Engineering Aspects of

THE MANJIL, IRAN EARTHQUAKE OF 20 JUNE 1990

A FIELD REPORT BY EEFIT

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SUMMARY

The behaviour of traditional masonry buildings in Iran during the Manjil earthquake of 20 June 1990 proved, yet again, the vulnerability of these buildings to seismic loading. In this respect very little could be learnt from the Manjil earthquake that had not already been observed and noted in numerous strong earthquakes of the past few decades in the country. In many other respects, however, the Manjil earthquake may be considered different to the past earthquakes, in that it struck a densely populated and relatively industrialised part of the country, affecting a large number of engineered and semi-engineered buildings and other structures. As a result, perhaps for the first time, the performance of such code-recommended measures as concrete ring-beams, engineered version of steel I-beam, jack-arch roofing system and the more recent concrete beam-block system in small buildings, as well as the behaviour of taller steel and concrete framed buildings could be studied in relative detail in the field.

In this report the performance of these semi-engineered and engineered buildings, as was observed during a post-earthquake field visit, will be discussed. Amongst many observations, the good behaviour of concrete ring-beams in mitigating the collapse of the roof and floor slabs is worthy of mention. The importance of providing principle or secondary load-bearing elements in the form of concrete columns was also evident in many cases. An important observation made on the response of the engineered version of steel I-beam jack-arch slabs (in which I-beams are restrained by transverse beams and/or tie-bars) was that such composite slabs are only suitable as roof slabs, simply supported on the walls via the ring-beam. Their behaviour as fixed-sided floor slabs in two-storey and higher buildings is less favourable. This is because the interaction between the brittle brick arches and the flexible steel beams under vertical (out-of-plane) vibration of the slab results in the disintegration and collapse of the brick arches.

The performance of the steel-framed buildings appeared very poor. Save for a few buildings situated in the epicentral area, they suffered heavy damage or collapsed. Those which were not badly damaged, survived the earthquake as a result of the incidental frequency range of the ground shaking which was much higher in that area than the fundamental frequencies of the buildings. The main point of weakness of the steel-framed buildings was in their welded joints.

Poor welding rendered weak connections which snapped before the steel sections could develop any significant dynamic stresses. The response of concrete-framed buildings, as a whole, appeared much more favourable than their steel-framed counterparts. An interesting aspect of the response of a number of framed buildings was the clear evidence of the effects of lateral-torsional dynamic interaction in these buildings. Such interactions are caused by stiffness and/or mass eccentricities in a building which result in amplified response of one side of the building.

Based on the observations made during the visit to the affected area and recalling many already established lessons and facts from the previous earthquakes, some recommendations are made on increasing the seismic strength of certain types of common buildings in Iran.

The region devastated by the Manjil earthquake is a well watered agricultural, and industrial area. As a result a number of large engineered structures such as dams, ground-based and elevated liquid storage tanks, silos, concrete and steel bridges, industrial plants and factories were affected by the earthquake. This report also examines the behaviour of some of these structures during the earthquake. The most important structure to be subjected to severe ground shaking was the 106m high, 425m long, aseismic designed buttressed Sefid-Rud dam. This dam is an important source for electricity generation and imperative to the agriculture in the area. Although the epicentre of the quake was determined as only 300m north-east of the dam it survived the estimated 0.65^{+} g ground acceleration with some cracking in central buttresses and the crown. Two other dams in the area, Sangar and Tarik, Both diversion dams also survived the quake. Failure in two of the thirteen steel gates of the Sangar dam and the spalling of concrete due to the pounding of the bridge deck against the piers in the Tarik dam was the main damage in these two dams. The only large structure to completely fail under the earthquake was a 47m high reinforced concrete, elevated water tank in the city of Rasht. The tower was apparently not designed to withstand earthquake forces. Two similar water towers in Rasht, however, survived with minor damage, mainly because they were empty at the time of the quake. Steel and concrete bridges, on the other hand, behaved well during the earthquake, partly as a result of their relatively low natural frequencies of vibration. None of those visited had suffered serious structural damage. Other large structures investigated, including a number of factories suffered varying degrees of damage.

Two important seismic design considerations, absent in most of the above structures include; (i) appropriate seismic joints and (ii) the safety of secondary elements or systems . Inadequate (or complete lack of) seismic joints between different sections of the structure which are invariably of different dynamic properties resulted in many local failures due to pounding. It was also apparent in many instances that in the seismic design of secondary structures and associated elements and in installation of systems and equipment, the secondary response of such elements and systems had been overlooked.

1 INTRODUCTION

At 20:43:12 GMT Wednesday 20 June 1990 (00:13:12 Thursday local time) a devastating earthquake struck the north western provinces of Gilan and Zanjan in Iran. The first reports placed the epicentre somewhere in the south-west of the Caspian Sea. However, later as the extent of the affected areas became apparent the epicentre was located at the mountainous Rudbar region of Gilan province near the town of Manjil.

The earthquake was strongly felt in Tehran 200 km south-east and Tabriz 300 km north-west of the epicentre and as far afield as Turkey and the Soviet Union. It devastated a large, densely populated rural area of northern Iran destroying a number of towns and hundreds of villages. The official reports of casualties put the number of dead at over 40,000 with half a million homeless. The material damage is estimated as over 7 billion dollars. The fact that it happened at night while people were in their homes and the poor resistance of masonry houses to the forces of the quake both contributed to the high casualty toll.

Different seismological centres gave the size of the quake as between 7.3 and 7.7 on the Richter scale. This makes the Manjil quake the strongest earthquake in recent years to strike a centre of population. The quake was associated with an 80 to 100 km long fault running east-west. Numerous rock falls and land slides followed the main event and the stronger after shocks, blocking roads and damaging structures. The quake also caused changes in the level of the water table, and resulted in soil liquefaction in vast areas.

In order to investigate the engineering aspects of the earthquake damage, on behalf of the British Earthquake Engineering Field Investigation Team (EEFIT) and in collaboration with the International Institute of Earthquake Engineering and Seismology in Iran (IIEES) and with the financial support of WS Atkins Group, the author visited the stricken area.

As in many previous earthquakes in Iran, the collapse of un-reinforced brick masonry roof and floor slabs of 1 to 3 storey houses was responsible for the majority of the casualties. The roofs of most of the collapsed buildings were either the traditional brick masonry dome type or the flat slab steel I-beam and jack arches. Neither type, when un-reinforced, have the ability to withstand the horizontal forces of an earthquake. Following the failure of the load-bearing masonry walls these roofs simply disintegrated and collapsed.

There were, however, many semi-engineered and engineered residential buildings in the area, particularly in larger villages and towns. These buildings in general behaved better and although in the epicentral area almost all buildings were damaged beyond repair, most maintained their integrity and did not collapse.

The semi-engineered buildings in the area have varying degrees of resistance to earthquake. A minimum code requirement which is the provision of reinforced concrete ring-beams at the roof level was observed in most of the more recently built houses. In the majority of cases where the ring-beams were supplemented with I-beams supporting the ends of the load-bearing steel beams of jack-arch roofs, the roof acted as a unit and stayed in place. This was despite the collapse of large portions of the supporting walls. Different aspects of the response of semi-engineered buildings is discussed in relative detail in Chapter 3.

There were also many engineered residential and non-residential buildings in the epicentral area. These were either reinforced concrete or steel framed buildings 1 to 5 storeys high. The reinforced concrete buildings generally behaved better than the steel-framed buildings, this was mainly due to the weak welded joints of the latter buildings which in many instances, simply snapped under the earthquake loading. In most of these buildings the floor slabs and roof, the main causes of casualties, stayed in place.

Further afield in the city of Rasht, home to 300,000 people, the damage was largely to the taller buildings. The ground acceleration at Rasht (60 km from the epicentre) was evidently less than in Manjil or Rudbar. Also the fact that mainly taller buildings (6-storey plus) were damaged indicates that the strong frequency range of the quake at Rasht, a city in the plain, was less than the frequencies of ground vibration in the mountainous Rudbar region, perhaps another reason why the concrete and steel framed buildings (3 storey plus) behaved better in the latter area. These aspects as well as a number of other observations made on the behaviour of engineered buildings are further discussed in Chapter 4.

Rudbar region of Gilan province is a well watered agricultural area and as a result there are a number of large engineered structures such as dams, water towers, silos as well as bridges and industrial plants and factories in the area affected by the quake. The response of these structures to the forces of the earthquake is the subject of discussion in Chapter 5.

Although the main subject of this report is the response of structures to the earthquake, the seismological aspects and other characteristics of the Manjil quake are also briefly discussed in the following chapter (Chapter 2). The information given in Chapter 2 is mainly based on the preliminary data provided by the investigators from IEESS, the Institute of Geophysics of Tehran University and the Research Institute of Ministry of Housing. Publication of more detailed and accurate account of these aspects by the above-mentioned organizations and other seismological teams from abroad who have visited the stricken area is anticipated in due course.

2 GENERAL CHARACTERISTICS OF THE EARTHQUAKE

2.1 SEISMIC HISTORY OF THE AREA

The area affected by the 20th June earthquake has a long, recorded history of seismic activities. Its position, in the Alburz mountain ranges of north Iran, part of the Alpine-Himalayan seismic belt, has made the area very vulnerable to recurring earthquakes. This is recognised in the Iranian Code for Seismic Resistant Design of Buildings (1988). In the above code the Manjil-Rudbar region is considered a high risk area where all buildings should be constructed to provide certain amount of protection against failure during earthquakes.

Many devastating earthquakes have been recorded in historical sources for this region of Iran. The instrumentally recorded earthquakes occurring in the region in this century are also numerous. However almost all of the more recent earthquakes have been of relatively medium magnitudes, causing little damage. Table 2.1 lists the $M > 6.0$ earthquakes either estimated or recorded in the last twelve centuries within a 200 km radius of the epicentre of the June 20 event [1].

2.2 MAGNITUDE

Different seismological centres around the world estimated the magnitude of the June 20 main event as between 7.3 and 7.7 on the Richter scale, including California, U.S.A. ($M=7.7$) and Aberdeen, U.K. ($M=7.4$). The Institute of Geophysics of Tehran University, however reported the magnitude as 7.3 on the Richter scale. Considering the proximity of this seismological centre to the epicentre of the event, this figure appears more plausible than the $M=7.7$ stated by California.

2.3 DURATION

The 20th June earthquake was a multi-shock phenomenon with two strong components being felt

List of Historical Earthquakes (M > 6.0) Within a 200 km Radius of Epicentre of Manjil Earthquake

Date				Location		Size
Year (AD)	Month	Day	Time	N	E	M
855				35.6	51.5	7.1
958	2	23		36.0	51.1	7.7
1119	12	10	18	37.7	49.9	6.5
1177	5			35.7	50.7	7.2
1485	7	15	18	36.7	50.5	7.2
1608	4	20	12	36.4	50.5	7.6
1678	2	3	6	37.2	50.0	6.5
1844	5	13	19	37.4	48.0	6.9
1879	3	22	4	37.8	47.9	6.7
1896	1	4	16	37.8	48.4	6.7
1905	1	9	6	37.0	48.7	6.2
1960	6	23	3	37.0	49.5	6.5
1962	9	1	19	35.7	49.8	7.2
1978	5	26	13	37.0	50.0	6.3
1978	11	4	15	37.7	48.9	6.1
1980	5	4	18	37.8	49.1	6.4

Table 2.1

separately at about 5 minutes interval in Tehran (some 200 km away). The two shocks may correspond to two separate events or they may be different components of the same event. However a period of 5 minutes appears too long for the latter to be the likely case. As a result, an established duration of the main event, as yet, has not been reported by the seismological centres.

2.4 FOCUS

The area shaken by the Manjil earthquake is extensive. However, the epicentral area where the extent of damage has been acute is relatively small. This points to a relatively shallow earthquake. The depth of the quake has been estimated by the Institute of Geophysics of Tehran University as about 10 km. Other estimates, in line with previous quakes in Alburz mountains, give shallow to medium depths of up to 30 km [2]. The relative shallowness of the earthquake accounts for the very high level of ground shaking in the epicentral area.

2.5 EPICENTRE

Reports from a number of seismological centres indicate that the epicentre of the Manjil quake was offshore, in the south-east corner of the Caspian sea. However, when considering the pattern of devastation relating to the intensity of ground shaking it becomes apparent that the location of strongest ground shaking and the ground ruptures associated with the quake was situated inland in the mountainous Rudbar region near the town of Manjil (some 150 km south of the reported epicentre). Bearing in mind the depth of the quake and the angle of the fault, it may well be possible that the focus of the earthquake was under the Caspian sea, however the main shock surfaced around its associated fault in the Alburz mountains. To avoid confusion, it should be noted that in the following the term 'epicentre' refers to the location of strongest ground shaking, rather than the point directly above the focus of the earthquake which may well be somewhere in the Caspian sea.

During author's visit, detailed seismological studies of the Manjil quake were underway by three Iranian research organisations, namely; the Institute of Geophysics of Tehran University (IGTU), The International Institute of Earthquake Engineering and Seismology (IIEES) and the Research Institute of Ministry of Housing (RIMH). Based on their initial findings, the IGTU considers town of Rudbar as the epicentre of the quake, whereas researchers from the two latter organisations believe the epicentre to be north of town of Manjil (36.75N, 49.40E), some 5 km south of Rudbar. The latter location appears more plausible, because, (i) ground ruptures associated with the recent event were discovered 300m north of Sefid-rud dam in Manjil and (ii) judging by the behaviour of buildings and even considering the different ground conditions in Manjil and Rudbar, ground shaking appeared more severe in Manjil than in Rudbar. The different opinions as to the exact location of the epicentre have arisen largely because of the multi-shock nature of the quake and the complex existing and new ground ruptures in the region. Also different ground conditions at various localities have made an isoseismic evidence of the epicentre more difficult.

2.6 INTENSITY

The Manjil earthquake was felt over an estimated 600,000 km^2 area (over 2.5 times the size of Great Britain). In Tehran, about 200 km south of its epicentre the quake caused widespread panic and some minor cracking in a number of tall buildings (I=V, on the MSK scale). There were however no reports of damage in Tabriz, 300 km north-west of epicentre where the quake was also strongly felt (I=IV). Other cities near the epicentre such as Zanjan (80 km) Gazvin (75 km) and Rasht (60 km) experienced stronger ground shaking with some damage to the weaker and taller buildings.

Detailed study of the ground intensity with a view to establishing the isoseismal map of the affected area is beyond the scope of this report. Investigators from the aforementioned Iranian research organisations have been involved in producing a definitive isoseismal map of the area and their findings are expected to be published in due course. A preliminary map published by the Ministry of Housing [2] is shown in Fig. 2.1. The present author's own observations support, to a large extent, this mapping. However the assignment of an intensity of I=X to the epicentre of the quake in Manjil is questionable. Although there was not a single building in Manjil without severe damage, nevertheless, the relatively poor standard of design and construction of the few semi-engineered and engineered structures meant failures at lower than expected levels of seismic loading. The intensity of ground shaking in Rudbar was evidently much less than that in Manjil. Considering the behaviour of some engineered structures such as two concrete and steel bridges, a few liquid storage tanks and a number of reinforced concrete

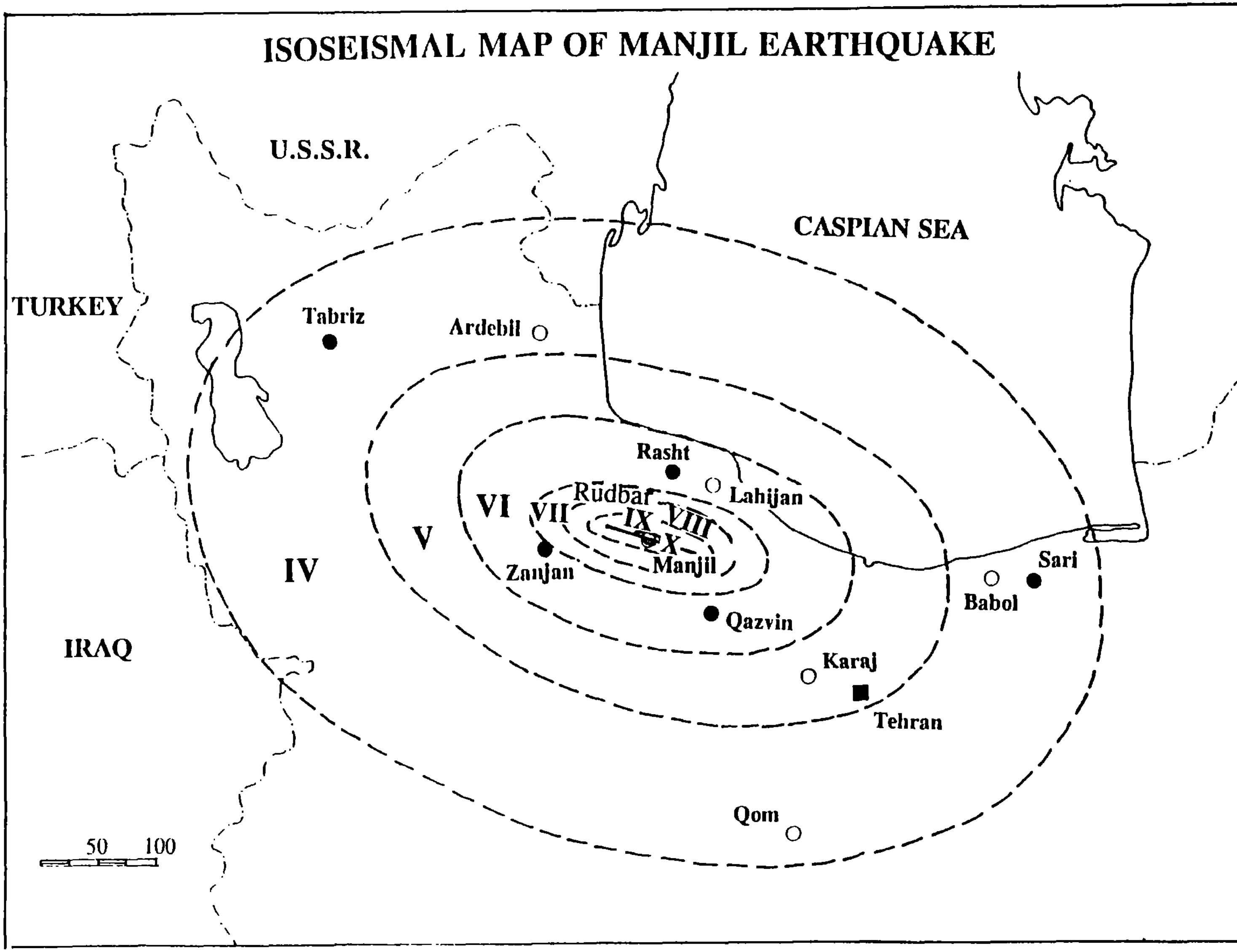

ISOSEISMAL MAP OF MANJIL EARTHQUAKE
U.S.S.R.
CASPIAN SEA
TURKEY
Tabriz
Ardebil
Rasht
Lahijan
Rūdbār
VI
VII
IX
VIII
X
V
IV
Zanjan
Manjil
Qazvin
Sari
Babol
IRAQ
Karaj
Tehran
Qom
50
100

Fig. 2.1

frame buildings, all of which survived without any significant damage, a lower intensity of ground shaking is indicated than suggested by Ministry of Housing's preliminary report.

2.7 GROUND ACCELERATIONS

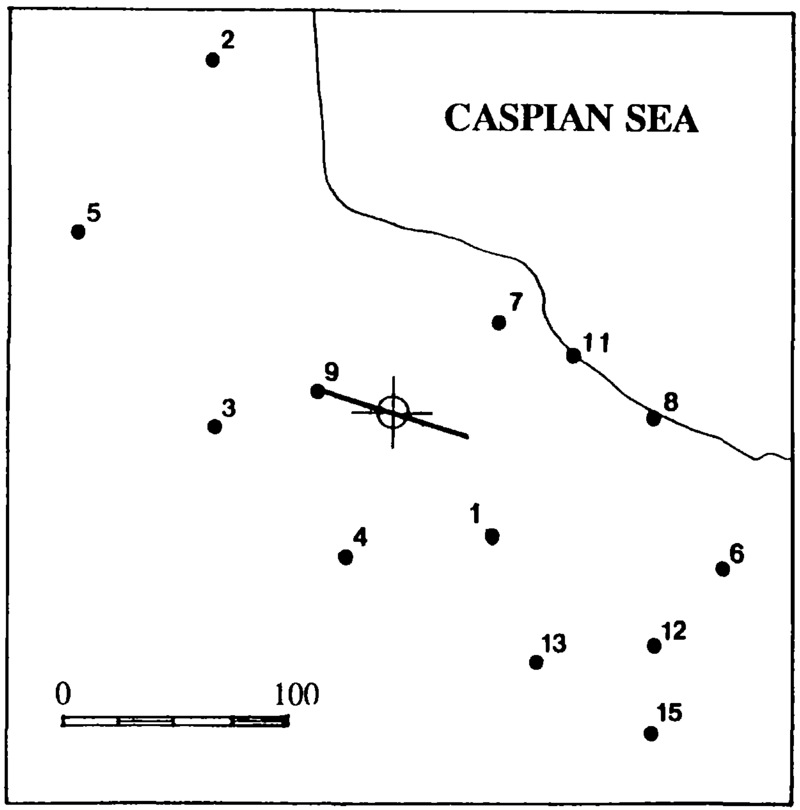
Although there are no direct records of the ground accelerations at the epicentre of the earthquake, a number of accelerographs situated within a 200 km radius of the epicentre recorded the ground accelerations of the main event. The preliminary readings from the nearest of these accelerographs at Abbar (situated near the Manjil fault some 40 km west of epicentre) revealed high maximum horizontal and vertical ground accelerations of 0.65g and 0.23g, respectively. One may safely assume accelerations in excess of the above figures for the epicentre of the quake around the Sefid-rud dam. Table 2.2 gives the list of preliminary readings from some of the operational accelerographs in the area (except those in Tehran) which recorded the main event [2]. A number of accelerographs in Tehran indicated maximum ground accelerations of between 2% and 3% of gravity.

2.8 FAULTING

As is already mentioned, the Rudbar region of Gilan province had been subjected to numerous strong earthquakes in historical times many of which were associated with rupture of the ground and faulting. A number of these recognised faults are noted in the geological and seismotectonic maps of the area. Of these the Rudbar fault runs in the east-west direction, the Masuleh fault runs north-south, intersecting the Rudbar fault and the Lahijan fault which runs in a southwest-northeast direction. The presence of this complex historical faulting system in the area suggested any probable faulting in the future to be associated with the existing faults and their intersections. However, as already mentioned, evidence of new faulting was found some 300m north of Sefid-rud dam in Manjil. This fault was traced by IIEES investigators running parallel to the Rudbar fault for a distance of 100 km from Abbar in the west to Jirandeh in the east and beyond.

The preliminary studies of the Institute of Geophysics and IIEES could not ascertain whether the ground ruptures associated with the 20th June quake are an extension of an hitherto undiscovered historical fault or an independent new fault. It is probable that the new ground ruptures are part of the complex Rudbar fault. There is evidence of a number of smaller intermediary ground

PEAK GROUND ACCELERATION RECORDED BY SOME ACCELEROGRAPHS



No	Instrument Location	Long. Comp. (%g)	Trans. Comp. (%g)	Vert. Comp. (%g)
1	Qazvin	19	8	9
2	Ardebil	2	9	2
3	Zanjan	10	6	6
4	Abhar	13	19	6
5	Mianeh	2	2	-
6	Gachsar	6	10	3
7	Lahijan	9	10	6
8	Tonekabon	11	6	3
9	Abbar	65	15	23
10	Kahrizak	3	2	2
11	Rudsar	7	5	6
12	Karaj	4	2	3
13	Eshtehard	6	7	5
14	Rudshur	3	3	1
15	Robat Karim	4	5	2

Table 2.2

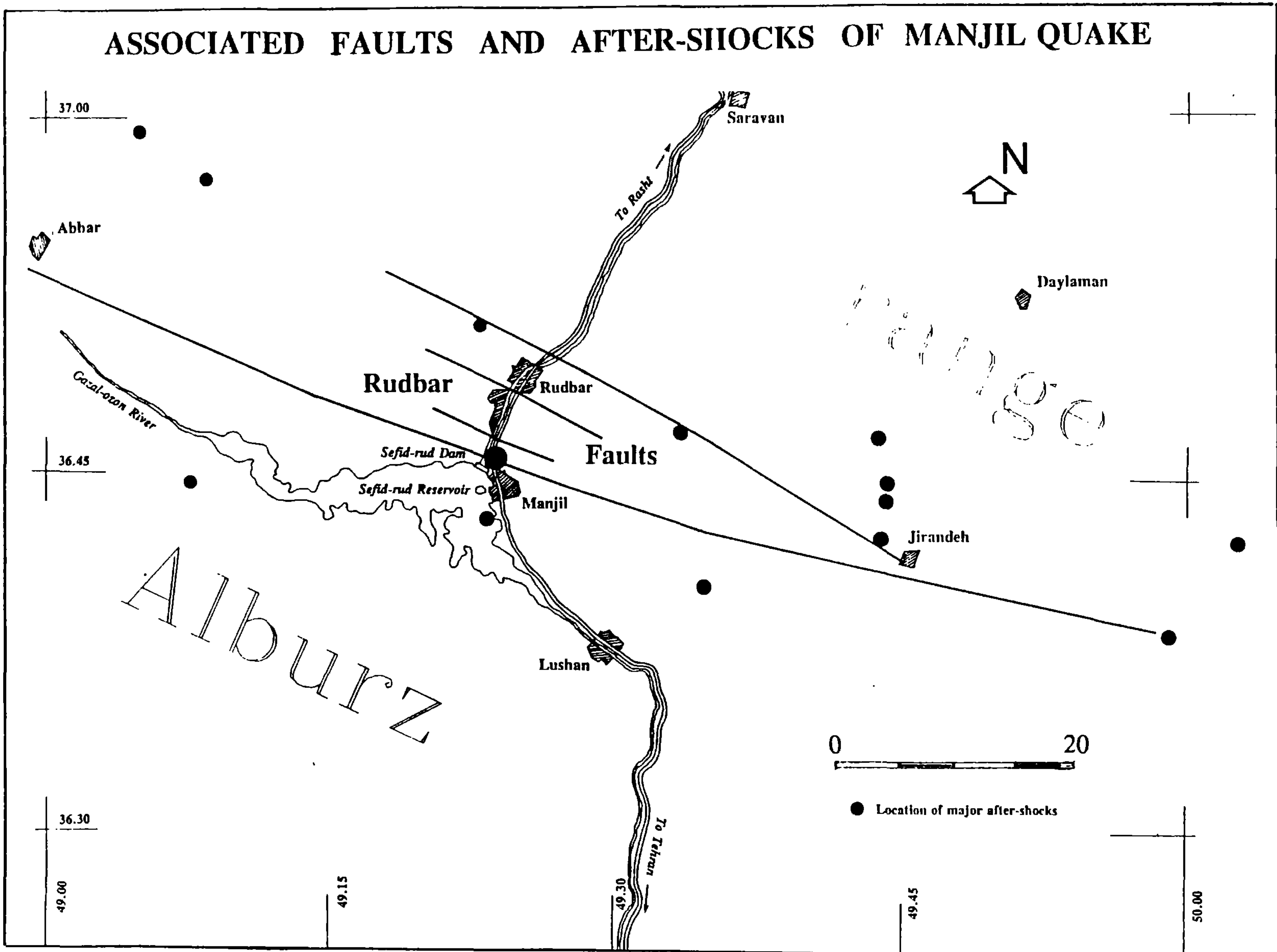

ASSOCIATED FAULTS AND AFTER-SHOCKS OF MANJIL QUAKE
37.00
Saravan
N
To Rasht
Abbar
Daylaman
Range
Rudbar
Rudbar
Faults
Gazal-ozon River
Sefid-rud Dam
Sefid-rud Reservoir
Manjil
36.45
Jirandeh
Alburz
Lushan
0
20
Location of major after-shocks
36.30
To Tehran
49.00
49.15
49.30
49.45
50.00

Fig. 2.2

ruptures between the old Rudbar fault and the new fault running north of Manjil (Fig. 2.2).

2.9 AFTER SHOCKS

The main event of 20th June was followed by numerous after shocks of varying magnitudes and strengths. At the time of the visit (40 days after the main event) these after shocks were still of an intensity to be clearly felt . The seismological centre of Institute of Geophysics of Tehran University recorded hundreds of these after shocks some of which were strong enough to be considered as medium to large size earthquakes. Figure 2.2 also illustrates the position of some of the after shocks (M > 5.0) dotted along the two main parallel faults.

2.10 GROUND FAILURES

a) Landslides: The Manjil earthquake caused many small and large landslides, blocking roads and damaging buildings. Further landslides caused by many after shocks hampered rescue operations and transportation. Although most of the landslides were small enough to be cleared and brought under control, there were however many large landslides which changed the topology of the area. Of these, one enormous landslide, not easily recognizable at first due to its shear size, was still threatening the town of Rudbar and its surviving buildings. The landslide measured a few kilometres in length.

b) Rock falls: Because of the mountainous nature of the stricken area numerous rock falls accompanied landslides in blocking the roads and destroying buildings. Evidence of secondary destruction of buildings due to rock fall could be seen in many instances in Manjil and Rudbar. One notable case was the complete destruction of a reinforced concrete guard-house at Sefid-rud dam (See section 5.1).

c) Soil liquefaction: Although no evidence of building failure due to soil liquefaction could be seen in the visited areas, there were reports of numerous cases of this type of failure further north in the Astaneh area and elsewhere.

Other phenomena usually associated with major earthquakes such as change in water table level and artesian phenomenon were also reported in many areas.

3 BEHAVIOUR OF SEMI-ENGINEERED BUILDINGS

3.1 INTRODUCTION

A large proportion of the residential buildings in the affected area were 'semi-engineered' buildings. These are buildings which although not designed to any particular seismic criteria, have certain earthquake resistant elements such as load-bearing steel frames or concrete ring-beams. It is difficult to categorize these buildings into definite types. However a number of general features are obvious. Buildings may have a rigid steel-framed roof (I-beam and jack-arch system) with or without a supporting concrete ring-beam. The ring-beam itself may be supported by concrete or steel columns or more often supported directly by load-bearing masonry walls. Alternatively the roof slab may consist of 'concrete beam-block' system, generally supported by a concrete ring-beam.

Because of the attraction of these two roofing systems (I-beams jack-arches and concrete beam-block) to the builders in Iran (due to their ease and speed of construction) it is appropriate to discuss them in more detail. Subsequently the behaviour of a number of semi-engineered buildings will be discussed with particular emphasis on some of their common forms of failure.

3.1.1 STEEL I-BEAM AND JACK-ARCH SYSTEM

This is a very popular method of roofing in Iran, parts of the Middle East and Eastern Europe. In the method a number of parallel steel I-beams are placed directly on the load-bearing walls spanning from one wall to the other. If a concrete ring-beam is included, the ends are simply supported on the beam. The distance between the two adjacent I-beams varies between 90 cm and 1.0 m. The space between the two neighbouring I-beams is then filled by a series of shallow brick arches (Fig. 3.1.a). The process is repeated until the whole roof is covered. A layer of lime mortar, mud or concrete is then placed on the brick arch and I-beams to create a flat surface. Another layer of reinforced clay mud 'kahgel', concrete slabs or bitumen on top of the slab forms the damp proof course. The slab is then plastered underneath to create a flat ceiling.

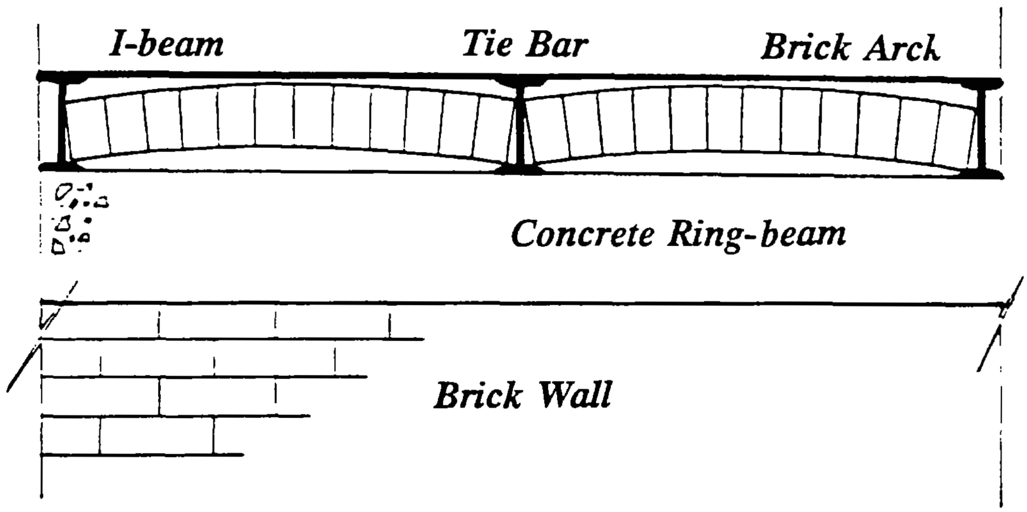
Roof slabs constructed in this way are stable under normal static conditions as the brick arches transmit the vertical load in compression along the arch to the supporting steel beams which in turn transmit the load to the walls. However, under dynamic loading the unrestrained, simply supported, I-beams tend to move apart resulting in the collapse of the brick arches. Restraining the I-beams by connecting them to transverse steel beams and the use of transverse steel tie bars across the span are two of the code recommendations for increasing the dynamic strength of the slab.

3.1.2 CONCRETE BEAM-BLOCK SYSTEM

In recent years another roofing method has been introduced which uses different materials and techniques but is similar in principle to the I-beam jack-arch system. In this method the steel I-beams are replaced by pre-cast reinforced concrete beams. The concrete beams are however placed more closely to each other at about 40cm apart. The gaps between the adjacent concrete beams are then filled with purpose-cast hollow concrete blocks. The concrete beams are T-shaped in cross section so that the concrete blocks can be supported on the bottom flanges (Fig. 3.1.b). The beam-block slab is then reinforced by the addition of a 7 to 10 cm thick in-situ reinforced concrete slab. In this way a reinforced concrete flat slab is formed without using scaffolding.

SEMI-ENGINEERED ROOFING SYSTEMS

a- Steel I-beam and Jack-arch Roofing



b- Concrete Beam-block Roofing

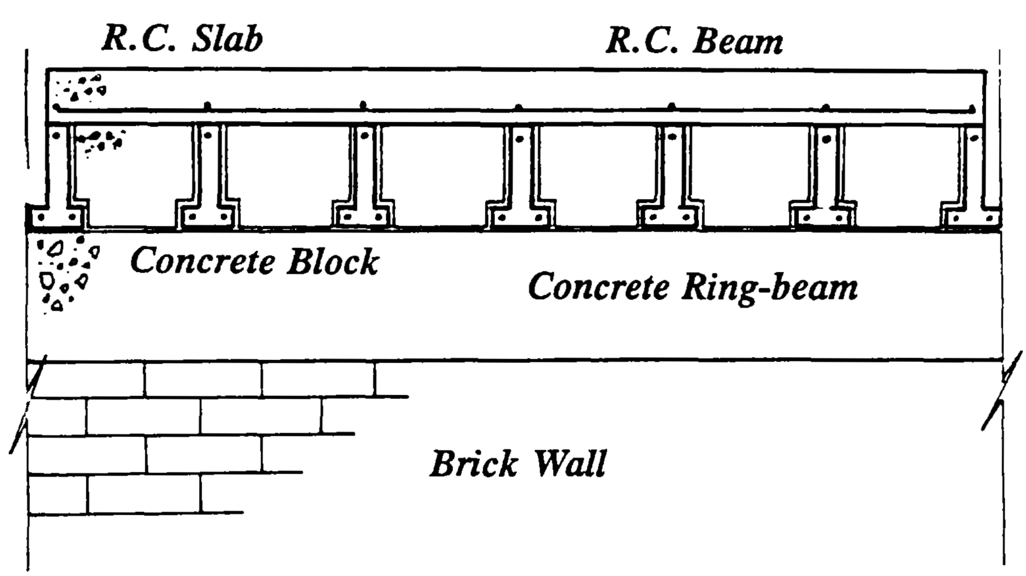


Fig. 3.1

3.2 CASE STUDIES

3.2.1 TWO-STOREY BRICK HOUSE, MANJIL

This is a two-storey un-reinforced masonry building with brick load-bearing walls. The main failure was in the long, transversely unsupported second floor south wall of the building with multiple out-of-plane bending cracks (Picture 3.1). Other failures include bending failure in parapet walls followed by overturning collapse of parts of those parapets, and separation of the perpendicular walls at their un-reinforced intersections.

Although the building is damaged beyond repair, the presence of the reinforced concrete ring-beams at first floor and roof level helped to keep the jack-arch floors in place.

3.2.2 TWO-STOREY STONE-CLAD RESIDENCE, MANJIL

The damage to this two-storey brick masonry house (Picture 3.2) was relatively less than its surrounding buildings. Apart from the collapse of parts of the unrestrained non-load-bearing wall of the second floor, the main load-bearing walls and the roof remained intact. No sign of failure could be seen in the load-bearing walls of the ground floor.

The main reason for the better behaviour of this masonry house appears to be that the stronger component of the quake coincided with the direction of the solid load-bearing shear walls of the ground floor (i.e. E-W direction, perpendicular to the view in Picture 3.2). If the strong component of the quake was in North-South direction, as there was practically no shear restraint in this direction, the building would have probably collapsed.

Another major failure in the building is the collapse of one of the second floor load-bearing walls (just visible in Picture 3.2). However, the presence of the reinforced concrete ring-beam has prevented collapse of the supported roof. Other failures could be seen in the free-standing front wall of the house in the form of flexural failure (vertical crack in the middle) and bending failure (long horizontal crack).

3.2.3 THREE-FOUR STOREY STONE-CLAD HOUSE, MANJIL

Despite its bizarre and complex architectural form, this masonry building maintained its integrity under the strong earthquake loading in Manjil (Picture 3.3). Its irregular architectural form however caused numerous failures in different forms. These include (Top down):

a. Bending failures at the short, top floor, brick column.

b. Shear failure in the wall of the top floor room.

c. Overturning of parts of the long unsupported parapet wall.

d. Multiple in-plane shear failures in the wider sections of the wall.

e. Multiple out-of-plane bending failures in the narrower sections of the wall.

3.2.4 WHITE STONE-CLAD HOUSE, MANJIL

This is another semi-engineered house which suffered heavy damage but did not collapse (Picture 3.4). It has a combination of reinforced concrete ring-beam and beam-block roofing system which again behaved as homogenous units at both the first floor and roof levels. The failures in the wall however were accelerated by the presence of large openings and possibly the failure of supporting ground.

An interesting feature of this building is the behaviour of door and window frames which, as was observed in a number of other instances, after the failure of the surrounding load-bearing walls or columns, became structural elements and acted as load-bearing supports. This observation supports the notion that if the concrete ring-beam were to be provided with auxiliary vertical supports such as steel or concrete columns, even in the event of complete failure of supporting walls these vertical supports would keep the roof in place, at least long enough for residents to escape.

The main failure at the ground floor is separation from the steel columns of the I-beams above the window frames (supporting the spandrel walls). The beams snapped at joints evidently under little dynamic bending or shear stress as the weak welds could not resist such stresses. After the joint failure the weight of the spandrel walls was transferred to the window frames causing them to buckle.

3.2.5 VALI-ASR HOSPITAL, RUDBAR

The construction of this hospital in Rudbar was completed in 1987. It consists of three main buildings with a combined built area of 4,000 m^2.

The hospital was constructed on the gradual slope of a hill and was largely founded on built ground. This may well have contributed to the severity of the ground shaking at the site. The main building has a combined steel and reinforced concrete frame (Picture 3.5). The columns are steel, however some reinforced concrete columns also accompany the steel columns at certain sections of the ground floor. The jack-arch slabs of the first floor and roof are supported by concrete ring-beams. The two smaller buildings of the hospital complex have no vertical steel or concrete elements and the load of the floor slabs and concrete ring-beams are directly transmitted through the load-bearing brick walls.

The northern section of the main building (Casualty Department) collapsed completely after the loss of it's load-bearing steel columns (Picture 3.8). The column supporting the south-east corner (the main entrance) had also failed and collapsed, bringing the first floor slab down with it. However, as is seen in Picture 3.6, the roof at this location survived despite the total loss of support at the corner. This behaviour is another example of the effectiveness of concrete ring-beams in maintaining the stability of the supported slabs.

The mode of behaviour of the building at this corner can be described as follows:-

> After the failure of the corner column, the concrete ring-beam of the first floor failed in cantilever action under the heavy load of the slab. The cantilever action of the roof however, was supported by the in-fill brick wall of the upper floor which, after failure of the corner column, acted as a load-bearing element and transmitted the load of the roof to the nearest surviving columns. The load path (as seen in Picture 3.6) is directly from the corner of the roof to the nearby column. That part of the wall which did not participate in the transmission of the load naturally collapsed under it's own weight.

The steel columns of those sections of the main building which did not collapse all failed in bending (Picture 3.7). The failure occurred mainly around the openings.

Two smaller buildings of the complex suffered heavier damage. One building completely collapsed on one side and was badly damaged on the other (Picture 3.9). The main load-bearing elements of these buildings were their un-reinforced brick walls.

Much of the concrete pavements in the complex had also suffered multiple cracking (Picture 3.10). Such cracking is indicative of the high intensity of the ground shaking and the failure of the built ground under the concrete slabs.

3.2.6 TWO-STOREY BRICK BUILDING, MANJIL

This semi-engineered brick masonry house which was damaged beyond repair, presents another good example of the vital role of the concrete ring-beams in maintaining the integrity of the floors and roof.

In the west side (Picture 3.11), although a large portion of the load-bearing wall has collapsed, a combination of ring-beam and concrete beam-block slab made the roof to act as a homogeneous unit, being kept in place by a minimum of surviving wall support. On the east side however the total destruction of the load-bearing walls of the upper floor resulted in the failure of the unsupported ring-beam and collapse of the roof.

A point to note is that the earthquake response of the concrete beam-block roofing system, as was used in this building appeared to have been much better than the steel I-beam and jack-arch system. The former system of roofing which has become increasingly more popular (partly because of the lower cost of concrete compared to steel) provides a stiffer and more homogeneous slab.

As in many other buildings in Manjil which escaped a complete collapse, the main load-bearing shear walls of this building run in the East-West direction which, as it happened, coincided with the direction of the strongest component of the quake. Shear failure in the long solid walls is therefore a common feature of these buildings. Such shear failures were clearly visible in the solid walls of the ground floor of this building (Picture 3.13). Apart from the associated diagonal cracking, the shear displacements are also clearly evident in these walls.

3.2.7 TWO-STOREY BRICK HOUSE, MANJIL

Another building which survived the worst effects of the quake was a two-storey brick masonry house. The building has no concrete ring-beams at the roof level. Large shear cracks in it's load-bearing walls indicate that the main force of the earthquake was resisted by these shear walls. Also, the steel I-beams of the jack-arch roof were supported at the ends by transverse steel I-beams which accounts for the good behaviour of the roof.

One serious fault in the lay-out of this house which caused serious damage was the presence of chimney flues inside the load-bearing walls (Picture 3.14). This weakened the walls by effectively dividing them into a number of wide brick columns susceptible to bending and flexural failures and incapable of transmitting in-plane shear forces.

3.2.8 TWO STOREY STEEL-FRAMED BUILDING, RUDBAR

The steel frame of this building (Picture 3.15) is a good example of building construction by instinct, carried out by self taught builders rather than engineers, designing according to engineering principles.

The steel truss beam appears to have successfully transmitted the roof load down to the columns. However, there are only two rather slender columns approximately 12 m apart, supporting the load. Although the steel frame has kept the jack-arch roof in place, the long span caused the truss system to fail in flexural bending.

The large in-plane and out-of-plane response displacements of the steel truss system also caused interaction between the flexible steel and the brittle in-fill brickwork, resulting in the failure and disintegration of the latter.

3.2.9 TWO-STOREY STEEL-FRAMED, JACK-ARCH ROOF BUILDING, RUDBAR

An example of a cantilever action of the upper floor due to lack of shear resisting elements may be found in the behaviour of this two-storey shop in Rudbar (Picture 3.17). The mode of failure of this building may be described as follows:

> Under the east-west component of the quake, the weak brick walls of the upper floor (north and south sides) failed in bending and shear and collapsed. After the loss of shear resisting walls the inadequate steel columns could not support the cantilever action of the roof and failed in bending. The inertia force of the now unsupported roof then caused it to be thrown westwards into the pavement, bending further the western columns under it's heavy weight (Picture 3.18).

An important point to note in the behaviour of this building is the effectiveness of the steel tie-beams (supporting the ends of the I-beams) in preserving the integrity of the brick arches. Despite heavy ground shaking and the subsequent heavy fall of the roof under it's own weight, the jack-arches remained, for the most part, in place and did not disintegrate.

Despite the complete failure and collapse of the upper floor, the ground floor escaped with little damage. The collapsed one-storey building on the south side probably helped to contain the ground floor of this building.

3.2.10 THREE-STOREY HOUSE, MANJIL

This 3-storey house (including the basement) appeared, at first, to have escaped the earthquake with only little damage visible from outside (Picture 3.19). However, the extent of damage became clear when visiting the interior of the building. Parts of the first floor had collapsed and non-load-bearing partition walls of both the first and second floors had failed and partially collapsed.

It is interesting to note that the brick jack-arches collapsed despite no apparent differential movements of the supporting steel I-beams (Picture 3.20). Similar failures were observed in a few other 2-storey buildings in Manjil and Rudbar. This behaviour indicates certain local amplifications in the response of the fixed-end, I-beams of the middle floors under earthquake loading.

3.2.11 GOVERNMENT BUILDING, RUDBAR

This two-storey semi-engineered building in Rudbar (Picture 3.21) consists of load-bearing brick walls and steel I-beam jack-arch slabs, supported by concrete ring-beams placed directly on the walls. There are however four reinforced concrete columns at the four corners of the building and one steel column supporting the relatively large span of the floor in the middle of the building. The I-beams of the first floor and the roof are restrained at their ends by transverse steel beams. This composite of steel and concrete created a relatively strong frame for the building which is the reason why it was still standing.

In the visible section no failures could be seen in either the concrete beams and columns or the steel work. However, the walls were heavily damaged and the brick jack-arches of the first floor had completely collapsed in many sections (Picture 3.22). The brick arches of the roof, on the other hand, remained intact. This behaviour, as is already noted in case study 3.2.10, can be attributed to the out-of-plane vibration of the floor slab which, as it's edges are placed between the ground floor and first floor walls, acted as a fixed-side slab. The brick arches were incapable of out-of-plane bending and therefore crashed and collapsed. On examining the fallen debris of the brick arches it was also noted that weak lime mortar and poor quality bricks had been used which did not help the resistance of the brick arches. The roof slab however is simply supported. As a result it was capable of displacing horizontally. This reduced the local vibration of the steel frame hence the bending forces exerted on the brick arches.

The load-bearing walls of the ground floor and first floor failed in different modes. This is clearly evident in the west side of the building (Picture 3.21). The strong E-W component of the earthquake caused bending failures (horizontal cracks) in the relatively narrow walls between openings of the upper floor, whereas similar walls at the ground floor failed in shear, presumably due to the force of the N-S component of the quake. There are identical modes of failure in the east wall of the building. The interior load-bearing walls also followed the same pattern. There was extensive diagonal (shear) cracking in the interior walls of the ground floor mainly in the N-S direction but smaller horizontal cracking of a bending type could also be seen in the walls of the upper floor. The explanation for this complex but consistent mode of failure may lie in the fact that the building was probably subjected to strong ground shaking in both E-W and N-S directions. The N-S component could have caused the shear failures in the ground floor walls running in that direction (and bending in the north and south walls) whereas the E-W component could have resulted in bending failures in the east and west walls of the upper floor (and shear in the north and south walls).

3.2.12 TWO-THREE STOREY STEEL-FRAMED HOUSE, RUDBAR

The steel frame of this two storey house behaved well and despite it's apparent lack of shear resistance in N-S direction, the presence of surrounding buildings helped to contain the structure. However, the damage to the upper section of the stair case (see Picture 3.23) was caused by the bending failure of vertical steel columns acting as cantilevers under the heavy load of the roof. As is seen in picture 3.23, there are no horizontal restraints for the slender steel columns.

3.2.13 TWO STOREY STEEL FRAME CIRCULAR RESIDENCE, RUDBAR

In the design of this two storey steel-framed building the earthquake forces were evidently not taken into consideration. The unsymmetrical architectural lay-out and the absence of any effective shear resisting elements in the upper floor resulted in complete collapse of the central section of the building and bending failures in the steel columns of its circular wings (Picture 3.24)

3.3 GENERAL OBSERVATIONS

3.3.1 FOUNDATIONS

- The majority of the foundations in the buildings discussed are strip footings made of 'shefteh' (lime-gravel). Occasionally the shefteh footing was supplemented by a concrete layer of varying depths. Ordinary concrete foundations were however not uncommon.

- In many cases the unfavourable ground condition was the main cause of foundation failure. This was clearly demonstrated in the case of the Rudbar hospital (Picture 3.10) in which failure of loose fill under ground shaking resulted in failure of the concrete strip foundations in the north and east sides of the building. Due to their location on the hill side, cut and fill built ground was common under the foundations of many buildings in Rudbar. This differential support condition was responsible for the partial collapse of a number of buildings in Rudbar.

3.3.2 WALLS

The failure of un-reinforced load-bearing masonry walls and the subsequent collapse of the roof was the most common form of failure in buildings throughout the affected area. Walls of the buildings were generally load-bearing, solid or perforated brick walls. However, in a number of buildings hollow concrete blocks were also used. The following short comings contributed to the failure of the walls.

- Poor quality of brick and mortar was evident in many of the collapsed walls. Mortar used in the brickwork was generally poor quality cement or lime mortar. In many cases the wall had collapsed in a heap of individual bricks separated cleanly from each other.

- Lack of mortar in the vertical joints of brickwork was another point of weakness of the walls. This results from the habit of laying the bricks on a bed of mortar and covering the newly laid course with another layer of mortar without paying much attention to filling the gaps in the vertical joints (Picture 3.11). The presence of these gaps and voids in vertical joints greatly reduces the out-of-plane flexural stiffness of the brickwork, hence reducing the overall strength of the wall.

- Flexural and bending failures were observed mainly in the long unrestrained masonry walls (Pictures 3.1 and 3.2). Neither the roof, nor foundations or the intersecting walls provide sufficient fixity for the brick walls, hence in the case of longer walls their resistance to the out-of-plane forces is much reduced.

- The strength of a wall depends mainly on the size of openings in that wall. Failures in walls containing large doors and windows were commonplace in the damaged buildings. The presence of such openings had effectively divided the load-bearing walls into a number of wide un-reinforced brick columns or spandrel walls unable to transfer or sustain any shear or flexural stresses. Examples of this weakness may be seen in Pictures 3.3 and 3.4. On the other hand, as for example in the case of the one-storey Rudbar post office (to be discussed in section 4), low opening to wall ratio resulted in a much stronger wall capable of resisting both shear and flexural stresses.

- The presence of chimney flues, within walls also created discontinuity and reduced the wall's shear and flexural resistance (Picture 3.14).

- Lack of proper connections between intersecting load-bearing walls is a serious point of weakness for a building. Such discontinuities are caused by the habit of building walls one at a time (for economy of scaffolding) leaving inadequate brick notches at the intersection points.

3.3.3 CONCRETE RING-BEAMS

Provision of a concrete ring-beam under the floor and roof slabs is considered by the Iranian seismic code as a minimum requirement for semi-engineered buildings. The function of the ring-beam is to provide additional support for the composite, often fragile, slab so that if parts of the load-bearing walls fail under the earthquake loading a homogeneous support such as a ring-beam could provide temporary support for the slab.

This code requirement was observed in some of the buildings investigated in the epicentral area. In most cases the ring-beams acted exceptionally well under the earthquake loading and kept the supported floor slabs and roofs in place. Good examples of their behaviour were seen (as already discussed) in Rudbar hospital (Picture 3.6) and the two-storey brick house in Manjil (Pictures 3.11 and 3.12). The ability of the ring-beam to re-distribute the load over whatever vertical support remains available was demonstrated in both cases. In Rudbar hospital after the collapse of load-bearing steel columns at the south-east corner of the main building, the weight of the roof was transmitted through a section of an originally non load-bearing wall to the nearest standing column. In the case of the brick house in Manjil, although the majority of the load-bearing west and south walls of the upper floor had failed and collapsed; a very small section of the wall left standing provided sufficient support to stop the collapse of the roof. It should be noted that in both cases transmission of the load via the ring-beams to the surviving wall sections increased the compressive stresses in the sections, which in itself helped their stability under further ground shaking. In another case, after the collapse of load-bearing steel columns, a window frame acted as a load-bearing element keeping the roof in place.

- The above-mentioned behaviour emphasises the importance of providing secondary floor support systems in masonry buildings which do not fail before the main load-bearing elements, and are activated after such failures. These support systems could be in the form of reinforced concrete or steel columns.

- An improvement to the strength of the ring-beam would obviously be achieved if a better quality concrete were used. In some cases collapse of the slab and the supporting ring-beam could be attributed to the poor quality of concrete and poor reinforcement in the ring-beam.

3.3.4 STEEL FRAMING

Although steel is an expensive commodity, it's use in the construction of houses and small commercial units has become increasingly more popular. The main reason for their popularity is the speed of erection and the somewhat false belief that the use of steel in any form makes the buildings stronger. The steel framing is usually carried out by ordinary builders using rule of thumb, with the size of sections being dictated by availability rather than proper engineering design procedures.

The main weakness of the majority of these steel framed buildings is in the welded joints. The welds were so weak that many of the joints snapped at the onset of the earthquake loading causing disintegration of the frame. The early failure of the joints could be deduced from the fact that the steel sections themselves showed no signs of failure. The state of welding in steel framed buildings is discussed further in section 4.

Inadequate footing under the columns is another structural weakness in most steel-framed buildings. It is common practice to cast the foot of the steel columns in concrete inside an empty oil barrel. This inadequate foundation, as is obvious, is not capable of providing the necessary support for the steel columns.

3.3.5 I-BEAM, JACK-ARCH SLABS

- Collapse of the unrestrained I-beam jack-arch roofs alone was responsible for the majority of fatalities in towns and larger villages. As observed in so many previous earthquakes in Iran, under horizontal ground vibrations the steel I-beams moved apart, causing the collapse of brick arches. This happened despite the ability of the load-bearing walls and ring-beams to resist the earthquake loading and escape failure.

- Restraining the ends of the I-beams with transverse beams or the use of steel tie-bars have long been recommended as ways of increasing the earthquake stability of such roofs [3]. These recommendations were adopted in many buildings with the beneficial results. Examples of the effectiveness of the above restraining methods were discussed in case studies (Pictures 3.16 and 3.17). In two cases, although due to the failure of steel columns and walls the roof had collapsed, it had however acted as a rigid slab with brick arches, in most parts, still in place.

- As is already mentioned, an interesting observation was made in a number of two-storey buildings having I-beams jack-arch system of floors and roofs. In these buildings the roof slab survived the earthquake without damage whereas the brick arches of the first floor slab had all collapsed with no apparent movement in the supporting I-beams. This different behaviour of slabs in the same building can be attributed to the fact that the roof slab is simply supported, capable of rigid horizontal movements under ground shaking with little out-of-plane flexural response. In the floor slab however, the I-beams are fixed at their supported ends. This results in local out-of-plane flexural vibration of the slab I-beams. It is clear that the brittle brick arches are unable to participate in such vibration hence the interaction between the flexible steel and the rigid brick arches results in the failure of the latter. Such composite slabs are therefore unsuitable for fixed floors where heavy local vibrations of the floors are likely under earthquake loading.

3.3.6 CONCRETE BEAM-BLOCK SLABS

. In general the behaviour of concrete beam-block floor and roofing systems was much better than their I-beam, jack arch counterparts. No cases were encountered where a concrete beam-block slab had failed and collapsed while the supporting walls or columns remained in place.

. The concrete beam-block slabs collapsed only when a large portion of the supporting walls or columns had been destroyed and even then the collapsed floor in many cases kept it's integrity and remained as a solid slab. A combination of concrete ring-beam and concrete beam-block slabs therefore appear to provide a suitable method of roofing for earthquake prone areas provided that the supporting walls and columns could also maintain their load-bearing capacity during an earthquake.

. For buildings of two-storey or higher however, the behaviour of a fixed-sided floor slab would be governed by the level of interaction between beams, concrete blocks and the concrete slab during local out-of-plane vibration of the floor. Although such interactions would be far less than is the case in steel I-beam, jack-arch slabs, care must be taken in reinforcement detailing of precast concrete beams and their positioning during construction phase. A few cases were noted in which parts of the supporting flange of the T-shaped beams had failed allowing the loose concrete blocks to fall. Closer examination of the failed concrete beams revealed lack of reinforcement in the critical flange section of the T-beams.

- As in other concrete elements seen in many buildings in the area, there is scope for improving the quality of concrete in the precast T-beams and the in-situ concrete slabs.

3.4 CONCLUSIONS

1. The underlying ground condition affect the dynamic behaviour of a building and determines the level and frequencies of earthquake loading to which the building is subjected. Differential stiffness of supporting soil (mainly as a result of cut and fill ground formation) was responsible for damage to many buildings in Rudbar.

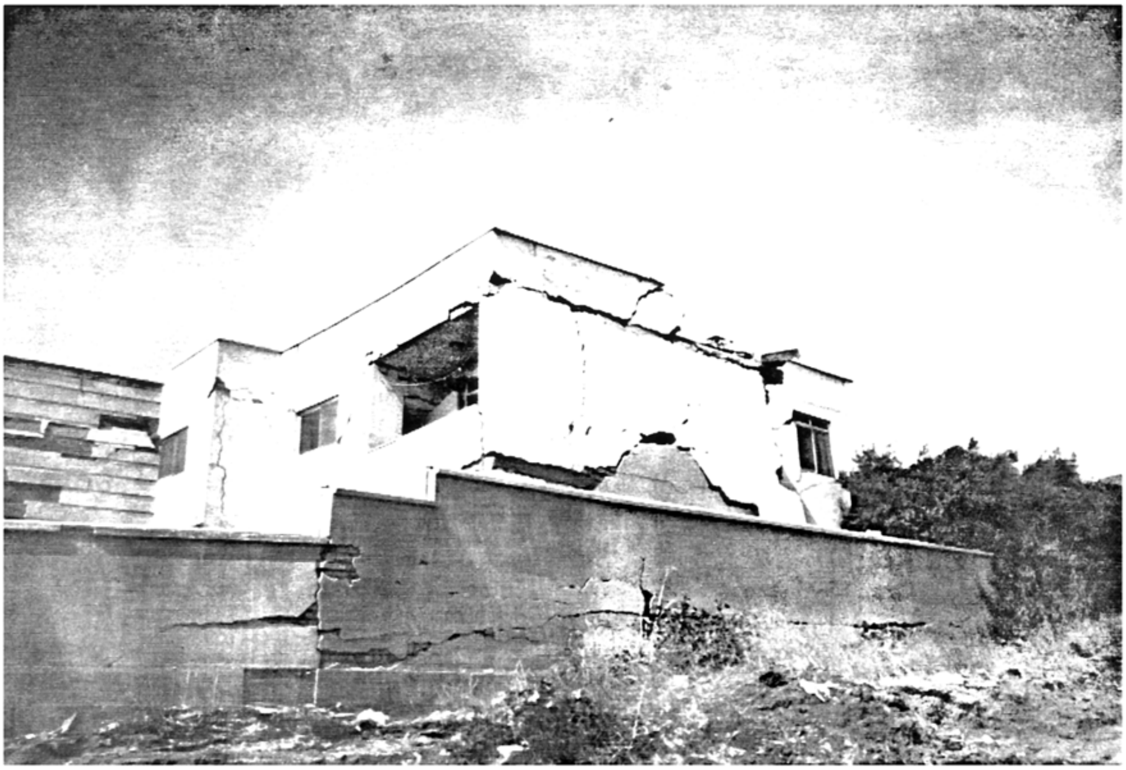
2. Poor material quality and workmanship together with inappropriate architectural lay-out were the main causes of failure in the load-bearing and in-fill masonry walls. The use of better quality bricks and mortar, providing mortar in vertical joints of brickwork, reducing the size and number of openings in walls and avoiding long unrestrained walls can greatly improve the strength of the un-reinforced masonry walls.

3. Concrete ring-beams proved to be useful elements in mitigating the collapse of floors and roofs.

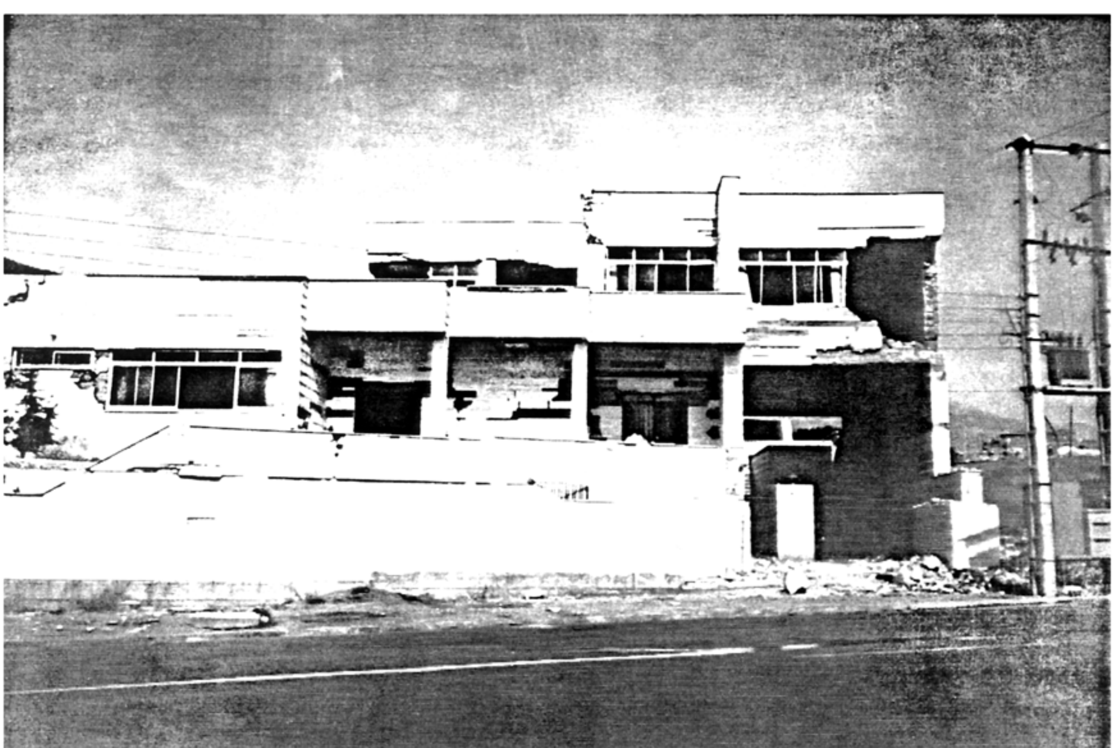
4. Poor quality of welding, inadequate steel sections and lack of proper foundations have placed the common steel-framed small building in Iran at serious risk for even medium sized earthquakes. Unless such practices can be changed and appropriate steel sections with bolted joints are designed and constructed by qualified engineers and builders, the use of steel frames for low-rise ordinary residential and commercial buildings should be discouraged. It should be pointed out that the cost of an equivalent reinforced concrete frame is probably less than the steel frame.

5. Steel I-beam jack-arch slabs survived when the I-beams were fully restrained at ends by transverse beams and along the span by steel tie-bars, but only when simply supported as a roof.

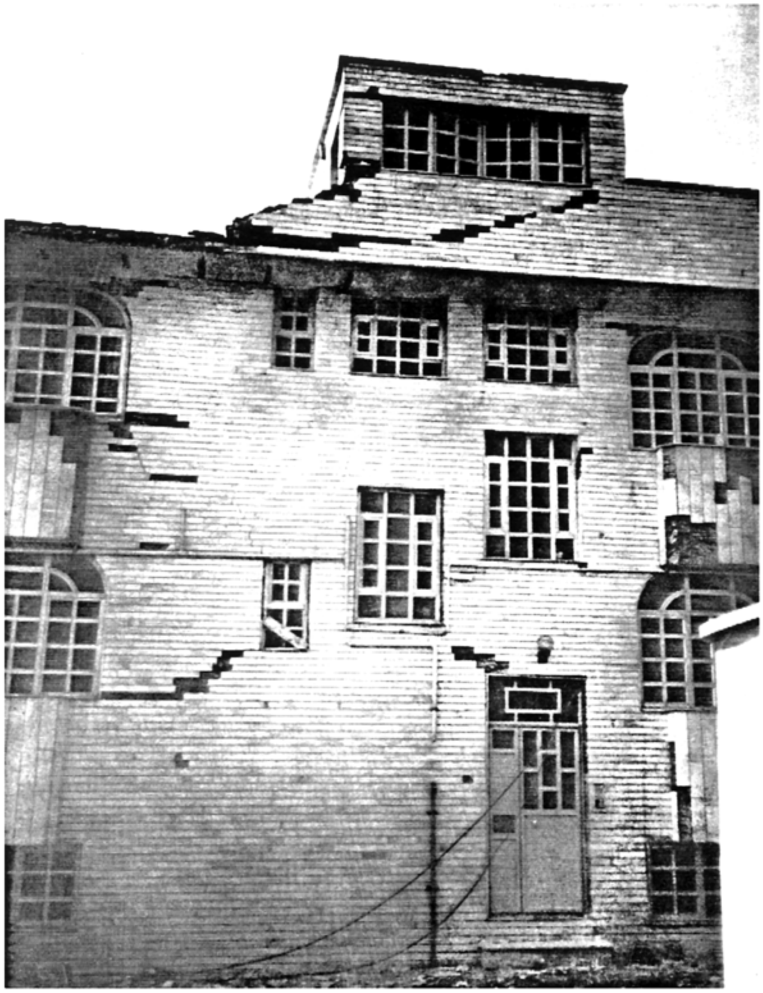
6. The better behaviour of concrete beam-block slabs was due mainly to their ability to act as a homogeneous unit. Reduced interaction between the concrete beams and blocks (as compared with the interaction between steel I-beams and brick arches) also helped the ability of the composite slab to withstand local vibration.



Picture 3.1 Two-storey Brick House, Manjil
Multiple out-of-plane bending failures in the wall



Picture 3.2 Two-storey Residence, Manjil
Collapse of the non-load bearing second floor wall



Picture 3.3	3-4 storey House, Manjil
Despite many failures due to its complex architectural form the building maintained its integrity

Picture 3.4	White Stone Clad House, Manjil
Window frame supporting the spandrel wall and the roof after failure of load-bearing steel frame

Picture 3.5 Vali-asr Hospital, Rudbar
Damage to the entrance of the main building

Picture 3.6 Vali-asr Hospital, Rudbar
excellent behaviour of roof concrete ring-beam after collapse of main load-bearing elements at this corner

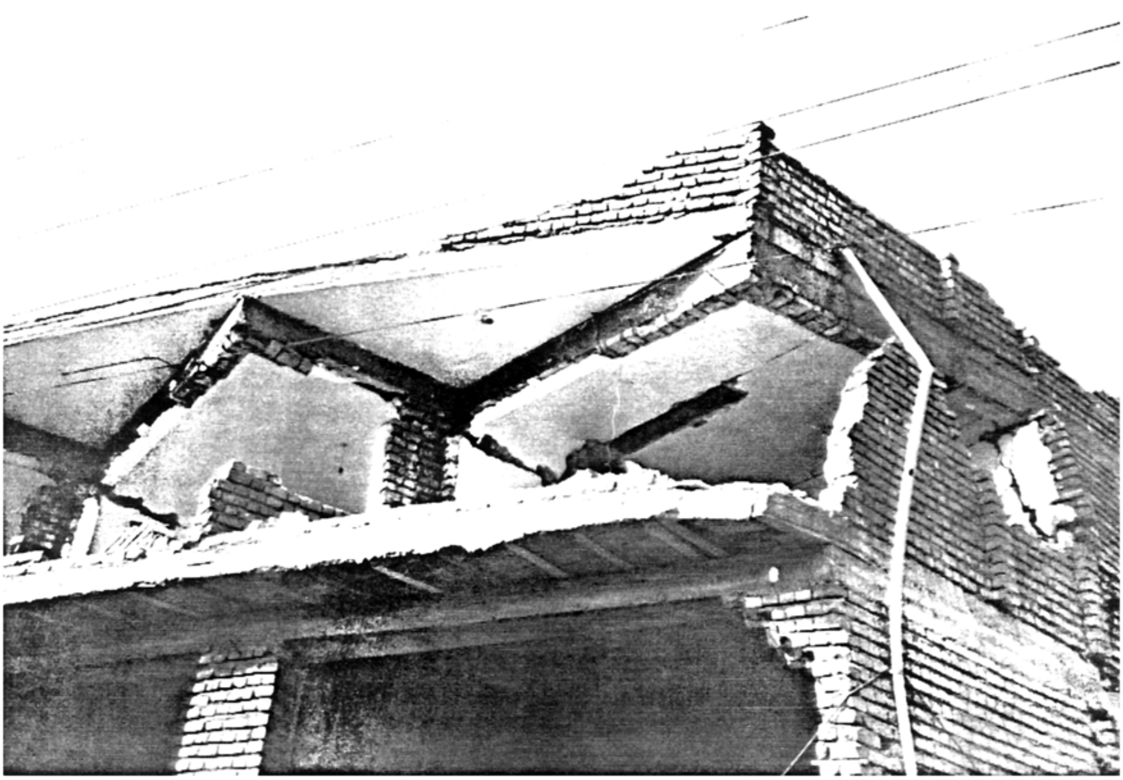
Picture 3.7 Vali-asr Hospital, Rudbar
Multiple fixed-end bending failures in columns of the main building

Picture 3.8 Vali-asr Hospital, Rudbar
Destruction of casualty department due to weak vertical supports

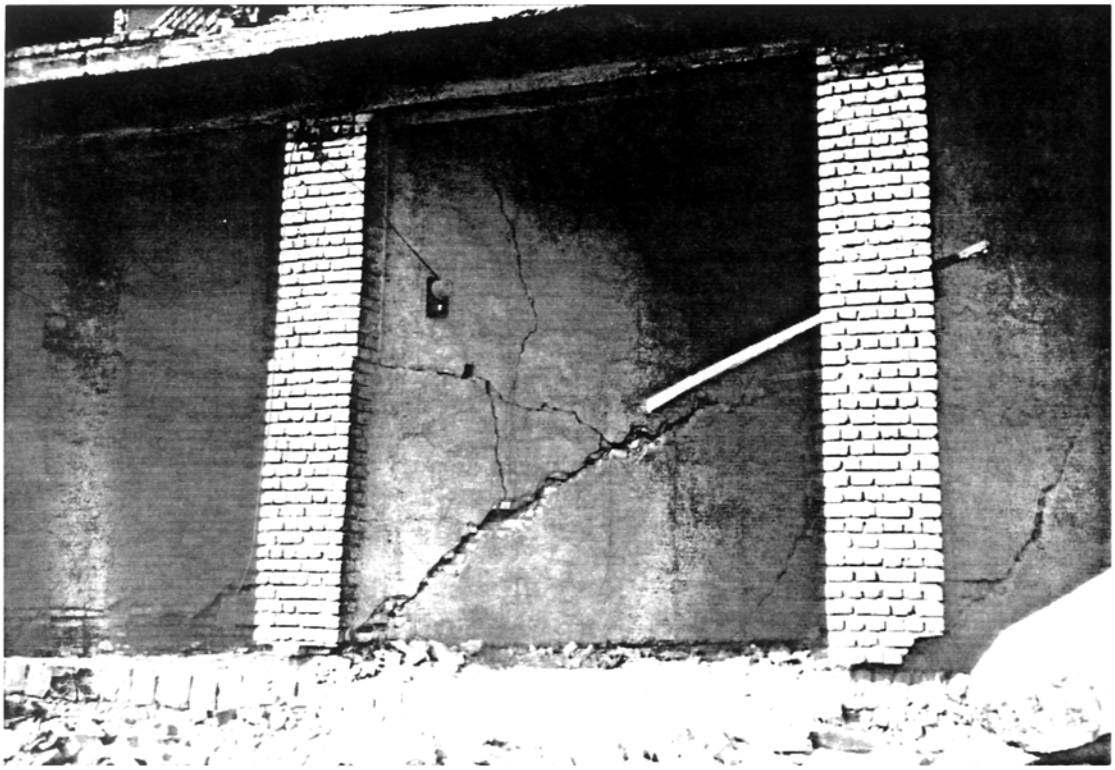
Picture 3.9 Vali-asr Hospital, Rudbar
Damage to the middle one-storey building

Picture 3.10 Vali-asr Hospital, Rudbar
Failure of built ground and the pavement concrete slabs

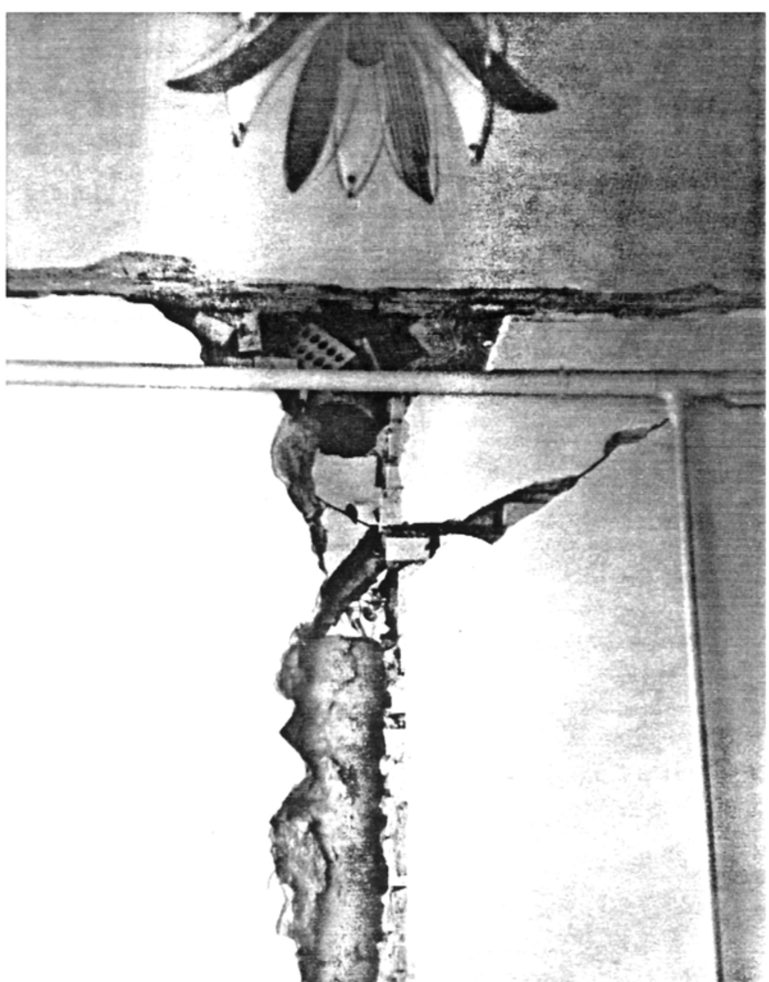
Picture 3.11 Two-storey Brick Building, Manjil
Collapse of the ring beam and roof of the east side
due to the complete loss of vertical support



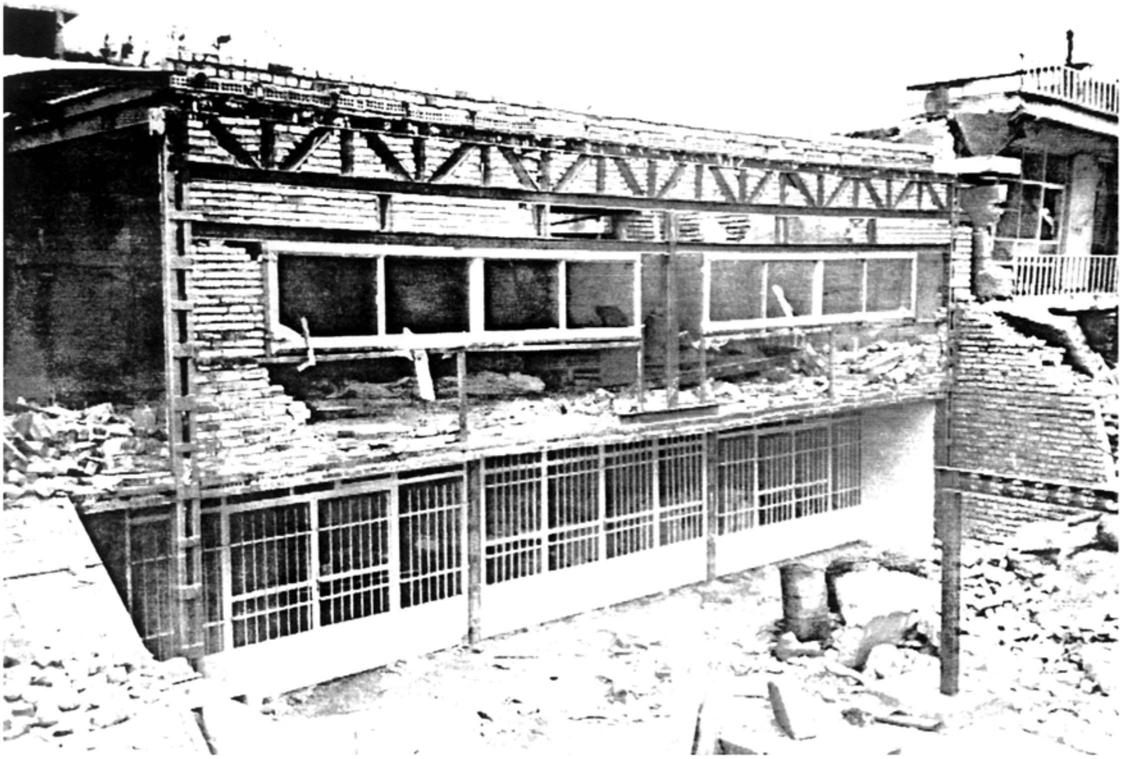
Picture 3.12 Two-storey Brick Building, Manjil
Despite loss of load bearing west wall elements
the concrete ring beam has kept the roof in place



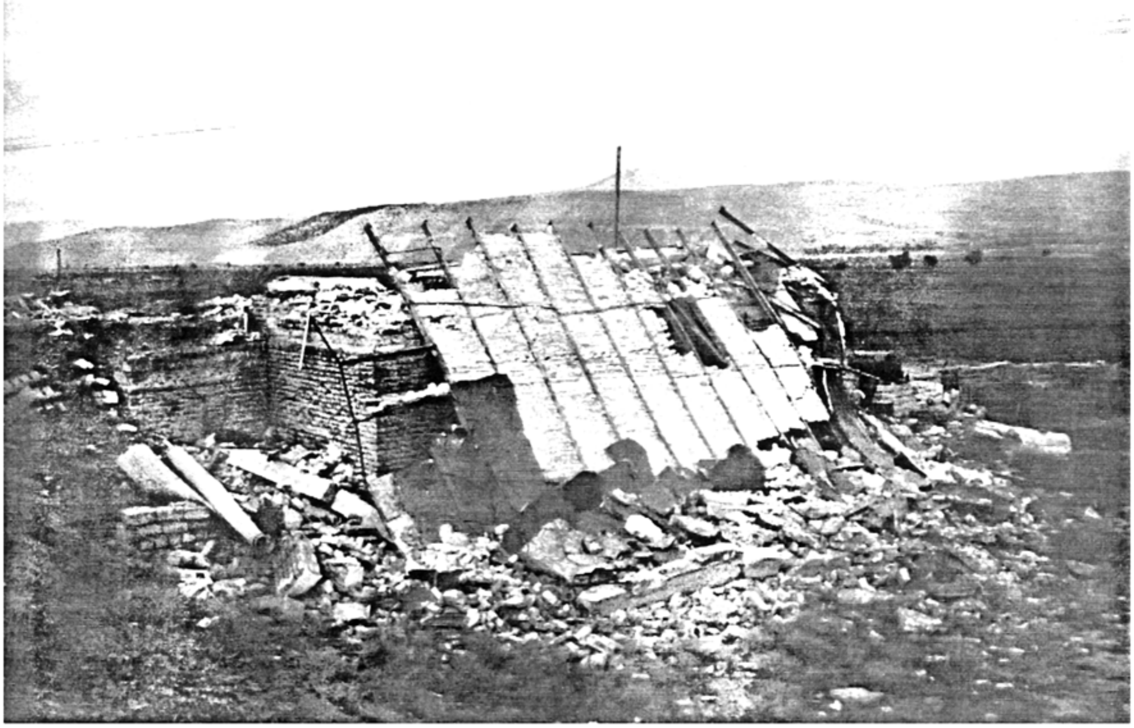
Picture 3.13 Two-storey Brick Building, Manjil
Shear failure in the ground floor walls



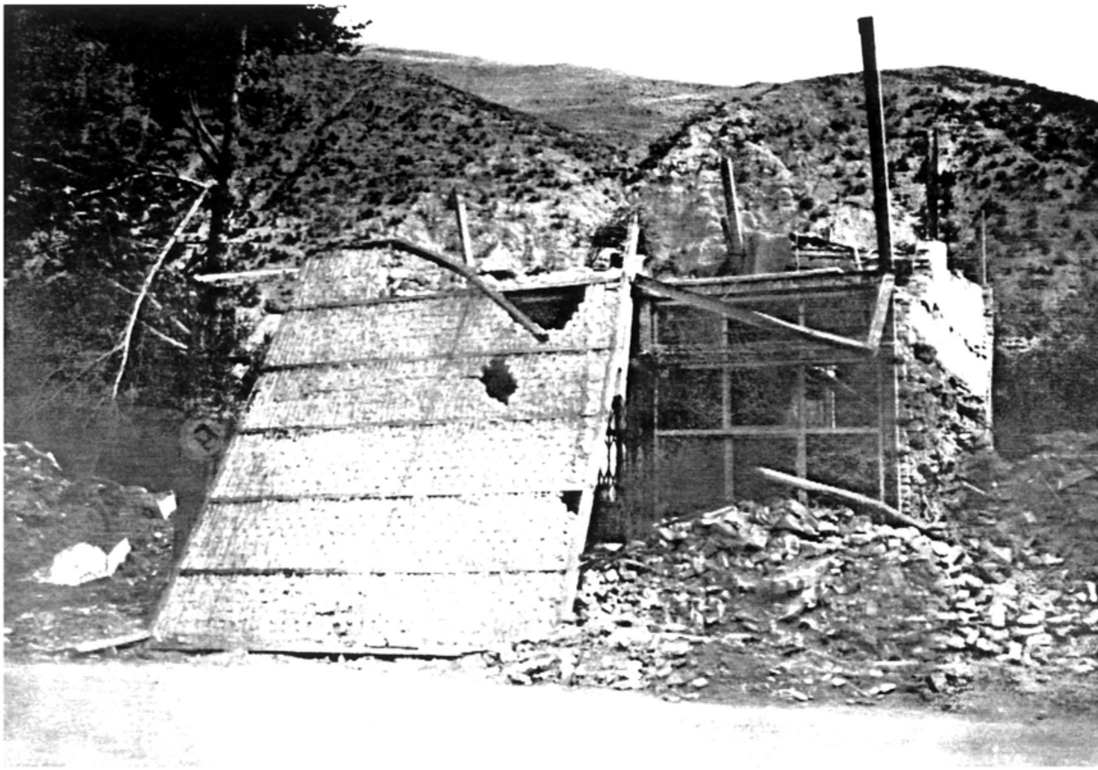
Picture 3.14 Two-storey Brick House, Manjil
Passage of chimney pipe through the wall has contributed to the weakness of the wall



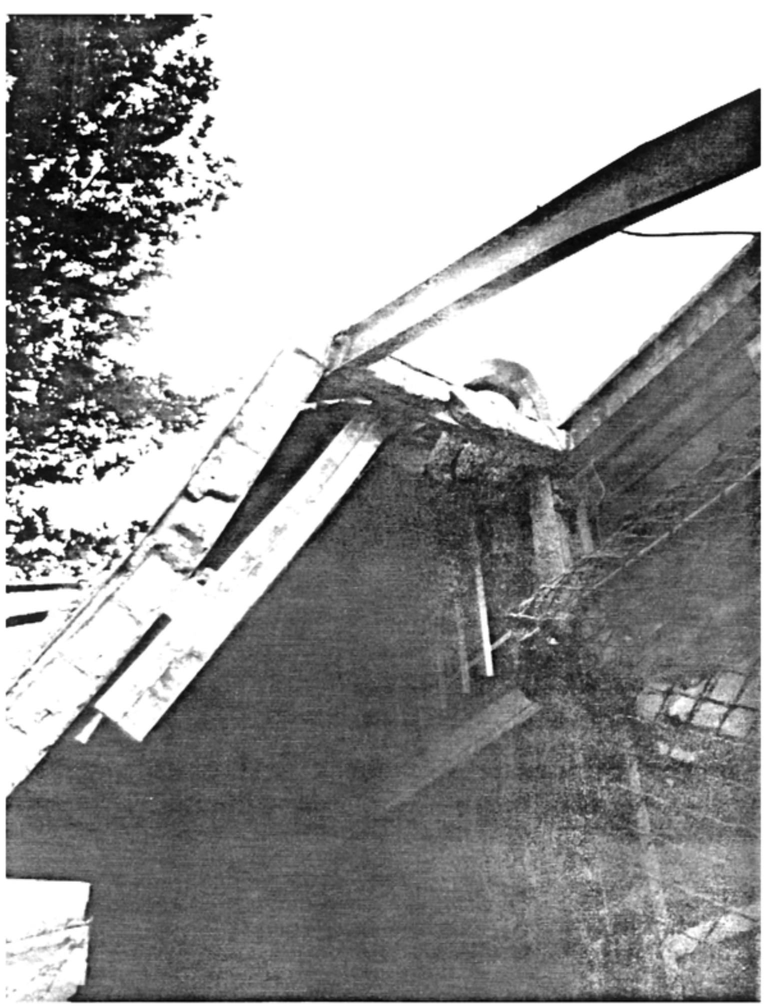
Picture 3.15 Two-storey Steel Frame Building, Rudbar
Long unsupported span of the steel frame sagging
under the load of the quake



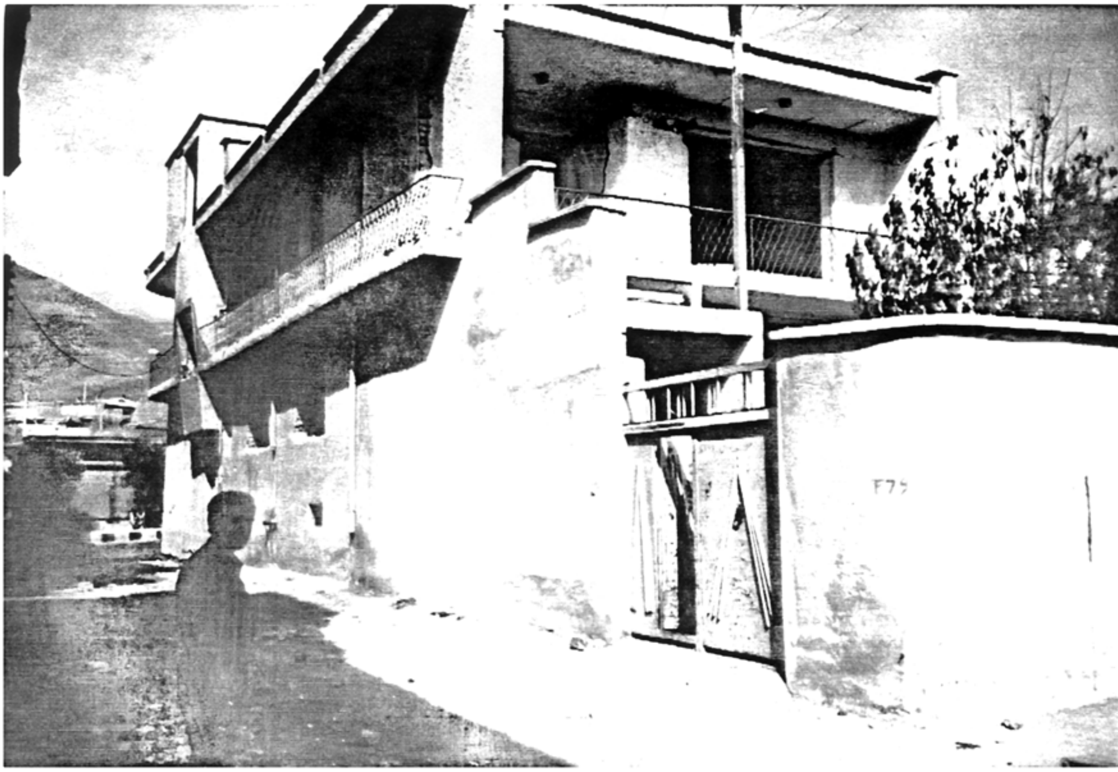
Picture 3.16 Two-storey Jack-arch Brick Building, Near Rudbar
Collapse of the roof due to inadequate vertical support but
steel bars keeping the I-beams and jack-arches together



Picture 3.17 Steel Frame Jack-arch Roof Building, Rudbar
Despite ground shaking and free fall under its own weight the supported I-beams of the roof slab kept the jack arches in place



Picture 3.18 Steel Frame Jack-arch Roof Building, Rudbar
Excessive bending of upper floor column under the weight of the roof



Picture 3.19 Three-storey Building, Manjil
Exterior of the house shows little sign of damage

Picture 3.20 Three-storey Building, Manjil
First floor jack arches collapsed despite no apparent separation of the supporting I-beams

Picture 3.21 Two-storey Government Building, Rudbar, General view

Picture 3.22 Two-storey Government Building, Rudbar
Collapse of first floor jack arches without dislocation of I-beams

Picture 3.23 Damage to Two-three Storey Steel Frame Residence, Rudbar

Picture 3.24 Two-storey Steel Frame Building, Rudbar
Bending failure in upper floor due to lack of shear elements

4 BEHAVIOUR OF ENGINEERED BUILDINGS

4.1 INTRODUCTION

There were a number of engineered residential and non-residential buildings in the area affected by the earthquake. Most of those investigated were situated in the city of Rasht (Fig. 2.1) some 60 km north of the epicentre where the intensity of ground shaking was relatively low.

The term 'engineered' here refers to the type of building in which either seismic loading was specifically considered in design or designed and constructed according to the normal engineering practices (which in themselves provide a certain amount of resistance to horizontal seismic loading). To earthquake engineers in general, the performance of these buildings is of more interest than the semi-engineered buildings described previously, although many lessons learnt from the behaviour of semi-engineered buildings are also applicable to the engineered ones.

The engineered buildings investigated were invariably concrete or steel framed buildings, ranging in height from one-storey to 10-storey. There were no high-rise buildings in the towns of Manjil and Rudbar. However a few steel and concrete framed three to five-storey buildings fared better than the one to two-storey buildings during the earthquake. On the other hand, most of the damaged buildings in Rasht were relatively tall (six to ten storeys). This indicates the low frequency range of the strong components of the quake in Rasht as compared to the towns of Manjil and Rudbar where, to judge from the type of buildings most damaged and the topography of the area, frequencies of ground shaking were much higher.

In the following, the behaviour of some of these buildings are examined in detail.

4.2 CASE STUDIES

4.2.1 TELECOM CENTRE, MANJIL

The three-storey Telecom centre at Manjil was one of a few buildings in the east side of the town left standing. Although it was damaged beyond repair, with most of the infill walls collapsed, the reinforced concrete frame of the structure kept the floors and roof in place (Picture 4.1).

The failure modes of this building were investigated by closely examining, as far as possible, the forms of failure in individual columns (28 off) and beams (90 off). The dimensions and lay-out of the building are shown in Fig. 4.1.a. It is a rectangular building 17.4m x 11.8m in plan dimensions and about 13m high. Because it was constructed on the slope of the hill the east and north sides of the ground floor were partially below the ground level. The main failure at the ground floor was in the form of shear, both in the reinforced concrete columns and the solid infill shear walls. The line of action of these shear walls, as many other buildings in Manjil, was in the same direction as the strongest component of the earthquake (i.e. E-W).
The shear failure in the walls of the ground and first floors was in the form of cross-diagonal (X) cracks which is typical of shear failure in the infill walls (Picture 4.2). As the building moved to one side the walls sheared in that direction and on the return of the building to the other side a second shear failure occurred in the opposite direction, hence the crisscross X shear cracks. Under further ground accelerations of the main event or under the forces of many strong after shocks which followed the main event, the failed free-standing shear walls underwent further bending and over-turning failures and partly collapsed (Picture 4.2).

There were no shear walls in the top floor interior of the building. The column failures at the floor level of this storey were therefore in the form of bending (Picture 4.3). The failure in columns at the first floor was however mainly in shear as seen in Picture 4.4. Not all the columns in this level had failed. It is interesting to note that a few columns which did not fail were all grouped in the north-east corner of the building. Figures 4.1.a and 4.1.b illustrate the type of failure in the columns at first and second floor levels, respectively. No failures were visible in the columns at the roof level from outside and unlike the ground floor and first floor it was not possible to investigate the possible failures at that level from within the building. One

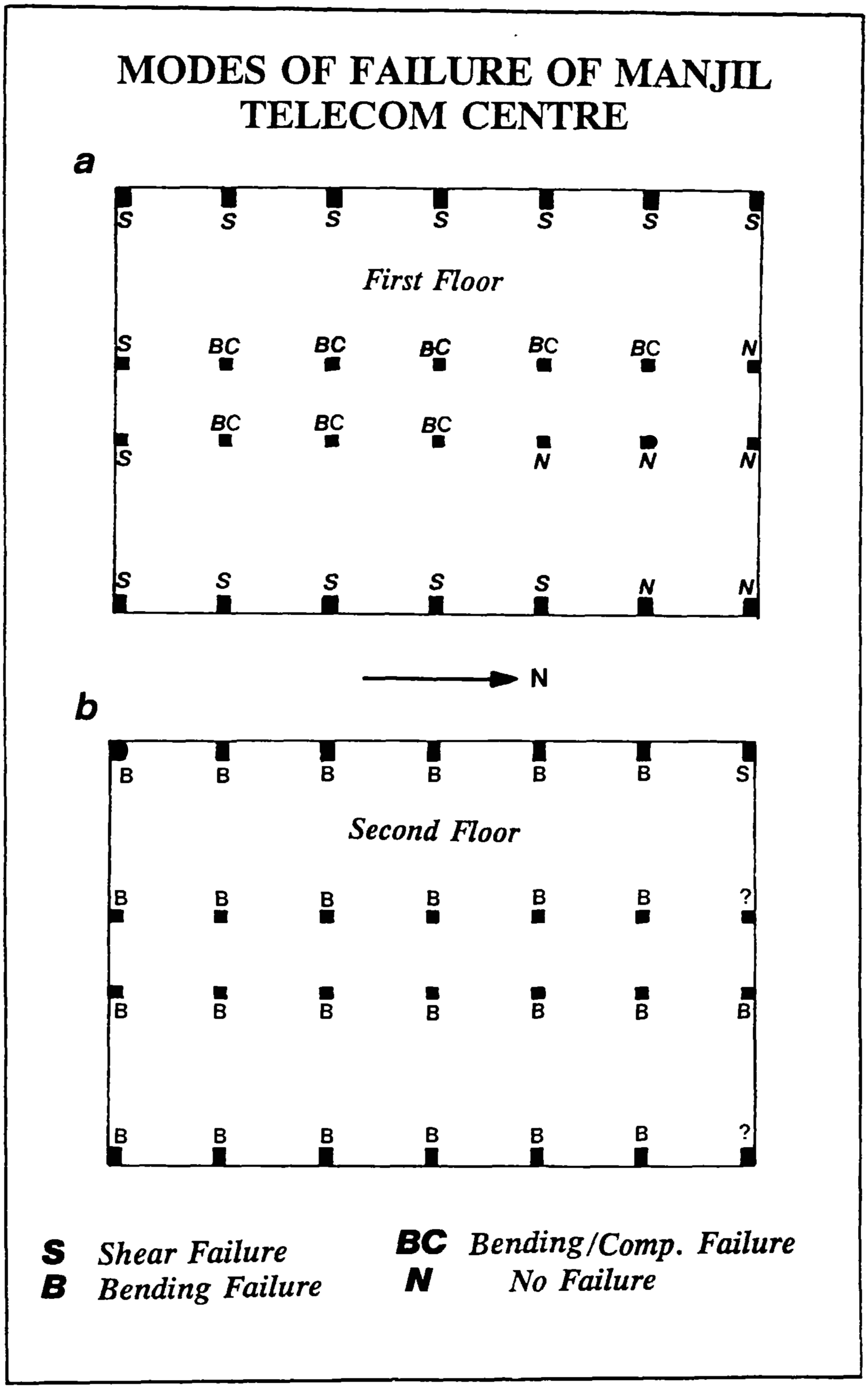

MODES OF FAILURE OF MANJIL TELECOM CENTRE
a
First Floor
S S S S S S S
S BC BC BC BC BC N
BC BC BC
S N N N
S S S S S N N
N
b
Second Floor
B B B B B B S
B B B B B B ?
B B B B B B B
B B B B B B ?
S Shear Failure
B Bending Failure
BC Bending/Comp. Failure
N No Failure

Fig. 4.1

can safely assume that if there were any failures they would be in bending rather than shear.

From the observed failure pattern of the columns, beams and the infill walls it is possible to construct the mode of behaviour of the building during the quake. Under strong east-west component, the initial force of the quake (top of the building moving to the east) was taken by the shear walls of the ground and first floors, causing shear failure in the walls and columns. On the return displacement of the building westwards, shear walls resisted in that direction and similarly failed. The failed columns could not resist any more shear forces. Since there were no shear walls in the second (top) floor of the building the sway of this floor on the return motion caused bending failure in the columns at that level. A diagrammatic illustration of these failure modes are shown in Fig. 4.2.

Although most of the elements failed in the mode mentioned above, there were exceptions particularly in the north-east corner of the building. Damage to that section was far less than the south-west corner, where all the infill walls had collapsed. This indicates that the response of the building in the south-west corner was much more than the north-east corner. Although the building is symmetrical in plan dimensions the presence of an extra floor in the north side (see Picture 4.1) created large mass eccentricities in the building. The behaviour of this building therefore provides a good example of the amplifying effects of mass and stiffness eccentricities in parts of a building and in this case, also possibly the differential stiffness of the underlying soil, providing different support conditions for different parts of the structure.

No flexural failures were observed in the beams. This was because of the short span and large depth of the beams. However, there were many shear and bending failures at the intersection of the beams and columns. A close examination of one of the failed joints indicated poor design and detailing of the reinforcement as the main cause of failure. The amount and size of the reinforcement appears sufficient, nevertheless their lay-out is inappropriate in that, whilst there are only three longitudinal bars in the depth (0.5m) there are four bars in the relatively narrow (0.3m) width of the column (Picture 4.4). Close spacing of excessive number of bars (in relation to the size of the section) not only did not increase the strength of the column but also created a weak plane where the joining beams could easily fail. The failure in the beam-column connections are mainly vertical and along this plane (Picture 4.4).

MODES OF FAILURE OF MANJIL TELECOM CENTRE

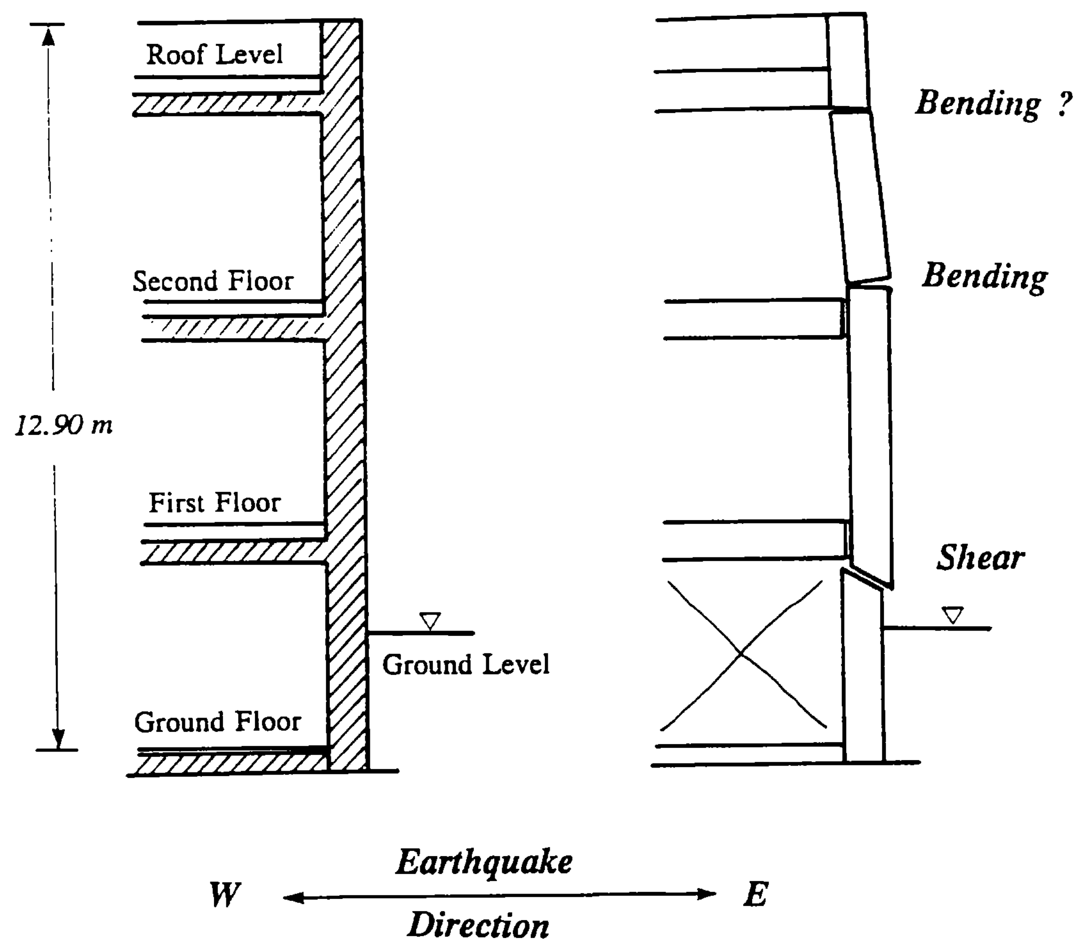


Fig. 4.2

4.2.2 THREE-FOUR STOREY STEEL-FRAMED BUILDING, MANJIL

This steel-framed brick building was one of the few structures in Manjil which escaped severe damage. It is a relatively well-constructed building with diagonal steel angles bracing the ground floor columns on the north and south sides (Picture 4.7). Details of the beam-column connections (seen in Picture 4.8) indicate better design and workmanship than many similar steel frame buildings in the area. The relative height of the building lending to its flexibility also contributed to its better behaviour. The main damage to the building was the loss of support of the staircase on one side. This failure made the staircase act as a heavy cantilever, producing high bending moments in the supporting south columns, which in turn resulted in failure of the columns.

One important design fault with this building is the presence of large openings in its east side (front, Picture 4.9). These openings have drastically reduced the shear stiffness of the building in North-South direction. However it survived, mainly because the strong component of the quake was in the East-West direction.

4.2.3 FOUR TO FIVE STOREY STEEL-FRAMED BUILDING, MANJIL

Another steel-framed structure which did not collapse in Manjil is this 5-storey building (Picture 4.10). Part of the infill unreinforced masonry walls had collapsed but the presence of steel cross bracing insured the stability of the frame as a whole. The significance of symmetrical cross bracing is not only in strengthening the steel frame but also in providing a certain amount of support for the otherwise free standing infill walls.

4.2.4 TWO-STOREY MARBLE STONE-CLAD HOUSE, RUDBAR

This two-storey steel-framed masonry house (Picture 4.11) is built on the slope of one of the western hills of Rudbar. The fact that the building survived with little damage is as much due to it's location as to it's strength. In building this house, large parts of the hill had been excavated thereby providing stiffer supporting soil. The intensity of ground shaking at this slope appears less than other locations in the town. The high frequency vibration of the stiff underlying soil during the earthquake, as compared to the fundamental frequency of the steel-framed building resulted in reduced response amplifications.

4.2.5 ONE-STOREY CONCRETE FRAME HOUSE, RUDBAR

This damaged house is situated on the same hill as the above-mentioned building (4.2.4). Considering that it is a one-storey reinforced concrete frame building, the extent of damage is at first surprising (Picture 4.12). However, unlike the previous building, it is relatively stiff (due to it's height and type of construction), which considering the high frequency range of ground shaking, resulted in an increased earthquake load on the building. Extensive shear and bending failure were observed in both the reinforced concrete frame and the infill walls.

4.2.6 TELECOM BUILDING, RUDBAR

In the vicinity of the one-storey house discussed in 4.2.5, the two-storey reinforced concrete framed Telecom Centre in Rudbar also suffered heavy damage (Pictures 4.13 and 4.14). The damage was largely in the infill concrete-block masonry walls of the building. The concrete frame itself acted well and despite cracking at some locations did not collapse. The good behaviour of the concrete frame also guaranteed the safety of the concrete beam-block floor slabs.

The extensive damage to the infill walls, apart from indicating the high intensity of the ground shaking, points to the heavy interaction of the infill masonry and the surrounding concrete frame. Such interactions may also occur in similar aseismic-designed reinforced masonry buildings. Reinforcement of the brick or block-work may help to mitigate the ultimate collapse of the wall, but is not capable of preventing the interaction between the flexible frame and the brittle masonry and the subsequent failure.

4.2.7 POST OFFICE BUILDING, RUDBAR

The construction of this one-storey building was completed shortly before the earthquake. It was the only building visited in Rudbar which suffered no damage. The building contains all the hall-marks of a good earthquake resistant building (Picture 4.15). Some of these may be summarised as follows:-

a. Low height of the building.

b. Symmetry in lay-out and simplicity of design.

c. Low opening-to-wall ratio (or high wall ratio).

d. Good solid foundation.

e. Reinforced concrete frame (consisting of vertical columns as well as lower and upper ring beams).

The building was inspected from outside and inside, save for a small crack in the plaster of the east wall (river side), the building survived the strong earthquake without any damage. This building behaved as a rigid box under the quake, it's structural elements (reinforced concrete frame and roof) were strong enough to withstand the increased seismic base-shear. The rigid-box type behaviour is ideal for short buildings, whereas for taller buildings to achieve such behaviour requires an oversized and complicated frame structure. As a result a controlled flexible behaviour is more favourable for the taller buildings.

4.2.8 BANK MELLI BUILDING, RUDBAR

Another building in Rudbar which behaved well during the earthquake was the one-storey Bank Melli building opposite the Post Office building. This building also had a reinforced concrete frame. The roof construction could not be ascertained (either reinforced concrete or concrete beam-block system). The building remained structurally sound and the only visible damage was cracking and collapse of parts of the cladding, covering the reinforced concrete frame (Picture 4.16).

4.2.9 THREE-STOREY STEEL-FRAMED HOUSE, MANJIL

A possible example of the effects of dynamic interaction on the response of eccentric buildings during earthquakes can be observed in the behaviour of this three-storey steel-framed building. As can be seen in Picture 4.17, the back of the building (west side) has completely failed and collapsed whereas the front section (east side, Picture 4.18) has remained relatively intact. This behaviour can be explained as follows:

Although the building is relatively symmetrical in plan there appear to be large stiffness and mass eccentricities in the building. At the back of the building the presence of a solid brick wall had made that side much heavier than the east side, which contains large openings, hence creating a large mass eccentricity towards the back of the building. This solid wall had also made the back of the building stiff. However the concentration of steel sections in the front of the building resulted in the centre of stiffness to be less eccentric (in relation to the centre of geometry) than the centre of mass. Under strong horizontal forces of the quake such a difference in the position of the centres of stiffness and mass in a relatively flexible building, possibly produced coupling between lateral and torsional modes of vibration resulting in high amplifications of response at the heavier side. The amplified response due to dynamic coupling is believed to be the cause of failure of that side. Although it is possible that the brittleness of this side caused it's premature failure before such dynamic coupling could take place.

4.2.10 EIGHT-STOREY UNFINISHED STEEL FRAME BUILDING, RASHT

The construction of this building (Picture 4.19) was started 15 years ago and was originally to be a four storey building, the steel frame of which was rapidly erected. After many years of abandonment and change of ownership, it was decided that four extra stories were to be added to the building. The steel columns were therefore extended using inferior steel sections and new steel beams were welded to the column extensions to support the new floor slabs. It was also decided that to strengthen the building, cross-diagonal steel bracing were needed. In process however only one side of the building was braced (Picture 4.19).

This hybrid of steel framing created a structure susceptible to heavy damage under a low intensity quake (I=VI).

The damage to this building includes:-

(i) a large sway (about 50 cm) to one side (see Picture 4.20),

(ii) bending failure in steel columns at the fifth floor level,

(iii) collapse of some of the infill walls at both the solid sections and the sections around the openings and

(iv) damage to the neighbouring building due to pounding.

The behaviour of this building during the quake may be summarised as follows:-

The steel cross-bracing at one side of the building created stiffness eccentricities resulting in an amplification of response at the opposite side due to dynamic coupling. The coupled mode of vibration being a combined sway and torsional mode. Since there are no shear walls or cross-bracing in the transverse direction the building could not resist the sway displacements hence failing in that mode. Simultaneously, the amplified torsional vibration of the building pounded the neighbouring building. This action not only damaged the adjacent building but also was responsible for the failure and collapse of the infill brick walls. Pounding also appeared responsible for a second bending failure in columns at the fifth floor level (same level as the roof of the pounded building) causing another sway in the opposite direction (towards the neighbouring building) of the upper floors.

The fact that the welded joints of the steel frame survived the ground shaking and the pounding is perhaps the only credit one may assume for this otherwise poorly designed and constructed building.

4.2.11 SIX-STOREY SHOP STORE, RASHT

With a similar history to the steel-framed building discussed in 4.2.10, this shop store (Picture 4.21) began it's life as a four-storey building. Despite initial resistance from the City Council, the owner was eventually allowed to increase it's height by adding a further two stories. The steel columns of the original building were simply extended by welding additional steel sections without strengthening the existing columns for the extra load. The facade of the building showed no sign of damage except for one shear crack in the central wall of the fourth storey (Picture 4.21). However, extensive damage could be seen inside the building, particularly in and around the fourth storey where the new section had been joined to the original building.

4.2.12 COLLAPSED SIX-STOREY STEEL FRAME BUILDING, RASHT

Picture 4.22 shows the remains of a six-storey steel frame apartment block in Rasht, one of a handful of tall buildings which suffered a similar fate in the city. The construction of the building had just been completed, luckily, as a result it was uninhabited at the time of the earthquake. However, collapse of the building on top of it's neighbouring shops and houses caused some injury to the residents of those houses. The shop next to the collapsed building (Picture 4.22) was completely destroyed by this secondary action and at the time of the visit (40 days after the event) had been rebuilt and just re-opened for business.

The material used and construction techniques adopted in the building of this apartment block is typical of recent construction of steel-framed buildings in Iran. Steel columns and beams as the main load-bearing elements, concrete beam-block slabs and light weight (hollow, corrugated) brick in-fill walls sum up the main building elements (Picture 4.23).

A typical steel column of this building is shown in Picture 4.24. The columns consist of two steel I-beams (3"x4") joined together by thin strips of steel at regular intervals of 25 to 30cm, giving final dimensions of 4"x9". The steel columns are then filled by concrete and broken brick-concrete rubble. Different sections of columns were then welded together to form a 20m high column supported only at the floor levels by similarly weak 3"x4" steel beams. Considering the building had only just been completed, the amount of corrosion in steel beams and columns indicates long exposure of the bare steel to the damp environment of this area.

The concrete beam-block floors of the building had, as expected, acted as rigid slabs and surprisingly, in most part, survived both the earthquake forces and the shock of collapse from a height (Picture 4.23). In areas where the concrete blocks had come loose, the breaking of the poorly reinforced flanges of the concrete beams was responsibly for the failure.

As is clearly evident the main cause of collapse of this building lies in the weak welded joints between different sections of the columns and between columns and beams. This is best illustrated in Picture 4.25. The very poor and inadequate welds (in many places spot welds) had simply snapped before the similarly weak beams and columns could undergo any significant stresses. All the steel beams and columns are thus un-deformed except a few which were deformed under the falling debris.

Poor design in the form of inadequate steel, poor workmanship in welding and poor supervision and quality control were the main causes of collapse of this building when subjected to relatively mild ground shaking.

4.2.13 COLLAPSED EIGHT-STOREY BUILDING, RASHT

Picture 4.26 shows the site of an eight-storey steel framed building in Rasht which completely collapsed under the earthquake loading, damaging a number of surrounding buildings in the process. The building was similar to the collapsed six-storey steel frame building discussed above (4.2.12) with inadequate steel columns and beams and weak welded joints.

4.2.14 SEVEN-STOREY WHITE STONE-CLAD BUILDING, RASHT

Picture 4.27 shows another damaged steel-framed building in Rasht. Close inspection of this rather old building however revealed good design and workmanship in the construction of the steel frame. As a result, the frame kept it's integrity and maintained the stability of floors and most of the walls. In a number of places, nevertheless, the brittle infill walls failed and collapsed under the amplified response accelerations the building (Picture 4.28).

4.2.15 TELECOM CENTRE, RASHT

This is another old steel frame building which suffered no structural failures (Picture 4.29). The only visible damage was in the in-fill walls between openings (Picture 4.30). The failures were in the form of diagonal (shear) cracks as well as horizontal and vertical out-of-plane bending and flexural cracks.

4.2.16 TERRACED BUILDINGS, RASHT

This cluster of four to six storey buildings in Rasht survived the quake with varying degrees of damage (Picture 4.31). The reinforced concrete frame building to the right of the picture behaved well and not only itself was undamaged but also greatly helped the stability of the adjoining weaker steel-framed buildings. The minor damage to this concrete building was at it's intersection with the steel building and due to pounding of the latter (Picture 4.32). When inspecting the interior of the steel building, numerous cracks (mainly shear) were seen in the infill walls. If the concrete building had not contained the steel building, the latter would have undoubtedly suffered heavier damage.

This is a clear example of the better behaviour of concrete frame buildings compared to their steel frame counterparts of recent construction. Apart from the higher strength of the frame, particularly at joints, the concrete frame buildings were also stiffer, putting the building frequencies outside the strong frequency range of ground shaking in Rasht.

4.2.17 NINE-STOREY APARTMENT BUILDING, RASHT

The reinforced concrete frame of this nine-storey apartment block survived the earthquake without damage (Picture 4.33) and although there were rather extensive cracking in the infill walls around the openings, the building as a whole is stable (Picture 4.34). The height of the building suggests a longer period of vibration, probably very near to the strong frequency range of the quake at Rasht. The damage to the in-fill walls indicate relatively high response of the building to ground shaking. However, as was the case with many other reinforced concrete frame buildings of similar type, the frame behaved well during the earthquake.

4.2.18 EIGHT-STOREY WHITE STONE-CLAD CONCRETE FRAME BUILDING, RASHT

This well constructed eight-storey reinforced concrete frame building (Picture 4.35) was the only tall building in Rasht which survived the earthquake without any damage (except a small crack in the white stone cladding (Picture 4.36). It is a well proportioned symmetrical structure with large reinforced concrete columns and deep concrete beams. The floor slabs are also concrete beam-block system giving it extra rigidity which helped it's dynamic response.

4.2.19 EIGHT-STOREY GREY STONE-CLAD CONCRETE FRAME BUILDING, RASHT

This is another well built eight-storey concrete frame building in Rasht, designed and constructed by the same engineers as the previous eight-storey building (4.2.18). It also escaped the earthquake without any structural damage (Picture 4.37). The stiff reinforced concrete frame ensured desirable dynamic response of the building.

As far as the earthquake resistant design of tall buildings is concerned, the only shortcoming of this building (Picture 4.38) and the one discussed earlier is in their unreinforced stone cladding. The fall of cladding can be as dangerous to lives as collapse of parts of the building itself.

4.3 GENERAL OBSERVATIONS

4.3.1 CONCRETE FRAME

- The few concrete-framed engineered buildings investigated behaved relatively well under the ground motion. In short buildings (one to two storey) of Rudbar the behaviour was exceptionally good considering the level of ground accelerations. This is attributed to the rigid box behaviour of the buildings made possible by their size and adequate reinforced concrete frames.

- There were no tall (five storey plus) reinforced concrete-framed buildings in the high acceleration epicentral area. The buildings visited were all situated in the city of Rasht (60 km away). Compared to the steel-framed buildings of similar size the concrete-framed buildings behaved well and no serious failures were apparent in the frame itself.

- The few failed concrete-framed buildings investigated in the epicentral areas had a few facts in common. These include use of low grade concrete and lack of proper curing together with use of inappropriate reinforcement bars and poor detailing. Also the buildings were intermediary (three to five storey) falling between short and tall buildings with reduced shear stiffness compared to the shorter buildings. As a result they were not able to act as a rigid box hence failing not only in shear but also in flexural modes.

4.3.2 STEEL FRAME

- In contrast to the concrete-framed buildings, the steel-framed buildings showed less resistance to the earthquake loading. Apart from the two main causes of weakness, ie poor welding and insufficient steel sections which will be discussed, another shortcoming appeared to be higher interaction between the more flexible steel frame and the rigid brittle in-fill masonry walls.

- Poor quality welding in steel construction has already been discussed in relation to the semi-engineered buildings. In the engineered buildings the quality of welding appeared only marginally better. In recent years in particular with increasingly higher demand for building materials and lower supplies, the state of steel constructions has deteriorated. Insufficient supply, together with lack of proper quality control has made the 'more sensitive to misuse' steel construction far inferior to the relatively 'less sensitive to misuse' concrete construction.

- Insufficient steel sections for load-carrying beams and columns noted in buildings of more recent construction also contributed to the seismic instability of the steel-framed buildings.

- In older steel-framed buildings the steel sections were notably larger and in some cases bolted joints were adopted. Those with welded connections also appeared to have better quality welds. The performance of these buildings was evidently better than their more recent counterparts.

4.3.3 ARCHITECTURAL LAY-OUT

- Complex architectural lay-out results in complex dynamic behaviour which sometimes is difficult to formulate or foresee. The majority of engineered buildings visited had geometrically symmetric lay-outs. A few which had somewhat asymmetric lay-outs suffered from the resulting complex dynamic behaviour.

- Large lateral sway in a number of tall buildings (six to ten storey) was caused by the absence of shear walls in some or all floors in at least one direction. The presence of shear walls in buildings of this type is essential for their stability and strength. A clear example of this mode of failure was observed in the eight-storey steel-framed building in Rasht (4.2.10). In that building no shear walls were present in the direction of sway.

4.3.4 MASS/STIFFNESS ECCENTRICITIES

The phenomenon of lateral-torsional dynamic interaction is likely to occur in buildings with large stiffness and/or mass eccentricities. This is when the centre of mass and the centre of stiffness of the building do not coincide. Such dynamic eccentricities give rise to high amplifications of response at one side of the building, particularly if the frequencies of first torsional and first lateral modes are close to each other. The behaviour is diagrammatically illustrated in Fig. 4.3 In a symmetric building where centres of mass and stiffness coincide, the lateral and torsional modes are uncoupled and the response in either mode is limited. In an asymmetric building the lateral and torsional modes are generally coupled, which depending on the amount of dynamic eccentricity, the response would be amplified.

The understanding of the lateral-torsional coupling mechanism under dynamic loading is relatively new to earthquake engineering. The phenomenon has been the subject of very recent theoretical modelling and experimental investigations in laboratories [4]. The behaviour was also observed in a number of engineered buildings and other structures under the real circumstances of the Manjil earthquake. The concrete-framed Manjil Telecom centre (case study 4.2.1), in which the damage to one side was noted to be more than the other, is one example of this type of behaviour caused by mass eccentricity. The pounding of the eight-storey steel-framed building in Rasht (case study 4.2.10) against its neighbouring building, caused by both stiffness and mass eccentricities, is another example of this coupling mode. However, perhaps the best and clearest example of this mode of behaviour was observed in the three-storey steel-framed building in Manjil (case study 4.2.9). In this building the mass eccentricity caused high amplitude response at the back of the building whereas the front benefited from the dynamic coupling with a much lower response amplitude. The pattern of damage in various parts of the building is clear indication of it's dynamic response under the earthquake loading.

4.3.5 INFILL WALL-FRAME INTERACTION

- A common form of damage observed in many engineered buildings was the failure of the exterior and interior infill masonry walls. This was particularly true for those taller buildings which did not behave as a rigid box. The interaction between the brittle rigid infill walls and the more flexible ductile reinforced concrete or steel frame may be considered as responsible for the majority of these failures. (Another reason was lack of proper connections between the frame and wall rendering the infill a free-standing wall.)

- In reinforced concrete-framed buildings the interaction caused only local failures at the interface between the frame and the wall. The failures were also mainly in the infill

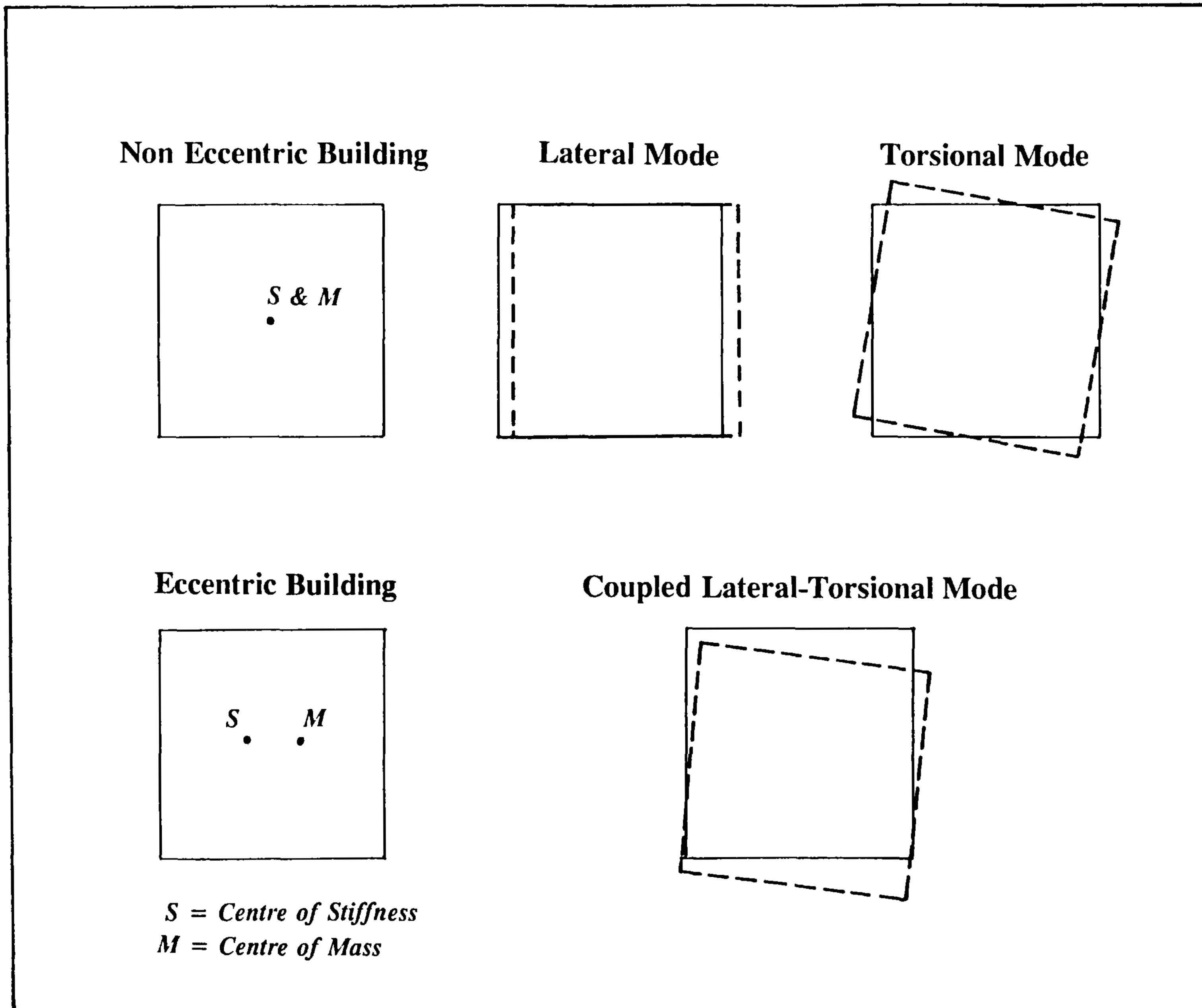


Fig. 4.3 Lateral-torsional Dynamic Coupling in Asymmetric Buildings

sections rather than the frame itself.

In the steel-framed buildings, on the other hand, the interaction was more intensive. This was due to the high differential stiffnesses of the frame and the walls. The higher interaction caused more serious damage to the in-fill walls and in a few observed cases resulted in failures in the steel sections or the welded joints of the frame.

4.4 CONCLUSIONS AND RECOMMENDATIONS

4.4.1 CONCLUSION

1. In general, the concrete-framed buildings behaved better than their steel-framed counterparts. For smaller buildings a 'rigid box' behaviour could be better achieved by a concrete frame than a steel frame. A concrete-frame is also less sensitive to bad workmanship or bad design or the lack of it.

2. The poor quality of welded connections and in some cases inadequate steel sections were the two main short comings of the steel-framed buildings of recent construction. In some older steel-framed buildings in which either bolted connections were used or the quality of welded joints was higher, the behaviour was more favourable.

3. Complex architectural lay-outs, geometric asymmetry and lack of adequate interior shear walls greatly contributed to the dynamic instability of some of the engineered buildings.

4. Lateral-torsional dynamic interaction occurred in buildings with high mass and/or stiffness eccentricities. The amplified response not only caused extensive failures in some cases, but in a number of other cases it was also responsible for secondary damage to neighbouring buildings due to pounding.

5. A common failure in many engineered buildings was the interaction between the frame and the masonry infill walls. The interaction which resulted in localised failure in the wall sections, was caused by differential stiffnesses of the rigid walls and the more flexible frame.

6. The fall of un-reinforced stone cladding was a common feature of many taller buildings which had otherwise survived the earthquake without damage.

4.4.2 RECOMMENDATIONS

1. The better behaviour of reinforced concrete-framed buildings as observed throughout the area hit by the Manjil earthquake, highlighted the already well established notion of suitability of reinforced concrete design for smaller buildings. This provides the desired action for the small buildings.

   Reinforced concrete frame, consisting of ground and floor level ring-beams and a sufficient number of connecting load-bearing columns together with either, reinforced concrete or concrete beam-blocks slabs (described in section 3.1) seems to provide adequate protection against collapse under a strong earthquake. The rigid-box frame should be founded on adequate foundations. The key to a rigid-body motion (for a small building) however lies with the masonry in-fill shear walls at all levels and directions. These should be solid with minimum opening area and ideally reinforced brick masonry walls connected to the concrete frame. The height of these buildings should be limited to two storey and as far as architecturally possible should be simple and symmetric in geometry.

   For higher buildings proper seismic design procedures as laid out in many seismic codes of practice including the Iranian code (1988) should be carried out by qualified engineers and their implementation verified during construction.

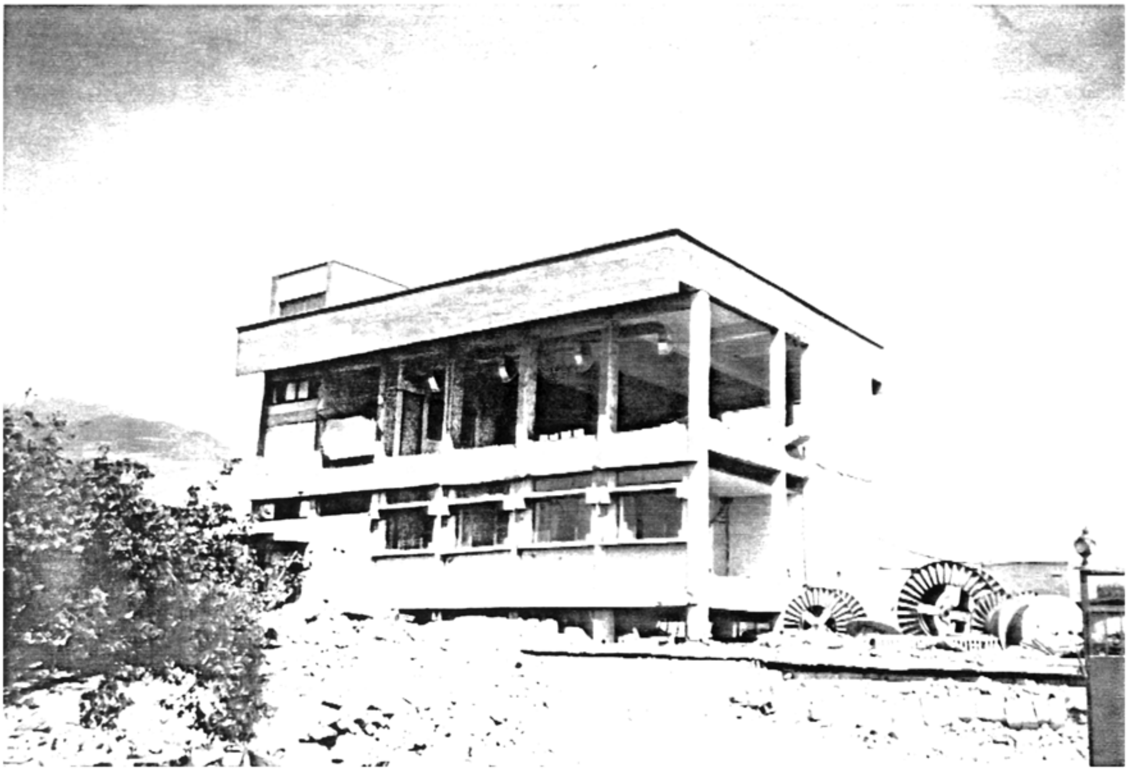
2. The seismic design and construction of steel-framed buildings requires not only in-depth analysis and calculations but also a stringent quality control during construction and maintenance. Such controls could be better achieved using bolted joints as the strength and quality of welding on site is difficult to check and verify.

   Unless such practices as proper seismic design and calculation for choosing appropriate steel sections and the use of bolted joints come into effect, construction of high-rise steel-framed buildings should be discouraged.

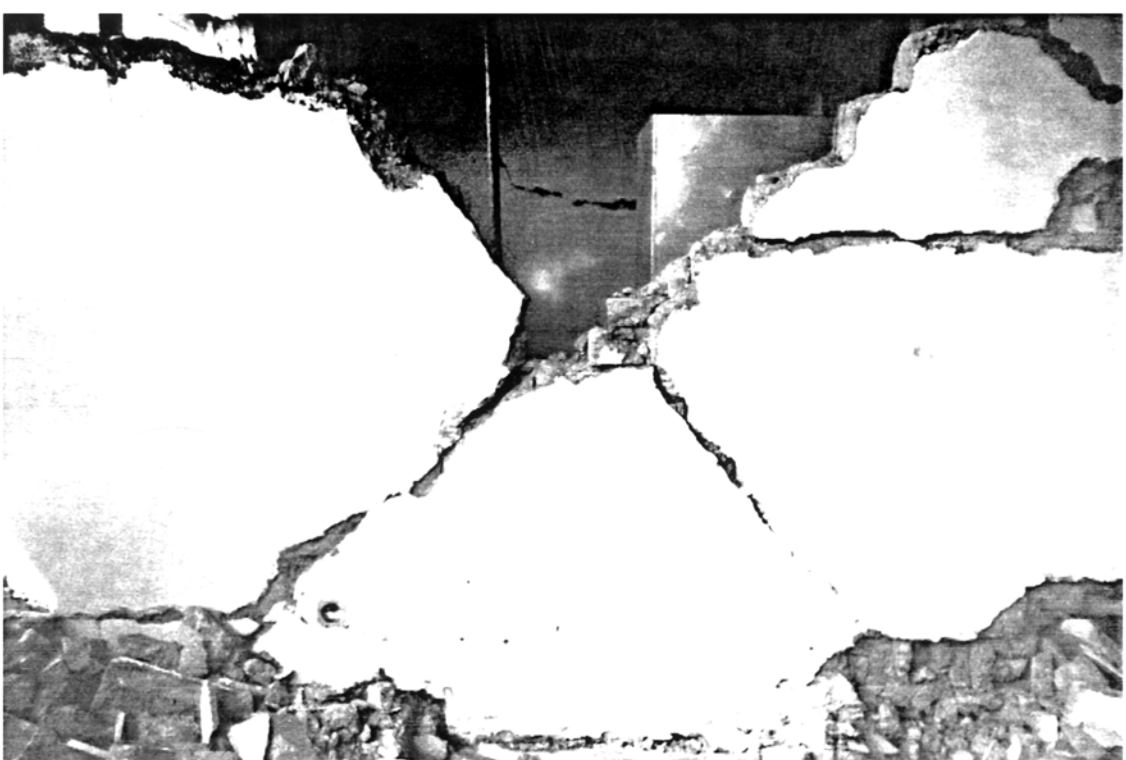
It should be noted that the cost of a steel-framed building in many instances may be higher than the equivalent concrete-frame.

3. A review of the 1988 Iranian Seismic Code should be undertaken to include the new understandings and trends in seismic behaviour and design and the lessons learnt from the Manjil earthquake of 1990.

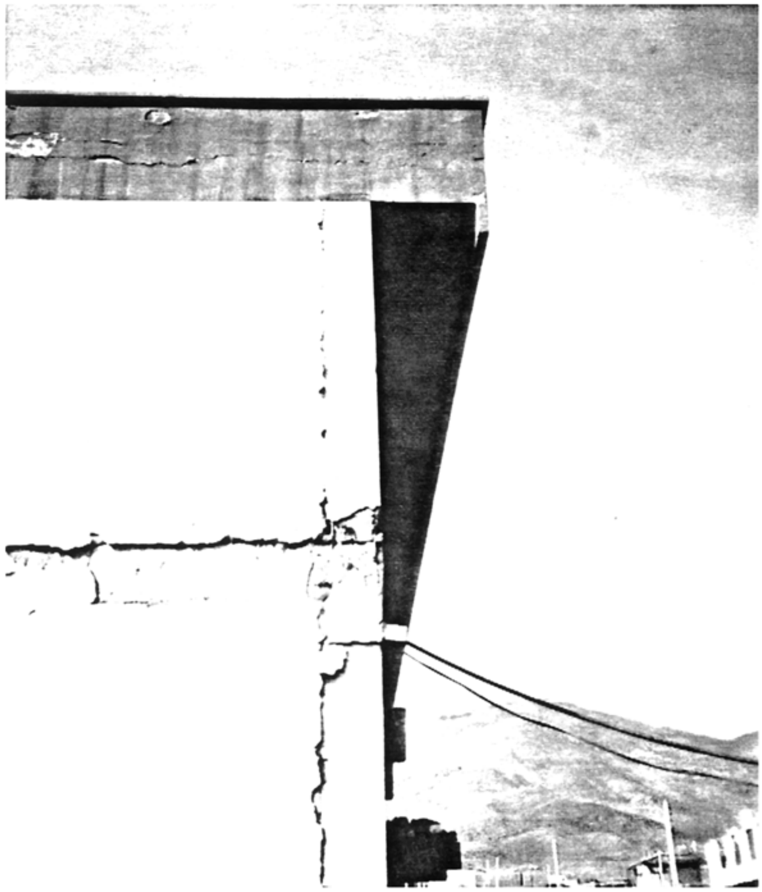
4. Perhaps more pressing than the quality of seismic codes of practice, the widespread application of such codes in the construction industry should be advocated. This is the most important lesson learned from the Manjil earthquake. A lesson not so much for individual engineers and builders but for the relevant authorities for exercising stricter control of the construction industry.



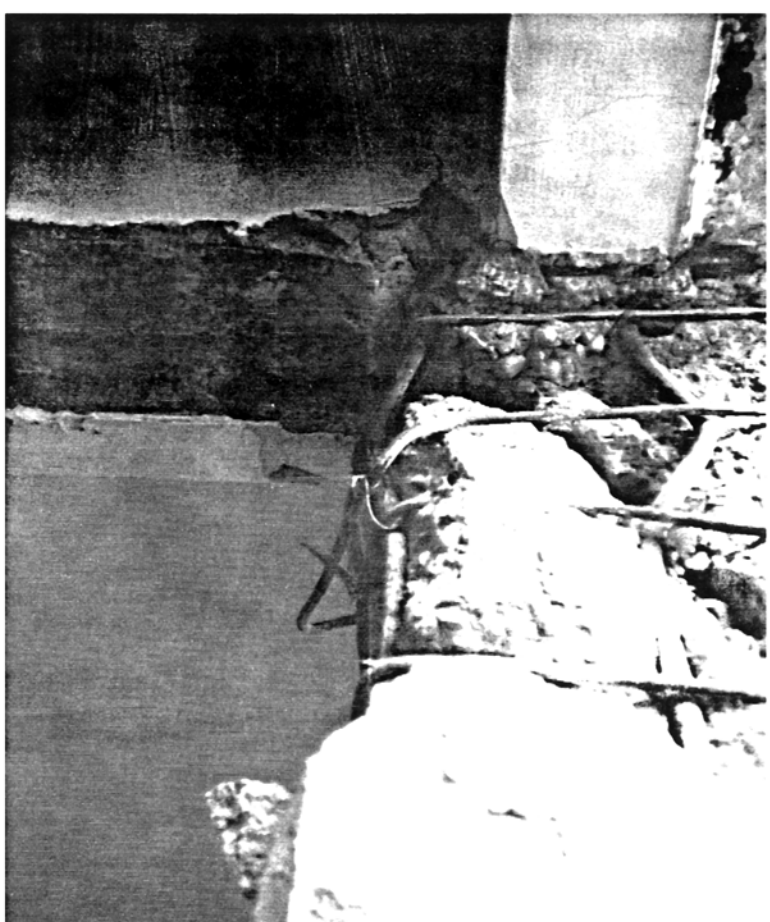
Picture 4.1 Telecom Centre, Manjil, West view



Picture 4.2 Telecom Centre, Manjil
Diagonal X shear failure in the shear wall followed by flexural and overturning failures

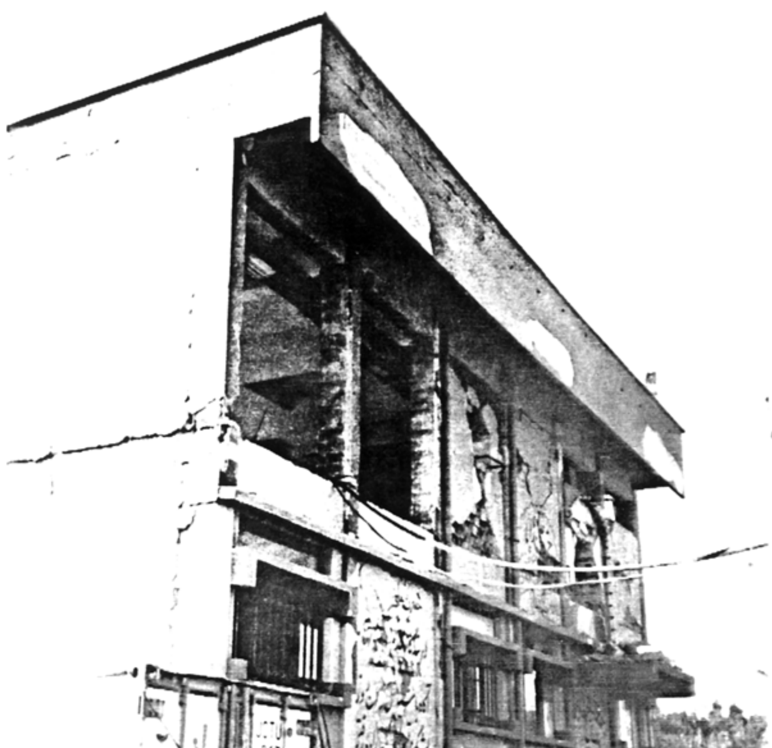


Picture 4.3 Telecom Centre, Manjil
Bending failure at the base of the upper floor column

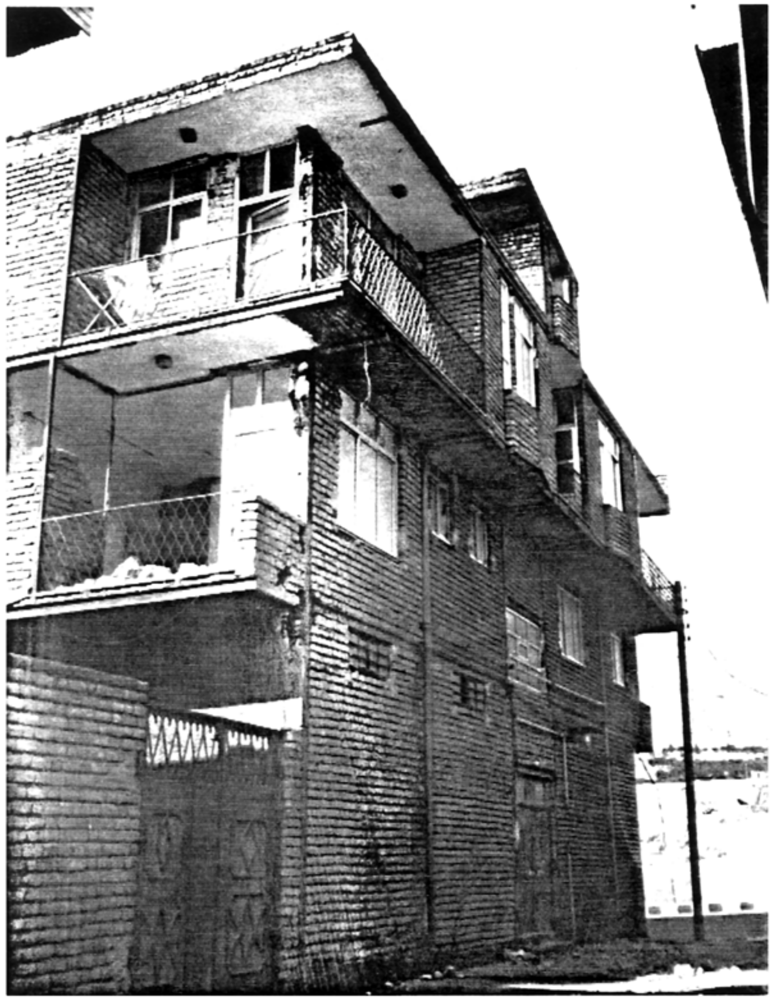


Picture 4.4 Telecom Centre, Manjil
Shear failure in the reinforced concrete column

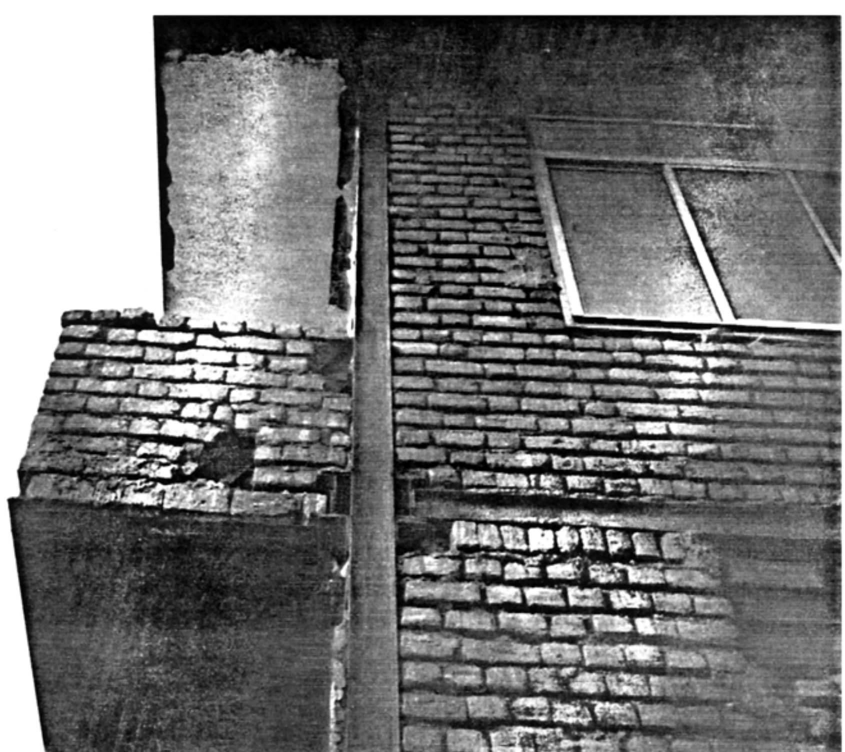
Picture 4.5 Telecom Centre, Manjil
Diagonal shear failure in the concrete block masonry wall extending to the reinforced concrete column



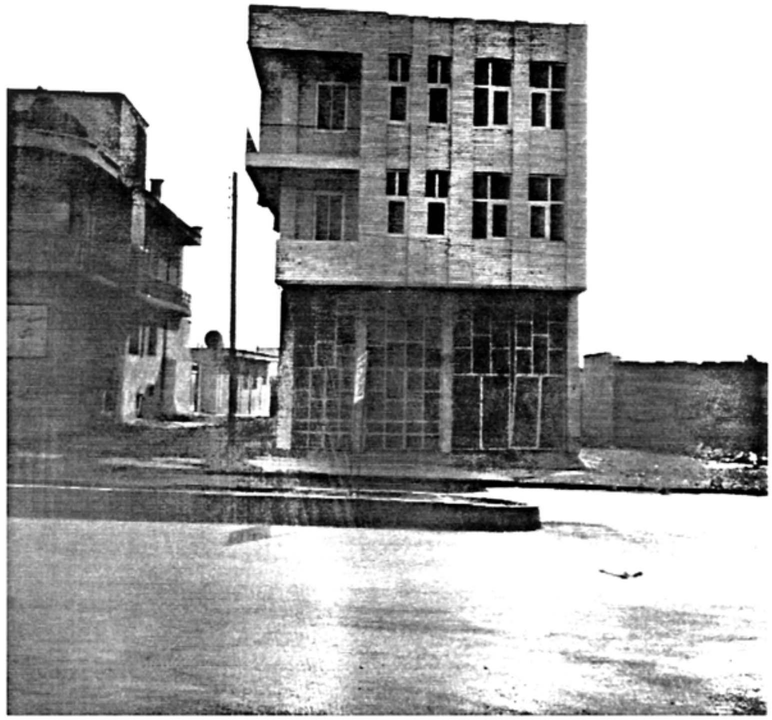
Picture 4.6 Telecom Centre, Manjil
View of east face of the building indicating lesser response at the south-east corner



Picture 4.7 3-4 Storey Steel Framed Building, Manjil
The good behaviour of this relatively well constructed building was probably due to its flexibility



Picture 4.8 3-4 Storey Steel Framed Building, Manjil
Details of the beam-column connection



Picture 4.9 3-4 Storey Steel Framed Building, Manjil
Lack of transverse shear walls at ground floor causing slight sway in that direction

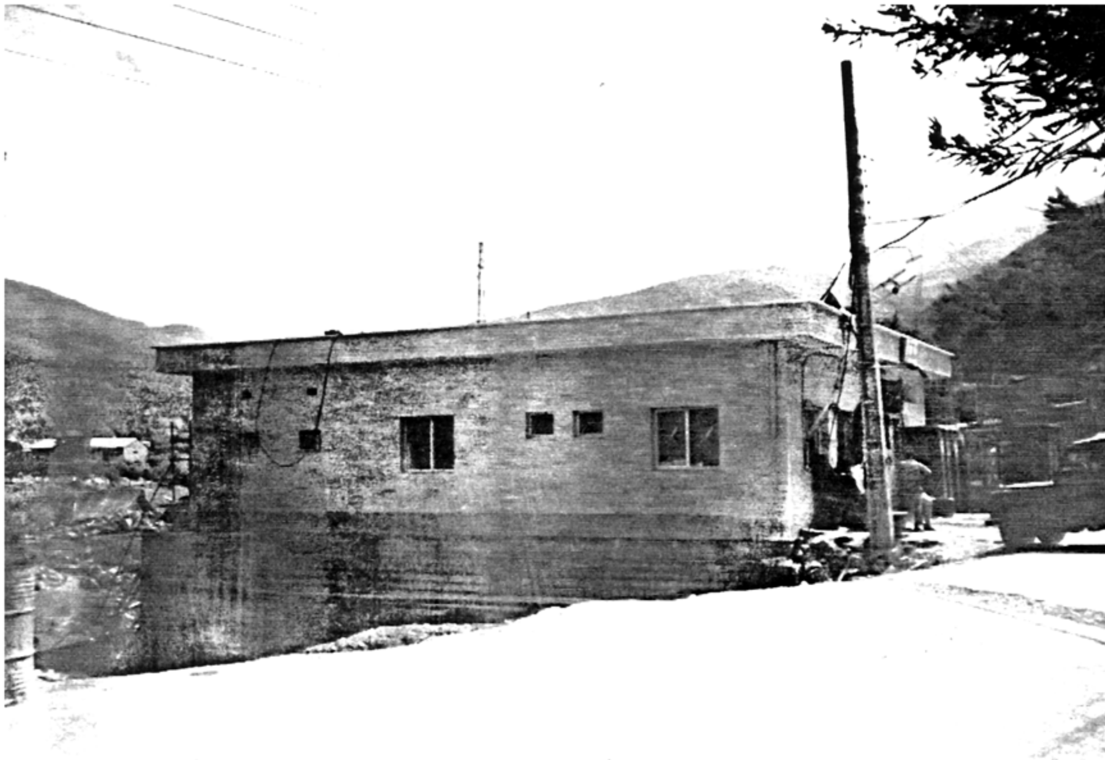
Picture 4.10 Better behaviour of this flexible steel frame building was as much due to its lower natural frequency of vibration as its relatively better construction, Manjil

Picture 4.11 Two-storey Stone Clad House, Rudbar
This relatively flexible steel frame house, founded on hard ground survived the quake with little damage

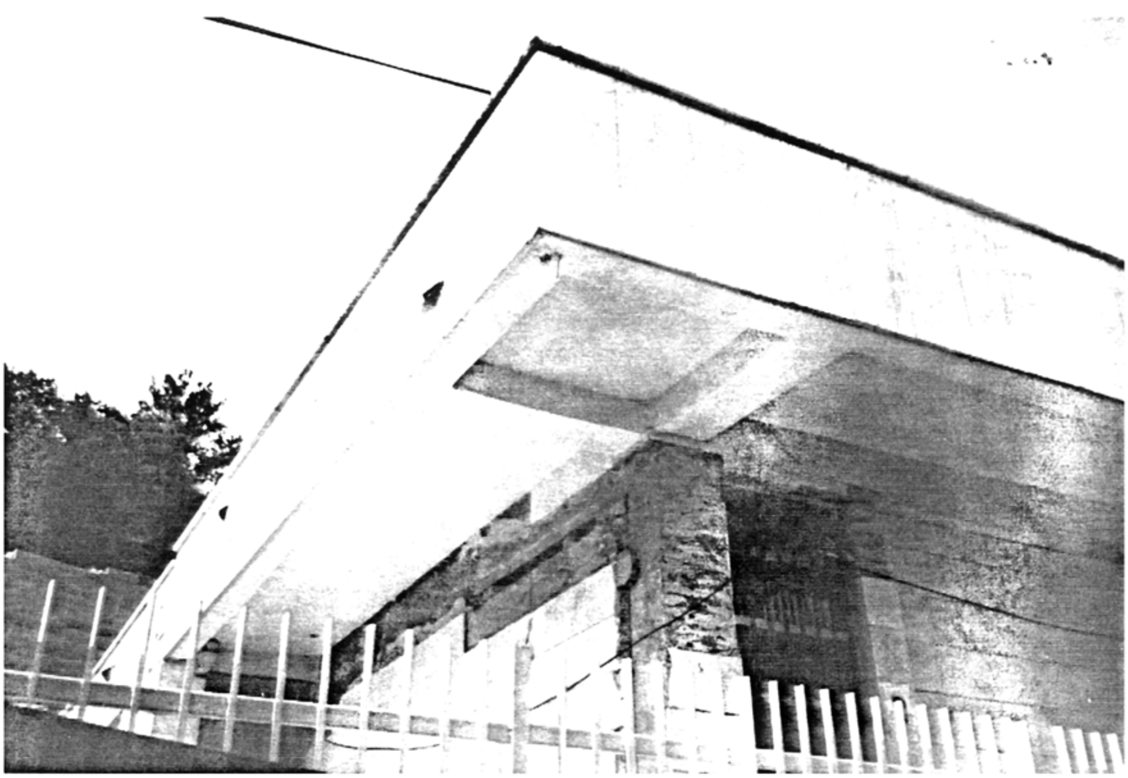
Picture 4.12 One-storey Reinforced Concrete Frame House, Rudbar
This relatively stiff house, founded on built ground suffered heavy damage due to high dynamic amplification

Picture 4.13 Telecom Building, Rudbar
Collapse of the infill masonry walls

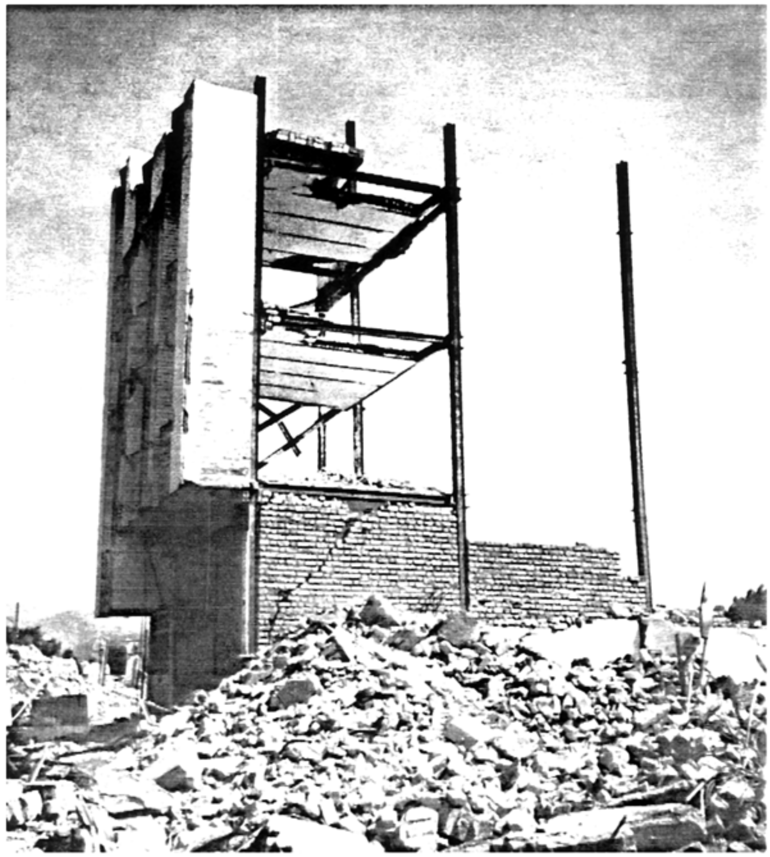
Picture 4.14 Telecom Building, Rudbar



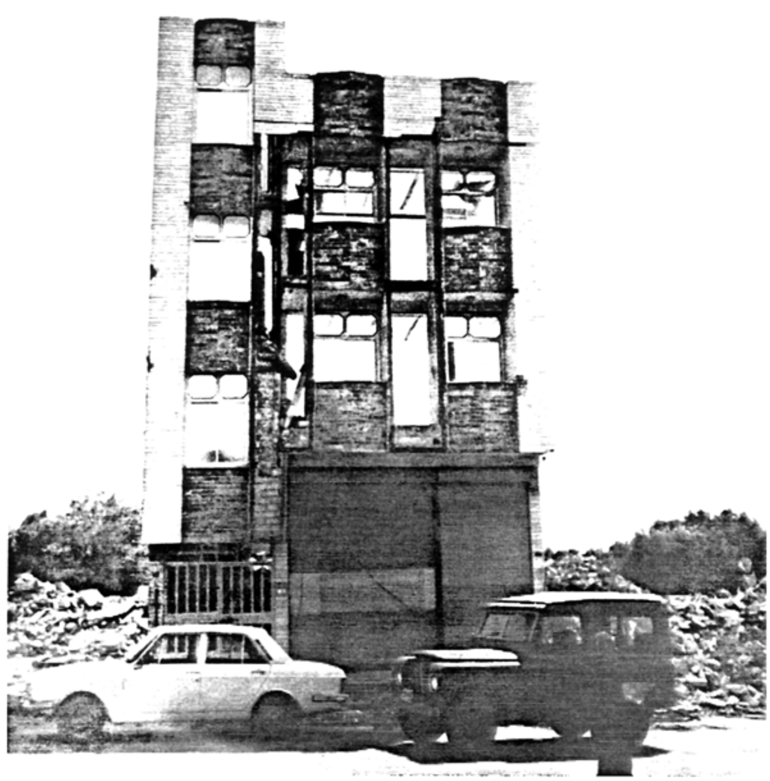
Picture 4.15 New Post Office Building, Rudbar
The only visited building in Rudbar without damage, good example of an earthquake resistant building



Picture 4.16 Bank Melli Iran, Rudbar
Good behaviour of reinforced concrete frame saved this one-storey brick building from major damage

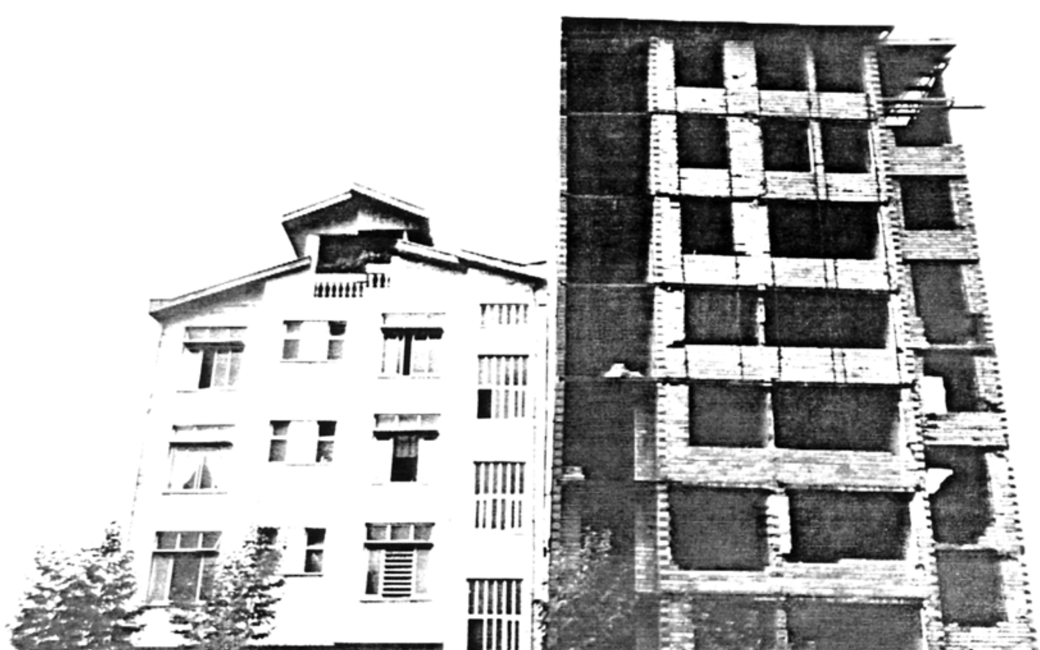


Picture 4.17 Three-four Storey Steel Frame House, Manjil
Amplification of response at the back of the building due to lateral-torsional coupling caused by both mass and stiffness eccentricities in the building



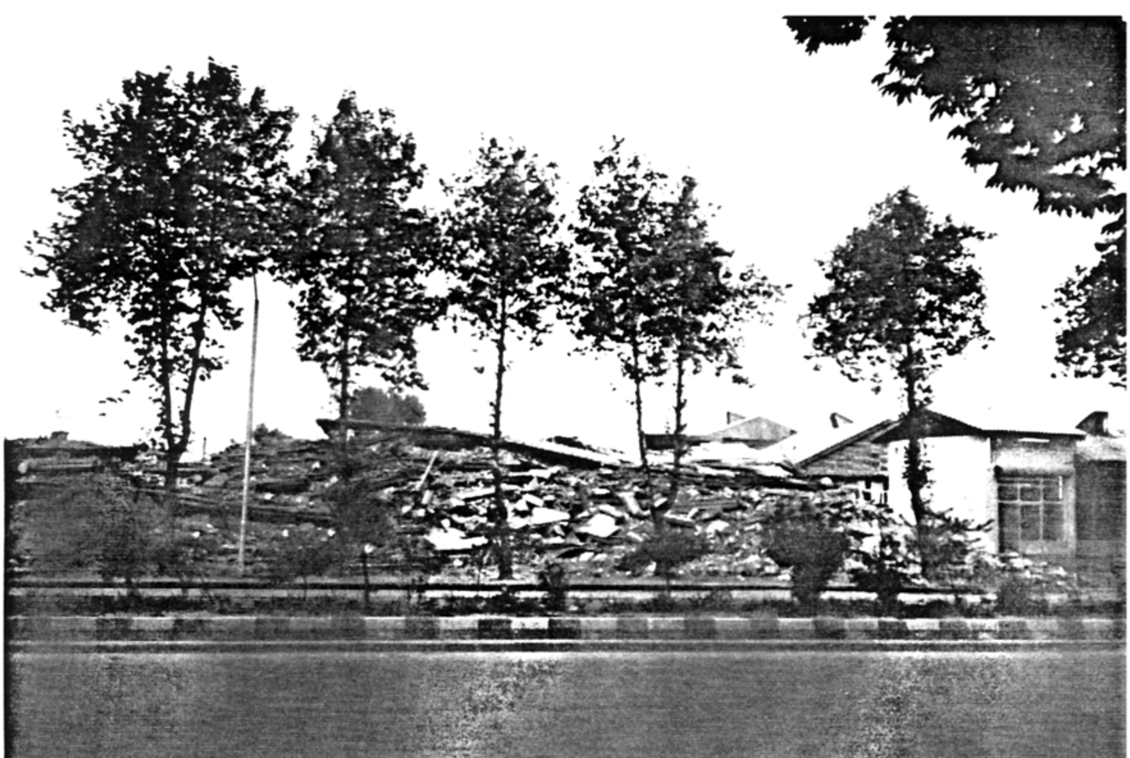
Picture 4.18 Three-four Storey Steel Frame House, Manjil
Front of the building relatively undamaged whilst the back completely collapsed

Picture 4.19 Unfinished Steel Frame Building, Rasht
Lateral-torsional coupling amplification in response of the building due to its eccentric stiffness



Picture 4.20 Unfinished Steel Frame Building, Rasht
Large sway of the building as a result of lack of shear resisting elements

Picture 4.21 Six-storey Shop Store, Rasht
Shear failure in the central infill wall



Picture 4.22 Six-storey Steel Frame Building, Rasht
Complete collapse of the building due to failure of welded joints

Picture 4.23 Six-storey Steel Frame Building, Rasht
Concrete beam-block floor slabs maintaining their integrity after collapse

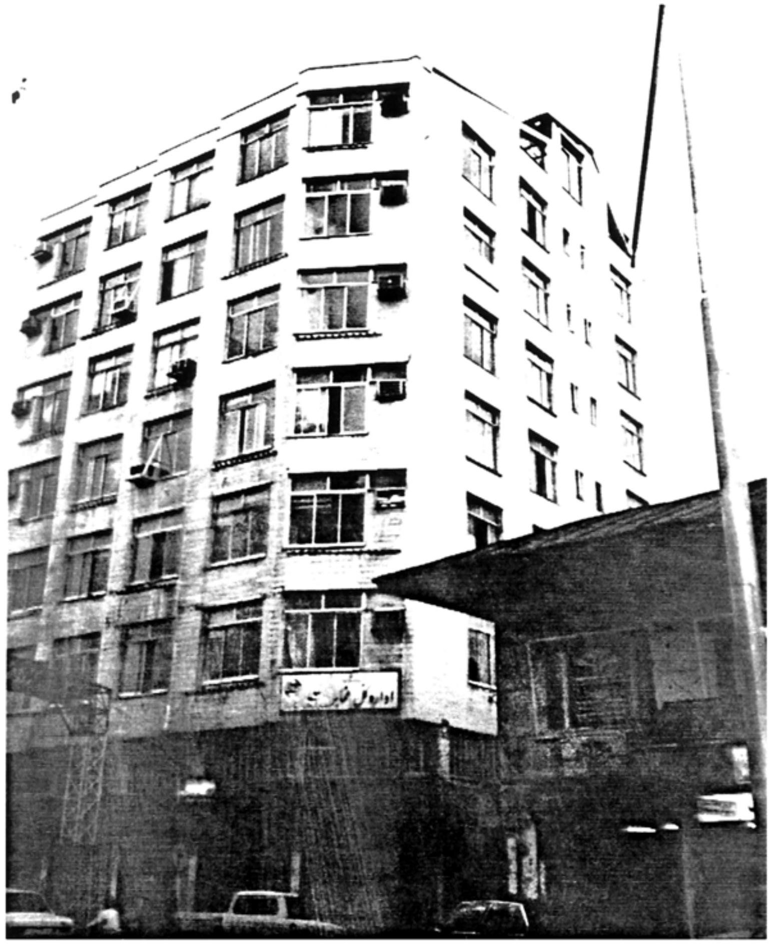
Picture 4.24 Six-storey Steel Frame Building, Rasht
Weak steel column consists of two undersize I-beams

Picture 4.25 Six-storey Steel Frame Building, Rasht
Weak welded joints simply snapped under small stresses

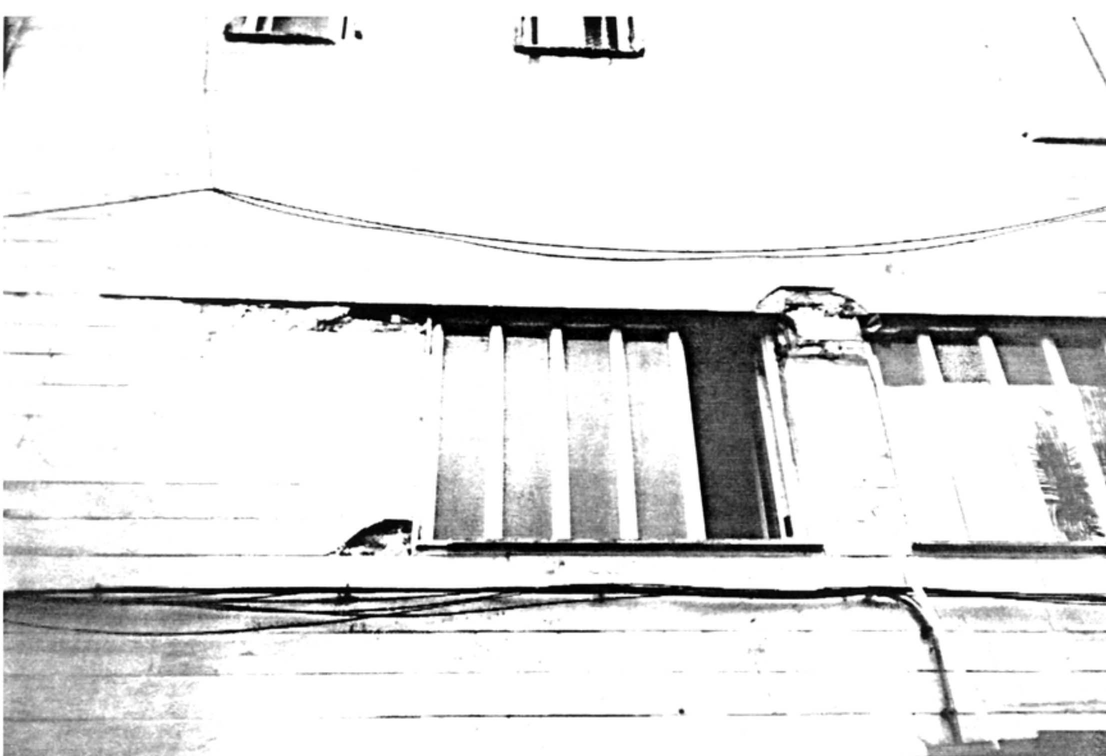
Picture 4.26 Site of a collapsed eight-storey steel frame building. Poor welded joints and weak steel columns the main causes of failure, Rasht

Picture 4.27 Seven-storey White Stone Clad Building, Rasht
A steel frame building with better design and workmanship

Picture 4.28 Seven-storey White Stone Clad Building, Rasht
Collapse of a number of the unsupported infill walls

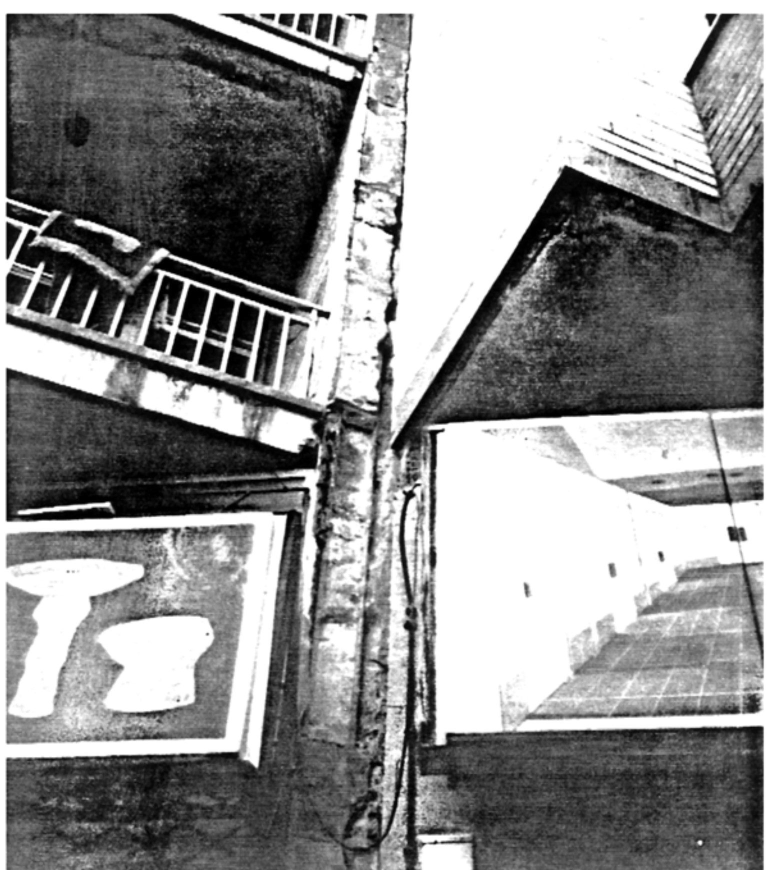


Picture 4.29 Telecom Building, Rasht
A well behaved steel frame building with minor damages



Picture 4.30 Telecom Building, Rasht
Failures in the infill walls and stone cladding

Picture 4.31 4-6 Storey Concrete and Steel Frame Buildings, Rasht
The better behaved concrete frame building also supported the weaker neighbouring steel frame buildings



Picture 4.32 4-6 Storey Concrete and Steel Buildings, Rasht
Separation due to differential responses of the relatively stiff concrete building and the more flexible steel frame building

Picture 4.33 Nine-storey Apartment Block, Rasht
The concrete frame behaved well

Picture 4.34 Nine-storey Apartment Block, Rasht
Shear cracks in the infill walls

Picture 4.35 Eight-storey White Stone Clad Building, Rasht
This well designed and constructed reinforced concrete frame building survived the quake undamaged

Picture 4.36 Eight-storey White Stone Clad Building, Rasht
Minor crack in the stone cladding the only damage in this building

Picture 4.37 Eight-storey Grey Stone Clad Building, Rasht
Another well constructed reinforced concrete building surviving the quake with minor damages

Picture 4.38 Eight-storey Grey Stone Clad Building, Rasht
Collapse of sections of stone cladding the only failure in this building

5 BEHAVIOUR OF OTHER STRUCTURES

5.1 SEFID-RUD DAM

The largest and probably the most important structure in the epicentral area of the Manjil earthquake is the Sefid-rud buttressed concrete dam (Pictures 5.1 and 5.2). It is situated approximately 2 km north west of the town of Manjil where it collects the waters of the Ghezelozan and Sefid-rud rivers. It is an important source for electricity generation for the region.

Sefid-rud dam is a 106m high, 425m long buttressed gravity dam. There are 23 buttresses, each 5m thick; the width of buttresses at the foundation level is about 100m. The slope of the dam on the downstream face is 1 in 0.6 and on the upstream side 1 in 0.4. It has a vertical crown section 14m high and 10.5m wide as illustrated in Fig. 5.1.

The reservoir was almost full at the time of the main event, the water level being 5m below the maximum level. The water outlets consist of two adjacent intake towers at the west end of the dam and 4 sluice gates at two different levels at both the east and west sides. At the time of the visit the reservoir was being emptied through two sluice gates and the water level was 60 to 70m below the crown.

As expected under the circumstances, the drainage of the reservoir started almost immediately after the quake. This was necessary not only to investigate the possible damage to the upstream face of the dam but also to reduce the level of hydrodynamic forces exerted on the dam under subsequent after shocks, as an already weakened dam would be very vulnerable to such after shocks.

The Sefid-rud dam was designed in the 1950's, construction began in 1958 and was completed by 1967. Because of the importance of the structure the seismic safety was an important consideration in design. In those days however seismic design of structures was carried out using the equivalent static approach. The dynamic behaviour of such structures was not fully understood and without the modern computational facilities the dynamic and hydrodynamic forces exerted on the dam could not be accurately calculated. The seismic factor adopted for the equivalent static design of this dam was 0.25 [2]. This is a rather high factor compared to other similar designs of the day, reflecting a conservative approach in design. It is very unlikely that a conservative factor was chosen because of the particular seismic conditions of the site, as there are no references in geological surveys of the day to the presence of a seismic fault crossing the downstream river only 300m north of the dam. Furthermore, if the presence of this fault had been known the dam would almost certainly not have been constructed on this site.

The higher seismic factor enabled the main structure of the dam to resist the high level of ground accelerations during the earthquake. The fact that, judging by failure modes of nearby structures, the stronger component of the quake happened to be almost parallel to the face of the dam also greatly helped it's behaviour during the quake. Unfortunately, there were no seismographs on or in the vicinity of the dam at the time of the main event to record the level of ground accelerations. The nearest accelerograph at Abbar close to the line of the seismic fault recorded maximum acceleration of 0.65 in the east-west direction and 0.2 in the north-south direction. It can therefore be assumed that the accelerations suffered by the dam were in excess of these values.

5.1.1 DAMAGE TO THE MAIN STRUCTURE

Although the main body of the dam behaved well and retained it's integrity, a number of cracks developed mainly in the buttresses but also in the upstream face of the dam. The structural damage visible can be summarized as follows (see also Fig. 5.1):-

1. Horizontal cracks; These were observed mainly in the upper parts of the buttresses at their intersection with the crown (Picture 5.5). Most of the 23 buttresses developed these cracks. The cracks were probably extended to the upstream (reservoir) face of the dam where at the same level some horizontal cracks were just visible. The cracks, associated in places with spalling of concrete at the surface to a width of 7 to 10cm, were probably caused by the out-of-plane bending (overturning) action of the crown under the North-South component of the quake. As the drainage of the reservoir was revealing more of the upstream face of the dam, more horizontal cracks could be seen at lower levels.

FAILURE MODES OF THE SEFID-RUD DAM

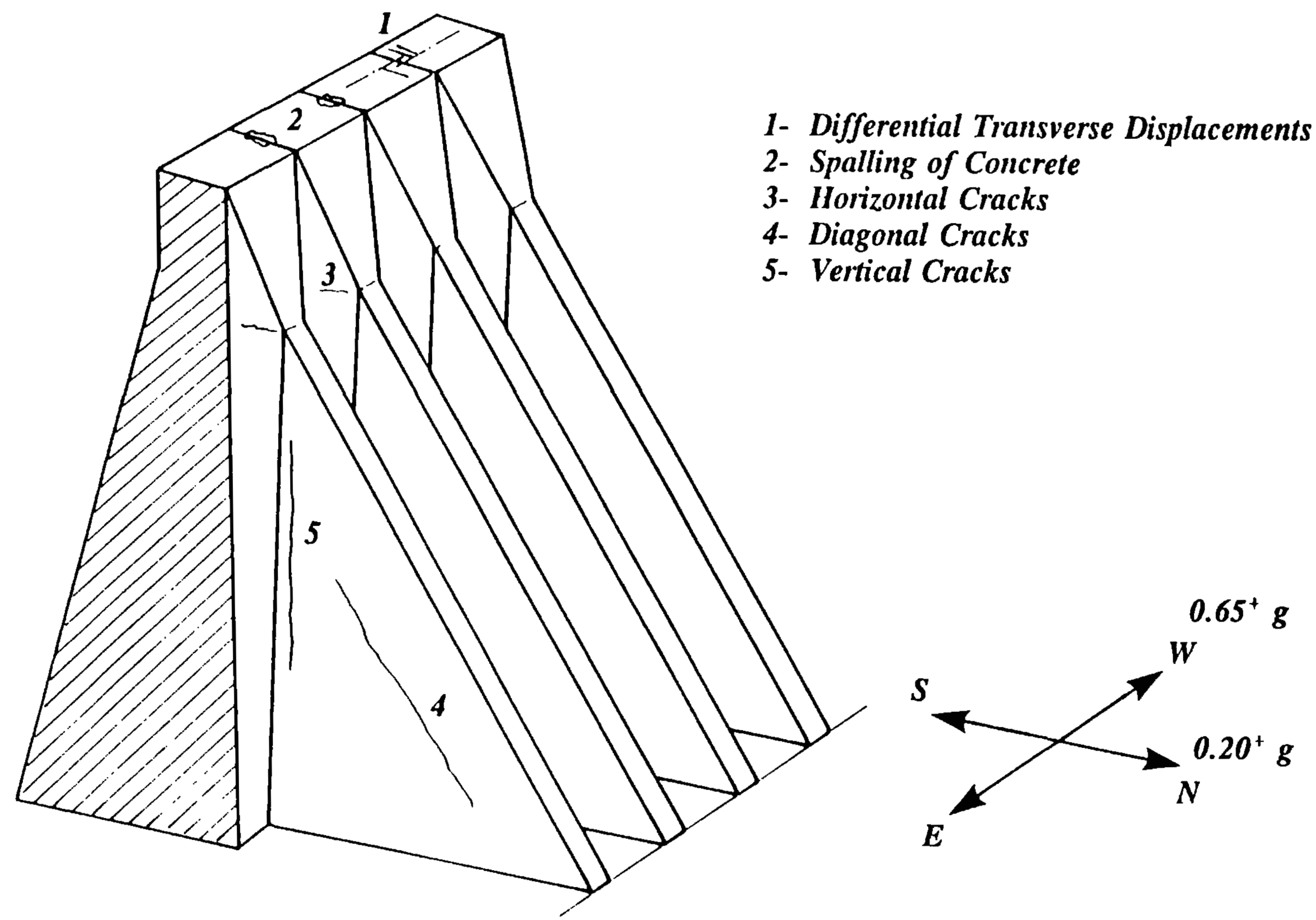


Fig. 5.1

There were also a number of horizontal cracks at the base of the buttresses both parallel and perpendicular to the dam.

2. Diagonal cracks; In a few central buttresses there were also a number of diagonal shear cracks. As the height of the dam in its central sections is more than the end sections the overturning and flexural responses of the structure under the North-South component were higher in those sections, hence resulting in shear failures in the supporting buttresses.

3. Differential displacements of the dam sections; The dam sections are separated from each other by construction and expansion joints. Under the horizontal ground motion some lateral differential displacements developed between these sections. An investigation carried out by the engineers from the Ministry of Energy revealed a maximum of 50mm difference in the alignments of the bench marks on some of these sections. Considering the overall dimensions of the dam, such relative displacements would be within the expected range.

4. Pounding of sections; Evidence of pounding of the dam sections against each other could be seen on the crest in the form of spalling of concrete at the joints (Picture 5.7). The spalling could be seen in most joints, however, the extent of the pounding damage at the interface of the adjoining sections could not be investigated. The type and size of seismic joints, if they were at all a consideration in design of the dam, were inappropriate to mitigate the damaging effects of pounding.

5.1.2 NON STRUCTURAL DAMAGE

. The main visible non structural damage to the dam was in the long unsupported parapet of the north side. Flexural failure at the central section of this reinforced concrete wall in the form of vertical cracks together with horizontal crack at the base due to the bending failure were responsible for the collapse of two relatively large sections of the wall (Picture 5.9).

. Subsidence of the fill at both ends of the dam adjacent to the concrete section of the dam

(Picture 5.9) caused by compaction of the loose fill under ground vibration.

- Shear and overturning failure in the majority of concrete and stone masonry guard blocks. Most of the blocks were thrown off their foundations by up to two meters in the west direction.

- Destruction of the guard post at the east side and guard house in the west side of the dam due to rock-fall. The reinforced concrete guard house was completely destroyed under the falling rocks (Picture 5.10), some a few meters in diameter, causing at least one fatality.

- Rock falls were evident at both east and west sides of the dam, some of which had blocked the access roads to the dam before being moved aside.

- Due to the shortage of time, a visit to the dam's systems and facilities could not be arranged. Considering the level of ground acceleration at the site, damage to the facilities and possibly the turbines can not be ruled out.

5.2 TARIK DAM

Another dam situated in the area affected by the Manjil earthquake is the Tarik diversion dam (Picture 5.11). This dam collects the water of the Sefid-rud river diverting part of it through a long underground channel to agricultural lands in Fuman some 70 km away.

The total length of the dam is 350m, of which the concrete section measures 230m, consisting of 10 concrete piers, 3m thick and 22m high. The piers measure 20m at the crest and 54m at the base. The flow of water is controlled by semi-cylindrical steel gates 15m long and about 8m high (Picture 5.12). The movement of these gates is controlled individually by an automatic pulley system. The bridge deck of the dam runs on the north side and level with top of the piers. The deck rests on columns supported by the piers (Picture 5.13).

The dam is situated some 40 km north of the epicentre of the Manjil earthquake. As a result, the accelerations experienced by the dam were less than those suffered by the Sefid-rud dam.

The intensity of the earthquake around the dam was put as VII (MSK) [2].

A close inspection of the upper parts of the piers above the water line revealed no direct failure of concrete under earthquake loading. The steel gates joined to the concrete pier via rotating steel arms also stayed in place without damage. At the time of the main event, two of these gates were open allowing the water through. This must have greatly reduced the high levels of impulsive hydrodynamic forces on the steel gates induced by the movement of dam against the mass of reservoir during the earthquake.

The only damage visible in this dam was the local cracking and spalling of concrete at the top of the piers in close contact with the bridge deck (Picture 5.14). This could be seen in almost all the piers. The failures were caused by the pounding action of the flexible bridge deck against the relatively rigid piers under ground shaking. The distance between the bridge deck and the concrete piers was not sufficient to accommodate the relative flexible responses of the two almost independent sections of the dam.

5.3 SANGAR DAM

Another dam in the area which suffered some damage during the earthquake is the Sangar diversion dam near the city of Rasht some 50 km from the epicentre. This dam, which is similar to the Tarik dam, consists of 13 steel gates. The gate movements however are controlled by counter balancing large concrete blocks in such a way that when the gates are shut (down) these blocks are in a raised position. During the earthquake the ground motion caused dislocation of some of the controlling cables off the pulleys. This resulted in the sudden lowering of the concrete blocks and therefore raising of the steel gates. In total six gates were opened in this way. Under further ground shaking two of the raised gates were reportedly thrown off their supports to a distance of approximately 200m downstream [2]. After the earthquake the replacement gates were quickly put into position, consequently during the visit no earthquake damage to this dam could be seen. The behaviour of the Sangar dam indicates the unsuitability of such designs for earthquake loading as the strong cantilever action of the heavy concrete blocks or the steel gates in a raised position under the horizontal acceleration would invariably result in failure. Apart from the mentioned failure of the gates, the earthquake caused no apparent structural damage to the dam itself.

5.4 RUDBAR CONCRETE BRIDGE

The Rudbar concrete bridge is a 190m long reinforced concrete bridge spanning across the Sefid-Rud river some 3 km north of the town. It consists of 5 piers and six deck sections each 30m long. The end sections are supported directly on two large end piers. The width of the bridge deck is 10.5m and it's height is about 10m. The piers are T shaped and are approximately 8m high and 10.5m wide at the top (Picture 5.15). The deck consists of four concrete beams running along the length of each section and simply supported on the piers. The four beams are joined together at 9.5m intervals along their length by deep concrete tie-beams 20cm wide. The above arrangement provides the stiffness of the bridge deck, the deck itself consists of a 35cm thick reinforced concrete slab. As far as could be gathered, the bridge deck sections were directly placed on the piers separated only by 20mm thick rubber pads. There were no rubber pads at the vertical gaps between the end sections and the piers.

Considering the high level of ground acceleration around Rudbar this concrete bridge behaved reasonably well during the earthquake. No transverse displacements of the simply supported bridge deck were apparent. The structural damage visible in the bridge are as follows:-

1. The spalling of deck concrete at both ends of the bridge, caused by the pounding action of the relatively flexible and free standing bridge deck against the rigid end-piers (Picture 5.16). The size of the vertical gap between the two sections was evidently very small. Insufficient gap together with lack of rubber pads or similar shock absorbent elements in the joints were the main causes of failure in these locations.

2. The second form of failure could be seen on the bridge deck at almost every joint between the deck sections. At these joints the pedestrian concrete paving had buckled (Picture 5.17). This was caused by the pounding action of the bridge deck sections against each other. Any possible pounding damage to the bridge deck itself could not be verified. It is probable that the presence of bitumen asphalt between the adjoining concrete deck sections prevented damage to the deck.

Other non structural failures in and around the bridge include the collapse of a reinforced concrete retaining wall next to the south end-pier (Picture 5.18) and an uncharacteristic shear failure in a masonry column of the bridge guard house (Picture 5.19).

5.5 STEEL BRIDGES

There were a number of small steel bridges in the epicentral area of the earthquake. Three of these bridges were investigated non of which showed any signs of failure or damage. These included Lushan bridge (Picture 5.20), Manjil bridge (downstream of the Sefid-rud dam) and Rudbar bridge (Picture 5.21). The bridges, all of a similar design and construction, have varying lengths of between 70m and 100m and widths of between 8.0m and 12.0m.

The Lushan bridge is a single-span bridge, hinged at both supports whereas the Manjil and Rudbar bridges are double-spanned with an off-centre supporting pier. The two latter bridges, as far as could be ascertained, were hinged at, at least, one end-support and simply supported on the central pier. In all the three bridges the stiffness of the long span is provided by vertical steel trusses.

The ability of the steel bridges to withstand the forces of the earthquake can be attributed to two main factors:-

i) The bridges are highly flexible structures with low fundamental frequencies of vibration. As is already mentioned, due to the mountainous nature of the epicentral area the frequencies of strong ground vibration were relatively high. As a result the dynamic amplification was much reduced during the earthquake.

ii) The bridges appeared well designed and constructed with all the joints bolted and without any signs of pre-earthquake weakness due to corrosion or other damage. As a result the load carrying elements behaved well under the reduced earthquake loading.

5.6 LUSHAN OLD CEMENT FACTORY

This 300 tonne capacity cement factory was constructed some 30 years ago. It is the smaller of two cement factories in and around the town of Lushan some 12 km south of epicentre (the other factory is a new 2100 tonne capacity factory which suffered no serious damage during the earthquake).

Although the factory was operational at the time of the visit it had suffered extensive damage particularly in it's associated industrial and residential buildings. Save for the main buildings housing the grinders, the furnace and the large cylindrical steel storage silos, all the other buildings in the factory suffered varying degrees of damage. The damage in the industrial buildings totalling some 1,500m^2 in area was generally in the form of partial collapse of non load-bearing masonry walls (Picture 5.23) or in collapse of the corrugated steel roofs. According to the factory manager some 1100m^2 office buildings and over 25,000m^2 residential buildings belonging to the workers and staff were damaged between 30% and 70%. The most important of the industrial buildings was the 500m^2 laboratory within which the majority of the instruments and equipment were damaged either due to the collapsing walls and roof or directly as a result of ground shaking.

The damage to the main parts of the factory were as follows:-

i) The rotating 300 tonne furnace was thrown off position along it's long axis for over 1.0m. Luckily, it did not roll over it's concrete support as the component of the earthquake in that direction was weaker (Picture 5.24). The furnace was re-positioned on it's supports shortly after the earthquake. However horizontal cracks could be seen in both of it's concrete supports. These cracks developed as a result of the flexural failure of the 2.0m thick, short, reinforced concrete legs of the supports under the inertia force of the furnace.

ii) Although no damage was visible in the large cylindrical steel cement silos the thick reinforced concrete foundation bases of these silos had also developed similar cracks as described in (i) above.

iii) Damage to one of the two large cylindrical grinders, which was still out of action at the time of the visit (Picture 5.25).

Also the main power supply to the factory was cut during the earthquake as the falling rocks and land slides damaged the power lines and pylons.

Because of the importance of the factory, particularly in view of the increased local need for building materials such as cement for the post earthquake reconstruction, the authorities considered it imperative to recommission the factory as soon as possible. As a result, despite loss of life amongst the workers and staff, the partial destruction of accommodation and offices and the damage to the main sections, the factory was operational again within a few days of the main event.

For reasons mentioned above the seismic safety of such plants as cement factories should be a prime consideration and in particular, the secondary response of important elements and installations under the earthquake loading should be given due attention.

In design and construction of the 30 year old Lushan cement factory, seismic safety was evidently not a consideration. About two or three miles away from this factory the much larger new cement factory was better equipped to withstand the earthquake loading. Although due to the shortage of time a visit to the latter plant could not be made, there were no reports of damage to that factory.

5.7 CONCRETE WATER TOWERS

The only large engineered structure which completely failed and collapsed in the Manjil quake was a 47m high reinforced concrete elevated water tank. This 20 year old tower (No 1), situated in the centre of the city of Rasht was not designed to resist earthquake forces. Two other similar water towers in the outskirts of the city however fared better and, although suffering some damage, did not collapse. The construction of these two towers (Nos. 2 and 3), identical in design and very similar to tower No 1, had just been completed and were empty at the time of the quake. This probably accounts for their better behaviour as compared to the ill-fated tower No 1.

WATER TOWER No. 1

This tower was situated in the grounds of the offices of the local water authority, some 6 to 7m west of the main building and only 20 to 30m away from the buildings of a nearby large hospital. Fortunately the failed tower was thrown away from the above buildings and collapsed in the central court of the compound (Picture 5.27).

The tower consisted of two thin-walled prestressed concrete cylindrical water tanks with a combined capacity of 1500m^3. The tanks were supported by a 25.50m high, 6.0m diameter and 0.30m thick reinforced concrete shaft, itself placed on a conical, double walled hollow foundation arrangement. The wall of the shaft was reinforced by two sets of 14mm dia. bars,- a reinforcement arrangement barely adequate to resist wind-induced stresses let alone horizontal ground loadings. As the architectural form of the tower suggests, its ability to withstand even a mild earthquake, as was the intensity of ground shaking in Rasht, was non-existent.

The tank was reported to be one third full at the time of the quake. Considering the weight of the tank and the centre of gravity of the tower (some 40m above the ground level) the earthquake induced bending stresses in the thin-walled, slender shaft of the tower would have been extremely high.

Another factor contributing to the dynamic weakness of the tower under the circumstances lies in its relatively low fundamental frequency of vibration. As has already been mentioned, the strong frequency range of the earthquake in Rasht was relatively low. As a result high response amplifications occurred in taller buildings and structures which were generally more flexible than smaller buildings. Considering the above two factors, the level of ground accelerations required for failure of the shaft and collapse of the tower may not have been very high .

The mode of failure of the water tower may be reconstructed from the debris as follows (see also Fig. 5.2);-

1) Possibly at the onset of the earthquake the bending stresses in the shaft exceeded the tensile capacity of the reinforcement bars, leading to bending failure in the shaft. The position of the bending crack, to judge from the remains of the shaft, appears to have occurred at about its mid-height.

2) The inertia force of the quake then forced the tank and upper section of the shaft westwards, opening the crack in the process and pushing the lower part of the shaft in the opposite direction.

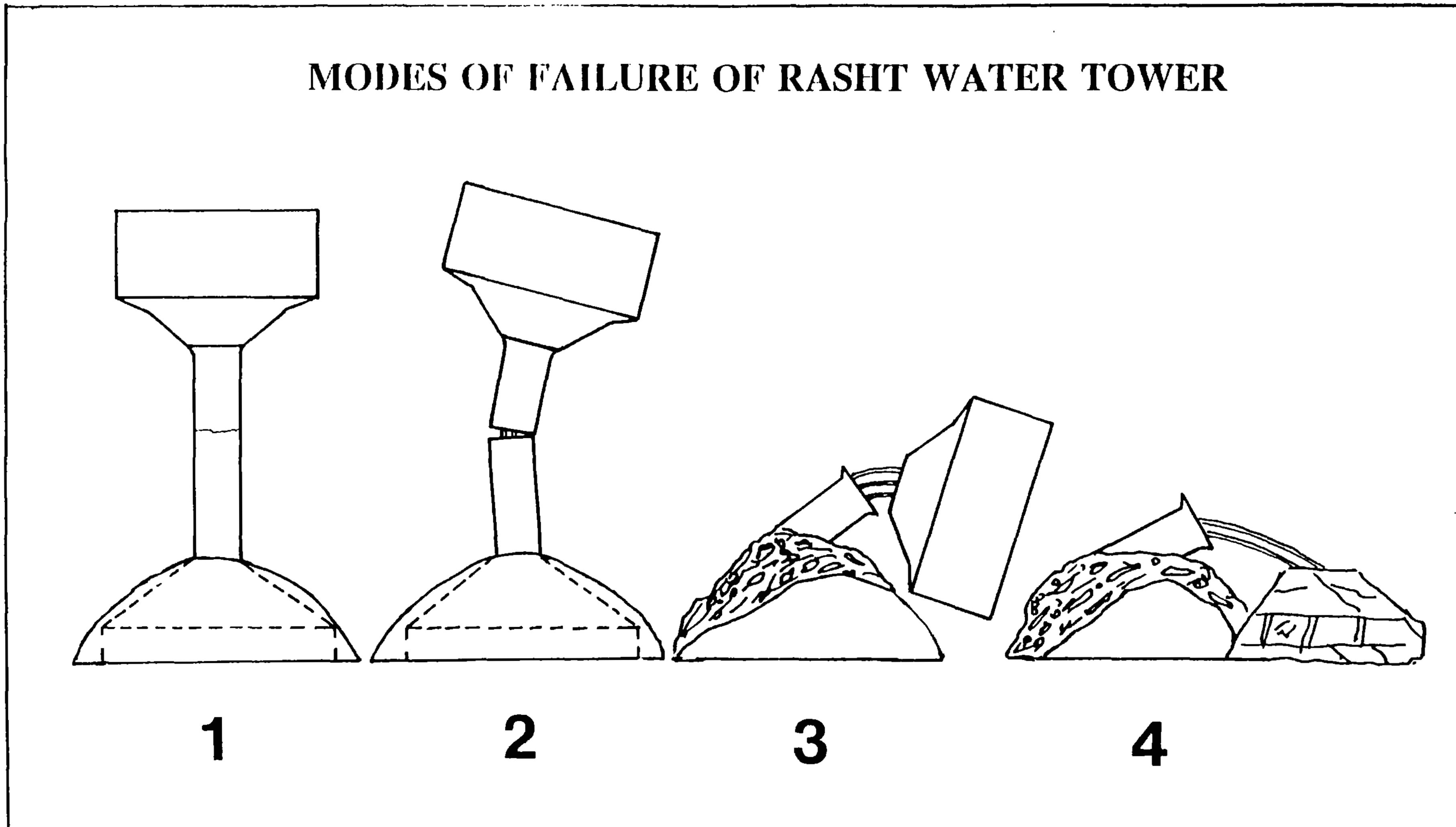


Fig. 5.2

3) Under the heavy compression and bending forces the lower half of the shaft crushed and disintegrated into a heap of broken concrete. The tank and upper part of the shaft, meanwhile, followed a free-fall mode , during which time the tank was separated from the shaft, turning over in process.

4) The final damage to the tank and upper part of the shaft was as a result of impact with the ground. On this impact the half of the upper shaft coming into contact with the ground also crashed whereas the other half remained relatively intact (Pictures 5.28 and 5.29). The fall of the heavy tank on the other hand caused multiple bending, shear and buckling failures in various concrete sections of its structure (Picture 5.30).

WATER TOWERS No 2 AND 3

The structural parts of these two identical elevated water tanks had just been completed when the earthquake struck (Pictures 5.31 and 5.32). They were designed by the same designers and built simultaneously by the same contractors. The tanks each have a water capacity of $2500m^3$. The height of tower from ground level is about 50m. Of this 24m is the length of the cylindrical shaft. The shafts of these towers are thicker and larger in diameter than tower No 1, being 0.5m and 8.0m, respectively. Both towers No 2 and 3 are supported by pile foundations. There are 24, 1.20m diameter, 30.0m long piles supporting each tower. A schematic illustration of the towers is shown in Fig. 5.3. Judging by the calculation files, copies of which were kindly supplied by the designers and the photographs showing the construction procedures, the design and construction of these two towers had been carried out professionally. However, one major consideration lacking in design was the effects of seismic loading.

Failure Mode of Tower No 2

Bending failure in the form of horizontal cracks could be seen all around the circumference of the shaft (Picture 5.33). The level of this continuous ring-crack varied at different sections of the shaft's circumference between 1.45m and 2.25m above the base of the shaft (Picture 5.34). The adjoining horizontal cracks running at different levels were joined together by short vertical cracks. One exception to this cracking pattern was an inclined form at the staircase (The reinforcement arrangement in the shaft adjacent to the staircase accounts for this). As already

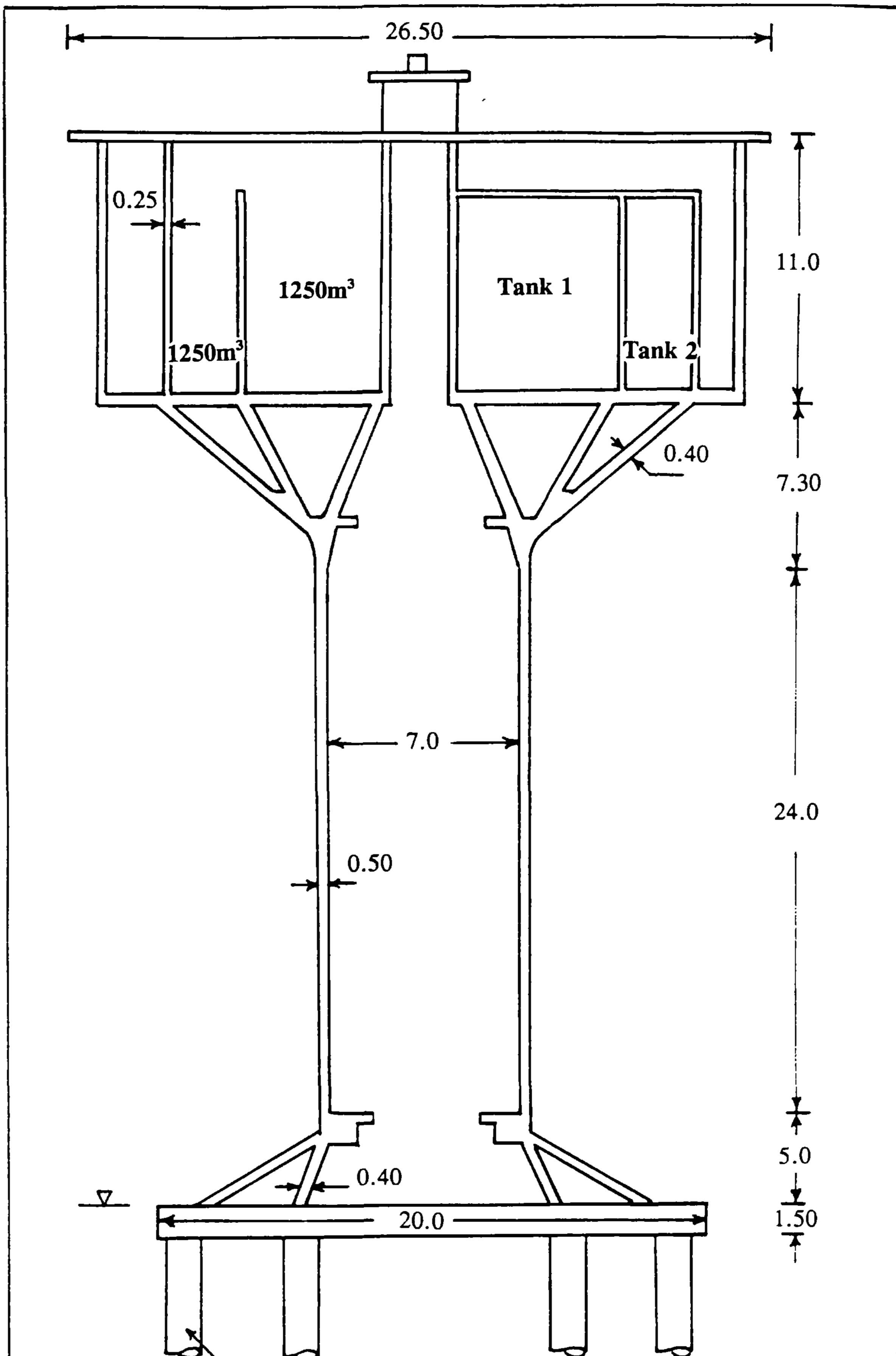


Fig. 5.3 A Schematic Illustration of Tanks No. 2 and 3.

mentioned, the cracks developed due to the high bending stresses in the section of the shaft with the weakest flexural capacity. In fact the crack line coincides with the construction joint between the base and the shaft, where the reinforcements from the two sections overlap.

According to the engineer in charge, examination of the tower after the main event of June the 20th revealed only a few hair-line horizontal cracks in the shaft. However, under the action of many aftershocks which followed the main event, the extent and size of the cracks grew to encircle the whole shaft.

Failure Mode of Tower No 3

Considering that towers 2 and 3 are identical in design and construction procedures, it is not surprising that their earthquake responses were also identical. The main failure of this tower was similarly in bending. The resulting horizontal crack was in the same region of the shaft as that observed in tower 2. However, unlike the crack in tower 2, this crack continued at a constant level of 1.25m (above the base of the shaft) around the circumference. Another form of failure, not seen in tower 2, was a number of vertical cracks in the shaft, starting from the horizontal crack-ring, running upwards for about 2.0m. Six or seven of these vertical cracks could be seen in one half of the shaft's circumference at intervals of two to three meters. The vertical cracks developed, according to the resident engineer, during the last strong aftershock. Excess hoop stresses developed in the shaft under bending vibration could be responsible for the formation of these vertical cracks.

5.8 STEEL LIQUID STORAGE TANKS

A number of elevated water tanks similar to the one shown in Picture 5.35 were seen in the affected area. None of those investigated showed any signs of failure except the water tank at Manjil, at the epicentre of the quake, in which some of the steel tie-bars had snapped under earthquake loading. Unlike the elevated concrete towers discussed in 5.7, the steel water tanks of the above design are well suited to resisting strong earthquake loading. The main reason for the tank's seismic strength is in its support arrangement. The earthquake-induced inertia forces of the tank can be safely transmitted, in tension and compression, through the steel columns and cross-bracing into the ground. In the event of increased earthquake loading on the tower the tie-bars may fail, causing a change in the dynamic characteristics of the tower and as a result reducing the earthquake loading. Behaviour of the steel water tower in Manjil demonstrated the above mode of response.

The earthquake performance of ground-based steel liquid storage tanks was also favourable. Examples of this were examined in one of Rudbar's petrol stations. None of the three small to medium size cylindrical petrol tanks in that station had failed (Pictures 5.36 and 5.37), while most of the low-rise masonry building around them were completely destroyed. The good performance of these tanks owes a great deal to the dynamic characteristics of both the steel shells and the earthquake. As far as an empty tank is concerned, its flexibility and low weight made it well resistant to high frequency earthquake loading. As for the hydrodynamic pressures exerted on the shell of the full tanks, the low response amplitudes of the shell also resulted in reduced impulsive hydrodynamic forces in the liquid [5]. The low frequency sloshing modes of the liquid also were not strongly excited by the higher frequencies of ground vibration in Rudbar. The only damage associated with these tanks was in their rigid stone or brick masonry footings, some of which were crushed under high compressive loads (Picture 5.36).

5.9 GENERAL OBSERVATIONS

5.9.1 Earthquake Design

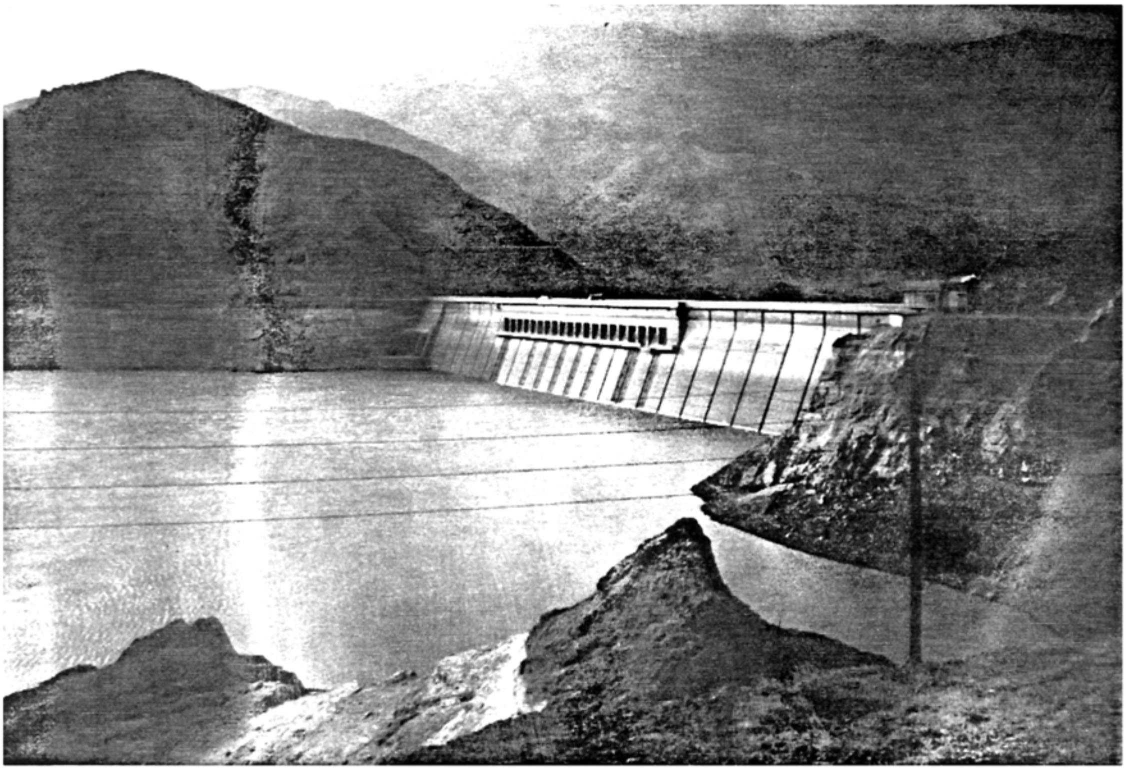
Most of the structures discussed in section 5 were not designed against seismic forces. Considering that these are major structures situated in an area known to be prone to severe earthquakes, lack of seismic considerations in design is surprising. The majority of these structures were however designed in the fifties and sixties when earthquake awareness in Iran was almost non-existent. After the devastating effects of a number of strong earthquakes in recent years (prior to Manjil quake) the earthquake awareness of the authorities and public alike still appears to fall far short of the real need. A seismic code of practice has been in existence since 1968. Nevertheless, for such recent constructions as the concrete water tanks in Rasht which are required to comply with the Iranian seismic code, the complete lack of compliance is a proof of such short-falls.

5.9.2 Seismic Joints

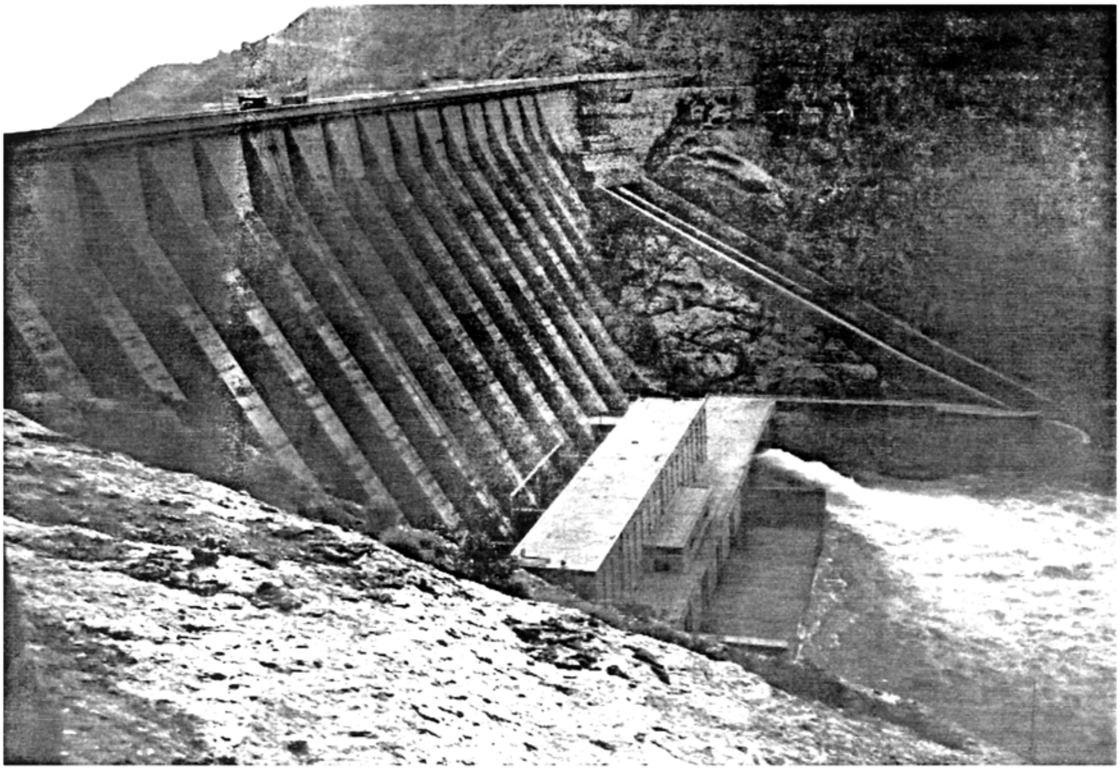
One of the main forms of failure noted in larger engineered structures was caused by pounding of adjoining sections and elements of the structure against each other. Such failures were observed, as mentioned previously, in the Tarik dam, the Rudbar concrete bridge and the Sefid-Rud dam. Although these structures had been designed to withstand the forces of earthquakes, nevertheless, the type and size of seismic joints separating different parts of the structure were inappropriate. Inadequate seismic joints is not the problem of the above structures alone, but the majority of earthquake designed structures around the world have insufficient seismic joints. This is because the interaction between neighbouring elements or structures has been largely ignored. In recent years, the pounding failure has become a common observation in many structures subjected to earthquakes and subsequent research in this area has shown that the codes of practice do not adequately provide for the design of such joints.

5.9.3 Secondary Elements and Systems

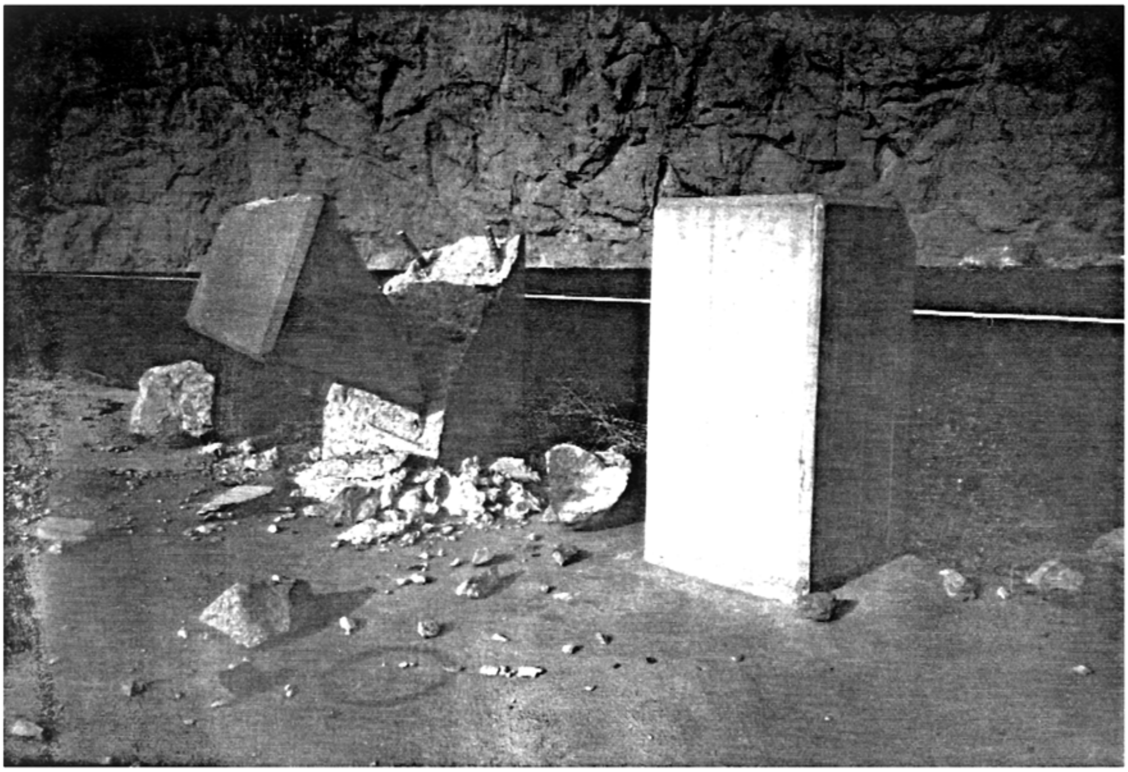
The behaviour of several important engineered structures during the earthquake illustrates the point that in seismic design of lifeline structures the safety and strength of secondary structures, systems or equipment is as important as the integrity of the main structure itself. The interaction between the bridge deck and the main structure such as was observed in the Tarik dam; the design weaknesses in steel gates as noted in the Sangar dam; and damage to equipment and facilities of the cement factory all reinforce this point.



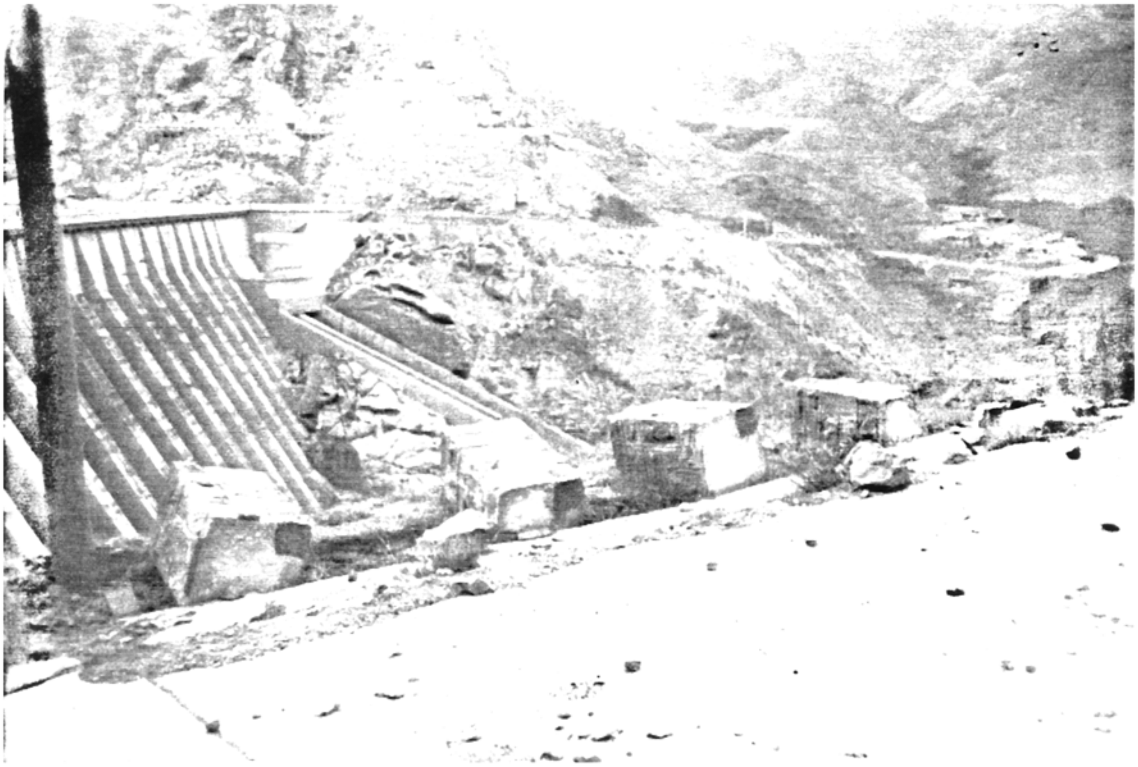
Picture 5.1 Sefid-rud Dam, Manjil, Upstream view



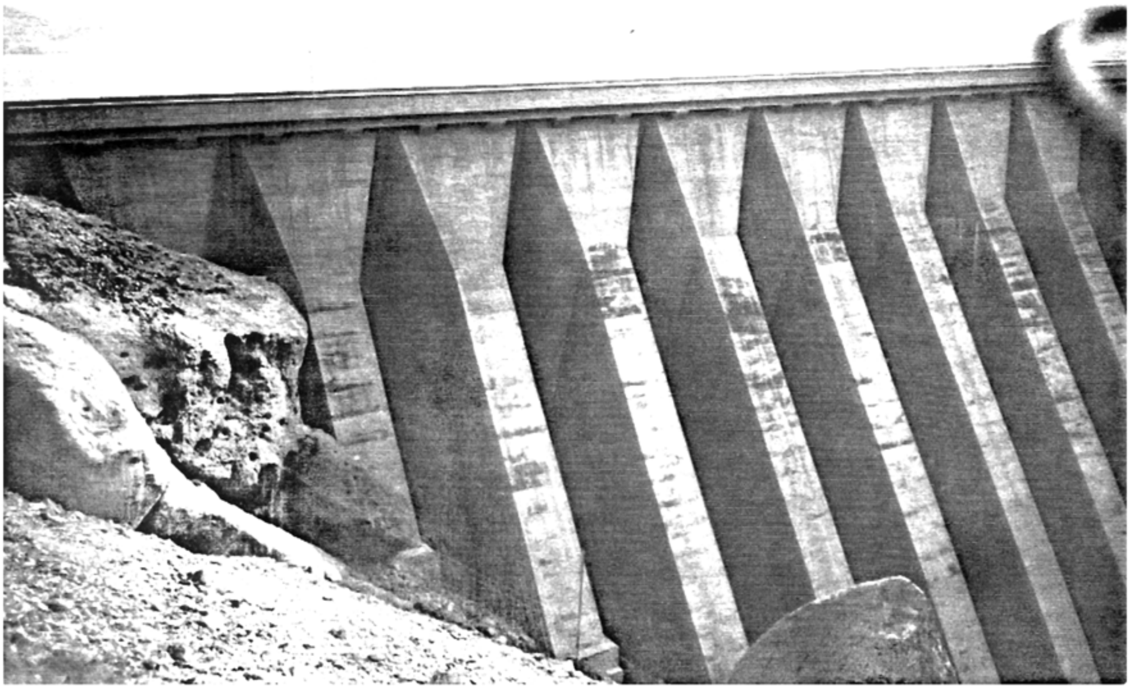
Picture 5.2 Sefid-rud Dam, Manjil, general view of downstream



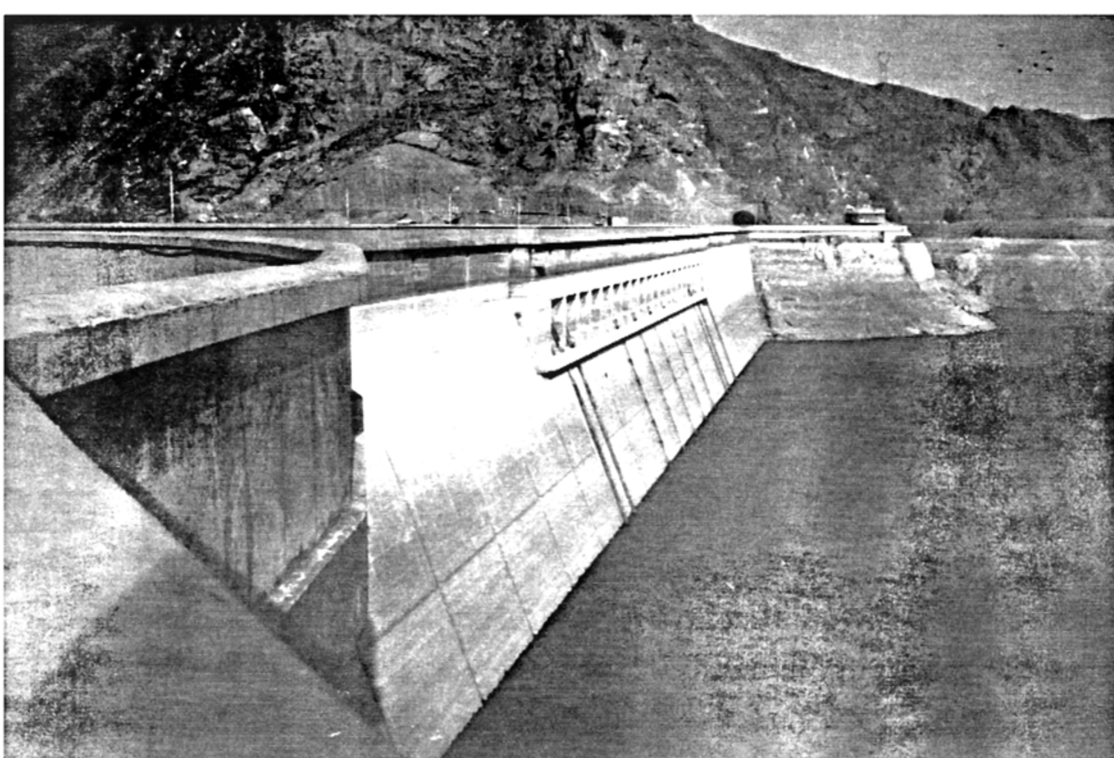
Picture 5.3 Sefid-rud Dam, Manjil
Shear failure in the reinforced concrete block at the side of the dam



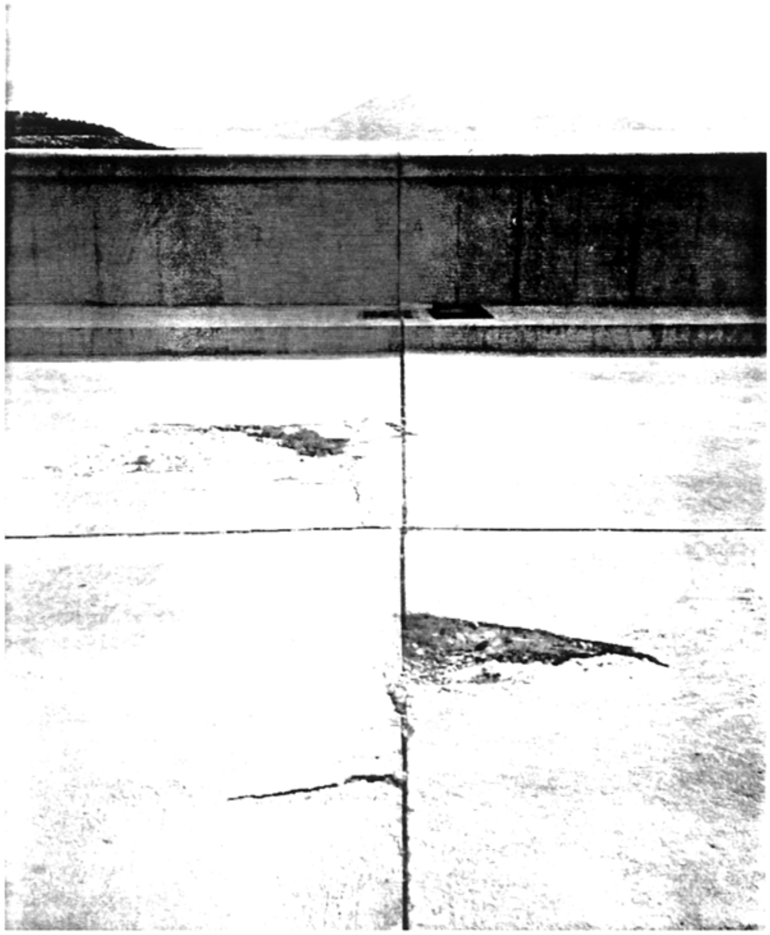
Picture 5.4 Sefid-rud Dam, Manjil
Overturning failure of stone masonry guard blocks



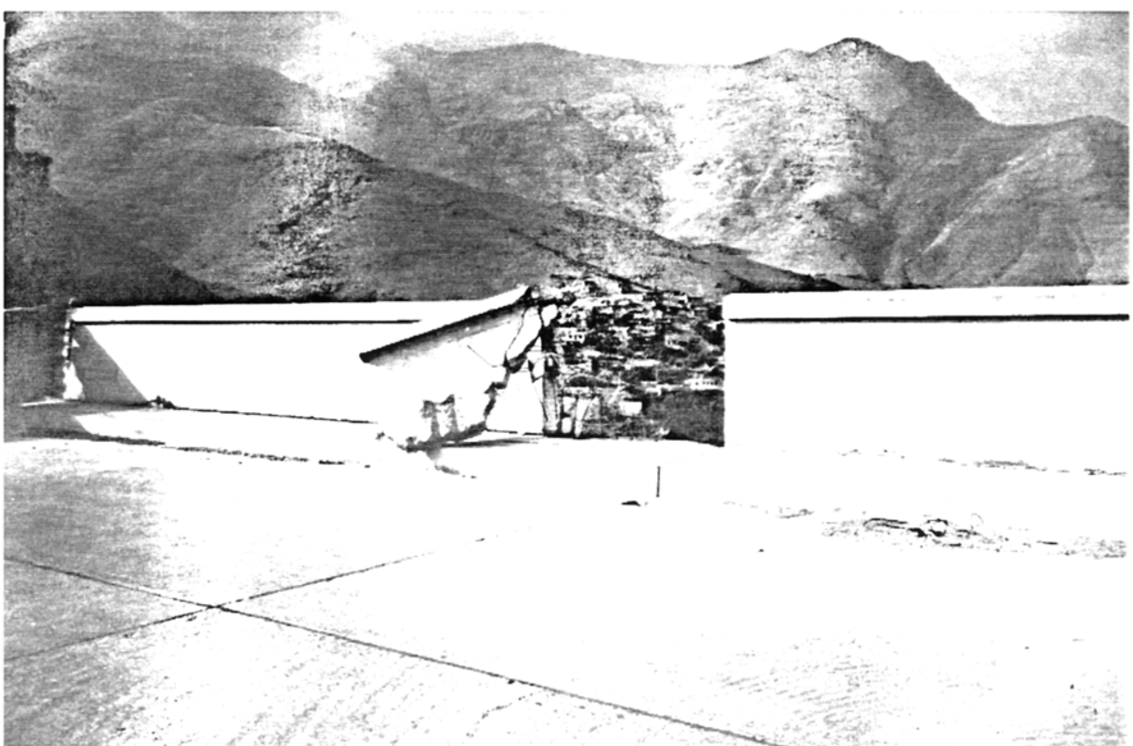
Picture 5.5 Sefid-rud Dam, Manjil
Horizontal cracks at the base of the crown and vertical cracks in buttresses not visible in the photograph



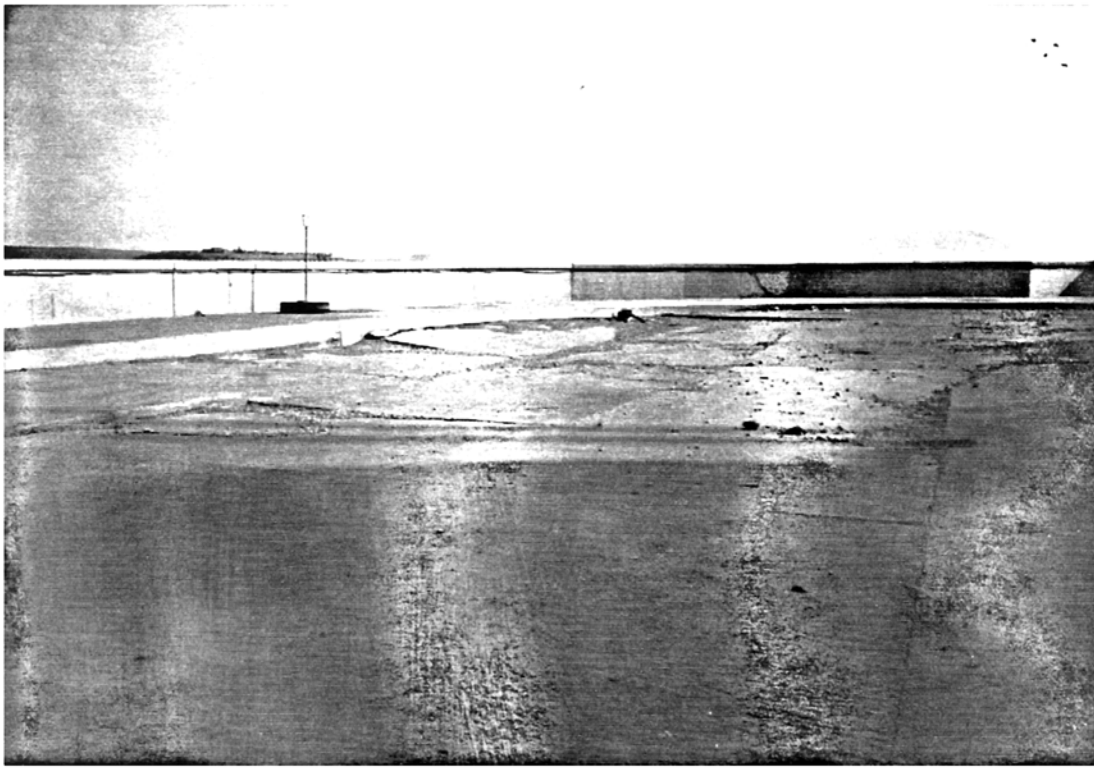
Picture 5.6 Sefid-rud Dam, Manjil
Some horizontal cracks can be observed in the upstream face of the dam



Picture 5.7 Sefid-rud Dam, Manjil
Spalling of concrete at the crest caused by pounding action of adjoining sections of the dam

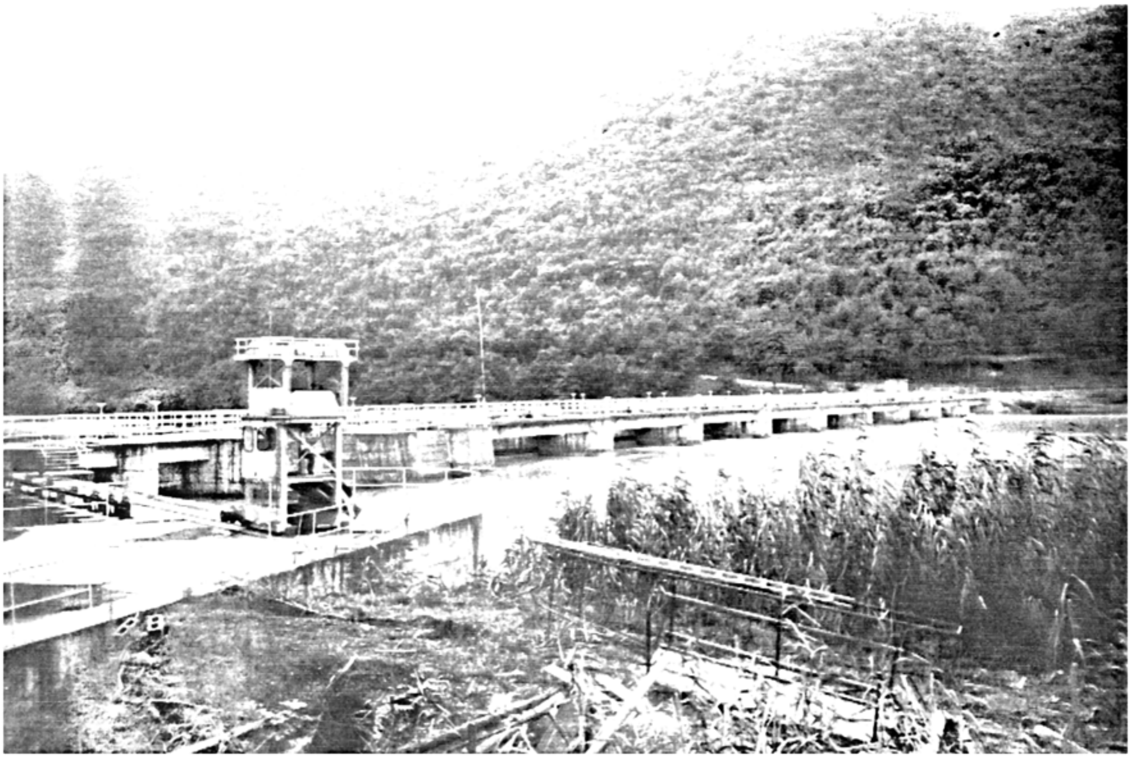


Picture 5.8 Sefid-rud Dam, Manjil
Flexural failure in the long, unsupported reinforced concrete parapet wall

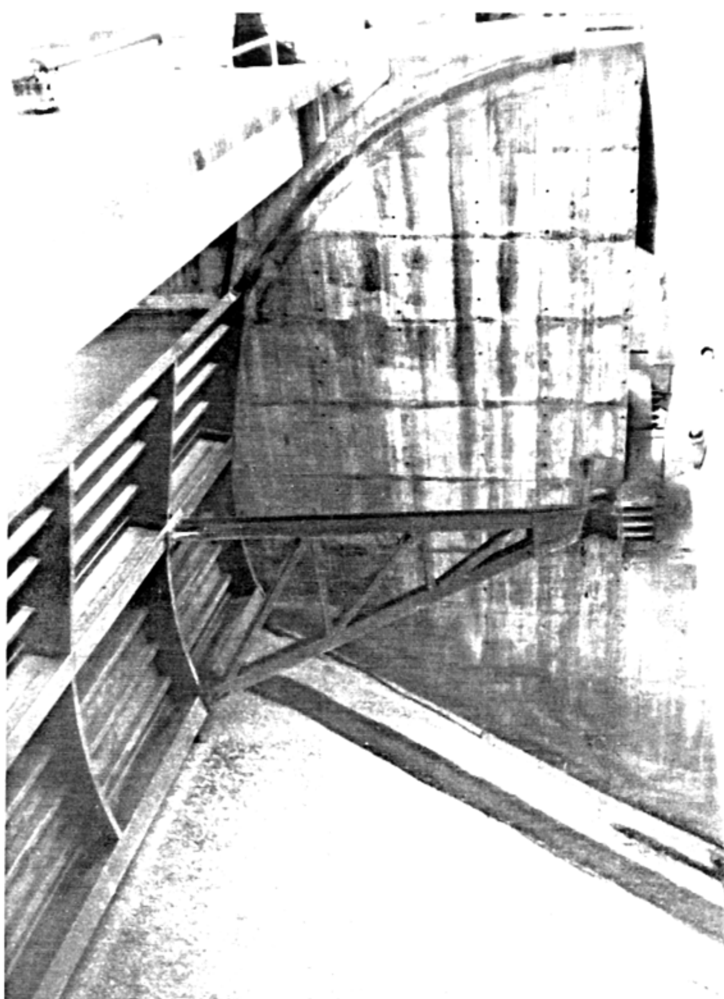


Picture 5.9 Sefid-rud Dam, Manjil
Subsidence of the fill at the west end adjoining the concrete section of the dam

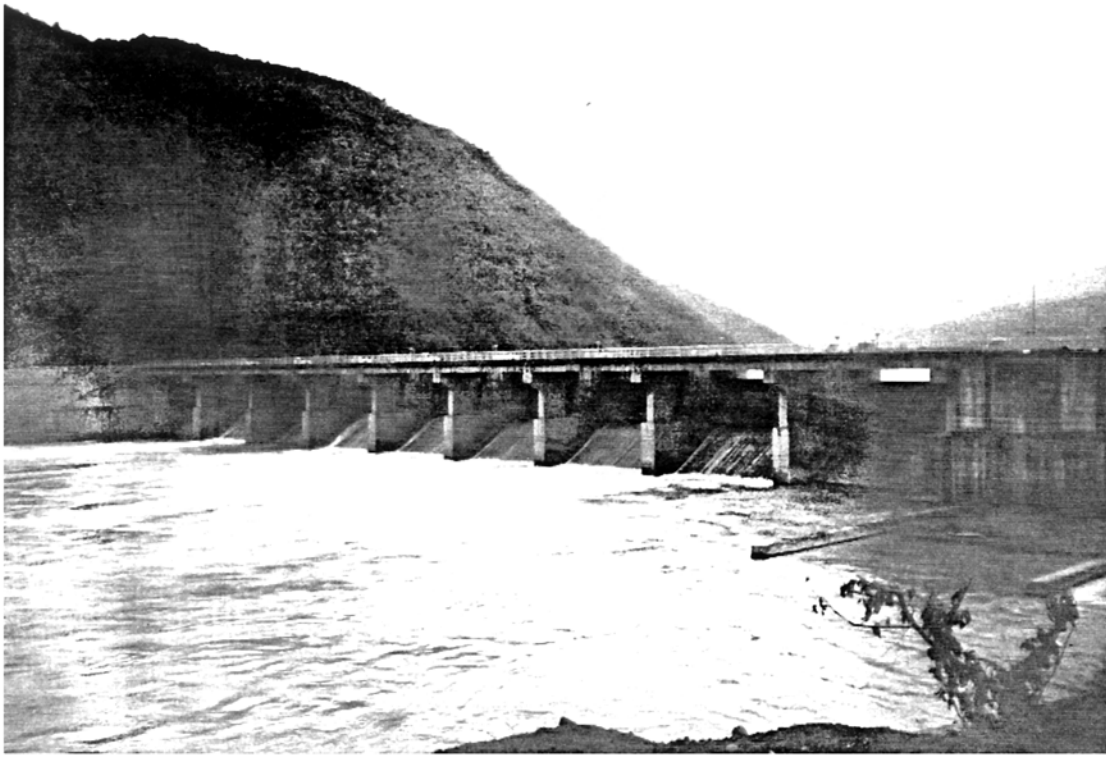
Picture 5.10 Sefid-rud Dam, Manjil
Destruction of the reinforced concrete guard house under the secondary action of the rock fall



Picture 5.11 Tarik Diversion Dam, Saravan, Upstream view



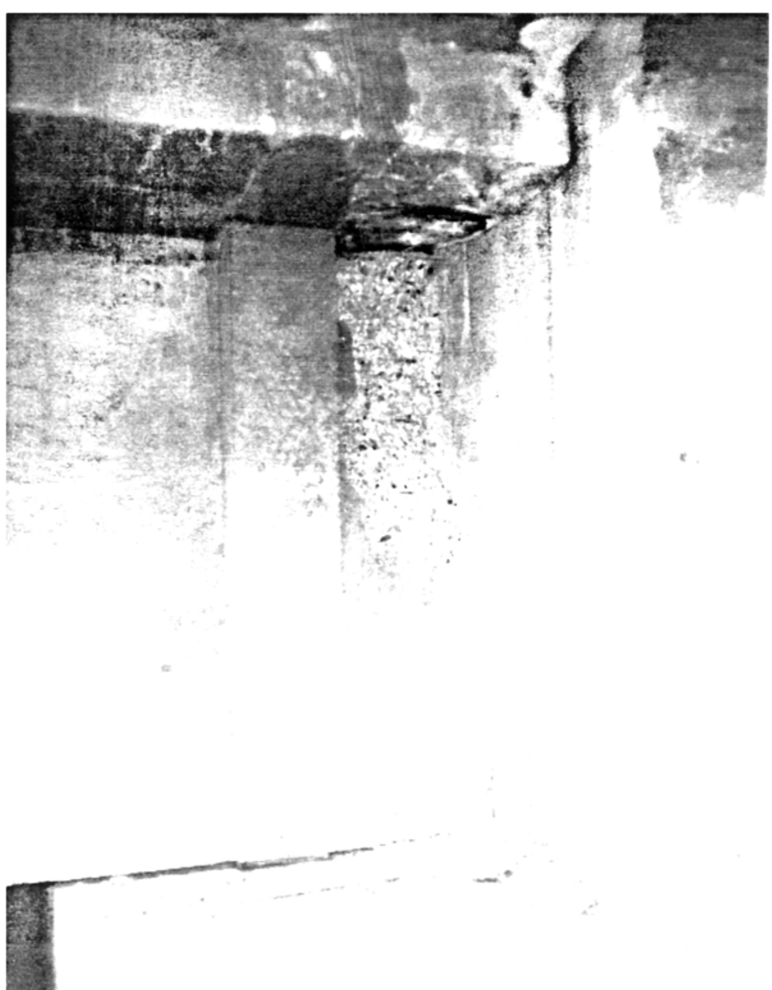
Picture 5.12 Tarik Diversion Dam, Saravan
The gate's arm and pier connection not designed to
withstand the earthquake induced hydrodynamic forces



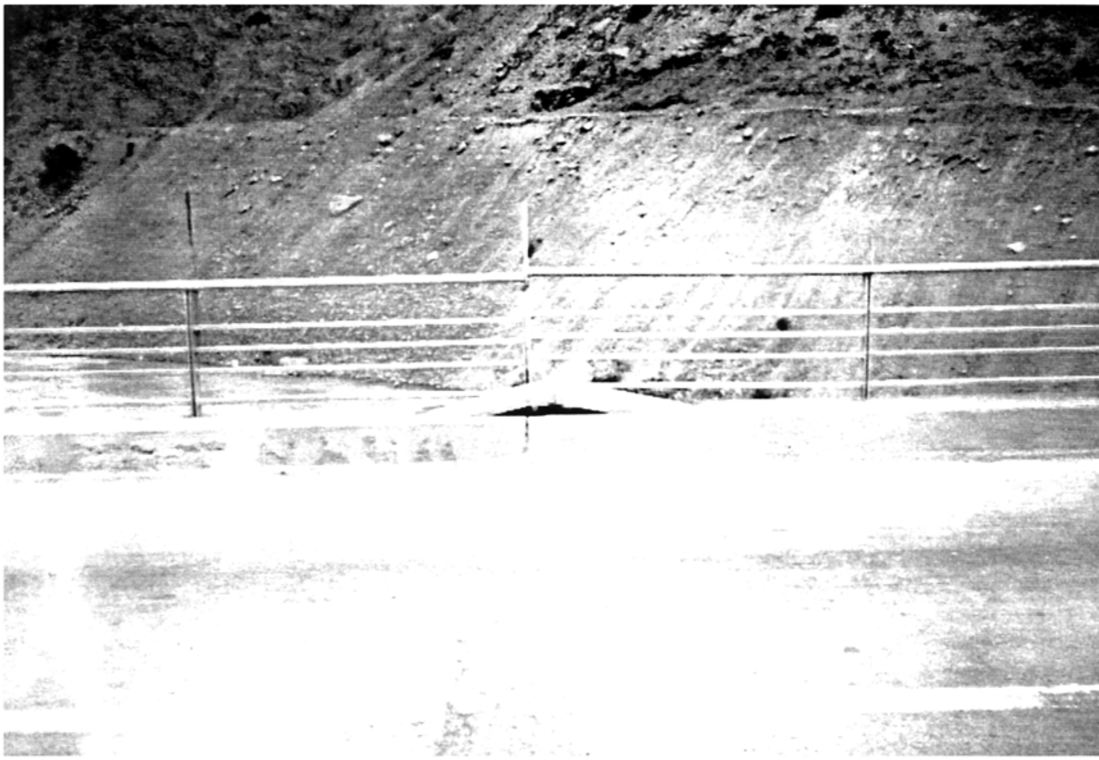
Picture 5.13 Tarik Diversion Dam, Saravan, Bridge deck

Picture 5.14 Tarik Diversion Dam, Saravan
Damage to the concrete piers and bridge deck due to pounding

Picture 5.15 Concrete Bridge, Rudbar, West view

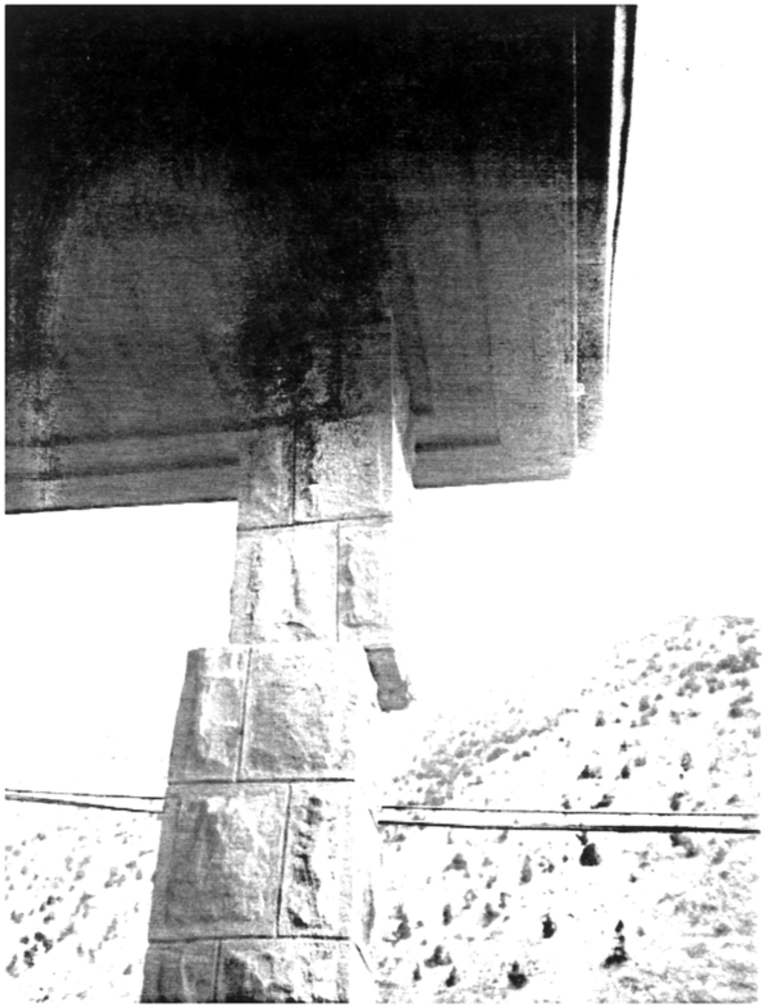


Picture 5.16 Concrete Bridge, Rudbar
Damage to the north end of the concrete deck due to pounding

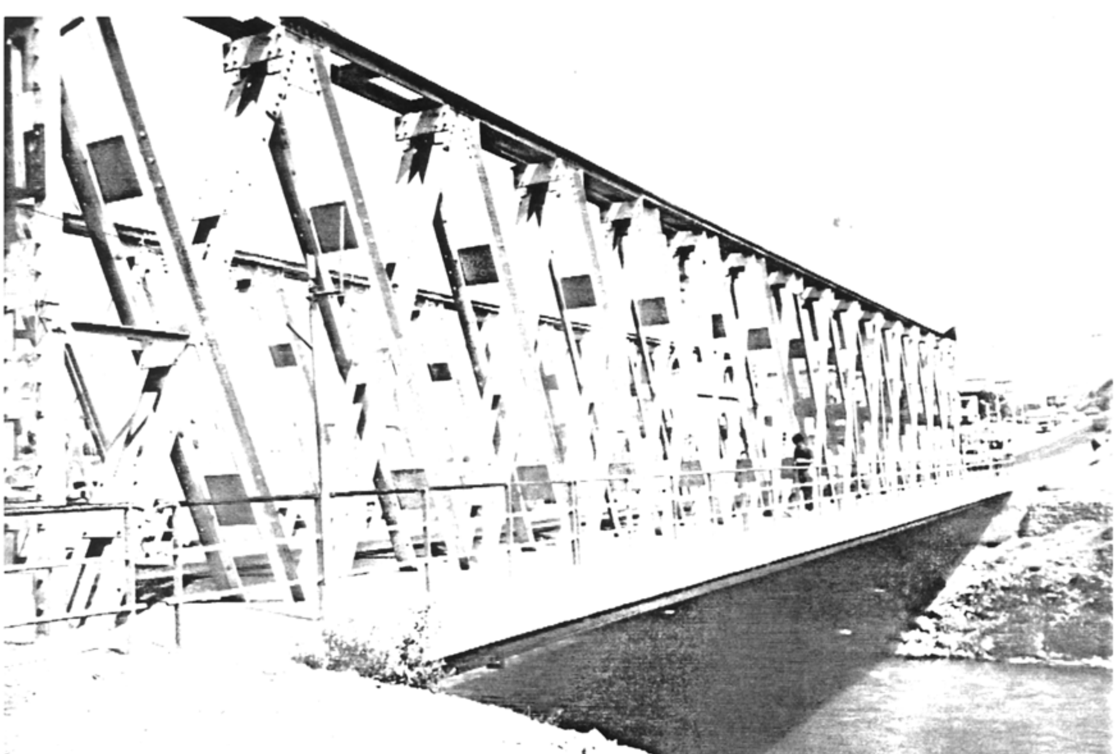


Picture 5.17 Concrete Bridge, Rudbar
Pounding failure at deck joints

Picture 5.18 Concrete Bridge, Rudbar
Failure of the south-west retaining wall

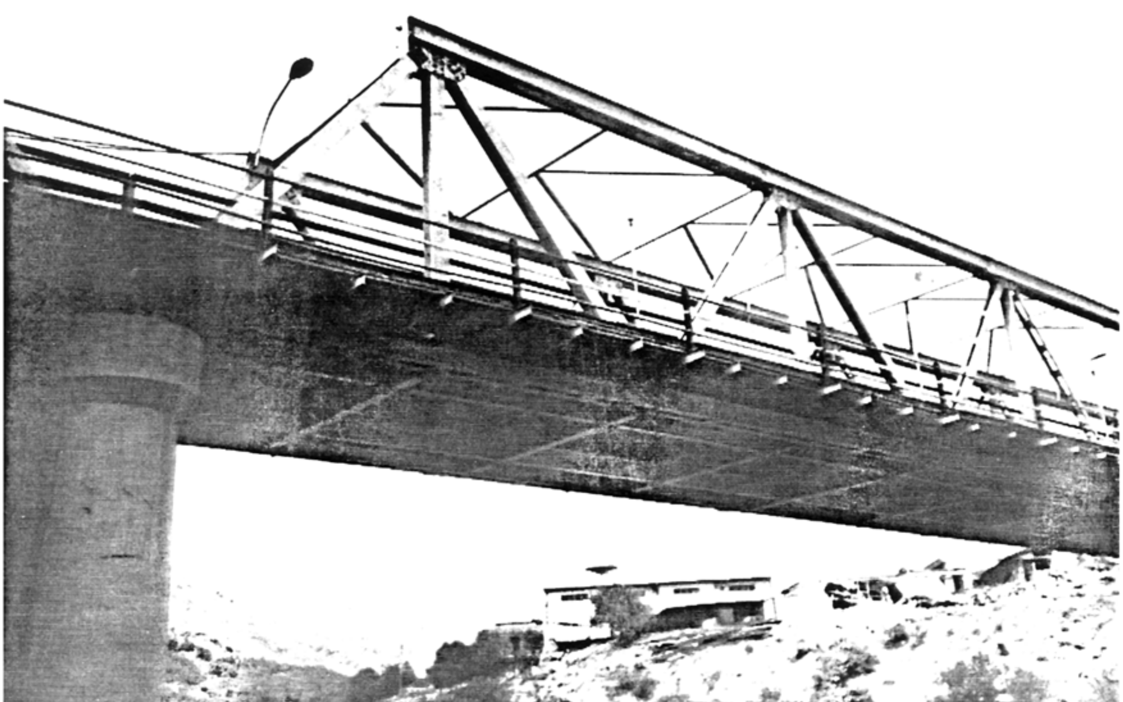


Picture 5.19 Concrete Bridge, Rudbar
Shear displacement at the masonry column of bridge guard house

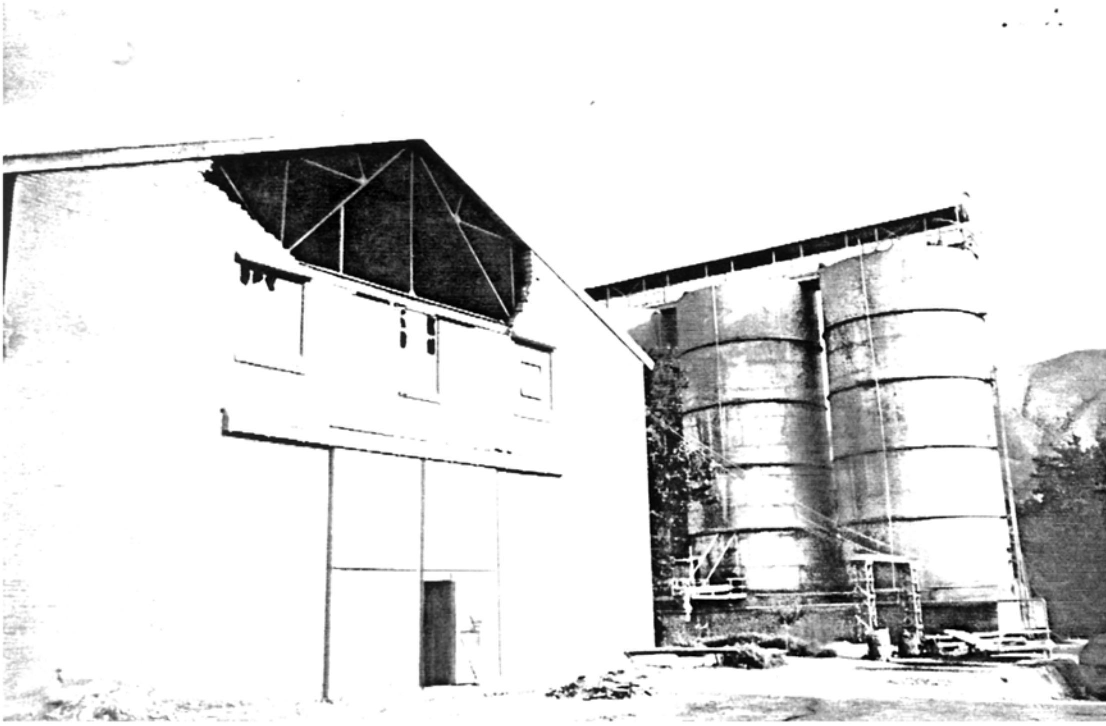


Picture 5.20 Steel Bridge, Lushan. No damage to this bridge

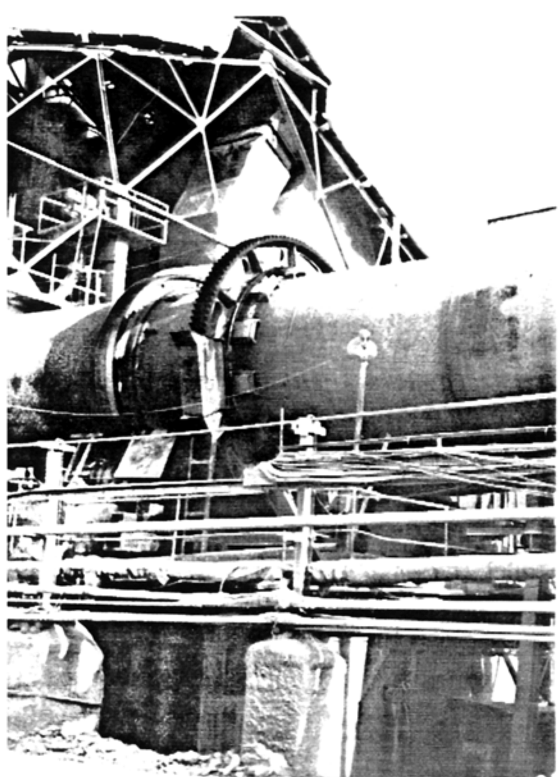
Picture 5.21 Steel Bridge, Rudbar. It behaved well during the earthquake



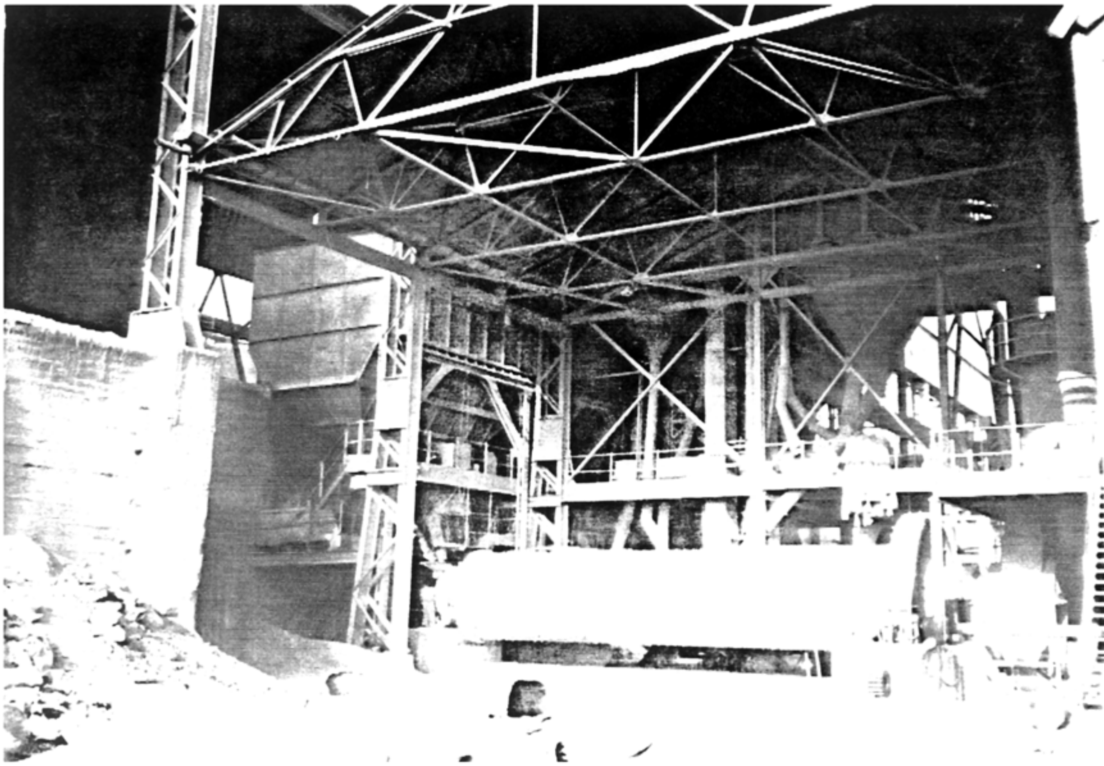
Picture 5.22 Steel Bridge, Rudbar, underside view



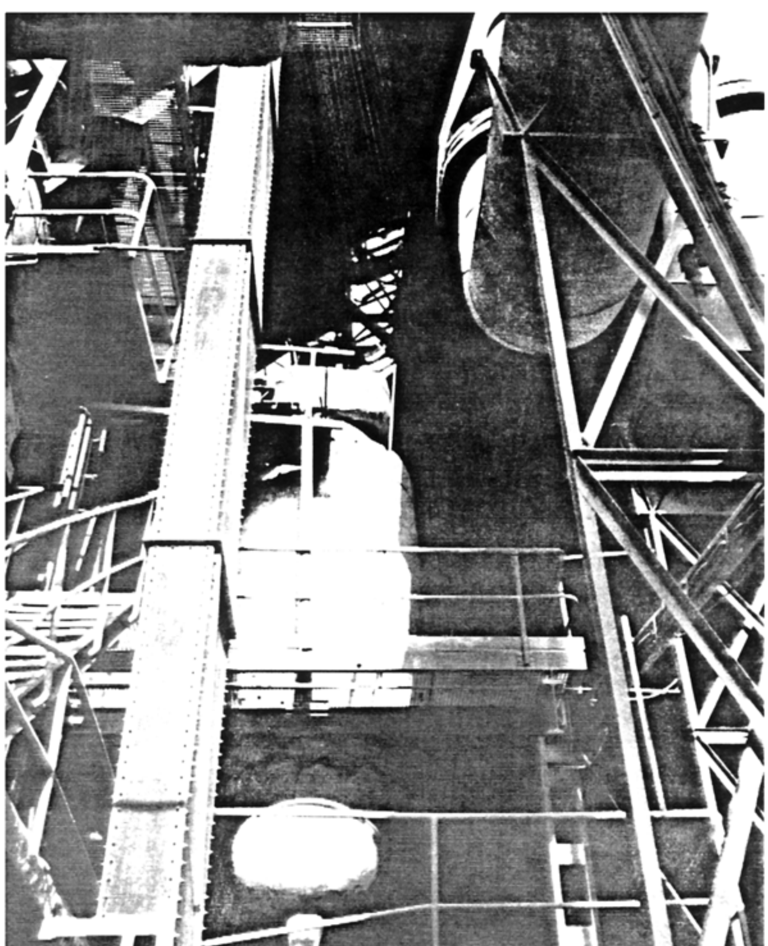
Picture 5.23 300-Tonne Cement Factory, Lushan
Collapse of un-reinforced brick masonry walls, typical damage to many of factory buildings and offices



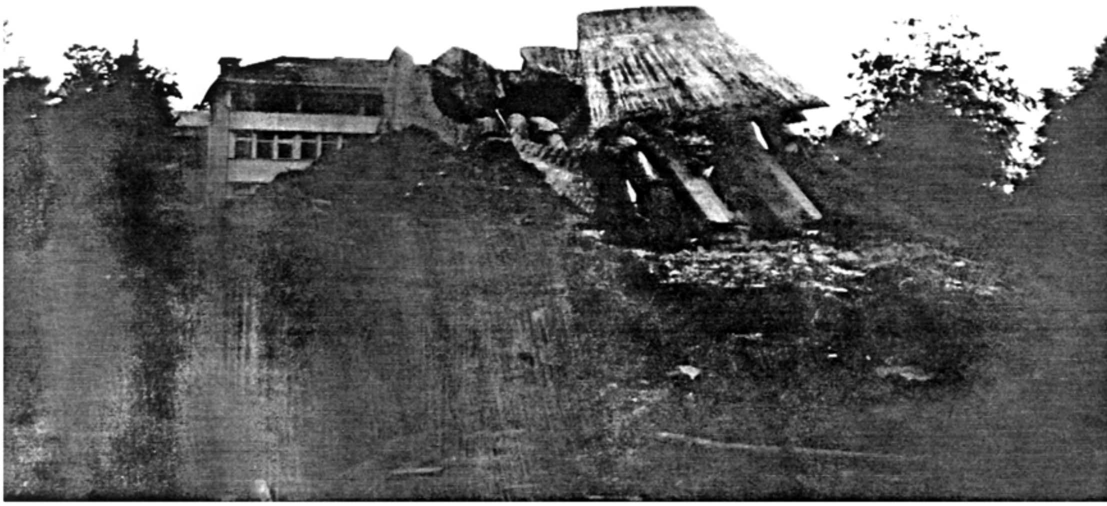
Picture 5.24 300-Tonne Cement Factory, Lushan
The 300 tonne furnace was slightly thrown off position by the force of the quake, some horizontal (bending) cracks in the two massive concrete bases of the furnace



Picture 5.25 300-Tonne Cement Factory, Lushan
Damage to one of the grinders



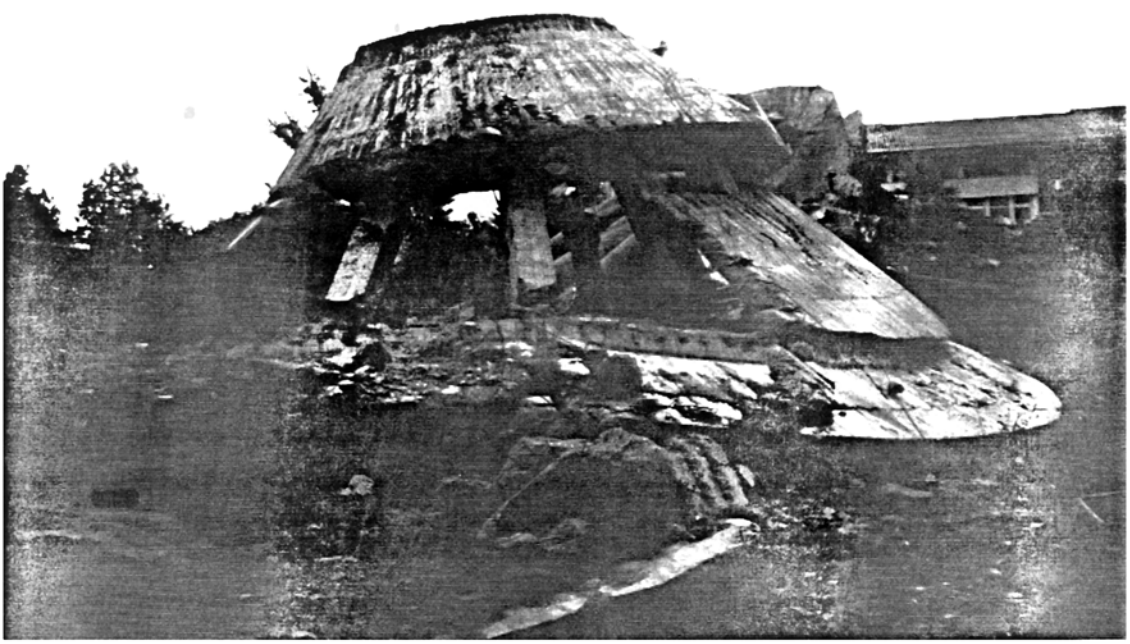
Picture 5.26 300-Tonne Cement Factory, Lushan
Damage to the roof and instalations



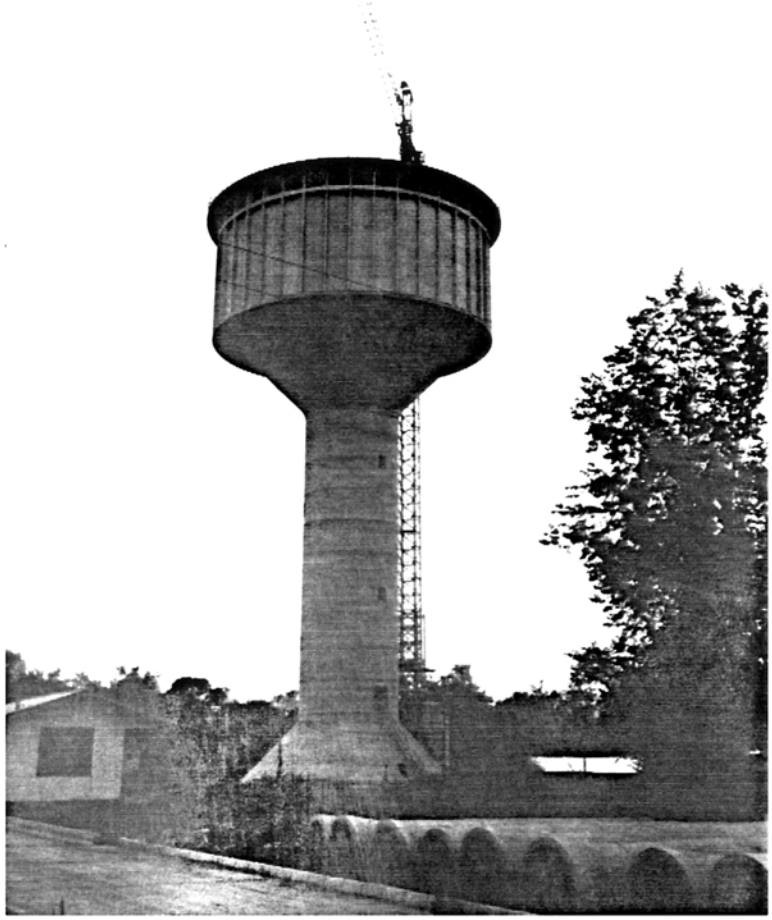
Picture 5.27 Concrete Water Tower No. 1, Rasht
Complete collapse of the tower due to the bending failure in the shaft

Picture 5.28 Concrete Water Tower No. 1, Rasht
Explosive compressive failure of concrete at the lower section of the shaft during collapse

Picture 5.29 Concrete Water Tower No. 1, Rasht
Destruction of the shaft on impact

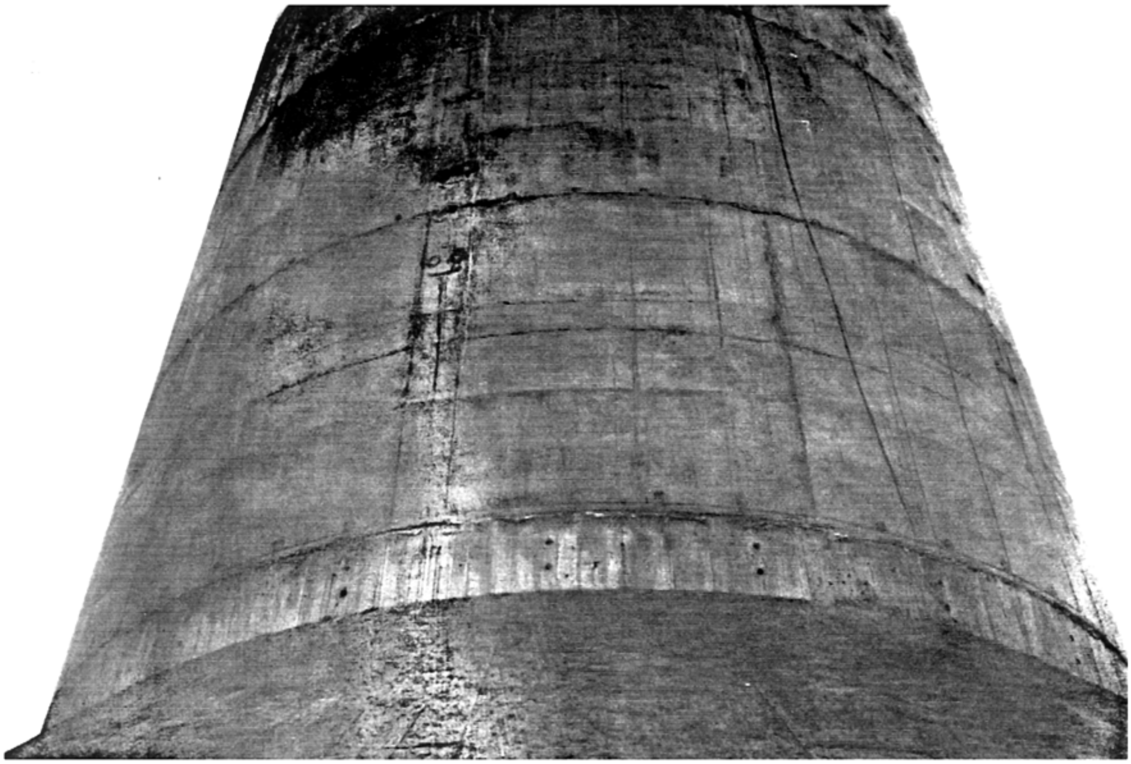


Picture 5.30 Concrete Water Tower No. 1, Rasht
Secondary bending and buckling failure of tank columns due to impact



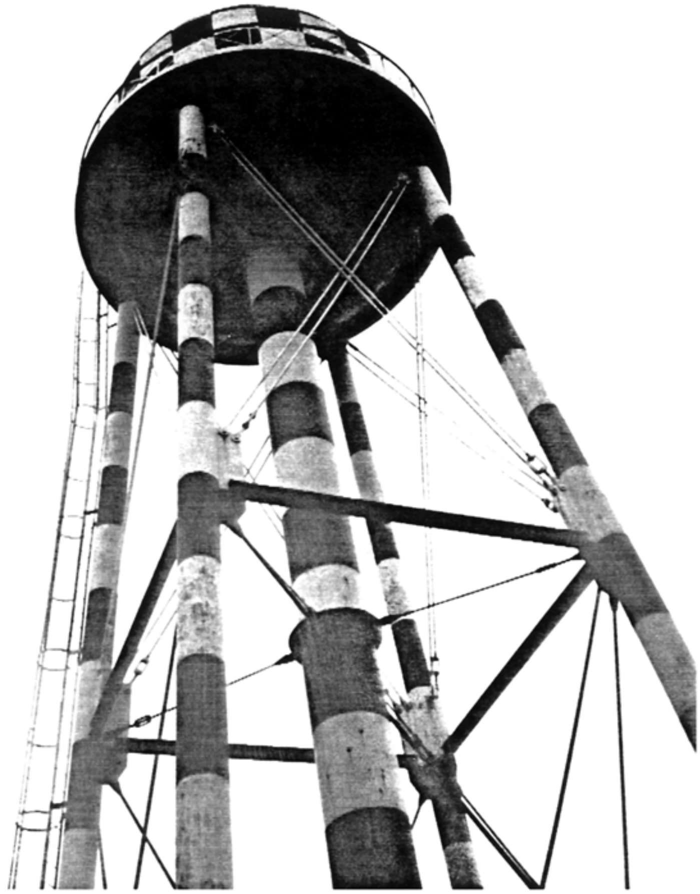
Picture 5.31 Concrete Water Tower No. 2, Rasht, General view

Picture 5.32 Concrete Water Tower No. 3, Rasht, General view

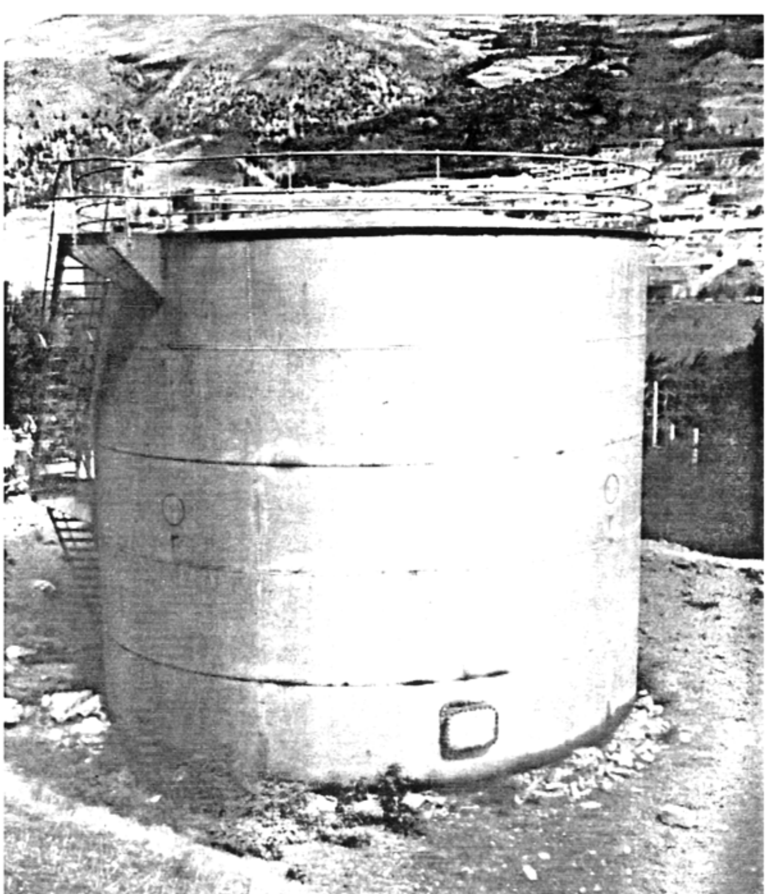


Picture 5.33 Concrete Water Tower No. 3, Rasht
Horizontal bending crack in the shaft coincides with the construction joint between the foundation and the shaft

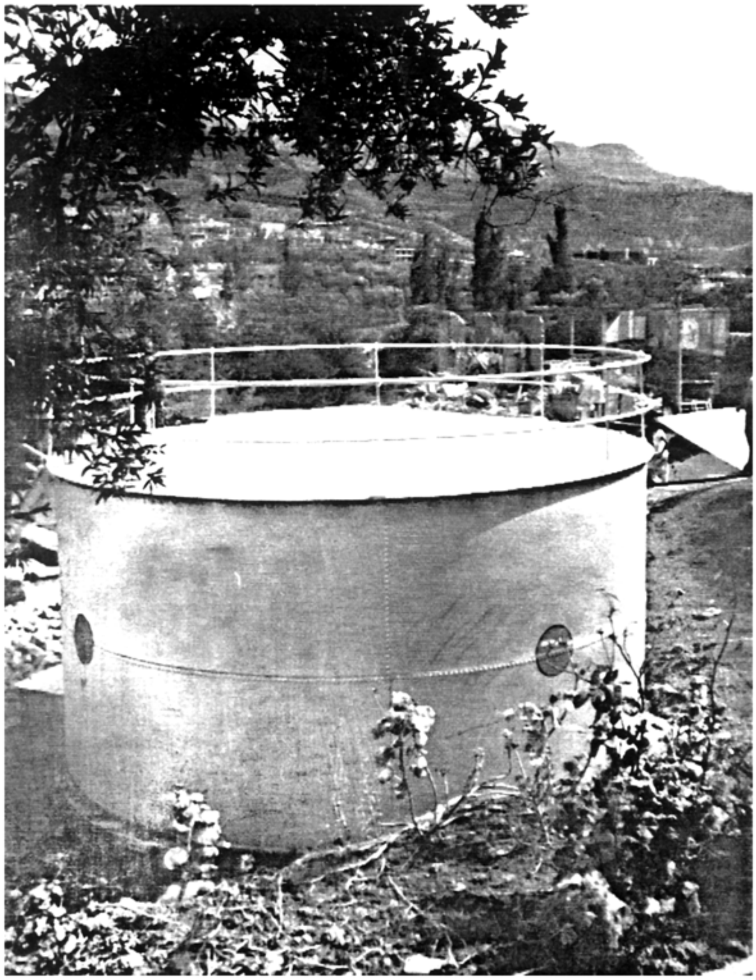
Picture 5.34 Concrete Water Tower No. 3, Rasht
Horizontal bending cracks joined by short vertical cracks indicate the path of the weakest section of the shaft in taking bending stresses



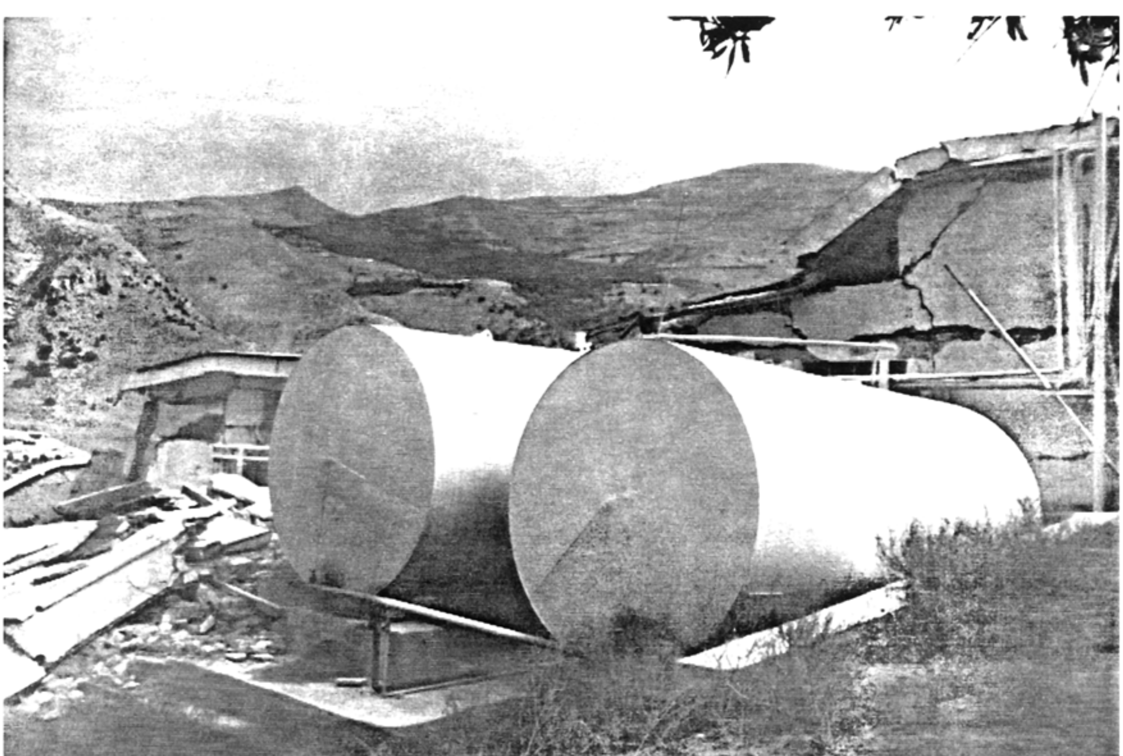
Picture 5.35 Elevated Steel Water Tank, Rasht
No damage to this well supported elevated water tank



Picture 5.36 Petrol tank No. 3, Rudbar
Steel tank survived the quake but the brick footing did not



Picture 5.37 Cylindrical Petrol Tank No. 2, Rudbar
The steel tank standing amidst collapsed buildings



Picture 5.38 Vali-asr Hospital, Rudbar
Collapse of liquid storage tanks due to support failure

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