes have a large amount of ductility.
- A drastic change in stiffness from story to story is an invitation for

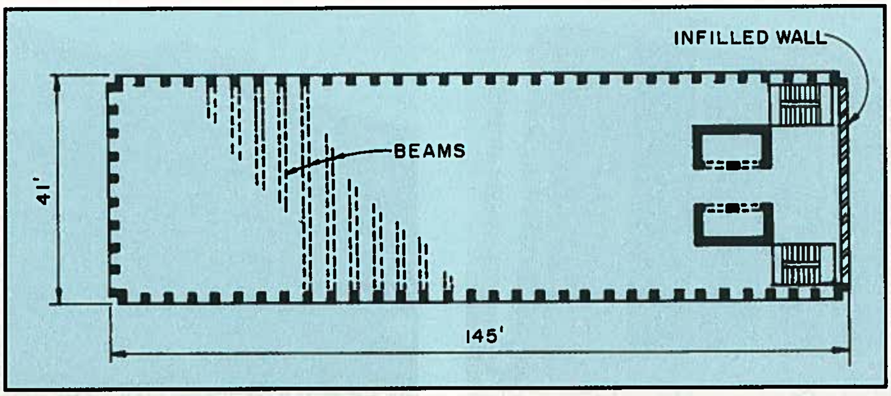

Fig. 27. Floor plan of Banco Central.


Fig. 28. Inside damage, Banco Central.


Fig. 29. Banco de America.

trouble, unless positive measures are taken to create a restoring force within the soft story.

- Shear wall-type structures show a superior response to earthquakes compared with frame-type structures by limiting the interstory distortions.

MANAGUA, NICARAGUA (DECEMBER 1972)

Managua, Nicaragua, is in Central America on the Pacific coast. It lies on the Circum Pacific Earthquake Belt that affects locations from New Zealand to Japan, Alaska, the west coast of the United States, and Central and South America.

The 1972 earthquake was of moderate magnitude, slightly more than 6 on the Richter scale. The epicenter of this shallow earthquake was located under the city, resulting in a disproportionately large amount of damage. About 50 city blocks were totally obliterated, causing the deaths of about 10,000 people, mostly inhabitants of traditional types of construction.

Banco Central and Banco de America

Of specific interest are two modern bank buildings constructed in the early 1960s (see Fig. 25). The buildings were located across from each other. Both banks were very carefully constructed with the best quality construction available, using current state-of-the-art techniques. Both structures were subjected to the same intense earthquake. They were located within the area of the severe earthquake damage and both behaved very differently because of their different structural systems.

The 14-story Banco Central (see Fig. 26) was a one-bay frame. The lateral resistance was provided by two small concrete elevator cores and a masonry infill wall, all located at the east end of the building (see Fig. 27). During the earthquake, this cluster of resistances acted as a pivot around which the entire structure rotated. In the assessment of the damage, it was found there was only limited structural damage, primarily tearing between the

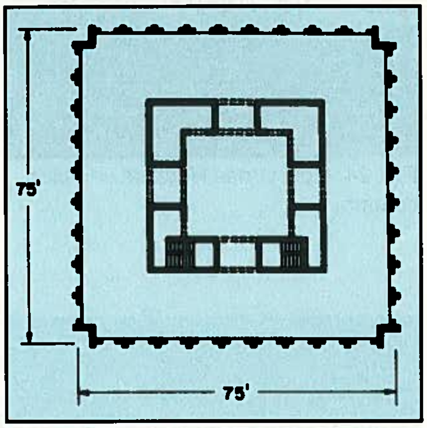

Fig. 30. Floor plan of Banco de America.


Fig. 31. Failed coupling beam in Banco de America.

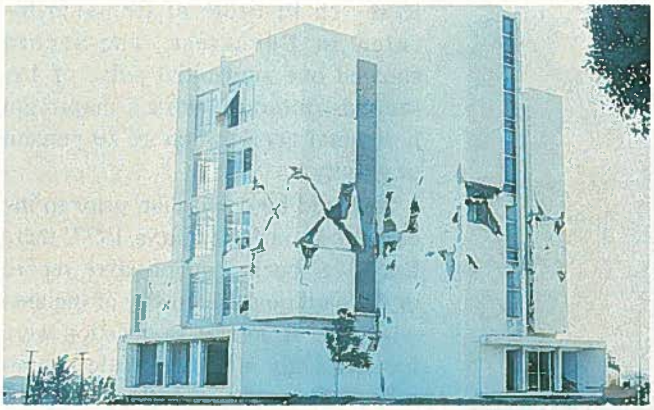

Fig. 32. Five-story Insurance Building, Managua, Nicaragua.

Fig. 33. Five-story Light and Power Building, Managua, Nicaragua.

Fig. 34. Modern structures in Bucharest, Romania.

Fig. 35. End of building failure, Bucharest, Romania.

cores and the floors. However, the non-structural damage was very severe as a consequence of the intense shaking of the building. The inside of the building was severely damaged (see Fig. 28) on all floors; all of the window sills were cracked. The building was subsequently torn down.

The 18-story Banco de America (see Fig. 29) responded to the earthquake in a completely different way. In the plan (see Fig. 30), a closely spaced peripheral column grid interacted with four centrally located elevator and stair cores to resist lateral forces. The cores were tied together with coupling beams on every floor level. The result was a rigid shear wall frame interactive system. There was neither non-structural nor structural damage, except for one coupling beam that failed in shear on the sixth floor (see Fig. 31). This was caused by over reinforcing in flexure of the short coupling beam with a large duct opening.

Five-Story Insurance Building and Five-Story Light and Power Building

These two buildings were not located close to each other; however, each was in an area of severe damage. The five-story Insurance Building (see Fig. 32) had a structural frame with filled-in hollow clay tile masonry around the periphery and the same clay partitions. There was no structural damage because the flexible structure had the capacity to withstand large distortions; however, there was severe non-structural damage to the exterior and interior partitions.

The five-story Light and Power Building (Fig. 33) had a central reinforced concrete core and an exterior frame of closely spaced delicate columns. There were several cracks in some columns. A corner of the core shear wall was damaged; however, there was no other damage and this structure continued its operations the day after the earthquake occurred.

Lessons Learned from the Managua Earthquake

The superior behavior of the shear wall structure exhibited in the two banks in addition to the cumulative experience in prior earthquakes initiated a move in the United States towards a change in the codes. Subsequently, it took about 15 years to incorporate shear wall buildings and shear wall frame interactive systems into the American seismic codes to recognize shear walls as a superior concrete framing system for earthquake resistance.

BUCHAREST, ROMANIA (MARCH 1977)

Bucharest, Romania, in eastern Europe, lies on the Alpide Earthquake Belt. The area has experienced earthquakes in the past, most recently in 1940. During that earthquake, many buildings were shaken up but none

Fig. 36. Failed Computer Building, Bucharest, Romania.

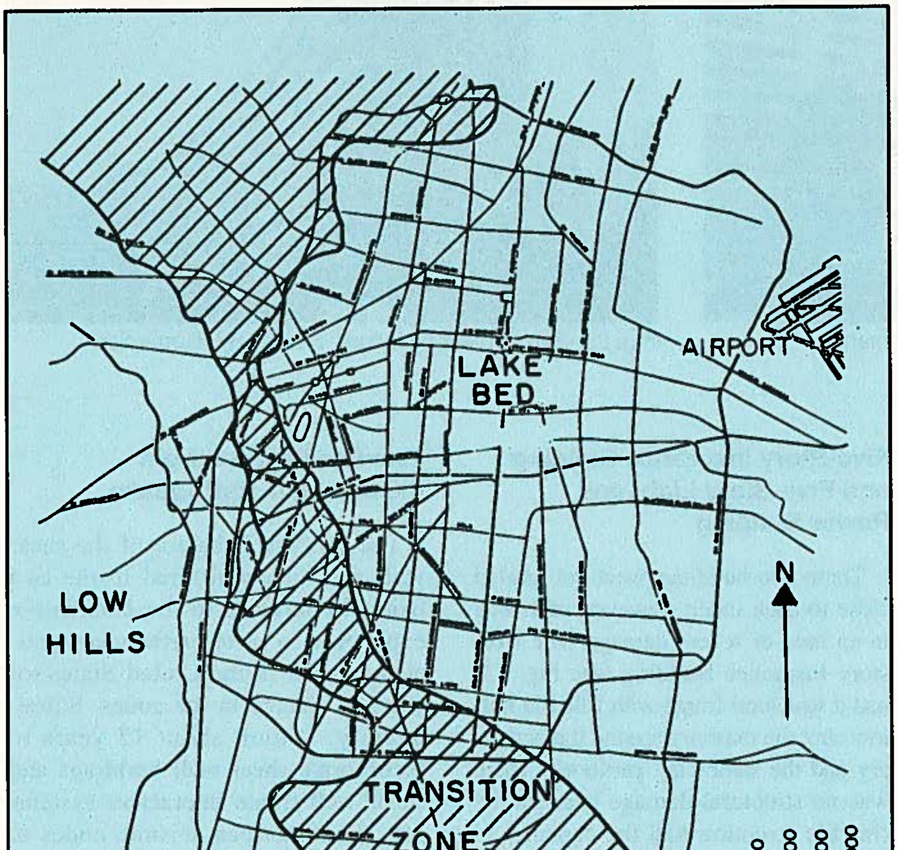

Fig. 37. Geological zones in Mexico City, Mexico.

collapsed; however, their period of vibration may have lengthened substantially due to cracking.

The earthquake measured 7.2 on the Richter scale. The epicenter was located in the Vrancea mountains 130 miles (210 km) north of Bucharest. It is significant to note that, for the first time, there was an accelerogram taken in the city by the Earthquake Research Institute of Romania located in Bucharest. The record showed one sinusoidal pulse of $1^1/_2$ seconds duration with a maximum horizontal acceleration of 20 percent of gravity.

It should be noted that, prior to the earthquake in Romania in 1977, there has never been a quantitative record of the earthquake intensity of the area where the structures in question were located. Only a Richter scale magnitude of the earthquake, describing the amount of energy released at the epicenter as registered at several observatories around the world, was available. However, from an engineering viewpoint, it is useful to know what the intensity of shaking is at the location of a building. Therefore, in all earthquakes prior to 1976, only a qualitative discussion about behavior of structures in an earthquake was possible and not an engineering correlation between the input force and the response of the structure.

During the March 1977 earthquake, 35 buildings collapsed in Bucharest, killing 1800 people. Out of the 35 collapses, 32 were 10- to 12-story buildings constructed prior to World War II. These were probably shaken up during the 1940 earthquake. As in most European cities, a city block consists of a number of buildings next to each other without separation joints, but not tied to each other. The 32 collapses were mostly of buildings located at each end of blocks. As a block moved back and forth in response to the earthquake, the end buildings were thrown off, similar to the stairwells in the Olive View Hospital in the San Fernando earthquake.

One can now speculate that most of these 32 buildings were in a period range of $1^1/_2$ seconds and may have, therefore, responded in resonance to the earthquake shocks having a period of $1^1/_2$ seconds.

There were 300,000 new dwellings constructed in Bucharest after World War II, and out of these, 70,000 were built using precast concrete. It was the first time that a large number of precast concrete buildings were tested in an earthquake. The precast concrete buildings built in Romania after

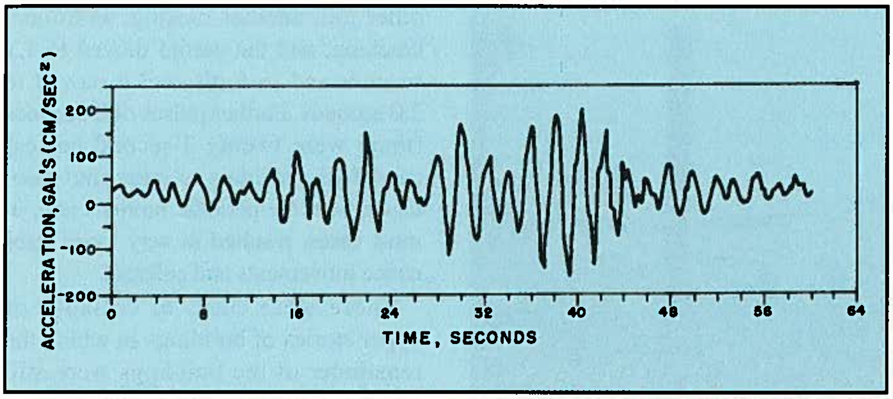

Fig. 38. Accelerogram at the Ministry of Transportation, September 19, 1985, Mexico City earthquake.

World War II were initially four stories in height. Subsequently 7-, 9-, 13-, and 15-story buildings were built (see Fig. 34).

Various structural systems were used for the precast concrete buildings. Some were precast concrete skeletons, but most of the others were shear wall structures. Of the shear wall structures, some had precast concrete slabs with cast-in-place walls while others had cast-in-place slabs and precast concrete walls. All of the precast concrete construction was properly tied together by welding of protruding reinforcement and use of cast-in-place jointing to achieve good structural continuity.

In general, modern construction of all types of buildings behaved very well. There were only three failures of modern type construction. One was an end of a block of a cast-in-place multistory building (see Fig. 35), the foundation of which was being underpinned at the time of the earthquake.

A dramatic failure was the three-story computer facility building of the Romanian Railroads (see Fig. 36). Judging from the debris, the collapse occurred by shear failures of the columns in the lower soft story due to the large interstory distortion.

Lessons Learned from the Romanian Earthquake

- 32 older 10- to 12-story buildings failed, in all probability, in resonance with the frequency of the shaking.
- New reinforced concrete buildings generally performed very well.
- Precast concrete and large panel (shear wall) buildings of various configurations showed minimum distress. It was the first time that a large number of precast concrete buildings were tested in an earthquake. However, it should be kept in mind that these buildings had periods of 0.6 to 0.7 seconds (estimated), while the earthquake motion had a period of $1^1/_2$ seconds.

MEXICO CITY (SEPTEMBER 1985)

The earthquake that shook Mexico City on September 19 and 20, 1985, was the most damaging earthquake experienced in the history of observation of modern structures. The two consecutive events had magnitudes of 8.1 and 7.5 on the Richter scale.

The epicenter of the earthquake was in the Pacific Ocean, about 20 miles (32 km) off the Mexican coast; it was about 250 miles (402 km) from Mexico City. The intensities measured on the Modified Mercalli scale were estimated to range between V and IX, depending on the neighborhood within the city.

Mexico City has several geological regions (see Fig. 37). The downtown area lies on an infilled lake. The lake was filled-in centuries ago, mostly with volcanic and alluvial deposits. The western suburbs are located on hills (solid ground) and there is an intermediate transition zone between the hills and the lake. The earthquake, being 250 miles (402 km) away, was mildly felt on the solid ground of the hills; the registered shaking was 4 percent of gravity.

Fig. 38 shows the accelerogram recorded within the severely damaged area of the city in the building of the Ministry of Transportation. This is a very unusual accelerogram. The record shows 20 consecutive 2-second intense harmonic pulses. The record resembles a response record more than

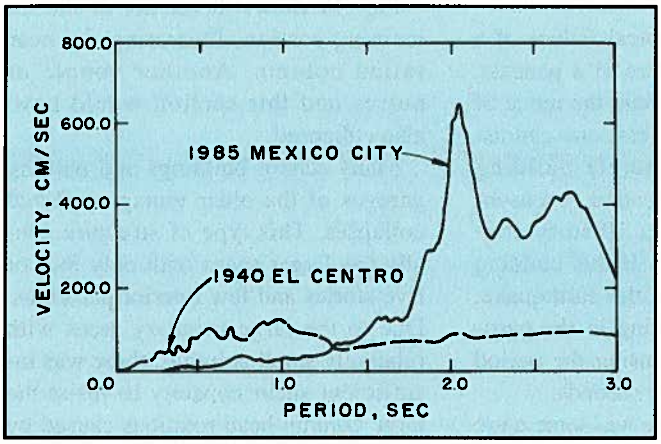

Fig. 39. Velocity response spectrum for 2 percent damping, Mexico City earthquake.

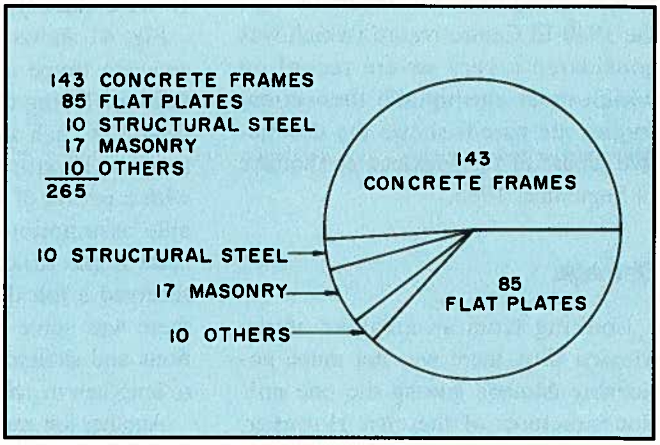

Fig. 40. Damage by structural types, Mexico City earthquake.

Fig. 41. Typical pancaked building, Mexico City, Mexico.

an accelerogram. It is actually the response of the soil layers of the infilled lake to the incoming long waves. While there were many different long waves that arrived at that location, the soil responded with its own characteristic dynamic period of vibration of 2 seconds.

Prior to this earthquake, it was considered that an intense phase of an earthquake input motion may be up to 15 seconds. A duration of 40 seconds has never been recorded before. The maximum acceleration (see Fig. 38) was up to 18 percent of gravity, showing an amplification of more than four times the acceleration registered in the hills.

Fig. 39 shows the velocity response spectrum (for 2 percent damping) derived from the input record. It shows that a building with a 2-second period would have a violent velocity response to that earthquake. A comparison with the 1940 El Centro record (which was considered a very severe record on which most earthquake theoretical studies are based) shows the destructive nature of the Mexican earthquake of September 1985.

Damage

Looking from an airplane above Mexico City, there was not much noticeable damage among the one million structures of the city. However, an inspection of particular neighborhoods revealed an amount of damage never before experienced in modern construction. The statistics, which were published by the Earthquake Engineering Institute two weeks after the earthquake (see Fig. 40), show that there were 180 buildings collapsed and 85 nearly collapsed, for a total of 265 buildings. Most of these buildings were between 6 and 15 stories.

According to official statistics, 9000 people were killed, although subsequent unofficial estimates indicated much higher casualties. Also, later information from the Engineering Societies of Mexico showed that 760 buildings were slated for demolition.

If those 265 collapsed buildings are characterized by structural type (see Fig. 40), it can be seen that 143 were concrete frames, 85 were flat plates (many of them so-called reticular plates), 10 were structural steel buildings, 17 were masonry buildings and 10 were other types.

Fig. 41 shows a typical failure of a concrete frame structure in a pancake fashion. Trying to explain the mode of failure of such structures, one can assume a 10-story concrete building with a period of 1.0 second (a reasonable assumption for a 10-story concrete frame structure). If this building received a jolt during the earthquake, there was some cracking in the partitions and skeleton, causing the period to lengthen to, say, 1.2 seconds.

Another jolt and there was some more cracking and yielding and the period further lengthened to, say, 1.5 seconds. Another jolt, another racking, additional cracking, and the period moved to 1.7 seconds and so forth until it moved to 2.0 seconds. Further pulses of 2 seconds (there were twenty 2-second pulses) caused this building to enter into resonance with the periodic motions and, in most cases, resulted in very large resonance movements and collapse.

There were cases of collapse of upper stories of buildings in which the remainder of the buildings were still standing (see Fig. 42). In this particular case, the neighboring building appeared to provide a buttressing effect, causing an amplification of the whiplash of the higher stories — the primary cause of the failure. In some cases, intermediate stories collapsed (see Fig. 43).

Fig. 44 shows a failure of a so-called reticular type slab. It is a variety of a waffle slab in which the waffles are formed by very thin concrete boxes, less than 1 in. (25.4 mm) in thickness, to create the voids. These flat plate buildings, 87 of which collapsed, failed in shear around the columns, with the slabs sliding down the columns into a heap on the ground.

A noteworthy case study was that of Nuevo Leon, a suburb of Mexico City constructed in the 1960s, in which about 750,000 people lived. There were a number of 14-story buildings, each consisting of three sections separated by expansion joints. Fig. 45 shows one remaining section standing while the other two collapsed. The primary resistance to lateral forces in these buildings was provided by rigid walls at each end; the rigidity of the walls was created by X-bracing.

Fig. 46 shows the center of the remaining section illustrating the near failed column. Another couple of pulses and this section would have also collapsed.

Many school buildings and parking garages of the older vintage suffered collapses. This type of structure usually has larger spans with only four or five stories and few interior partitions. Due to the large tributary areas with relatively small columns, there was insufficient shear capacity to resist the large column head rotations caused by interstory distortions of the highly flexible frames.

Fig. 42. Failure of upper stories, Mexico City, Mexico.

Fig. 43. Collapse of intermediate stories, Mexico City, Mexico.

Fig. 44. Failed flat plate building (reticular), Mexico City, Mexico.

In contrast, a number of modern parking structures performed excellently. The five-story garage, shown in Fig. 47, across from the collapsed hospital performed extremely well. Its structure had substantial columns, heavy beams in both directions interconnecting the columns and there were concrete walls within the ramp area.

It should be pointed out that many collapses were adjacent to buildings that withstood the earthquake very well.

Lessons Learned from the Mexico City Earthquake

- Concrete nonductile frame buildings not stiffened by shear walls fail in severe earthquakes.
- Flat plate structures without stiffening walls are not suitable for earthquake resistance.
- Knowledge of the period of vibration of the underlying soil may help in the design of structures to avoid failures in resonance.
- Failure of the 10 structural steel buildings showed that the choice of concrete or steel does not ensure seismic resistance. Rather, it is proper selection of a structural system and incorporation of suitable detailing that ensures seismic resistance.

CHILE (1985)

The 1985 Chilean earthquake received relatively little attention in the engineering profession, despite the fact that its magnitude was similar to that of the Mexican earthquake of the same year. This earthquake went almost unnoticed, probably because there were no dramatic collapses, the severity of the event notwithstanding.

The primary reason for the minimal damage was the widely used engineering practice in Chile of incorporating concrete walls into buildings to control drift. It should be noted that the detailing practice for shear walls in Chile generally does not follow the ductile detailing requirements of seismic regions in the United States, but rather follows conventional detailing as required in previous ACI Building codes.

The exceptionally good performance of Chilean buildings during the 1960 and, particularly, the 1985 earthquake bears testimony that drift control provided by shear walls can protect relatively nonductile framing elements.

ARMENIA (DECEMBER 1988)

Armenia, located in the Caucasus (see Fig. 48), lies on the Anatolian earthquake fault, which is at the junction of the Eurasian and Arabian plates. This shallow earthquake [focal depth of 9.3 miles (15 km)] had a Richter magnitude of 6.9. It was located very close to the city of Spitak, which was almost completely destroyed. Two other cities, Leninakan and Kirovakan, suffered severe damage.

The loss of life was unofficially estimated at up to 50,000 people. The casualties were especially heavy among the school population. The earthquake occurred at 11.41 a.m., only 4 minutes

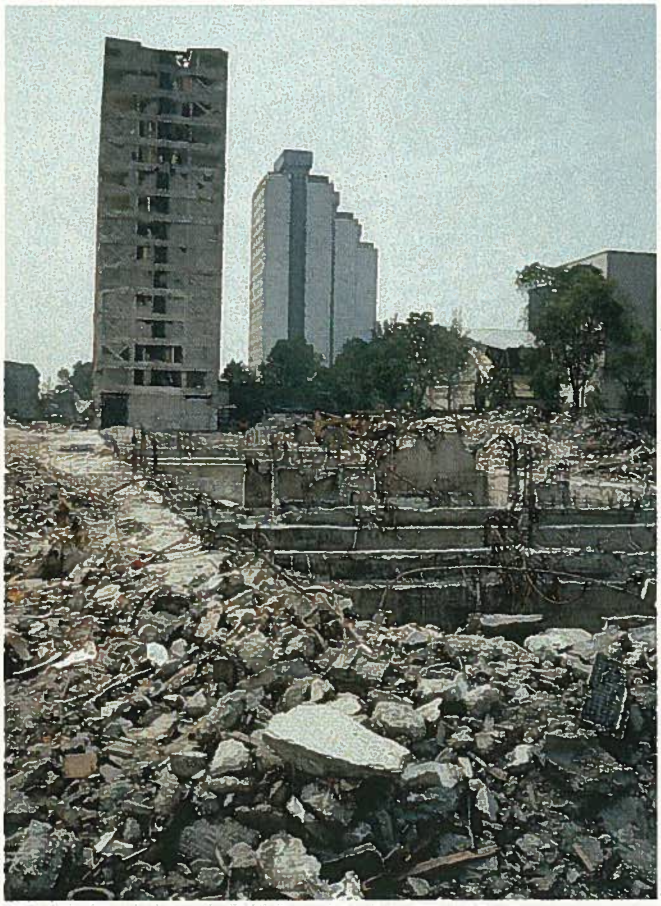

Fig. 45. Nuevo Leon Building, Mexico City, Mexico.

Fig. 46. Remaining section of building, Nuevo Leon, Mexico.

Fig. 47. Undamaged parking structure, Mexico City, Mexico.

before the children are dismissed from the classrooms for the lunch break.

Spitak, a city of 24,000 inhabitants located in the epicentral region, received an almost direct hit from this shallow earthquake. The city was almost completely devastated (see Fig. 49); all of its schools, hospitals, public facilities and most of the housing was destroyed. Loadbearing masonry and masonry with frames up to five stories were the predominant residential construction types in the city. About 90 percent of these structures collapsed — none escaped damage. Of the residential buildings in the city, only a five-story large precast concrete panel building survived the earthquake undamaged (see Fig. 50).

Leninakan, a city of 290,000 inhabitants located 20 miles (32 km) from the epicenter, suffered an enormous amount of casualties in collapsed modern residential high rise buildings (mostly nine-story precast concrete frame structures; see Fig. 51), schools,

Fig. 48. Map of Armenia.

Fig. 49. View of damaged city, Spitak, Armenia.

Fig. 50. Undamaged shear wall structure, Spitak, Armenia.

Fig. 51. Nine-story precast frames, Leninakan, Armenia.

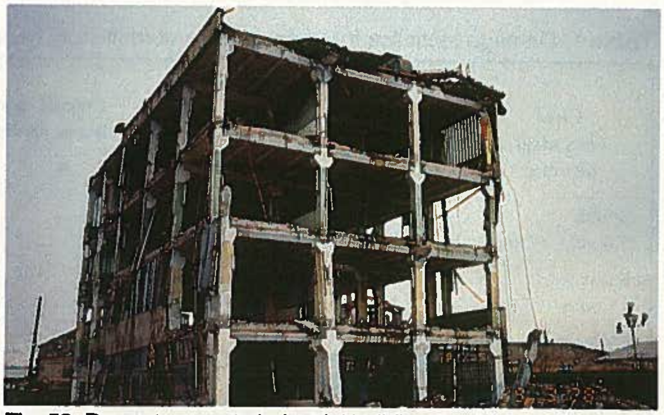

Fig. 52. Precast concrete industrial building, Leninakan, Armenia.

institutional, and industrial buildings (see Fig. 52). Some new residential sections of the city consisting of many high rise buildings were devastated. Many churches and other historical monuments were destroyed. The Technical University suffered a partial collapse of a major building, burying in the debris a large number of students. The 16 high rise precast concrete large panel structures remained undamaged (see Fig. 53).

Kirovakan, a city of 100,000 inhabitants located 15 miles (25 km) from the epicenter, suffered significantly less destruction than Leninakan. None of the 108 five- and nine-story precast concrete frame buildings collapsed or were severely damaged, although 88 of them needed some repairs or strengthening (see Fig. 54). Also, the four large precast concrete panel buildings in the city remained intact.

Summary of Damage Statistics

Table 1 is a compilation of a post-earthquake survey of buildings in the affected area. The four building types in the table are:

- Large precast concrete panel structures
- Precast concrete frame structures
- Composite frame/masonry structures
- Loadbearing masonry structures

The four levels of performance during the earthquake, denoted by the letters A, B, C, and D, are described at the bottom of Table 1.

It can be seen that in the precast concrete frame structures category, all of the collapsed and severely damaged buildings were in Leninakan. In the composite frame/masonry and loadbearing masonry structures categories, about 80 percent of collapses and heavy damage occurred in Spitak, Kirovakan, and Leninakan.

In analyzing the damage figures for the various building categories, the following becomes apparent:

- Large panel precast concrete structures performed very well. Periods of vibration, for nine-story buildings measured before and after the earthquake, were around 0.35 seconds. No damage was apparent in this structure category in the three cities listed in Table 1.
- Precast concrete frame structures, mostly residential nine-story buildings, had measured periods of vibration of about 0.6 seconds prior to the earthquake.

In Leninakan, 95 percent of these buildings either collapsed (54 percent) or needed to be demolished (41 percent). Only 5 percent could be repaired. None escaped damage.

In Kirovakan, which is closer to the epicenter than Leninakan, none of the precast concrete frame structures collapsed or needed to be demolished, while 19 percent escaped damage. The remaining 81 percent needed repairs or strengthening.

This significant difference in the performance of the precast concrete frame structures in the two cities suggests an examination of possible factors contributing to this diverse behavior, including:

- Quality of construction

Fig. 53. Nine-story large panel structure damaged by collapsing neighbor, Leninakan, Armenia.

Fig. 54. Nine-story precast concrete frame building, Kirovakan, Armenia.

Table 1. Damage statistics for various types of multistory residential buildings in the 1988 Armenia earthquake.*

City/ epicentral distance	Large panel precast concrete structures				Precast concrete frame structures				Composite frame/masonry structures				Loadbearing masonry structures			
	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
Spitak 5.6 miles (9 km)	—	—	—	1	—	—	—	—	43	9	7	—	20	2	3	—
Kirovakan 15 miles (25 km)	—	—	—	4	—	—	88	20	41	89	414	27	45	53	145	—
Leninakan 20 miles (32 km)	—	—	—	16	72	55	6	—	27	115	67	20	24	160	154	150
Total in three cities	—	—	—	21	72	52	94	20	111	213	488	47	89	215	302	150
Total in all Armenia	—	—	13	65	72	57	130	77	137	288	719	307	104	317	402	263

Levels of performance: A – Collapsed
B – Heavily damaged; to be demolished
C – Damaged; to be repaired or strengthened
D – No significant damage; usable

* Extracted from a compilation by Professor Der Kiureghian of the University of California at Berkeley based on Russian documents.

- Structural characteristics
- Earthquake intensity

There is no reason to believe that there should be a drastic difference in construction quality and in structural characteristics in the two cities because the working drawings and specifications were all prepared in Yerevan and most of the precast concrete elements were produced in centralized precasting plants.

Regarding earthquake intensity, Leninakan is founded on about 1000 ft (300 m) of alluvial deposits overlying volcanic tuff. Kirovakan is founded on firmer soil. It was also reported that in the months before the earthquake, the ground water level in Leninakan was unusually high. Historically, saturated alluvial deposits are associated with heavy earthquake damages.

It is, therefore, considered conceivable that the extreme damage in Leninakan was caused primarily by soil amplification of those frequency ranges in the spectrum that were close to the fundamental periods of some buildings, particularly the nine-story frame buildings. Consequently, resonance with the soil frequencies caused their collapse. Framing systems and construction quality (although both unquestionably important) may have played only a secondary role in this particular case.

Lessons Learned from the Armenian Earthquake

- Analysis of the damage distribution between various construction types and building heights in Leninakan and Kirovakan strongly suggests the influence of the soil period of vibration to be the primary factor of damage distribution.
- While hundreds of frame structures collapsed, not a single large panel structure was destroyed.

SUMMARY

In previous decades, significant attention was devoted to ductility details of structural systems. Some of these systems proved inappropriate for seismic resistance of concrete structures. Ductility details incorporated into the wrong structural system are wasteful and can create a false sense of security.

During the early days of earthquake engineering, many professionals confused ductility with flexibility. As a result, a large number of flexible buildings were built in many seismic areas of the world. Although some of these buildings may inadvertently have a reasonable degree of built-in ductility, their responses in future earthquakes have the potential to cause large economic losses, or even collapse, due to large interstory distortions.

In modern buildings, the cost of the structure may be as low as 20 percent of the total cost, while the remaining 80 percent is for the architectural, mechanical and electrical components. Thus, it is of primary importance to select a structural system with the best chance of providing both life safety and property protection in future earthquakes. For concrete structures, shear walls have demonstrated the ability to fulfill both of these requirements at the lowest cost.

Considering the suitability of structural systems as related to the functional requirements of buildings, we can divide the universe of multistory buildings into residential and commercial occupancies. There is no question that for residential buildings, shear walls can be used as the primary, or even the only, vertical load carrying elements, thus serving the double function of carrying the loads and dividing the space. In commercial buildings where large, unobstructed space is a functional requirement, a shear wall-frame interactive system (with sufficient shear walls) provides both rigidity and space flexibility.

Low rise and medium rise parking structures and school buildings are particularly vulnerable to earthquakes. Having large tributary areas with relatively small columns creates very flexible frames. Only stiffening such frames with sufficient shear walls (core and periphery) can substantially improve their earthquake resistance.

Ductility details for shear walls, which were developed as a result of recent laboratory tests and analytical investigations and incorporated into some codes, have not yet been tested in actual earthquakes. The inclusion of ductility details in shear walls will unquestionably improve the ductile properties of the walls. However, the extent to which shear wall ductility is actually utilized during earthquakes, and how such ductility affects the performance of the connected frames, remains to be determined using sophisticated dynamic response studies or in actual earthquakes. Also, the usually neglected rotation of the shear wall base due to soil flexibility may drastically reduce the moments induced into the shear walls during an earthquake.

In order to design a shear wall to behave in a ductile manner, which requires that its strength be governed by flexure rather than by shear, its shear capacity must be known and must be larger than the shear corresponding to its moment capacity. We need to know not only the ultimate shear capacity but also what happens between the onset of shear cracking and shear failure.

Whether and to what extent the grinding within shear cracks, caused by reversible cycles of lateral movement, can serve as an energy dissipation mechanism needs to be determined and has not yet been sufficiently investigated.

CONCLUDING REMARKS

During the earthquakes of the last three decades, buildings containing shear walls have exhibited very satisfactory earthquake performance. In most cases, the shear walls were reinforced in the traditional manner for gravity and overturning, without consideration given to special details for ductility, as required in recent United States building codes.

To the best knowledge of the author, who investigated and reported on the behavior of modern structures in a dozen earthquakes throughout the world, starting with the Skopje earthquake of 1963 through the Armenian earthquake of 1988, not a single concrete building containing shear walls has collapsed. While there were cases of cracking of various degrees of severity, no lives were lost in these buildings. Of the hundreds of concrete structures that collapsed, most suffered excessive interstory distortions that in turn caused shear failures of columns. Even where collapse of frame structures did not occur and no lives were lost, the large interstory distortions of frames caused significant property losses.

The above statement should not be taken to imply that frame structures built by the present advanced American seismic codes would also collapse in severe earthquakes. It has been demonstrated, however, that buildings containing even only conventionally reinforced shear walls, and also those of reinforced masonry or masonry infilled frames, have the capacity to withstand severe earthquakes, in many cases without damage.

After observing the devastations and the resulting loss of life in the many earthquakes, particularly those in Managua in 1972, Mexico City in 1985, and Armenia in 1988, the author believes it to be the responsibility of engineers and architects to make certain that residential buildings, in particular, be constructed with significant shear walls. Whether such walls are made of plain concrete, traditionally reinforced or reinforced for ductility will depend on the economic capacity of a given society and on engineering judgment; however, they all protect life and in most cases also provide good protection of property.

We cannot afford to build economical concrete buildings to resist severe earthquakes without shear walls.

POSTSCRIPT

The conclusions drawn in this artcle are based on the author's personal observations of structures damaged in a dozen major earthquakes, starting with the Chilean earthquake (May 1960) and ending with the Armenian earthquake (December 1988). Therefore, this article is not intended to be an exhaustive study of the performance of shear wall buildings during all earthquakes. In particular, the reader should consult two important articles related to the Northridge and Kobe earthquakes:

1. Iverson, James K., and Hawkins, Neil M., "Performance of Precast/Prestressed Building Structures During Northridge Earthquake," PCI JOURNAL, V. 39, No. 2, March-April 1994, pp. 38-55.
2. Ghosh, S. K., "Observations on the Performance of Structures in the Kobe Earthquake of January 17, 1995," PCI JOURNAL, V. 40, No. 2, March-April 1995, pp. 14-22.

In the Kobe earthquake, no shear wall buildings collapsed.

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13. Fintel, Mark, and Ghosh, S. K., "Case Study of Aseismic Design of a 16-Story Coupled Wall Structure Using Inelastic Dynamic Analysis," *ACI Journal,* V. 79, No. 3, May-June 1982, pp. 171-179. See also Discussion in *ACI Journal,* V. 80, No. 2, March-April 1983, pp. 151-152.
14. Fintel, Mark, "Performance of Precast and Prestressed Concrete in Mexico Earthquake," PCI JOURNAL, V. 31, No. 1, January-February 1986, pp. 18-42.
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