Fig. 52. Agadir Municipal Building, After the Earthquake

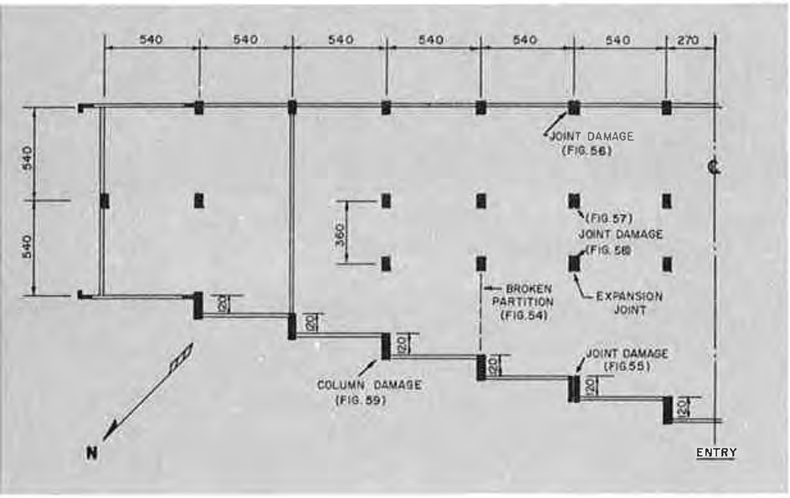

Fig. 53. Sketch of Column Layout, Municipal Building

Fig. 54. Municipal Building, Damage to Tile Partitions

The Municipal Building (City Hall)

A few of the newer reinforced concrete frame structures in Agadir had been constructed with moment resistant joints and were well tied together. Although the lateral force design of these structures took account of wind forces only, and thus provided relatively low seismic resistance, their performance during the earthquake was noticeably better than that of the concrete frame structures designed only for vertical load. One of the most attractive of these concrete frame buildings was the Municipal Building, shown in Fig. 52, which was built about 1955. The planned height of this structure was eight stories, and the structural design had been based on the full height. However, at the time of the earthquake it had been completed only to four stories with the intention of adding the additional stories at some future date. For this reason, the structure was considerably stronger than a normal building of this size.

The basic dimensions of the structure are shown in Fig. 53. It is apparent that the stepped pattern of the front of the building made it expedient to use columns which were rather deep in the NW-SE direction. Thus the building was quite resistant to lateral forces along this axis, which happened to coincide with the direction of the principal earthquake shock.

The view of the building shown in Fig. 52 gives the impression that the structure was not damaged by the earthquake. However, closer inspection revealed that non-structural damage (to partitions, interior plaster, windows, etc.) was very extensive, and that a significant amount of structural damage also occurred. Fig. 54 is a view of the front left part of the lobby of the

Fig. 55. Fracture of Girder-column Joint, Front Interior, Municipal Building

building, just inside the front doors. The clay tile partition which had spanned between columns at the left side of the photograph has been shattered because its rigidity greatly exceeded that of the columns. As the columns reacted to the lateral earthquake force (toward the left in this picture), they deflected, and the partition which was too stiff to accommodate this deflection (and too weak to support the earthquake force) ruptured. Damage to other interior partitions is also evident in this figure.

A typical example of the structural damage caused by the earthquake is presented in Fig. 55, which shows the girder-column joint located at the expansion joint at the front of the building (location noted in Fig. 53). Damage at the equivalent location to the rear of the building is shown in Fig. 56, and at the interior columns in Figs. 57 and 58. It is quite likely that part of the damage shown in Figs. 55 to 58 is due to relative movements of the structure on the two sides of the expansion joint. However, the principal point to be noted is that a relatively deep girder was provided between columns along the expansion joint, while at the other column lines the columns framed directly into the floor system. Thus, the effect of relative rigidities again is demonstrated. The frames along the expansion joint were stiffer than the other frames in the structure and were not able to accommodate as large displacements as the others. Damage to column joints were severe only at those locations where the girders were framed in. Where the column attached directly to the floor systems (providing greater flexibility), very little damage was observed. However, some evidence of working was apparent at the base of most of the columns, outside the structure, as shown in Fig. 59.

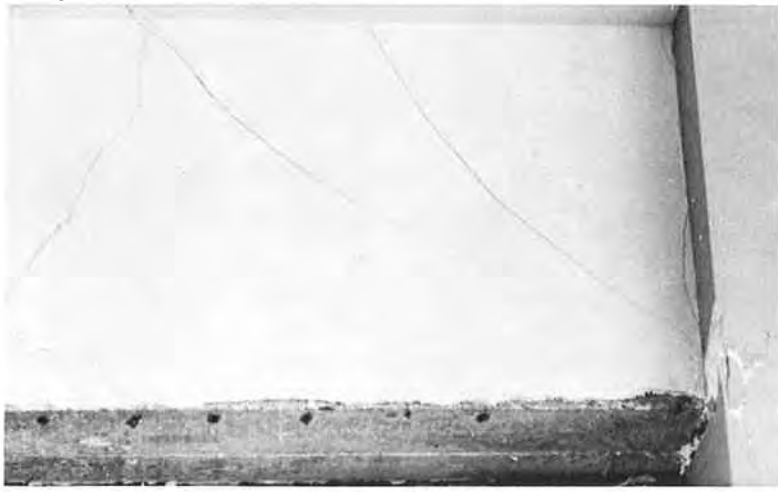

Fig. 57. Fracture of Joint at Interior Column, Municipal Building

Fig. 58. Fracture of Joint at Interior Column, Municipal Building

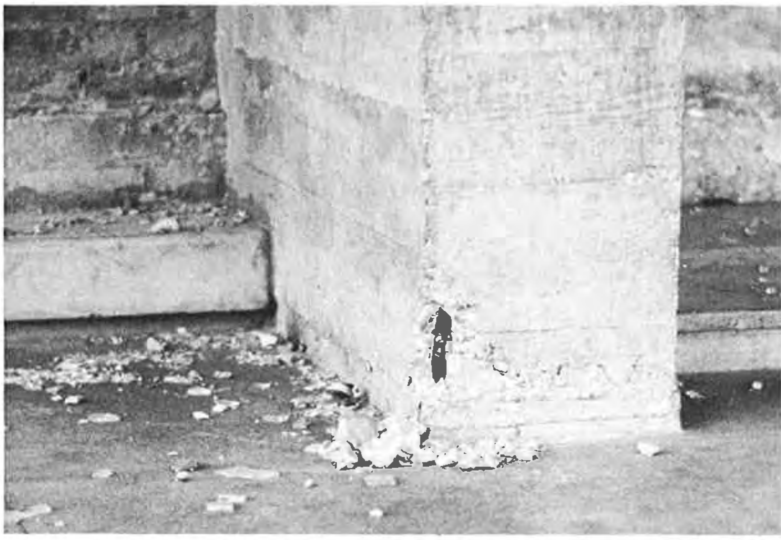

Fig. 59. Buckling of Reinforcement at Base of Column, Front Exterior of Municipal Building

Fig. 56. Fracture of Girder-column Joint, Rear Interior, Municipal Building

Fig. 60. Immeuble Paternelle in the New City, from the South

Immeuble Paternel

Another modern concrete frame structure of striking appearance is the Immeuble Paternel, shown in Fig. 60. This building housed the "Banque Populaire of Agadir" on thè first floor, while the upper three stories were used for apartments. The concrete frame of this structure was severely damaged at the first floor level where the lateral force was resisted primarily by the exterior tapered columns. Two of the columns shown in Fig. 60 failed completely; Fig. 61 gives a closer view of one of these. The failure appears to be predominantly one of compression; reinforcing bars have buckled completely, with the result that the column is offset by more than an inch at each of two levels.

The cracking of the exterior filler wall in the projecting end of the building shows considerable working of the frame at the second floor level, the same type of damage being shown on the other side of the building as well (Fig. 62). Typical damage suffered by the interior partitions at the second floor level is shown in Fig. 63. It will be noted that the interior structural column in the center of the picture also had suffered appreciable damage.

Fig. 61. Immeuble Paternelle, Detail of Column Fracture

Fig. 62. Immeuble Paternelle, from the Southeast

Fig. 63. Second Story Interior of Immeuble Paternelle, Showing Partition Damage

Fig. 64. New City Municipal Market

Fig. 65. Second Story of Market Building, Two Sections Collapsed, One Still Standing

Fig. 66. Damage to Second Story Floor System of Market, Resulting from Collapse of Roof

Fig. 67. Detail Demonstrating Large Yield Capacity of Market Frame Structure

Municipal Market

The market building of the New City, shown in Fig. 64, was also a concrete frame structure having moment resistant joints; but it did not perform as well during the earthquake as did the Municipal Building and the Immeuble Paternel. The first story of this building had very heavy concrete frames, designed to support heavy warehouse loadings of produce on the second floor. The roof of the upper story having a very light live load, however, was supported by rather slender columns. The entire structure was separated into three parts (each approximately 65 to 100 feet in plan) by expansion joints. The roof over two of these sections collapsed completely, while the third section was severely distorted as shown in Fig. 65. The collapse of these roof sections undoubtedly caused heavy vertical dynamic loadings on the unloaded floor system below, resulting in tremendous cracking and yielding of the slabs, joists, and frames as shown in Fig. 66. The earthquake itself, however, had practically no direct effect on the unloaded first floor structure, as was demonstrated by the excellent condition of the structure beneath the section of roof which did not collapse. The distortions which the lower story accommodated without collapse (Fig. 67) are of great interest in the design of earthquake resistant structures. A properly designed structure built of suitable materials can absorb a large amount of energy in plastic deformations; thus it is unrealistic to assume that a structure must behave elastically in the maximum "design earthquake." The manner in which this energy absorption concept has been incorporated into the most recent building codes will be discussed in Chapter 4 of this report.

Fig. 68. Racking of Reinforced Concrete Columns

Fig. 69. Fracturing of Masonry Filler Walls Reveals Reinforced Concrete Frame

Fig. 70. Reinforced Concrete Frame Carries Load in Spite of Damage to Masonry

Miscellaneous Structures

It may be instructive to consider a few other examples of structures having reinforced concrete frames. FIG. 68 shows a building which was located across the street from the Sud Building. It clearly shows the effect of the strong earthquake ground motion acting in the northwesterly direction. The reinforced concrete columns of the first story have been severely damaged at base and top. The lateral strength of the upper stories was greater than that provided by the first story columns, and all major damage was confined to the first floor level. The gap which has opened between this building and its neighbor to the left is of interest. There is no evidence of pounding between the two buildings, which indicates that there was no acceleration pulse of the ground toward the southeast to match the intensity of the pulse toward the northwest which opened the gap. Clearly this was an earthquake of short duration with only one or possibly two principal acceleration pulses.

Other examples of the damage to reinforced concrete frame structures are shown in FIGS. 69 and 70, in which it will be noted how the damage generally was concentrated in the lower stories of the structure. Such results would be expected from an earthquake consisting of a single sharp acceleration pulse, because in such a case the total lateral force at any level depends primarily upon the mass of the structure above that level. The principal lateral strength of these structures was provided by the masonry filler walls; thus the strength was nearly constant over the height, and the failure, therefore, occurred in the zone of strongest forces. It is interesting to note in FIG. 69 how the masonry walls have cracked away (characteristic "x"-cracks), exposing the concrete skeleton which still supports the structure.