STRUCTURAL DAMAGE CAUSED BY
THE SKOPJE EARTHQUAKE OF 1963

by

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INTRODUCTORY NOTE

The city of Skopje, Yugoslavia, was struck by a severe earthquake on 26 July 1963. The writer was in Skopje during the period 17 to 23 September 1963 as an observer for the Committee on Masonry and Reinforced Concrete of the American Society of Civil Engineers and for the American Concrete Institute. This report is based on his observations of structural damage.

Over 90 percent of the construction in Skopje was unreinforced masonry. The damage to such construction was heavy and had tragic consequences. Although examples are given in the report, no effort was made to study the response of unreinforced masonry buildings in any detail since this type of construction holds no promise for earthquake-resistant design.

There were a considerable number of buildings with complete or partial reinforced concrete frames, about 500 according to the writer's estimate. In view of the fact that these buildings were not designed for earthquake effects, their behavior was surprisingly good. For the same reason, it was not good enough and a study of their failings emphasizes and reinforces the experience gained from previous earthquakes.

There were reported to be two industrial plants with steel frames on the outskirts of the city. These buildings were not visited by the writer.

A brief description of the town and the 26 July 1963 earthquake in the preliminary chapters is followed by a general discussion of the damage to various types of construction visited. Four case studies of damage to reinforced concrete buildings, to varying degrees of depth permitted by the available information, is described in the final chapters.
SKOPJE

Skopje is the capital of Macedonia, one of the six republics of Yugoslavia (Fig. 1). It is, after Belgrade and Zagreb, the third largest city of Yugoslavia and occupies a strategic position on the communication routes of the Balkan Peninsula. Skopje is located very close to the middle of a 220-mile line joining Tirana, capital of Albania, to Sofia, capital of Bulgaria. Belgrade lies 220 air miles NNW and the Mediterranean seaport of Salonika 125 air miles SE. Skopje is on the 42nd parallel in the Northern Hemisphere. Chicago, Illinois, is a few miles south of the same parallel.

Although founded earlier as an Illyrian settlement, the recorded history of the town dates back to the Roman conquest of Macedonia when the Romans fortified Scupi, located about 2.5 miles NW of the present town, in the Third Century B.C. to serve as the capital of Dardania. This settlement was decimated by an earthquake in 518. Justinian I had the town rebuilt in its present site as an outpost of the Eastern Roman Empire. Skopje served as the capital of King Milutin of Serbia in the 13th century and as a provincial governor's seat of the Turkish Empire from the 14th to the 20th century during which time it was again struck by a devastating earthquake in 1555. Serbia annexed Skopje and surrounding territory in 1912. Since World War II Skopje has been the capital of the Socialist Republic of Macedonia. In 1923, the inhabitants of the town numbered about 40,000. There was little change in this number up to 1949 when it was 49,000. However, the population burgeoned in the fifties. At the time of the earthquake, Skopje contained 213,000 people, 198,000 of which were permanent residents.

Skopje straddles the Vardar, a river emptying into the Aegean and is surrounded by high terrain (Fig. 2). The elevation of the town is 800 ft. The slopes of the Vodno, a mountain rising to 3500 ft, start at less than a mile south of the town. The Jakupica, a mountain further south, has an elevation of 3300 ft. The southern ranges of the Crna Gora and the northern part of the
Sar Planina range wall the north and the west. Ground elevations are not as high, but still mountainous, on the east side of town.

The surrounding mountain ranges are metamorphic formations. The town is built on alluvial deposits supported by Tertiary sandstones and marls. The region features several faults and is considered to be one of the active zones of the Alpide Belt.

The Vardar separates the old town, on the north bank, from the new town, on the south bank (Fig. 3). Very little building activity occurred south of the river up to the turn of the present century. The old town is built on rising terrain, marked by the Kale hill which towers some 125 ft above the Vardar, while the new town is relatively flat between the river and the railroad. A series of photographs (Fig. 6-11) taken from Kale present a general view of the new town. The suburb Karpos (Fig. 3) is photographed from a hill located about 5 miles SW of the city in Fig. 12. A view from Kale shows part of the old town in Fig. 13.

South of the railroad, the ground rises to form the slopes of the Vodno, seen in the background of Fig. 9. The focus of the current community may be considered to be Marshal Tito Square (H6 in Fig. 3) with an important axis formed by Marshal Tito Avenue which joins the square to the railroad station on Zeleznika Street (K6 in Fig. 3). Marshal Tito Square is seen in the upper left-hand portion of Fig. 10.

The foundation soils for the town are good, though not uniform. Reportedly, the depth of gravel mixed with sand was about 30 ft near Marshal Tito Square. In the eastern suburbs of the new town, gravel was found in borings up to 300 ft. The water table was reported to be about 30 ft below ground level. An oversimplification compares the foundation characteristics on the two sides of the river: the allowable bearing pressure was 4 tons per sq. ft in the old town and 2 tons per sq. ft in the new town. The better foundation characteristics of the old town are due to the proximity to the surface of the metamorphic formations.
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THE EARTHQUAKE OF 26 JULY 1963

As mentioned earlier, the Skopje region had been subjected to major earthquakes in 518 and 1555. Although the historical descriptions are not completely reliable, there is little question about the fact that these two earthquakes had been very severe. Earthquakes of intensity VII on the Modified Mercalli Scale have been recorded in 1890 and 1921 along with numerous quakes of lower intensity throughout the history of the region. This active record coupled with the knowledge of the structural faults in the area should have taken the surprise element out of the earthquake of 26 July 1963.

The earthquake occurred on a Friday morning at 5:17 a.m. local time (4:17 GMT). The USCGS preliminary report puts the epicenter at 42.1°N, 21.5°E, about 7 miles and N15°E of town, on the slopes of the Crna Gora (Fig. 2). The preliminary estimate by the USCGS of the magnitude of the earthquake was 5.4 on the Richter scale. Other stations reported the magnitude as follows:

- Athens : 6.0
- Belgrade : 6.2
- Strasbourg : 5.8
- Stuttgart : 6.7
- Tokyo : 6.5

No reliable strong motion records were available to the writer. Reportedly, the seismological facilities of the University of Skopje were damaged during the earthquake. Eyewitness reports varied in relation to the direction of the ground motion, but almost all persons interviewed agreed that the shock waves hit the town in three bursts at ten to fifteen second intervals, with the second burst, presumably strong transverse waves, being the most violent.

It can be speculated on the basis of movements of walls with respect to fixed objects that the ground motion may have been at least four in. in town. There were reports of some factory machinery having been moved as much as eight in. by the earthquake. However, the writer was not able to confirm these reports.
The writer could find no evidence to contradict the epicenter location indicated by the USCGS even though many observers reported the epicenter to be within the town. On the contrary, the damage to most of the buildings in town fitted in with the assumption of an epicenter outside but near the town at about N 15° E. Furthermore, studies by Dr. Ambraseys of aftershock data recorded at the temporary seismological station at Skopje indicated the epicenters of the aftershocks to be from 7 to 9 miles NNE of town.

AMOUNT AND DISTRIBUTION OF DAMAGE

Damage

While the writer was in Skopje, general statistics of property loss were in a liquid state. There was a serious crisis related to the problem of housing the population before the onset of winter, and sufficient manpower could not be diverted to checking and rechecking the damage to each building. It was not always a straightforward task to separate superficial from structural damage. Consequently, the over-all damage figures were not firm. The writer still does not possess official and final statistics. However, the following figures should not have changed by more than 10 percent of each category.

Skopje contained 35,600 residential units providing 19 x 10^6 sq. ft. of floor area. The classification of damage was described as follows:

<table>
<thead>
<tr>
<th>Percent of Total</th>
<th>Residential Units</th>
<th>Floor Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Destroyed</td>
<td>8.5</td>
<td>7</td>
</tr>
<tr>
<td>Heavy Damage</td>
<td>34</td>
<td>30</td>
</tr>
<tr>
<td>Medium Damage</td>
<td>36</td>
<td>40</td>
</tr>
<tr>
<td>Light Damage</td>
<td>19</td>
<td>20</td>
</tr>
<tr>
<td>No Damage</td>
<td>2.5</td>
<td>3</td>
</tr>
</tbody>
</table>

Less than 60 percent of the apartment units survived the earthquake. Only about 20 percent were in a habitable condition. The relative damage to public and industrial buildings was comparable. The writer was told that about 70 percent of public and industrial buildings were seriously damaged. A news item recorded
that of the 45 industrial buildings 14 had been completely and 13 partially
destroyed. Only 18 of the 45 had survived with minor damage.

The official count of fatalities was 1070 as of September 1963. A
total of 2900 persons had received medical aid at hospitals, 1110 of these being
serious cases.

**Distribution of Damage**

A view of town from a distant point would reveal a town hardly
damaged. Figure 12 is a photograph of the suburb Karpos, a heavily damaged
area, taken from a hill SW of Skopje. Unless one compares this photograph
with one taken previously and on a microscopic scale, it is difficult to con­
clude that the town has been subjected to an earthquake. Figure 14 shows the
skyline along the river before and after the earthquake. Again, the change
in the skyline does not seem to agree with the statistics provided in the
previous section. The reason for this appearance is that most of the buildings
which influenced the skyline of the city, the tall buildings, had framed
construction and survived the earthquake. However, the damage was extensive
to the lower buildings which were not salient in the skyline of the town.

There was very heavy damage to the old construction on the north
bank of the river. However, the results of this destruction were not as
serious because most of these houses were limited to one or two stories and
had relatively light roofs. Figure 15 shows a street in the old town.

The most critical damage occurred in the new town, between the river
and the railroad tracks south of the river. The fact that this was also the
area of inundation in the November 1962 flood is not coincidental. However,
the relationships between the flood area and the earthquake damage area may
be one of association rather than one of cause and effect. The flooded area
was the alluvial valley of the Vardar and, as indicated before, the foundation
conditions were different in this low area compared with areas on the north bank of the river and south of the railroad tracks. It was also reported that some of the buildings on Marshal Tito Square had developed foundation problems after the flood.

The immediate damage of the earthquake was limited to structural failures caused by the ground shock. Only one case of fire is known to the writer. A fire was started in the second floor of a technical highschool building. But this fire was extinguished before spreading since there was no critical damage to the water supply system of the city.

Some structures in the new town were observed to have foundation problems after the earthquake (tilting). These cases were not studied by the writer.

**DAMAGE TO UNREINFORCED MASONRY STRUCTURES**

**Old Construction**

The predominant type of residential construction on the north bank of the river was the adobe building. Some examples of this system were also to be found scattered on the south bank of the river. These buildings, limited to two-stories in height, were made of unbaked brick and with a rather poor quality of mortar (Fig. 15). The walls were strengthened by pieces of timber arranged vertically and diagonally. The timber roof trusses or beams were covered with clay tiles.

These buildings sustained heavy damage throughout the town. Financially, this was not a critical loss since a very large percentage of these buildings had outlived their usefulness and were about ready to fall down or to be taken down. The number of lives claimed by the destruction of these buildings was low because these buildings lacked heavy roof or floor slab systems. The inclusion of timber beams in the walls appeared to have helped their resistance to lateral forces.
New Construction

The new unreinforced masonry construction in Skopje was the seat of the tragedy. Most of these buildings, all below six stories in height, were built to accommodate the population boom. On the surface, they appeared to be solid and well finished structures. The interiors were well planned and equipped with modern conveniences. They could have adorned any suburb in post-war Europe and, the writer suspects, similar structures do.

The brick bearing walls were 10 in. thick and perforated with many windows. The roof was covered with either a flat reinforced concrete roof slab or timber roof trusses supporting clay roof tiles. In general, the floors comprised precast reinforced concrete beams supporting plain concrete slabs cast in place. Heavy reinforced concrete beams encircled the walls. Lintel beams were also made of reinforced concrete.

This type of construction was used in almost all of the new residential units and for some of the public buildings built after World War I.

The failures were catastrophic. In one instance, a residential unit covering one quarter of a block collapsed with the floor slabs stacked on top of each other. A case that received much publicity because of the international background of its victims was the Hotel Makedonia shown in Fig. 16. The Army Club (Fig. 17) was a noteworthy failure in that it exemplified the behavior of the early 20th Century type of public building with heavy, pretentious domes, slender columns and walls, and very little strength. Figures 18 and 21 show the fate of apartment buildings with brick bearing walls. Figure 22 shows the inclined cracks in the perforated bearing wall of an apartment building in Karpos.

There is little to be said about the magnitude, undesirability, and consequences of horizontal strength to stiffness ratio of brick walls.
However, one wonders by how much the fatalities would have been reduced if the reinforced concrete slab, a heavy element, had not been combined so freely with the masonry bearing walls. Furthermore, the mortar used in the construction of the brick bearing walls was of rather poor quality since lime was used as a binder. The walls collapsed into rubbles of individual bricks and any mortar that stuck to the bricks could be scratched off very easily. The windows (Fig. 22) obviously did not alleviate the problem.

The behavior of two small two-story brick houses in Karpas is of interest. These buildings were completely intact after the earthquake and struck quite a contrast with the unoccupied buildings and destruction around them. One of them is shown in Fig. 23. There were other two-story brick houses a few blocks north of this building and they suffered heavy damage.

The east elevation of the Skopje City Theater, on the north bank of the Vardar (G6 in Fig. 3), is shown in Fig. 24. This masonry building was also damaged heavily and may have to be demolished. The random fall of the statues (eight before the earthquake) at the edge of the roof is noteworthy in relation to the arguments for an epicenter in the middle of town inspired by the random damage.

Another masonry structure, possibly dating from the 15th Century, the Mustafa Pasha Mosque is shown in Fig. 25. The slender masonry minaret, which had its crown sheared, survived the earthquake as did several others (Fig. 13).

**Hybrid Construction**

Many modern buildings in Skopje which appeared, on the surface, to have complete structural frames included a masonry bearing wall as part of the load-carrying system. Sometimes one and sometimes a whole group of columns would be replaced by a masonry bearing wall. Reportedly, the bearing wall was used in order to reduce the cost of the construction.
This combination of two unlike structural elements, often led to the destruction of the bearing wall and that part of the building depending on the bearing wall for support. An example of this antipathetic symbiosis is described in detail in a following chapter (Koco Racin Apartment Buildings). Although this type of building was classified under the category of framed structures by some observers, it should belong in the classification dictated by its weakest element, the unreinforced masonry wall.

DAMAGE TO REINFORCED CONCRETE STRUCTURES

Design and Construction

According to unofficial sources, about 5 percent of the buildings in Skopje had reinforced concrete frames and two thirds of this group had survived the earthquake with light damage. The total number of buildings with reinforced concrete frames was about 500. However, the writer believes on the basis of his sampling that at least 10 percent of these might be hybrid construction not to be classified as reinforced concrete.

Despite the fact that none of these structures had been designed or detailed for an earthquake approaching Intensity IX, their behavior was surprisingly (but not uniformly) good. One case of collapse and several cases of irreparable damage are discussed in the following pages. On the other hand, it should be noted that no fatalities were reported in reinforced concrete buildings, this clean a record being partially providential since not many persons could have survived the collapse of Exhibition Hall No. 1. The Trade Union Building (Fig. 26), a 13-story modern reinforced concrete structure (located in J7, Figure 3), survived the shock with the damage limited to the cracking of a few partition walls. Two other buildings which came through the earthquake with minor damage are shown in Figs. 27 and 28.
Despite insistent inquiries, the writer could not obtain specific information about the building code in force in Skopje or in the Republic of Makedonia and was led to believe by the variation of the answers that the building specifications would change depending on the ownership of the building and the contracting enterprise.

The materials most often used were 3,000 psi concrete and plain round bars of structural grade with a yield stress of about 34,000 psi. Reportedly, higher strength concrete was used in the lower story columns of the high-rise buildings. The writer was told of one building in which high strength twisted bars (Isteg) were used.

Generally, the frames were heavy in one direction, usually the short plan dimension of the building, and were connected with relatively light beams in the opposite direction. The lateral forces were assigned completely to shear walls. The frame was not designed to carry any lateral loading. The taller structures were reportedly designed for a lateral load equal to two percent of the gravity load. (The wind load was said to be about 20 psf.)

The labor force was not skilled. However, the supervising engineers were competent and had a good tradition in reinforced concrete construction.

The critical factor in the design and construction of the buildings appeared to be the shortage of time and materials, especially reinforcement. The minimum amounts of reinforcement were extremely low. Column reinforcement could be as low as 0.3 percent. Transverse reinforcement was minimal in beams and in columns.

Inspection of concrete and formwork seemed to be spotty. The writer observed some excellent work. But the city had more than its share of substandard work. There were too many cold joints. An extreme example of bad
formwork was a column which bowed to one side for a maximum deflection of 1.5 in. In several building constructions visited, the floors sagged because of inadequate shoring.

It would be unfair not to mention that the construction standards in Skopje were as good as any that could be found in Eastern Europe. Had it not been for earthquake, the buildings would have fared well. Furthermore, it is not too much of a generalization to mention that the main cause of the tragedy was not poor construction but inadequate design and detailing. The excuse for the latter was the pressure of the population boom and the gamble that had to be taken to house the people. Skopje was no worse prepared for an earthquake than any other city in the Balkans.

The following paragraphs describe some examples of damage to reinforced concrete frames in Skopje. Without wishing to indulge freely in the pleasures of hindsight, the writer would like to point out that none of the failures observed were unpredictable and that all could have been avoided with relatively little cost had the contingency of an earthquake been considered in the design and planning of these structures.

**Strength of the First Story**

Because of its use for business or because of architectural requirements, the first story of many of the buildings in Skopje was taller than the other stories and had fewer walls. The upper stories were stiffened by brick walls enclosing the story completely in addition to the interior partition walls. Thus, the whole building approached a relatively simple system formed by a rigid mass supported by flexible elements permitting

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*The gamble had to be taken again. While the writer was in Skopje, he observed damaged reinforced concrete columns being replaced by masonry columns.*
translation, torsion, and rocking.

Such a system can be used intentionally to reduce the effect of the shock on the upper stories. However the first story elements must possess the strength to restore the structure to its original position. Since a violent ground motion was not anticipated in the design of such buildings in Skopje, the first story columns did not have adequate flexural strength and sustained permanent deformations. Although the upper stories suffered little damage because of the shock-absorbing ductility of the first story, the whole structure was made useless because of lack of strength in the first story.

Southwest elevations of two apartment buildings on Zeleznika Street shown in Fig. 29 and 30 exemplify this problem.

Torsion

Many of the buildings with open first stories had staircases and walls arranged nonsymmetrically in plan. However, the eccentricity was not large and the stiff elements were not very strong. Therefore, there were not many cases of distress caused by torsion. A rather severe example occurred in the Student Union Building to be discussed in detail in a later chapter.

Hammering

The writer did not observe many cases of severe hammering damage in Skopje not because this problem had been anticipated but because the tall buildings were spread at random throughout the city. Hammering damage was observed at the expansion joints of the Student Union and Transformer Buildings (see pertinent chapters).

Interaction with Nonstructural Elements

This was the most common cause of damage to reinforced concrete columns. An example is shown in Fig. 31. The shear on the column, caused
by the lateral movement, was enhanced by the stiffening effect of the wall which extended over only half the story height. A similar example is discussed in relation to the failure of the Student Union Building.

Any solid element in contact with the frame will affect the behavior of the building. Classifying a wall as a nonstructural element does not relieve the structural engineer from the responsibility of considering this interaction which can be used to advantage but which, if ignored in design, can lead to disaster.

**Connections**

The critical point in the earthquake-resistant design of the reinforced concrete frame is the connection between vertical and horizontal elements, since that is where the moments and shears usually reach a maximum. The problem is not simplified by the fact that the connection is also the place where it is convenient to splice the reinforcement.

The connection must satisfy one basic requirement: the paths of tensile and compressive forces must be continuous through it even after large deformation reversals. Two factors essential to this requirement were lacking in Skopje: (1) Adequate splicing using deformed bars and/or special connection devices, and (2) transverse reinforcement to improve the ductility of the concrete, to counteract the bursting stresses that may be caused by the splice, and to resist shear. These shortcomings were aggravated quite often by construction defects such as segregated concrete and misplaced reinforcement at the connections. Although it cannot be blamed entirely to a poorly designed connection (since the connection was supposed to be elsewhere), the failure shown in Fig. 31 represents an example. A more serious example is shown in Fig. 32-34. The failure shown in the first story was repeated in the basement.
Many others are discussed later in this report in relation to the Transformer Building and the Student Union Building.

Most frame failures are ultimately connection failures, although initiated by other causes. Consequently, the connection must be designed, detailed, and constructed with concern for continuity.

Reinforcement in Columns

The amount of longitudinal reinforcement used in columns was seldom greater than one percent and usually half a percent or less. The flexural strength and energy-absorbing capacity of a column depends critically on the amount of longitudinal reinforcement. Underreinforcement of the columns was one of the primary causes of "First-story weakness" encountered in Skopje. Combined with the lack of adequate lateral reinforcement, it led to failures as illustrated in Fig. 34.

Reinforcement in Beams

As in the case of columns, the beams were lightly reinforced. Since the energy-absorbing capacity of an underreinforced beam is practically independent of the amount of reinforcement, this situation would not have provided any problems. Furthermore, the top reinforcement appeared to be sufficient to prevent yielding in the beam. However, there were three features of beam reinforcement that caused trouble most frequently: (1) Since moment reversals were not anticipated, the bottom reinforcement was not anchored in or made continuous through the columns. (2) The top reinforcement was cut off, sometimes completely, close to the column. (3) Transverse reinforcement was light, if any. None of these would have resulted in weakness under gravity loads. However, they proved inadequate for lateral loads. The description of the damage to the Transformer Building provides an example.
Staircases

Almost every staircase the writer saw in Skopje was damaged, especially at the first story. Although evidently not considered in the design calculations, the staircases developed considerably large lateral forces as a result of the structural deformations resulting from the earthquake. While they were rigid enough to develop these forces, their reinforcement was very light. An example of damage is shown in Fig. 35. Figure 36 shows the damage inflicted on a girder by the rigidity of the staircase shown in Fig. 35.

Shear Walls

The behavior of shear walls and buildings with shear walls was generally satisfactory. Three problem sources were observed.

Some concrete shear walls were completely or partially unreinforced. For example, the shear walls of the City Hotel (Fig. 27) were reinforced only up to the fourth floor. Fine diagonal cracks were observed in the wall at the fourth floor.

It appeared to be standard practice to dig channels in the shear walls for electrical conduits. These caused unsightly crumbling of the plaster covering and must have weakened the whole structure.

There were some cases of parallel shear walls in the same plane connected by beams. These beams cracked badly and it is doubtful whether they were of any significant help to the structure, since the beams were shallow and lightly reinforced.

THE TRANSFORMER BUILDING

Location and Description

The Transformer Building*, which housed the business offices of the local power enterprise and the equipment for power distribution, was completed

*Designation assigned to the building by the writer.
only about four months before the earthquake. It was a modern reinforced concrete structure situated at the intersection of Leninova and Debarce streets as indicated in Fig. 3. The front of the building was parallel to Leninova Street which was oriented at N 24°E.

The neighboring buildings had sustained heavy damage. Several masonry buildings in the immediate vicinity had collapsed. The apartment building shown in Fig. 18 was in the same block and about 150 ft NE of the Transformer Building.

The building comprised two structurally independent parts (Fig. 37). The main part, L-shaped in plan, included a basement and three stories rising to approximately 50 ft above ground. A tall first story housed electrical equipment while offices occupied the second and the third stories. The smaller part, curved in plan, contained the reception area and some offices at the first floor and a large meeting room at the second. It rose to approximately 30 ft above ground.

Various elevations of the Transformer Building can be seen in Fig. 38 through 43. The camera angles for the photographs are indicated in Fig. 37. Both parts of the building had overhangs in front at the second story level (Fig. 38 and 39).

**Structural System**

The structural system of only the main part of the building is described here since the critical damage occurred in the main part. This part of the building had heavy reinforced concrete framing in the shorter dimension of its plan. In the longer direction, the frames were joined by relatively light beams.

The arrangement and plan dimensions of the columns are given in Fig. 44 which also indicates the frame designations. The layout in the vertical plane...
of Frame A is also given in Fig. 44. The transverse frames, excluding the cantilevers for the overhang, are shown in Fig. 45.

Frame 1 (Fig. 45) had three columns and four girders. Except for the space taken by two windows and a door, which can be seen in Fig. 43, a 10-in. brick wall filled the frame.

Frames 2 through 5 (Fig. 45) differed significantly from Frame 1. The members were heavier. However, the girder spans were longer and the first story bay extended about 33 ft above the foundation. As may be seen in Fig. 42, 10-in. brick walls butted against the East columns for a height of about 20 ft from ground, but there were no walls within the frame below the second story. Partition walls (approximately 5-in. thick, one brick plus plaster) occupied the second and third story bays as indicated in Fig. 37.

Frames 6 and 7 (Fig. 45) were in the wider portion of the building and although the west bay of these frames were identical to Frames 2 through 5, they were stiffened by two additional bays. The west bay of Frame 6 had a 10-in. brick wall throughout the height of the building (Fig. 41). Frame 8 had four bays in each story and lighter members as in the west bays of Frame 7.

The floor systems of the Transformer Building was of a type very popular in Skopje. Precast joists, approximately 3 by 12 in., and spaced at about 2 ft spanned between the transverse frames and supported a cast-in-place plain concrete slab less than 2-in. thick. A 3 by 12-in. diaphragm was used at mid-span. (See Fig. 36 for an example with smaller joist spacing)

No definite information was available on the amount and arrangement of the reinforcement and the quality of the materials. According to a construction foreman who had worked on the building, the design strength for the concrete was about 3000 psi (cylinder strength) and the yield stress for the plain bars about 34000 psi.
Inspection showed the quality of the concrete to be quite satisfactory in most places. However, it appeared that the concrete in some of the columns had been cast on top of the debris that had fallen into the form. It also appeared that the reinforcement had not been securely tied during casting: there was considerable variation in concrete cover.

The plaster cover was quite heavy on both the inside and outside walls. The thickness of the plaster was usually one in. and approached 1.5 in. in many places.

**Damage**

The lateral movements of the building during the earthquake were reflected by the extensive but superficial damage suffered by the heavy layers of plaster. In fact, these thick layers acted as a brittle stress-coating and, in many places, made the damage appear more serious than it was.

The damage to the plaster on the external walls can be seen in Fig. 38 through 43. Although superficial, the falling plaster was a hazard because of its weight. It should be noted that almost all of the exterior plaster damage occurred on the east-west walls (Fig. 38, 41, and 43).

Plaster damage also revealed hammering between the two parts of the building (Fig. 40). Hammering damage was limited, possibly because of the low height of the building and the direction of the major earthquake shock.

Another salient sign of damage, again superficial, was the fall of the cupolas on the air ducts. Only one of these five cupolas, located nearest to the stiffer part of the building in the east-west direction, remained in place after the earthquake (Fig. 42).

Inside the building, severe nonstructural damage was observed at the third floor level where part of the corridor west wall, between frames 3 and 2, had fallen toward the east (Fig. 46).
Figure 38 reveals some spalling on the west column of Frame 7 at a height of approximately five ft from ground level. Figures 47a and b show close-ups of the spalled area. Evidently, the spalling occurred at a column splice and the problem was one of bond stress.

The connection between the first story column (east) and the second story floor girder of Frame 4 is shown in Fig. 48. The inclined crack in the column (only two-thirds of the column depth is seen in the photograph) indicate a serious failure dominated by combined bending and shear stresses. The vertical crack in the girder was caused by moment reversal at the joint.

A series of photographs shown in Fig. 49 through 55 show the east-west frames at the second story level. (The camera angles are indicated in Fig. 37.)

There was no damage to Frame 1 (Fig. 49) except for some cracks indicating moment reversal at the corner shown. An outside view of most of Frame 1 is seen in Fig. 43. The cracks in the wall at the first story level next to the door and the lack of severe cracking elsewhere indicate that this frame had behaved as a stiff beam or shear wall. (It is quite likely that a column on the west side of the door would have eliminated the wall failure.)

Figures 50 and 51 show the north and south sides of Frame 2 at the second story level. An inclined crack is seen about six ft from the inside face of the east column. The cracks in the plaster did not penetrate the partition wall which had little damage. A similar inclined crack appears in the girder of Frame 3 (Fig. 52 and 53). The partition wall contained in Frame 3 is seen to be damaged. Figures 54 and 55, showing Frame 4, reveal a very serious inclined crack in the girder and serious damage to the partition wall. Frame 5 was not damaged.

Figure 56 summarizes the damage to the main part of the Transformer Building. The nonstructural damages to the third-story corridor wall and the
air-duct cupolas have been shown because they provide an indication of the
direction the major inertia force and the relative deformations of the frames.

The cupolas and the third-story corridor wall fell toward the east.
In a rigid structure, this would imply a strong ground motion toward the west.
The existence of such motion is confirmed by the orientation of the wall cracks
seen in Fig. 41. Furthermore, the structural damage to the first-story column
and the third-story floor girders is compatible with a ground motion toward
the west: the inclined crack in the column indicates a westward horizontal
reaction applied by the foundation and the cracks in the girders indicate
upward reactions by the east columns.

The collapse of the partition wall occurred in the top story and was
confined to the middle of the narrower portion of the structure. The damage to
the cupolas was also more severe in the same area. The cupolas next to frames
3 and 4 had the most damage (Fig. 42). These two observations suggest that the
horizontal movement was greater for frames 3 and 4 than for frames 1 and 6
and, of course, that it increased with height above the ground. As discussed
before, frames 1 and 6 were stiffer in the horizontal direction than the inter-
mediate ones and the floor system was not sufficiently rigid to limit relative
deflections of the frames.

The discussion of the structural damage must involve some speculation
since information on the amount and arrangement of reinforcement was not avail-
able to the writer. The column failure of Frame 4 shown in Fig. 48 was apparently
one in combined bending and shear. Although the column was slender with a free
height to over-all depth ratio of about 15, it could have been sufficiently
stiffened by the masonry wall bearing against it on the east side of the frame.
to develop a large shearing force.
The splice failure in Frame 7 is compatible with the direction of the strong motion suggested by other observed damage. Its appearance in Frame 7 rather than in one of the intermediate frames which apparently underwent larger deformations is attributable to variation in construction quality.

The third-story floor girder failures shown in Fig. 49-55 have the appearance of failures in combined bending and shear. However, it is difficult to attribute the initiation of these failures to combined bending and shear for two reasons: (a) the longitudinal reinforcement ratio required to develop the girder end moments which would be compatible with a unit shear sufficient to cause inclined cracking is on the order of four percent, considerably beyond practical limits and (b) the crack was located about six ft from the face of the nearer girder while the typical inclined crack would be expected to be in the immediate vicinity of the connection where the moment is greater.

The top of the inclined crack, not seen in the photographs because of the ceiling, approached the quarter-point of the span. It is not too speculative to assume that the negative moment reinforcement was reduced drastically, if not discontinued, at that point. Thus, when the moment at that point was reversed from positive to negative, the section was virtually unreinforced. A flexural crack formed and, being unrestrained by adequate reinforcement, penetrated rapidly into the girder, becoming inclined as the unit shear stress increased because of the reduced section. The final almost horizontal portion of the crack relates to the level of the positive moment reinforcement. On the basis of this hypothetical explanation of the failure phenomenon in the girders, it appears that the primary weakness was in the arrangement of the flexural reinforcement.
THE STUDENT UNION BUILDING

Location and Description

The Student Union Building was situated near the intersection of JNA Boulevard and Lole Ribera Street (J5 in Fig. 3). The two-story reinforced concrete building was the front one, on Lole Ribera Street, of a complex of three interconnected buildings. Two views of the Student Union Building are shown in Fig. 57 and 58. The eight-story dormitory building is seen in the background of Fig. 57. Between the dormitory and the Union was a one-story restaurant which can be seen partially in the right-hand side of Fig. 58. There was no structural connection between the three buildings. This discussion relates only to the Student Union Building.

The floor plans of the two stories of the building and its orientation are shown in Fig. 59 and 60. The building was divided into two structurally independent parts by an expansion joint at the center of Column Line 4. The framing for the west part was fairly uniform: each frame had two stories and two bays. In the second story of the east part, the middle columns (line B) were omitted for frames 6-9.

The height of the first story was about 10 ft. The height of the second story was approximately 14 ft at Frame 1, 12 ft at Frame 4 and 17 ft at Frame 9. The columns were spaced at about 18 ft in the longitudinal and 20 ft in the transverse direction. They were approximately 15 in. round or square (plus a one-inch layer of plaster). At the first story level, all but columns A2-A6 were round. At the second story level, the round columns were C1-C9 and B1. The main girders of the frames ran in the N-S direction. The E-W connections between the frames were provided by relatively light beams.

The first-story round columns contained 8 plain round bars with a diameter of approximately 0.5 in. tied by 0.25-in. single round ties at 7.5
in. Reportedly, the target strength of the concrete was approximately 3000 psi and the yield stress of the reinforcement 34,000 psi.

**Damage**

Nonstructural damage to the building was limited primarily because there were a limited number of partition walls. Approximately the top half of the north exterior wall of the west room in the second floor was completely destroyed. There were large diagonal cracks in the west wall of the same room (Fig. 57). There was little damage elsewhere to the exterior walls of the second story. The east wall on that floor showed virtually no signs of damage (Fig. 58). Figure 61 shows the damage to the ornamental wall at the southeast corner of the first story. There was considerable damage to glass on all sides and at both levels of the building.

The critical structural damage to the building occurred east of the expansion joint and was intimately related to it. The photographs in Fig. 62-65 show various views of Frame 4. The east and larger part of the building had virtually rotated clockwise in the horizontal plane about column C4.

The southward movement of the east portion of column C4 at the top of the first story was less than one in. while the southward movement of the corresponding point of column C9 was close to five in. Columns C8 and C9 had been able to accommodate this movement with only local crushing at the top of the first story (Fig. 67). Column B9 had serious crushing at the bottom (Fig. 68 and 70). An ornamental wall had caused serious consequences for columns A8 and A9. (Fig. 68 and 69) The only other column damage was the spalling on the east side of the base of column B5 at the first story level.

Torsional rotation of the east part of the Student Union Building is consistent with the distribution of its mass and structural framing: the structure was heavier near frames 8 and 9 while the framing was stiffer for frames
4 and 5 with the second floor bay of Frame 5 stiffened considerably by a partition wall. Without flexurally strong first story columns or without major changes in the structural layout, the permanent horizontal deformation of the east end was unavoidable. However, the shear failure of column A8 and the flexural failure of column A9 could have been prevented by separating the "ornamental" wall from the columns.

TWO APARTMENT BUILDINGS ON KOCO RACIN STREET

Location and Description

The damage to two identical apartment buildings located on the north side of Koco Racin street is of interest because these buildings were hybrids combining a reinforced concrete frame with brick bearing walls. In the following paragraphs, these buildings will be referred to as KR1 and KR2.

The location of the two buildings is indicated in Fig. 3 (K7). Both buildings were on the north side of Koco Racin Street which they faced. Building KR1 was approximately 300 ft south of KR2.

At the time of the earthquake, the KR buildings were in final stages of construction. Work was almost complete on KR1 (Fig. 71) and the top story framing had just been completed for KR2 (Fig. 72).

The ground floor plan for the two buildings is given in Fig. 73. Longitudinally, there were 14 bays, each bay about 12 ft long center-to-center of columns. The column spacing was about 15 ft 9 in. in the transverse direction. The building included a basement, 5 stories and a penthouse and rose to about 63 ft above ground. The first story height was 13 ft. At the second-story level, the building was cantilevered for 3 ft 3 in. from the column center line on the SE and NW sides (front and back of the building).

At the ground floor, columns with 10 by 16-in. rectangular sections were used only for the two end frames, with the 40-in. dimension oriented in
the transverse direction. All other columns had circular sections with a diameter of 16 in. In the stories above, the columns were rectangular in section and measured 10 by 16 in. They were oriented to suit the wall arrangement. Girders with 10 by 16-in. sections spanned between the columns only in the transverse direction. Precast beams with 2.5 by 12-in. sections with a 2-in. cast-in-place plain concrete slab spanned between the girders at a spacing of 16 in.

The critical feature of the structural system was the omission of framing at three locations as shown in Fig. 73. Columns were replaced by 10-in. brick bearing walls around the staircases in the two sides and at the center of the building.

No information is available to the writer on the amount and arrangement of reinforcement which consisted of plain round bars. There were no special features of the structural system to lead one to think that the usual materials (3000-psi concrete and 34,000-psi steel) had not been used in the structure. Reportedly, the columns rested on isolated footings at a depth of approximately eight ft.

**Damage**

The photographs in Fig. 71 and 73, both taken after the earthquake, reveal virtually no damage. The photographs show only the front and one side which were completely framed. Unfortunately, the damage was serious in the back of the buildings where the framing was not complete.

Figure 74 shows a partial view of the back (north end) of KRL. The damage to the wall was unimportant structurally since the wall was not a bearing wall. The cracks occurred because the wall could not accommodate the movement of the tall first story.

Figure 75 shows one of the staircase walls in KRL. The brick wall was on the verge of disintegration. So were the other two staircase walls
in KRL. Figures 76, 77, and 78 describe the disaster around the north staircase walls of KR2. The other bearing walls in the structure were also heavily damaged and on the verge of collapse.

The damage to the reinforced concrete was slight and limited to local spalling of the concrete at girder-column connections in the first story. If the unreinforced brick bearing wall had not been combined with the reinforced concrete framework, all indications were that both buildings would have survived the earthquake with only superficial damage.

**EXHIBITION BUILDING NO. 1 OF THE SKOPJE FAIR**

**Location and Description**

The Skopje Fair is situated on the north bank of the Vardar on the east side and at the foot of the hill on which the Kale stood. Exhibition Building No. 1 was one of a proposed complex of three buildings, of which only two had been built at the time of the earthquake. The location of Exhibition Building No. 1 is indicated in Fig. 3 (F6) and its orientation is shown in Fig. 79. Figure 80 shows a photograph of the building, which had a shell roof, taken before the earthquake. The building was completed in 1960.

The major structural feature of the building was the roof, a translational shell supported on four approximately 3 by 3-ft columns on 98 ft (30m) centers. The columns rose to about 27 ft above ground level. The shell was formed from two orthogonal parabolas with rises of about 10 and 13 ft at center. The edges of the shell were stiffened by beams, and 30-in. deep prestressed tie girders connected the tops of the supporting columns. The shell thickness was about 2.5 in. (6cm).

A series of columns supported a 33-ft wide second-floor gallery which was continuous around the building (Fig. 80). The gallery floor, about 15 ft above ground, was cantilevered from the columns and extended 25 ft inside
and 8 ft outside from the column center lines. Ribs of varying depth emanated from the columns to carry the floor. The four heavy columns supporting the shell were incorporated in the floor system. The flat roof of the gallery was presumably supported by the tie girder at the inside edge and by a series of light columns at the outside edge.

No definite information was available to the writer on the reinforcement and quality of the materials.

**Damage**

East and west views of the structure after the earthquake are shown in Fig. 81 and 82. The shell had collapsed and the roof of the gallery had fallen, with a slight rotation, to the level of the floor.

The heavy columns supporting the shell on the north, east and south corners of the building are shown in Fig. 83, 84, and 85, respectively. The columns were destroyed above the gallery floor. The tie girder can be seen on the gallery floor in the right-hand side of Fig. 83 and the left-hand side of Fig. 84. A portion of the tie girder, lying on the ground, is shown in Fig. 86. The south corner of the tie girder and the shell edge beam can be distinguished in the upper part of Fig. 85.

Damage to the system supporting the gallery floor was very little despite the heavy vertical impact to which it must have been subjected. The only serious damage was noted on the west corner of the building where two of the cantilevered ribs had failed. Cracks had formed in other ribs indicating the inadequacy of top reinforcement for heavy vertical forces applied near the ends of the ribs, a condition which would not have been considered in their design.

From the ruins of the building and what is known about the framing of the structure, it appeared that the failure of the shell was caused by the
failure of the supporting columns above the first floor. This costly failure could have been avoided by providing adequate bending strength in the columns supporting the shell or by improving the framing system of the second floor so that most of the lateral forces set up by the top-heavy structure did not have to be resisted by the four main columns.

SUMMARY

Object of Report

The city of Skopje, Yugoslavia, was struck by an earthquake of magnitude 5.4 (USCGS preliminary estimate) on 26 July 1963. The epicenter was estimated to be within 10 miles of the city. Since over 90 percent of the construction in Skopje was in unreinforced masonry, the structural damage was serious, reaching an intensity of IX on the Modified Mercalli Scale.

The writer spent six days in Skopje in September 1963. This report records his observations of structural damage. After providing general information about the town, the earthquake, and the damage, the report concentrates on describing the damage to two buildings with reinforced concrete frames, two hybrid structures combining a reinforced concrete frame with unreinforced brick bearing walls, and a shell supported on reinforced concrete framework.

Skopje

At the time of the earthquake, Skopje (Fig. 1), population 213,000, was the third largest city of Yugoslavia. It is situated on the banks of the Vardar (Fig. 2) in mountainous Macedonia. Alluvial deposits supported by tertiary sandstones and marls form the foundation medium for the buildings many of which had been built since 1950. A street map is given in Fig. 3. Figures 4-15 show various views of the town. The region is in the Alpide Belt and features several active faults. Two previous major earthquakes, in
518 and 1555, were recorded. A recent flood (November 1962) inundated the area between the river and the railroad causing heavy damage.

**Damage**

As implied by the preponderance of unreinforced masonry construction, the buildings were not designed for earthquake effects. There were about 500 (writer's estimate) structures with complete or partial reinforced concrete frames and the taller buildings (6 to 14 stories) were designed for lateral loads amounting to approximately 2 percent of the gravity load (or about 20 psf). In design, the walls were assumed to resist all of the lateral loads. There were reported to be two steel industrial buildings on the outskirts of town. These buildings were not visited by the writer.

According to information obtained locally, 42.5 percent of the 35,600 residential units were lost in the earthquake, 36 percent suffered structural and 19 percent nonstructural damage. Only 2.5 percent of the residential units survived the earthquake without damage.

The damage was heavy to the old adobe buildings. However, the fatalities were relatively light in these buildings since they had light floor and roof systems (Fig. 15). The most costly damage, in terms of human life and economic value, occurred in the unreinforced brick masonry buildings (Fig. 16-24). Although not designed for earthquake effects, the damage to buildings with reinforced concrete frames was relatively light (Fig. 26-28).

Failures observed in reinforced concrete buildings were recurrent types that have been observed after various other earthquakes in buildings not planned to resist earthquakes: relative weakness of a story in the lateral direction (Fig. 29 and 30), interaction of frame components with nonstructural elements (Fig. 31 and 69), inadequate column reinforcement (Fig. 32-34), staircase damage (Fig. 35 and 36), hammering (Fig. 40), inadequate beam reinforcement (Fig. 55) and torsion.
The Transformer Building

The floor and framing plans of the transformer building are shown in Fig. 37, 44, and 45. The series of photographs in Fig. 38-43 describe the building externally. (Camera angles are indicated in Fig. 37.) The damage to the main part of the building is summarized in Fig. 56 and described in Fig. 46-55.

The longitudinal axis of this building pointed approximately in the direction of the earthquake epicenter, so that the transverse ground motion must have been in virtually the same plane as the heavier frames which spanned the short plan dimension (Fig. 37). The frames at the two ends of the main part of the building were laterally stiffer than those in the middle (Fig. 45). The relative deflections of the frames can be traced in the damage to the third-story floor beams (Fig. 49-55) and the cupolas (Fig. 41). No information on the reinforcing scheme was available to the writer. However, it appeared that the fatal inclined cracks in the girders had been initiated by inadequate longitudinal reinforcement.

The Student Union Building

The two-story Student Union Building (Fig. 57 and 58) suffered heavy damage because of torsional rotation in the horizontal plane. The floor plans are given in Fig. 59 and 60. An expansion joint through column line 4 divided the structure into two. During the earthquake, the part of the building east of the construction joint virtually pivoted about column C4 causing heavy damage to the columns supporting the east end (Fig. 68-70). The damage to columns was enhanced by the presence of an ornamental wall.

Koco Racin Apartment Buildings

These two buildings represent examples of the hybrid building combining a reinforced concrete frame with masonry bearing walls. The facades of
the two buildings after the earthquake, are shown in Fig. 71 and 72. The ground floor plan (Fig. 73) shows where reinforced concrete columns had been replaced by masonry bearing walls. Figures 76-78 describe the damage caused by the failure of the unreinforced walls.

**Exhibition Hall No. 1 of the Skopje Fair**

Figure 80 shows a photograph of Exhibition Hall No. 1, a translational shell supported on reinforced concrete columns, before the earthquake. Figures 81 and 82 show the same structure after the earthquake. The plans for the ground and gallery floors are given in Fig. 79. The 98-ft square shell was supported on four columns, the lateral thrust being resisted by four pre-stressed tie girders spanning between the tops of the columns. Over the height of the first story, the main columns were strengthened by a series of columns serving to support the gallery floor. Over the height of the second story, the main columns were the primary source of lateral strength. From the ruins of the building, it appeared that the fall of the roof shell and the gallery roof was caused by failure in bending of the main columns (Fig. 83-85) which evidently were not designed to resist the large inertia forces related to the mass of the shell.
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FIG. 2 SKOPJE IN RELATION TO NEARBY TOWNS AND THE EPICENTER OF THE 26 JULY 1963 EARTHQUAKE
FIG. 3 TOWN MAP OF SKOPJE
1. The Transformer Building
2. The Student Union Building
3. Koco Racin Apartment Building
4. Exhibition Building No. 1 of the Skopje Fair
FIG. 4 KALE BEFORE THE EARTHQUAKE, SOUTHEAST VIEW
FIG. 5  KALE AFTER THE EARTHQUAKE, EAST VIEW
FIG. 7 VIEW OF SKOPJE FROM KALE, LOOKING ESE
FIG. 8 THE NATIONAL ASSEMBLY BUILDING
FIG. 11 VIEW OF SKOPJE FROM KALE, LOOKING SSW
FIG. 13  VIEW OF SKOPJE FROM KALE, LOOKING NORTHEAST
FIG. 14  SOUTH BANK OF THE VARDAR NEAR MARSHAL TITO SQUARE
FIG. 17 THE ARMY CLUB BEFORE AND AFTER THE EARTHQUAKE
FIG. 18  UNREINFORCED MASONRY APARTMENT BUILDING NEAR THE INTERSECTION OF LENINOVA AND GURO GAKOVIK STREETS
FIG. 20  UNREINFORCED MASONRY APARTMENT BUILDING IN KARPOS
FIG. 22 HEAVILY DAMAGED UNREINFORCED MASONRY APARTMENT BUILDING IN KARPOS
FIG. 23 TWO-STORY UNREINFORCED MASONRY BUILDING IN KARPOS
FIG. 24  THE THEATRE BUILDING, EAST ELEVATION
FIG. 28  KARPOS TOWER NO. 3
FIG. 29 PARTIAL SOUTH VIEW OF APARTMENT BUILDING 1 ON ZELEZNIKA STREET
FIG. 30 SOUTH VIEW OF APARTMENT BUILDING 2 ON ZELEZNIKA STREET
FIG. 31 DAMAGE TO COLUMN
FIG. 32. SOUTHEAST VIEW OF KARPOS TOWER NO. 1
FIG. 37 PLAN OF THE TRANSFORMER BUILDING (SECOND AND THIRD FLOORS)
FIG. 40  PARTIAL VIEW OF EAST ELEVATION SHOWING THE JOINT BETWEEN THE TWO PARTS OF THE TRANSFORMER BUILDING
FIG. 43 PARTIAL VIEW OF NORTH END OF BUILDING
FRAME A

Width of Roof Girders = 8"
Width of Floor Girders = 10"

FIG. 44 THE TRANSFORMER BUILDING, COLUMNS AND LONGITUDINAL FRAME A
FIG. 45 THE TRANSFORMER BUILDING, FRAMES 1-6
FIG. 46 DAMAGE TO CORRIDOR WALL IN THE THIRD STORY OF THE TRANSFORMER BUILDING
FIG. 47 SPLICE FAILURE IN THE TRANSFORMER BUILDING
FIG. 48 DAMAGE TO COLUMN OF FRAME NO. 4,
FIRST FLOOR, EAST SIDE, TRANSFORMER BUILDING
FIG. 49 FRAME NO. 1, SECOND FLOOR, EAST SIDE, SOUTH ELEVATION, TRANSFORMER BUILDING
FIG. 50 FRAME NO. 2, SECOND FLOOR, EAST SIDE, NORTH ELEVATION, TRANSFORMER BUILDING

FIG. 51 FRAME NO. 2, SECOND FLOOR, EAST SIDE, SOUTH ELEVATION, TRANSFORMER BUILDING
FIG. 52 FRAME NO. 3, SECOND FLOOR, EAST SIDE, NORTH ELEVATION, TRANSFORMER BUILDING

FIG. 53 FRAME NO. 3, SECOND FLOOR, EAST SIDE, SOUTH ELEVATION, TRANSFORMER BUILDING
FIG. 54 FRAME NO. 4, SECOND FLOOR, EAST SIDE, NORTH ELEVATION, TRANSFORMER BUILDING

FIG. 55 FRAME NO. 4, SECOND FLOOR, EAST SIDE, SOUTH ELEVATION, TRANSFORMER BUILDING
FIG. 56 DAMAGE TO THE TRANSFORMER BUILDING
FIG. 57 SOUTHWEST VIEW OF THE STUDENT UNION BUILDING
FIG. 59 FIRST FLOOR PLAN, STUDENT UNION BUILDING

FIG. 60 SECOND FLOOR PLAN, STUDENT UNION BUILDING
FIG. 62  SOUTH VIEW OF EXPANSION JOINT AT COLUMN C4, STUDENT UNION BUILDING
FIG. 63 WEST VIEW OF COLUMN C4, STUDENT UNION BUILDING
FIG. 64 SOUTH VIEW OF EXPANSION JOINT AT COLUMN A4, STUDENT UNION BUILDING

FIG. 65 FLOOR SEPARATION AT SECOND STORY, NEAR COLUMN B4, NORTH VIEW OF COLUMN BASE, STUDENT UNION BUILDING
FIG. 70 CLOSE-UPS OF DAMAGE TO COLUMNS A9 and B9
FIG. 73 GROUND FLOOR PLAN FOR THE KOCO RACIN APARTMENT BUILDINGS
FIG. 74 BACK WALL OF KOCO RACIN BUILDING NO. 1, NORTH CORNER
FIG. 76 DAMAGE AROUND STAIR SHAFT OF Koko RACIN NO. 2, LOOKING UP AND SOUTHWEST
FIG. 77 DAMAGE AROUND STAIR SHAFT OF KOCO RACIN NO. 2, LOOKING UP AND SOUTH
FIG. 85  EXHIBITION HALL NO. 1, SOUTH MAIN COLUMN