No. 2

# REPORT OF JAPAN DISASTER RELIEF TEAM
# ON
# EARTHQUAKE AT SPITAK, ARMENIA, USSR

FEBRUARY 1990

JAPAN INTERNATIONAL COOPERATION AGENCY
(JICA)

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## PREFACE

JICA (Japan International Cooperation Agency), on behalf of the Japanese Government, dispatched a JDR (Japan Disaster Relief Team) from 18th to 28th December, 1988, to conduct a survey of the damage of Spitak Earthquake and to advise engineer and officials of Armenia SSR and USSR the technical know-how for the rehabilitation and restoration measures to be taken in the damaged area.

The JDR team was headed by Dr. Shigeji Suehiro, a councilor of the Ministry of Foreign Affairs (a former Director General of the Meteorological Agency) and composed of 10 experts of earthquake engineering, earthquake-resistant design etc.

According to the report of the Team, the second expert team, composed of 17 experts, was dispatched from 19th February to 15th March, 1989.

The present report outlines major results of the activity of the both teams.

I really hope that this report would contribute for the restoration of the disaster-striken area and also express my sincere gratitude to all people concerned of USSR and Armenia SSR.

February 1990

Sekai Nishino
Vice-President
Japan International Cooperation Agency

## Background for JDR Expert Team Dispatch

1) An earthquake registering an intensity of nearly 7 occurred near Spitak, Armenia, Soviet Union, at 11:40 on December 7, 1988, causing huge damage including the loss of about 35,000 lives in the area containing the three cities of Leninakan, Spitak and Kirovakan.

2) Ministry of Foreign Affairs decided to offer a financial aid of one billion yen and provisions including blankets, tents and medicines through the Japanese Red Cross Society, and dispatched an advance group of four members including one medical doctor (headed by Yutaka Iimura, Director of Technical Cooperation Division). The group made a disaster survey, and discussed with experts from the authorities of Armenia and the Soviet Union concerning the dispatch of a Japan Disaster Relief team.

3) The Soviet personnel requested Japan through the above-mentioned advance group to dispatch a Disaster Relief team specializing in various areas including seismic engineering, in which Japan, one of the most earthquake-ridden countries, has the most advanced expertise in the world.

4) In response to this request, the Japanese government immediately started the selection of experts in cooperation with the Ministry of Construction, National Land Agency, Ministry of Education and other agencies concerned, and decided on the dispatch of a Japan Disaster Relief Team consisting of 10 experts to perform a survey during the period from December 18 to 28.

5) As the result of the survey, this JDR Team reported to the Government of Japan the necessity of continuing technical advice in detail.

6) According to this recommendation, the second JDR Team composed of 17 experts, was dispatched from 19 February to March 1989.

7) The Report of the first JDR team is a translation from the Japanese version and the Report of the second JDR team is compiled based on the documents prepared for the International Seminar or Spitak-Armenia Earthquake held by UNICEF in May 1989 in Yereban.

**Members**

The team dispatched was consisted of experts specializing in construction, civil engineering, seismology, geology, disaster prevention, administration and other fields.

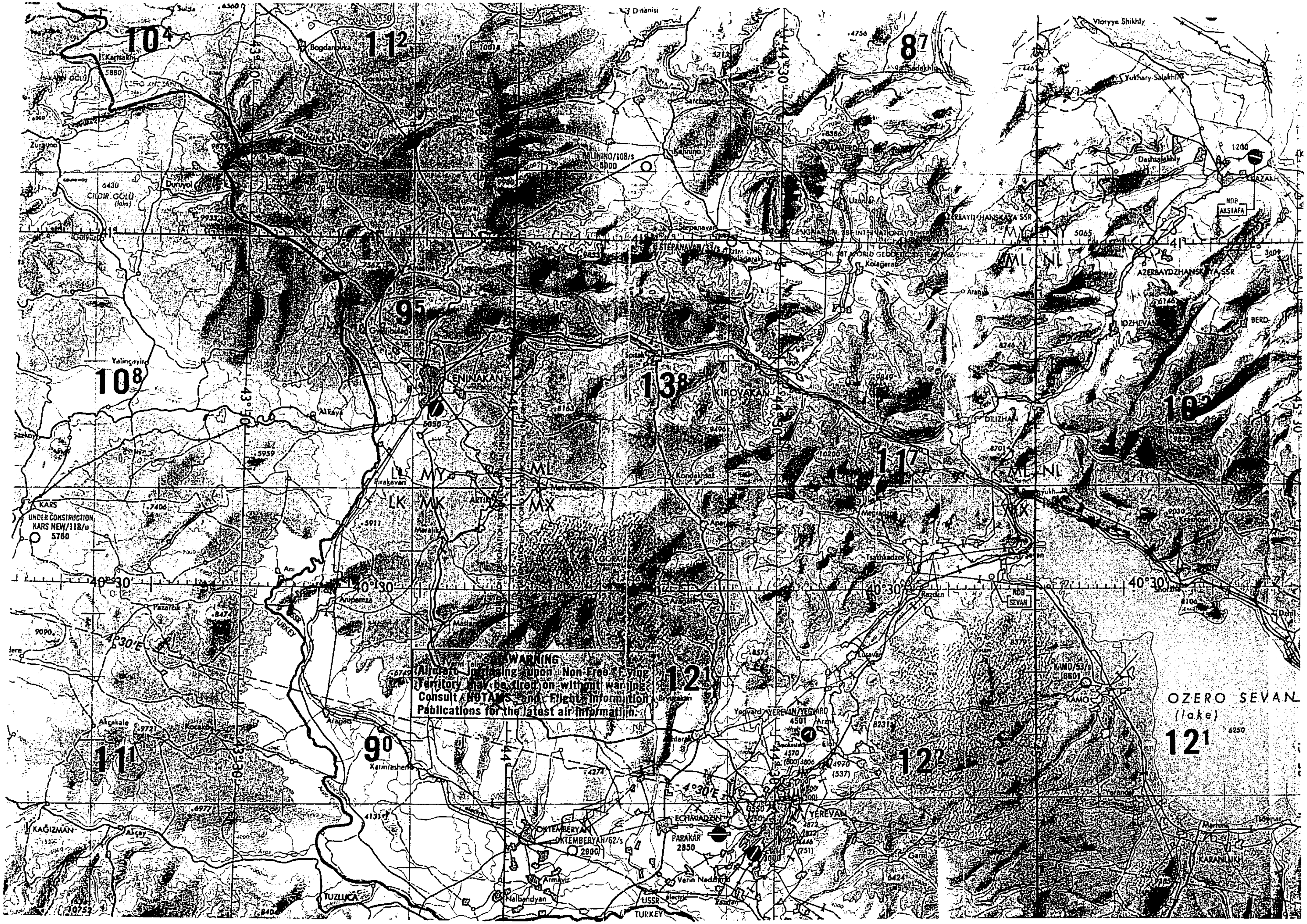
(1) The First JDR Team

| | | |
|---|---|---|
| Shigeji Suehiro (Head, Seismology) | : | Special Assistant to the Ministers for Foreign Affairs and for Science and Technology (a former Director General of the Meteorological Agency) |
| Kenji Ishihara (Geotechnical Engineering) | : | Professor, Department of Civil Engineering, University of Tokyo |
| Tsuneo Okada (Reinforced Concrete Structure, Seismic Design) | : | Professor, Institute of Industrial Science, University of Tokyo |
| Tadao Minami (Earthquake Engineering) | : | Professor, Earthquake Research Institute, University of Tokyo |
| Sakaaki Oda (Disaster Countermeasures) | : | Director, Earthquake Disaster Countermeasures Division, Disaster Prevention Bureau, National Land Agency |
| Yasunori Yamanaka (Building Disaster Prevention) | : | Director of Building Disaster Prevention, Housing Bureau, Ministry of Construction |
| Masaya Hirosawa (Precast Reinforced Concrete) | : | Director, International Institute of Seismology and Earthquake Engineering, Building Research Institute, Ministry of Construction |
| Kazuhiko Kawashima (Construction Engineering) | : | Director, Earthquake Resistance Laboratory, Earthquake Disaster Countermeasures Division, Public Works Research Institute, Ministry of Construction |
| Atsushi Yoshida (Coordinater) | : | Official, Soviet Division, European and Oceanic Affairs Bure_u, Ministry of Foreign Affairs |
| Koji Kawai (Coordinator) | : | Deputy Director, Disaster Relief Division, Medical Cooperation Department, Japan International Cooperation Agency (JICA) |

(2) The Second JDR Team

| | | |
|---|---|---|
| Shigeji Suehiro (Head, Seismology) | : | Special Assistant to the Ministers for Foreign Affairs and for Science and Tecnology |
| Kenji Ishihara (Geotechnical Engineering) | : | Professor, Department of Civil Engineering, University of Tokyo |
| Tsuneo Okada (Reinforced Concrete Structure, Seismic Design) | : | Professor, Institute of Industrial Science, University of Tokyo |
| Hiroyoshi Kobayashi (Engineering Seismology) | : | Professor Emeritus, Tokyo Institute of Technology |
| Masaya Murakami (Earthquake Engineering & Seismic Design) | : | Professor, Department of Architectural Engineering, Faculty of Engineering, Chiba University |
| Yasunori Yamanaka (Building Disaster Prevention) | : | Director of Building Disaster Prevention, Housing Bureau, Ministry of Construction |
| Shuji Mukunoki (Disaster Countermeasures) | : | Deputy Director, Earthquake Disaster Countermeasures Div., Disaster Prevention Bureau, National Land Agency |
| Yoshimi Akiba (Structure Design) | : | Deputy Director of Building Div., Government Build. Dept., Ministry of Construction of Japan |
| Masaya Hirosawa (Precast Reinforced Concrete) | : | Director, International Institute of Seismology and Earthquake Engineering, BRI, MOC |
| Keiichi Ohtani (Earthquake Engineering) | : | Head, Earthquake Engineering Laboratory, National Research Center for Disaster Prevention, Science and Technology Agency |
| Toshihide Kashima (Earthquake Engineering | : | Researcher, International Institute of Seismology and Earthquake Engineering, BRI, MOC |
| Atsushi Yoshida (Coordinator) | : | Official, Soviet Div., European and Oceanic Affairs Bureau, Ministry of Foreign Affairs |

| | | |
|---|---|---|
| Yoshio Inoue<br>(Structure Design of Precast Reinforced Concrete) | : | Chief Structural Engineering, Design Section of Architecture and Construction Department, Housing and Urban Development Corporation ( HUDC ) |
| Toru Takahashi<br>(Prefabiricated Construction Engineering) | : | General Manager, Design and Proposal Dept. Marketing and Sales Div., Taisei Prefab Corporation |
| Noboru Sakaguchi<br>(Structural Engineering) | : | Senior Research Engineer, Structural Engineering Dept., Institute of Technology, Shimizu Corporation |
| Shunsuke Sugano<br>(Reinforced Concrete Structure, Seismic Design) | : | Chief Research Engineer Takenaka Corporation, Technical Research Laboratory |
| Koji Kawai<br>(Coordinator) | : | Deputy Director, Disaster Relief Division, Medical Cooperation Department, Japan International Cooperation Agency (JICA) |
| Yuji Kashihara<br>(Coordinator) | : | Staff, System Management Division, General Affairs Department, Japan International Cooperation Agency (JICA) |


WARNING
Aircraft infringing upon Non-Free Flying Territory may be fired on without warning. Consult NOTAMS and Flight Information Publications for the latest air information.
LENINAKAN
KIROVAKAN
DILIZHAN
YEREVAN
ECHMIADZIN
OKTEMBERYAN
KARS
UNDER CONSTRUCTION KARS NEW/118/u 5760
CILDIR GÖLÜ (lake)
OZERO SEVAN (lake)
KAĞIZMAN
TUZLUCA
AKSTAFA
IDZHEVAN
BERD
KAMO
STEPANAVAN
Spitak
Ani
Armavir
Nalbandyan
USSR
TURKEY
AZERBAYDZHANSKAYA SSR
40°30'
41°

Report of Japan Disaster Relief Teams on
Earthquake at Spitake, Armenia, Soviet Union.

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# I The First JDR Team

# I. The First JDR Team

## 1. Situation in Armenia — Focusing on Seismic Measures

### 1.1 General Information

(1) Natural and Geographic Features

Highlands at altitudes of 1,000m or above account for more than 90 percent of the total area of the Armenian SSR, with the average altitude being as high as 1,800m. Mt. Aragats, a 4,090m-high extinct volcano, is the highest mountain in Armenia. Lake Seven, located in a highland area at an altitude of about 1,900m, was formed by the damming of a river caused by the activity of the volcano. Mt. Ararat (5,165m), which the Noah's ark is said to have reached after drifting, is currently in the territory of Turkey.

(Altitudes of major cities: Yerevan 950m, Leninakan 1,500m, Kirovakan 1,350m)

Most part of Armenia is covered with volcanic deposits. In particular, volcanic ejecta including andesite, basalt, tuff, pumice and pearlite from currently inactive volcanoes account for a great part of the surface soil. Tuff with colors in the range from pink to lilac occurring in Armenia is widely used as a building material, as reflected in the unique color tone of the streets in Yerevan, Armenia's capital.

Though being located at subtropical latitudes, the Armenian SSR has complex meteorological features due to the effects of high altitudes and the existence of the Caspian Sea, Black Sea and Asia Minor highland. The basins and mountenous areas in Armenia, including capital Yerevan, are characterized by continental hot summers, moderate winters and small precipitation. The average temperature of 24 to 26°C and -5°C in July and January, respectively, and the precipitation is in the range of about 200-400mm. In the highland district (including the earthquake-stricken area in question), on the other hand, the average temperature is 18-20°C and in the range of -4 to -6°C in July and January, respectively, and the precipitation is around 550mm.

All rivers in Armenia belong to the Caspian river system. Specifically, part (24 percent of total Armenia) of northern Armenia belongs to the Kura system, with the Kura River flowing through the middle of the two neighboring republics of Georgia and Azerbaijan towards the east into the Caspian Sea. The remaining great part (76 percent of the total area) forms the borders of Armenia with Azerbaijan, Turkey and Iran, and belongs to the Araxes system, with the Araxes river joining the downstream Kura River. (The former river system contains Spitak and Kirovakan while the latter contains Leninakan, Yerevan and Lake Sevan.)

(2) People, Language and Religion

Of the fifteen republics in the Soviet Union, the Armenian SSR is the smallest in area (29,800$km^3$) and the second highest in population density (104.7/$km^2$ in 1987) following the Moldavian SSR.

The Armenian SSR had a population of 3,412,000 as of 1987, accounting for 1.2 percent of the Soviet total population. In a racial breakdown, Armenians account for 90 percent, Azerbaijans for 5 percent, Russians for 2.3 percent, and Kurds for 1.7 percent. There are 4,150,000 and about 180,000 Armenians (1979) respectively in the Soviet Union and in about sixty other nations (450,000 in the U.S., 200,000 in Iran, 200,000 in France, 180,000 in Lebanon, 150,000 in Syria, 60,000 in Argentina, and 25,000 in Iraq) (figures for nations other than the Soviet Union cited from 1967 statistics).

The origin of the Armenians can be traced back to the ancient people living in the northeastern part of Asia Minor, who later settled in the Armenian highland as agriculture propagated and developed in a form closely linked with livestock farming. They experienced repeated conflicts with people in surrounding areas including Arabs, Byzantines, SELDJOUK Turks, Mongolians and Tatarians all through the periods of slavery and feudalism. The Armenians started to develop a capitalistic-type society at the beginning of the 19th century, when eastern Armenia was incorporated in the Russian Empire, and later became a republic when the Union of Soviet Socialist Republics was established in November of 1920.

Armenian is an independent language belonging to the Indo-European group. Its alphabet was established in the beginning of the 5th century based on the alphabets of Greek and Aramic (used in ancient Syria and Paletine).

It has been reported that Christianity was propagated to Armania by missionaries around the 2nd century as the religion spread from the Roman Empire. Armenia and the Armenians were the first state and people that had officially accepted Christianity (301 A.D.), and they are a rare example in which the tradition has been kept up to now while maintaining the religion. GRIGORIOS, who played the leading role in making Christianity the state religion of Armenia, designated Echmiadzin (meaning "the advent of the only-begotten"), which is located about 20km to the west of Yerevan, as the place for the bishop of the Armenian church. (Currently, Echmiadzin is still regarded as holy place for the Armenian church.)

(3) Industry

1) Agriculture and Stock Farming

In a breakdown of land by form of utilization, pastures, orchards and grassland account for a great part of the Armenian SSR. The republic is mostly dry with small precipitation, and more than 23 percent of the total area is irrigated. Major agricultural products are fruits including grape, industrial crops including tobacco and sugar beet, and other crops including wheat and potato. Major animals for pasturing include cattle, sheep and goats.

2) Industry

Major industries include machinery production, metal processing, chemistry, petrochemistry, textile product (knitwear) production, shoe production, foods production (wine, brandy, canned vegetables/fruit), nonferous metal production, and construction materials production. Since the republic is rich in water resources, hydraulic power generation is actively performed. In addition, thermal power generation is also conducte and there is the Armenian Nuclear Power Plant near Yerevan (about 30km to the west of Yerevan, with an output of 815,000kW).

Volcanic rocks (tuff, pumice, pearlite, basalt, granite, marble, etc.) that occur in large amounts in the area have been used as material for construction and decoration. The area has been well known for the production of wall material, light-weight concrete aggregate, and heat-resistant construction material made from tuff.

(4) Outline of Earthquake-Damaged Cities

1) Leninakan (called ALEKSANDROPOLI until 1924)

The city is located on a volcanic plateau at an altitude of higher than 1,500km. It has a population of about 290,000 (according to a Soviet newspaper issued in December 1988; 213,000 in 1981).

In this area, there were settlements as early as in the 5th century B.C. A town called Kumairi was formed in the Middle Age, and a Russian fortress was built in 1837. The area was the center of workers' movements during the period from the end of the 19th century to the beginning of the 20th century, followed by the establishment of the Union of Soviet Socialist Republics in 1920.

Currently, Leninakan is a center of the textile and spinning industry, and other industries include the production of machinery, precision machinery, foods and furnitures. There is an education college and a branch school of the Yerevan Polytechnical Institute in the city.

2) Kirovakan (called KARAKLIS until 1935)

The city is located in a basin at an altitude of 1,350m between the BAZUM Mountains and the Pamback Mountains. It has a population of about 170,000 (asccording to the same data as above; 153,000 in 1981). The city began to prosper at the end of the 19th century, when a railroad was laid from Tiflis (currently Tbilisi) to ALEKSANDROPOLI (currently Leninakan) via KARAKL.

At present, Kirovakan is a center of the chemical industry and chemical fiber industry including the production of mineral-based fertilizers. Other industries include the production of machinery, precision machinery, foods and fiber/textile. There is an education college and a branch school of the Yerevan Polytechnical Institute.

3) Spitak (called AMAMUR until 1984)

Spitak, reorganaized as a municipality in 1960, is located in the basin of the Pamback River, which belongs to the Kura River system. It has a population of about 20,000 (according to the same data as above; 13,300 in 1975).

Major industries in the city include the production of sugar/butter/cheese, elevator, artificial leather, flour, and knitwear.

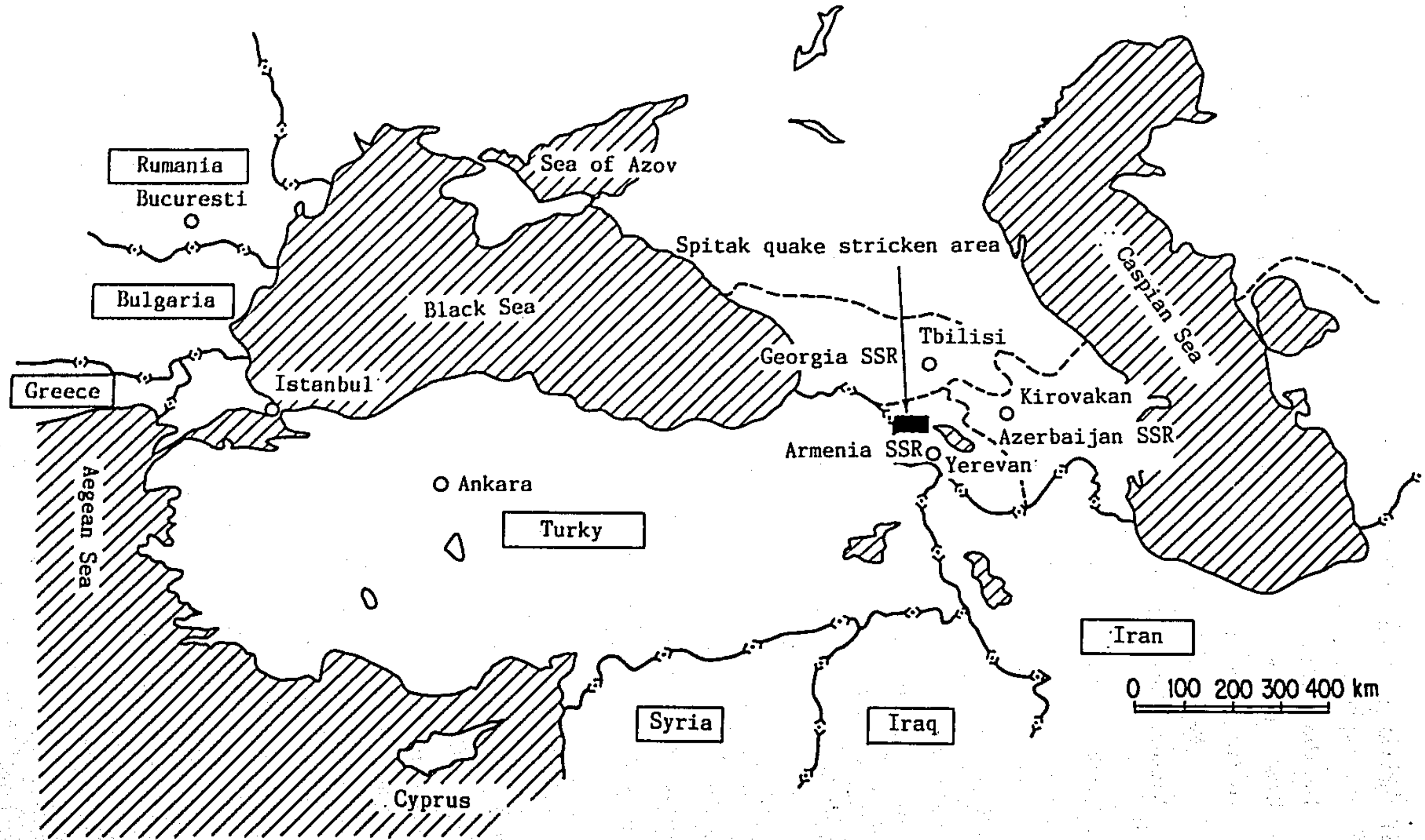

Rumania
Bucuresti
Bulgaria
Greece
Istanbul
Black Sea
Sea of Azov
Spitak quake stricken area
Georgia SSR
Tbilisi
Caspian Sea
Kirovakan
Azerbaijan SSR
Armenia SSR
Yerevan
Ankara
Turky
Aegean Sea
Cyprus
Syria
Iraq
Iran
0 100 200 300 400 km

Fig. 1.1.1

## 1.2 Geology and Ground Condition

### (1) Outline

The damaged area is located in highland on the small Caucasus Mountains ranging between the Black Sea and Caspian Sea. It is at altitudes of 1,500-1,700m and characterized by an inland-type climate. The annual precipitation is about 500mm, which is approximately one-third that in Japan. There was no rain during the week prior to December 7, when the earthquake occurred, suggesting that the ground was dry.

Fig. 1.2.1 roughly illustrates the area around the epicenter of the earthquake. The Pamback River runs eastward from the watershed located between Spitak and Leninakan. It flows through the middle of Spitak, turns to the north in Kirovakan, penetrates the plain portion of Azerbaijan, and reaches the Caspian Sea. Many tributaries, mostly from the south, flow into the Pamback River. The streets of Spitak developed around the junction of one of the tributaries.

Kirovakan, located 20km to the east of Spitak, developed in the east-west direction on a terrace along the Pamback River. A river coming from Lake Sevan to the east of Kirovakan joins the Pamback River and turns to the north in the city. It is estimated that the river currently is approximately 5m wide and 1m deep with a flow rate of approximately $5m^3/sec$.

Leninakan is an old city developed among forests on a plain basin surrounded by gently-sloping mountains. There are two rivers, running on the east and west of the city towards the south. The area contains major water sources. In fact, three tributaries branch out in the city, though little water was seen during the survey probably because it was the dry season.

As stated previously, the earthquake-stricken area is covered with volcanic deposits. There are many small cliffs and the highland is scattered all over with pumice.

### (2) Spitak

The city of Spitak developed around the junction of the Pamback and a large tributary coming from the south. The topography around the city is outlined in Fig. 1.2.2. and its rough cross section cut in the north-south direction is shown in Fig. 1.2.3. As obvious from these figures, the major part of the city can be divided into two parts: one part consists of hills at the foot of a mountain and plateaus on the terrace of rivers, while the other contains lowlands extended over dry riverbeds. There is a steep cliff 15-20m high between the lowland part and the plateau part on its south. The area on its north comprises gentle slopes, and there are no definite boundaries. A small mountain with a relative height of 50-70m exists at the center of the plateau on the south.

A group of five- to six-storied apartment houses located in Zone A in Fig. 1.2.2 suffered the greatest damage from the latest earthquake. The zone is on a plateau containing hills. In Zone B in the same figure, three-to four-storied buildings and one-storied private houses were destroyed almost completely. Zone C, a terrace plateau, contained many factories varying in size, which were also heavily destroyed. Damage was smaller in Zone E, where one-storied private houses stood on hill slopes, and in Zone D on the southeast side of the tributary. Data show that Zone D is low in altitude and comprises a low terrace 3-5m above the riverbed level. The smaller damage in Zones D and E can be attributed to the fact that most of the houses were stone-built ones which were low and new. Contrary to this, old, weak, low houses accounted for a large part in Zone B.

The low land on the riverbeds in Spitak comprised railroad banking and scattered farm houses, and there were no newly constructed apartment houses. Many of the one-storied farm houses collapsed, and the railroad banking was destroyed at some points, bending rails heavily. It is inferred from past examples of damage caused by earthquakes that the destruction in these zones is attributable to the fluidization of the ground.

The soil column pattern in Spitak is not known because boring data are not available. Judging from the topography, the plateaus contain gravel and clay formed from weathered tuff, and the groundwater tabler is estimated to be very low. It is not exactly known whether the plateaus are river terraces formed through the errosion of deposits in old rivers, or they are simply terraces formed from hill sides with weathered soil remaining over them. Observations seem to indicate that the greater part of them have been formed through the latter mechanism. Most of the low land on the riverbeds has been formed from soil deposited after being carried by the rivers. The soil contains much clay and is generally unlikely to be fluidized. The groundwater table is estimated to be low, possibly at about 1-2m from the ground surface. It would be interesting to investigate the depth of the deposits layer in this region, though no data are available.

On the west of Spitak, there is a valley formed between step mountains existing on the north and south of it. Further to the west, there is another gently sloping region. The intensity of the earthquake has been reported to be IX to X in this region, and the villages along the Pamback River suffered heavy damage.

(3) Kirovakan

Kirovakan, located 20km to the west of Spitak, is a long, narrow city extended along the Pamback River. There are many factories for machinery manufacturing, spinning, and chemicals production. The major part of the city is located on a hillock terrace 1-2km wide between two ranges of mountains extending in the east-west directions. The geology there is basically the same as that in Spitak.

(4) Leninakan

The city is about 4km wide in the east-west direction and about 8km long in the north-south direction. As shown in Fig. 1.2.4, three tributaries (one on the west is not shown) run from the center of the city to join the Aknurian River. Another river is flowing southward on the east of the city. The tributary running through the middle of the city gets into an underground conduit and comes up to the ground in the south of the city, though little water was seen during the survey. We obtained east-west cross sectional profiles of the ground at two points in the north part of the area shown in Fig. 1.2.4. The cross sections are illustrated in Fig. 1.2.5. Tuff occurs at depths of 10-20m and is overlain by layers of sand and clay that appear to have been brought by the river. Observations suggest that the groundwater table in the low region is generally at 4m or deeper from the surface, though its depth seems to be smaller at some points.

It is reported that researchers in Leninakan since around the beginning of 1970 have been conducting ground condition studies and underground exploration based on boring data and elastic wave measurements, respectively. Fig. 1.2.6 gives a geological map cited from a report issued by the Academy of Science of the Armenian SSR. Fig. 1.2.7 shows the geological profile of the K-K' cross section (east-west direction) of the area in Fig. 1.2.6. The comparison of these figures indicates that Point 15 in Fig. 1.2.6 corresponds to a point at the foot of the slope from a monument standing on the hill in the west. Point 23 corresponds to a spot at the foot of a slope slightly to the west of the Cherketz River while Point 17 to a spot on the Kumairi River flowing southward through the underground conduit penetrating the middle of the city. Topographically, the northern part of the city is about 5-10m higher than the other part while the ground is low around the center of the southern region (mostly on the south of line K-K'). This may have resulted from the erosion by small rivers.

It is inferred from Fig. 1.2.6 that tuff formed from volcanic lava occurs in the northwest part of the city. The tuff layer is overlain by surface soil of 2-3m thick. The north part of the city contains sandy clay layers sandwiching thin gravel layers, which are considered to be river deposits. Sandy and conglomeratic soil prevail in the south of the city. Probably, they were exposed after the removal of surface soil by erosion. The Cherketz and Kumairi River have a bottom at depth of 3-5m, and there are slipped cliffs at some points. Fig. 1.2.7 illustrates the strata contained in the K-K' cross section, suggesting that sandy and conglomeratic soil prevail in the south of the city. Observations of the water level in boreholes indicate that the underground water table is at 4m or more below the surface as shown by the broken line while the groundwater table is high only in the low regions along the rivers. The depth of the groundwater table is illustrated in detail in Fig. 1.2.6, also revealing that the position of the groundwater table is shallower by 4m in the regions along the rivers and in the northeast of the city. Groundwater does not exist in deep soil, but the thickness of the aquifer is in the range of 7-9m in the regions from the center to the northeast of the city and 2.5-5m in the low regions.

Boreholes 2 and 23 shown in Fig. 1.2.7 are only about 35m and 15m deep, respectively. It is reported that deeper boreholes have been formed and that the lacustrine clay widely distributed over this area reaches as deep as 250-300m maximum. This is also seen from the fact that the base part in Fig. 1.2.7 is mostly covered with this lacustrine clay. These observations may suggest that there was a large lake in this area, containing Leninakan, in ancient times, which was filled up over a long period of time to form lacustrine clay and then overlain by gravel deposits brought by rivers or tuff brought by the activities of nearby volcanoes. This may also be inferred to some extent from current maps, which show that the Armenian area is still dotted with a large number of lakes different in size including Lake Sevan.

Table 1.2.1 lists data on the propagation velocity of the longitudinal and transverse wave in each soil layer determined from elastic wave measurements made at the points in Fig. 1.2.6.

Table 1.2.1 Characteristics of Soil in Leninakan

| Type of Soil | Measuring Point | Vp (m/sec) | Vs (m/sec) | Depth of Ground-water Table |
|---|---|---|---|---|
| Firm tuff | 9, 9[a] | 1600∿1800 | 720 | |
| Weathered tuff | 9[a] | 530 | 163 | |
| Lacustrine clay | 14[a], 15, 16 | 1440∿1760 | 450 | |
| Unsaturated lacustrine clay | 14 | 680 | 390 | |
| Sandy clay | 13 | 830 | --- | |
| Unsaturated sandy clay | 1∿4, 6, 7, 10∿13 | 300∿500 | 145∿340 | |
| Saturated sandy clay | 3 | 2150 | --- | 6.5∿7.5 |
| Unsaturated sandy soil | 5, 14[a], 15, 16 | 290∿560 | 165 | |
| Dry, firm sand | 4, 8, 18 | 660∿700 | 280 | |
| Sandy gravel | 11 | 1500∿1520 | 1000 | |
| Earth filling | 2, 6, 14[a] | 270 | 160∿170 | |

* see Fig. 1.2.6.

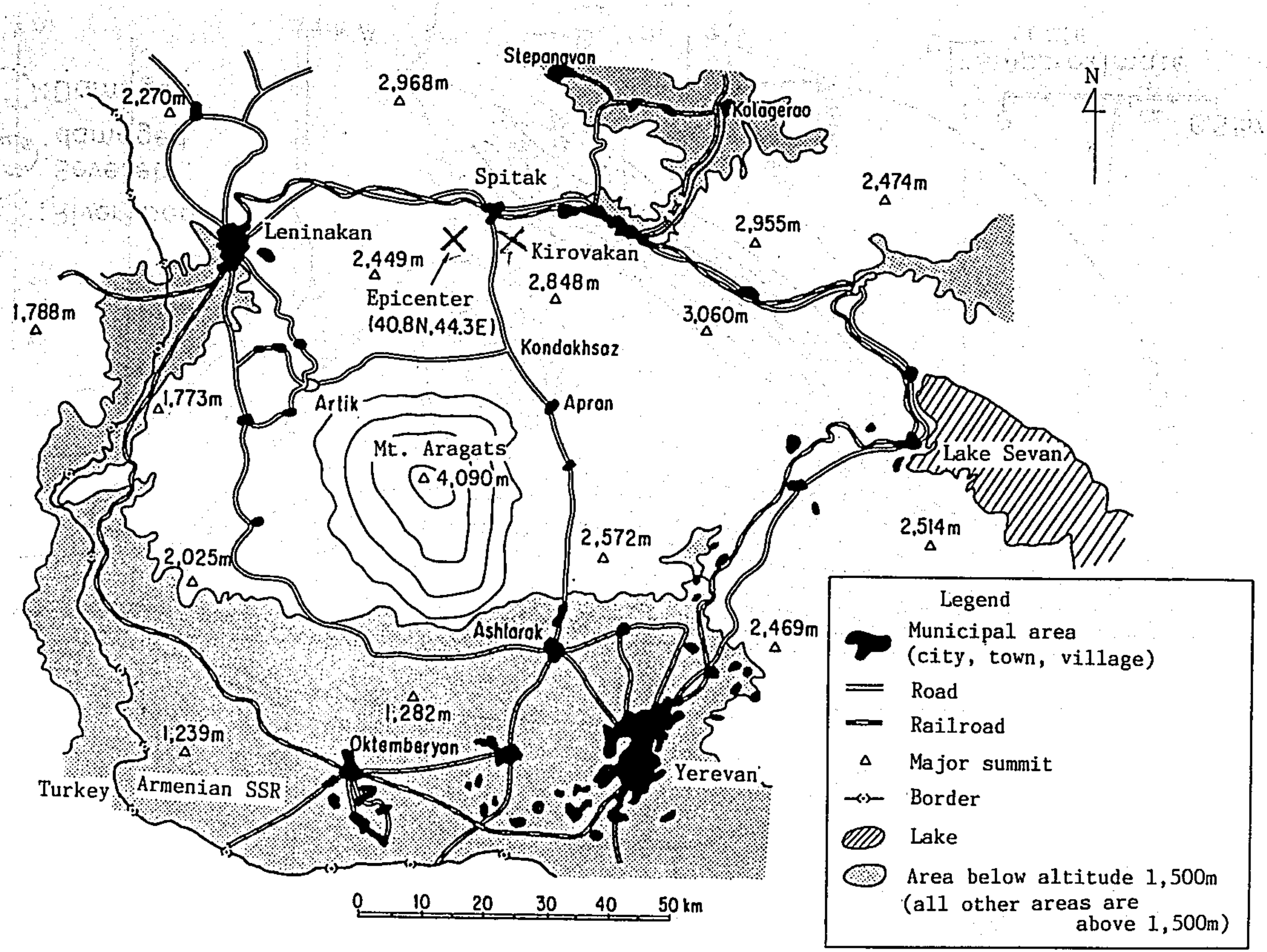



Fig. 1.2.1 Topography of Earthquake-Stricken Area

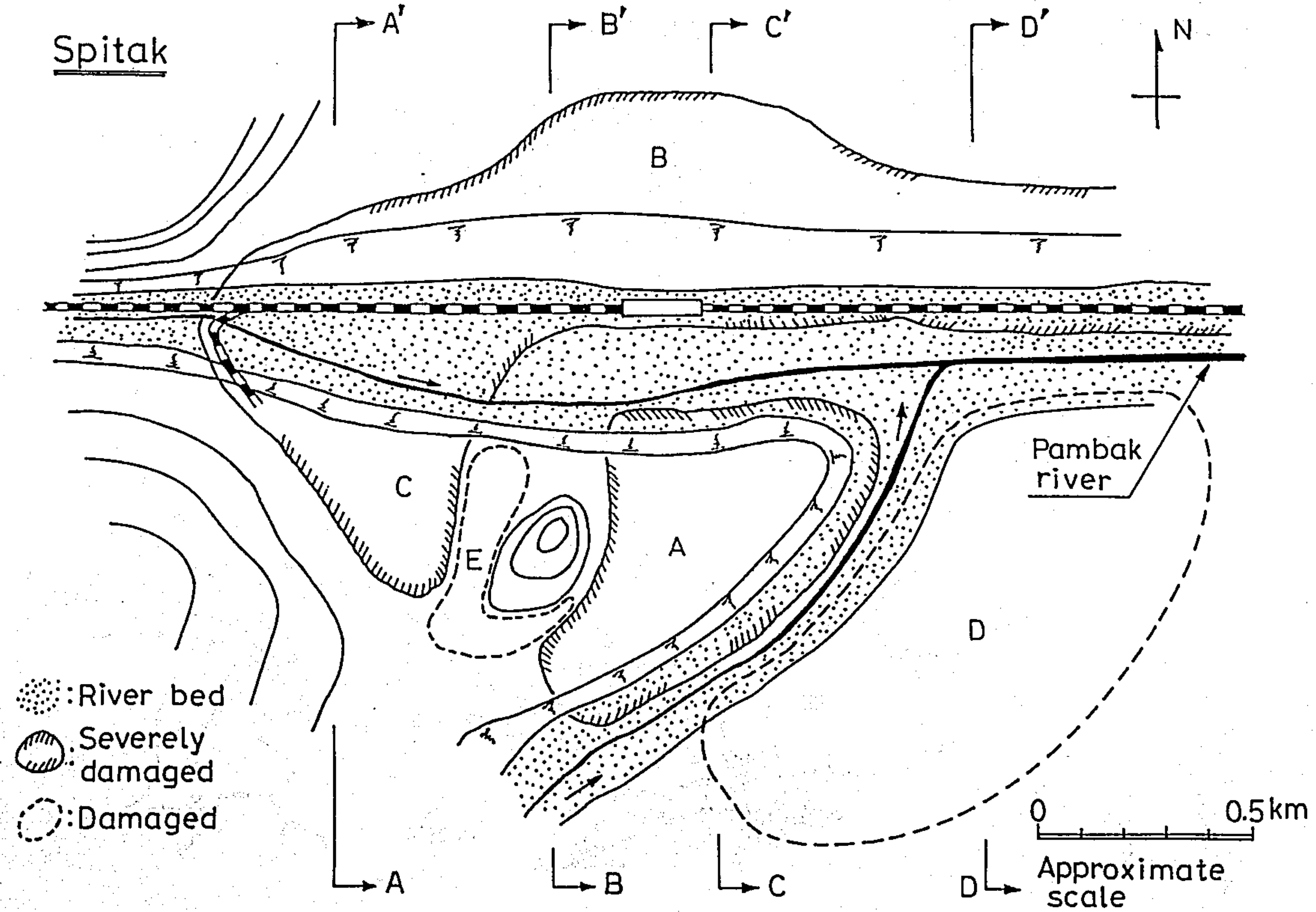

Spitak
A'
B'
C'
D'
N
B
C
E
A
D
Pambak river
:River bed
:Severely damaged
:Damaged
A
B
C
D
0
0.5km
Approximate scale

Fig. 1.2.2

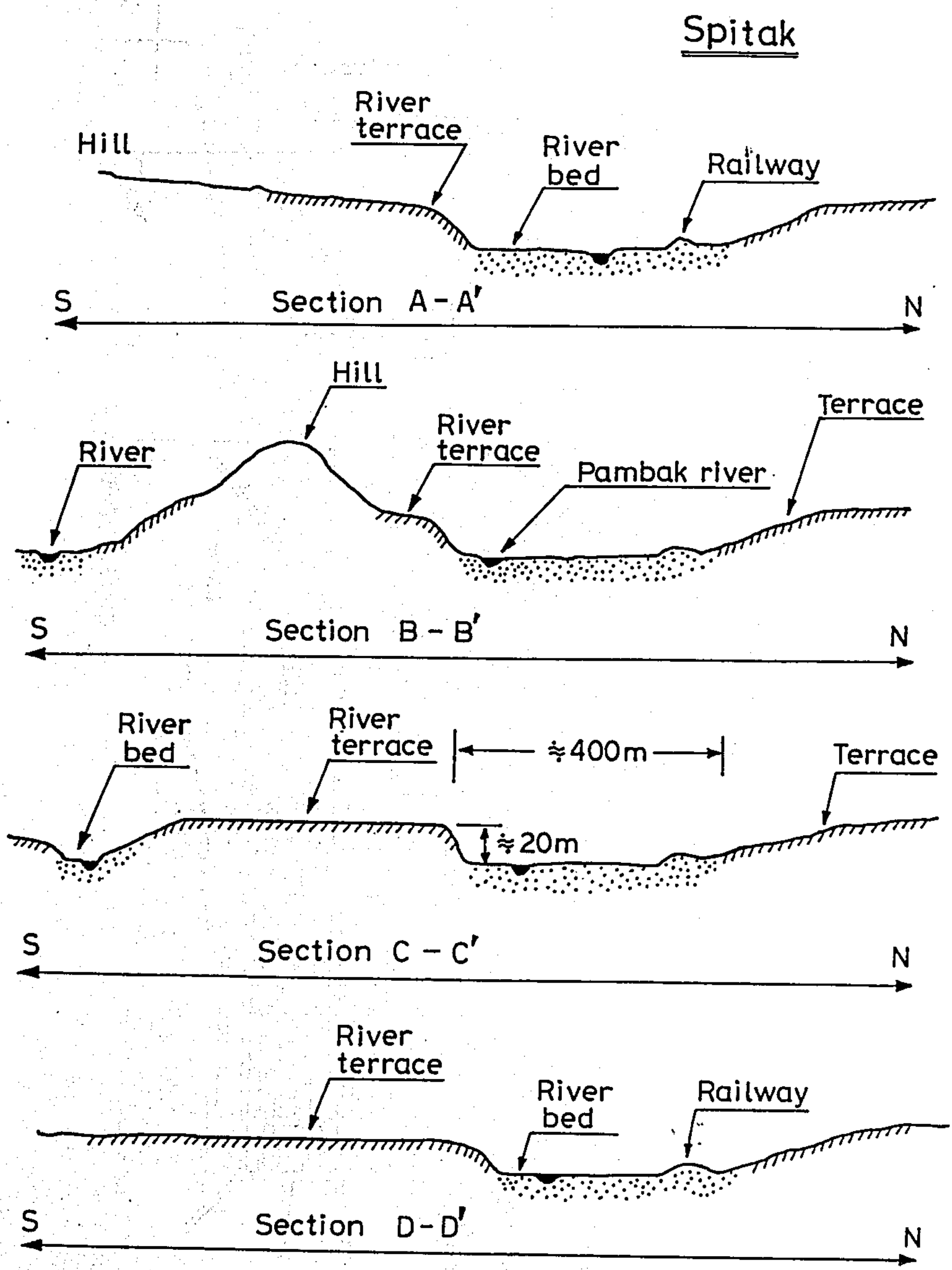

Spitak
Hill
River terrace
River bed
Railway
S
Section A - A'
N
Hill
River
River terrace
Pambak river
Terrace
S
Section B - B'
N
River bed
River terrace
≒400m
Terrace
≒20m
S
Section C - C'
N
River terrace
River bed
Railway
S
Section D-D'
N

Fig. 1.2.3

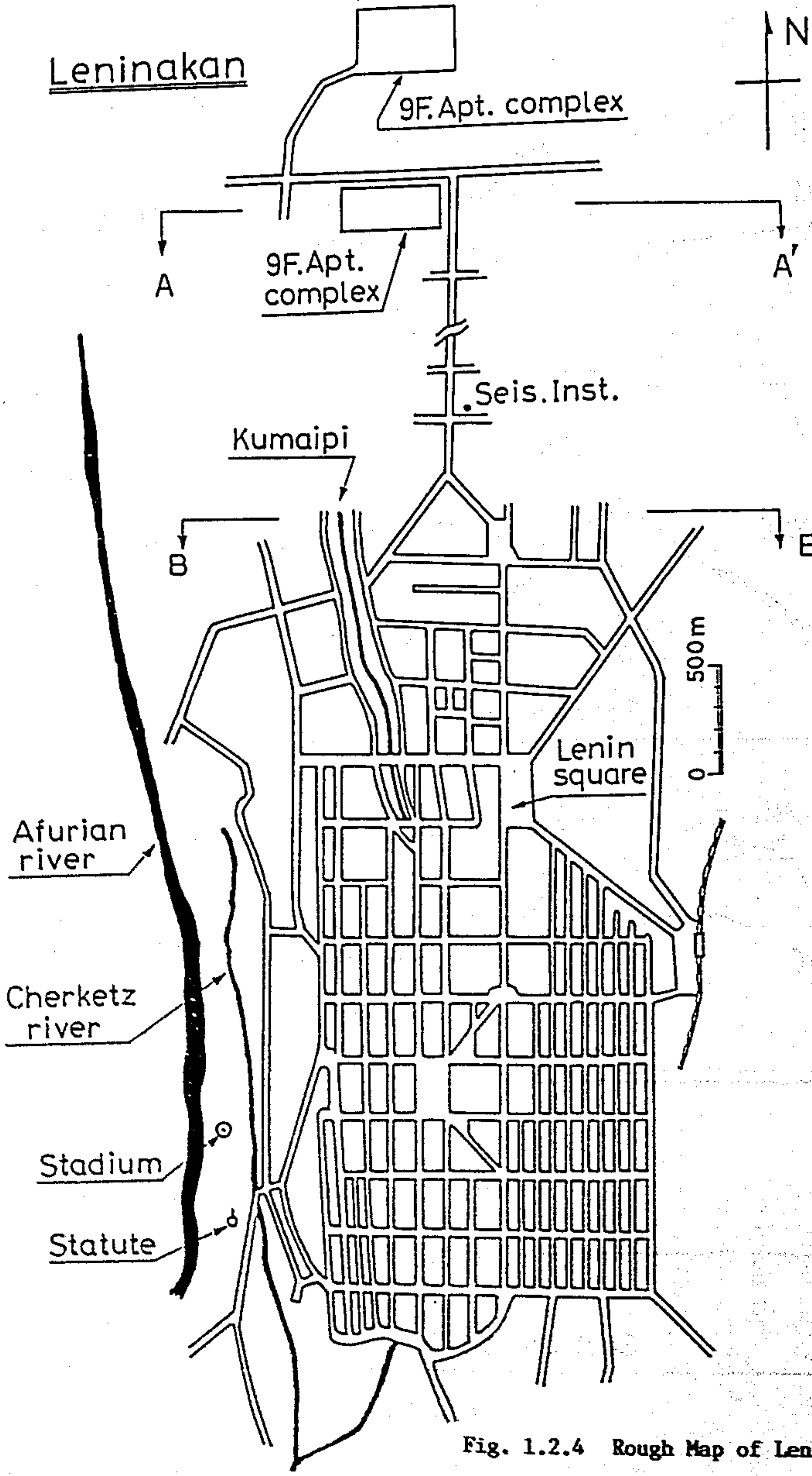
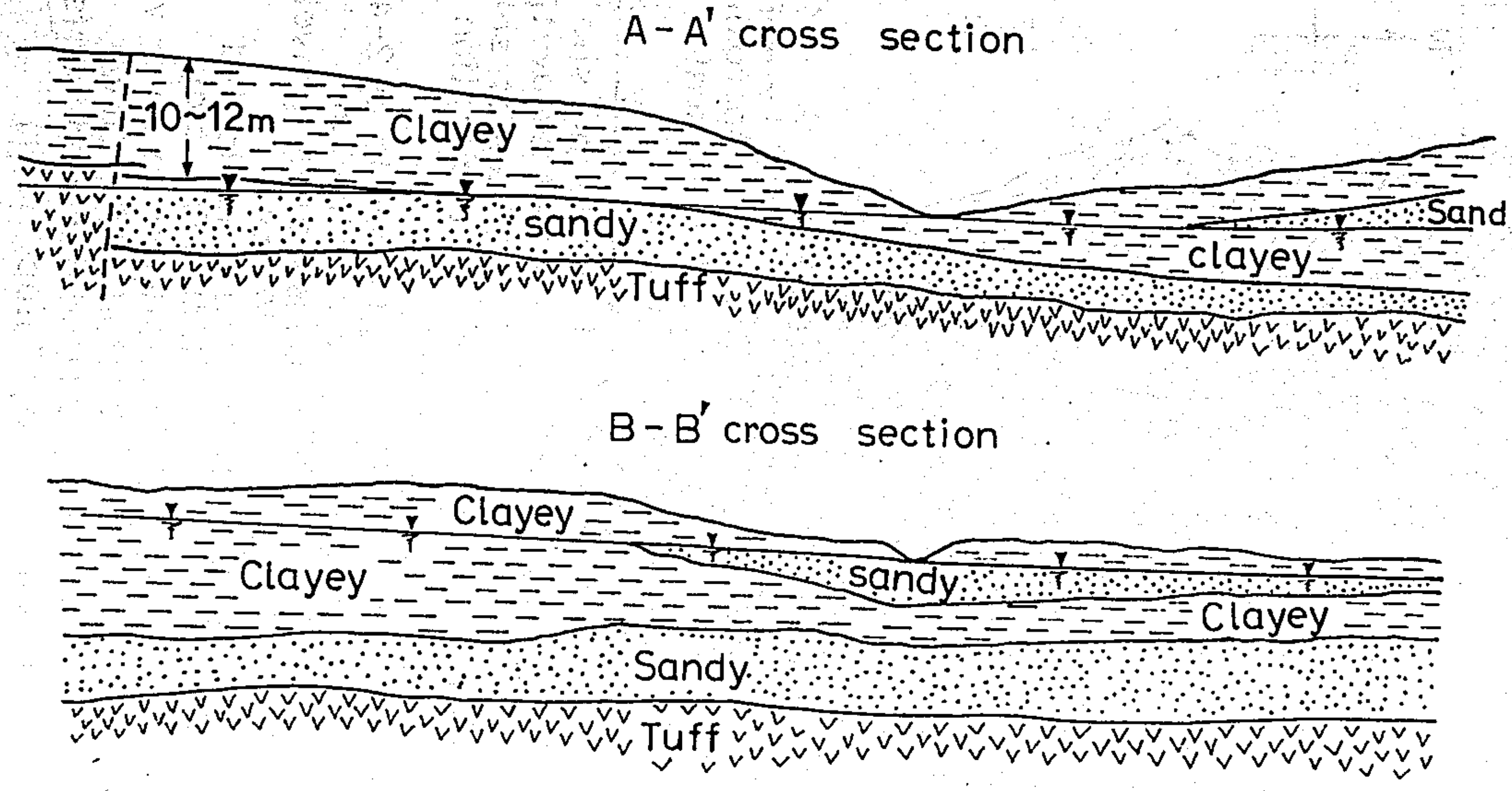
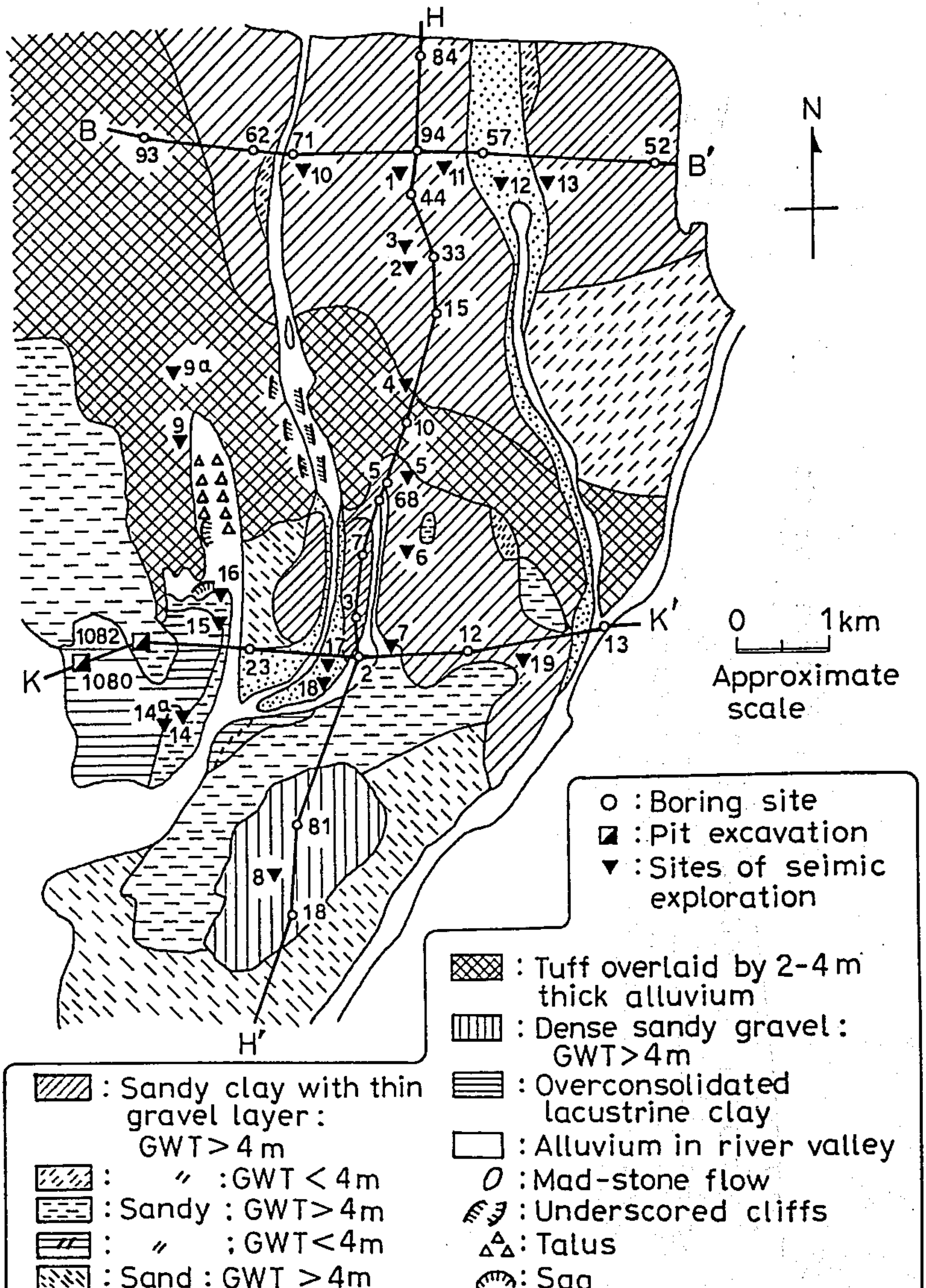



Fig. 1.2.4 Rough Map of Leninakan




Leninakan

Fig. 1.2.5



H
84
B
93
62
71
94
57
52
B'
10
1
11
12
13
44
3
33
2
15
9a
4
9
10
5
5
68
7
6
16
3
15
1082
K'
1080
K
23
17
2
7
12
13
18
19
14a
14
81
8
18
H'
N
0
1km
Approximate scale
o : Boring site
: Pit excavation
▼ : Sites of seimic exploration
: Tuff overlaid by 2-4 m thick alluvium
: Dense sandy gravel : GWT>4m
: Overconsolidated lacustrine clay
: Alluvium in river valley
: Mad-stone flow
: Underscored cliffs
: Talus
: Sag
: Sandy clay with thin gravel layer : GWT>4 m
: 〃 :GWT < 4m
: Sandy : GWT>4m
: 〃 ; GWT<4m
: Sand : GWT >4m
: 〃 : GWT < 4m
*GWT: Ground water table


Fig. 1.2.6

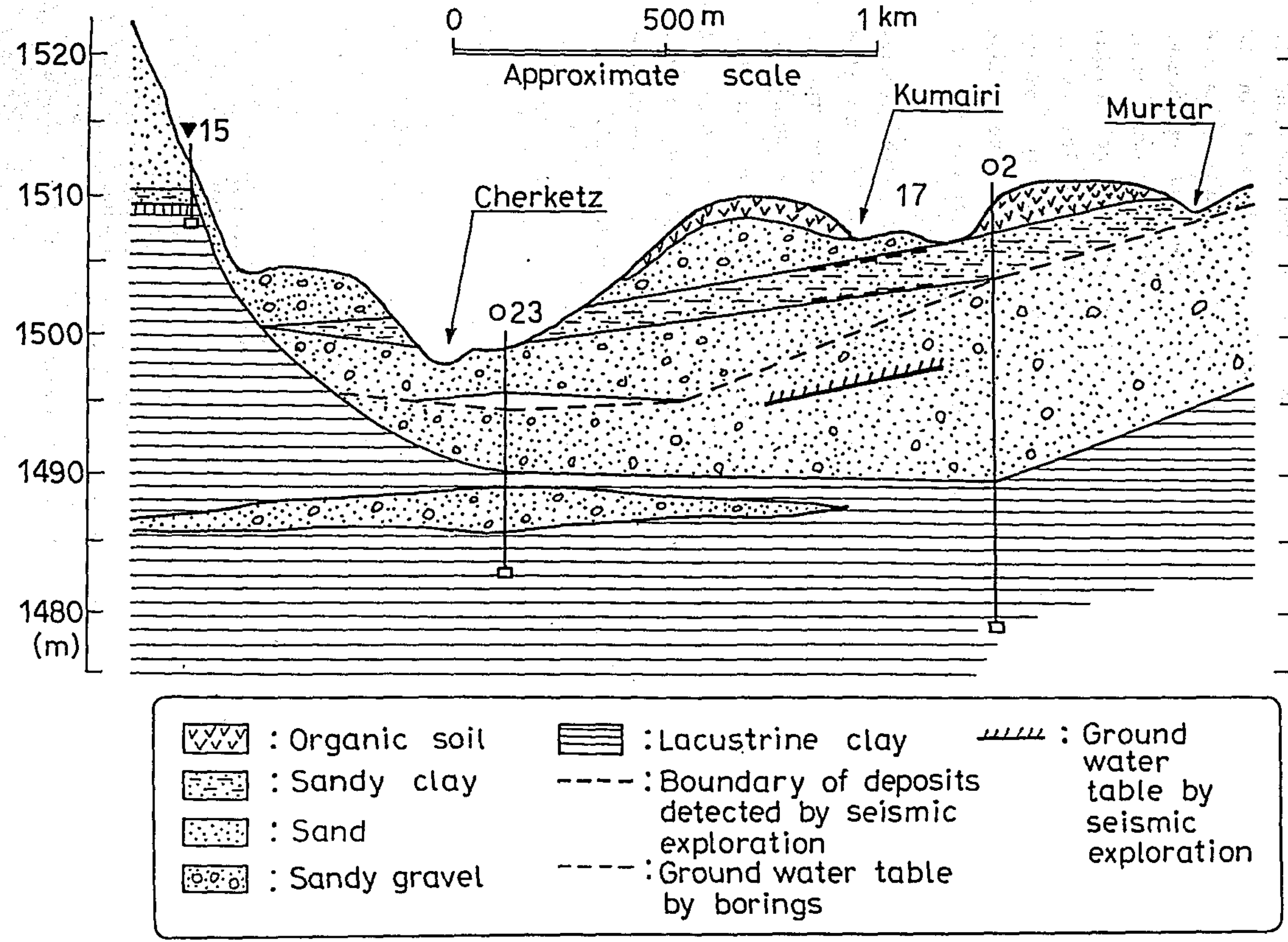
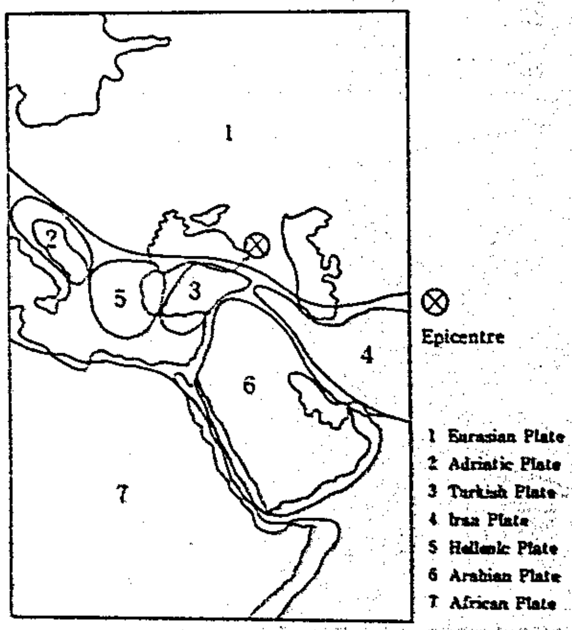

0
500 m
1 km
Approximate scale
1520
1510
1500
1490
1480
(m)
▼15
Cherketz
o23
Kumairi
17
o2
Murtar
: Organic soil
: Sandy clay
: Sand
: Sandy gravel
: Lacustrine clay
: Boundary of deposits detected by seismic exploration
: Ground water table by borings
: Ground water table by seismic exploration

Fig. 1.2.7

## 1.3 Seismicity

The latest Spitak earthquake occurred in the north of the Armenian SSR, which is located in the Caucasia district in the Soviet Union. The area, sandwiched between the Caspian Sea and the Black Sea, constitutes part of the Europe-Asian earthquake belt. The plate-tectonic structure is complicated in this area. The Armenian SSR is located near the southern edge of the Eurasian plate, which is in contact with the Turkish, Arabian and Iran plates (see Fig. 1.3.1). The seismic activities on and near the contact lines among these plates have been attributed to the stress developing along the contact lines between the plates and the resultant strain in the crust.

The seismicity in this area is lower than that in the circum-pan-Pacific earthquake belt, which contains Japan. Within this area, the seismicity in Caucasia is slightly lower than that in Turkey and Iran, which are located on the south of the Black Sea and on the south of the Caspian Sea, respectively. However this is a mere comparison of relative seismicity, and Caucasia did experienced many damaging earthquakes during the present century, indicating that it is always facing the danger of an earthquake that may cause large damage (see Fig. 1.3.1 and Table 1.3.2).




CONTINENTAL SHELVES
IN THE MIDDLE AND NEAR EAST AREA

**Fig. 1.3.1**

PAST EARTHQUAKES

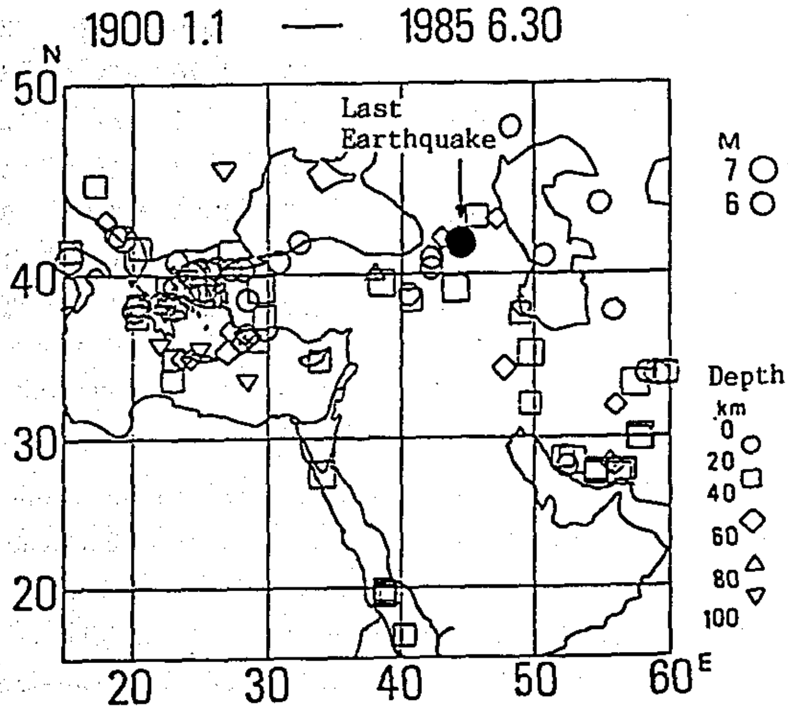



Fig. 1.3.2 Past Earthquakes in the Middle and Near East Area

Table 1.3.1 Recent Damaging Earthquake

| No. | Year | Month | Day | Intensity on Richter Scale | No. | Year | Month | Day | Intensity on Richter Scale |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 1926 | X | 22 | 5.6 | 11 | 1972 | III | 22 | 4.8 |
| 2 | 1930 | VII | 06 | 7.2 | 12 | 1975 | I | 12 | 5.0 |
| 3 | 1931 | iV | 27 | 6.5 | 13 | 1976 | III | 25 | 4.8 |
| 4 | 1932 | III | 15 | 5.6 | 14 | 1976 | IV | 30 | 5.5 |
| 5 | 1935 | V | 11 | 6.0 | 15 | 1976 | XI | 24 | 7.3 |
| 6 | 1954 | X | 30 | 6.0 | 16 | 1978 | II | 15 | 4.8 |
| 7 | 1940 | V | 07 | 6.5 | 17 | 1983 | X | 30 | 7.1 |
| 8 | 1962 | IX | 11 | 7.3 | 18 | 1984 | X | 18 | 4.8 |
| 9 | 1963 | VII | 16 | 6.0 | 19 | 1986 | V | 13 | 5.6 |
| 10 | 1968 | IV | 29 | 5.3 | | | | | |

The No.1 and No.4 earthquakes are the damaging ones occurred in the Armenian SSR in the current century, but their intensities are below 6. Those which occurred near its border are the No.3, No.6, No.13 and No. 19 earthquakes, with a maximum intensity of 6.5. Historically, Armenia experienced a large-scale earthquake with an estimated intensity of approximately 7 in the 11th century.

These data suggest that the latest earthquake is one at the maximum intensity level which is expected to occur in this area every several hundreds of years or every thousand years.

Nevertheless, the distribution of earthquakes with an intensity of 8 or above that have occurred during the last eight years incicates that there are no clear seismicity gaps near the epicenter of the latest earthquake.

## 1.4 Construction Administration/Organization and Disaster Prevention Countermeasures

The team had several meetings with the staff of the Academy of Science of the Armenian SSR and the Armenia Scientific Research Institute of Civil Engineering and Architecture, most of whom specialized in seismic engineering. But the team could not have contact with administrative agencies of construction. It was impossible, therefore, to perform a detailed, comprehensive study on the construction administration, construction organizations, and disaster prevention measures. Information on construction administration obtained from the Research Institute of Civil Engineering and Architecture and other organizations is outlined below.

The Armenia Scientific Research Institute of Civil Engineering and Architecture, which is an archetectural research organization belonging to the National Construction Commitee of the republic, was our major counterpart for exchanging opinions. Though being a research organization, the institute seemed to be deeply involved in establishing design standard for buildings. Thus, necessary data including standard design drawings for damaged buildings were obtained from this institute.

The National Construction Committee is an administrative agency belonging to the Ministerial Council, which performs the function of the republic's government, and Japanese may regard it as a "ministry of construction" at the republic level. In the Soviet Union in recent years, there are moves towards strengthened autonomy of republics, but (according to Japanese Embassy at Moscow) the Kremlin still has strong powers and the committee is under strict guidance of the National Construction Committee of USSR.

Construction standards including anti-earthquake standards for buildings have been established by the Soviet National Construction Committee. We have heard, however, that each republic proposes construction standards for buildings constructed of materials peculiar to each area, and the republic's Research Institute of Civil Engineering and Architecture is directly involved in the planning of these standards.

According to information we obtained, a responsible department of the National Construction Committee of the Armenian SSR carries out inspection during the planning and costruction phases to ensure that buildings will be completed to meet the construction standards. The committee is a large organization covering all fields of construction and comprising departments that correspond to the construction administrative agencies in Japan, and those which are directly in charge of construction works (corresponding to construction companies).

During our latest survey, the Institute of Geology, a research organization within the Academy of Science of the Armenian SSR, offered facilities to us. This institute was also in charge of the U.S. team, which participated in the survey several days behind us. Apparently, it seemed to have been decided that the institute be in charge of all foreign teams.

It appeared that the Institute of Geology was playing the role of a contact point between the central government and Armenian agencies in handling matters related to the earthquake, as infered from the fact that the first meeting in which the vice president of Science of USSR took part was held in this institute.

## 1.5 Aseismic Design Method

(1) Buildings

Design seismic intensity for buildings damaged by the latest earthquake is discussed below in connection with the references given below concerning seismic design codes of the Soviet Union. Most of the damaged buildings were found to have been designed according to the 1977 code, which are not largely different from the 1969 code. Thus, the 1969 code are discussed below based on references a, b and c.

a) Toshihiko Hisada, Kyoji Nakagawa and Hikaru Saito, "Earthquake Engineering and Aseismic Design Code", Kenchiku Zasshi 1961-6.

. The first codes (1957 ed.) that were formulated in the same form as the current aseismic design code (1981 ed.) are outlined.

b) Construction in Earthquake Regions Design Code
SNSP II-A, 12-69
Translated from the Russian by the Building Research Station Garston, Watford, England

. English translation of the 1969 code. IAEE World List 197.

c) CTRONTEЛblbE HORMbl N ЛPABNЛA
CHNЛ、II－7－81 MockBa 1982

. The current code (1981 Rev.). Russian original. Obtained from the Armenia Scientific Research Institute of Civil Engineering and Architecture.

1) History of Aseismic Design Code in Soviet Union

It was in 1940 that the first nationwide aseismic design code was established in the Soviet Union, and the first aseismic design code formulated in the same form as the current one was issued in 1957 (according to Reference a). Later, the code was revised in 1969, 1977 and 1981, but no significant modifications have been made since 1957. (According to information from Armenia Scientific Research Institute of Civil Engineering and Architecture)

2) Outline of Aseismic Design

i) General

The design earthquake force, Sik, base shear of the i'th other mode is expressed by Equation 1.

$$S_{ik} = Q_k \cdot K_c \cdot \beta_i \cdot \eta_{ik} \quad (1)$$

where Qk is the weight of the Kth story. The following parameters are used as load coefficients.

| | |
|---|---|
| Fixed load | -0.9 |
| Permanent | -0.8 |
| Temporary floor load and snow load | -0.5 |

$K_c$ : design base shear coefficent as given in Table 1.5.1

$\beta_i$ : i'th order acceleration response ratio.

$\eta_{ik}$ : i'th order mode coefficient as expressed by Equation 3.

ii) Design base shear coefficient Kc is set up as in Table 1.5.1 based on the design intensity.

Table 1.5.1 Design Ground Motion

| Design base shear coefficient | 7 | 8 | 9 |
|---|---|---|---|
| Kc | 0.025 | 0.05 | 0.1 |

The design base shear coefficient is established by the following procedure: (1) find an intensity, 6, 7, 8 or 9*, for the standard ground (sands and clays and low ground water level -6m or below) given in the regional maps, which have been developed for all parts of the Soviet Union (the map for Armenia is shown in Fig. 1.5.1), (2) then revise the values taking into account the hardness or softness of the ground (Table 1.5.2), and (3) further correct them taking into account the use of each building (Table 1.5.3a). (* nearly the same as the MSK intensity)

iii) Acceleration Response Ratio $\beta i$

$\beta i$ is calculated by Equation 2. Its value should be between 0.8 minimum and 3.0 maximum.

$$\beta i = 1 / Ti \tag{2}$$

where Ti is the i'th order natural period of the building.

iv) Mode Coefficient $\eta ik$

In general, this coefficient is calculated by Equation 3.

$$\eta ik = Xi(xk) \sum_{j=1}^{n} Qj \cdot Xi(xj) \Big/ \sum_{j=1}^{n} Qj \cdot Xi^{2}(xj) \tag{3}$$

where $Xi(xk)$, $Xi(xj)$ are the displacement in case of free vibration at Point k (k-th story) and Point j (j-th story), respectively.

v) Other Rules Related with $\beta i$ and $\eta i$

Detailed rules for $\beta i$ and $\eta i$ are as follows.

a. The value of $\beta i$ should be increased by 50 percent for tower-like structures including towers, masts and chimneys.

b. The earthquake force should be multiplied by the following factor for building which have more than five stories.

- General buildings

  1 + 0.1(n-5), but should not be more than 1.4,

- Large panel buildings and solid RC wall buildings

  1+0.06(n-5), but should not be more than 1.3,

  where n represents the number of stories.

c. Design earthquake force is multiplied by 0.8 for factory buildings of one story frame construction with beam height of no more than 8m and span of no more than 18m.

d. $\beta i$ may be assumed to be 3.0 and $\eta i$ may be calculated by Equation 4 for buildings having no more than five stories.

$$\eta ik = Xk \sum_{j=1}^{n} Qj \cdot xj / \sum_{j=1}^{n} Qj \cdot xj^2 \tag{4}$$

where $Xk$ and $xj$ represent the height of Point k and Point j, respectively, from the foundation.

e. The $\beta\eta k$ values given in Table 1.5.4 may be used for earthquake-resistant wall buildings having no more than five stories.

b) Design Stress

Where the first order natural period is 0.5 sec or longer, the first to third order modes should be taken into account in calculating the design earthquake force. The mode superposition is performed by Equation 5.

$$Nr = N_{max}^2 + 0.5 \sum_{1}^{n} Nj^2 \tag{5}$$

where $Nr$ = stress,

$N_{max}$ = stress of an order with largest effect,

$Nj$ = stress of other order excluding $N_{max}$.

Only the first mode may be taken into account where the first order natural period is shorter than 0.5 sec.

c) Design of Members

The design of members has been done based on the ultimate strength theory according to information from the Armenia Scientific Research Institute of Civil Engineering and Architecture, and there seem to be other design standards that specify more details.

3) Rough Calculation of Design Shear Coefficient Used for Damaged Building

Calculation is made below of the design shear coefficient (base shear coefficient $C_B$) at the first story of five-storied, stone-built apartment houses in Spitak, Leninakan and Kirovakan.

The following equation is used.

$$C_B = K_C \times \beta \times \frac{1}{n} \sum_{1}^{n} \eta k \times (\text{correction factor})$$

The effects of higher orders modes are ignored since they do not affect the value of $C_B$ significantly.

a) Design Intensity (for residence)

Spitak 7

Leninakan 8

Kirovakan 7

b) Design Ground Motion Factor $K_C$

Spitak and Kirovakan $K_C = 0.025$

Leninakan $K_C = 0.05$

c) Five-Storied Masonry Apartment House

i) Leninakan

The average $\beta\eta k$ is 2.46 if it can be assumed that all layers have the same weight and that $\beta\eta k$ is as given in Table 1.5.5.

$C_B = 0.05\times2.46 = 0.123$

Then, $C_B \fallingdotseq 0.11$ assuming that the load factor is approximately 0.9.

ii) Spitak and Kirovakan

$C_B = 0.05\times2.46 = 0.06$

d) Nine-Storied, PC-Frame Apartment House

i) Leninakan

- $T_1 = 0.6\sim0.85$ sec (according to information from the Armenia Scientific Research Institute of Civil Engineering and Architecture)

$\therefore \beta = \frac{1}{0.6}\sim\frac{1}{0.85} = 1.7\sim1.2$

(normally $1.7\sim1.2$, according to information from the same institute)

- $\Sigma\eta k/n \fallingdotseq 0.86\sim0.9$ where all layers has the same rigidity.
- Correction factor for high building

$1+0.1(n-5) = 1+0.1\times4 = 1.4$

$\therefore C_B = 0.05\times(1.2\sim1.7)\times(0.86\sim0.9)\times1.4 = 0.07\sim0.107$

$C_B = 0.06\sim0.10$ (if the load factor can be assumed to be approximately 0.9.)

ii) Kirovakan

$C_B = (0.07 \sim 0.107)/2 = 0.04 \sim 0.05$

$C_B = 0.03 \sim 0.05$ (if the load factor can be assumed to be approximately 0.9.)

e) Nine-Storied PC-Panel Apartment House

i) Leninakan

- $T_1 = 0.35 \sim 0.4$sec (according to information from the Armenia Scientific Research Institute of Civil Engineering and Architecture)

$\therefore \beta = 2.5 \sim 2.85$

- $\Sigma \eta k/n$ $0.86 \sim 0.9$
- Correction factor for high building

$1+0.06(n-5) = 1+0.06\times4 = 1.24$

$\therefore C_B = 0.05\times(2.2 \sim 2.85)\times(0.86 \sim 0.9)\times1.24 = 0.133 \sim 0.16$

$C_B = 0.12 \sim 0.14$ (if the load factor can be assumed to be approximately 0.9.)

f) Table 1.5.4 lists values of the base shear coefficient for design for different cities (rough calculations).

Table 1.5.4 Rough Calculation of Design Base Shear Coefficient for Different Cities

| | Spitak | Leninakan | Kirovakan |
|---|---|---|---|
| Five-storied stone building | 0.06 | 0.11 | 0.06 |
| Nine-storied PC-frame building | - | 0.06-0.1 | 0.03-0.05 |
| Nine-storied PC-panel building | - | 0.12-0.14 | - |

4) Aseismic Design Code of 1981

Details are unknown because only the Russian original is available. Information obtained from the Armenia Scientific Research Institute of Civil Engineering and Architecture is outlined below.

a) Design Earthquake Force

$$S_{ik}=K_1 \cdot K_2 \cdot Q_k \cdot A \cdot \beta_i \cdot K\phi \cdot \eta_{ik} \qquad (1')$$

$K_1$: permissible damage coefficient (Table 4.5.5)

This is a newly adopted coefficient and has the following values.

1.0 for important building

0.25 for general building

0.12 for warehouse, etc.

$K_2$: improved form of an existing correction factor related with the height

$K_2 = 1+0.1(n-5)$ for frame structure

$K_2 = 0.9+0.075(n-5)$ for panel structure

$K_2 = 1.3$ for panel structure

A: design earthquake motion

This corresponds to the conventional $K_c$ parameter. It is four times as large the latter and has the values of 0.1, 0.2 and 0.4 for the intensities of 7, 8 and 9, respectively.

After being multiplied by $K_2$, however, it gives 0.025, 0.05 and 0.1 for general houses, which are equal to the values of the conventional parameters.

$\beta_i$: one of three values is selected according to the type of soil profile

$$\beta_i = \frac{1}{T_1} \leqq 3.0 \quad \text{for Type I soil profile} \qquad (3')$$

$$\beta_i = \frac{1.1}{T_1} \leqq 2.7 \quad \text{for Type II soil profile} \qquad (4')$$

$$\beta_i = \frac{1.5}{T_1} \leqq 2 \quad \text{for Type III soil profile} \qquad (5')$$

$K\phi$: conventional multiplier for tower-like structure in the range of 1.0-1.5.

$\eta_k$ : conventional factor

b) Design Stress

Effects of high-order modes are calculated by Equation 8.

$$N_p = \sum_{j=1}^{n} N_j^2 \qquad (8)$$

c) Others

The zoning map is slightly modified (Fig.4.5.3).

Table 1.5.2 Correction of Design Intensity in Terms of Ground Condition and Groundwater Table

| Type of Ground | Soil Condition | Standard Intensity 7 | 8 | 9 |
|---|---|---|---|---|
| I | Near-rock soil base, cracked igneous rock, metamorphic rock, Sedimentary rock -- granite, Gneiss, limestone, sandstone, conglomerate | 6 | 7 | 8 |
| | Slightly near-rock soil base -- mudstone, oxidized clay, clayey sandstone, tuff, shell rock, gypsum | | | |
| | Dense ground, very large grain size, groundwater table 15m or deeper | | | |
| II | Clay, loam sand, clayey sand Groundwater table 8m or deeper | 7 | 8 | 9 |
| | Large particle size / Groundwater table 6-10m. | | | |
| III | Clay, loam, sand, clayey sand Groundwater table shallower than 4m | 8 | 9 | >9 |
| | Large particle size Groundwater table 3m or shallower | | | |

Note: The value of design intensity for important buildings should be set up after obtaining approval of the National Construction Committee of the Soviet Ministerial Council.

Table 1.5.3a Correction of Design Intensity for Building and Other Structure

| Building and Structure | Seismic Intensity for Construction Site | | |
|---|---|---|---|
| | 7 | 8 | 9 |
| (1) Building and structure for residence, public use or manufacturing, excluding those under 2, 3 or 4 below. | 7 | 8 | 9 |
| (2) Important building and structure used by Soviet or Republic agencies*. | 8 | 9 | 9** |
| (3) One-storied factory, small workshop, etc. with 50 or less workers and without expensive equipment, including agricultural buildings for long-term use by farmers. | 7 | 7 | 8 |
| (4) Building and structure whose destruction does not cause loss of human lives or damage to expensive equipment (excluding building and structure which should be protected from damage by aftershock). Including agricultural building and temporary building which are not used for living. | Effect of earthquake is not taken into account. | | |

* Buildings and structures in this category are designated after obtaining approval from the Soviet or Republic Construction Committee.

** For structure design of buildings on this category, the load corresponding to the design intensity of 9 is further multiplied by 1.5.

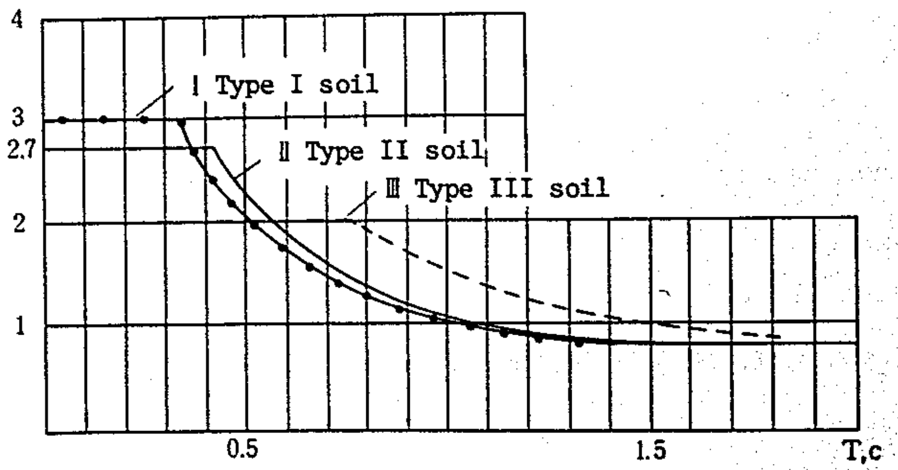



Fig. 1.5.2

Table 1.5.3b Correction of Design Intensity for Road of Different Importance and Structure of Different Size

| Type | Standard Intensity | | | |
|---|---|---|---|---|
| | 6 | 7 | 8 | 9 |
| (1) Large-size bridge on national-level Class I or II road or railroad, urban expressway, or important road in city. | 7 | 8 | 9 | – |
| (2) Large-size bridge on national-level Class III or IV road or railroad, or in city. Medium-size bridge on national-level Class I or II road or railroad, urban expressway, or important road in city. | 6 | 7 | 8 | 9 |
| (3) Medium-size bridge on national-level Class III or IV road or railroad, important road in city, or spur to factory. Small-size bridge, culvert, retaining wall, wooden bridge on road of all classes. | 6 | 6 | 7 | 7 |

Table 1.5.4 Value of $\beta\eta k$

| Story | Number of Stories of Building | | | | |
|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 |
| 1st story | 3 | 1.8 | 1.3 | 1 | 0.8 |
| 2nd story | – | 3.6 | 2.6 | 2 | 1.6 |
| 3rd story | – | – | 3.9 | 3 | 2.5 |
| 4th story | – | – | – | 4 | 3.3 |
| 5th story | – | – | – | – | 4.1 |

Table 1.5.5 Value of $K_1$

| Permissible Damage to Building and Structure | Value of Coefficient $K_1$ |
|---|---|
| (1) Structure which should be free from residual deformation or local damage (depression, crack, etc.)* | 1 |
| (2) Building and structure which are permitted to suffer residual deformation, crack, or individual member's destruction that will not affect the safety of human life or preservation of the facility though it may not function normally. (Building and structure for residence, public use, manufacturing, agriculture. Structure for irrigation and transportation. Energy and water supply system, garage for fire engine, fire fighting system, certain structure for communication, etc.) | 0.25 |
| (3) Building and structure which are permitted to suffer large residual deformation, crack, or individual member's destruction that may hinder the use of the facility temporarily, unless it affects the safety of human life. (One-storied building for manufacturing or agriculture free from expensive equipment) | 0.12 |

* List of structures of the first category should be approved by GOSSTROI USSR.

Table 1.5.6 Value of $K_2$

| Structural Type of Building | Value of Coefficient $K_2$ |
|---|---|
| (1) Frame structure large-block building having composite wall. The number of stories (n) is five or more. | $K_2 = 1+0.1(n-5)$ |
| (2) Building having large panel type walls or monolithic reinforced concrete walls. The number of stories is less than five. | 0.9 |
| (3) Same as above, but the number of stories is five or more. | $K_2 = 0.9+0.075(n-5)$ |
| (4) Building whose lower portion (one or more stories) is of frame structure and upper portion has load bearing walls or has a frame comprising diaphragm or filler. In the upper portion, filler is not used or, if in use, has little effect on the rigidity. | 1.5 |
| (5) Building with load bearing walls comprising stone blocks or bricks that are laid manually without using filler to increase the binding capability. | 1.3 |
| (6) One-storied, frame building with a height of the beam from the bottom of the truss of less than 8m and a span of 18m or less. | 0.8 |
| (7) Agricultural building supported by pile-type columns and constructed on Type III soil (Table 7). | 0.5 |
| (8) Building and structure not belonging to any of the above categories. | 1 |

Note: The value of $\phi$ and $K_2$ should not exceed 1.5.

The value of $K_2$ may be set up based on test results after consulting GOSSTROI.

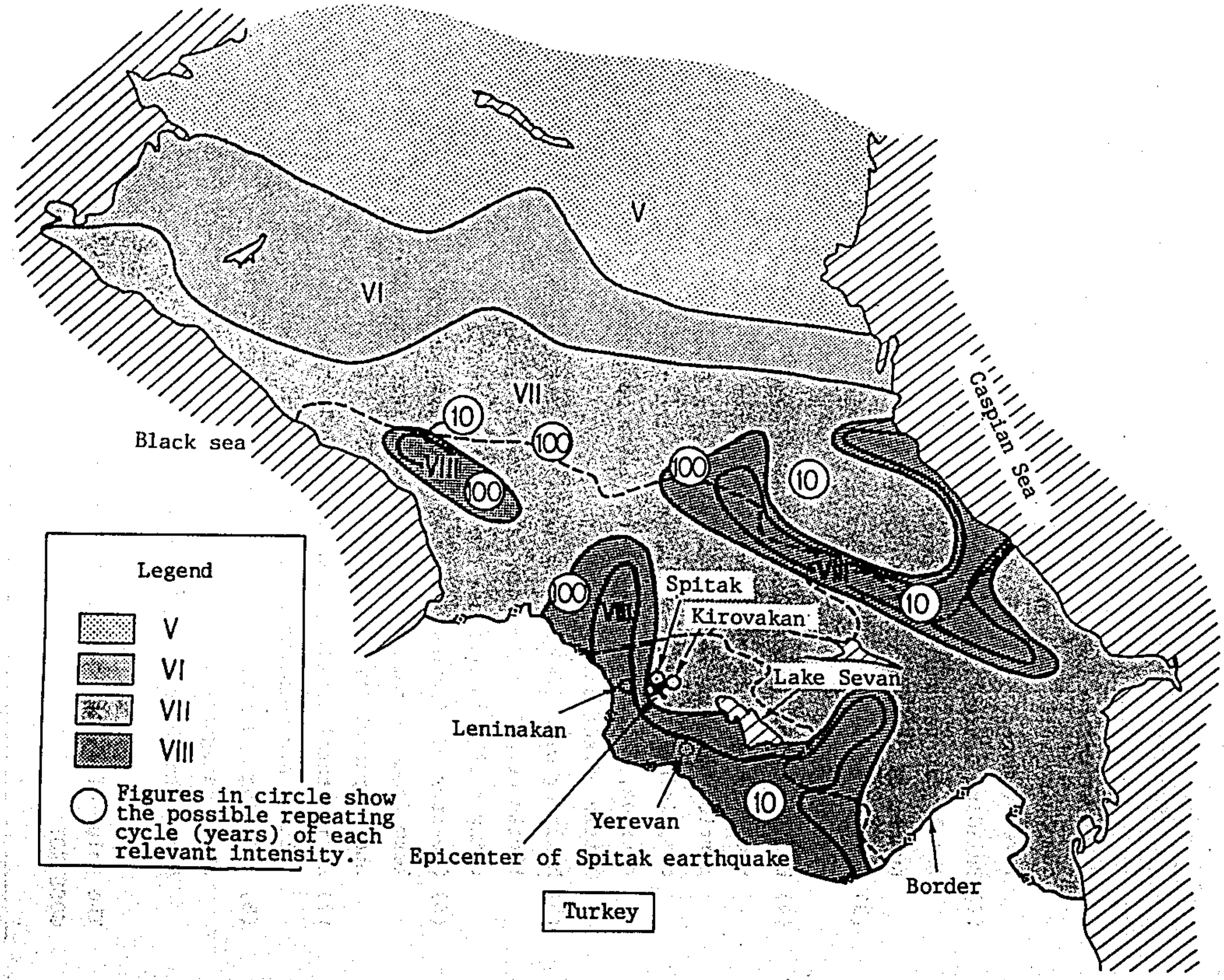



Fig. 1.5.1 Design Intensity Used for Aseismic Design

(2) Civil Engineering Structure

1) General

The 1969 aseismic design standards (SN&P II-A. 12-69) specify requirements for techniques and methods for aseismic design not only for buildings but also for traffic facilities (on road and railroad) including bridge, culvert under banking, retaining wall, and tunnel. The standards cover aseismic design methods for buildings to be constructed in seismic zones where the design intensity is 7, 8 or 9. It is required that basically the following three approaches be used to increase the seismic capability.

1) A site with the lowest earthquake risk should be selected for the construction.

2) Seismic design techniques should be used.

3) Construction works should be carried out in an appropriate way.

2) Design Earthquake Force

a) Design Intensity

In setting up a design intensity, the standard intensity (see Fig. 1.5.1) should be corrected to meet the ground conditions given in Table 1.5.2, as in the case of ordinary buildings, and the use of the related road or railroad as shown in Table 1.5.3b should be taken into account.

b) Design Earthquake Force

The design earthquake force is calculated by the equation given below.

$$S_{ik} = Q_k \cdot K_C \cdot \beta_i \cdot \eta_{ik} \qquad (1)$$

where,

$S_{ik}$ = design earthquake force at Point $\underline{k}$ in i'th degree mode,

$Q_k$ = weight at Point $\underline{k}$. Calculated from live load and dead load excluding hydrostatic pressure,

$K_C$ = design ground vibration intensity given in Table 2,

$\beta_i$ = i'th order acceleration response ratio expressed by the following equation.

$$0.3 \leq \beta i = 1/Ti \leq 3.0 \tag{2}$$

where, Ti is the igen period of the i'th order,

$\eta_{ik}$ = mode coefficient of i'th order expressed by the following equation,

$$\eta_{ik} = X_i(X_k) \sum_{j=1}^{n} Q_j \cdot X_j(x_j) \Big/ \sum_{j=1}^{n} Q_j \cdot X_j^2(x_j) \tag{3}$$

c) Calculation of Design Forces

Where the fundamental natural period is shorter than 0.5 sec, the aseismic design calculation may be carried out for the earthquake force expressed by Equation 1 while taking only the first order mode into account. Where the fundamental natural period is longer than 0.5 sec, the design forces (axial tension, shear force, bending moment) should be calculated by using the following equation to take into account the modes up to the third order.

$$N_r = N_{max}^2 + 0.5 \sum_{j=1}^{n} N_i^2 \tag{4}$$

where $N_r$ = cross sectional force,

$N_{max}$ = cross sectional force due to the mode having the largest effects on the relevant cross section,

$N_i$ = cross sectional force imposed on the cross section with Nmax due to all modes excluding Nmax mode.

d) Design of Members

Strength and stability (shape and location) should be well considered in performing the design of cross sections.

It appears that the calculation of strength has been performed based on the first limit state load bearing capacity, though details are unknown. A service factor (Mkr), which takes the following values, is used in this calculation.

Mr = {
- 1.4 for steel and wood member
- 1.2 for plain-concrete, RC, PC or stone structure
- 1.0 steel member, RC strut or welded portion in stone structure which suffers from shearing or compression

3) Trial Calculation of Design Earthquake Force Used for Civil Engineering Structure in Damaged Area

Rough calculation of the design intensity for civil engineering structures in the disaster-stricken area is shown below.

a) Standard Earthquake Intensity

7 for Spitak and Kirovakan

8 for Leninakan

b) Correction of Standard Earthquake Intensity for Soil Condition

The soil is considered to be of Type I or II, though its type has not been identified accurately.

If it is of Type I,

6 for Spitak and Kirovakan,

7 for Leninakan.

If it is of Type II,

7 for Spitak and Kirovakan,

8 for Leninakan.

c) Correction of Standard Earthquake Intensity for Type of Road and Size of Structure

Necessary data are not available, but if Type II is assumed,

7 for Spitak and Kirovakan,

8 for Leninakan.

d) Design Intensity

Reliable information is not available as to what methods have been used to superpose the effects under b) and d) for the correction of standard intensity. If they are taken into account independently, the design intensity will be as follows.

If the soil is of Type I,

6 for Spitak and Kirovakan,

7 for Leninakan.

If the soil is of Type II,

7 for Spitak and Kirovakan,

8 for Leninakan.

e) Design soil vibration intensity Kc

Table 1.5.7

| | Type I Soil | Type II Soil |
|---|---|---|
| Spitak and Kirovakan | - | 0.025 |
| Leninakan | 0.025 | 0.05 |

f) $\eta_{ik} \doteqdot 1.0$ for a bridge with an upper structure of a very large weight.

g) The natural period can be assumed to be in the range of 0.3-0.5 because the damaged area contained only small bridges.

$$\beta_i = \frac{1}{0.3}\ (<3) \sim \frac{1}{0.5} = 3 \sim 2 \qquad (5)$$

h) Thus, the calculation of design horizontal intensity gives the following values (natural period 0.3-0.5 sec).

Table 1.5.8

| | Type I Soil | Type II Soil |
|---|---|---|
| Spitak and Kirovakan | - | 0.05-0.075 |
| Leninakan | 0.05-0.075 | 0.1-0.15 |

4) 1957 Aseismic Code

Hisada et al. have reported that the earthquake force for road and railroad structures was defined as follows in the 1957 Anti-Earthquake Code (CH-8-57).

$$S = Q\,Kc\,\alpha \qquad (6)$$

where,

$Q$ = dead load,

Kc= design soil vibration intensity Kc (given in Table 1.5.7),

$\alpha$ = coefficient that takes into account the dynamic characteristics of structure.

a) For high wall, tower and bridge pier with a shear span ratio of 5 or more, $\alpha = 2$ at the top of the structure, $\alpha = 1$ at the top of the foundation, and linear interpolation is performed for points in between.

b) $\alpha = 5$ for anchor bolt and joint between main girder and supporting point.

c) $\alpha = 1$ for other structures.

5) 1981 Aseismic Code

It has been reported that the 1981 Aseismic Code required the calculation of the earthquake force as shown under Section 1.5.1 (4), as in the case of buildings.

6) Aseismic Design of Bridge

For aseismic design of bridges, a total of 28 sections (Sections 4.17-4.34 for upper structure and Sections 4.35-4.44 for lower structure) are dedicated to structural and other detailed requirements for structures. Major ones are outlined below.

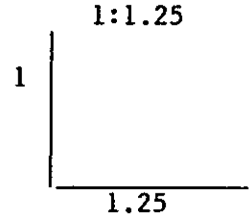
a) Bridge Members Except Basement Structure

1) The gradient of a slope of the banking underlying an abutment in a seismic zone should be one rank higher than that for non-seismic zones, as shown in Table 1.5.9.

Table 1.5.9

| Area | Gradient of Slope | | |
|---|---|---|---|
| Non-seismic zone | 1:1.25 | 1:1.5 | 1:1.75 |
| Seismic zone | 1:1.5 | 1:1.75 | 1:2 |

Slope of 1:1.25




2) The use of a frame structure is recommended where reinforced concrete is adopted.

3) A girder bridge should be preferably used as continuous girders or comprise a completely independent upper structure.

A antilever structure having a hinge should not be used in an area with a standard earthquake intensity of 9.

4) The girders of a girder bridge should not jump off at the supports. The supports should be firmly fixed using special anchors, etc., to prevent shear fracture of at the top of the pier.

b) Basement Structure

1) Bedrock is desirable as the stratum for supporting the foundation. The use of a direct foundation is more desirable than a pile foundation. The bottom face of the foundation should be flat. In general, the bench-cut foundation should not be used except where base rock is available.

2) The abutment should have a simple shape. A stone abutment should not be used in areas with a standard earthquake intensity of 9. If possible, its use should also be avoided in areas with a standard intensity of 7 or 8.

3) A single-pillar type concrete or masonry pier should not be adopted in areas with a standard earthquake intensity of 9. If possible, its use should also be avoided in areas with a standard intensity of 7 or 8. Where frame-type piers are used, they should be installed on the same footing except where base rock is available.

4) At the joints of a pier with the footing or the upper structure, the main reinforcement should be fully extended and a particularly high strength should be ensured in sections whose width is equal to the effective width of the bridge. In areas with a standard earthquake intensity of 9, a bridge should not have abrupt changes in cross section, and joint faces should be strengthened by using gently curved faces or diagonal reinforcement.

5) Where a large cross section is required, concrete piers may be built by binding large-block type precast members. However, the joint portions should be connected properly to ensure that they are fully resistant to tensile and shear forces to allow the piers to behave in harmony.

7) Requirements for Aseismic Design of Culvert under Banking

1) A culvert should preferably have an arched shape. A culvert made of unreinforced concrete should have a box shape and a lid of PC structure should be provided.

2) When built along hill in an area with a standard earthquake intensity of 8 or 9, a culvert for railroad should be of RC construction and a culvert for road may be of RC construction, unreinforced concrete, or masonry.

3) Where a culvert is divided in its long direction, all portions should be installed on ground in the same geological condition.

8) Requirements for Aseismic Design of Retaining Wall

A retaining wall should be of RC construction, unreinforced concrete, or mortar-mixing masonry. Sun-dried bricks may be used for very small ones. Retaining walls which are not of RC construction should meet the height requirements given in Table 1.5.10.

Table 1.5.10

| Material | Standard Earthquake Intensity | |
|---|---|---|
| | 8 | 9 |
| Unreinforced concrete | 12m | 10m |
| Motar-mixing masonry | 12m | 8m (railway)<br>10m (road) |
| Sun-dried brick | 3m | |

9) Requirements for Aseismic Design of Tunnel

1) A tunnel with a its depth of up to 50m that exists in an area with a standard earthquake intensity of 9 should be provided with a lining of RC construction considering the movement of the ground.

2) In a tunnel in an area with a standard intensity of 8 or 9, sections with the depth of 15m or less near a mouth should also be provided with an RC lining. Precast members may be employed,

but should be connected adequately by means of cast-in-place concrete.

3) Water should be removed adequately from the natural ground around the mouths to prevent the tunnel from being blocked due to collapse of the ground near the mouths.

## 1.6 Construction Method

### (1) Construction Method Used in Armenia and Structural Feature

#### 1) Construction Method

The major feature concerning the construction methods used in Armenia is that stone and precast reinforced concrete are employed very frequently for various types of buildings while there are few buildings made of cast-in-place concrete, wood or steel frame.

Stone buildings account for by far the greatest part of the apartment houses having five or less stories. Most of the other low buildings including temples, government buildings and private houses are also built of stone. The former are strengthened by using reinforced concrete (hereafter referred to as strengthened stone and RMS building), while the latter are mostly unreinforced.

The precast reinforced concrete buildings are roughly divided into two groups: rigid-frame type (hereafter referred to as RPC building) and wall type (WPC). The RPC buildings account for by far the greatest portion of the total. The RPC technique is used also for constructing one- or two-storied factories. Some buildings are constructed of special precast concrete or by the combination of precast concrete and cast-in-place RC.

#### 2) Structural Feature

Major structural features, other than technical features, of damaged buildings in Armenia are outlined below.

a) Very Frequent Use of Stone Material

Armenia is very rich in tuff, which is widely used not only as structural material for stone buildings but also as exterior finishing material or concrete aggregate for RPC and WPC buildings.

The stone is porous and its color is mostly purple and ranges from blackish to yellowish. Its compressive strength is in the relatively low range of about 25-400kg/cm$^2$, which is comparable to that of Oya-ishi in Japan.

Photograph 1.6.1 shows apartment houses along the main street in capital Yerevan. The high building is made of RPC with tuff exterior while the lower ones are RMS buildings using tuff as structural material. Thus, the color of tuff constitutes the basic tone in all cities and towns in Armenia.

Photograph 1.6.2 shows tuff embedded as exterior finishing material in precast wall panels. The tuff in Photograph 1.6.3 is used as aggregate for concrete. In Photograph 1.6.3, the entire concrete has a purplish color and aggregate is heavily cracked, indicating that the aggregate does not have a sufficient strength.

These observations suggest that the use of a large amount of weak stone material for main structural members has worked to increase the weight of the buildings and bolster up their undesirable behavior.

b) Wide Use of Precast (Void) Slab as Flooring Material

Precast void slabs are used as flooring system in almost all apartment houses, which account for by far the greatest part of the buildings that suffered heavy damage by the latest earthquake, as seen from Fig. 1.6.1 and Photograph 1.6.4. This material seems to be used not only in RPC buildings as shown in Photograph 1.6.4, but also in most of the RMS-built apartment houses. These void slabs are unidirectional simple support plates about 25cm thick. Reinforcing bars for increasing the bending strength are used in many cases and as a result, the typical span is relatively long, about 6m.

For factory roofing, ribbed unidirectional precast slabs are used as seen Fig. 1.6.1 and Photograph 1.6.5.

Thus, these buildings have a relatively long span and comprise floor panels insufficient in earthquake resistance due to inadequate bonding with beams or walls, leading to a decreased integrity of these buildings. The wall-to-wall and the pillar-to-pillar distance is long in RMS and RPC buildings, respectively. Furthermore, the weight of the floor is supported only on the frame provided along either the long edge or short edge of the floor. All these may have been a cause of the heavy damage of these buildings.

c) Structure of Gable Roof of Reinforced Stone Building

Most of the buildings having about three stories and part of the five-storied ones in the old sections of the cities have gable-like roofs which comprise a wooden roof truss provided directly on the roof plate or gable wall. This represents one of the standard building forms in the areas. In most cases, however, the top portion of the bargeboard of the gable is not reinforced at all. As a result, many of these buildings suffered the destruction of the gable wall. Compared to this, the portion containing the gable wall is stronger than other portions in many of the buildings that have a gable wall strengthened by reinforced concrete up to the top.

These findings suggest that the heavy damage to stone buildings may be partly attributed to the insufficient detailed standard requirements for the strengthening of the top portion of masonry walls.

d) Foundation for Direct Support

Methods of installing the foundation widely used in Armenia are illustrated in Fig.1.6.2.

The foundation method was observed for only a few of the damaged buildings. According to the Research Institute of Civil Engineering and Architecture, a large part of the buildings have cast-in-place continuous foundation, and piles are not used even in any nine-storied building. According to the institute, however, standard penetration test has not been performed in any case though boring exploration has generally been carried out.

During the field survey, a few buildings were found to have foundation that seemed to be made of precast concrete, as shown in Fig.1.6.2.

Specific cracks associated with such causes as differential settlement were not observed because most buildings had exterior walls decorated with stone material, etc. A doubt remains as to whether there were heavy buildings such as nine-storied ones in which foundation did not have sufficient supporting capability, depending on the quality of ground.

(2) Reinforced Masonry Buildings (five-storied ones in particular)

This type of buildings consist of structural walls of masonry strengthened with reinforced concrete, precast concrete (PC) floors and roof plates. Many of them were found to have wooden trusses (PC trusses in some cases) provided directly on masonry walls.

Masonry structural walls can be divided into two groups depending on the shape of stone used as shown in Fig.1.6.3 and Photograph 1.6.6, respectively, or they can be divided into three groups depending on the tensile strength of the mortar used for joint or filling ($0.6-1.2kg/cm^2$, $1.2-1.8kg/cm^2$, and $1.8kg/cm^2$ or higher).

In addition, the aseismic design code (SN&P II-A 12-69, 1969 ed.) contains requirements for the size, shape, material and reinforcement of the walls of reinforced masonry buildings. Major requirements are outlined below. (Figures in brackets show the number of each provision.)

i. The same wall material should be used in the same residential zone (each story is of a different material in the building in Photograph 1.6.6). It is recommended that windows and piers have the same width. [3-29]

ii. The design of a masonry building to be built in an area with an intensity of 7, 8 or 9, should takes into account the effect of vertical force of 15 percent (area with intensity of 7 or 8) or 30 percent (area with intensity of 9) in addition to the effect of horizontal force. [3-30]

iii. Masonry buildings are divided into three classes depending on the tensile strength of the mortar used for joint or filling, and areas where buildings of each class can be built are designated. [3-31]

iv. The height of each story of the buildings in an area with an intensity of 7, 8 or 9 should not be more than 6m, 5m and 4m, respectively. The ratio of the height of a story to the thickness of the wall should not exceed 12. [3-36]

v. The wall-to-wall distance should not be more than the values specified for each construction site and wall class (for instance, the distance should not be more than 12m for a Class-1 building in an area with an intensity of 9). [3-37]

vi. Calculation procedures are shown for the dimensions of wall members, etc. The required minimum width of piers (1.16m for a Class-1 building in an area with an intensity of 9), upper limits to the width of openings (2.5m) and lower limits to the ratio of the width of piers to that of openings should not be less than the specified values. [3-38]

vii. Aseismic beams of cast-in-place RC should be provided at the top to floor levels of all walls. The beams at the highest story should be directly connected to the wall using reinforcing bars. [3-39]

viii. The beams should be as thick as or thicker than the wall and 15cm or more in height. The concrete used should be 150kg/cm$^2$ or more in strength and the reinforcing bars should be 4-10mm or more in diameter (4-12mm in area with intensity of 9). [3-40]

ix. The joints of walls should be strengthened by means of 2m of mesh provided at every 70cm heights (50cm in area with intensity of 9). [3-41]

x. To achieve an increased earthquake resistance, masonry walls should be integrated by providing cast-in-place RC portions within them. The vertical members of a masonry wall should be connected to the beams. [3-43]

The requirements for beams are clearly specified quantitatively. Compared to this, the requirements for vertical strengthening, which is as important a strengthening element as beams, cannot be clearly specified, as seen from the provision under x.

Photograph 1.6.7 shows the degree of vertical strengthening.

(3) Nine-Storied Building of Framed Precast Reinforced Concrete (RPC) Construction

Fig.1.6.4 and Photograph 1.6.8 roughly show the construction method for these buildings, and Figs.1.6.5 and 1.6.6 illustrate the plan and elevation of a typical building constructed with this method. Basically, a building of this type is constructed by in-site welding of factory-produced elements including pillars, beams, floor panels and floor panels. Photograph 1.6.8 shows a frame structure consisting of beams provided in two directions while Photograph 1.6.9 shows a frame with unidirectional beams. In the latter case, wall panels may be provided perpendicular to the beams (example in Leninakan described later), or may not be provided at all (example in Spitak given in Photograph 1.6.9). Outer walls of precast concrete are attached to the outer face of the frame in many buildings (Photograph 1.6.9).

Of the major joints in these buildings, pillar bars are connected to each other by butt welding using special guideplate in some cases (see Photograph 1.6.10). Beam bars is welded by using support bars or doubling plates.

For this RPC method, the above-mentioned aseismic design code show no specific requirements except for those concerning floor structures.

Nine-storied apartment houses account for by far the greatest part of the buildings constructed by this method.

(4) Nine-Storied Building of Wall-Type Precast Reinforced Concrete (WPC) Construction

This type of buildings comprise large precast floor and wall plates as shown in Fig. 1.6.7 and Photograph 1.6.15. Their major feature is that they have no members corresponding to pillars or beams. Concerning the connection of plates, the welded edges of vertical wall plates and that of horizontal ones are called vertical and horizontal joints, respectively, while the welded edges of floors are referred to as floor-floor joints. In general, a joint between a floor and wall is included in the horizontal joints. These joints can be divided into two types depending on their structure, namely, dry joint and wet joint. A dry joint is formed when steel members anchored within plates are connected by welding while a wet joint is formed when plates are connected by concrete provided between them. In Armenian buildings constructed by the WPC method, vertical joints are of wet type (see Photographs 1.6.16 and 1.6.17) while horizontal joints (see Photograph 1.6.18) and floor joints are of dry type. To make a vertical joint, horizontal reinforcing bars are welded together in the case of linear wall arrangement (Photograph 1.6.16 ) while reinforcing loops are intersected when connecting walls perpendicular to each other

(Photograph 1.6.17). Most of the techniques used for the WPC method are the same as those in Japan. Major differences are that the projections (called shear key) provided at the edge of plates used in these countries have different shapes and that connecting bars for horizontal joint used in Japan are larger in cross section.

Major requirements for this type of WPC method specified in the Aseismic Design Code (1969 rev.) are outlined below (figures in brackets showing the number of each provision).

i. For large-panel buildings, structural design concerning the vertical force and earthquake force should be performed taking the following matters into account to ensure that the wall plates and floor plates coordinate with each other to allow the building to behave as an integral three-dimensional structure.
   ° Each wall plate or floor plate should be as large as possible. Use room-size ones where possible.
   ° Wall plates and floor plates should be combined by using widened reinforced joints filled with low-shrinkage concrete, or other joints equivalent to them.
   ° Horizontal forces should be beared by external walls. Walls should be arranged appropriately in terms of balanced rigidity.
   ° The wall-to-wall distance should not be more than 6.5m.

ii. The wall plates should comprise double reinforcing bars. Keys cotter for concrete filling should be provided at the edge faces of each floor plate. [3-26]

iii. Wall plates and floor plates should be connected by welding the anchoring bars. It is recommended that the joint portions are filled with concrete after carrying out rust prevention treatment. [3-27]

iv. Calculation procedures for the cross section of jointing bars are shown. It should not be less than $1cm^2$ per meter of the linear portion of a joint. [3-28]

In Leninakan, we identified 6 nine-storied apartment houses and a five-storied one that had been constructed by this method. Their number is much smaller than that of apartment houses constructed by the RPC method.

(5) Other Method

Other types of buildings found in the area are included the following.

i. Unreinforced stone buildings, most of which are one-storied houses.

ii. One- to three-storied factory buildings constructed in the RPC method (braces are used in some of them) (see Fig.1.6.8 and Photograph 1.6.5).

iii. High-rise apartment houses constructed by the combination of the cast-in-place RC and RPC methods (see Photograph 1.6.19).

Buildings constructed by the following methods exist in Yerevan's suburbs which are not included in the latest earthquake damaged area. The information given under v. was obtained from architectural institutes in the areas.

iv. Steel-frame, one-storied residences and factory buildings (see Fig.1.6.9).

v. High-and medium-rise apartment houses of small open PCa frame construction (see Fig. 1.6.10).

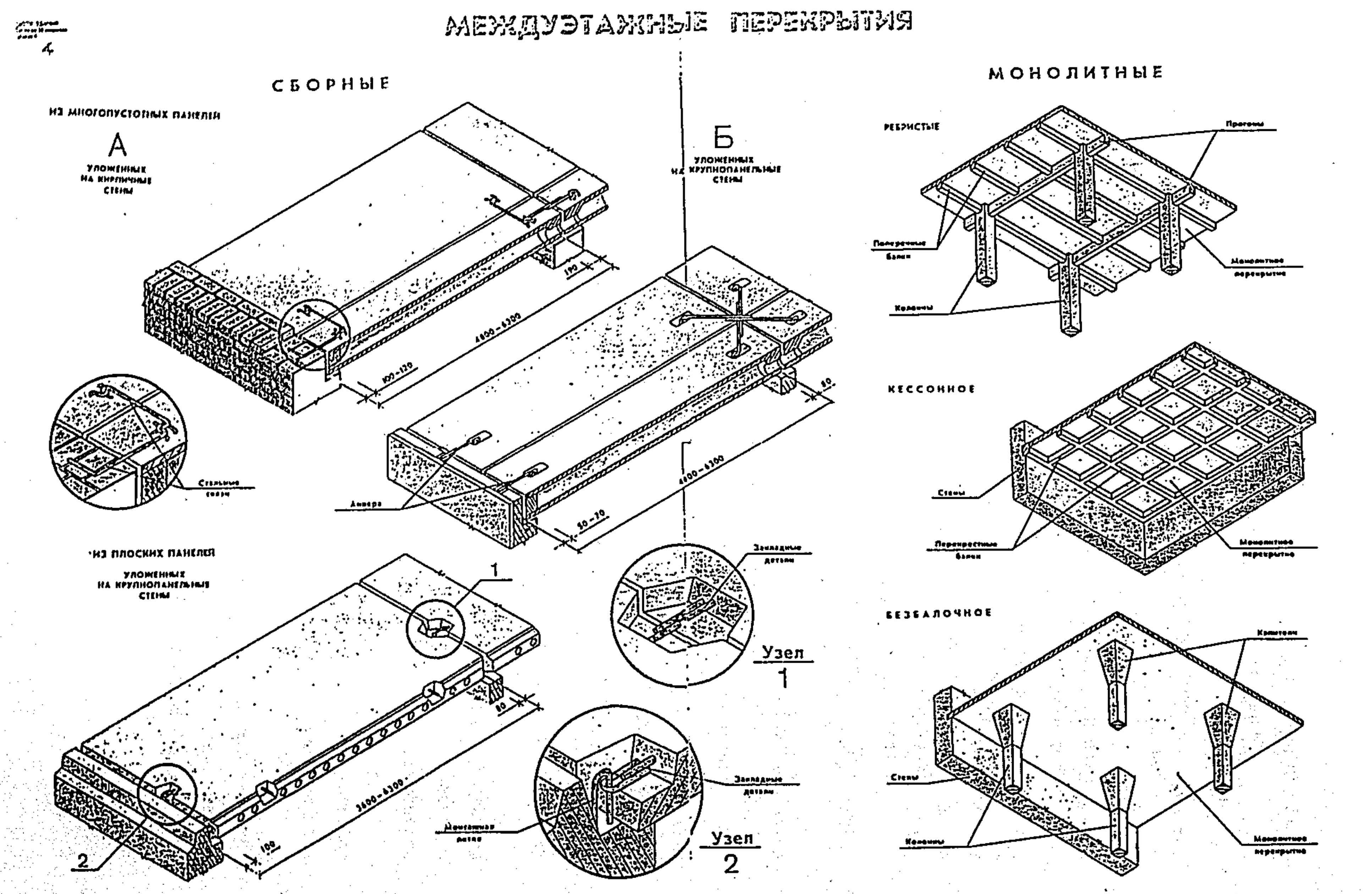



Fig. 1.6.1 Floor System

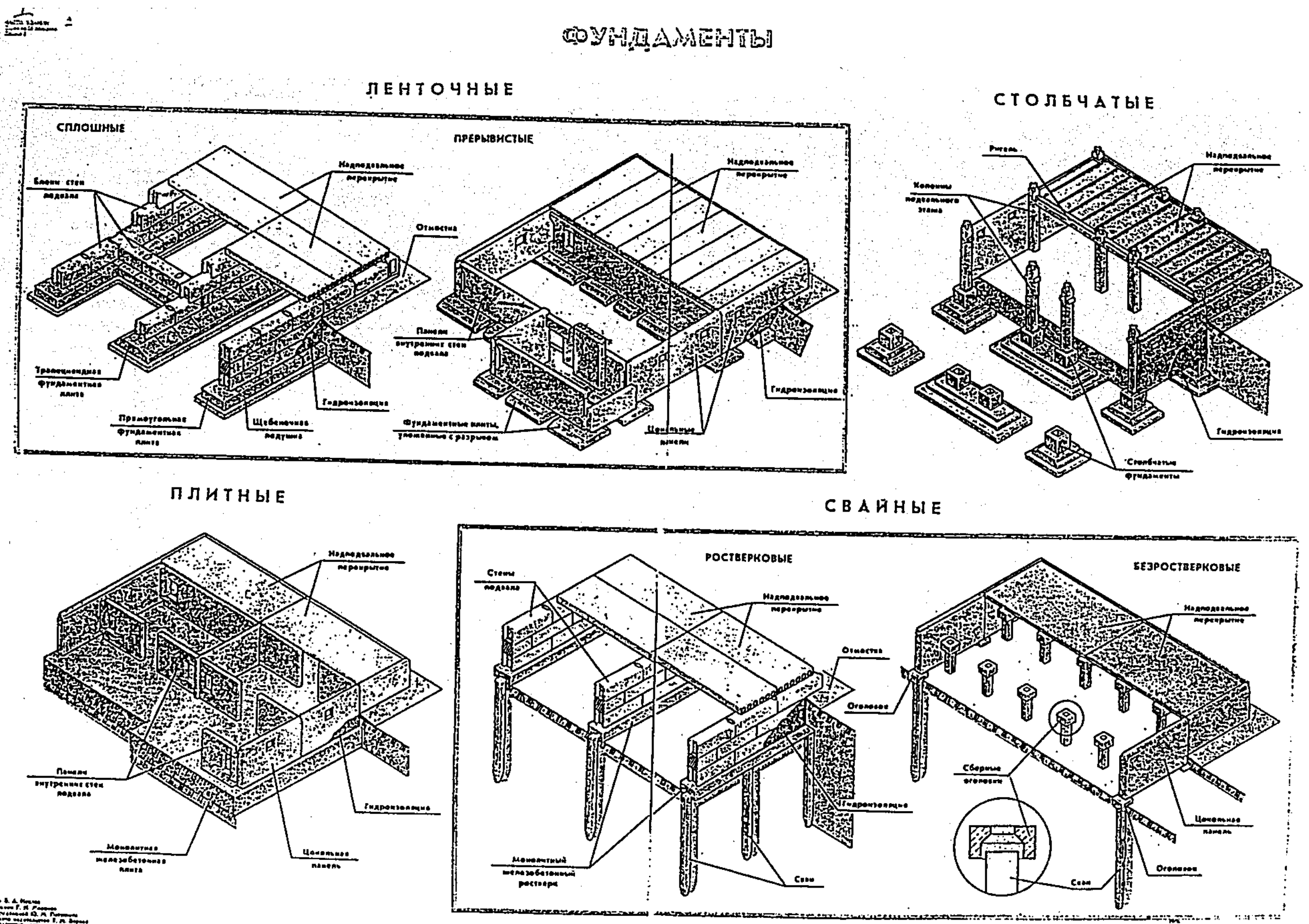


Fig. 1.6.2 Foundation System

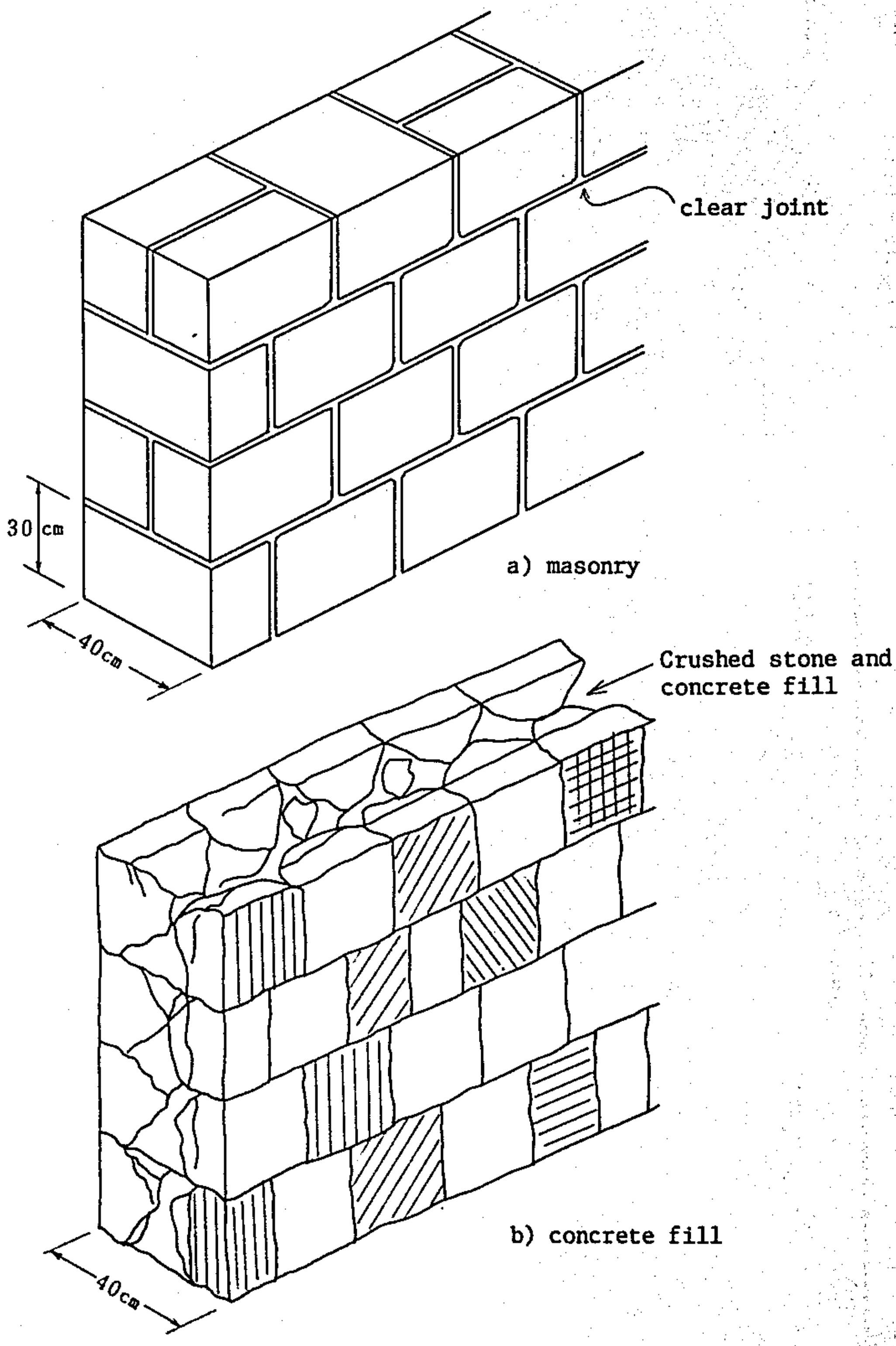



Fig. 1.6.3 Two Types of Stone Wall

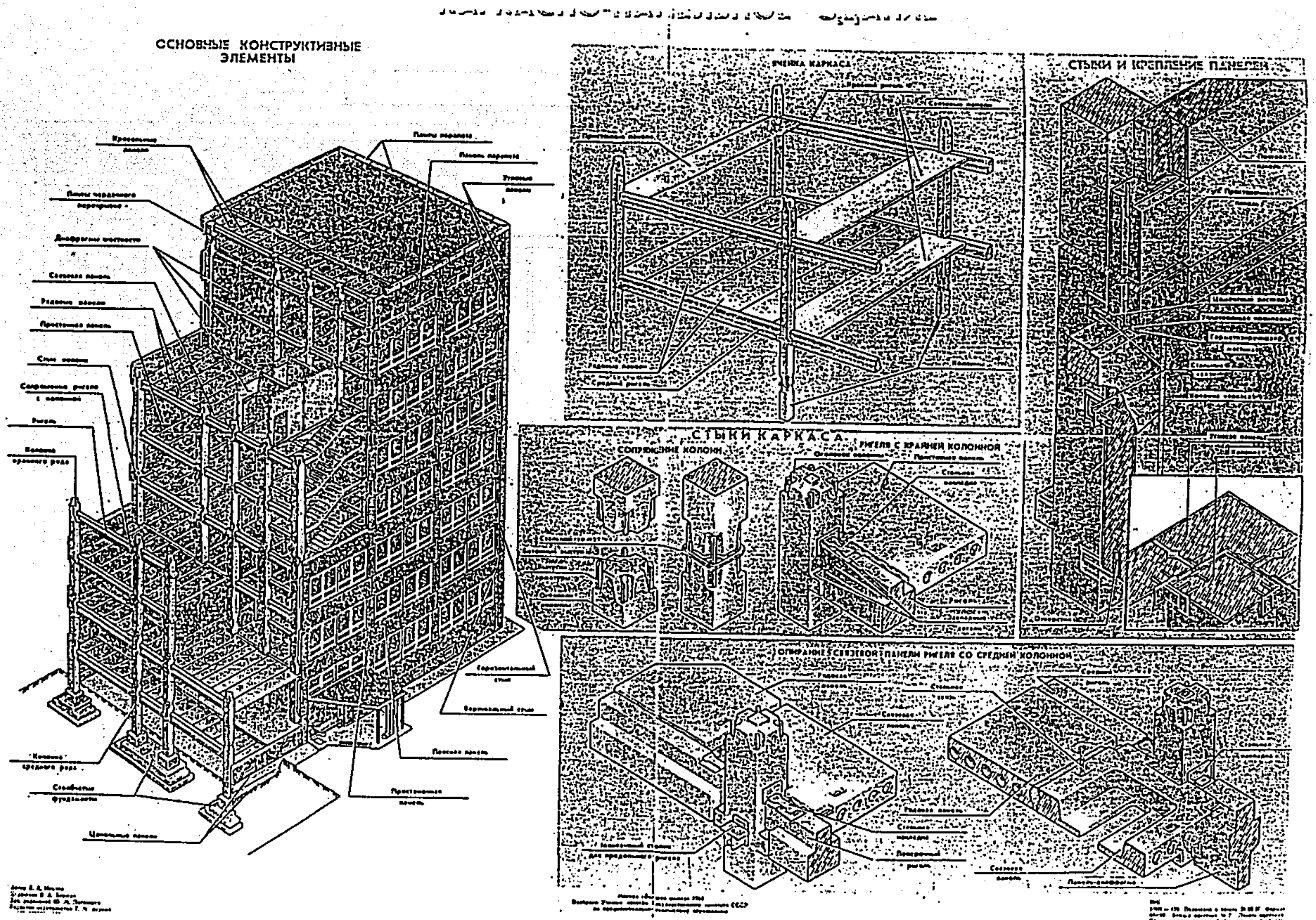



Fig. 1.6.4 Rigid-Frame Precast Reinforced Concrete (RPC) Construction (unidirectional panel)

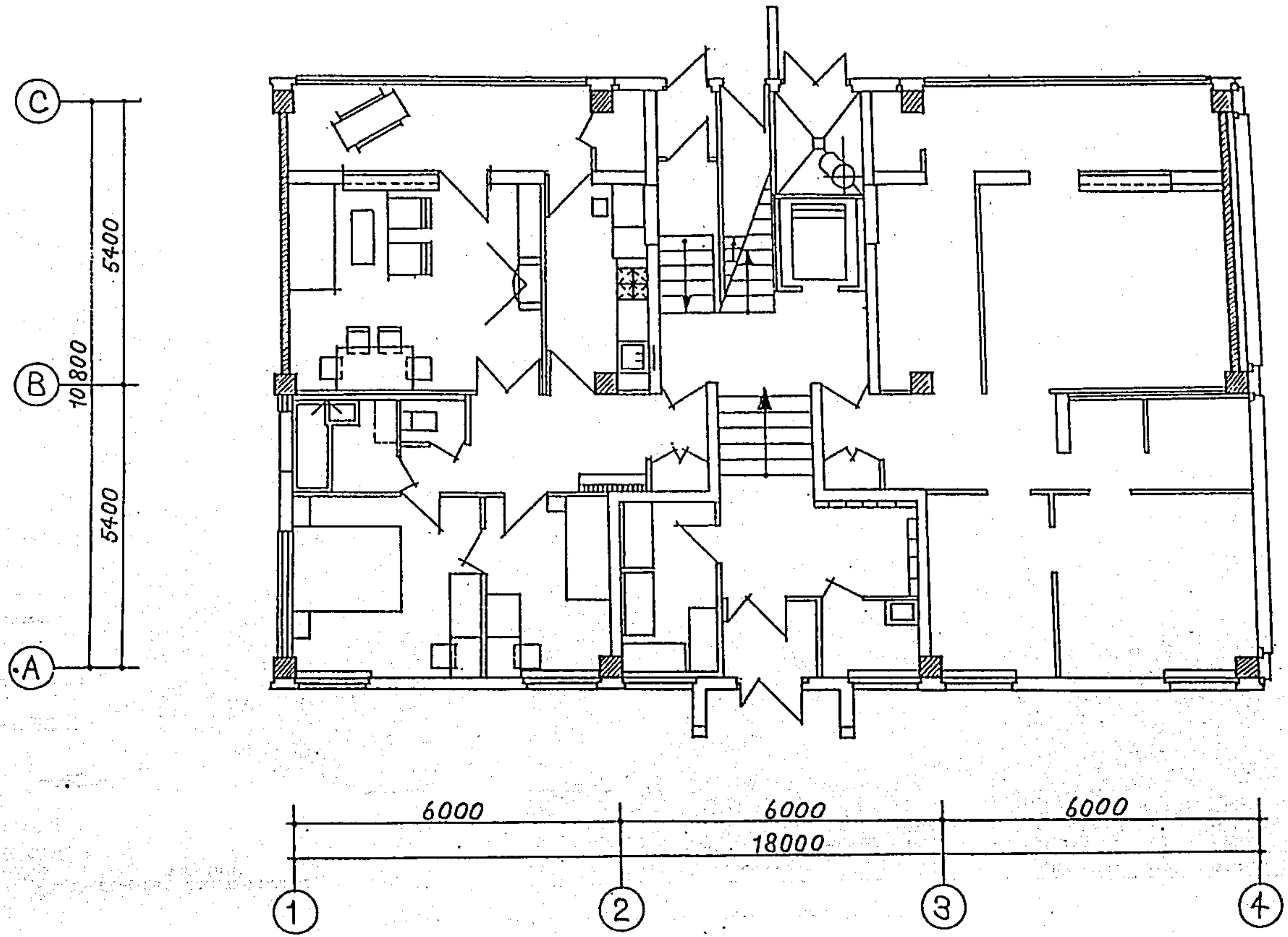


Fig. 1.6.5 Plane of Apartment House of RPC Construction

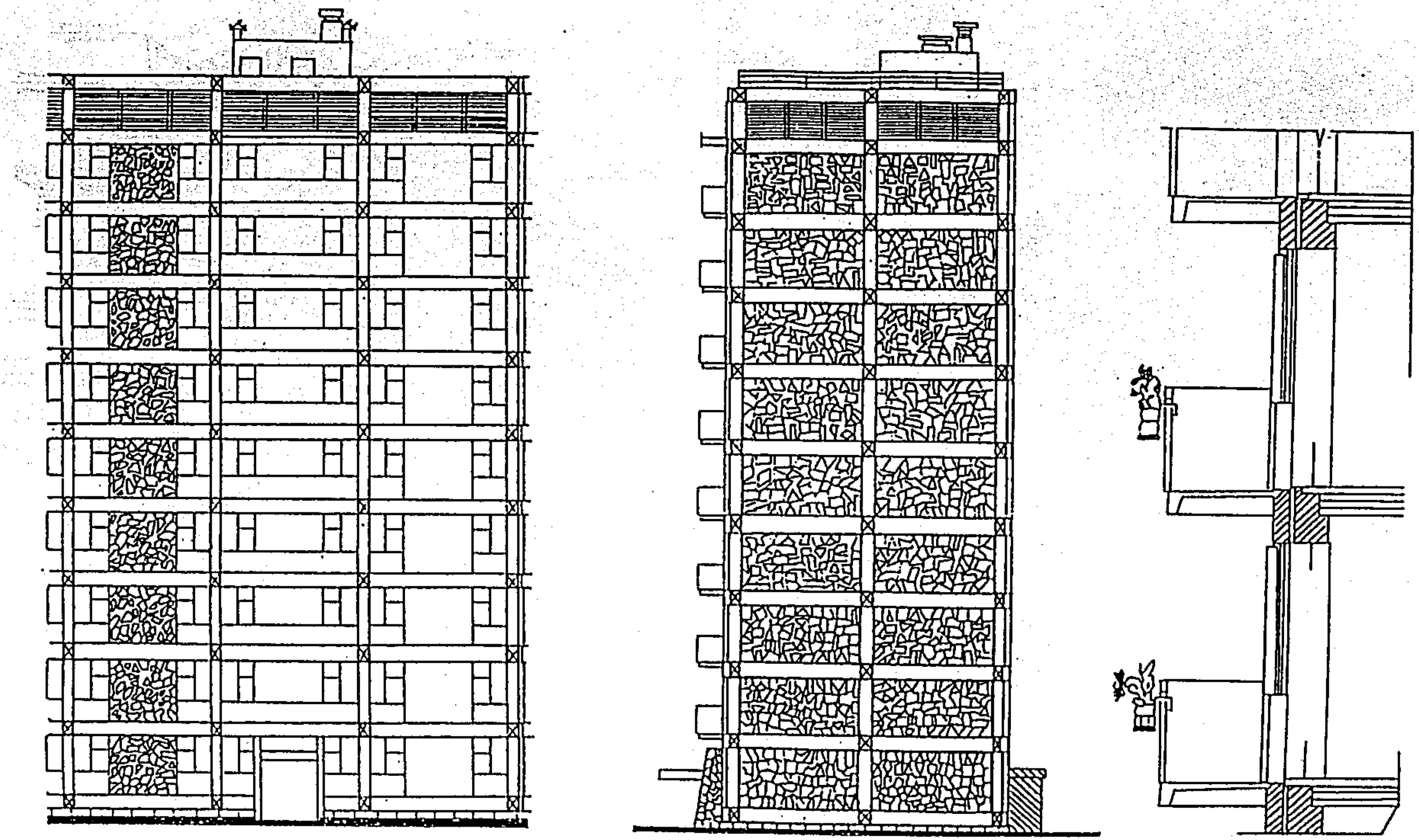


Fig. 1.6.6 Elevation and Cross Section of Apartment House of RPC Construction

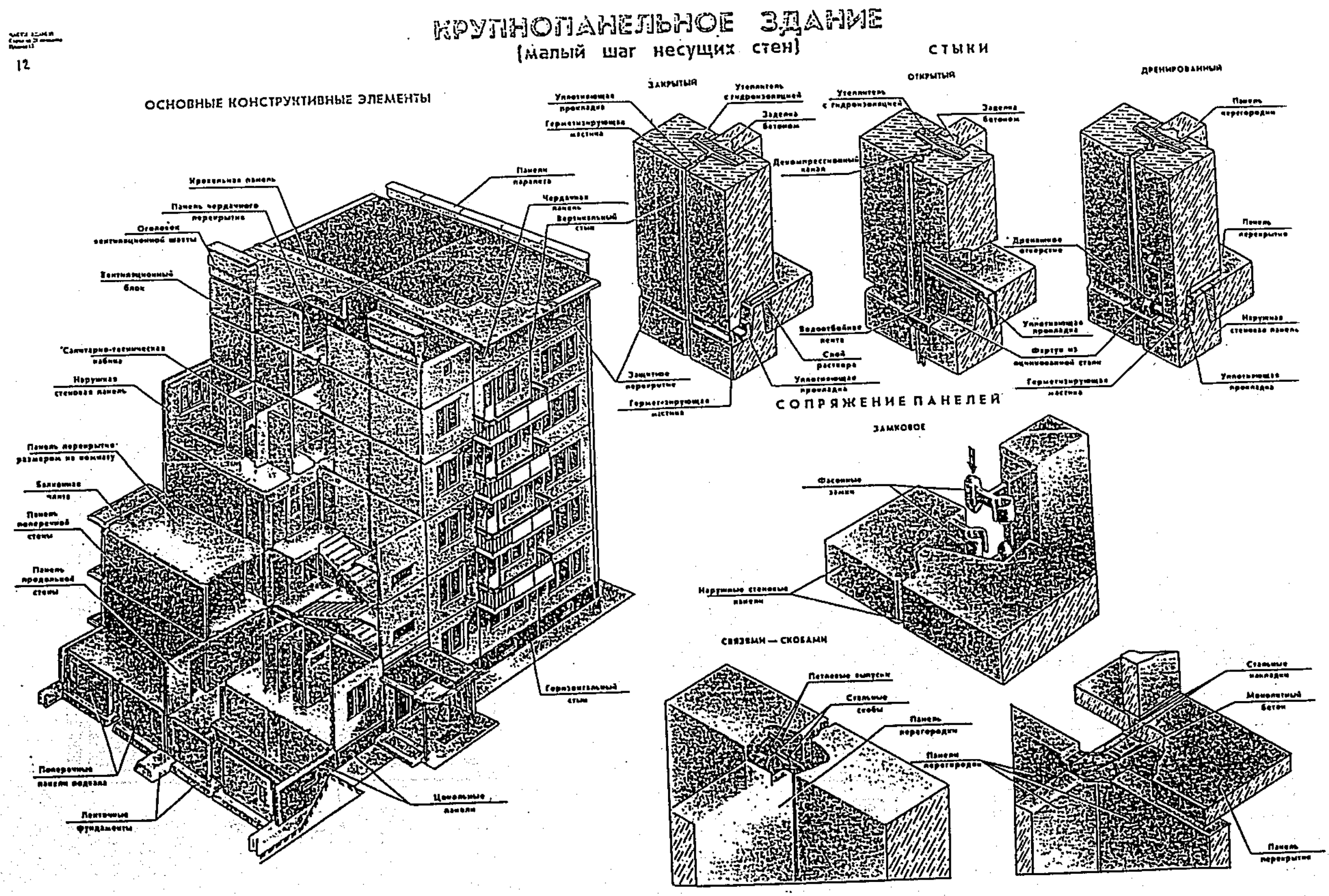



Fig. 1.6.7 Large PC Plate (WPC) Construction (I)

ЖЕЛЕЗОБЕТОННЫЕ КАРКАСЫ ОДНОЭТАЖНЫХ ПРОМЫШЛЕННЫХ ЗДАНИЙ

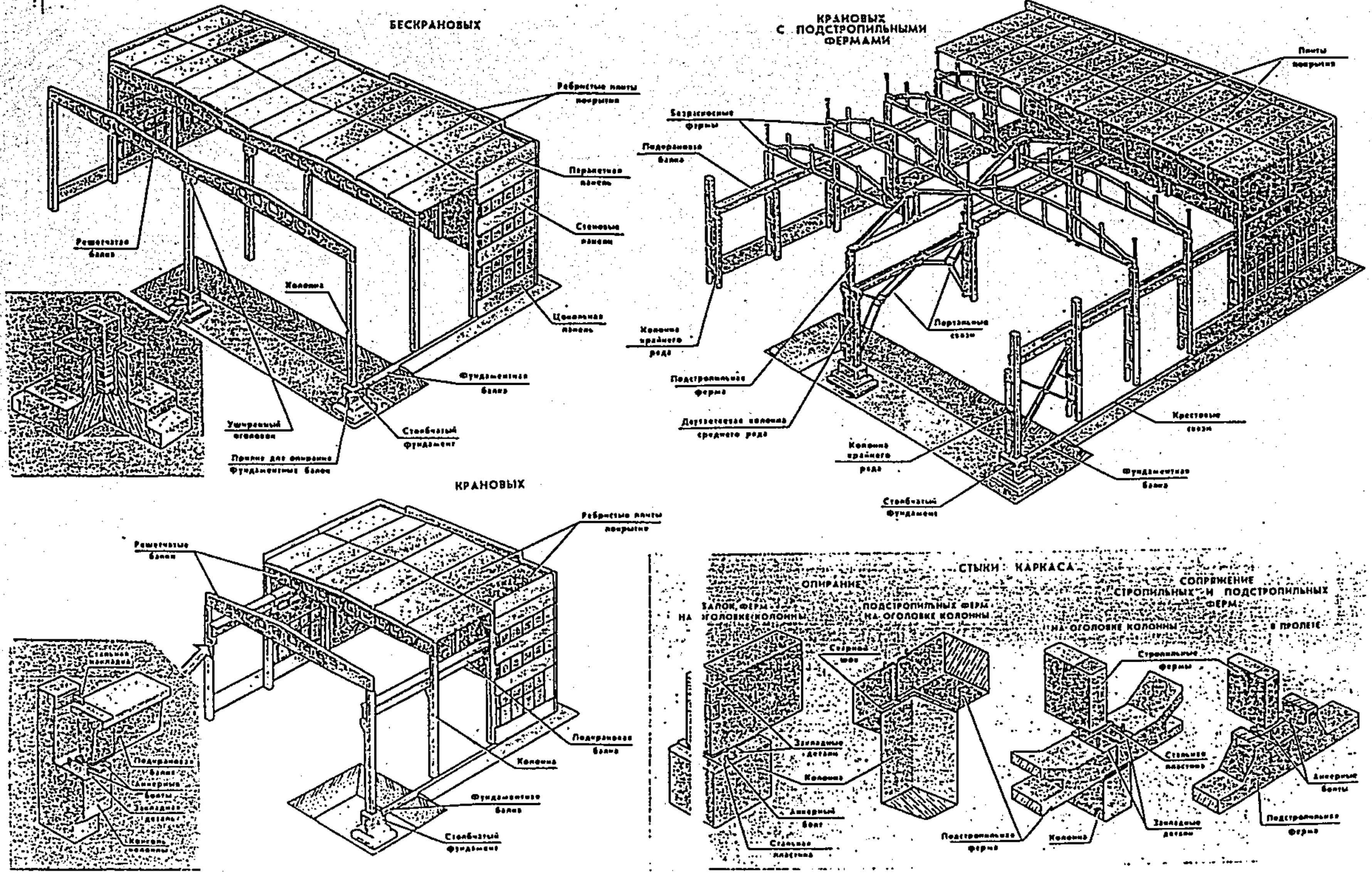
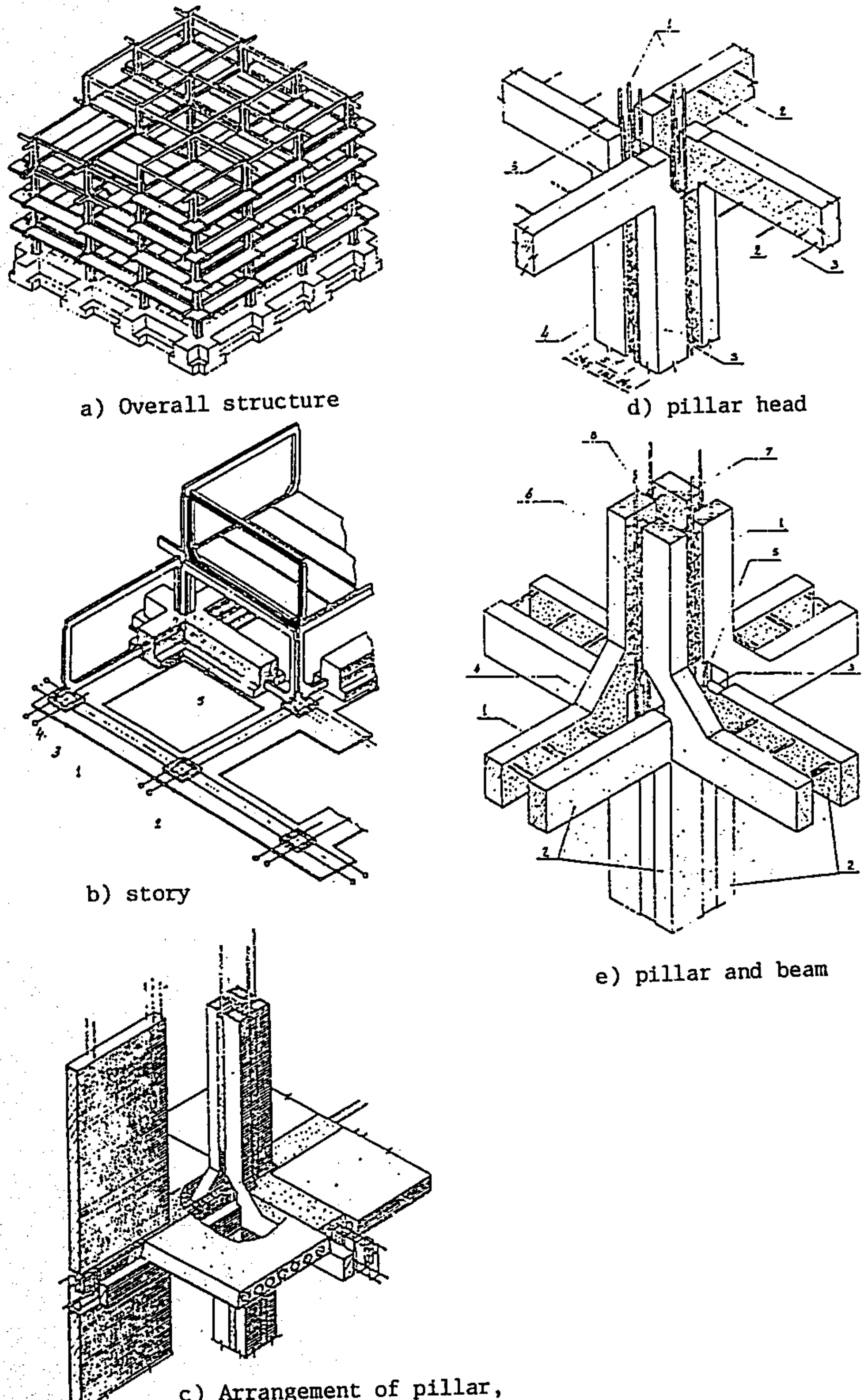



Fig. 1.6.8 One-Factory Building of Light-Weight Steel-Frame Construction


Fig. 1.6.9 Steel-Frame Factory Building



a) Overall structure

b) story

c) Arrangement of pillar, beam, floor and wall

d) pillar head

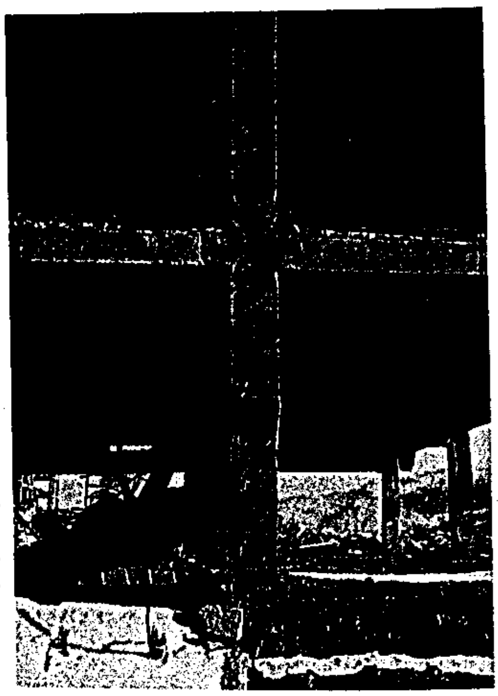
e) pillar and beam

**Fig. 1.6.10 Building of Framed Precast Reinforced Concrete Construction Using Small PC Frame Members**

Photograph 1.6.1
Eleven-storied apartment house of framed precast reinforced concrete (RPC) construction and five-storied apartment houses of reinforced masonry (RMS) construction standing along the main street of capital Yerevan. (In most of the buildings in Armenia, purplish tuff is used as structural and exterior finishing material over the external faces.)

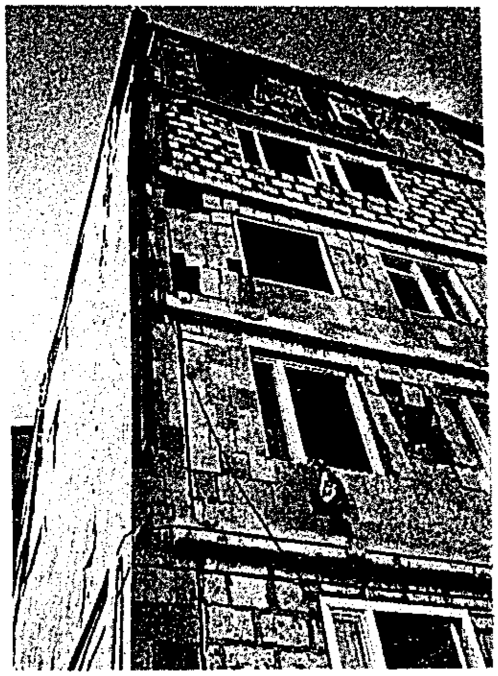
Photograph 1.6.2
Tuff embedded as exterior finishing material for large precast plate (at a construction site in Leninakan)

Photograph 1.6.3
Tuff used as aggregate for concrete. The entire concrete has a strong purplish color. (Reinforced concrete in destroyed five-storied RMS apartment house in Spitak



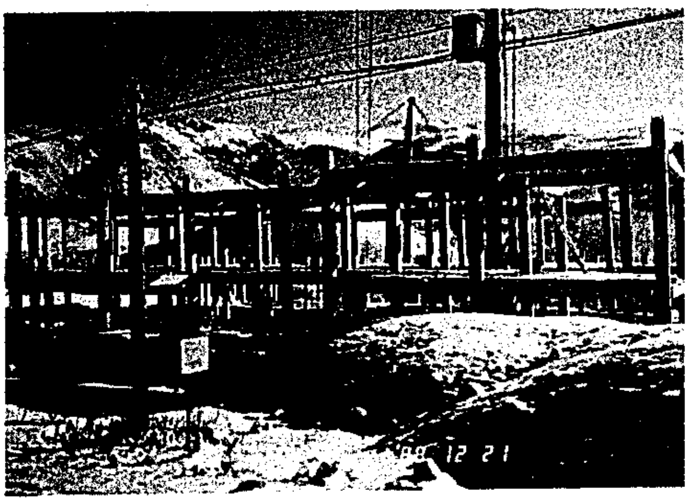
Photograph 1.6.4
Precast void slab used as floor material in RPC building heavily destroyed during construction. Each end is supported on the beam and cast-in-place concrete is laid on the support portion. The span is about 5m in most cases. (Spitak)

Photograph 1.6.5
Ribbed precast panels used for roofing of laboratory building of precast reinforced concrete. Braces were used in some portions of this building, but its external masonry walls were destroyed almost completely. (Leninakan)

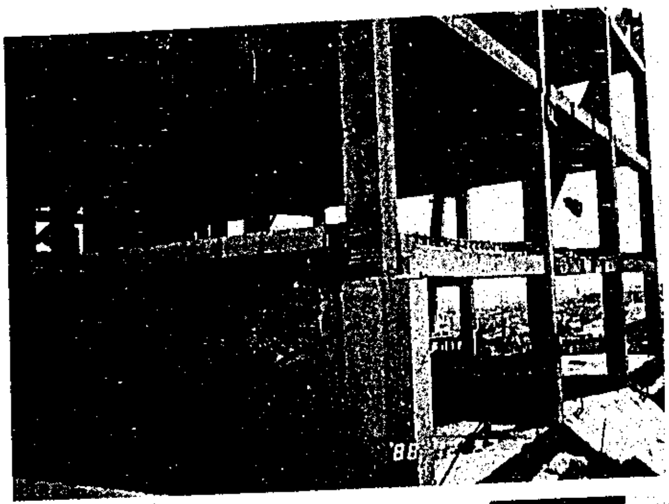


Photograph 1.6.6
Gable of five-storied RMS apartment. Two apparently different laying methods are used for alternate stories. (Spitak)

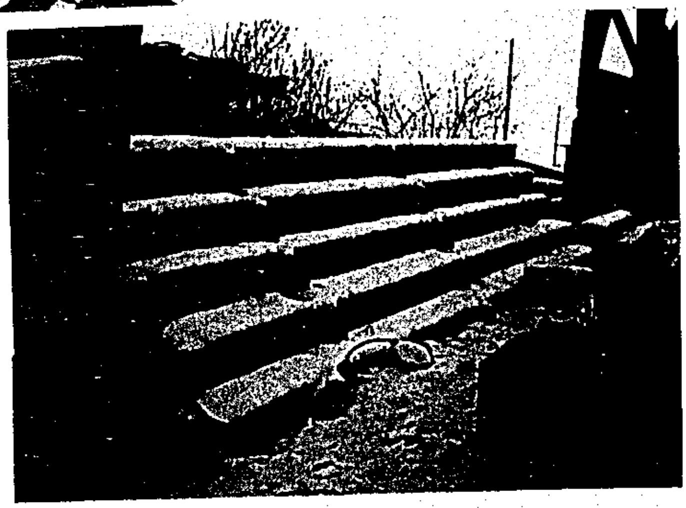
Photograph 1.6.7
RMS apartment house under construction. Baskets of reinforcing bars used in reinforced concrete is seen between masonry walls. The photograph shows the typical arrangement of reinforcement in RMS buildings about five-story high. (Spitak)



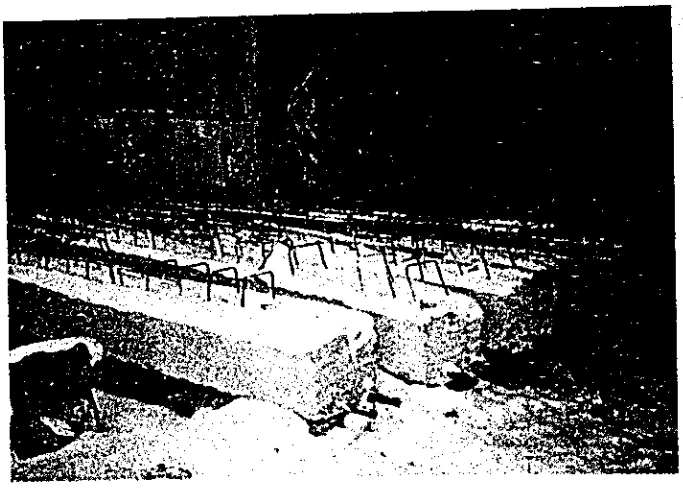
Photograph 1.6.8
Framework of rigid-frame precast reinforced concrete (RPC) building damaged during construction. This example has beams in two directions, though buildings with unidirectional beams account for a much larger part. (Suburb of Spitak)



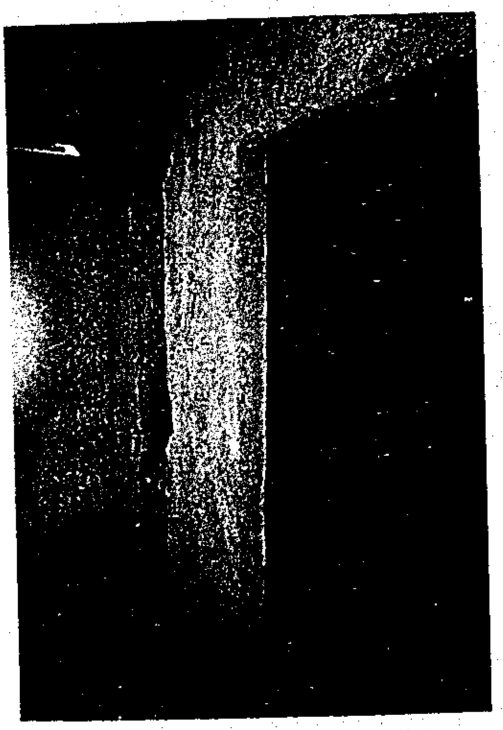
Photograph 1.6.9
Framework of RPC building under construction, as in the previous photograph. Unlike the previous example, beams are provided only in one direction. Precast plates and masonry walls are provided in the direction perpendicular to the beams. (Spitak)



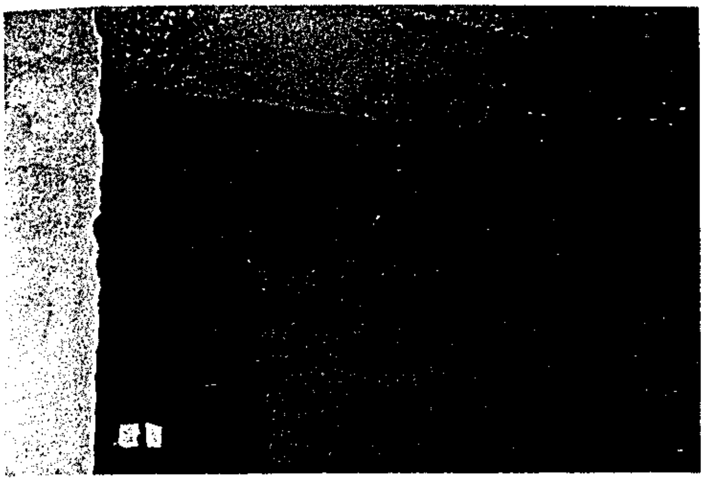
Photograph 1.6.10
Precast pillar members left on construction site of RPC building destroyed during construction. One member covers two stories. Exposed main bars are provided at each end for pillar to pillar joint, and a pair of bars and steel angles are provided at two points between the ends for pillar-beam joint. (Leninakan)



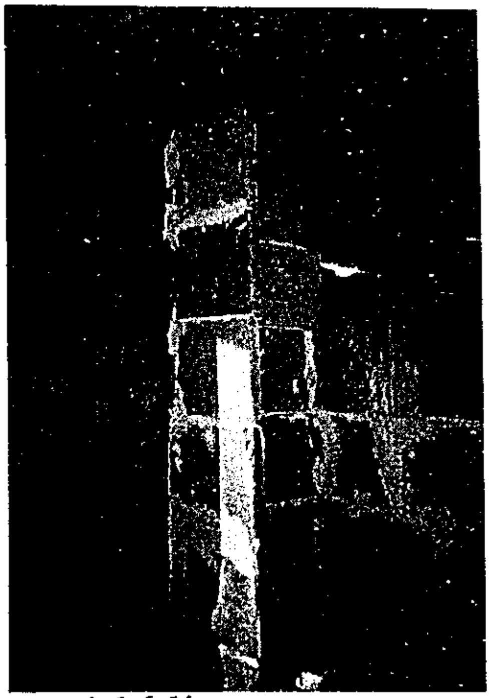
Photograph 1.6.11
Precast beam members left on construction site of RPC building, as in the previous photograph. A member covers one span. Top of the beam reinforcing bars is exposed at a portion where cast-in-place concrete is to be laid for connection with a floor panel. After assembling beam members, two main bars are provided at the two solid steel angle portion of the reinforcing bars, and then the main bars are welded, followed by placing of concrete. (Leninakan)



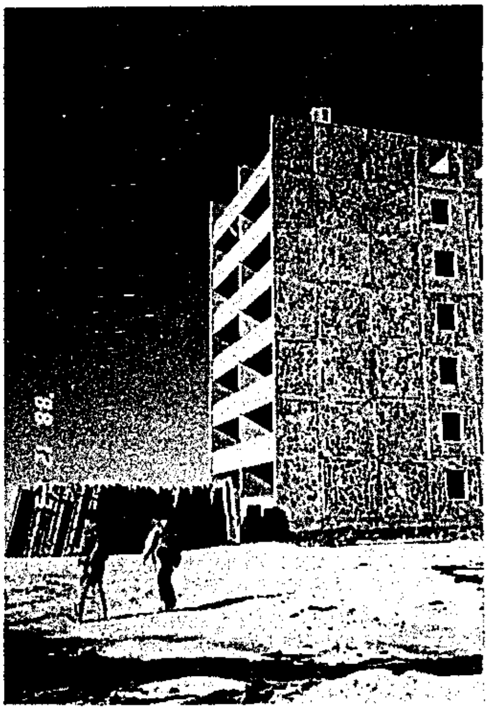
Photograph 1.6.12
Joint between pillar and precast opening wall (probably for structural use) in RPC building under construction, as in the previous photograph. (Suburb of Leninakan)



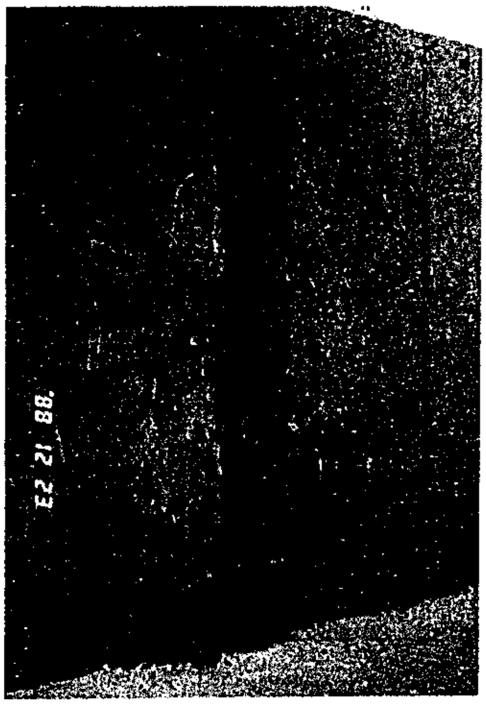
Photograph 1.6.13
Masonry partition wall in RPC building under construction. About 20cm thick and not reinforced at all. (Leninakan)



Photograph 1.6.14
Partition wall of small precast panel in RPC building under construction, as in the previous photograph. Not reinforced at all, either.

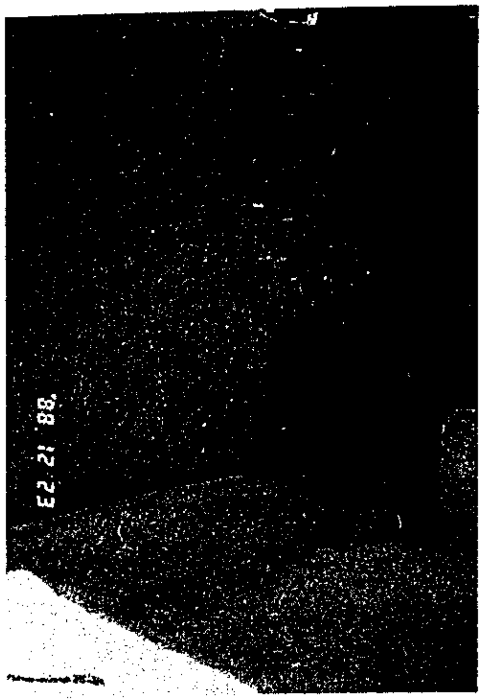


Photograph 1.6.15
Construction site of nine-storied apartment house comprising large precast panels of reinforced concrete (WPC method). Seven stories had been completed at the time of the earthquake. The damage was limited to the fall and collapse of the PC floor panels of the eighth floor. Precast exterior walls incorporating tuff as external finish material are seen beside the building. (Suburb of Leninakan)



Photograph 1.6.16
Vertical joint between wall panels in WPC building, as in the previous photograph. Horizontal reinforcements of the wall are welded together. This type of buildings have many features that are common to similar buildings in Japan, excluding the shape of the end portion of the concrete panels.

Photograph 1.6.17
Vertical joint of wall panels arranged in T-shape in WPC building. Linearly arranged panels are connected by welding their horizontal bars while the horizontal bars in perpendicular panels have exposed loops, which are filled with concrete to form vertical joints after placing vertical bars at the site. This technique is used in many of the similar buildings in Japan.



Photograph 1.6.18
Details of horizontal joint between upper and lower wall panels in WPC building. Reinforcing bars in the lower panel are welded to a steel panel embedded at the bottom of the upper panel. A similar technique is also used in Japan, though reinforcing bars with larger diameters are employed.

Photograph 1.6.19
The only high-rise building (sixteen storied) in Leninakan. Precast material is used for pillars and floor slabs in the peripheral portions while cast-in-place concrete is used for anti-earthquake walls installed in core-like arrangement in the middle portion of the building. The core anti-earthquake walls suffered heavy damage, causing most of the horizontal bars to bend. The damage was so severe that it seemed impossible to assure the safety of the building under normal load. Later, the building was pulled down completely.

## 2. Outline of Armenia Spitak Earthquake

### 2.1 Seismic Phenomenon

(1) Hypocentral Element and Earthquake Mechanism

In the Soviet Union, seismological survey has been conducted by the Academy of Science of USSR in cooperation with the Academy of Science of each republic. There are 14 seismological stations in the Armenian SSR. An analysis of the records was made promptly, and the following results were obtained.

Time of occurrence: 11:41 December 7, 1988
Epicenter: 40°50'N, 44°15'E
Depth: 10km
Earthquake magnitude: 7.0

These are the most reliable results available at present since the epicentral elements can be determined most accurately from the data obtained at seismological stations near the epicenter. The epicenter is located about 6km to the south of Spitak, where there was heavy damage. Data obtained at the Matsushiro Seismological Station of Meteorological Agency of Japan show that the amplitudes of the surface waves were much larger than those of the body waves, including P and S, suggesting that the depth of the hypercenter was small.

According to an analysis by the Soviet Union, the earthquake occurred through a reversed fault mechanism: an earthquake fault was formed about 60km to the west of the hypercenter which acted as the origin of the crustal destruction, and the land on the north side of the fault moved upward in the nearly vertical direction (about 80° in angle) above the level on the south side while horizontal side occurred slightly to the west. The displacement due to sliding has not been determined yet.

These analytical results are consistent well with the fact that much heavier damage occurred in Leninakan, which is located as much as 35km to the west of the epicenter than in Kirovakan, located 17km to the east, that both Spitak and Leninakan were on the moving side of the fault line, where the intensity is generally large, and that the damping of the seismic indensity was larger on the south than on the north of the fault line.

(2) Foreshocks

An earthquake was recorded at about 16:00 on December 6 at a seismological station in Stepanavan to the north of Spitak, and a larger earthquake was recorded at 19:30 on the same day at several stations. The hypocenter of the latter earthquake was very close to that of the main shock. Needless to say, it was impossible for the present-day seismology to judge that they were the foreshocks prior to a strong earthquake with a magnitude of 7.

(3) Aftershocks

In general, an earthquake with magnitude of 7 is accompanied by many aftershocks according to data on the past earthquakes in Japan, where seismological stations have been installed densely and the largest volume of data are available in the world. The frequency of aftershocks decreases nearly monotonously with times. The magnitude of the strongest aftershocks is generally smaller than the main shock by one rank, and the aftershocks occur on the opposite side of the fault. Most of aftershocks take place within two to three days of the main shock.

In the present case in Spitak, an aftershock with a magnitude of 5.8 occurred four minutes later than the main shock. Their hypocenters were located at almost the same position. All aftershocks following this were below 5 in magnitude. The aftershock region was a near-circular ellipse with a major axis length of 60km. This size is normal for the aftershocks accompanying an earthquake with a magnitude of 7.

However, an attention should be given to the fact that the damping in the frequency of the afterhsocks was not monotonous and that the number of the aftershocks was small on the side of the seismic fault opposite to the epicenter of the main shock. Close observation should be continued to investigate the aftershocks activities.

In Japan, an earthquake with a magnitude of about 6 occurred in Shimane Prefecture in March 1943, followed by another earthquake with an intensity of 7.4 in September, or six months later, whose hypocenter was located at nearly the same position as the former one. The comparison of the two earthquakes indicates that the one in March was a foreshock precedent to the one in September. There were many aftershocks, but their frequency did not decrease smoothly. In view of this example, an emphasis should be placed on the importance of the observation of the aftershocks following the Armenian earthquake.

(4) Premonitory Phenomenon

A study following the earthquake has identified some phenomena which appear to be premonitory signs.

1) Crustal Deformation

Gradual land displacement had been observed on roads near the epicenter during five years prior to the latest earthquake.

2) Groundwater

The groundwater level began to drop and water became muddy two days before the earthquake.

3) Inclination and Extension

An inclinometer and extensometer recorded abnormal measurements of about $1.3 \times 10^{-6}$ 2-3 days prior to the earthquake. The measuring point was near Leninakan, and the extensometer detected "shrinking" movements. A shrinkage of $5 \times 10^{-7}$ or more is regarded as abnormal in the Tokai District special measurement scheme in Japan.

4) Atmospheric Phenomenon

It was abnormally warm on the evening before the earthquake. It was also reported that a mist lay thick in the morning of the day of the earthquake, and it disappeared immediately before it occurred.

All the above information was obtained at the Yerevan's Institute of Seismology from Mr. Nersesov and officials at the Academy of Science of USSR. We had a chance to check the abnormal measurements recorded on the measuring instruments.

However, we only had a short meeting and the area was still in turmoil. Further efforts are required for the quantitative investigation. However, it is clear at present that there were some premonitory phenomena including those recorded on the measuring devices.

## 2.2 Earthquake Motion and Intensity

(1) Earthquake Motion

One of the major difficulties in studying the damage to buildings by the latest earthquake is that no strong motion seismograms were recorded in the disaster-stricken area. (It was found later that measurements had been taken in Gukasyan, which is located to the north of Leninakan.) The only record we obtained was of the horizontal I component, one of the three strong motion seismogtraphic components, observed at the Yerevan Institute of Architecture (Fig. 2.2.1). Even its azimuth has not been identified. Yerevan is located about 70km to the south of the epicenter and there were almost no damage to the buildings there. Thus, it is difficult to estimate the earthquake motion from these measurements, but rough features of the earthquake can be drawn. The maximum acceleration was 62 gals. The magnitude of the surface wave (ms) was converted into the local magnitude ($M_L$) and various distance-damping equations were applied. Results showed that existing empirical equations were met satisfactorily (Fig. 2.2.2). An analysis of frequency components indicated that the waves with extremely short periods continued for a long time. Waves around 10Hz prevailed during the first nearly 10 seconds. Waves around 5Hz prevailed during the next 10 seconds, followed by the waves with slightly longer periods. According to some seismologists, the profiles of the P-wave components observed at the distant places indicate that the latest earthquake consisted of three events that took place successively at intervals of about 9 seconds. It does seem possible to interprete the above results to support this assumption. In fact, vibrations with amplitudes larger than 2/3 that of the maximum continued for as long as about 25 seconds, indicating that the vibration duration was very long in spite of the fact that short-period components prevailed. Fig. 2.2.3 illustrates acceleration response spectra calculated from these data (according to the Public Works Research Institute, Ministry of Construction of Japan). Short-period components of 0.1-0.2 second were extremely prominent, but no significant peaks were identified. In view of the fact that the Yerevan Research Institute of Architecture is located on a thick tuff layer, the measurements may not reflect the effect of the amplification in the surface ground. The area comprises relatively firm ground consisting mainly of tuff and basalt and may not suffer from the modulation caused by the propagation of sesmic waves. It is considered, therefore, that the measured spectral characteristics reflect the mechanism of the earthquake almost directly.

Concerning areas other than Yerevan, we heard that strong motion seismograms had been taken at the nuclear power plant at Oktemberyan, located 40km to the west of Yerevan, though they were not available to us. Some say that the maximum acceleration was 18 gals while others insist that it was 80 gals. Strong motion seismograph had been installed in Leninakan, where the buildings suffered heavy damage. Again, some maintain that they could not be recovered because the building was destroyed while others (including Professor Steinberg at the Research Institute of Geophysics in Moscow) say that the seismographs installed at the top and bottom of the building were alive and being recovered. It is expected that they, if recovered, would have provided extremely valuable data helpful to analyze the process of the collapse of the buildings.

In addition to strong seismograms, seismoscopic measurements were taken at the Institute of Geology and Engineering Seismology at Leninakan and the Research Institute of Architecture at Yerevan. The seismoscopes recorded the traces of seven pendulums with different natural periods in the range of 0.1-1.0 second. The damping constant of the pendulums were reported to be 5-8 percent (logarithmic decrement 0.3-0.5). Dr. Khatchian said that the maximum acceleration calculated from the amplitude of the pendulum with a period of 0.25 second was closely consistent with the maximum amplitude (about 60 gals) recorded on the strong motion seismographs, though the calculation procedure used for the conversion has been unknown. At the Institute of Geology and Engineering Seismology, the maximum acceleration had been calculated for each period of the pendulums and the results had been compiled in the form of a graph. We copied the graph freehand on our notebook, which is shown in Fig. 2.2.4. The two curves represent the maximum acceleration in two perpendicular directions. There are two peaks in the range of 0.2-0.3 seconds and the long-period components are few, as in the case of the strong motion seismograms taken at Yerevan. It is unknown, however, whether the vertical axis represents the maximum acceleration of the ground motion or the maximum response of the pendulum. In general, it is difficult for this type of devices to measure the maximum amplitude of the ground motion. Assuming that the measurements show the acceleration response spectra with a damping constant of 5-8 percent, the maximum acceleration of the ground motion is calculated by dividing it by 2.5 giving 400-520 gals (according to Newmark's standard spectrum, the response magnification factor is about 3 and 2 for a damping constant of 5 and 10 percent, respectively, in the period range where the acceleration response is constant). Compared to this, it has been reported that the maximum amplitude of ground motion was 17-18cm in Leninakan. If these were recorded on a seismoscope, pendulums with long periods should have been used. If the resonance of a pendulum the with longest period (1.0 second) is considered, the maximum base acceleration is estimated at about 70-110 gals.

Japanese workers at an office of a sewing machine manufacturer who experienced the earthquake in Kirovakan said that they had heard an underground rumbling immediately before the earthquake. This represents a feature of an inland earthquake generally experienced near the epicenter. In Leninakan, several witnesses including experts (geologists) reported that the short-period ground motion had continued for about 4 minutes, followed by the long-period shakes, making people unable to move, and most of the damaged buildings had been destroyed during these shakes.

It is difficult to estimate the ground motion in the disaster-stricken area from the data available at present. It seems, however, that the components with the short periods of around 0.2 second prevailed, as inferred from the acceleration response spectra drawn from the strong motion seismograms taken at Yerevan (Fig. 2.2.3) and the acceleration response estimated from the seismoscopic measurements made at Leninakan. This is partly attributed to the fact that since the disaster-stricken area is located among mountains, firm ground of tuff of basalt is exposed in many places and thick deposits of soft alluvial soil are rare seen. Many high buildings constructed based on earthquake resistant design, but in view of the experience with the earthquakes in the past, it is difficult to assume that such heavy damage was caused by these short-period components. Thus, the waves must have contained some frequency components with their periods around 1 second (possibly in the range of 100-150 gals).

(2) Intensity Scale

Rescue of people from under the debris had been nearly completed at long lasting and detailed surveys of the disaster had not started when we visited the site. In particular, little information has been available on the damage to the buildings in the mountainous villages excluding Leninakan, Spitak and Kirovakan, where population is large. Under such circumstances, it would be almost impossible to determine the intensity distribution over the entire disaster-stricken area. The degree of damage must vary largely among small villages among mountains depending on their conditions in the location. For instance, two small villalges (about 4km apart from each other), called Narbanse and Kegasar, which contain several tens of houses or more than a hundred houses exist about 15km away from Leninakan towards Spitak. Old two-storied masonry houses were completely destroyed in Narbanse while there appeared to be little damage in Kegasar when viewed from a distance. Discussed below is the intensity in the above-mentioned three cities, where we visited to make a brief survey. According to local experts, the proportion of destroyed buildings was 100, 80 and 20 percent in Spitak, Leninakan and Kirovakan, respectively. Apparently, several buildings remained standing in Spitak without suffering complete destruction. More than half of the buildings remained standing in Leninakan, and damage was so small in Kirovakan that it was difficult to find the destroyed buildings from the inside of a car. Nevertheless, even the remaining buildings were found to have revealed large damage when checked at close range or from inside. In Spitak and Leninakan, in particular, most of the buildings were found to have suffered "serious damage" according to the Japanese damage classification. The above destruction rate may be regarded as reasonable, though they appeared to be slightly overestimated. Another problem in extimating the intensity is the evaluation of earthquake-resistant performance of each building. It may be possible to some extent to compare their performance with that of Japanese buildings on the basis of earthquake resistant design standards, but it is difficult to estimate the critical earthquake force at which a given building actually constructed in an area would be destroyed. According to information obtained in each city, the intensity was 10 or more (MKS intensity scale, an intensity of 11 have also been reported) in Spitak, 9-10 in Leninakan, and 8-9 in Kirovakan. The value of seismic load used in designing is considerably smaller than that adopted in Japan, and it cannot be assumed that sufficient ductility and been achieved in the joints of the rigid-frame prefabricated houses, which received particularly heavy damage. Considering these points and adopting the lowest possible value, the intensity on the Japanese scale was nearly VII (10 on MSK scale) in Spitak, VI (9) in Leninakan, and V (8) in Kirovakan. According to the zoning based on the earthquake resistant design code, Leninakan (and Yerevan) belongs to a zone of intensity 8 while Spitak and Kirovakan belong to a zone of intensity 7. However, there seemed to be no significant differences in earthquake-resistant performance among the buildings in these cities as far as judging from the visual inspection of the buildings in Leninakan and Kirovakan. Some buildings in Kirovakan appeared to be more resistant than those of the same types in Leninakan. Comparison in the degree of damage taking account the above observations indicates that the estimated earthquake intensities in these areas are reasonable. Specifically, the earthquake motion largely exceeded the design load in Spitak while the intensity was one rank higher on the MSK scale in Leninakan. In Kirovakan, the degree of earthquake motion was nearly the same as or slightly greater than the design load. The intensity in Leninakan, located 35 km away from the epicenter, was higher than in Kirovakan, 15 km away from the epicenter, probably because the destruction of faults propagated toward the west from the hypocenter. The intensity in Yerevan, where damage was smaller, is estimated at IV (6-7 on MSK scale). It might be very reckless to determine the intensity distribution over the disaster-stricken area from these data alone, but a rough estimation of intensity distribution is schematically shown in Fig. 2.2.5.

As stated previously, an earthquake with a magnitude (Ms) of 5.7 took place 12km to the south of Leninakan in 1926, causing destruction of 10 nearby villages and damage to 4,200 buildings in 44 villages and towns including Leninakan. The average intensity in Leninakan was 8 (on MSK scale). The population of Leninakan at that time was much smaller than at present, but a detailed survey was carried out to reveal the conditions of damaged buildings. The Leninakan Institute of Geology and Engineering Seismology later established micro-zoning of Leninakan based on the survey results and the data on the ground conditions. Fig. 2.2.6 illustrates the zoning map, which we copied freehand. An attention should be given to the fact that the multi-storied (mostly nine-storied) apartments of moment resisting frame prefabricated construction that suffered the greatest damage by the latest earthquake were located on the firm ground comprising tuff where damage was small at the time of the previous earthquake. Though not shown in the figure, the areas with an intensity of 8 are classified into two groups in the original map, and it is indicated that damage was slightly larger in the areas where the depth of underground table was 4-5m or less. Shacks stood densely on a dry riverbed in an area with an intensity of 9, but they appeared to have received no significant damage when viewed from a distance.

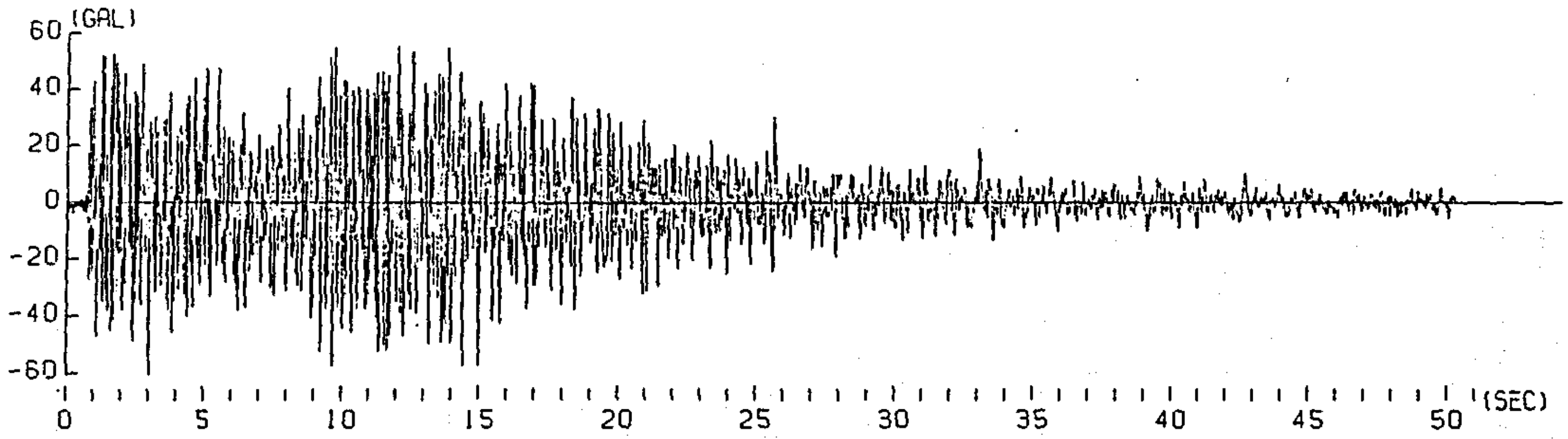



**Fig. 2.2.1 Strong Motion Seismogram of Spitak Earthquake (recorded in Yerevan)**

(Maximum acceleration 61 gals; quantitative analysis by Public Works Research Institute, Ministry of Construction of Japan)

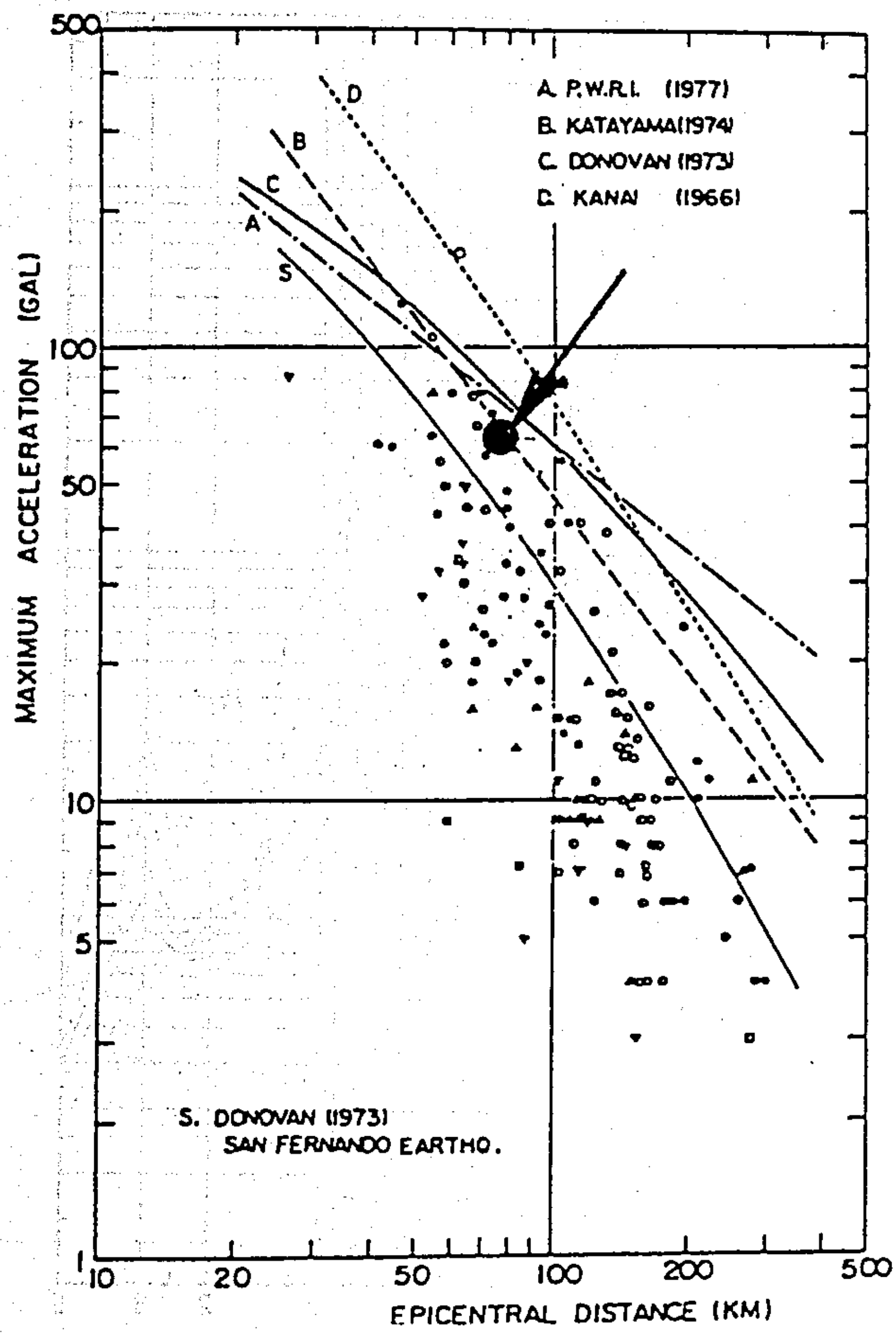



Fig. 2.2.2 **Comparison of Damping at Different Epicentral Distance Observed and Calculated by Empirical Equation (M=7.0)**

( ● refers to measurement in Yerevan for the Spitak earthquake.)

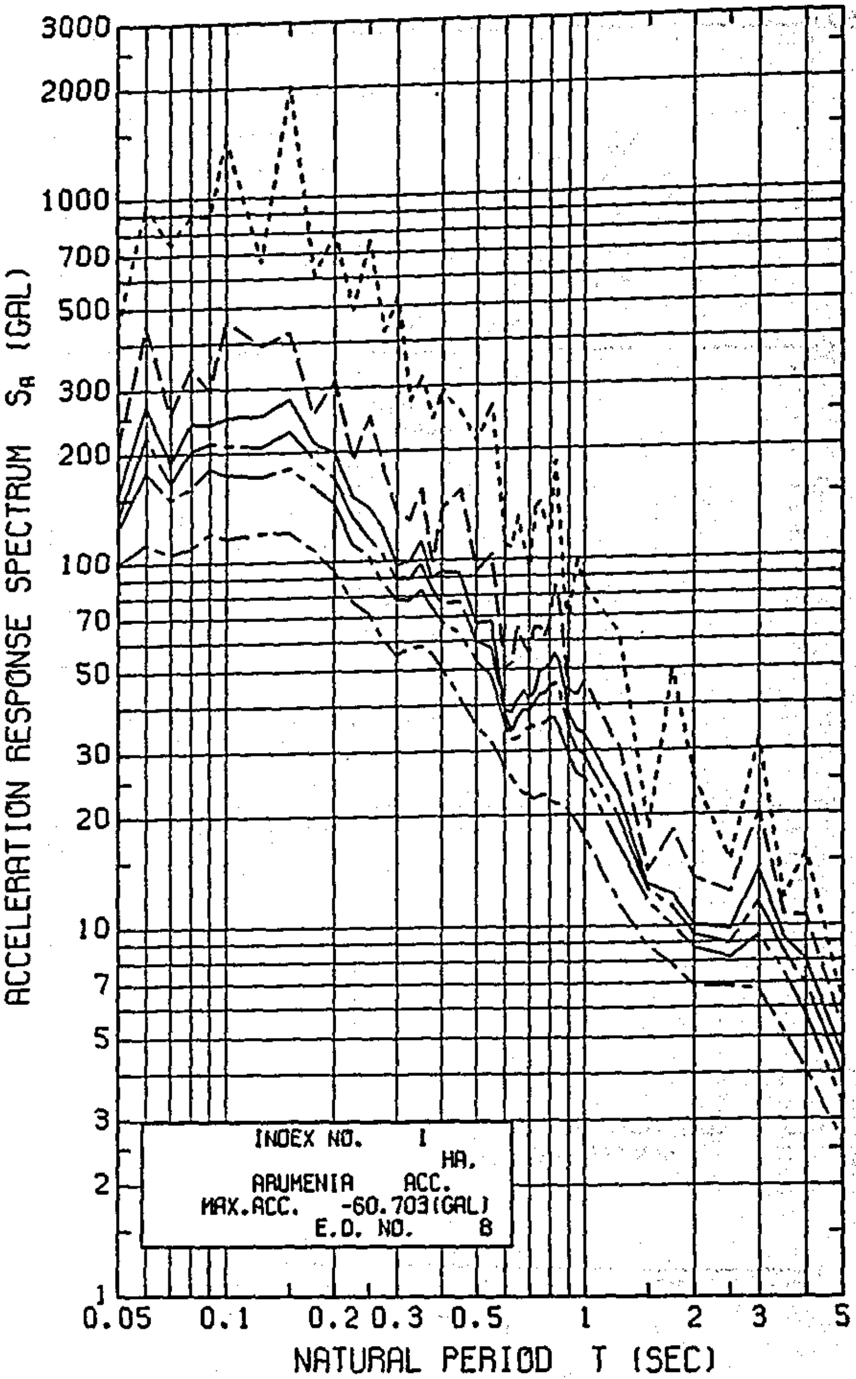



**Fig. 2.2.3 Acceleration Response Spectrum Estimated from Strong Motion Seismogram Taken in Yerevan**

(Quantitative analysis by Public Works Research Institute, Ministry of Construction of Japan)

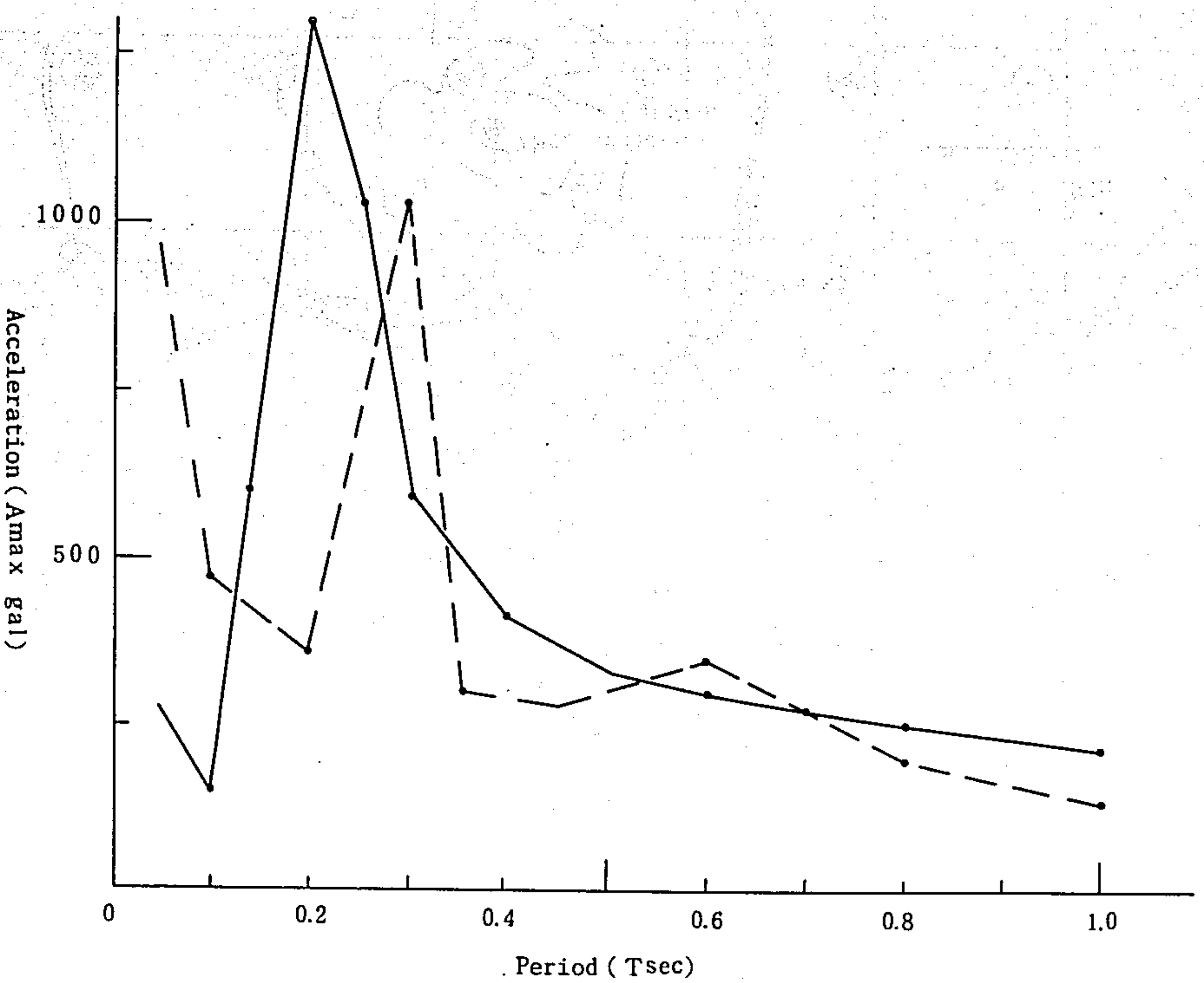



**Fig. 2.2.4 Acceleration Calculated from Seismoscopic Measurement Taken in Leninakan (rough plotting)**

(It is unknown whether the data shows the ground motion or pendulum motion.)

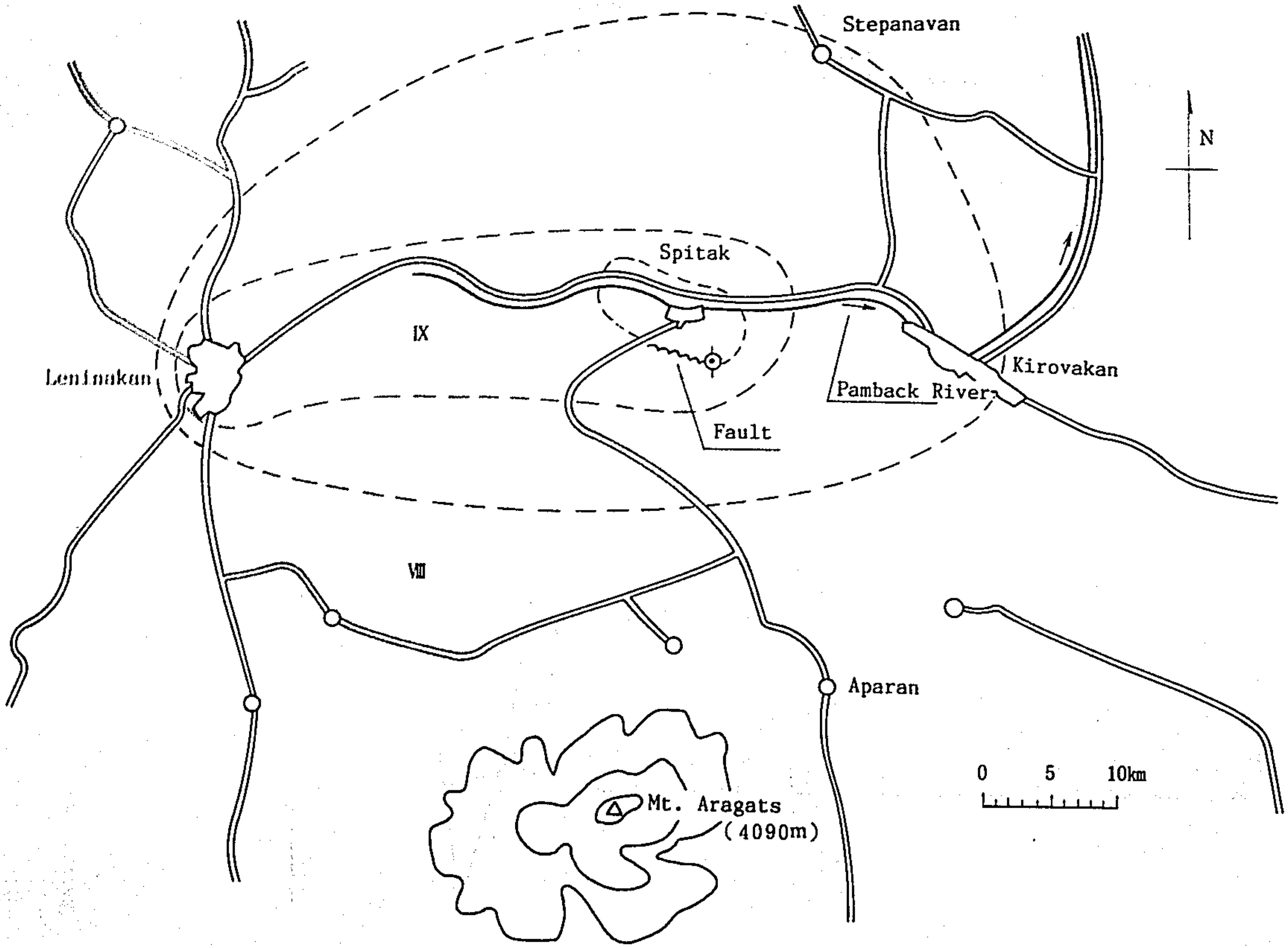



Fig. 2.2.5 Intensity of Spitak Earthquake at Different Points

(Size of square: 1km x 1km)

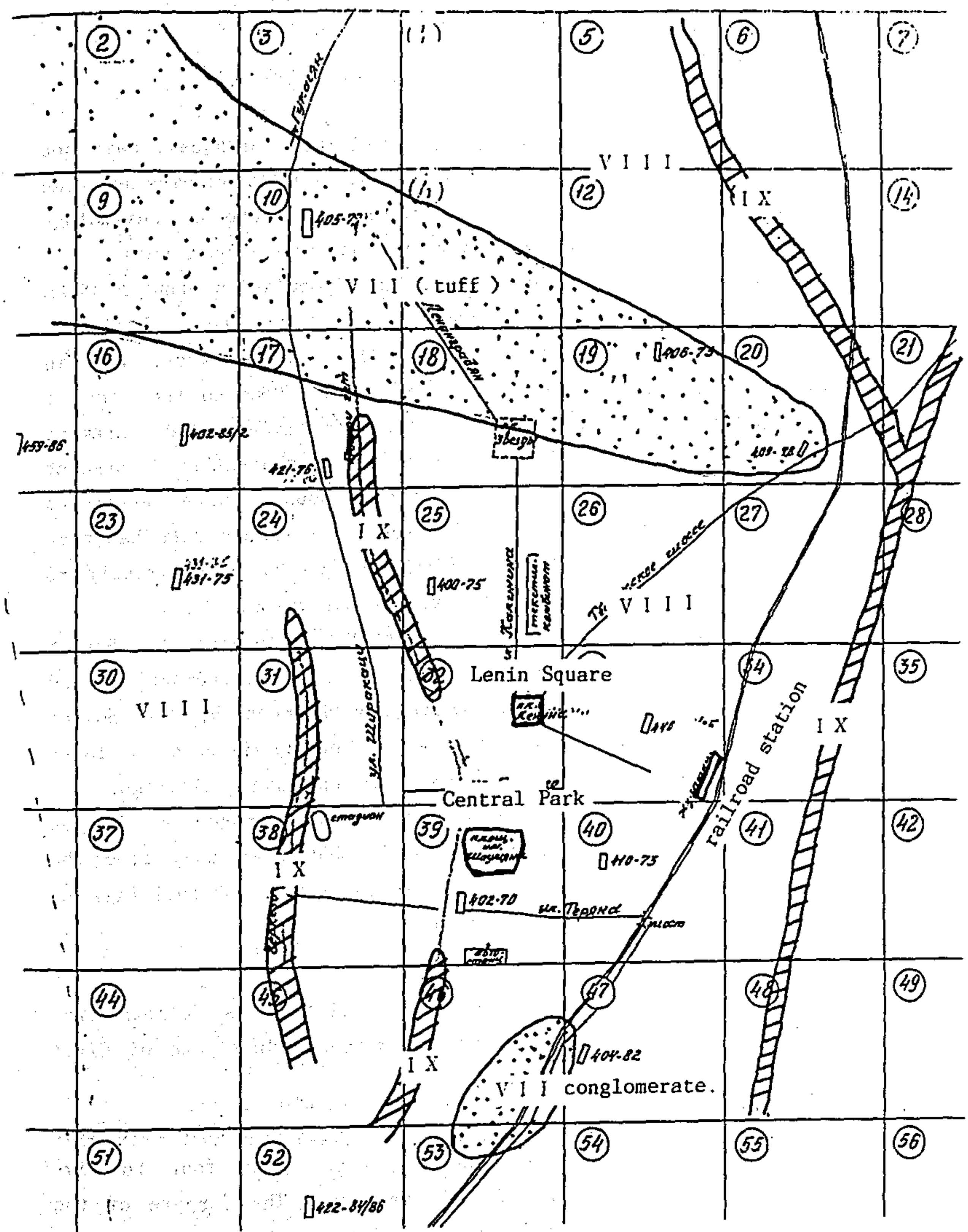



Fig. 2.2.6 Micro-Zoning Map of Leninakan (developed after the 1926 earthquake)

# 3. Damage by Spitak Earthquake

## 3.1 Damage to Building

(1) Outline

Statistics of the damage by the latest earthquake have not been compiled yet (as of December 23). Outlines below are the information obtained from the Armenia Institute of Seismology and estimations from the survey by the Japanese expert team.

Heavy damage was caused in Spitak (population about 20,000), located nearly directly above the hypocenter, Leninakan (population about 300,000), about 35km to the west, and in Kirovakan (population about 200,000), about 20km to the east of the epicenter. Nearly 300 of the four- to five-storied apartment houses made of reinforced masonry and nine-storied precast reinforced concrete frame apartment houses were destroyed completely causing loss of many lives. The damage rate has been reported to be nearly 100 percent in Spitak, approximately 80 percent in Leninakan, and 20 percent in Kirovakan.

The number of death was 90,000 according to Soviet ministries in charge of international trade (December 8) and 45,000 according to the official announcement by the Soviet government (December 10). The number of death must at least 40,000-50,000,judging from the number of destroyed buildings.

The latest earthquake caused little damage in Yerevan (population about 1,000,000), located about 80km away from the epicenter although there were a large number of buildings of the same type as seen in Leninakan.

1) Damage to Apartment House

Damage to apartment houses is estimated as follows from a survey by the Armenia Scientific Research Institute of Civil Engineering and Architecture.

i. Reinforced Masonry: Buildings of this type which have been constructed during the last 30 years, have four to five stories and are made of tuff masonry. The corners of the walls are strengthened with reinforcing bars and concrete filler. PC panels are used as flooring material in most buildings. The damage in each city is estimated as follows.

Leninakan : Approximately 100 buildings destroyed.

Spitak : All buildings (about 60) except a few destroyed.

Kirovakan : Approximately 15 buildings destroyed.

Though the total number of buildings in Leninakan and Kirovakan has been unknown, about 40 percent of the total buildings in the three cities are reported to have been destroyed.

ii. Precast reinforced concrete frame construction: Buildings of this type have been constructed during the last about 10 years and have nine stories. Precast concrete curtain walls are used as exterior material in most cases. The damage in each city is estimated as follows.

. Leninakan : All buildings (about 100) destroyed.

. Spitak : There were no buildings of this type. (According to the field survey, however, all 30-50 buildings except for 2 (about 60) were destroyed. They are estimated to have five stories.)

. Kirovakan : There were about 70 buildings, but none of them collapsed.

iii. Large precast concrete panel construction: Buildings of this type have been constructed since 1985 and have five to sixteen stories.

. Leninakan : There were 7 nine-storied buildings including those under construction, but none of them collapsed.

. Spitak : There was one five-storied building, which did not collapsed.

iv. Conventional Masonry: Old-type masonry of tuff.

In Leninakan, there were a large number of two- to three-storied old-type masonry houses constructed since the 1920s. Some were destroyed though details have been unknown.

There were one- to two-storied private houses of conventional type masonry construction in villages near the epicenter. It has been reported that there was heavy damage in some villages, for instance in Shelak, located about 15km to the northeast of Leninakan, while damage was small in Jajuk, a nearby village, but details have been unknown.

2) Damage to School Building: Two- to three-storied school buildings of precast reinforced concrete suffered from damage as follows.

- Leninakan : 7-8 buildings destroyed (total number unknown).
- Spitak : 7 of total 8 destroyed.
- Kirovakan : No school building destroyed.

(2) Damage to Building in Major Earthquake-Stricken Area

1) Spitak

Spitak is the closest city to the epicenter with a population of about 20,000, as illustrated in Fig. 3.1.1. Manufacturing accounts for the major part of the businesses in the city. The Pamback River flows towards the east through the lowland area extending in the east-west direction in the middle of the city, with a railroad running parallel to it, as shown in Fig. 3.1.2. Urban district extends towards the hills on the north and south of the lowland area. A small hill with a stone-built memorial tower standing at the top exists at the center of the south part. A tributary of the Pamback flows from southwest towards northeast through the southeast part of the city.

The buildings in this city can be classified as shown below. None of them has six or more stories. There are no buildings of WPC construction.

i. Five-storied apartment houses of RPC construction
ii. Five-storied apartment houses of RMS construction
iii. One- or two-storied houses, etc., of masonry construction
iv. One- to three-storied factory buildings of RPC construction
v. Large silos and factories of precast reinforced concrete construction

These buildings can be classified by the degree of damage as described below, as illustrated in Fig. 3.1.2.

i. Area I-1: Almost all five-storied apartment houses of RPC construction (total number unidentified, probably several tens) collapsed. There was only one building (comprising many PC wall panels) that suffered little damage and three that narrowly avoided destruction (Photograph 3.1.1).

ii. Area I-2: Most of the five-storied RMS apartment houses and stone residences suffered complete or nearly complete destruction (Photograph 3.1.2).

iii. Area I-3: Most of the RPC and stone-built factory buildings and one- to two-storied stone residences collapsed or heavily damaged (Photograph 3.1.3).

iv. Area II-1: Most of the five-storied RMS apartment houses and stone residences suffered considerably heavy damage (Photograph 3.1.4).

v. Area II-2: Most of the RPC and stone-built factory buildings and stone residences suffered considerably heavy damage.

vi. Area II-3: Most of the stone residences suffered heavy damage.

vii. Area III-1 and III-2: Stone buildings accounted for a great part of the total buildings. Judged from appearance, they appeared to have suffered little damage (Photographs 3.1.6 and 3.1.7).

viii. Area IV: There was a graveyard on the side of a small hill. (Their bottoms were mortared. Whether they were embedded is unknown.) All gravestones including those with large height/thickness ratios remained standing (Photograph 3.1.8).

Fig. 3.1.2 shows the degree of damage to major structures including the memorial tower on the hill (slightly damaged; Photograph 3.1.9) and large silo (partly destroyed; Photograph 3.1.10).

2) Leninakan

Leninakan is located about 35km to the west of the epicenter as illustrated in Fig. 3.1.1. There are plateaus on the north and east of the city while small hills exist on the west. Surrounded by these features, the city extends along basin-like land open towards the south. About 5km to the west of the center of the city, there are two parallel rivers, called Aknurian and Cheketz, running in the north-south direction. A tributary flows in the north-south direction through the middle of the city, partly through a closed conduit. A railroad runs in the north-south direction along the east edge of the city and the Leninakan Station is located near the center of the east edge, as illustrated in Fig. 3.1.3. Leninakan, which developed in years before Christ, is currently the second largest city in Armenia, with a population of approximately 290,000. The central and south parts constitute the old section of the city, where there are old masonry churches, public buildings and office buildings. Compared to this, high-rise (mainly nine-storied) residential buildings, mostly apartment houses, began to be constructed about 10 years ago in the north-western edge of the city. Construction of them has been continuing up to now. There are various types of buildings which are of different construction and have been built by different methods as described under Section 4.6, but tuff is used as exterior material whether it is structural or not, in all these buildings. We surveyed key spots in many parts of the downtown section of the city, though the section expanded so widely that it was impossible to make a survey of all buildings there.

According to this limited survey, major types of building and degree of damage in each area defined in Fig. 3.1.3 are as follows.

i. Area I-1: This is a residential area where nine-storied RPC apartment houses account for most of the buildings. Around 20 percent (estimated) of the apartment houses were narrowly saved from collapse, but most of the rest were destroyed completely (Photographs 3.1.11. and 3.1.12).

ii. Area I-2: This area contains various types of buildings including five-storied RMS apartment houses and RTC department stores addition to old stone buildings. Many buildings were destroyed completely. (Photograph 3.1.13).

iii. Area II-1: This area is the central part of the city, where there are many low buildings, mostly three-storied or lower, including churches and public buildings. It appeared that many buildings were destroyed, while most of the small-scale one- to two-storied buildings had suffered little damage (Photograph 3.1.14).

iv. Area II-2: Many three-storied stone apartment houses (apparently non-reinforced) are built along both sides of the main street from the station to the Lenin Square. Largely destroyed buildings mainly with damaged gables and apparently undamaged buildings coexisted in this area (Photograph 3.1.15).

v. Area III-1: This is a residential area located at the north edge of the city. Several (at least 6) nine-storied apartment houses of WPC construction stood there. One of them was under construction and wall panels had been laid up to the seventh floor. All these buildings were reported to have suffered little damage, except for the one under construction (floor slabs of the eighth floor) (Photograph 3.1.16).

vi. Area III-2: Nearly 10 five-storied apartment houses of (apparently) RPC construction stood along the street coming from the Yerevan area into Leninakan. When viewed from outside, these apartment houses, nearly free of broken windowpanes, did not appear to have significant damage, except for uniform cracks formed diagonally along the edges of wall panels which were located at the ends of the buildings and had small openings (Photograph 3.1.17).

vii. Area III-3: This area contains many old one-storied residences, most of which were of stone. Damage to them was relatively small though some stone walls were partly broken or dropped (Photograph 3.1.18).

viii. Area IV: Further to the west of the Aknurian River, a village comprising mainly one-storied stone residences existed on a hill side sloping down towards the cast. It was reported that the village did not suffer damage (Photograph 3.1.19).

In addition, Fig. 3.1.3 shows the locations of major buildings and structures including a statue standing on the slope in the southwestern part (undamaged; Photograph 6.1.20), a stone-built stadium (undamaged; Photograph 3.1.21), the Leninakan Station (two-storied masonry, undamaged; Photograph 3.1.22), and an sixteen-storied apartment house of mixed RC and PC construction (heavily destroyed; shown previously), which is the highest building in the city.

3) Kirovakan

Kirovakan is located about 20km to the west of the epicenter. The city, having a population of approximately 170,000, is located between hill areas on its north and south and extends along a river flowing mostly in the east-west direction. We had only the chance to survey a limited part of the city though it had been reported as stated previously that about 15 apartment houses of RMS construction had collapsed. Apparently, the degree of damage seemed low as compared to the two above-mentioned cities, Spitak and Leninakan. We observed the inside of the first story of an apartment house standing next to the five-storied RMS apartment house (Photograph 3.1.23) which had been pulled down and removed after being partly destroyed by the earthquake. It did not suffer damage and was almost free of cracks. Photograph 3.1.24 shows the RMS factory buildings and the apartment houses standing over the slope in the northeast part of the city. They appeared to have suffered little damage.

4) Other Cities

For areas other than the three above-mentioned cities, Fig. 3.1.1 outlines the damage in the towns and the villages located along roads connecting these three cities.

i. Small Village about 10km to the West of Spitak

It is reported that there were one-storied masonry houses, most of which collapsed though mostly having wood roofs. A PC platform in a station near the village dropped (see Photographs 3.1.25 and 3.1.26).

ii. Ashitarak about 15km to the Northwest of Yerevan

One- to two-storied stone houses are scattered spacely on both sides of a valley about 15m deep, but they suffered little damage (Photograph 3.1.27).

iii. RC factory buildings and stone houses consisting of slender members as shown in Fig. 3.1.28 are scattered sparcely near Mralik, which is located about 25km to the south of Leninakan (about 40km to the west-south-west of the epicenter), but they suffered little damage (Photograph 3.1.28).

iv. Yerevan

The city, capital of Armenia with a population of about 1,000,000, is located approximately 80km nearly to the south of the epicenter. The city contains a great number of buildings including five-storied RMS apartment houses and ten-storied RPC apartment houses. In addition, many high-rise apartment houses probably of RPC construction are under construction in the suburb on the north of the city. A strong motion seismograph installed at the first floor of the city's Research Institute of Architecture showed a maximum strong motion of 63 gals. No particular reports have been published as yet concerning the damage to buildings in Yerevan (Photographs 3.1.29 and 3.1.30).

### (3) Typical Damage to Different Type of Building

#### 1) Low Stone Building

The low stone-built buildings can be classified into some groups by use and size: one- or two-storied private residences, and two- or three-storied apartment houses, offices, shops, churches and public buildings. By the structure of the stone wall, they can be grouped into two categories as illustrated in Fig. 1.6.3.: in one, shaped stones are laid with the jointing mortar (referred to as masonry type stone wall), while in the other, stones in which only the external face is shaped are placed like a form the inside and outside of the wall with the gaps filled with concrete or stones (referred to as filled-type stone wall). Private houses may be either of the two types while the greater part of the buildings with three or more stories is of the filled-type.

Damage to private houses was very large in Spitak. In particular, most of them were completely destroyed regardless of structural type in the low land areas in the center of the city (Areas I -3 and II -3), as shown in Photographs 3.1.31 and 3.1.33. Compared to this, damage to private houses appeared to be small in the hill area in the south part of Spitak and in almost all parts of Leninakan, as shown in Photographs 3.1.6 (described previously) and 3.1.34. The conditions of these damaged private houses are similar in many aspects to those of two- to three-storied buildings described later.

For two-to three-storied stone buildings other than private houses, damage was heavy in Leninakan though damaged ones were seldom seen in Spitak. Damaged ones in Leninakan are shown in Photographs 3.1.35-3.1.38.

A survey of buildings which were destroyed severely but not completely revealed that the major part of them suffered damage to the top or corners of the gable which did not directly support the floor plates or trusses.

2) Reinforced Stone-Built Apartment House

Of the stone-built apartment houses, many of the four- to five-storied ones were reinforced to meet the anti-earthquake requirements given under Section 4.6 while some of the one- to three-storied ones were found to be unreinforced.

Nevertheless, the degree of reinforcement and the volume of walls (expressed by the length of wall per unit floor area) varied among different buildings. Some of them could be regarded as a structure like rigid-frame construction because of having large openings.

About 450 buildings of this type have been constructed during the last 30 years in the three cities of Leninakan, Kirovakan and Spitak. Of them, about 100 in Leninakan were completely or heavily destroyed. Only two or three remained undestroyed. With all of the remaining about 60 were completely or heavily destroyed in Spitak. In Kirovakan, about 15 were reported to have been destroyed completely.

These damaged buildings are shown in Photographs 3.1.39-3.1.45.

Many of the buildings of this type suffered damage to their gables (Photographs 3.1.41 and 3.1.43). There were many types of the destruction including a local damage to intermediate stories (Photograph 3.1.42). Since the exterior walls were built of stone, it was not easy to identify cracks in them. However, clear cracks (widening of joints or connections) were seen around small openings in the external walls in some buildings which appeared to have unevenly distributed walls.

3) Rigid-Frame Precast Reinforced Concrete (RPC) Apartment House

According to information obtained during the field survey, all RPC apartment houses in the area have nine stories. None of them were in Spitak. It was reported that all of the about 100 in Leninakan had been destroyed completely while about 70 suffered damage in Kirovakan. In the northern hill area in Spitak (Area I-1 in Fig. 3.1.2), however, there was a housing development (Photograph 3.1.1) where all of the several tens of apartment houses (probably five-storied) were destroyed completely except for a few, as stated previously. The ones remaining undestroyed include 2 five-storied RPC buildings under construction, a five-storied apartment house of similar construction, and a five-storied WPC apartment house. PC members regarded as beams and pillars as well as floor void slabs were found in the apartment houses in the development which were destroyed, suggesting the possibility that they might be of RPC construction.

Destroyed buildings had already been removed when we visited Spitak. The observations outlined below are mainly focused on Laninakan.

Photograph 3.1.46 shows typical conditions seen in RPC housing developments in Leninakan. All apartment houses except for the three shown in this photograph were completely destroyed and reduced to debris, forming a large number of rubble heaps all over the area. In the building at the center which narrowly remained undestroyed, the first story was damaged severely while the second and third stories expanded largely, making the building appear to be ready to collapse. Photograph 3.1.47 shows lower stories in the remaining part of a building that suffered damage to its central portion. Several beams almost free of significant cracks hang from the structural pillars. All of the pillars and beams on the external face of the building and the precast members appearing to be walls are exterior finishing materials, rather than structural materials. Major features of the damage include the following: no significant damage is seen in these finishing materials; wall materials are destroyed almost completely though precast structural members such as pillars, beams and void slabs account for most part of the members that had fallen without being destroyed; the PC members that have dropped do not have significant cracks appearing to be caused by horizontal forces; and many beams remain hanging while all void slabs have fallen rather than remaining hanging.

Photograph 3.1.48 shows another building which remained undestroyed. This is a rare example in which the exterior decorative panels existing from the second to fourth stories suffered large cracks, with many of the windowpanes in that portion being broken.

Photograph 3.1.49 (overall view of the building in Photograph 3.1.47) and Photograph 3.1.50 (five-storied building in Spitak which might not an apartment house) also show rare examples in which the central portion of the buildings were destroyed. Photographs 3.1.51 and 3.1.52 depict the joints of PC members in destroyed buildings respectively.

Photographs 3.1.53-3.1.55 show RPC apartment houses that were destroyed during construction. The one in Photograph 3.1.54 is a five-storied building comprising PC wall panels for decoration while the five-storied one in Photograph 3.1.55 has no beams in any direction and uses small PC panels as wall material. In particular, attention should be given to the fact that the latter, in which walls had been constructed to a considerable degree, suffered less damage than the former two, which had no walls laid when they were destroyed heavily.

Of the RPC apartment houses, most of which were destroyed heavily or completely, the nine-storied one in Kirovakan depicted in Photograph 3.1.56 suffered no damage though its exterior was built of masonry-type stone walls, and the five-storied one in Photograph 3.1.57, in which many PC decorative panels were used as exterior wall materials, also remained undestroyed. Attention should be given to the fact that these apartment houses and the building shown in Photograph 3.1.55 were not damaged significantly. The five-storied WPC apartment house in Photograph 3.1.58, as detailedly described later, was the only building that remained undestroyed in the most heavily stricken area in Spitak, that is, Area I -1 in Fig. 3.1.2. The major features common to many of these less damaged buildings are that the ground motion at the site was small (Kirovakan) and that the volume of walls that appeared to be effective (in spite of being decorative walls) was large against the input.

4) Wall-Type Precast Reinforced Concrete (WPC) Apartment House

There were only six apartment houses of WPC construction in the area: 5 nine-storied ones in the suburbs of Spitak (Photograph 3.1.16, another shown in Photograph 1.6.15 which had been completed up to the seventh story) and a five-storied one in the most heavily damaged area in Spitak (Photograph 3.1.58). All these buildings except for a few are reported to have suffered little damage.

Damaged RPC buildings are shown in Photographs 3.1.59 and 3.1.60. The former depicts floor panels dropped from the eighth story in the apartment house shown in Photograph 1.6.15, which had been completed up to the seventh story, while the latter shows diagonal cracks seen in part of the panels with openings used in five-storied apartment houses shown in Photograph 3.1.58. These observations indicate that WPC buildings suffered much less damage than RPC ones, reflecting their high anti-earthquake strength, which is one of the anti-earthquake features peculiar to the WPC buildings.

5) Building of Other Construction

Outlines below are conditions of damaged buildings of structures other than those described above.

i. RPC Building Other than Apartment House

It appeared that the buildings for different uses other than apartments had been constructed by the RPC method, but it was difficult to identify them from their external appearance unless they suffered significant damage. Some heavily damaged ones are shown in Photographs 3.1.61–3.1.63. As inferred from these photographs, there seemed to be many other damaged buildings that had been constructed by the RPC method.

Photographs 3.1.61 and 3.1.62 show a front view and a side view, respectively, of a four-storied department store destroyed completely (Leninakan). Photograph 3.1.63 depicts apparently an office building which appears to be of masonry construction (Leninakan).

The factory buildings given in Photographs 3.1.64 and 3.1.65 are of precast rigid-frame construction (partly braced) in one direction and of rigid-frame or truss construction in the other direction. In the buildings of this structural type, the roofs of precast plates were not damaged severely, but the surface of the exterior masonry mostly fell off.

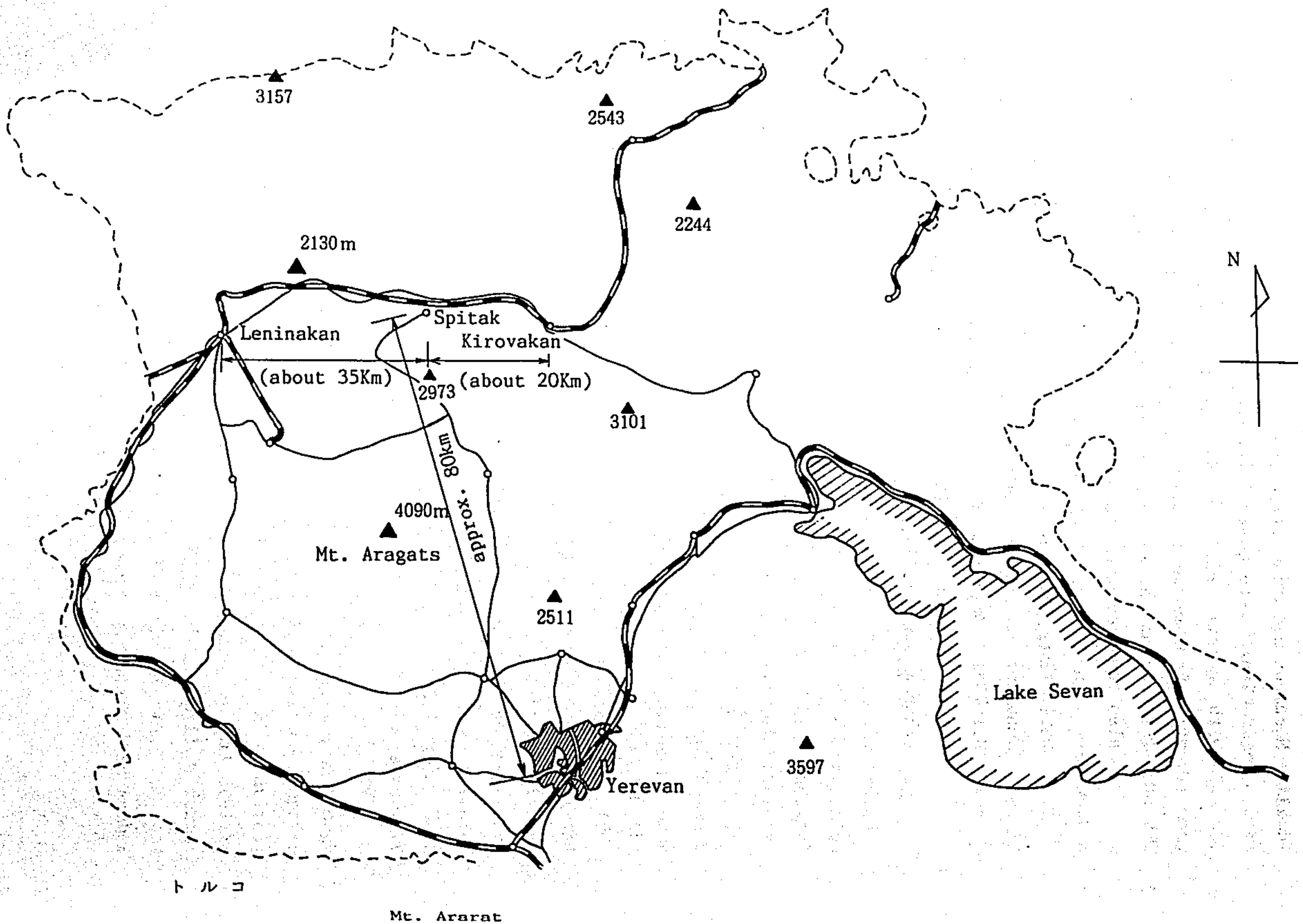
ii. Photographs 3.1.66 and 3.1.67 show an overall view and damaged portion, respectively, or a sixteen-storied apartment house of a mixed structure in which the central core part of the building is made of cast-in-place reinforced concrete (RC) while the floor slabs and pillars are of PC material (Leninakan). The core in the first story was heavily destroyed and the exterior was also damaged considerable as seen from the photographs, though its pillars appeared to have suffered no damage. However, most of the vertical reinforcing bars in the RP core wall had buckled largely, and the structure as a whole was in heavily damaged condition.

Photographs 3.1.68–3.1.71 depict buildings which though apparently seem to be of reinforced masonry, appear to comprise a relatively small number of stone walls and a relatively large number of reinforced concrete pillars or pillar-like members at the portions corresponding to the pillar forms. These RC portions were destroyed heavily in many of the buildings of this type, probably because they had a less number of walls.

iii. Monument and Other Structure

There were plate-like bus stop structures and monuments with a tower ratio of 4-10 along a road to Leninakan, but all of them were found to remain standing. The monument (statue) at the top of the hill in the southwest of Leninakan also remained undestroyed.

In Spitak, simple structures like those at gas stations were destroyed completely (Photograph 3.1.72) while a stone monument on a hill suffered only slight damage (Photograph 3.1.73). Gravestones standing nearby (their bottom seemed to be embedded in the ground) remained almost undamaged (Photograph 3.1.74).


N
3157
2543
2244
2130m
Spitak
Leninakan
Kirovakan
(about 35Km)
2973
(about 20Km)
3101
approx. 80Km
4090m
Mt. Aragats
2511
Yerevan
3597
Lake Sevan
トルコ
Mt. Ararat

[redacted]

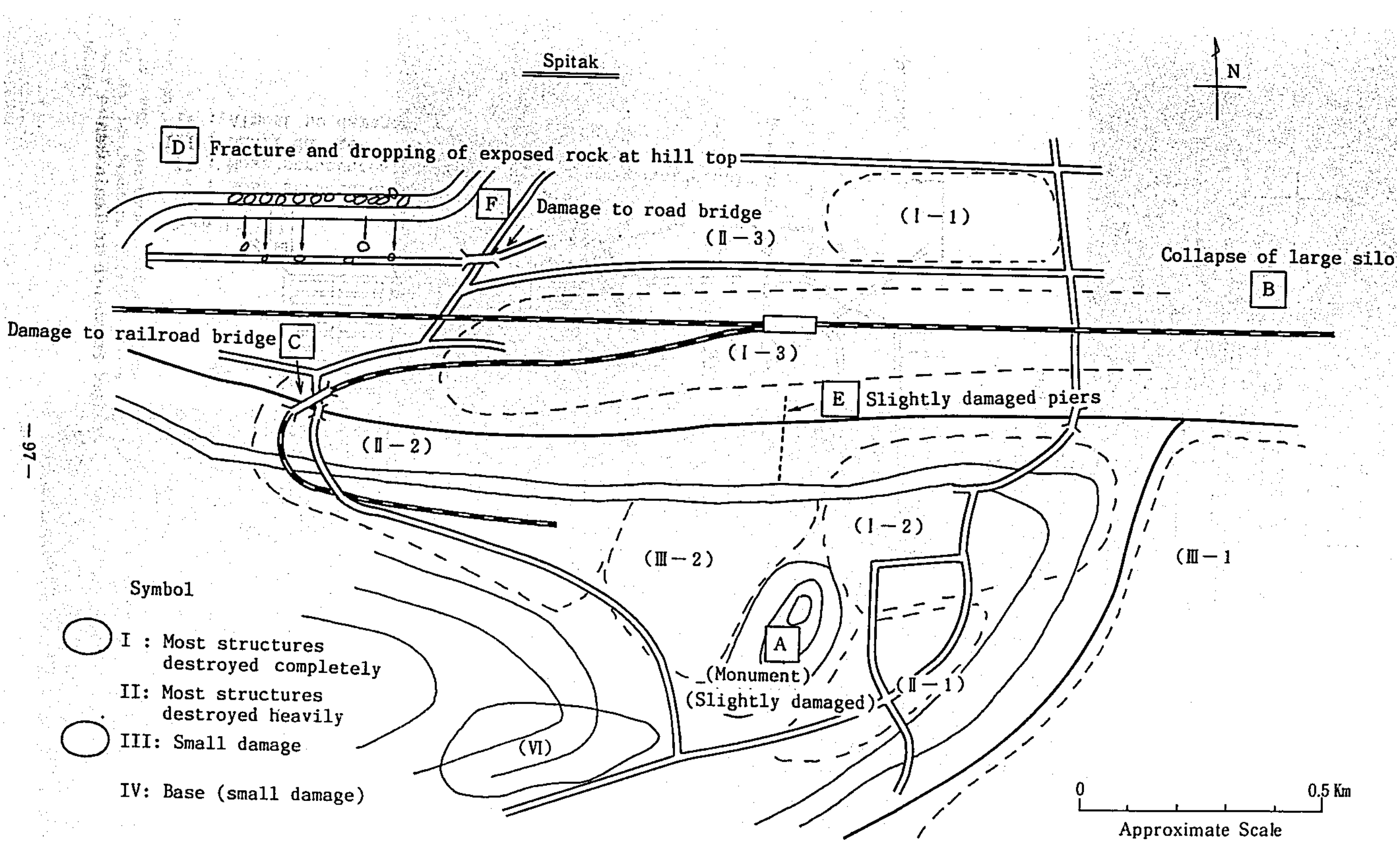



Fig. 3.1.2 Distribution of Damaged Structure in Spitak

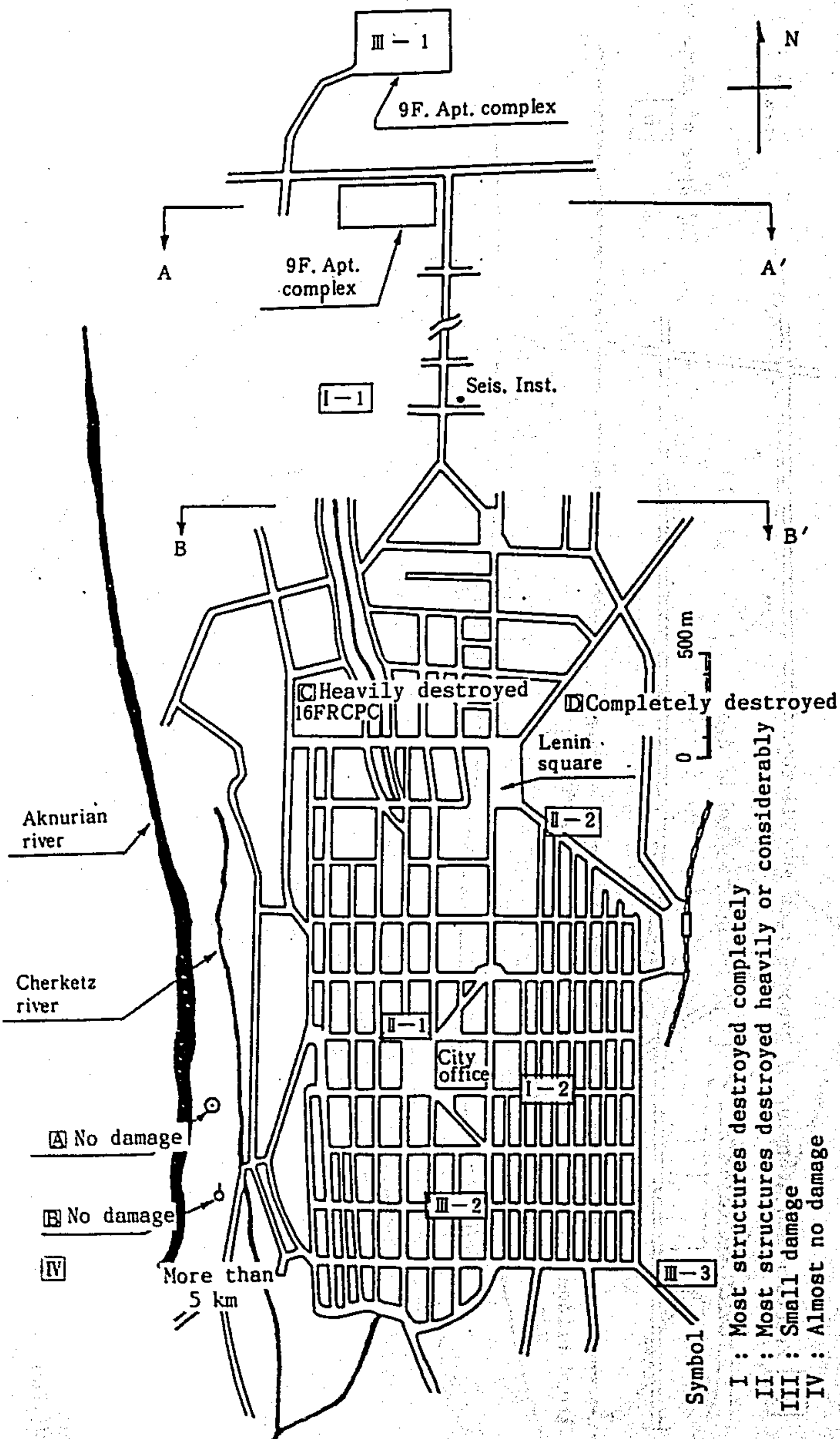


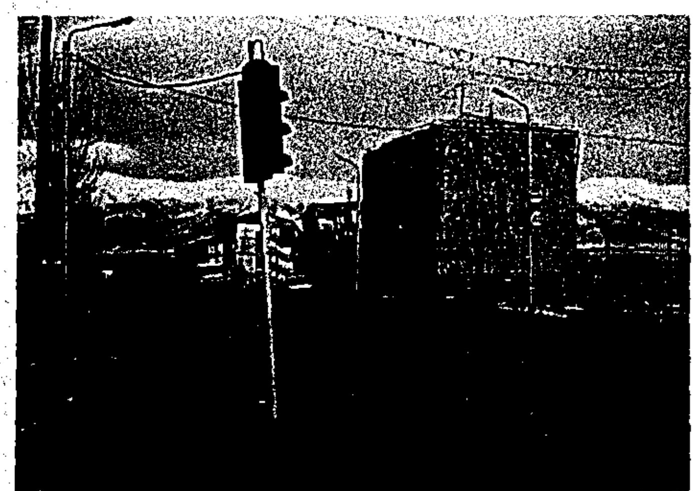

Fig. 3.1.3 Distribution of Damaged Structure in Leninakan

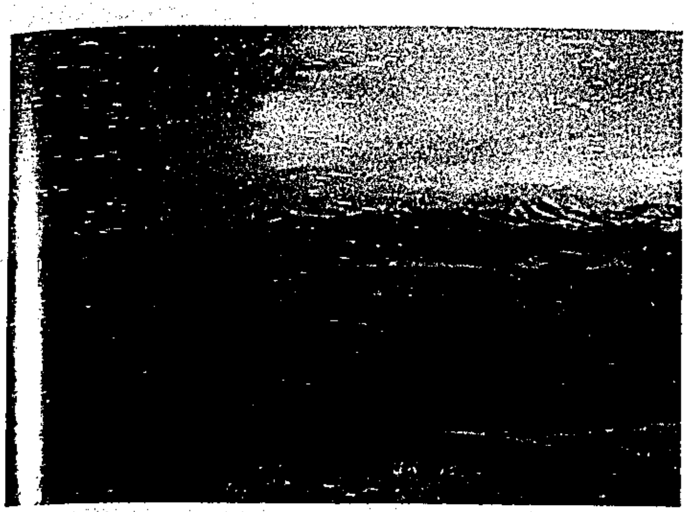
Photograph 3.1.1
A housing development consisting of five-storied RPC apartment houses located at hill area in northern Spitak. At least about 20 buildings were completely destroyed while only three or four including those with many walls or under construction remained understroyed.

Photograph 3.1.2
Destroyed five-storied RMS apartment houses and other structures in area near hill slope in the both sides of Spitak. At least more than 10 appartment houses were destroyed completely to debris in this area.

Photograph 3.1.3
Damaged buildings in lowland area along Pamback River flowing in east-west direction through central portion of Spitak. Most of the low RPC factory buildings and low stone residences were destroyed heavily or completely.

Photograph 3.1.4
Damaged RMS apartment houses along street in southeastern Spitak. There are many partially or heavily destroyed buildings.

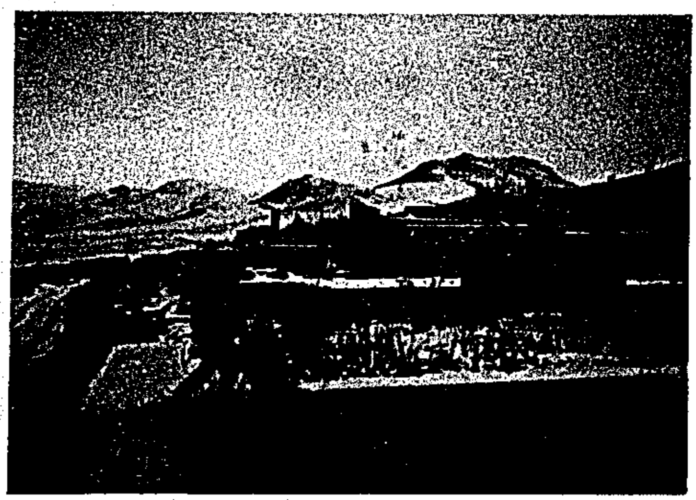




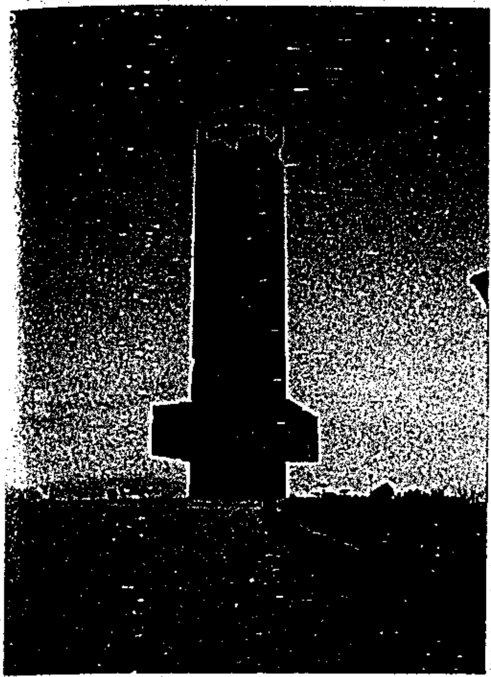
**Photograph 3.1.5**
Damaged buildings in south part (center of photograph) of lowland area in central Spitak viewed from hill in northwest of the city. There are heavily destroyed factories and other buildings.

**Photograph 3.1.6**
Low houses in eastern Spitak (foot of mountain) viewed from hill at center of south area of the city. Damage appears small.

**Photograph 3.1.7**
Low residences in the west of central Spitak (bottom of photograph) viewed from the hill at center of south area of the city. Damage appears small.

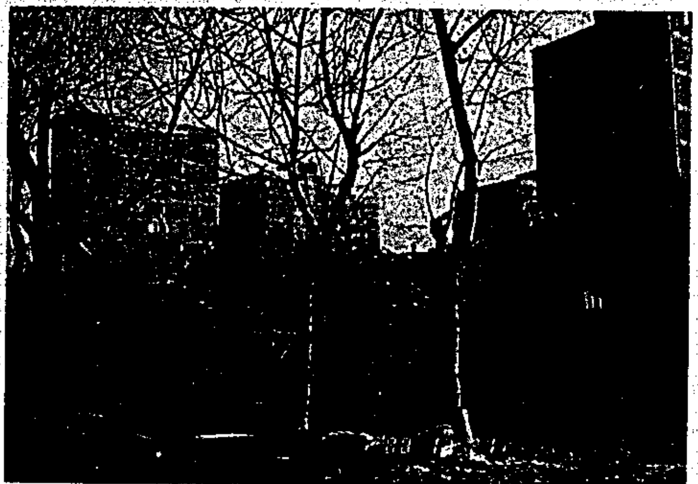


**Photograph 3.1.8**
Graveyard on small hill near south edge of Spitak. All gravestones appear to remain standing, including those with large thickness-height (H/t) ratio looking towards the north. The depth of the gravestones which embedded into their base stones is unknown, but mortar is used.



Photograph 3.1.9
A stone monument at the top of the hill at center of southern area of Spitak. The monument as a whole is considerably or slightly damaged, except for part of the projected stone member in the middle partially broken and fallen.

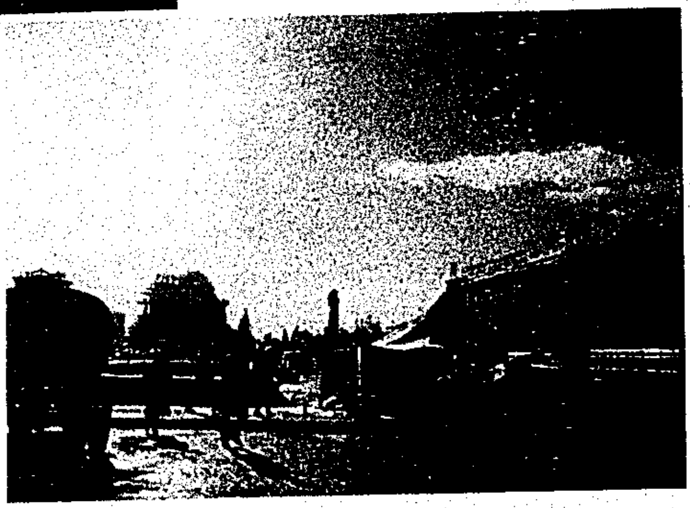
Photograph 3.1.10
A damaged large silo and auxiliary facilities located at western edge of north part of Spitak. A silo filled with grains has fallen down and the auxiliary facilities have partially been destroyed heavily or collapsed.



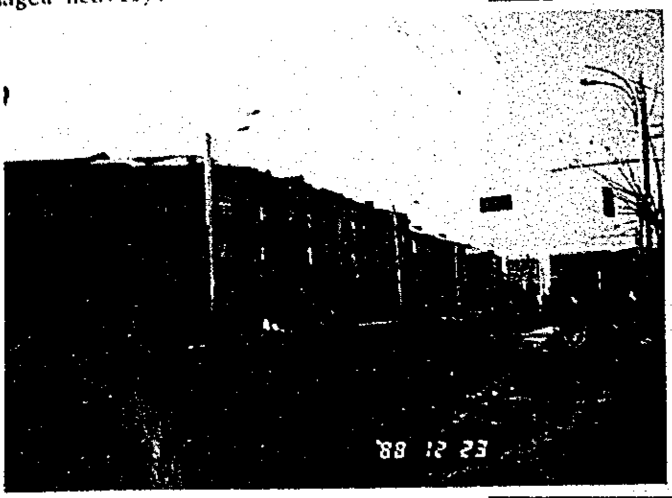
Photograph 3.1.11
Completely destroyed nine-storied RPC apartment houses in housing development located northwest part of Leninakan. All apartment house narrowly remaining standing were also destroyed heavily.

Photograph 3.1.12
Same site as above. PC floors and PC pillars almost free of rupture are found among the debris of buildings.

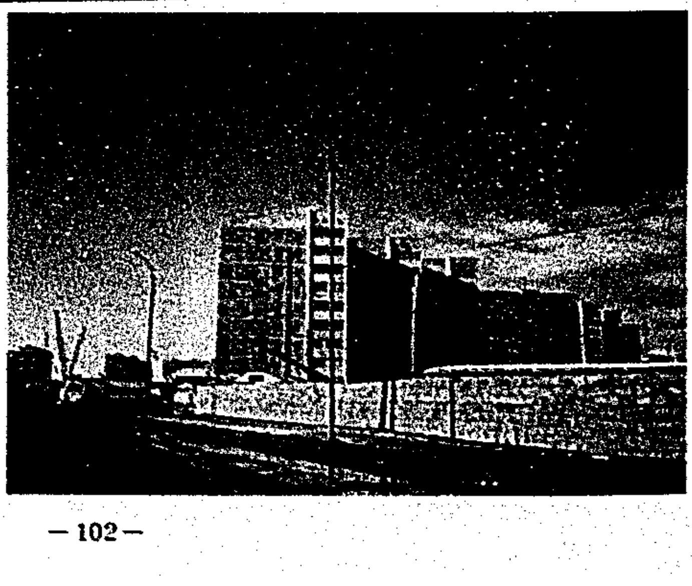
Photograph 3.1.13
Completely destroyed RMS apartment houses among old stone buildings in central part of Leninakan. They were completely reduced debris, leaving a huge number of rubble heaps of tuff used as wall material.



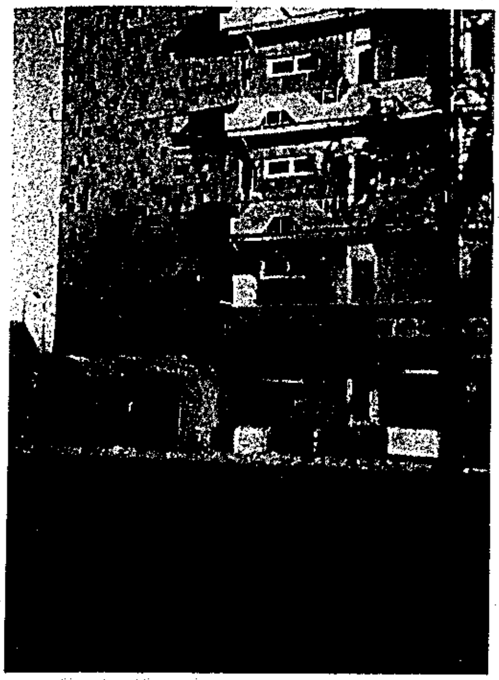
Photograph 3.1.14
Damaged old stone buildings in Central Square in Leninakan. The stone-built church in the middle of the photograph had its hall part completely destroyed. The part was located on the left of the gable wall, which stood like a small tower to the right of the church. Compared to this, the three-storied masonry hotel in the right part of the photograph suffered little damage, while the three-storied masonry government building at the opposite side of the square had its fourth story damaged heavily.




Photograph 3.1.15
Old three-storied stone-built apartment houses standing on either side of principal road running from Lenin Square to railroad station located at east edge of Leninakan. With small windows, most of those apartment houses suffered only small damage. There was large damage (collapse of gable, etc.) to many of the relatively new three-storied stone apartment houses and five-storied RMS ones which stood behind the ones in the photograph.

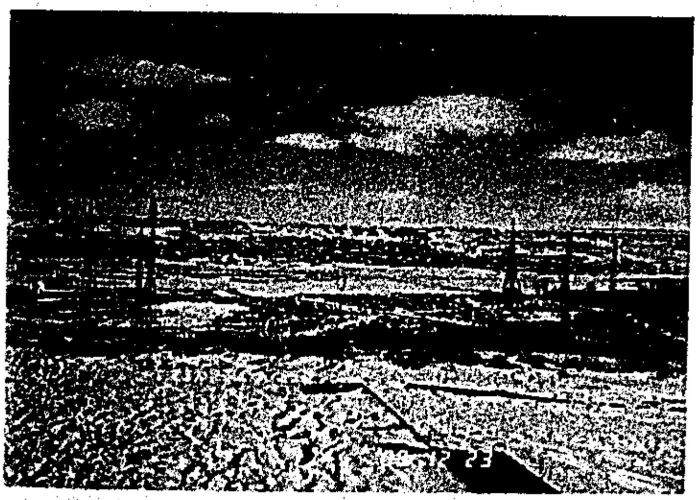


Photograph 3.1.16
Wall-type precast reinforced concrete (WPC) apartment houses under construction in new housing development at north edge of Leninakan. Tuff cut thinly as decorated material was precast with the outer structural wall panels. They seem to have worked effectively as anti-earthquake walls. These buildings suffered no significant damage.



Photograph 3.1.17
Damaged outer wall with small openings of five-storied, (probably) RPC apartment house standing on a road coming from the south into Leninakan. Similar damage was seen in all other buildings along the road, though they were almost free of other types of damage.

Photograph 3.1.18
An urban zone mainly comprising old one-storied stone residences has developed in the north part of Leninakan. Most buildings appeared to have suffered only slight damage.



Photograph 3.1.19
A community comprising one- to two-storied stone residences has developed on a slope located to the west of the Aknurian River flowing in the north-south direction about 5km to the west of Leninakan. These buildings were reported to be almost free of damage.

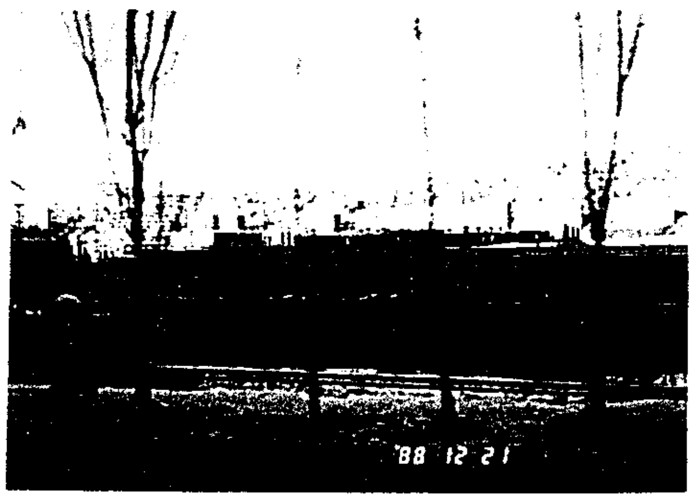
Photograph 3.1.20
Statue on stone base located on small hill at southwest edge of Leninakan. It remains completely free of damage.

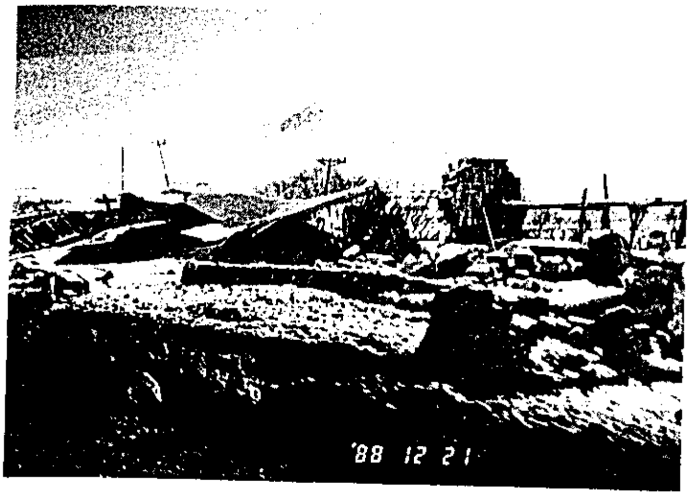
**Photograph 3.1.21**
A part of Leninakan viewed from top of a small hill at the southwest edge of the city in direction of the north-east. A stone-built stadium is seen in the left-middle of the photograph. No report was available on its damage.


**Photograph 3.1.22**
Two-storied stone-built Leninakan Station. Free of damage. The exterior stone material of a rusty color is tuff. The railroad tracks in this zone remain normal.

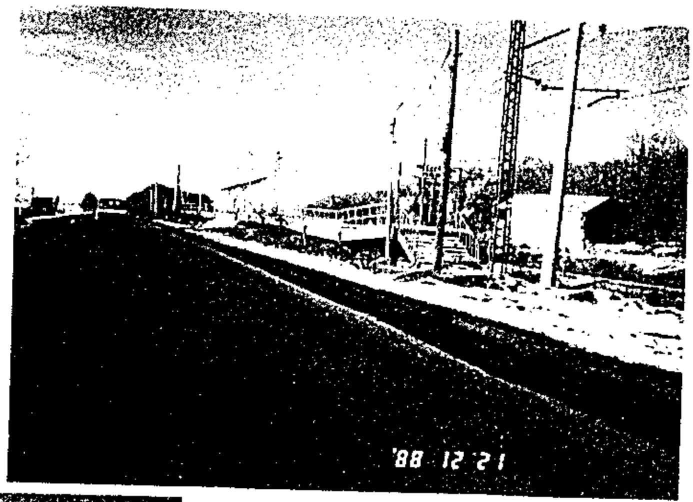
**Photograph 3.1.23**
The left part of a five-storied RMS apartment (remaining right part seen in photograph) in Kirovakan was completely destroyed. It was pulled down and removed. There were many similar apartment houses in this area, but many of them were reported to have suffered only slight damage.

**Photograph 3.1.24**
Stone residences on a hill slope and three-storied stone factory buildings at its foot in the northeast part of Kirovakan. All these buildings appeared to have suffered only slight damage.

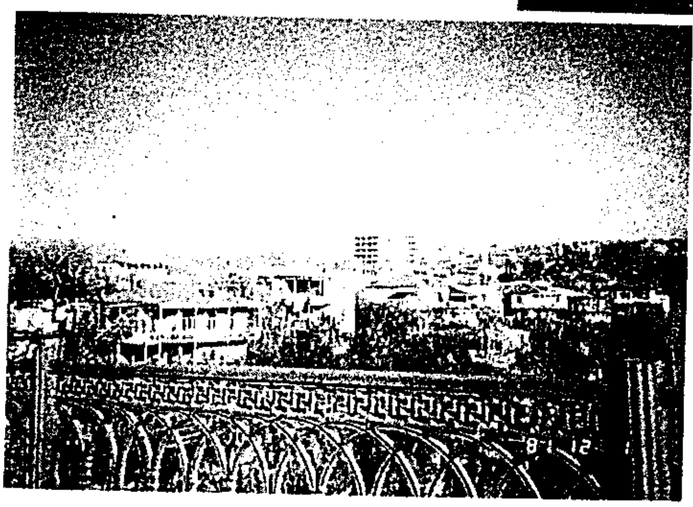




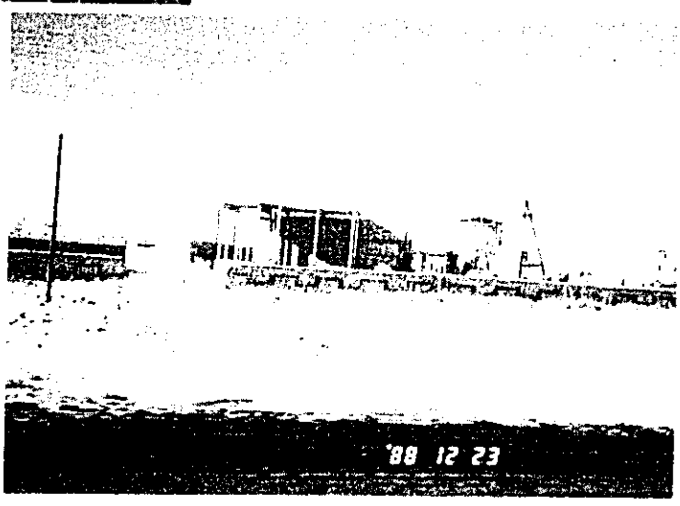
Photograph 3.1.25
Damaged buildings in a small village located about 10km to the west of Spitak (about 10km to the west of the epicenter). There were many one-storied stone houses with wooden roofs, most of which were destroyed completely.



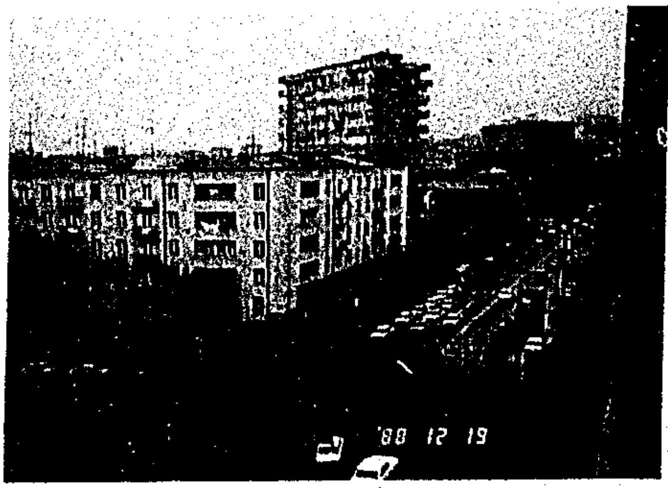
Photograph 3.1.26
A railroad station in a small village about 10km to the west of Spitak. A precast platform was depressed.



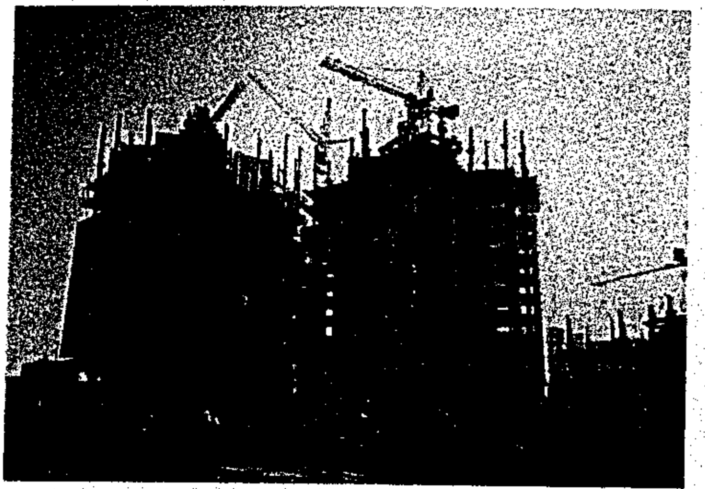
Photograph 3.1.27
A community in Ashtarak located about 15km to the northwest of Yerevan (about 65km to the south of the epicenter). There are one- to two-storied stone-built residences on either side of a valley. They did not suffer damage.



otograph 3.1.28
urb of Maralik located about 25km the south of Leninakan (about 40km the south-southwest of the epicent- ). There were factory buildings mprising thin RPC pillars and one- two-storied stone-built residences, ich seemed to remain undamaged.

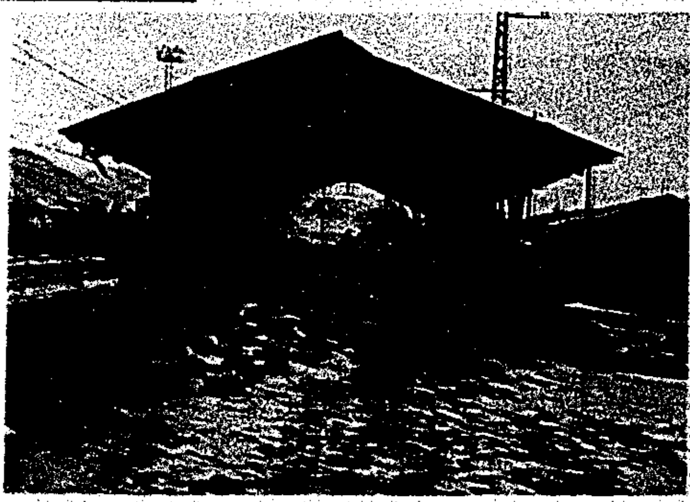



**Photograph 3.1.29**
Buildings in downtown Yerevan, a city with a population of 1,000,000 about 80km to the south of the epicenter. The only one set of strong motion data currently available was obtained on the first floor of the Institute of Architecture located in this city. The maximum acceleration was 62 gals. Yerevan contains a variety of buildings including twelve-storied RPC apartment houses. Most of them seemed to remain undamaged.

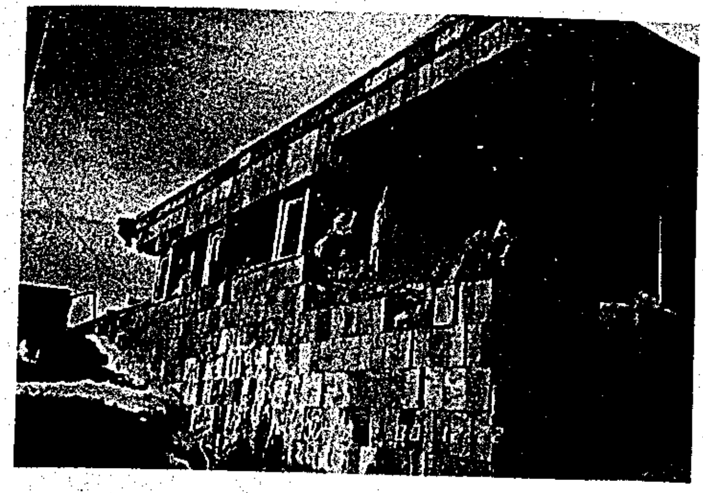


**Photograph 3.1.30**
A large housing development comprising mainly mixed PC/RC high-rise apartment houses, which are under construction in a northern suburb of Yerevan (about 70km to the south of the epicenter). They appeared to remain undamaged.

**Photograph 3.1.31**
Destroyed low resisdences on a hill in the north part of Spitak. There were many unreinforced masonry houses with wooden roof, most of which were completely destroyed.

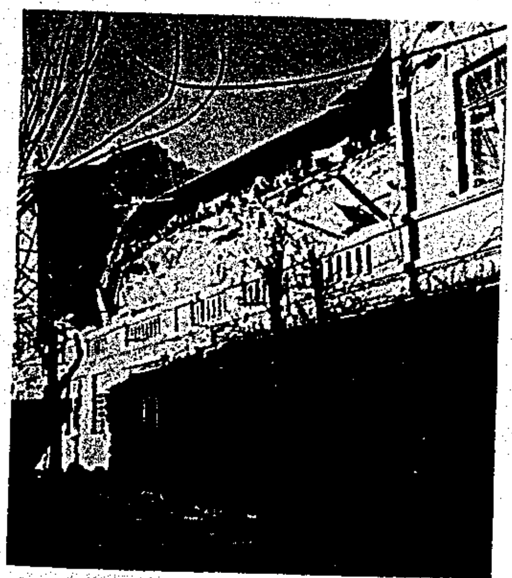


**Photograph 3.1.32**
Railroad station with wooden roof in Spitak. Masonry walls land between pillars of the railroad were destroyed. Gables and corners were also destroyed in the two-storied house behind it. Morter containing clay is used.



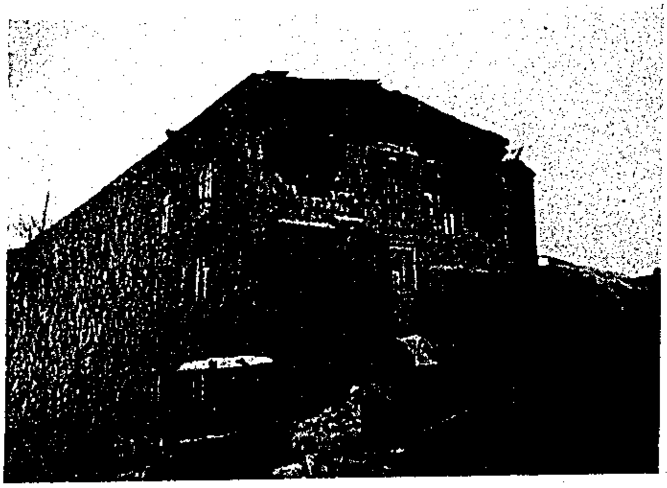
Photograph 3.1.33
A damaged one-storied stone house along railroad slope in Spitak. The walls are of concrete-filled type and with thickness of 30cm. The piers on either side of the window were heavily destroyed.

Photograph 3.1.34
A slightly damaged one-storied stone house in the old section of Leninakan. The wall appears to be of masonry type, judging from the conditions of the joints and lintel. Soil is placed thickly on the ceiling probably for heat insulation and a wood roof is installed over it.

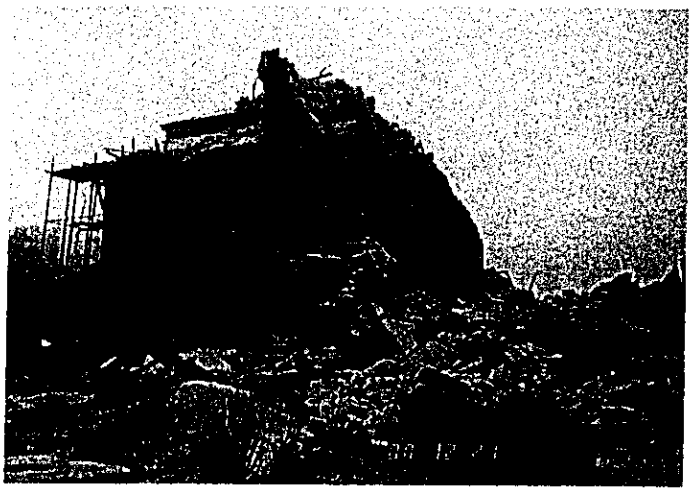
Photograph 3.1.35
Wall material has dropped from the gable at the front porch provided in the longer side of a building. The top of the wall was not reinforced and appeared to have been peeled off by an external force. Similar damages seen in many other buildings. The walls are of filled type and have suffered little damage (Leninakan). The shape of stone material can be seen from the photograph.

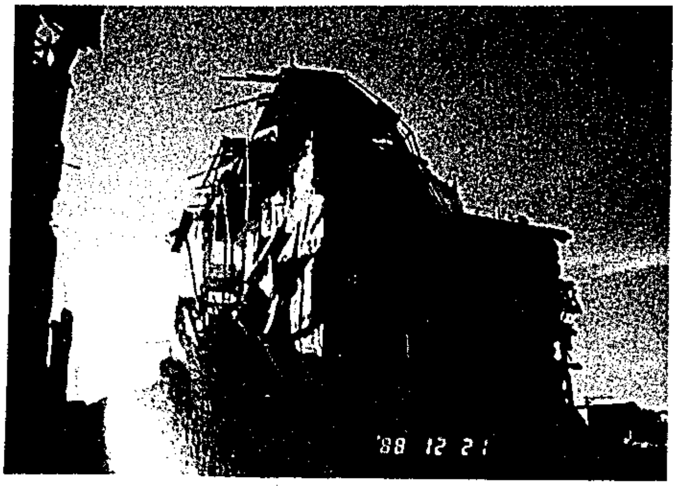
Photograph 3.1.36
Damaged gables at the second and third levels of a three-storied stone-built apartment house. The wall is of filled type, though apparently of masonry type, and the floor is of wood. It is unknown from where the long PC panel has dropped (Leninakan).



Photograph 3.1.37
Damaged gables at the third level of a three-storied filled-type stone-built apartment house. Void slabs are used as floor material in this building, but the roof is of wood. No members such as anchoring bars are seen between the top of the wall at the third level and the horizontal decorative member in the building (Leninakan).

Photograph 3.1.38
A destroyed stone-built church which stood looking towards the Central Square in Leninakan. Many other churches suffered damage including the loss of spires. The roof is of wood and no reinforcing bars are seen in the debris, suggesting that the structure is not reinforced.



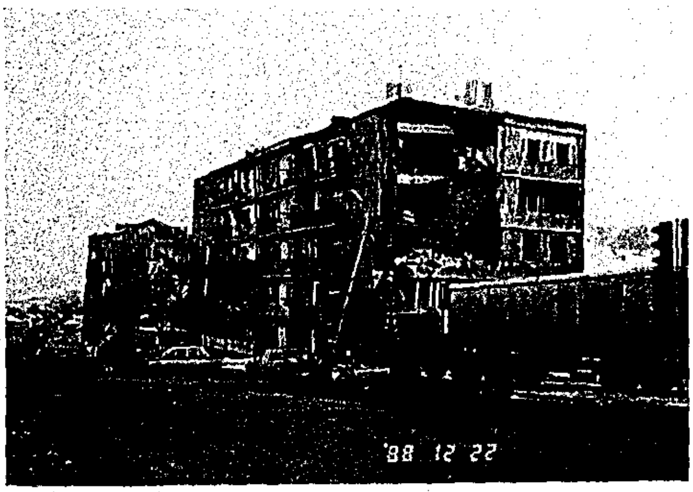


Phot 3.1.39
Heav stroyed five-storied rein-
forc apartment house. The
walls filled type and void slabs
are us floor material. Slabs
are hang rom the fourth and fifth
stories. no beams or reinforcing
pillars a een, suggesting that the
building as a whole does not have an
integrated structure (Spitak).

Photograph 3.1.40
Heavily destroyed five-storied reinforced stone apartment house. The walls appear to be of mixed filled- and masonry-type. The reinforcing bars that have dropped do not have concrete around them, suggesting insufficient fixation (Spitak).

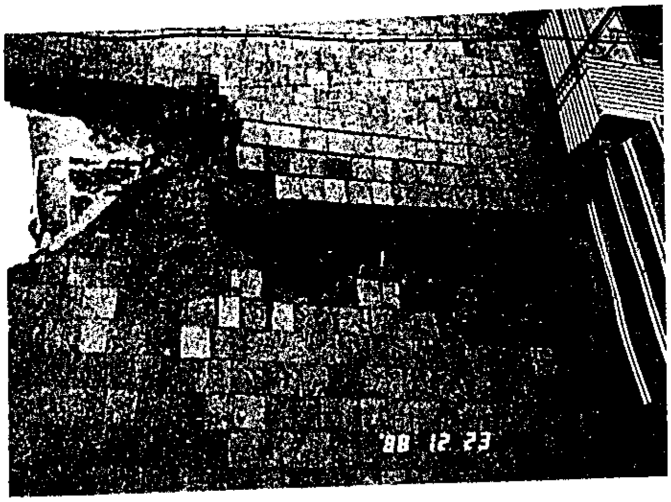
**Photograph 3.1.41**
A four-storied reinforced stone apartment house with damaged gable. This is an old-type building, in which reinforcement is not used in large amounts though the wall volume is large (Leninakan).

**Photograph 3.1.42**
A five-storied apartment house suffering from local damage to intermediate stories. The walls are of filled-type, but no reinforcement is seen in the wall at the edge of the shorter face (Spitak).



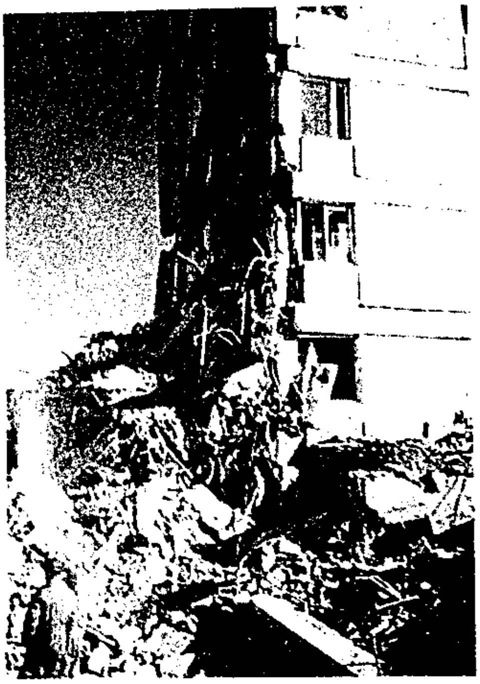

**Photograph 3.1.43**
A filled-type stone-built apartment house suffering from heavy damage to corners of higher stories. In the longer face, walls are provided only at the edges. In the shorter face, the gable has large openings and there no walls of the edges (Spitak).

**Photograph 3.1.44**
A detailed view of the damaged part of the same building as above. Reinforced concrete is seen at the edge of the wall in the longer face. Beams connecting to an RC portion is seen in the fifth story and roof. The slabs used in the building is of precast type though not of void type. Anchoring bars probably used for connection are seen in the shorter side but not in the longer side (Spitak).

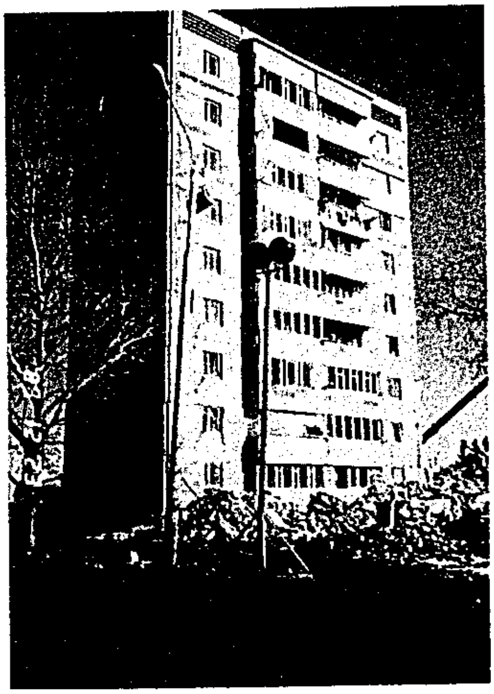



Photograph 3.1.45
A detailed view of a damaged portic
in the same building as in Photogra
3.1.42. Vertical members of reinforc
concrete is seen at two portions
the damaged filled-type stone wal
but the horizontal members probab
provided for reinforcement are n
connected to the vertical ones. O
vertical reinforcing bars and t
horizontal ones are seen between them.

Photograph 3.1.46
Typical damage seen in a housing development comprising nine-storied RPC apartment houses. Most of the apartment houses were destroyed completely, and all of the remaining ones were also damaged heavily. The second and third story of the building in the photograph is so deformed that the portion as a whole appear to be expanded.
A large number of victims still remained undiscovered (Leninakan).



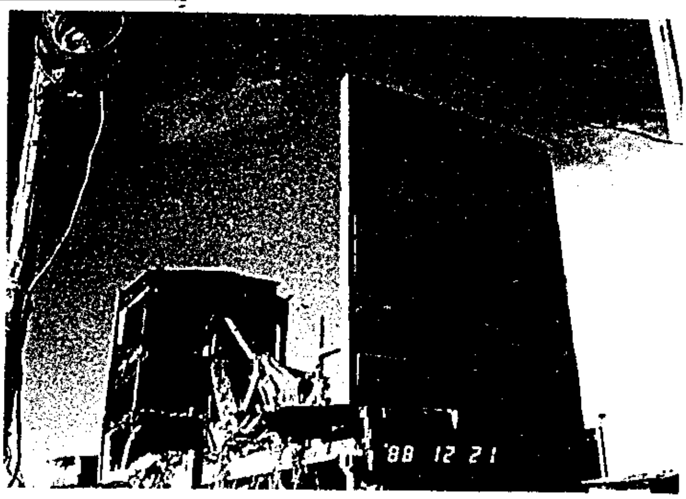
Photograph 3.1.47
A detailed view of the destroyed portion of a nine-storied RPC apartment house suffering from local damage to its central part. Non structural precast members are seen around the building. They did not suffer cracks. PC beam members hang from the building and all void slabs had dropped off (Leninakan).



Photograph 3.1.48
A RPC apartment house narrowly remaining standing. The second and third stories suffered large cracks in the outer wall and damaged to their windowpanes (Leninakan).

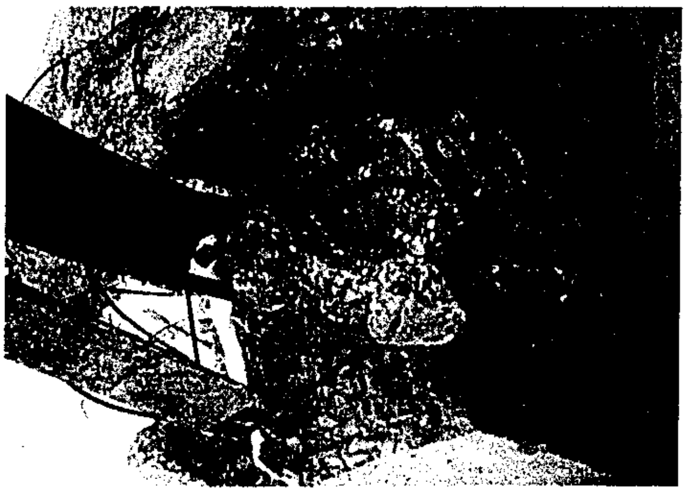
Photograph 3.1.49
An apartment house suffering the destruction of its middle portion, leaving the side portions undestroyed (overall view of the building in Photograph 3.1.47). As a rare case, the building was televised in a local TV program. The portions that appear to be pillars, beams or walls are broken sections of finishing material provided over structural members. The building is damaged in the span direction particularly in the second story. The four-storied building behind appear to be free of damage (Leninakan).

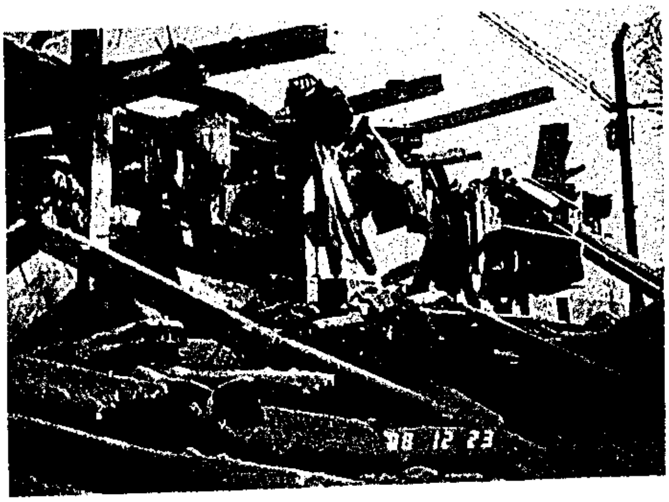
Photograph 3.1.50
A five-storied RPC building suffering the destruction of its middle portion, leaving only the gables undestroyed. The shape of the gable wall suggest that the building was not designed for residential use. Part of the front portion is smoke-stained probably due to fire (Spitak).




Photograph 3.1.51
Precast pillar members found in completely destroyed nine-storied RPC apartment house. Angles and plates embedded for connecting reinforcing bars in beams are seen in the center of the photograph. Each main bar arranged at the four corners of the pillars is welded by using a 30cm-long auxiliary reinforcing bar. The reinforcing bars are about 7mm in diameter though the hoop spacing is small.

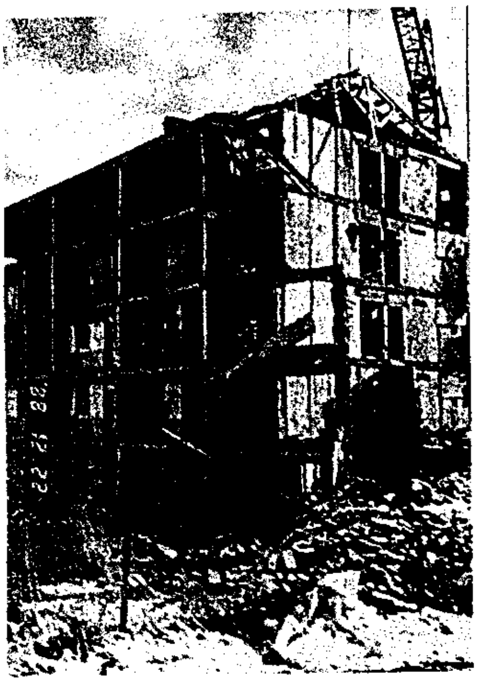
ɔtogtraph3.1.52
end of a precast beam member found completely destroyed nine-storied apartment house. A slippage is n in the welded portion between bottom bar in the beam and an angle hored to a pillar. A U-shaped plate used for connection of the top bars. aggregate grains are broken in e cross section of the ruptured conete. The stirrup is about 7mm in diameter. The cross section of the beam has a convex shape to accept floor plates on it, but there is no evidence to show that the reinforcing bars in slabs were used for anchorage.



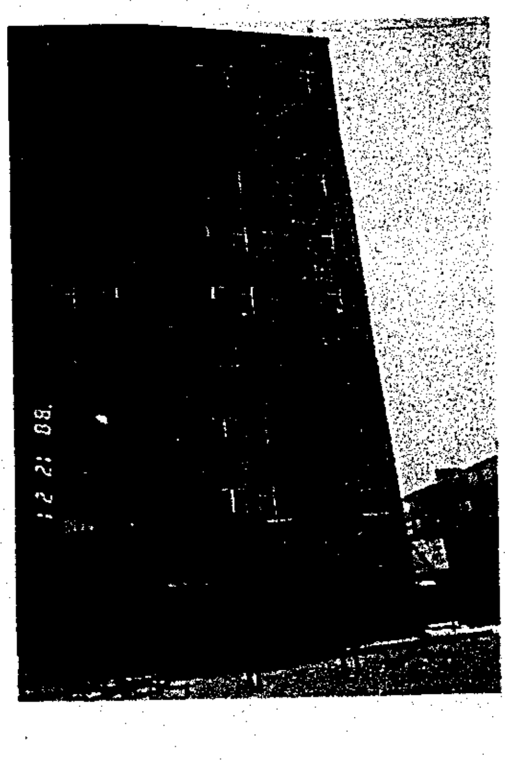


Photograph 3.1.53
An apartment house destroyed during construction. Some precast pillar members have fallen down in the same direction. Precast exterior material is attached to the outer faces of structural members. The building is estimated to have been a nine-storied one, but the cross-sectional dimensions of the pillar members are 40cm×40cm even in the first story (Leninakan).

Photograph 3.1.54
An RPC structure damaged during construction. This structure may have been designed to have more than five stories, judging from the comparison of the pillars with those seen in five-storied apartments in Spitak, though the cross-sectional dimensions of the pillars were about 40cm×40cm. Beams are provided only in one direction while in the perpendicular direction, floor slabs are connected to pillars by using bars. Pillars are connected in every three stories, that is, in the first and fourth story.



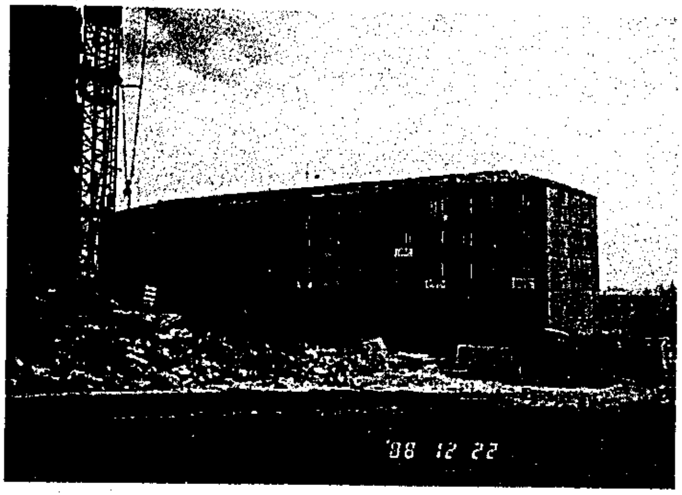
Photograph 3.1.55
A five-storied precast apartment house damaged during construction. It has beams in the long direction, but no PC pillars. A small PC wall panels are provided for each span, and the other parts are made of masonry walls. No beams are provided in the span direction, and many small PC wall panels are used instead. PC truss material is also used in the roof (Spitak).

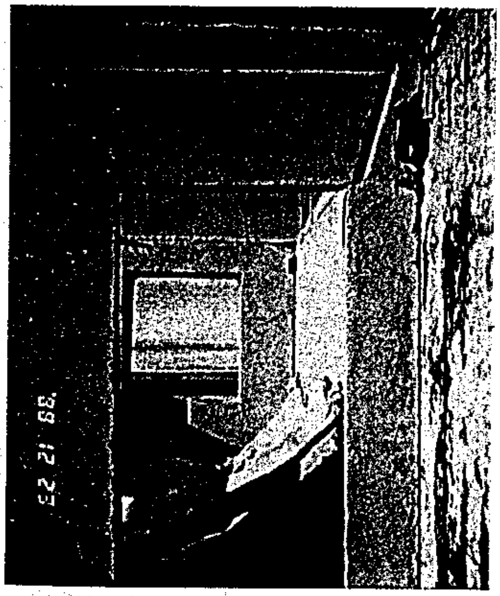


Photograph 3.1.56
An undamaged nine-storied RPC apartment house in Kirovakan. This building did not suffer even a crack in the outer walls though it was reported that 70 apartment houses of similar construction were damaged in Kirovakan.

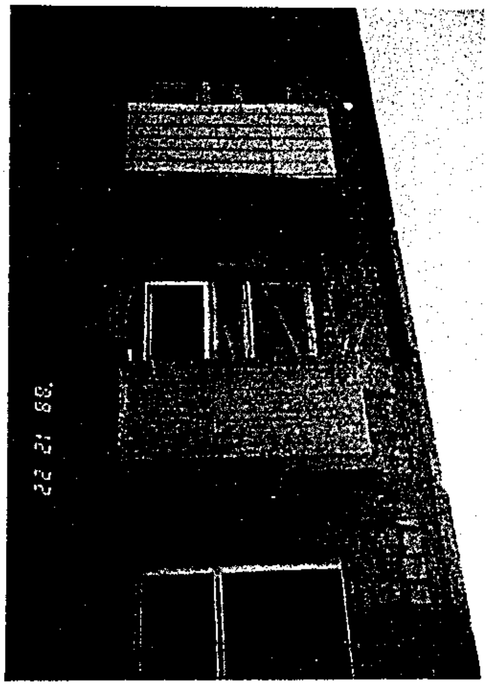
Photograph 3.1.57
2 five-storied RPC apartment houses built near the construction site shown in Photograph 3.1.53. Though many walls are installed, pillars and beam forms are seen, which is a key feature indicating that these are not of WPC construction. No significant damage is seen except that, on the building in the foreground, the first-story portion of the gable appears to be deformed.

Photograph 3.1.58
An almost undamaged WPC apartment house suffering only the loss of some outer wall panels which stood among many completely destroyed five-storied RPC apartment houses in Spitak. The absence of pillars and beam forms is the key feature indicating that it is of WPC construction.

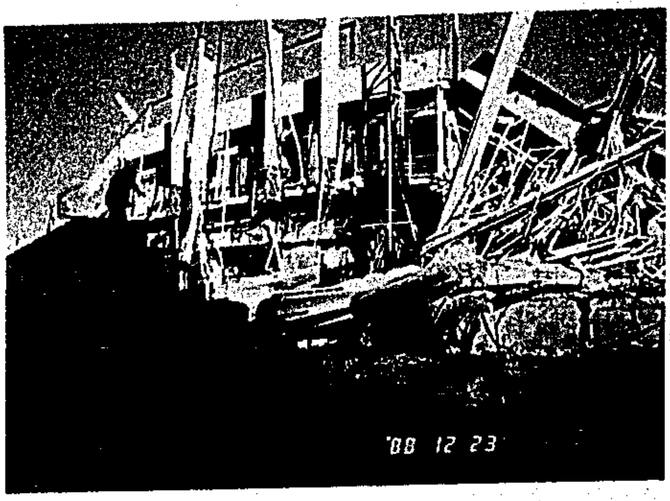




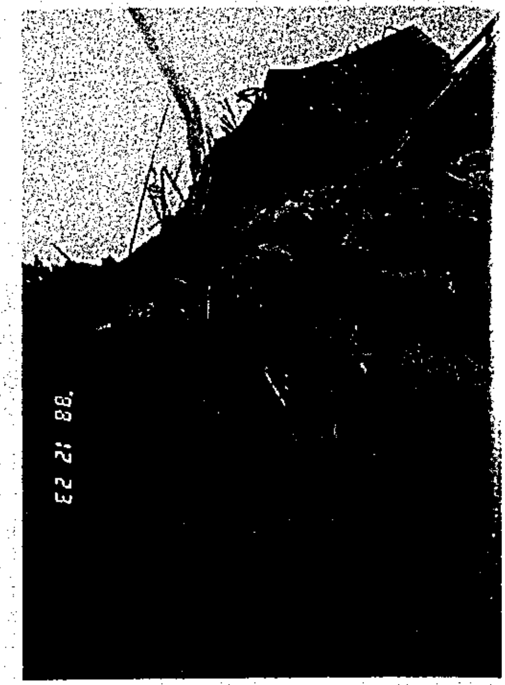
Photograph 3.1.59
Floor panels that have fractured and dropped from the eighth floor to the seventh floor in the nine-storied WPC apartment house under construction shown previously in Photograph 3.1.15. It is unknown whether the connection of floor panels had been completed when they fell in the earthquake. However, other panels and connecting portions suffered little damage (joint concrete had not been laid in the seventh story (Leninakan).



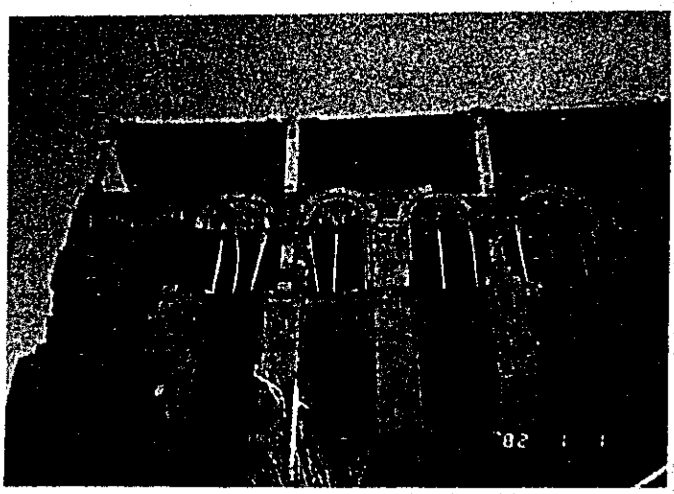
Photograph 3.1.60
Diagonal cracks formed in the outer wall panels on the five-storied WPC apartment house shown previously in Photograph 3.1.58. Cracks are seen in the wall on the right of the wall panels at the second-story level, but other panels and joints did not suffer cracking (Spitak).




Photograph 3.1.61
A completely destroyed four-storied RPC department store. The front part originally had three stories. The second story was destroyed and the third floor dropped (Leninakan).



Photograph 3.1.62
A side view of a destroyed four-storied RPC department house. The store was so completely destroyed that its original form could not be estimated (Leninakan).

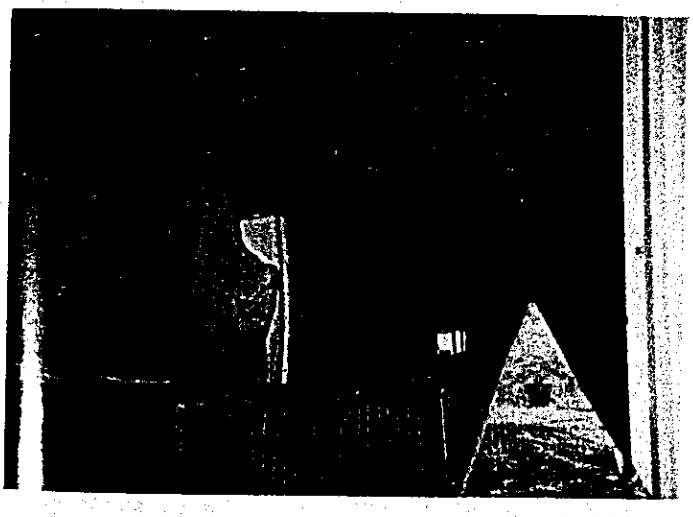


Photograph 3.1.63
An apparently RPC office building probably under construction. The outer walls have been finished in a masonry appearance. The stone exterior is damaged heavily. The framework appear to have suffered little damage because of a low rigidity (Leninakan).

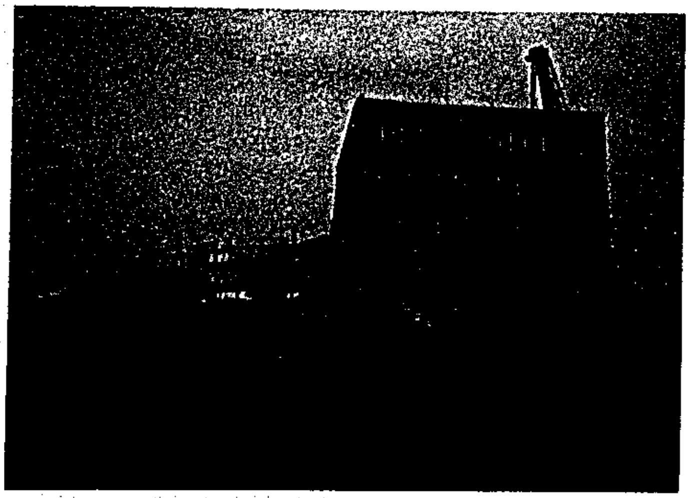
Photograph 3.1.64
A one-storied laboratory building comprising precast pillars trusses. Braces were used in the end span, but their rigidity was low and all masonry outer walls collapsed.

Photograph 3.1.65
A three-storied factory building comprising precast pillars and beams and ribbed slabs. Stone outer walls at the second and third story levels, where the amplitude was large, were peeled off, though most of the other members including pillars, floors and roof slabs remained undamaged.

Photograph 3.1.66
The lower part of a sixteen-storied apartment house, which is the highest building in Leninakan. The core part is constructed of cast-in-place rainforced concrete, and the floor slabs, internal pillars and 12 external pillars at the four corners are of precast reinforced concrete. Pillars did not suffer significant damage though outer walls and partitioning walls were slightly damaged.

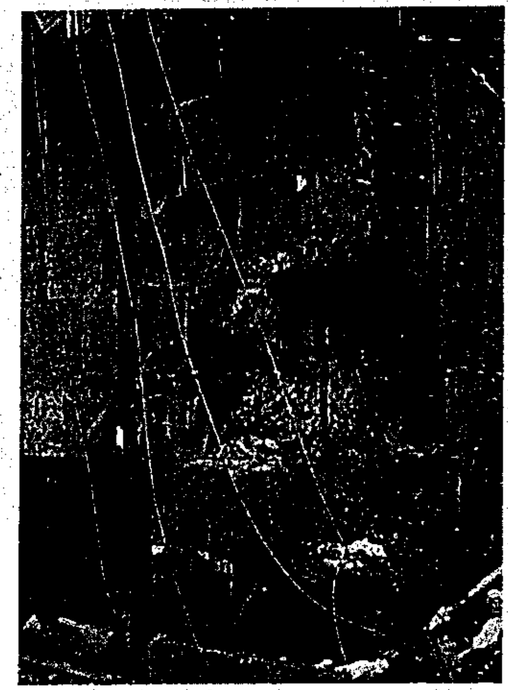


Photograph 3.1.67
Damaged cast-in-place RC core wall at the center of the first story in a sixteen-storied apartment of mixed RPC and RC construction. The wall is heavily damaged and the vertical bars are buckled largely. The partitioning wall is also deformed showing a reduction in the height of the story, though the two inner pillars suffered no cracks.



Photograph 3.1.68
A largely deformed six-storied apparently office building standing in the old section of Leninakan. Its construction type cannot be identified from the external view.

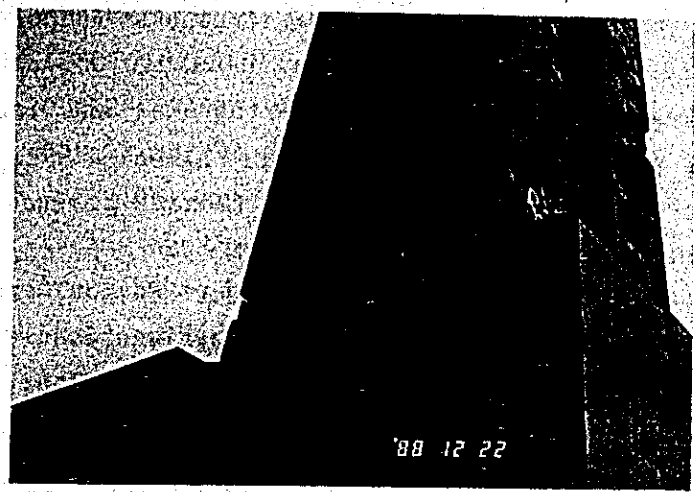
Photograph 3.1.69
When viewed at close range, many window-panes are broken and vertical members which appear to be reinforced concrete pillard suffered large cracks.



Photograph 3.1.70
Reinforcing members of reinforced concrete are seen in the stone wall at a corner of the building. In addition to this, there are many buildings which do not appear to be constructed to stone but comprise no apparent RC members.

Photograph 3.1.71
A damaged building which appear to be of similar construction to the ones in the previous three photographs. Reinforcing members of reinforced concrete are seen in the destroyed stone pillar. Windows are large and the wall volume is small.

Photograph 3.1.72
A gas station of RC construction in Spitak.




Photograph 3.1.73
A damaged reinforced stone monument on a small hill in the south part of Spitak.

Photograph 3.1.74
A graveyard at the foot of a hill in the south part of Spitak. Thin gravestones have not fallen down probably because their bottom parts were embedded in the ground.

## 3.2 Damage to Road and Railroad

(1) General

Fig. 1.2.1 shows roads and railroads in the suburbs of the cities of Spitak (population about 20,000), Leninakan (population about 290,000) and Kirovakan (population about 170,000), which were heavily damaged by the latest earthquake, and Yerevan (population about 1,000,000), the capital of Armenian SSR. The railroads must have been classified by type (managing agency) and degree of importance though details are unknown.

We did not have the chance to contact Soviet experts specializing in transportation facilities partly because the Japanese expert team had little time for a survey. Thus, the following description on damaged structures is based only on the short field survey by the expert team.

(2) Damage to Road

1) Outline

The total length of the roads within the earthquake-stricken areas is about 600km, of which sections totaling about 60km are reported to have suffered heavy damage. Since all these stricken areas are located in a mountenous district at altitudes of 1,500m or more, the major mishaps included collapse of slopes, cracks in road surface and sharp drop in road level. In urban areas, buildings were brought down, blocking up roads. As of February 21, on which the expert team arrived, the roads had resumed their functions nearly completely because an army was mobilized specially for the repair of roads, though further restoration work was still needed. Slight damage to roads was seen in many places, but there seemed to be no heavy destruction that would cause long-term traffic interruption, though details were unknown. At least, heavy damage to the roads running through Yerevan-Leninakan-Spitak-Kirovakan and Yerevan-Spitak were not found during the survey by the expert team.

2) Damage to Road Bridge

It is reported that the earthquake-stricken areas contained about 140 bridges, mostly of small-scale, of which 7 suffered slight damage.

Photograph 3.2.1 shows an elevated bridge on a bypass currently under construction in a suburb of Spitak, the city closest to the epicenter. It consists of a three-span, simple-support, pretentioned girder supported on a RC frame. The lower structure consists of precast cross beams connected to precast piers at the site. The type of foundation is not known, but it may be of direct type.

The top and base portions of the piers suffered damage including the peeling of concrete as shown in Fig. 3.2.2. The cross beams have been slightly rotated and some piers have been inclined towards the bridge axis. Judging from Photograph 3.2.2.(b), the main bars, which are of deformed type, are about 25mm in diameter, and the hoops (deformed reinforcing bars) of about 9mm in diameter are provided at intervals of about 30cm. Ordinary type aggregate, rather than tuff, is used, but its strength appears to be low (roughly estimated at around 250kg). Compared to other bridges described later, this one obviously has a smaller cross section and there seems to be a large difference in design intensity.

Photograph 3.2.3 shows a five-span, simple-girder bridge (effective span 33m, 5- 165m) installed in Leninakan. Its upper structure consists of two series of six PC T-girder beams provided at different heights while the lower structure consists of cross beams installed on two RC piers. The structure of the foundation is unknown, but it may be of direct type since the ground is made of tuff. The bridge is firmly constructed and very largely different from the above-mentioned one on the bypass in suburban Spitak with respect to the cross section of piers and quality of concrete finishing. Information on the intensity of the concrete used was not available for most bridges, but we heard at a construction site in Yerevan that the intensity adopted there was 400kg/cm.

In the bridge in question, a residual displacement parallel and perpendicular to the bridge axis as shown in Photograph 3.2.4 was caused between the second and third span from the left bank. The damage, however, is so small that it would not have a significant effect on the performance of the bridge. A sixteen-storied building standing on the left bank was heavily damaged and ready to collapse, clearly indicating the difference in earthquake resistance between the bridge and building.

Photograph 3.2.5 depicts the piers (undamaged) for an elevated bridge being constructed on a bypass. It has 14 spans and its pier height is considerably large, 50m maximum. The span is approximately 30m. All piers and part of the cross beams had just been installed and become ready to accept some girders when the earthquake took place. The piers were constructed by connecting precast members at the site. Concrete had not been laid on the connecting portions, but the piers were fortunately free from damage because the upper structure had not been installed.

3) Damage to Road Banking, Road Surface, etc.

Photograph 3.2.6 shows a cracked road surface found in suburban Spitak. The road runs along the edge of a river terrace, and the cracks are considered to have resulted from the slippage of banking. Photograph 3.2.7 depicts cracks and fallen rocks on a bypass under construction in Spitak. Damage to a culvert as shown in Photograph 3.2.8 was also found on the same bypass. However, the number of damaged portions such as shown in Photographs 3.2.6-3.2.8 was not large for most roads.

4) In Photograph 3.2.9. destroyed roadside buildings blocked up a road or the entrance of an underpass. In particular, the blocking of roads by destroyed buildings had serious effects on the traffic. In some places, traffic was regulated for fear of the collapse of a damaged building even if it remained standing.

Photograph 3.2.10 shows a truck recovered from rubble after being buried under a fallen building. In particular, removal of rubble on trucks caused serious traffic congestion in central parts of urban areas.

(3) Damage to Railroad

1) Outline

The main line connecting Leninakan-Spitak-Kirovakan had been reopened by December 21, when the expert team arrived, though it was reported that railroad service had been suspended in many places imediately after the earthquake.

2) Railroad Bridge

Photograph 3.2.12 shows a railroad bridge that collapsed in suburban Spitak. This seemed to be a spur track connecting the Spitak Station and factories in the city, rather than the main line between Spitak and Kirovakan. The two-span simple deck bridge has lost a girder (span about 30m) close to the left bank. According to other information, however, it was not that the girder dropped, but it was pulled down for replacement after suffering heavy damage. Details were unknown. Photograph 3.2.13 depicts the bridge under repair. Restoration work is under way using a steel girder which seems to have been procured hastily. Near the left bank, the abutment is being restored using tuff blocks to suit the length of the girder.

Photographs 3.2.14 and 3.2.15 show railroad bridges (both undamaged) in Leninakan and suburban Yerevan, respectively. The former, in particular, is supported by very firm piers. Its design horizontal intensity is estimated at 0.2 or more.

3) Tunnel

A tunnel is suburban Spitak was found to suffer cracks at its mouth as shown in Photograph 3.2.16. We did not have time to check the conditions within the tunnel, but railroad service had been reopened by December 21, suggesting that it did not suffer serious damage.

4) Banking

Photograph 3.2.17, taken by a Soviet expert immediately after the earthquake, shows destroyed banking. Its restoration seemed to have completed when the expert team arrived, and its location could not be identified. Judging from the photograph, slippage appeared to have taken place not only in the banking but also in the ground around it.

Photograph 3.2.18 depicts damaged small banking found in the yard of the Spitak Station.

5) Other Damage

Photograph 3.2.19 shows a fallen concrete utility pole seen in suburban Spitak and a temporal pole used for restoration.

(4) Summary

Sufficient data are not available because we did not have the chance to contact Soviet experts specializing in traffic facilities. According to the survey by the Japanese expert group, however, damage to roads and railroads seem to be generally small as compared to buildings. At the present stage, major factors in this difference are considered to be as follows, though further studies are required with respect to design and construction.

1) Large-scale damage did not take place because there were few large structures in the earthquake-stricken area.

2) The ground is generally in good condition in most parts of the earthquake-stricken area since it is in a mountainous district. Consequently, the area did not suffer large-scale destruction of banking and slopes caused by the deterioration in the ground condition.

3) Depending on the degree of importance of the road or railroad, small-scale bridges generally have a short natural period and therefore, their design horizontal intensity is not small in most cases as compared to buildings that suffered heavy damage. Most bridges are made of concrete with considerably high strength (400kg/cm in some cases) and finished fairly well. This feature indicates a large difference from heavily damaged buildings.

## 3.3 Damage to Lifeline Facility

Sufficient information has not been obtained concerning the current conditions and damage to lifelines (including damage to management system) or emergency restoration, because we did not have the chance to contact Soviet experts specializing in lifeline facilities. Summarized below are data on damage, though fragmentary, obtained from interviews at the Institute of Physics and Engineering Seismology and the short field survey by the Japanese expert team.

Damage to lifeline facilities identified in Leninakan by December 21 is as follows.

- Electric power, water supply

  These facilities suffered considerable damage, but have been restored gradually.

- Gas supply, sewerage

  These facilities must have suffered large damage, but surveys have not been made sufficiently. Further surveys are needed. Unrestored.

Gas supply facilities are provided with cutoff valves that automatically stops the gas supply when detecting vibrations, and these valves are said to have been actuated.

Photograph 3.3.1 shows pipelines (undamaged) crossing a road in suburban Leninakan (found immediately before entering the earthquake-stricken area). Similar pipelines were seen in many places. They are considered to be used for irrigation or tap water supply, though details are unknown. In places other than roads, pipelines are installed directly on the ground surface or on a shallow level in the ground.

Photograph 3.3.2 depicts a damaged weld in a steel pipe and a bent fitting found on a bypass under construction in suburban Spitak.

Photograph 3.3.3 shows a cutoff valve removed as a temporary measure to stop the leak from a damaged fitting in a water supply pipe in Leninakan.

Photograph 3.3.4 shows a culvert being built near a building under construction in Leninakan, and a steel pipe installed in it. The use of the pipe is unknown. It seemed that the pipe laid in the culvert would be covered with reinforced concrete lids, followed by back filling.

Photograph 3.3.5 depicts underground cables being restored. They appeared to be cables for control of traffic lights. They are passed through steel pipes and buried on a shallow level under the pavement.

Photograph 3.3.6 shows tanks in an industrial complex in Lininakan. Whether they were damaged is unknown.

## 3.4 Damage to Ground

The earthquake-stricken area is located in a mountainous district at altitudes of 1,500m-1,700m. The ground consists of weathered tuff and basalt, consequently, the ground contains clay as the principal component and the layer of deposits is thin, which appears to be the major reason for the fact that the area did not suffer significant damage due to rupture of the ground by fluidization, etc. Another possible reason is that the area is in a dry district with no precipitation before the earthquake, and the ground contained little water and was high in strength.

### (1) Spitak and Its Suburbs

Railroad banking on a dry riverbed in Spitak suffered large-scale collapse, heavily deforming the rails (Photograph 3.4.1). Large-scale slippage and destruction took place in the middle of the banking, suggesting that the slippage resulted from a sharp reduction in supporting force of the ground caused by fluidization. Destroyed masonry was found on the Pamback River in western Spitak (Photograph 3.4.2). In the north of the west part of the city, large rocks were scattered on a road after falling from the top of a precipice (Photograph 3.4.3). Many cracks had been formed in a road running in the east-west direction in the north part of the city, and partial destruction and depression were seen in many places.

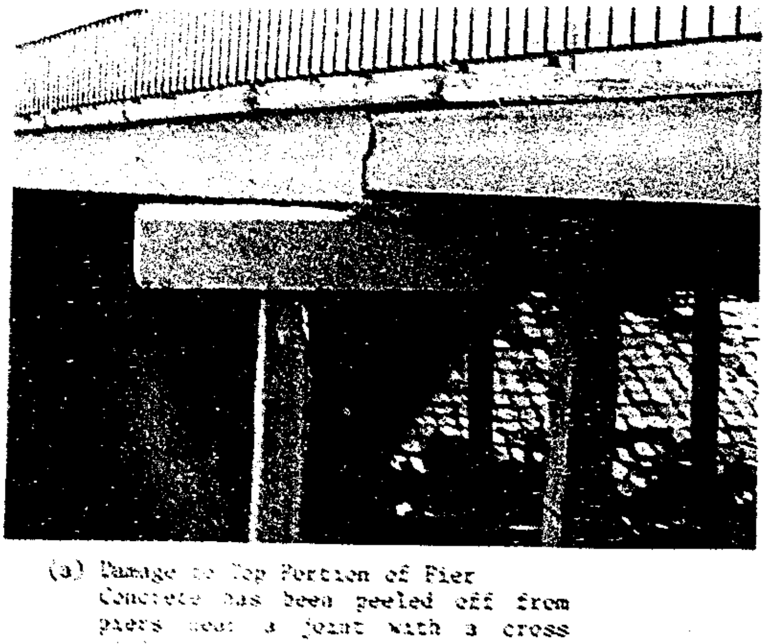
The intensity was X in the area 5-6km to the west of Spitak, where roads and banking were reported to have suffered heavy damage, though details are unknown. We found small-scale damage to roadside slopes in many places. Electric poles on the dry riverbed of the Pamback River were inclined, which also appeared to have caused by fluidization of the ground.

### (2) Leninakan

The ground appeared to be free of damage, probably because land was generally flat and the groundwater level was low. Detailed information is not available for the south part of the city because we did not have the chance for a survey there.

Leninakan appears to be located on a thick lacustrine clay layer of 250-300m maximum, as described under Section 4.2. This clay layer is largely overconsolidated, and as seen from Table 4.2.1, the shaer wave propagation velocity ($V_s$) is a considerably high 450m/sec. There is the possibility, however, that the clay layer worked to amplify long-period input waves, suggesting that waves with periods of 1-2 sec might have prevailed to some extent. If so, it may become possible to explain the collapse of many nine-storied apartment houses. Anyway, more detailed studies are required in the future.

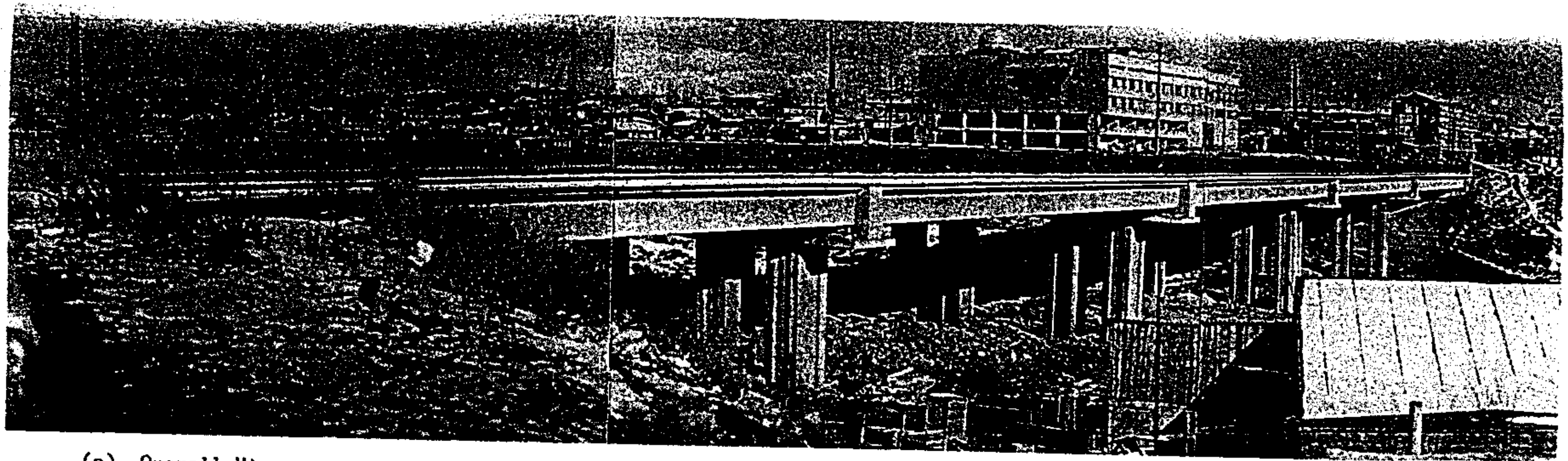
Photograph 3.2.1: Elevated Bridge under Construction (Spitak)
Cross beams are installed on precast piers to form a frame, on which pretentioned girders are laid. Some of the piers on the left are inclined irregularly. Of the bridges surveyed, this one had the smallest cross section.



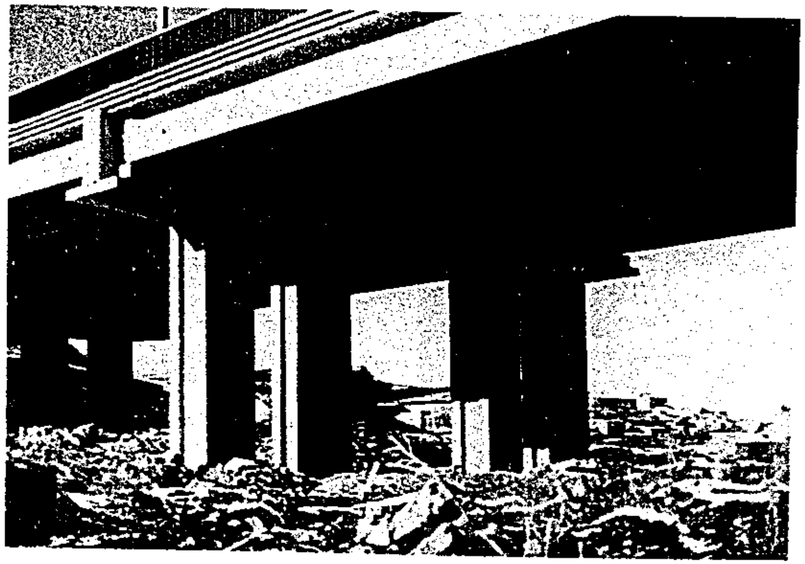
(a) Damage to Top Portion of Pier
Concrete has been peeled off from piers near a joint with a cross girder.

(b) Damage to Base Portion of Pier
Protective concrete has come off and the main bars has been slightly bent. The concrete is low in strength and finishing is inadequate.

Photograph 3.2.2: Damaged Pier (Spitak)



(a) Overall View
The five-span simple bridge is almost free of damage except for displacement of about 7cm perpendicular to the bridge axis.



(b) Pier and Support
The design horizontal intensity may be least about 0.2 judging from the cross section of the piers. The piers and supports are free of damage.

Photograph 3.2.3: Bridge across River (undamaged, Leninakan)

(a) Displacement of Main Girder Perpendicular to Bridge Axis (about 7cm)

(b) Torn Handrail
Particular joints are not provided at the end of the girder. The pavement is laid continuously. The handrail is pulled off by about 10cm, probably reflecting a residual displacement of the girder.

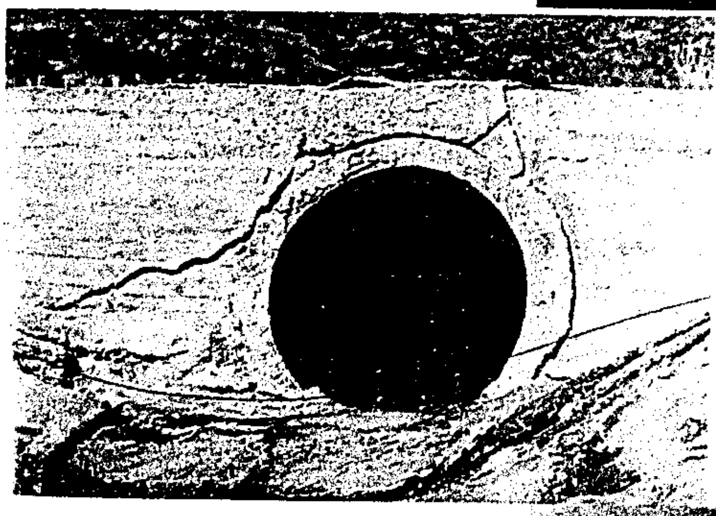
Photograph 3.2.4: Damage from Displacement of Main Girder (Leninakan)

This large-scale bridge is planned to have 14 spans and to be about 40-50m in length and about 40m in pier height. The precast pier members (hollow) seen at the left bottom of the photograph will be connected to form piers. Cross beams will be placed on each pair of piers (some cross beams have already been installed on the piers at the farther end), followed by the installation of girders on them. Proper connection of precast piers is of key importance. The piers seem to have suffered no damage.

Photograph 3.2.5
Road Bridge under Construction

**Photograph 3.2.6:** Crack and Fallen Rock on Banking of Road under Construction (Spitak)
The road is under construction and the slope is not protected. Rocks have fallen from a height of 50m or more from the road surface. The crack in the road surface was caused by slippage in the banking. The slope of the banking is formed to the right of the portion in the photograph.

**Photograph 3.2.7:** Crack in Road Surface (Spitak)
A crack has been formed due to slippage in the banking. The number of roads suffering from cracks like this was not large within the scope of the survey.



(a) Mouth

Photograph 3.2.8 ↑ →

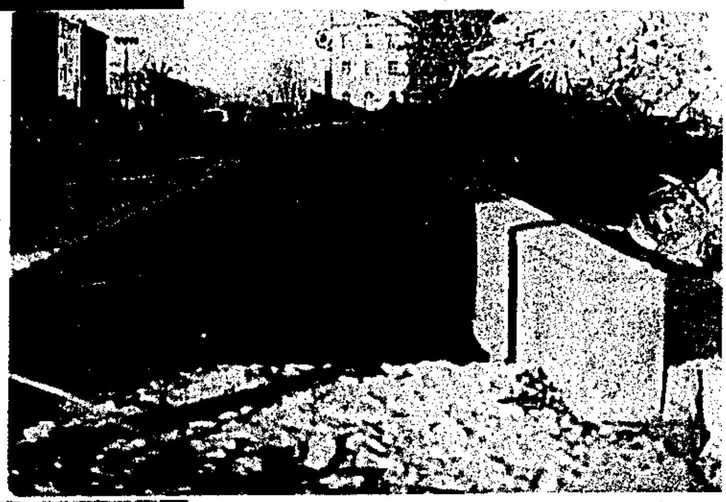
**P tograph 3.2.8:** Damaged Culvert under R d (Spitak)
T s culvert was installed through t banking of the road under construc- t n cited in Photograph 3.2.6. The s ppage in the banking caused a gap b ween the culvert and the breast- w l.

(b) Inside

(a) Building Torn Down onto Road
Many roads in the city were blocked by destroyed buildings. Damage to roadside facilities should be taken into account in performing restoration work for the roads.

Photogtraph 3.2.9 ↑ →

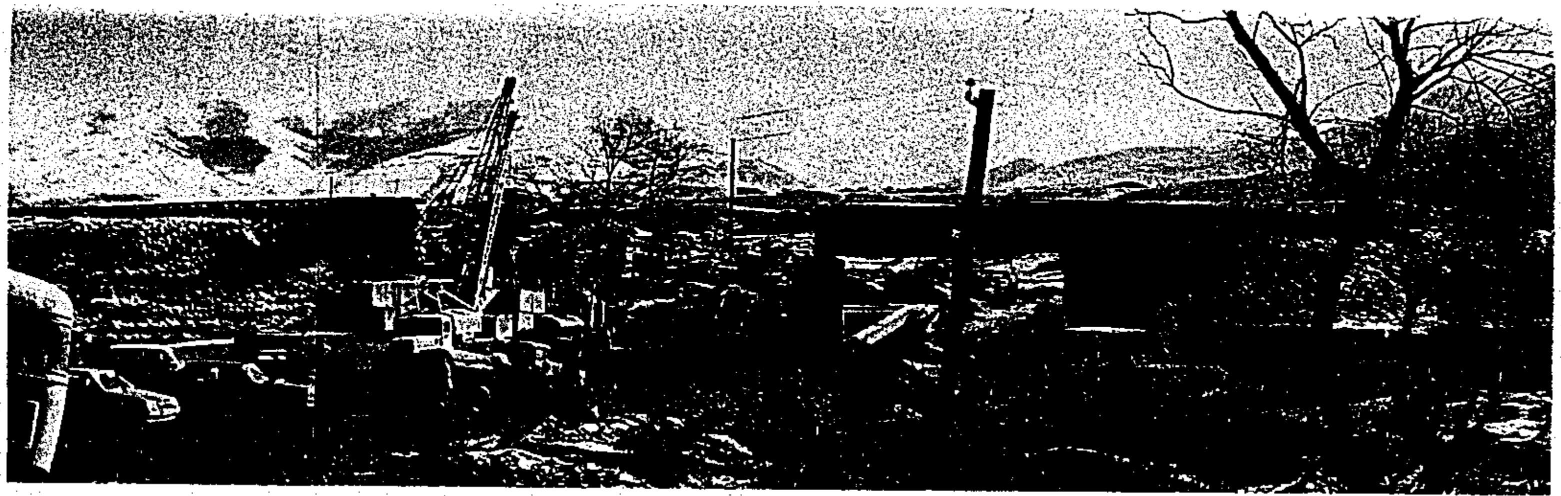
Photograph 3.2.9: Road Blocked by Destroyed Roadside Building



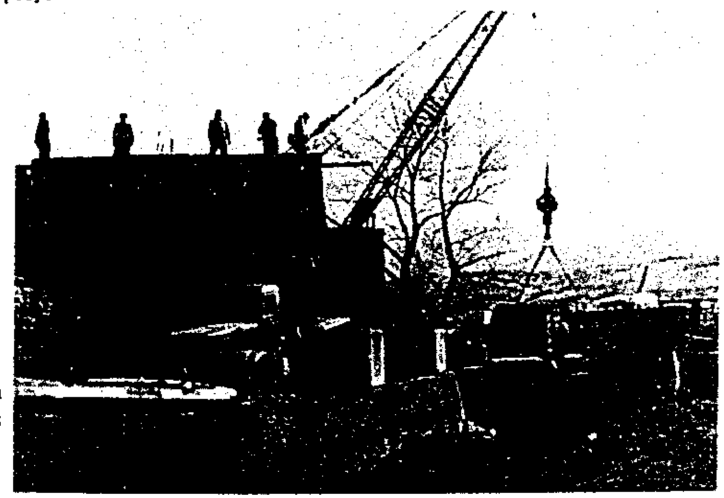
(b) The mouth of an underpass is blocked by rubble of a destroyed building.

Photograph 3.2.10: Truck Recovered from Rubble of Destroyed Roadside Building (Spitak)

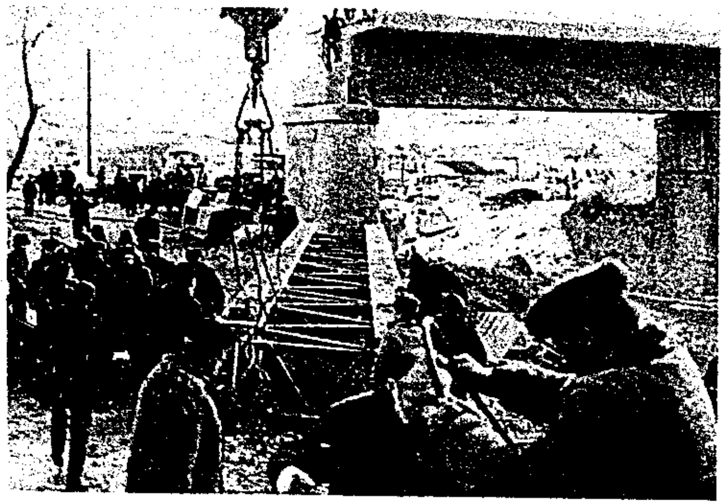
Photograph 3.2.11: Removal of Rubble (Spitak)
It is extremely difficult to remove and transport debris with many projected reinforcing bars, though an easier case is shown in the photograph. There were a great number of rubble heaps. Removal and disposal of the rubble will be an important matter.



Photograph 3.2.12: Damaged Railroad Bridge
The left part of the two-span bridge has dropped while the right girder did not suffer damage. It was also reported that the girder was simply being replaced after being heavily damaged, instead of dropping in the earthquake. The cause of the damage is unknown. The piers in the central portion do not suffer from peeled concrete.

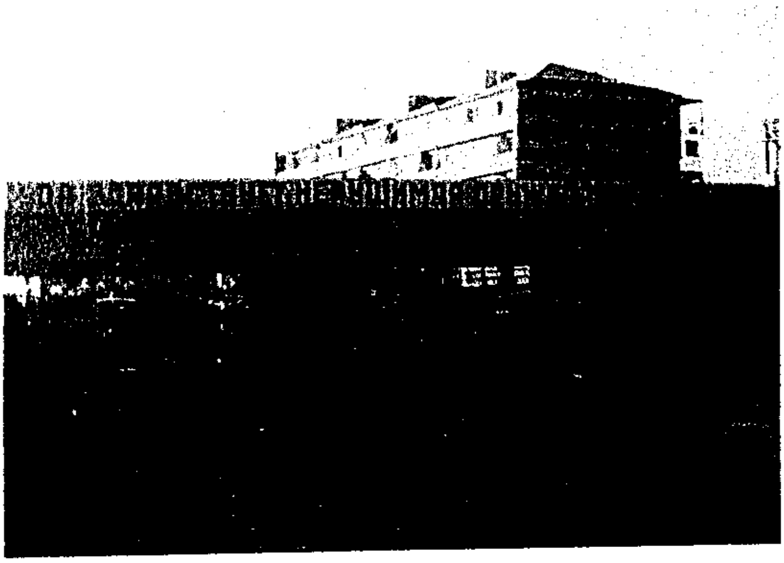


(a) Steel Girder for Restroation
Steel girders are procured and restoration work is performed by placing blocks on the piers to girder height. Steel girders arrived only in two days after the taking of Photograph 3.2.12, suggesting that the restoration work was performed very promptly.



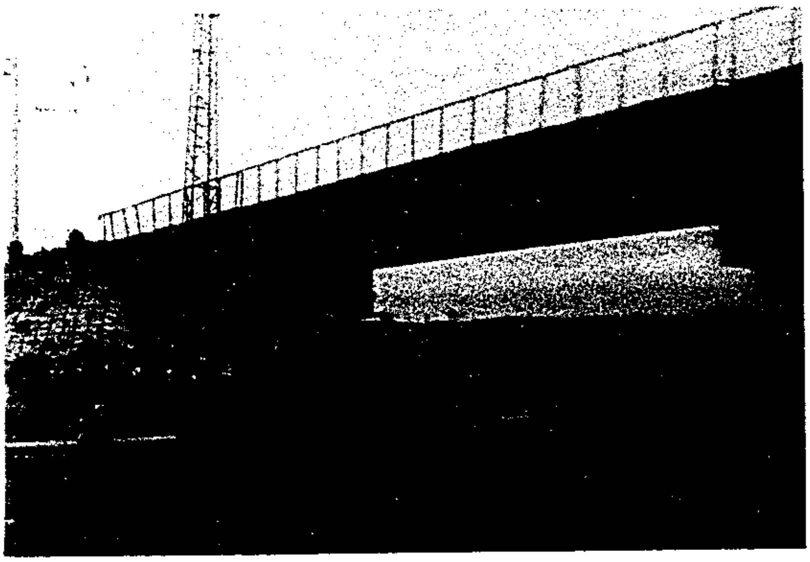
(b) Restoration of Abutment: Bank
For restoration, tuff block laid to length and height of the girders that have been procured hastily.

Photograph 3.2.13: Restoration Work (Spitak)

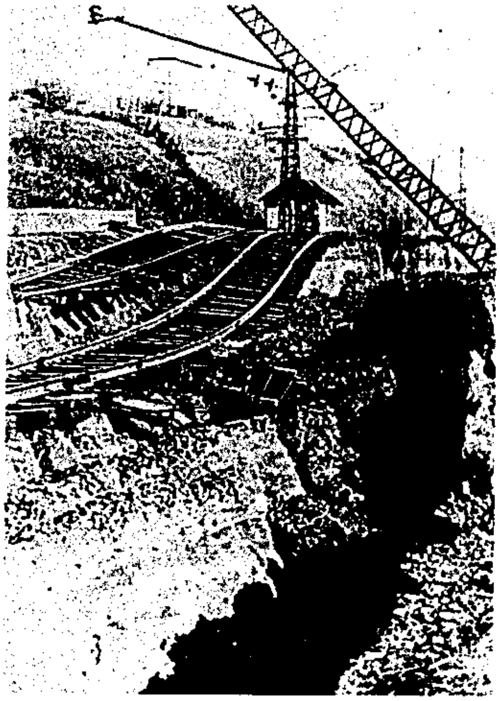


**Photograph 3.2.14** Railroad Bri e (Leninakan)
The bridge is undamaged. The pi s are of rigid-frame construction d large in cross section, indicat g that the performance of the bri e is largely different from that of e one in Photograph 3.2.1. The des n horizontal intensity is roughly esti ed at about 0.3.

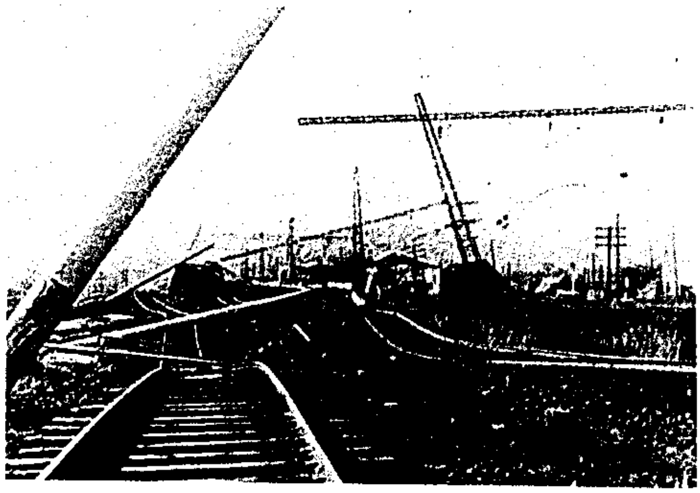
**Photograph 3.2.15** Railroad Bridge (suburban Yerevan, undamaged)
The bridge, which is undamaged, is located outside the disaster-stricken area.
The piers are not very large in cross section, but they are of rigid-frame construction and estimated to have a sufficient rigidity.



**Photograph 3.2.16** Crack at Mout of Railroad Tunnel (suburban Spitak)
Restoration work is under way. D age seemed to have occurred inside the tunnel though railroad service had reopened.

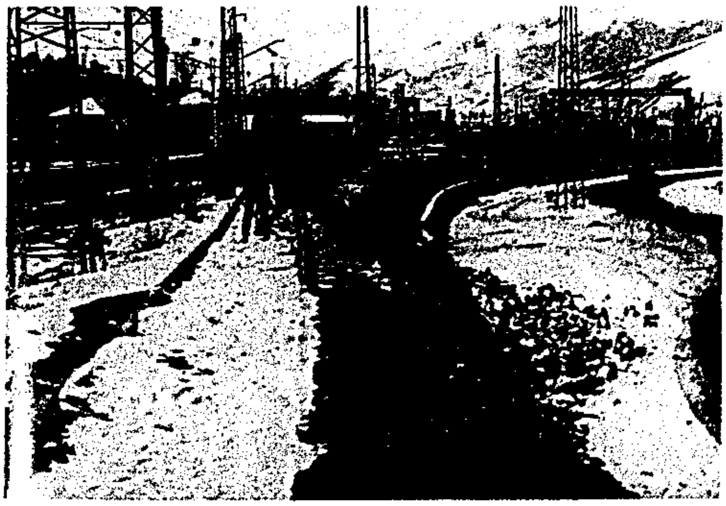


(a) The hill in the left appears to be a river terrace. The bankig in the low area slipped to destroy the track.



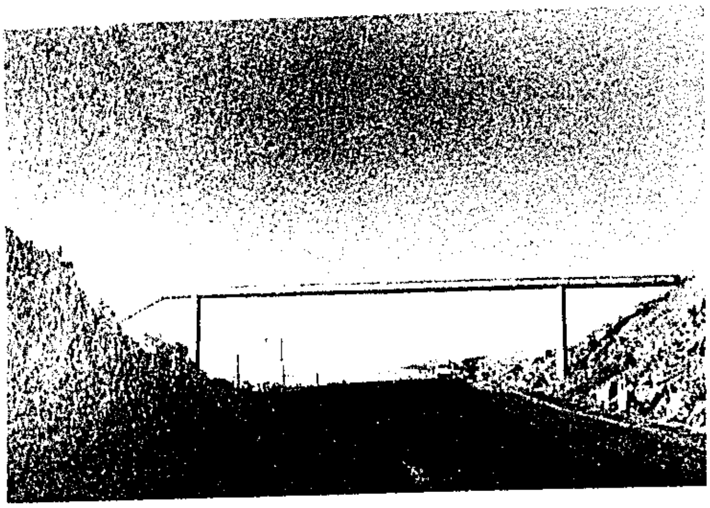
(b) The track is heavily damaged and rails are largely deformed. A train (probably freight train) is also largely inclined. All poles for aerial line and signs have fallen down, reflecting the serious disaster immediately after the earthquake.

**Photograph 3.2.17:** Damaged Railroad (photographed by Soviet expert immediately after the earthquake)

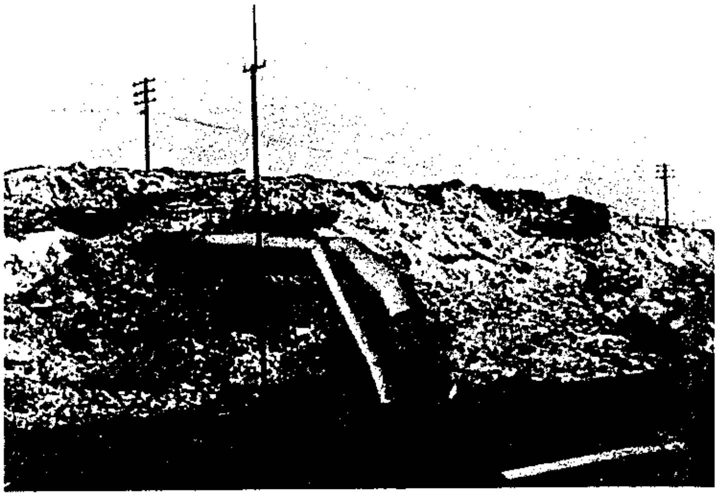


**Photograph 3.2.18:** Damage to Banking (Spitak Station)
Heavy damage like that in Photograph 17 was not found during the survey. In this example, the sandguard retaining wall is free from damage, and the shoulder is damaged only slightly.

**otograph 3.2.19:** Damaged Concrete ectric Pole and Temporary Restoration pitak)
considerably large earthquake force required to destroy electric poles.

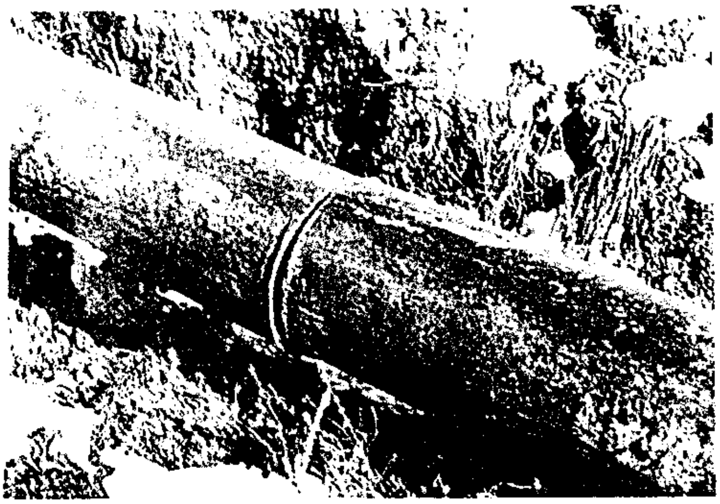


(a) A pipeline installed over a roa



(b) A pipeline crossing a road.

**Photograph 3.3.1**: Pipeline Across Road (steel pipe, probably for irrigation, undamaged)
There are pipelines like this in many places probably because the area is high in altitude and small in precipitation. Detailed information on their damage is not available.

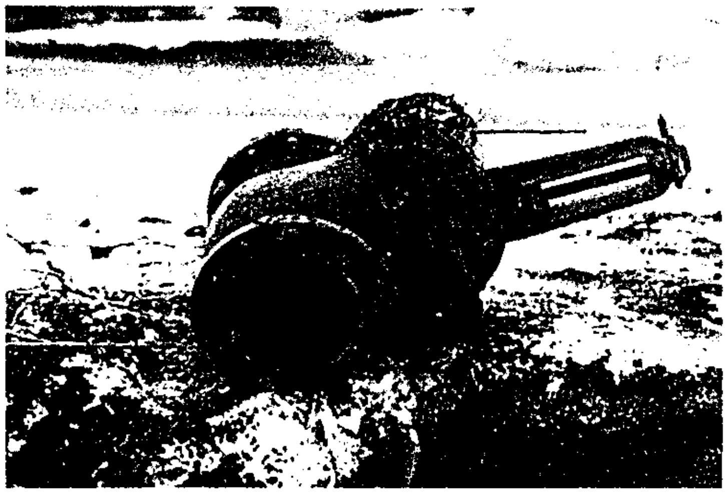


(a) Damaged Weld (approximately 25cm in diameter).

(b) Deformed Fitting (approximately 15cm in diameter)

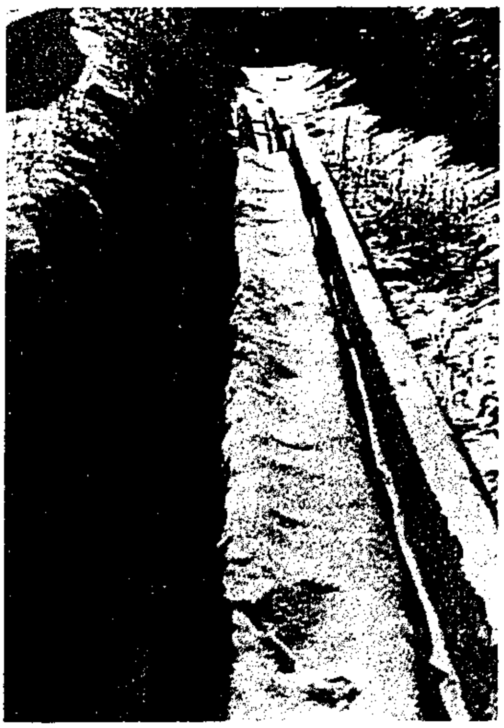
**Photograph 3.3.2**: Damaged Steel Pipeline (Spitak)

(a) Temporary Treatment of a Fitting



(b) Cutoff Valve

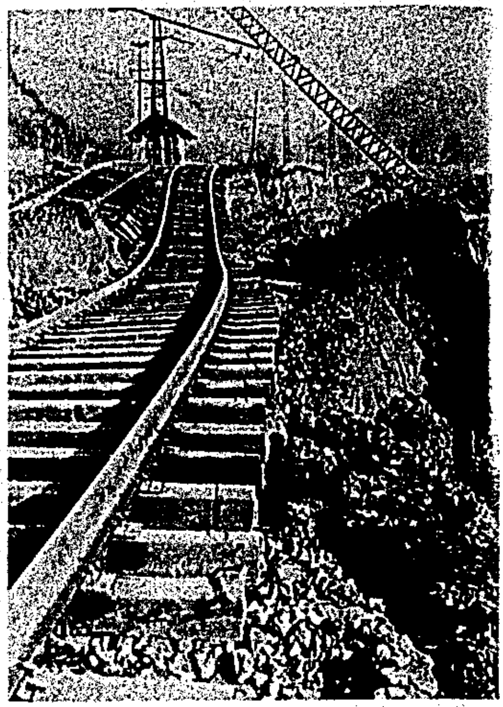
**Photograph 3.3.3**: Damaged Water Supply Pipe
It seemed that full-scale restoration had not started. Restoration work was seldom seen.



**Photograph 3.3.4**: Steel Pipe (Probably for Water Supply) Installed in Culvert at Building Construction Site(Leninakan)
It appeared that RC lids would be laid, followed by back filling.

**Photograph 3.3.5**: Restoration of Apparently Electric Cable under Pavement (Leninakan)

Photograph 3.3.6: Tanks in Industrial Complex (Leninakan)
When viewed from a distance, they did not appear to be damaged by fire, etc.



Photograph 3.4.1: Destroyed Railroad Banking

Photograph 3.4.2: Destroyed Masonry along Pamback River

Photograph 3.4.3: Fallen Rocks in Suburban Spitak

## 4. Discussion with Authorities Concerned

We made discussions with members from authorities concerned regarding such matters as seismic phenomena, damage, anti-earthquake design methods and restoration work in the future, as described below. The books and references we contributed on this occasion are listed in Chapter 10.

(1) Academy of Science of Armenian SSR and Institute of Geology of Armenia SSR

a) December 19 (Mon.) 18:00-

We had a meeting with Mr. Nicolay Lavelov, vice-president of Academy of Science of USSR and members from the Academy of Science and Institute of Geology of Armenian SSR. We obtained information and offered advice on the danger of earthquakes in the future as estimated from data on aftershocks.

Determination of the degree of damage to buildings and review of anti-earthquake design methods were the urgent matters for Mr. Lavelov, who requested us to make a discussion on them with members from the Armenia Scientific Research Institute of Civil Engineering and Architecture.

The Soviet participants are listed below.

1. Nicolay Lavelov
   Vice-President, Academy of Sci., USSR
2. Grigorian Sergei
   Corr, Member, Acad. of Sci., Arm. SSR
3. Gabrielian Arshaluis
   Academician, Acad. of Sci., Arm. SSR
4. Sedrakian David
   Academician, Acad. of Sci., Arm. SSR
5. Kejlis Borok
   Academician, Acad. of Sci., USSR
6. Shebalian Nicolay
   Professor
7. Karapetian Ashot
   Doctor of Sci.
8. Khatchian Eduard
   Doctor of Sci.

9. Nicolaev Alexei
Professor
10. Shahinian Suren
Doctor of Sci.
11. Djerbashian Ruben
Doctor of Sci.

b) December 20 (Tue.) 10:30-12:00

We honored the president of Academy of Science of Armenia SSR with a visit.

c) December 23 (Fri.) 11:00-12:00

We made a discussion following the one on December 19 at the Institute of Geology (Suehiro, Takamatsu).

(2) Armenia Scientific Research Institute of Civil Engineering and Architecture

1) December 20 (Tue.) 14:00-18:00 (?)

We had a meeting with 11 members including Director Shaginian S. G. to obtain information on damage to structures, construction methods and anti-earthquake design methods, and to offer cooperation as stated below.

i. We showed data on reinforced concrete prefabricated houses in Japan, the type of housing which suffered the most serious damage by the latest earthquake. We offered some papers and reports we had brought with us.

ii. We showed data on methods used in Japan for diagnosis of the earthquake resistance of existing buildings, strengthening of them, determination of the degree of damage to buildings, and strengthening of damaged buildings. We also offered papers and reports.

iii. The Soviet participants told us that the urgent matters included the evaluation of the safety of the remaining buildings in Leninakan and other cities and buildings in areas free of damage by the latest earthquake such as Yerevan.

iv. We received information on the strong motion records (data on one of the two horizontal components, those on upper and lower components not available). We were requested to carry out a numerical analysis of the recorded component and the calculation of response spectrum in Japan, and we promised to do them.

All members except Suehiro of the Japanese team took part in the meeting. The Soviet participants are listed below.

1. Shaginian S. G.
   Head of ARMNIISA
2. Badalian R. A.
   Head of Frame Building Lab.
3. Khachian E. E.
   Doc. of Eng. Sci., Chief of Lab. of Earthquake Eng.
4. Karamian K. O.
   Doc. of Eng. Sci., Chief of Lab. of Ind. House-building
5. Balian G. G.
   Chief of Economic Design Section
6. Shakhnazarian B. X.
   Cand. of Eng. Sci., Chief of Section
7. Aroichik A. B.
   Doc. of Eng. Sci., Expert of EDS
8. Ter-Stepaniain G. I.
   Head of Lab., Institute of Geophysics, Leninakan
9. Papian V. V.
   Chief Designer of ARMGOSPROEKT
10. Shakhsuvarian L. V.
    Doc. of Eng. Sci., Head of Group of Earthquake - Eng. Laboratory
11. Airapetova T. S.
    EDS Chief Designer
12. Dautian S.D. (interpreter -- English)

2) December 23 (Fri.) 11:20-12:20

We resumed the opinion exchange following the previous one (Okada, Yamanaka, Oda).

Major subjects included factors in damage, anti-earthquake design methods, plan on the transfer of the city Spitak, and disaster prevention measures adopted in Japan.

3) December 24 (Sat.) 10:00-12:30

Okada, Hirosawa, Minami and Yoshida visited the institute again to make a discussion on the design intensities in the earthquake-stricken area. They had a chance to see the experimental facilities there.

(3) Institute of Geology and Engineering Seismology

December 21, morning, and December 22, 15:00-19:00

We met institute members including director Badalian, S. V., to exchange opinions on aftershocks, ground motion, damage, and design intensity.

(4) Yerevan Polytechnical Institute, Road and Bridge Construction Department

a) December 24 (Sat.) 10:00-12:00

Kawashima visited the department to collect information on the anti-earthquake design methods used for bridges in Armenia, and received papers and reports concerning bridges. He made an explanation on anti-earthquake design methods for road bridges and measures protection of roads against earthquakes which have been adopted in Japan, and offered papers and reports.

| | |
|---|---|
| Abovian G.A. | Vice Director |
| Doozgazian S. M. | Professor |
| Ionnisian S. G. | Associate Professor |
| Azoian P.S. | Associate Professor |
| Azootuuian F.Z. | Associate Professor |
| Avefigian A. M. | Associate Professor |

(5) Armenian Ministry of Transportation

a) December 24 (Sat.) 13:00-13:30

Kawashima visited the ministry to obtain information on damage to road facilities from the Minister of Transportation.

(6) Department of Geophysics, Academy of Science, USSR

a) December 26 (Mon.) 16:00-17:00

We made a discussion with department members including Professor Bune and Steinberg.

We received information on seismic sonation, and exchanged opinions on actual design intensities in the earthquake-stricken area and design intensities to be used for anti-earthquake design.

(7) National Committee of Construction, USSR

a) December 27 (Tue.) 10:00-11:30

We made a rough report to seven members including vice-chairman Chizhevsky and exchanged opinions on factors in damage and review of anti-earthquake codes. The Soviet participants are listed below.

Chizhevsky, M.V.

vice-chairman, National Committee of Construction

Director, Bureau of Science and Technology, the same Committee

Vice-director, the same Bureau, the same Committee

Director, Basic Scientific Institute for Building and Bridge, USSR

Vice-director, Central Scientific Institute for Construction and Structure

Section Chief, Public Relations Division, National Committee of Construction

Chief Expert, Bureau of Science and Technology, National Committee of Construction

## 5. Future Measures Required in Seismic Region in Soviet Union

Specific measures required in the future for the seismic zones in the Soviet Union will be largely different in some aspects from those needed in Japan, though the basic concept may be the same.

Countermeasures against earthquakes in Japan have been carried out in line with the "Guidelines for Promotion of Countermeasures against Earthquake in Large City" adopted in the Central Disaster Prevention Conference in 1971. In particular, the Promotion Guidelines requires the following: 1) promotion of disaster prevention measures in cities, 2) strengthening of the disaster prevention system and uplift of consciousness towards disaster prevention, and 3) promotion of earthquake prediction.

1) Implementation of disaster prevention measures in urban areas is one of the major pillars for establishing disaster-free cities. In the Soviet Union, however, the enhancement in the earthquake resistance of existing buildings, let alone newly constructed ones, against the ground motion itself is more important than the implementation of disaster prevention measures.

2) Enhancement in the public consciousness towards disaster prevention and strengthening of disaster prevention measures may also be important in the Soviet Union. Considering the large difference in national consciousness, customs and population density, however, it would be only natural that specific measures required in the two nations would be different.

3) Needless to say, earthquake prediction is of great importance. Measures should be taken based on adequate studies on the characteristics of specific earthquakes.

## 6. Conclusion

The present report outlines the activities and results of a survey by the First Team of Japan Disaster Relief (JDR), dispatched by the Japanese government to offer technical advice for the rehabilitation and restoration of the Armenian SSR suffering desaster from the Armenian Spitak earthquake, which occurred on December 7, 1988.

JDR Team is generally dispatched in accordance with the "Law Concerning Dispatch of Japan Disaster Relief Team (1987)", which was promulgated and enforced on the basis of the experience with the volcanic eruption in Colombia and the Mexico earthquake in 1985. The Team consists of a rescue team (rescue of victims), medical team (first aid, disinfection) and expert team (emergency measures and restoration from disaster). In the latest case, the Soviet Union after making a discussion with an advance group requested Japan to dispatch an expert team. At the time of the departure of the first team, we had obtained reports indicating that many people had been killed under buildings destroyed by the earthquake and that medium-height apartments of precast reinforced concrete or stone material had accounted for a great part of the destroyed buildings, but no detailed information was available that would be useful for the activities requested by the Soviet Union. Consequently, the Team carried up-to-date technical data covering methods for earthquake resistant design, determination of the degree of damage and rehabilitation of earthquake-stricken areas, focusing on reinforced concrete buildings and other structures.

The organizations for joint investigations in Armenia were the National Committees of Construction and Academies of Science of USSR and Armenian SSR. The rehabilitation work was being performed through nationwide efforts. For these reasons, we had meetings with both Soviet and Armenian organizations.

The National Committee of Construction of USSR is an organization similar to the Ministry of Construction of Japan. The participants in the meeting included members from a department in charge of the construction standards applicable to the entire Soviet Union, and members from the central Research Institute of Architectural Structure and Scientific Research Institute of Basic Structure, which are under the Committee.

The National Committee of Construction of Armenian SSR, which acts as a construction ministry of the republic and was playing the leading role in the restoration work, was in charge of the survey of the conditions of damaged buildings, judgment as to whether each building should be pulled down or strengthened for reuse, study on methods for their strengthening, and planning of new residential zones.

The Academies of Science are similar in many aspects to the Japan Academy. Unlike the Japan Academy, however, they are acting practically as research authorities with research institutes under their direct control. Following the latest earthquake, they were in charge of coordination among other authorities concerned and their subsidiary research institutes in the fields of the study on the characteristics of the earthquake. They also performed a joint study with U.S. and French teams to make investigation on aftershocks. The participants in the meetings with us included members from the Research Institute of Geophysics and a joint committee of seismology and seismic engineering, which are under the Academy of Science of USSR, and members from the Research Institute of Geology, which is under the Academy of Science of Armenia SSR.

Prior to meetings in various fields, the Team made a field survey of damaged buildings and other structures in major areas including the three cities of Spitak, Leninakan and Kirovakan.

Through meetings with these organizations, we understood that the Soviet Union was facing difficulties in urgent issues as follows:

a. Whether certain damaged buildings should be pulled down or strengthened for reuse.

b. How to review the earthquake resistant design standards reflecting the lessons learned from the experience with the earthquake.

c. Whether the city of Spitak, which suffered serious damage, should be transferred to other place for the reconstruction of a new Spitak.

d. Possibility of strong aftershocks, though none of such ones had occurred.

Concerning these points, we presented our views based on our field survey while showing technical data that we had brought with us. The present report outlines the results of the survey by the Team, contents of the meeting and advise offered to Soviet organizations the above-mentioned matters, and lists the books and reports that we contributed to them.

The Soviet Union is one of the leading countries in the field of research in seismology and seismic engineering. The latest earthquake, however, was much higher in intensity than ones expected in the Armenian SSR. This was obviously one of the major factors in the destruction of a large number of buildings.

Considering this, we concluded that disaster prevention techniques developed in Japan would be very useful for the restoration of the damaged area and mitigation of the effects of the disaster in the future because earthquakes with strong ground motion as seen in the latest case have been assumed in actual design in the country.

Before arriving at the damaged area, we were anxious as to whether we would be permitted to take photographs there and whether necessary technical data would be available, though we received active cooperation from organizations there and this proved to be needless worry. Our survey was assumed to complete in a single visit. However, the field survey has revealed that the disaster was so heavy and there were so many serious problems to be solved promptly for restoration. We, as members of the Team, strongly hope that another team carrying data and information requested by the person we met in Soviet Union, will be dispatched again as early as possible within the existing framework.

## 7. List of Contributed Books and Reports

1. Measures against Seismic Damage to Roads (Preventive Measures)
2. Measures against Seismic Damage to Roads (Restoration Measures)
3. Outline of Manual for Repair Methods for Civil Engineering Structures Damaged by Earthquakes (by Iwasaki et al.)
4. Evaluation of Seismic Vulnerability of Highway (Bridges in Japan (by Kanashima et al.)
5. Present Status of Seismic Design of Highway Bridge in Japan (by Kawashima et al.)
6. Design and Construction of Wall-Type Precast Reinforced Concrete Structure (Japanese Society of Architecture, 4,900), Oct. 1986
7. Teaching Material for Study on Structure, (Japanese Society of Architecture, 1,800), Mar. 1985
8. Proceedings of the Seminar on Repair and Retrofit of Structure, Workshop, U.S.-Japan Panel on Wind and Seismic Effect, UJNR, May 1987
9. General Earthquake Engineering in Japan, IISEE Lecture Note, 1988/89, M. Hirosawa
10. Earthquake Resistance Diagnosis and Anti-earthquake Enforcement Standards, IISEE Lecture Note
11. Guidelines Arrangement of Reinforcement in Wall Structure, Nov. 1987
12. Experiment with Vibration of Buildings, Dec. 1978
13. Data for Anti-earthquake Design of Buildings, Apr. 1981
14. Seismic Load -- Present Status and Future Perspective, Nov. 1987
15. Yield Strength and Deformation Performance in Anti-earthquake Design of Buildings, Jun. 1981
16. AIJ Standards for Structural Calculation of Reinforced Concrete Structure and Explanatory Notes (in Japanese), Oct. 1988
17. AIJ Standard for Structural Calculation of Reinforced Concrete Structures(in English), Jul. 1985

18. Data on Ultimate Strength Design of Reinforced Concrete, Sep. 1987

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# II The Second JDR Team

## II. The Second JDR Team

### 1. Outline of Evaluation

#### 1.1 Outline of Seismic Design Methods in Japan

(1) General

The general philosophies accepted in most countries of the world for the seismic resistant design of buildings are that (1) only minor damage without significant loss of economy is allowable under moderate earthquakes and (2) severe damage or collapse followed by loss of human life should be prevented under severe earthquakes. For these philosophies general buildings lower than roughly ten-story height in Japan are designed by the allowable stress method with the base shear coefficient of Co = 0.2 for moderate earthquakes, and by the ultimate method with the coefficient of Co = 1.0 for severe earthquakes. Higher buildings are required further design studies based on analytical and/or experimental investigations. The required ultimate lateral load carrying capacity of a building, however, may be decreased, as shown in the following Eq (2), corresponding to the level of ductility while it must be increased with the effect of torsion and/or soft story.

$$Qu \geqq Qun \quad (1)$$

$$Qun = Ds \times Fes \times Qud \quad (2)$$

$$Qud = Z \times Rt \times Ai \times Co \times W \quad (3)$$

here

Qu : Lateral load carrying capacity
Qun : Required lateral load carrying capacity
Ds : Structural factor dependent on ductility (Ds=0.25-1.0)
Fes : Shape factor (Fes=1-2.25)
Z : Zone factor (Z=1.0-0.8 for mainlands)
Rt : Spectral coefficient corresponding to soil condition (Rt=1-0.25)
Co : Standard base shear coefficient (Co=1.0)
W : Weight of building
Ai : Lateral force distribution factor along the height

The lateral load carrying capacity of a building is basically calculated for the failure mechanism formed by flexural yielding of beams, columns and shear walls. Although the capacity is calculated to one loading direction, the effect of floor slabs and members perpendicular to the direction must be taken into account. Lateral displacement of a building subjected to the forces corresponding to Co = 0.2 shall not exceed 1/200 the story height, however, the use of deformable non-structural members or deformable joint systems may raise the displacement up to 1/120.

(2) Ds Value for Reinforced Concrete Buildings

Seismic capacity of a building is controlled by strength and ductility capacities (ref.2). Design lateral load carrying capacity may be decreased with ductility. The Ds factor to represent ductility is quantitatively regulated for reinforced concrete, steel or timber structures, respectively, corresponding to the level of expected ductility. In the case of most ductile building, the required capacity may be decreased to 25% the required capacity of most brittle building, thus the Ds may take the value of 0.25.

Ds value for R/C buildings are obtained as follows. Firstly, each member is classified to one of the four members shown in Tables 1 and 2 corresponding to ductility rank. The ductility rank is determined with the level of ultimate shear stress and other structural factors. Secondly, the ductility rank as a group of columns or shear walls is determined, as shown in Tables 3 and 4, based on the ratio of total strength of columns or shear walls in the same ductility rank to that of all the columns or shear walls. Thirdly, the Ds value at each story shown in Table 4 is determined on the basis of (1) a combination of ductility ranks of groups of columns and shear walls, and (2) ratio of capacity of shear walls to that of the story. Boundary values in the tables are determined with a great deal of experimental and analytical data.

## 1.2 Comments on Design Seismic Forces in the USSR Codes

Following comments are described with emphasis on comparisons of design seismic forces with those by the Japanese codes.

### (1) Design Seismic Intensity and Zone Factor

Design seismic forces in the USSR codes are expressed in Eq.(4) in terms of acting external forces Sik at the k-th floor and the i-th mode. The design seismic intensity A in the area I is shown in Eq.(5).

$$Sik = K1 \times K2 \times Qk \times A \times \beta i \times K\Phi \times \eta ik \qquad (4)$$

$$A = 0.4 \times 2^{(I-9)} \quad I = 7,\ 8,\ 9 \qquad (5)$$

The maximum intensity 0.4 in Eq.(5) is almost same to that in the Japanese codes, however, the zone factor is different. Relative values of zone factor in several countries are listed in Table 5. The USSR codes indicate most significant change for different zones. Due to the following facts, the Japanese codes moderate the reduction of zone factor for mainlands by 20% at maximum. (1) There is an uncertainty in the presumed intensity of each earthquake on an available earthquake catalogue. (2) The duration of the catalogue is not long enough compared with return periods of earthquakes.

### (2) Dynamic Amplification Factor Corresponding to Soil Condition

The dynamic amplification factor $\beta$ in Eq.(4) is regulated as shown in Fig. 1. Relative values of $\beta/\beta 0$ in the Japanese codes are shown in Fig. 2, where $\beta 0$ is the value for short period structures on hard soil. In the Mexico City codes, design base shear coefficients corresponding to soil conditions were revised after the earthquake of 1985 from those in Fig. 3 to those in Fig. 4. Due to following facts, the reduction of the factors on a long period range is moderated in the Japanese codes. (1) Exact classification of soil condition is difficult in several cases. (2) It was observed that predominant periods of medium soil tended to increase under severe earthquakes.

**(3) Reduction Factor K1 for Allowable Damage Degree** The factor K1 in Eq.(4) is defined only by the use of building and is regulated as 0.25 for general buildings. The factor is adopted to allow moderate damage which does not harm human life in a building under severe earthquakes. However, the building should not collapse at a large deflection resulting from reduced lateral load carrying capacity. That is, a ductile building capable of withstanding a large deflection should be designed to maintain permissible performance even under decreased lateral load carrying capacity.

It is considered necessary to divide K1 into two factors, the importance factor dependent on the use of building and the structural factor dependent on ductility of building. It is suggested to provide a factor similar to the previously described Ds factor and to regulate adequate provisions necessary to provide ductile performance. For reference, the importance factor and the structural factor in the Japanese codes are shown in Tables 6 and 7.

### (4) Rough Comparison of Design Seismic Foreces and Designed Members

A rough Comparison of design seismic forces and designed sections and reinforcements in an example building in accordance with both the Japanese and USSR codes is shown in Fig. 5. A 9-story precast R/C frame building on the medium soil in Leninakan area (I = 8) was taken as an example. The design forces and designed sections would be close to those by the Japanese Codes when the design intensity took I = 9.

### 1.3 Design Study of Prototype Buildings

Two prototype R/C buildings of precast large-panel structure and of wall structure were studied following the Japanese practice in both the design and analysis. They were previously designed in accordance with the USSR codes.

#### (1) Study of a 4-Story Precast Large-Panel Structure Building

1) Type of Structural System and Method of Analysis : As the structural system of the building was similar to precast large-panel structures in Japan, the structural calculation was performed based on the Japanese regulations. Floor plans and sections are shown in Fig. 6. Different structural properties from those of Japanese buildings were that it consisted of thick walls, light weight and low strength of concrete and heavy unit weight. Floor panels were precast void panels, however, precast wall panels were monolithic.

The ultimate strength method with Co = 1.0 as well as the allowable stress method with Co = 0.2 were applied for the study. Although the Ds value might be selected from 0.45 to 0.55, it took 0.5 because of existence of short beams. Details of each connection were as follows. (1) Vertical joints : wet joints with shear keys or welded reinforcements. (2) Horizontal joints : dry joints by direct joint system. (3) Floor to floor joints : wet joint with welded anchor reinforcements

2) Structural Dimension and Analysis for Co=0.2 : The stress analysis was done assuming (1) equal unit shear stress in walls, (2) standard ratio of inflection point to story height, and (3) local redistribution of elastic stress. Flexural reinforcements of beams were determined by working stress while shear reinforcements by the minimum requirements. Both flexural and shear reinforcements of walls were determined by the minimum requirements.

3) Lateral Load Carrying Capacity : The base shear coefficient corresponding to the reinforcements determined by the allowable stress method was 0.45. It reached roughly 0.5 when increasing flexural reinforcements at the base of walls of the 1st floor. Vertical and horizontal joints were checked with the ultimate strength method for the coefficient 0.5. Floor joints were checked by the allowable stress method for the local seismic coefficient 1.0.

4) Results : The obtained main results were the increases of (1) shear reinforcement of beams, walls and connections, (2) flexural reinforcement of beams and walls and (3) strength of foundation beams, compared with those in the previous design.

#### (2) Study of a 4-Story Cast-in-Place Wall Structure Building

1) Type of Structural System and Method of Analysis : Plan and section of the building are shown in Fig. 7. The building consisted of mixed systems of wall structure (box wall structure) and walled frame structure (frame structure with wall columns), however, the total system was rather similar to typical wall structures in Japan. Although the length of wall per unit floor area (13.8cm per square meters) did not satisfy the minimum requirement (15cm per square meters), the area of wall (425.5 square cm per floor area) satisfied the requirement (15cm per square meters times the wall thickness ) for the

wall structure. Therefore, it was decided to study the building in accordance with the standard regulations for wall structures. The building of wall structure in general may be designed by simplified methods without detailed calculation. However, here the lateral load carrying capacity and the ductility capacity were investigated and the elasto-plastic response analysis was carried out because of insufficient amount of structural walls.

2) Elastic Stress Analysis and Required Reinforcements for Co=0.2 : As a result of the analysis, the stress concentration to wall plane was fairly big. Mean shear stress of the wall plane reached more than 2 times the walled frame plane ($\tau 1$ : $\tau 2$ = 0.14 : 0.36 MPa). The stress distribution was influenced by the rigidity of the ground and foundation beams. Flexural and shear reinforcements of walls and beams were determined by the minimum requirements.

3) Lateral Load Carrying Capacity and Ductility : Flexural reinforcements necessary to obtain the required capacity were calculated by simplified method assuming a failure mechanism and referring to the item 2) above. The required Ds value was 0.45 since all the walls were judged as the rank WA because the maximum value of $\tau u/Fc$ (0.07) was less than 0.1. The calculated lateral load carrying capacity was 0.52 in term of base shear coefficient. The obtained failure mechanism consisted of flexural yielding of beams and overturning of both the exterior walls and exterior walled frames.

4) Elasto-plastic Response Analysis : The analysis was carried out under the following conditions and the obtained results are summarized as follows.

a. Input earthquake motions : Recorded ground motions, Hachinohe-EW 1968 and Yerevan 1988 normalized with the maximum acceleration of 400 gals.
b. Dynamic properties and hysteresis rule : Fundamental period T1=0.24 sec, damping factor=5% and elasto-plastic type hysteresis rule.
c. Maximum ductility factor : $\mu$ = 2.4 induced by the Yerevan record and $\mu$ = 1.2 by the Hachinohe record.
d. Maximum story drift : $\delta$ = 0.82cm at the 2nd story induced by the Yerevan record and $\delta$ = 0.39cm at the first story by the Hachinohe record.

## 1.4 Seismic Strengthening for Damaged Buildings

### (1) Criteria of Seismic Capacity

The recommended criteria for rehabilitation of damaged buildings (repair, strengthening, demolition) in Japan are shown in Table 8. The aimed seismic capacity of damaged or undamaged R/C buildings is expressed in terms of Is index. The index Is generally takes 0.7 for low to medium-rise buildings subjected to 0.3 to 0.4g level of ground motion, where g is the specified gravity, and to approximately 3 time the response amplification.

### (2) Seismic Strengthening of a 5-Story Precast R/ C Frame Building

Three types of strengthening methods were studied for a 5-story damaged frame building in Leninakan which were under construction. It was indicated that the strengthening with each of steel braces shown in Fig. 8 and concrete shear walls shown in Fig. 9 could provide as much seismic capacity as that generally required in Japan. Some details are shown in those figures.

### (3) Recommendations for Strengthening Design and Construction

Careful attention must be paid to (1) space of seismic joints, (2) rigid floor diaphragm, (3) rigid foundation system, (4) design and construction of strengthening elements of steel, (5) quality control of construction, (6) columns subjected to very high axial compression, and (7) joint of nonstructural elements to structural members.

## 1.5 Comments on Seismic Microzoning Method Based on Measurement of Microtremors

### (1) General Characteristics of Microtremors

1) Definition of Microtremors : Microtremor is defined as ground vibration by artificial sources caused by traffics or machinaries in the city area, and it differs from microseismo caused by natural meteological phenomena. The amplitude of microtremor is smaller than 10 micrometer and the range of period is 0.05 to 2 or 3 sec (sometimes 4 to 5 sec). Its amplitude is changed with the time, that is, it may be 3 times larger in day-time than in night-time.

2) Layered Structure of Subsurface Soil and Predominant Period of Fourier Spectrum of Microtremors : In the layered soil structure, seismic waves propagate with multiple reflection. As a result of the reflection, it has own period. This period appears as the predominant period of microtremors.

3) Relative Assessment of Amplitude of Fourier Spectrum of Microtremors : The shape of Fourier spectrum of microtremors is similar to that of strong-earthquake ground motions. The amplitude of Fourier spectrum of the ground motions is equivalent to that of microtremors.

### (2) Microtremors Measured in and around Leninakan

1) Seismic Bedrock of Leninakan : The microtremors by long period seismometer found the predominant period of 3 - 4 sec. This predominant period was caused by very deep sedimentary layer on the seismic bedrock. The seismic bedrock of Leninakan area was located at several kilometers bellow the ground surface.

2) Mean Period of Microtremors and Degree of Building Damage : The mean period of micrtremors of the ground in northern area (north part from Lenin square) was 0.5 - 0.6 sec, and this was close to the natural period of 9-story precast R/C frame buildings. So it was guessed that this fact might be one of the causes of serious damage to this kind of buildings. The mean period in the central area of the city was 0.2 - 0.3 sec and was close to the natural period of low-rise stone-masonry buildings. This fact might be one of the causes of serious damage to this kind of buildings. It would be necessary to implement more detailed survey, because there were underground flows of rivers. The predominant period in the south area was close to that in the north area, however, it's amplitude was larger.

3) Microtremors measured in the New Developing Area : The microtremors by long period seismometer were measured at two sites in the new developing area of north Leninakan. The predominant period was about 3.5 sec. The predominant period in the new developing area of south Leninakan was close to that in the north area. Therefore, the seismic bedrock in and around Leninakan area seemed to be located at almost the same depth.

### (3) Microtremors Measured in and around Spitak

1) Appeared Seismic Bedrock : The microtremors of the ground were measured at five sites in and around Spitak. The predominant period was very short (less than 0.1 sec) at the foot around the hill where a monument was constructed at its top. The seismic bedrock was located at very shallow portion in the area.

2) Mean Period of Microtremors and Degree of Building Damage : The predominant periods in the heavily damaged area of Spitak were in the range between 0.2 and 0.5 sec, and these were close to natural periods of damaged buildings.

3) Microtremors Measured in New Developing Area : The predominant period in new developing area was close to that in the damaged area. Therefore, it would be necessary to construct more reinforced and stiffened buildings.

(4) Microtremors Measured in and around Kirovakan
Just in front of the City-office, the predominant period of the ground was 0.15 sec and the amplitude was about 1/3 the amplitude in heavily damaged area. In the damaged area closely located to the City-office, the predominant period was 0.3 to 0.5 sec. This fact might be one of the causes of serious damage to buildings. The predominant period in the new developing area was slightly longer than that in the damaged area.

## 1.6 Measured Dynamic Properties of Buildings in Armenia

(1) Results of Microtremors Measured on Buildings The microtremors on the buildings were measured at 19 sites in Yerevan, Leninakan and Kirovakan. The measurements in Yerevan were recognized for dynamic properties of undamaged buildings while those in Leninakan and Kirovakan were recognized for damaged buildings. The measurement results are listed in Table 9.

(2) Comparison of Dynamic Properties between Undamaged and Damaged Buildings
1) 5-Story Stone-Masonry Building : The measured natural periods and critical damping coefficients of undamaged buildings were 0.28 - 0.33 sec and 2.2 - 7.2%, respectively. Those of damaged buildings were 0.21 - 0.55 sec and 1.9 - 4.4%, respectively. The change of the natural periods in damaged buildings were significant, presumably because of the deterioration of both the stiffness and strength of masonry walls.

2) 9-Story Precast Frame Buildings : The natural periods in the longitudinal and transverse directions of undamaged buildings were 0.61 - 0.63 and 0.46 - 0.53 sec, respectively. Those of damaged buildings were 0.83 - 1.07 and 0.81-1.00 sec, respectively. The natural period of damaged buildings was increased by 30 - 40% compared with the period of undamaged buildings. The critical damping coefficient of damaged buildings was increased by more than 25%.

3) 9-Story Large Panel Buildings : The natural periods and critical damping coefficients of undamaged buildings were 0.38 - 0.39 sec and 1.2-2.5%, respectively. Those of damaged buildings were 0.40 - 0.46 sec and 2.6 - 4.5%, respectively. The natural periods were not so changed between undamaged and damaged buildings, but the critical damping coefficients of damaged buildings were almost 2 times those of undamaged buildings.

## 1.7 Concluding Remarks

The 1988 Spitak Earthquake has given a very strong impact to structural engineers over the world because a number of modern buildings designed with seismic design codes had been totally collapsed. The lessons to be learned from this earthquake seemed to be almost the same as those learned from destructive earthquakes for past several decades in Japan. The activities of the Japanese missions have been done based on those lessons. Main results and comments obtained form the works and discussions are summarized as follows.

1) Insufficient strength and/or ductility capacity of precast frame buildings compared with the amplitude level of the seismic motions would have caused the collapse of many buildings. Poor construction quality would have accelerated the damage. It must be emphasized that the joints between precast members such as column to beam, beam to slab and wall to slab were not provided with sufficient strength nor ductility.

However, there is a question how nonstructural members such as external walls in damaged precast frame buildings behaved during the earthquake. How the damage to such walls played roles to the collapse of the buildings ?

The collapse and/or falling of walls would have reduced both the strength and stiffness of several stories while the response forces would have not been reduced so much because of small change of fundamental period. This would have resulted in concentrated displacement at particular stories. Consequently, it is important to investigate the process up to collapse of buildings, that is, which part was damaged first and how the buildings behaved up to collapse.

2) In the affected area, a total volume of concrete used for both structural and nonstructural members in a building seemed to be almost the same as that in Japan. Effective use of concrete as structural material, more amount of both longitudinal and lateral reinforcements, and good quality control of construction could have improved the seismic capacity of the buildings. Severe climate conditions in addition to these matters might restrict available structural systems. For example, wall structures, walled frame structures and large-panel structures might be recommended.

3) To incorporate design seismic forces, reasonable amplitude level and characteristic of ground motions should be estimated considering the dimension of fault rupture, and topography and soil condition of the site. This subject might still be one of the problems which need tremendous efforts to solve in the field of earthquake engineering.

4) The ductility capacity of structural members and frames should be evaluated based on available experimental data.

5) Careful attention must be paid to the following items and design requirements should be provided. (1) Space of seismic joints. (2) Rigid floor diaphragm. (3) Rigid foundation system. (4) Joints of non-structural elements to structural members. (5) Possible overturnings and fallings.

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Table 1 Ductility Rank of Framing Members

| Ductility Rank | | FA | FB | FC | FD |
|---|---|---|---|---|---|
| Column | $h_o/D$ | $\geq 2.5$ | $\geq 2.0$ | – | Others |
| | $\sigma_o/Fc$ | $\leq 0.35$ | $\leq 0.45$ | $\leq 0.55$ | |
| | Pt | $\leq 0.8\%$ | $\leq 1.0\%$ | – | |
| | $\tau u/Fc$ | $\leq 0.1$ | $\leq 0.125$ | $\leq 0.15$ | |
| Beam | $\tau u/Fc$ | $\leq 0.15$ | $\leq 0.20$ | – | |
| Common Condition | | Qsu > Qm | | | |

Qsu : Shear capacity. Qm : Shear force at mechanism
$h_o$,D : Clear height and depth
$\sigma_o$ : Compressive unit stress at mechanism
Pt : Tensile reinforcement ratio
Fc : Design compressive strength of concrete
$\tau u$ : Nominal ultimate shear stress $Qm/(b \cdot D)$
here b:width

Table 2 Ductility Rank of Shear Walls

| Ductility Rank | | WA | WB | WC | WD |
|---|---|---|---|---|---|
| Upper limit $\tau u/Fc$ | General *1 | 0.2 | 0.25 | -- | Others |
| | Box Wall *2 | 0.2 | 0.125 | 0.15 | |
| Commom Condition | | Qsu > Qu | | | |

*1 shear walls with boundary columns
*2 walls in box wall type struchere

Table 3 Ductility Rank as a Group of Members

| | $\Sigma Qa / \Sigma Q$ | $\Sigma Qb / \Sigma Q$ | $\Sigma Qc / \Sigma Q$ |
|---|---|---|---|
| Rank A | $\geqq 50\%$ | – – – | $\leqq 20\%$ |
| Rank B | – – – | – – – | $< 50\%$ |
| Rank C | – – – | – – – | $\geqq 50\%$ |

$\Sigma Q = \Sigma Qa + \Sigma Qb + \Sigma Qc$
$\Sigma Qa$ : Total strength of FA(or WA) members
$\Sigma Qb$ : Total strength of FB(or WB) members
$\Sigma Qc$ : Total strength of FC(or WC) members

Table 5 Relative Value of Zone Factor in Current Codes

| | I | II | III |
|---|---|---|---|
| U.S.S.R(1981) | 1.0 | 0.5 | 0.25 |
| Japan (1981) | 1.0 | 0.9 | 0.8 |
| U.S.A. (1985) | 1.0 | 0.75 | 0.375 |
| Mexico (1985) | 1.0 | 0.7 | 0.4 |

Table 4 Ds-Value for Reinforced Concrete Structure

| Ductility Rannk of Column Group | Ductility Rank of Wall Group | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | WA | | | WB | | | WC | | | WD | | |
| | βu ≦0.3 | 0.3 ⁞ 0.7 | βu >0.7 | βu ≦0.3 | 0.3 ⁞ 0.7 | βu >0.7 | βu ≦0.3 | 0.3 ⁞ 0.7 | βu >0.7 | βu ≦0.3 | 0.3 ⁞ 0.7 | βu >0.7 |
| FA | 0.3 | 0.35 | 0.4 | 0.35 | 0.4 | 0.45 | 0.35 | 0.4 | 0.5 | 0.4 | 0.45 | 0.55 |
| FB | 0.35 | 0.4 | 0.45 | 0.35 | 0.4 | 0.45 | 0.35 | 0.45 | 0.5 | 0.4 | 0.45 | 0.55 |
| FC | 0.4 | 0.45 | 0.45 | 0.4 | 0.45 | 0.5 | 0.4 | 0.45 | 0.5 | 0.45 | 0.5 | 0.55 |
| FD | 0.45 | 0.5 | 0.55 | 0.45 | 0.45 | 0.55 | 0.45 | 0.5 | 0.55 | 0.45 | 0.5 | 0.55 |
| βu = 1 *1 | --- | --- | 0.45 | --- | --- | 0.5 | --- | --- | 0.55 | --- | --- | 0.55 |

*1 Box Wall Type Structure

Table 6 Importance Factor I in Japan

| | Type of building |
|---|---|
| 1.0 | General building |
| 1.25 | Important building (Governmental or public) |
| 1.5~3.0 | Nuclear Power Plant<br>1.5 for moderate earthquake<br>3.0 for severe earthquake |

Table 7 Strucural Coefficient Ds for Reinforced Concrete Structure

| Ds Value | Type of Structure |
|---|---|
| 0.3 | Ductile Moment Frame Structure |
| 0.4 | Ductile Shear Wall Structure |
| 0.45~0.55 | Shear Wall Structure |
| 0.45~0.55 | Box Wall Structure<br>Large Panel Structure |

Table 8 Recommended Permanent Treatment for Damaged buiuldings

| Seismic Intensity *1 | Identification of Damage and Permanent Treatments | | | |
|---|---|---|---|---|
| | Slight | Small | Moderate | Severe or Collapse |
| Less than or equal to IV ($\alpha \leq 80$) | Repair | Repair or Strengthening (2nd Level Check) | Strengthening or Demolition | Strengthening or Demolition |
| Equal to V ($80 < \alpha \leq 250$) | Repair | Repair | Repair or Strengthening (2nd Level Check) | Strengthening or Demolition |
| Greater than or equal to IV ($250 \leq \alpha < 450$) | Repair | Repair | Repair | Repair or Strengthening (2nd Level Check) |

Note *1 Japan Meteorological Agency, $\alpha$ : Ground Surface Acceleration (gal)

Table 9 Dynamic Properties of Damaged and Undamaged buildings

| Type of Structure and No. of Story | Damage *1 | Translation | | | | Torsional | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Longitudinal | | Transverse | | Longitudinal | | Transverse | |
| | | T sec | h % | T sec | h % | T sec | h % | T sec | h % |
| Stone Masonry 5 | U | 0.30 | 4.8 | 0.33 | 7.2 | 0.35 | 2.6 | 0.35 | 4.6 |
| | U | 0.28 | 2.2 | 0.28 | 4.6 | 0.23 | 1.8 | 0.23 | 1.1 |
| | D | 0.49 | 4.0 | 0.28 | 1.9 | 0.41 | 4.7 | 0.41 | 3.6 |
| | D | 0.44 | 2.6 | 0.21 | 1.9 | 0.40 | 3.6 | 0.40 | 4.5 |
| | D | 0.52 | 4.0 | 0.52 | 7.4 | 0.55 | 3.2 | 0.56 | 4.1 |
| | D | 0.55 | 3.4 | 0.55 | 4.4 | 0.56 | 5.9 | 0.59 | 4.7 |
| Precast Frames 9 | U.U | 0.57 | 1.6 | 0.79 | 1.1 | 0.50 | 9.3 | 0.54 | 15.0 |
| | U.U | 0.43 | 11.1 | 0.63 | 1.4 | 0.41 | 9.9 | 0.41 | 4.7 |
| | U | 0.61 | 3.2 | 0.46 | 1.7 | 0.41 | 0.6 | 0.42 | 2.5 |
| | U | 0.63 | 0.5 | 0.53 | 2.3 | 0.55 | 3.4 | 0.55 | 3.4 |
| | D | 0.93 | --- | 0.93 | --- | 0.87 | --- | 0.85 | --- |
| | D | 1.07 | --- | 0.96 | --- | 0.95 | --- | 0.95 | --- |
| | D | 1.00 | --- | 0.81 | --- | 0.80 | --- | 0.77 | --- |
| | D | 0.83 | --- | 1.00 | --- | 0.77 | --- | 0.78 | --- |
| Large Pnel 9 | U | 0.38 | 2.2 | 0.38 | 1.2 | 0.30 | 2.3 | 0.30 | 0.5 |
| | U | 0.38 | 1.8 | 0.39 | 2.5 | 0.29 | 1.4 | 0.29 | 0.5 |
| | U | 0.46 | 4.5 | 0.40 | 2.6 | 0.46 | 3.4 | 0.45 | 1.8 |
| Monolithic 4 | U | 0.51 | 2.6 | 0.32 | 2.2 | 0.27 | 0.9 | 0.27 | 2.9 |
| Monolithic 16 | U | 1.10 | 7.8 | 1.00 | 6.1 | 0.96 | 1.9 | 1.18 | 1.3 |

--- unknown *1 U:Undamaged D:Damaged U.U:Undamaged and Under Construction

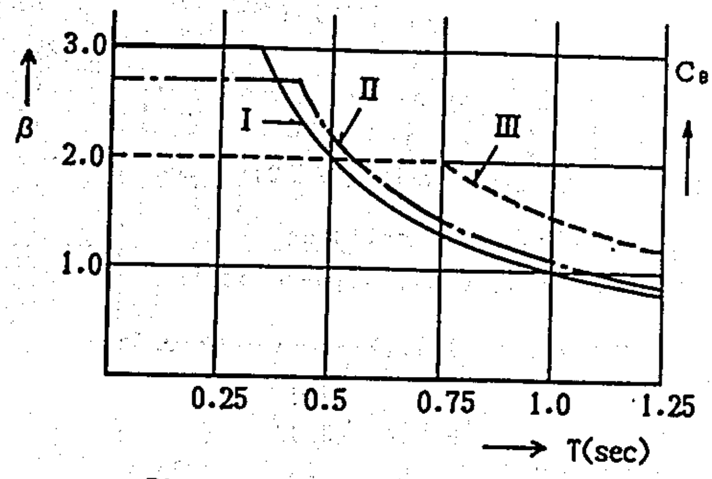



Fig.1 β-$T_1$ (U.S.S.R., 1981)

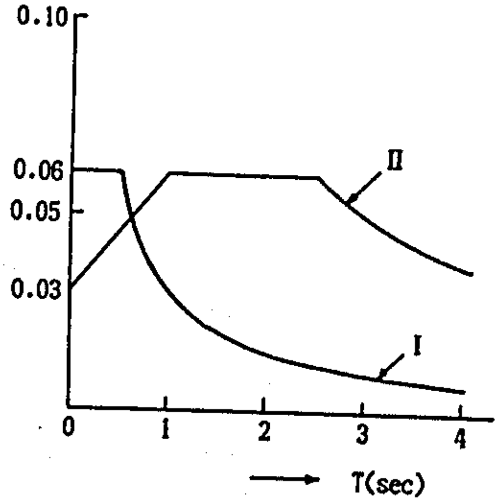



Fig.3 $C_B$-$T_1$ (Mexico, 1966)

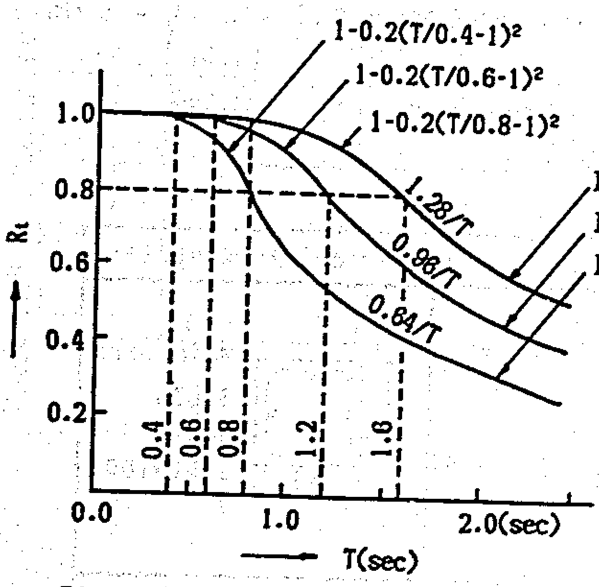



Fig.2 (β/βu)-$T_1$ (Japan, 1981)

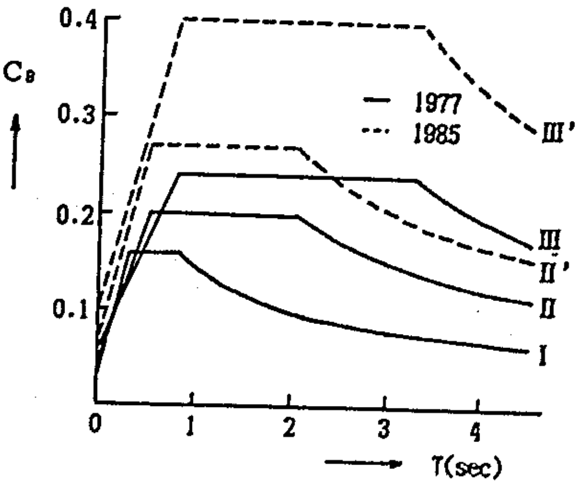
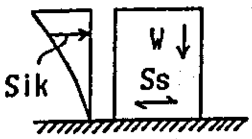



Fig.4 $C_B$-$T_1$ (Mexico, 1977, 1985)




$$C_B = \frac{S_s}{W} = \sum_{k=1}^{n} Sik \Big/ \sum_{k=1}^{n} Q_k$$

[Japan] $\quad C_{BJ} \geq \boxed{Ds} \cdot Fes \cdot \underline{Z \cdot C_0 \cdot Rt} \cdot A_1$

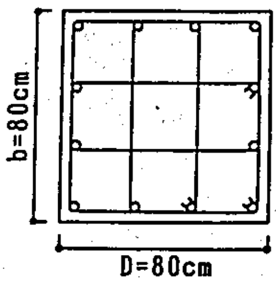
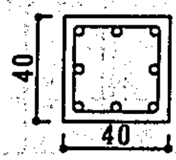
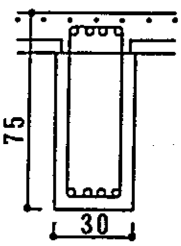
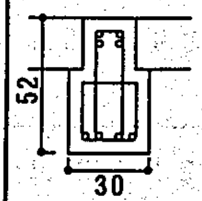
[Armenia] $\quad C_{BJ} \geq \boxed{K_1} \cdot K_2 \cdot \underline{A \cdot \beta} \cdot (\Sigma\, \eta\, ik \cdot Qi / \Sigma\, Qi) \cdot K\Phi$

Example : 9-story precast concrete building

z=1.0 (Japan), I =8 (Armenia), $T_1$=0.6 sec (Type II Soil)

$$C_{BJ} \geq \underbrace{\begin{pmatrix} 0.35 \\ \\ 0.55 \end{pmatrix}}_{\boxed{0.35}} \times 1 \times \begin{pmatrix} 1 \\ 0.9 \\ 0.8 \end{pmatrix} \underbrace{\times 1 \times 1 \times 1}_{\boxed{1.0}} = \begin{pmatrix} 0.35 \\ \\ 0.55 \end{pmatrix} \times \begin{pmatrix} 1 \\ 0.9 \\ 0.8 \end{pmatrix} \Rightarrow 0.35$$

$$C_{BA} \geq \underbrace{0.25 \times 1.4}_{\boxed{0.35}} \times \begin{pmatrix} 1 \\ 0.5 \\ 0.25 \end{pmatrix} \underbrace{\times 0.4 \times 1.1/0.6 \times 0.9 \times 1}_{\boxed{0.66}} = 0.23 \times \begin{pmatrix} 1 \\ 0.5 \\ 0.25 \end{pmatrix} \Rightarrow 0.115$$

| | [Japan] $C_{BJ}$=0.35 | [Armenia] $C_{BA}$=0.115 |
|---|---|---|
| 1st Floor Column |   Main Bars 12-D28 Hoops 4-D10@100 |   Main Bars 8-D28 Hoops 2-D6@100 |
| 5th Floor Beam |   Main Bars 4-D28 20-D13 Stirrups 2-D10@200 |   Main Bars 4-D28 Stirrups 2-D6@200 |

For $S_s/(bDFc) < 0.1$,

$$B(D) = \sqrt{\frac{6m \times 6m \times 1.1t/m^2 \times 9(\text{Floors}) \times 0.35(C_B)}{0.1 \times (Fc = 200kg/cm^2)}} = 79.4 \rightarrow 80cm$$

Fig.5 Rough Comparison of Design Seismic Forces and Member Sections

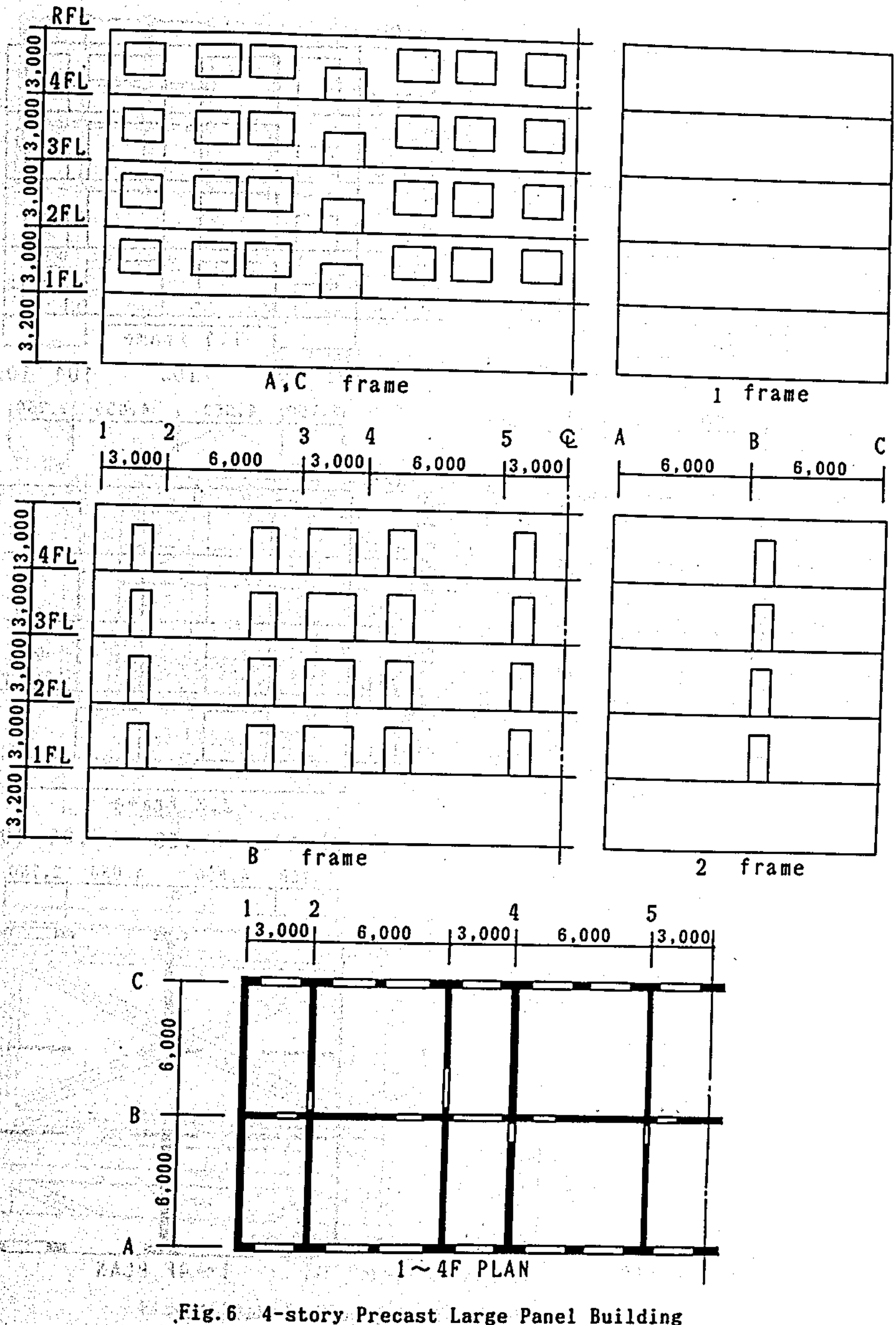



Fig.6 4-story Precast Large Panel Building

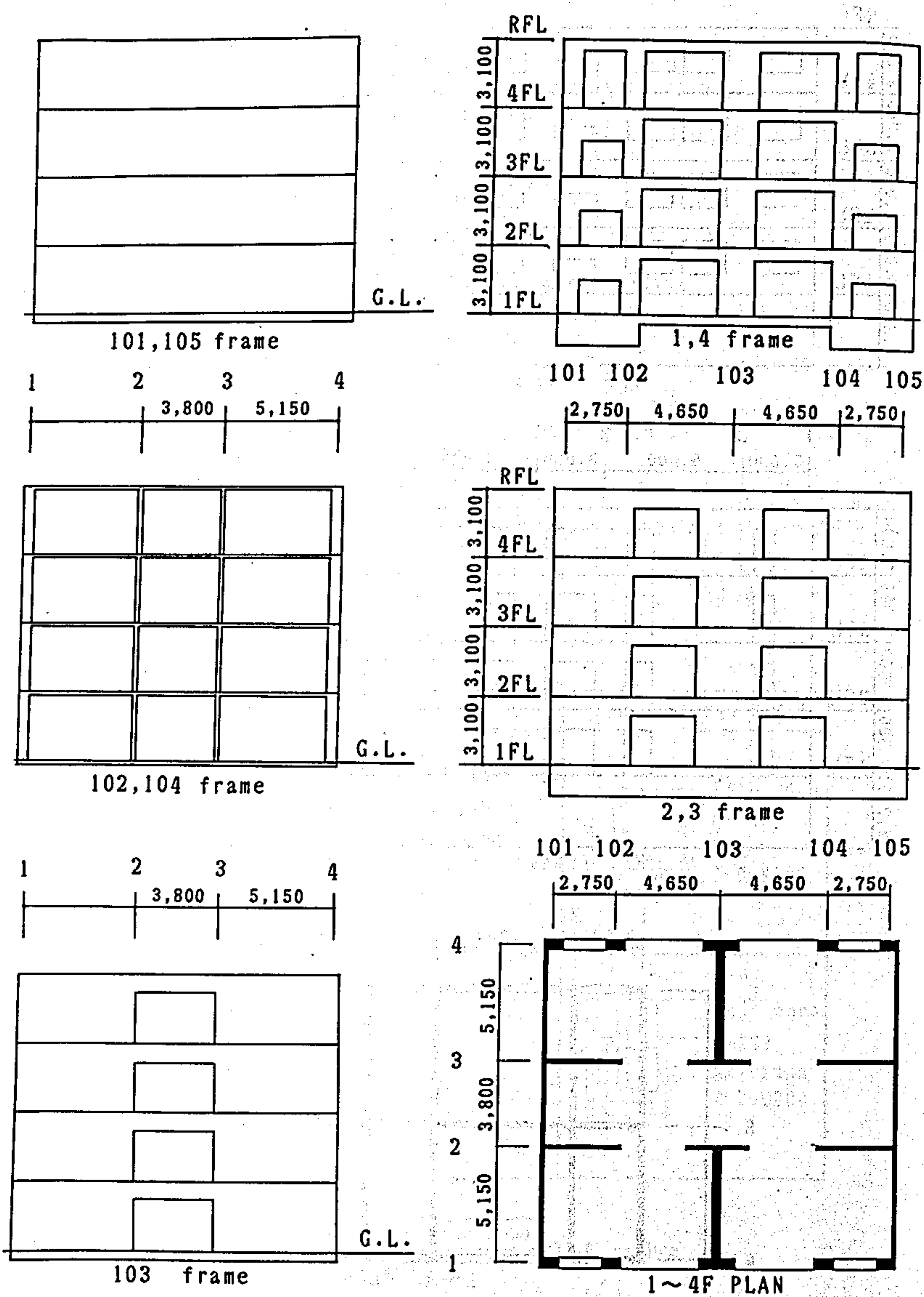
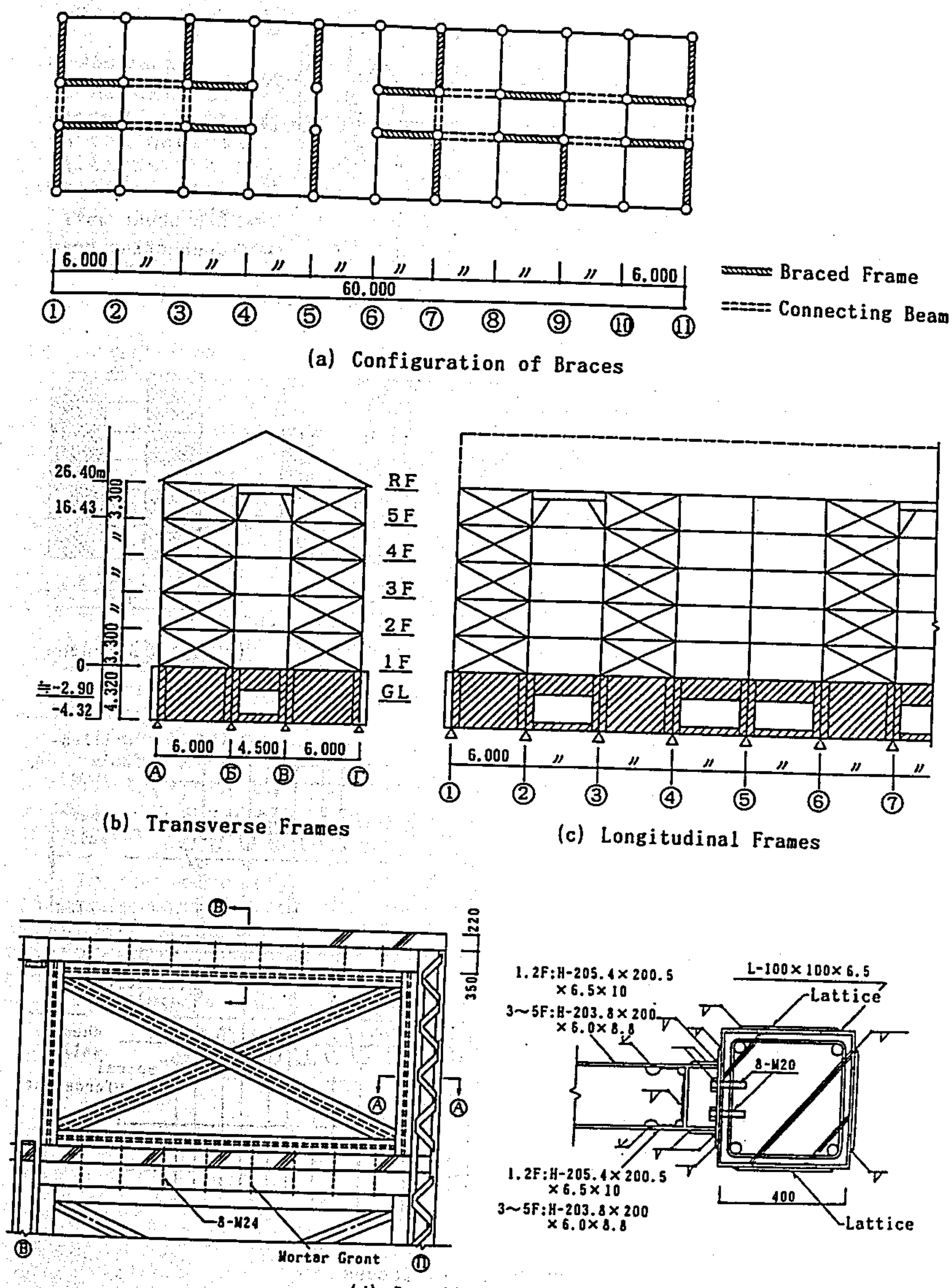



Fig.7 4-story Cast-in-place Wall Building




(a) Configuration of Braces

(b) Transverse Frames

(c) Longitudinal Frames

(d) Detail of Braces

Fig.8 Seismic Strengthening with Steel Braces

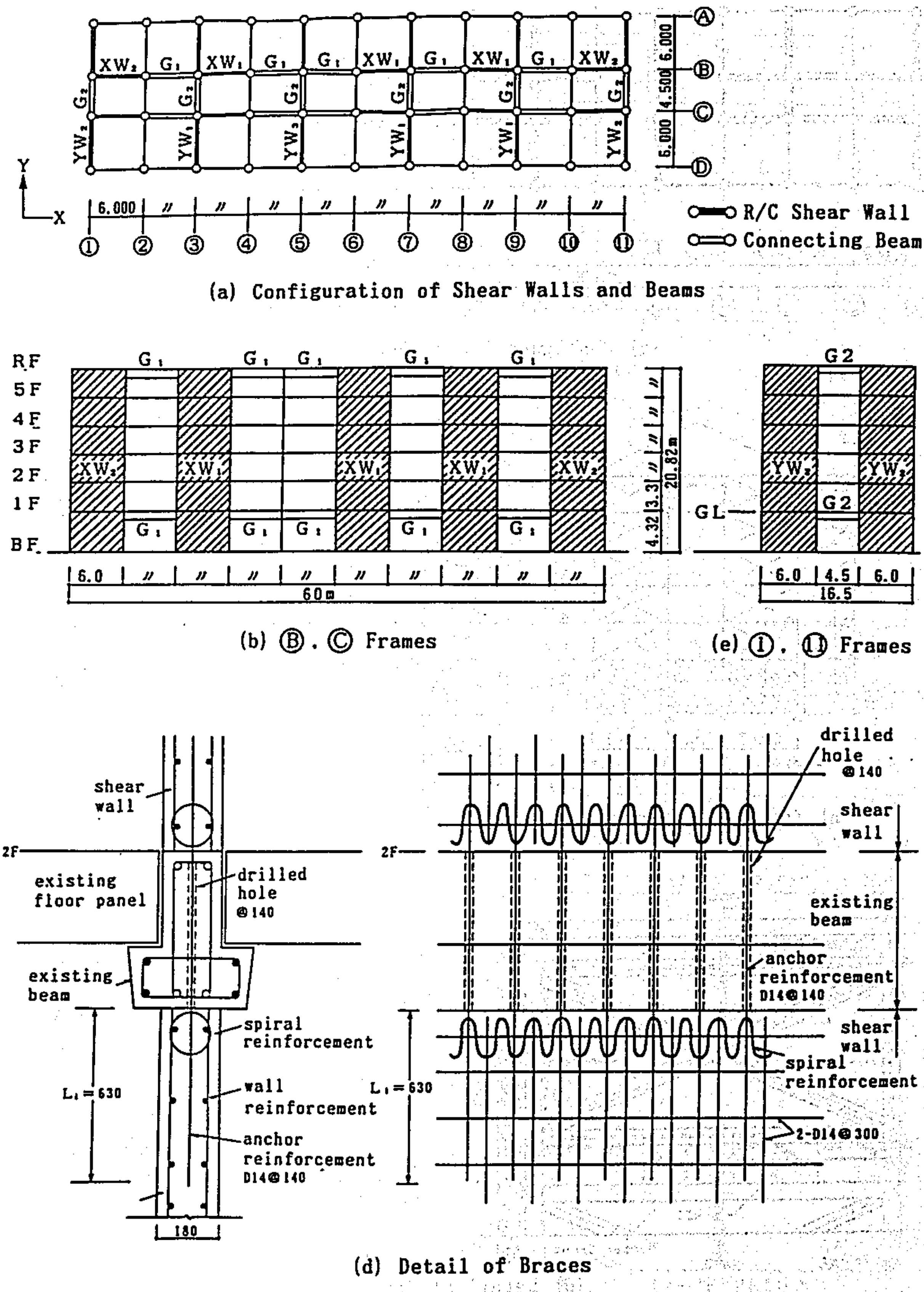



Fig.9 Seismic Strengthening with R/C Shear Walls

## 2. Effects of Ground Conditions of the Damage in the Spitak Earthquake

### 2.1 Abstract

Results of soil investigations in-situ and in the laboratory performed in the earthquake-damaged area are presented, together with the general feature of geological and typographical conditions. A set of conceivable problems on effects of subsurface soil conditions on the amplification of seismic motions is addressed as a basis for further investigation to clarify causes of the destructive damage during the Spitak earthquake.

### 2.2 Introduction

On 7th December 1988, an earthquake with a magnitude 7.0 occurred in the highlands of Caucasus bringing about disastrous damage in the city areas of Spitak, Leninakan and Kirowakan located in the northern region of the Armenian Republic, USSR. The epicenter of this earthquake is reportedly about 10 km northwest of Spitak where the damage was most dreadful. This earthquake generated about 15-km-long east-west trending, northward-dipping zone of surface rupture along the fault line starting from the south part of Spitak as shown in Fig. 1. The faulting is of right-lateral type with a maximum vertical offset of 2 m at the middle point of the main rupture line. As is often the case, the area located on the uplifting part of fault movement suffered a greater level of damage as compared to the area on the opposite side. In the Spitak earthquake, the city of Spitak and small villages in its vicinity located north of the fault line appears to have experienced accordingly a stronger shaking and hence heavier damage. The city area of Leninakan is an enclave where the damage to buildings was very severe. The reasons for the concentration of the damage in this city are not completely known.

In the severely affected areas, most of the damage were to old unreinforced rubble-stone masonry wall buildings and to newer 5-9 storey multiple dwelling apartment units constructed of reinforced concrete frame with infill masonry walls. However, the degree of damage to each type of these buildings appears to be governed, by and large, by the subsurface soil conditions and

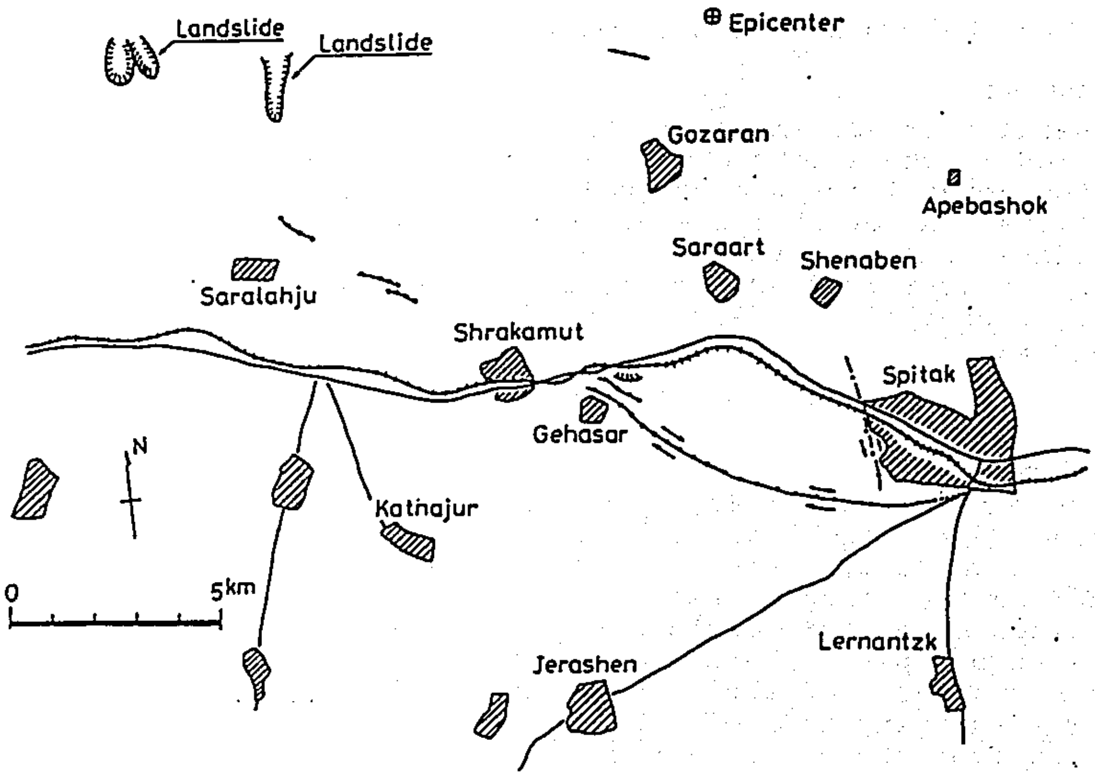



Fig. 1 Faults in the Spitake area

the topographical regime in the affected region. Thus, while the author stayed in Armenia as a member of the Japanese mission, efforts were made to investigate the soil conditions both in-situ and in the laboratory and to collect information on the geological setup of the damaged area. The objectives of this paper are to present a set of raw data on soil conditions hitherto obtained and to address some conceptual problems associated with effects of the ground conditions on the damage feature of the Spitak earthquake.

### 2.3 Outline of Geological Setup and Soil Conditions

The earthquake was centered in the highland region where there is a predominance of rocks and soils of Tertiary volcanic origin, principally from the Miocene age. The large expanses of the affected area is covered by soils derived from weathering of basaltic rock of igneous origin and of sedimentary tuff deposit.

#### (1) Spitak

The city of Spitak develops in a widely opened area near the junction of the Pamback river and its tributary streams as shown in Fig. 2. The worst-hit city area consists of low-lying portion where the Pamback river flows easterly and of terrace portion on the north and south sides of the city center. No boring data is available in Spitak, but visual observation of the exposed strata

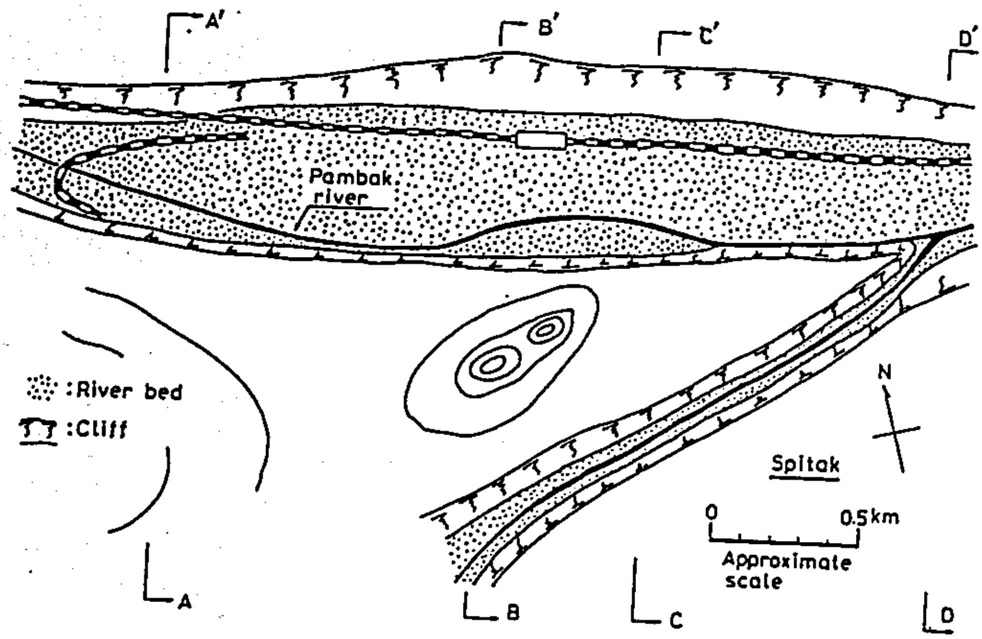



Fig. 2 (a) Plan view of Spitak

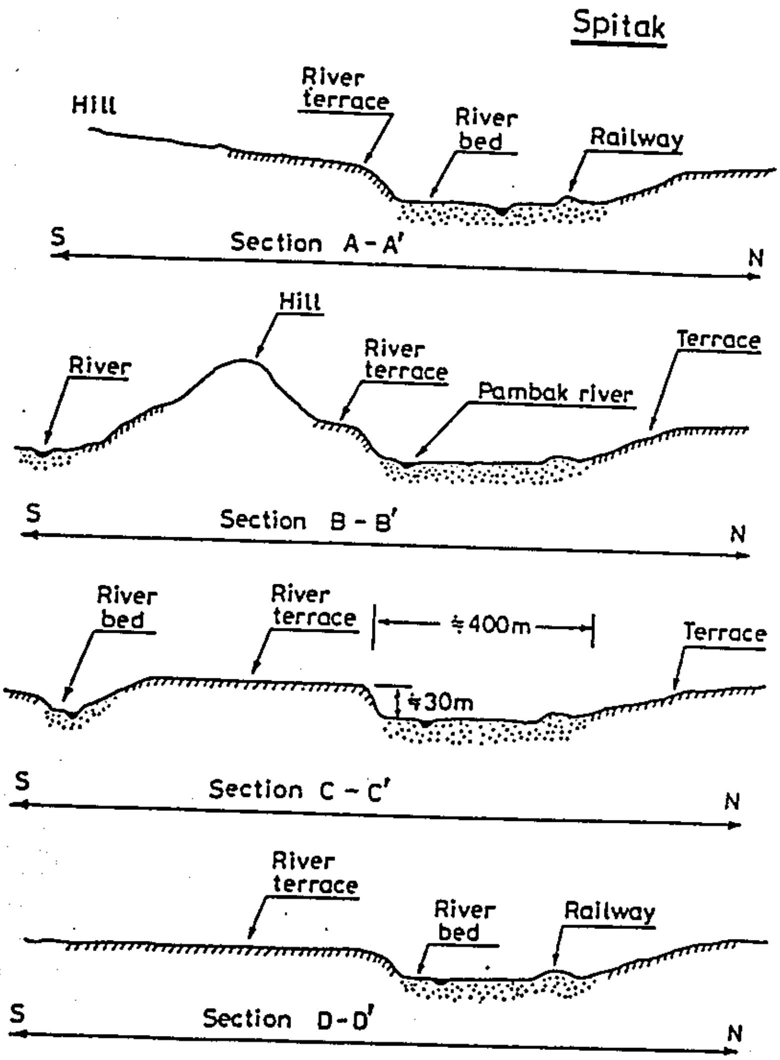



Fig. 2 (b) Cross-sectional views in Spitak on the cliffs shows that the terrace is formed of the black-colored basaltic rock overlaid by layers of gravel-rich soils which appears to have been derived from the disintegration of the mother rock. The low-lying riverbed area appears to have been formed as a result of erosion by the Pamback river. The ground surface in this zone is predominantly covered by black-colored silt or clay with high plasticity. The strata below this surface soils consist of gravelley soils underlaid by the hard basaltic rock.

(2) Leninakan

The city of Lininakan is reported to lie over thick lacustrine deposits which had been formed over the Oligocene and Miocene era through natural sedimentation in a bowl-shaped basin surrounded by hills (Saakyan, 1964). The depth of the buried lake deposit is reported to be 250 to 300 m and this deposit is said to exist in a wide area beyond the city limit. Overlying the lacustrine deposit is layers of diluvial and alluvial deposits composed mainly of gravel, sand and silt. In the northern part of Leninakan, there exists a mantle of tuff deposits covering the gravelly sand layers of diluvial origin. The structure of subsurface geology in the city of Leninakan is described by Babayan et al. (1974), as shown in Fig. 3, where it

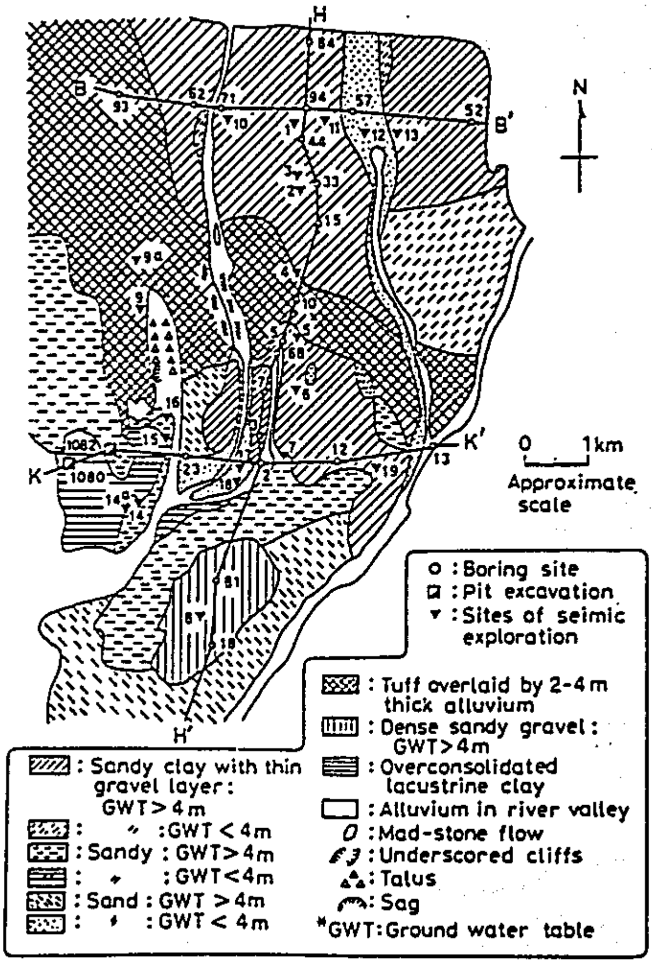



Fig. 3 Subsurface geology in Leninakan (Babayan et al. 1974)

may be seen that sand-rich deposits prevail generally in the southern part whereas gravelly deposits exist in the northern portion of the city. It is to be noted that the sedimentary tuff deposit 10 to 15 m thick extend toward southeast from northwest. In the south of the city, there exists dense sandy gravel deposit. The cross section in the east-west direction through the city center as indicated by k-k' in Fig. 3 is demonstrated in Fig. 4, where it may be seen that there exist layers of gravel-containing deposits of varying thickness to a maximum depth of about 20 m.

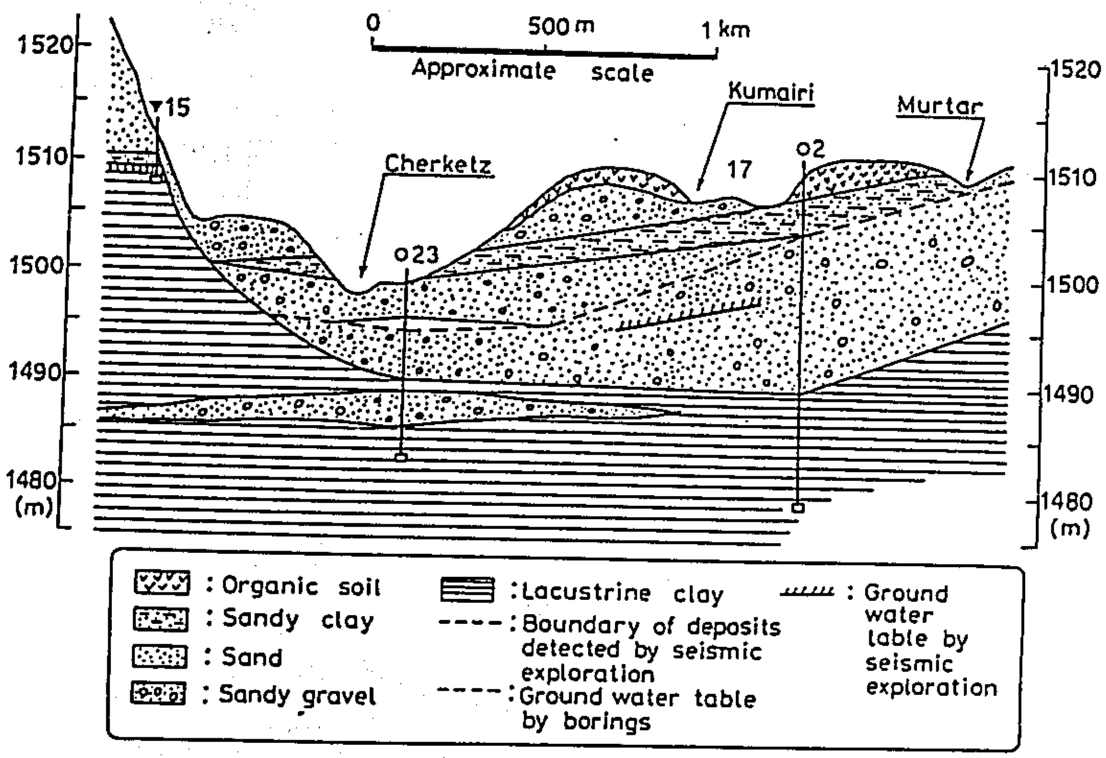



Fig. 4 Cross sectional view of soil profile in Leninakan (Babayan et al. 1974)

By means of the refraction method of elastic wave propagation, a series of in-situ tests were performed by Babayan et al. (1974) to determine the shear wave velocity of the soil deposits in the city of Leninakan. The results of this test are shown in Table 1.

To see the soil profile to deeper deposits, data on deep borings were collected as summarized in Fig. 5. Individual sites designated by No. 1, No. 7 and Ba are indicated in Fig. 6. Fig. 5 indicates that the lake deposit pinches out as the ground elevation rises toward the north. It can also be seen in Fig. 5 that there exists the basaltic rock formation underlying the lake deposit.

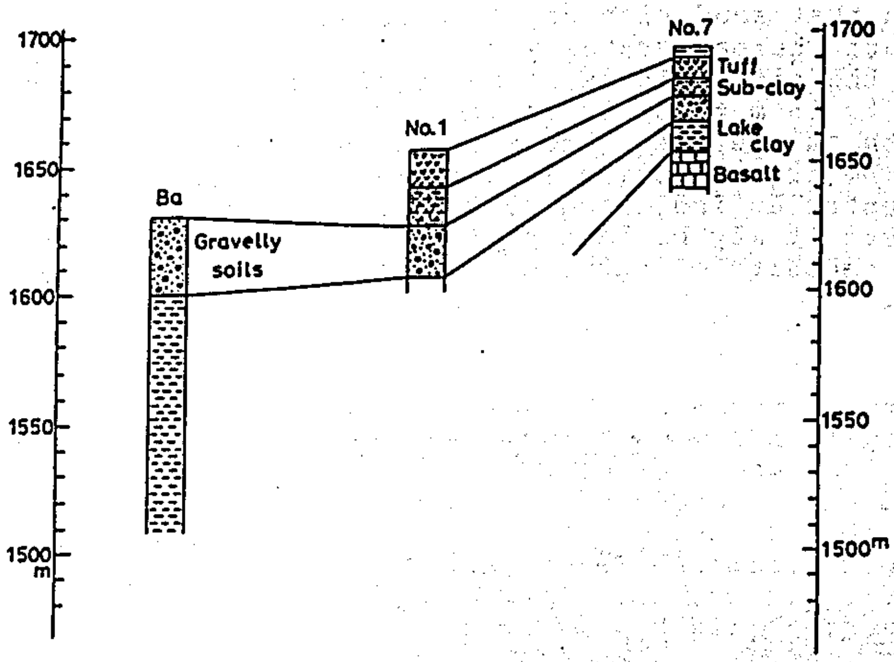



Fig. 5 Deep boring data in Leninakan

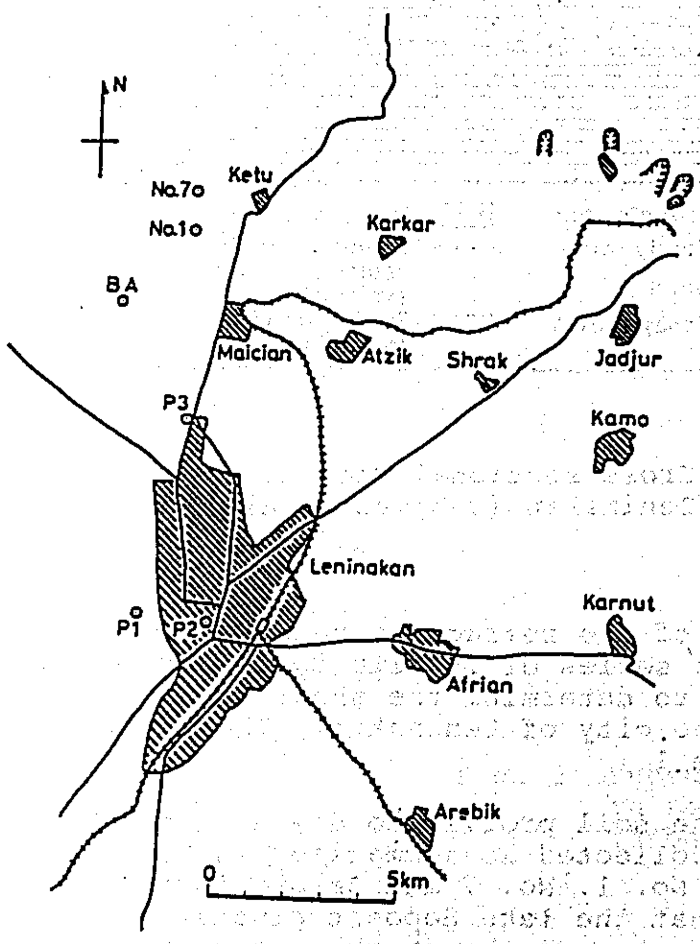



Fig. 6 Locations of deep borings and penetration tests in Leninakan

Table 1 Soil characteristics in the city area of Leninakan (Babayan et al. 1974)

| Materials | Site of observation* | $V_p$ (m/sec) | $V_s$ (m/sec) |
|---|---|---|---|
| Hard tuff | 9, $9^a$ | 1600 ~1800 | 720 |
| Weathered tuff | $9^a$ | 530 | 163 |
| Lacustrine clay | $14^a$, 15, 16 | 1440 ~1760 | 450 |
| Unsaturated lacutrine clay | 14 | 680 | 390 |
| Sandy clay | 13 | 830 | — |
| Unsaturated sandy clay | 1~4, 6, 7 10~13 | 300 ~500 | 145 ~340 |
| Saturated sandy clay | 3 | 2150 | — |
| Unsaturated sandy soil | 5, $14^a$, 15 16 | 290 ~560 | 165 |
| Hard dry sand | 4, 8, 18 | 660 ~700 | 280 |
| Sandy gravel | 11 | 1500 ~1520 | 1000 |
| Fills | 2, 6, $14^a$ | 270 | 160 ~170 |

* Refer to map

(3) **Kirovakan**

There are about 20 boring data showing soil profiles at different places through the city area. All of these data show that the ground in Kirovakan is composed of gravelly soils about 2 to 5 m thick underlaid by the basaltic rock. The ground conditions are generally good and suitable for directly placing building foundations as they have been done thus far.

(4) **Stepanavan**

No boring data is available in the city area of Stepanavan. However, judging from the rock outcrop on the cliffs of deep canyon, it is obvious that rock formation comes up to the surface in the downhill area near the river. In the uphill area to the south, the thickness of soil strata seems to become large, helping to amplify the motions during the earthquake.

2.4 In-situ Investigations of Subsurface Soil Conditions

In view of an important role played by the lake deposit in amplifying the seismic motions, efforts were made to measure the stiffness of the lacustrine clay in the field by means of a portable cone penetration test apparatus. This is a simple manually operated device to monitor the force required to penetrate a cone tip into the ground. The cone has an appex angle of 30 degree and a cross sectional area of A = 3.23 cm², as illustrated in Fig. 7. The force is measured by a proving ring equipped beneath a hand hold. The resistance of cone penetration, $q_c$, is defined as

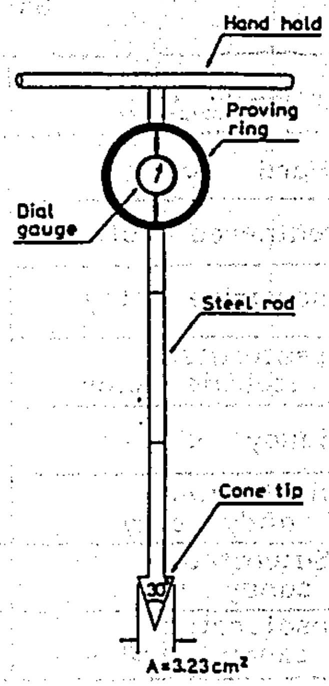



Fig. 7 Portable hand cone

$$q_c = \frac{Q}{A} \qquad \text{..... (1)}$$

where Q is the measured force in kgf and A is the cross sectional area in cm². Since the cone resistance is nothing but an indirect measure of soil stiffness, the $q_c$-value as defined above must be converted to some physically meaningful index parameters of soils. A parameter of engineering significance that can be derived from the cone resistance would be the velocity of shear wave propagation through soils. There has been no reported study regarding the correlation between the cone resistance and shear wave velocity particularly for clayey soils. However, some empirical correlations have been proposed between the shear wave velocity, Vs, and the blow count (N-value) in the standard penetration test. For diluvial clay, Imai (1977) proposed a correlation,

$$Vs = 114\ N^{0.294} \qquad \text{..... (2)}$$

For stiff clayey soils, the relationship between $q_c$-value and N-value has been known to be approximately,

$$N \fallingdotseq q_c/2 \qquad \text{..... (3)}$$

Thus, introducing this relationship into Eq. (2), one obtains

$$Vs = 93\ q_c^{0.294} \qquad \cdots\cdots (4)$$

This relationship will be used in the present study to approximately assess the shear wave velocity of soils from measured values of the cone resistance.

The penetration tests were performed at three sites indicated in Fig. 6 by P1, P2 and P3 in the suburb of Leninakan. The first site P1 is located about 200 m east of the river channel in the flood plain of the Afurian river. Intact surface of the lacustrine deposit was exposed by excavating a pit by means of a power shovel and the penetration test was performed at the bottom of the pit. In the vicinity of the pit, an exposed surface of the lake deposit was found and the cone penetration test was also conducted in this location. The material is clayey silt with occasional gravels which appears to have been derived originally from a series of volcanic activities in the mountains in this region. A grain size distribution curve of this material is shown in Fig. 8. The results of the cone tests are shown in Table 2, where it can be seen that the point resistance, $q_c$-value, is on the order of 20 to 25 kg/cm². The equivalent N-value in the standard penetration test (SPT) is supposed to be within the range of 10 to 15 in accordance with the correlation expressed by Eq. (4).

Table 2 Results of penetration tests in Leninakan

| Soil type | Site | qc - value (kgf/cm²) | N - value in SPT | Vs (m/sec) |
|---|---|---|---|---|
| Lacustrine clay in Leninakan | Afurian river site , P1 | 20 ~ 25 | 10 ~ 15 | 300 ~ 400 |
| Basalt - derived black clayey silt | Site at city center , P2 | 10 ~ 15 | 5 ~ 7 | 100 ~ 200 |
| Limestone - derived clayey silt | Relocation site , P3 | 25 ~ 35 | 15 ~ 18 | 350 ~ 450 |

The second site P2 is located at the road side one block east of the city square in Leninakan. Two pits about 2 m deep excavated for pipe line inspection happened to be there. The penetration tests were conducted at the bottom of the pits which approximately coincide with the elevation of the ground water table. The $q_c$-value was about 10 to 15 and the equivalent N-value would be 5 to 7, as accordingly shown in Table 2. The material is silty clay with dark blown color.

The third site P3 is located in the northern outskirt of Leninakan which is chosen to be the land for relocating the appartment complex destroyed by the earthquake. About 2 m deep excavation was underway for the foundation work of buildings. The material is clayey silt derived probably from weathering of limestone. The $q_c$-value was as high as 25 to 35 with an estimated N-value of about 15 to 18, as indicated in Table 2.

## 2.5 Physical and Mechanical Properties of Representative Soils

In the earthquake-affected region, the soils of geotechnical significance that can easily be tested in the laboratory are the black plastic clay from the riverbed area in Spitak and the silty clay from the lake deposit in Leninakan. These soils were tested in the laboratory to obtain data on their physical and mechanical properties. The results of the laboratory tests are summarized in Table 3. The physical property data on the lake clay referred to as USSR were kindly offered by a geologist who was engaged in the reconstruction work in Leninakan. The data on the velocity of wave propagation of the lake deposit in Leninakan were quoted from the data in Table 1. The other test data are those obtained by Kisojiban consultants of Japan.

### (1) Black Plastic Clay

The test sample was obtained from the riverbed at Spitak. This black clay derived from weathering of basaltic rock is shown in Table 3 to have a plasticity index of 27 which is high enough to show apparent cohesiveness and stickiness in the sense of touch. This type of soil exists in a form of silt or clay mixed with sand and gravel over a widespread area devastated by the Spitak earthquake from Kirowakan westwards to Leninakan. It was a surprising fact that visible signs of liquefaction were not observed in any of the low-lying plains near the river channels. According to experiences in many of large-scale earthquakes in the past, liquefaction was of frequent occurrence in sandy deposits in the low-lying area where the ground water table is generally high. In the flood plain along the Pamback river between Spitak and Shrakamut (Fig. 1), there are many places where liquefaction is likely to occur because of the predominance of sand contained in silt and clay. Nonetheless, evidence of liquefaction was not found as far as the author walked around. In the downtown area in the city center of Leninakan, sand-rich deposits are known to exist, as shown in Fig. 4, with a high elevation of the ground water table. It is no wonder that signs of liquefaction were observed in this area, but this was not actually the case. The non-occurrence of liquefaction as above in the most seriously affected area in the Spitak earthquake can thus be attributed, with good reasons, to the fact that the fraction of clay and silt contained in the sand possesses highly cohesive characteristics as represented by a plasticity index as high as 27.

(2) **Lacustrine Clay in Leninakan**

The test sample was secured from the bottom of the excavated pit and has a grain size distribution curve shown in Fig. 8. The clay is over-consolidated and cemented. The plasticity index is 34 as shown in Table 3. The void ratio of undisturbed samples are 1.45 and 1.18. The shear wave velocity, Vs, measured in-situ by the method of elastic wave refraction is reported by Babayan et al. (1974) to be 450 m/sec as indicated in Table 1, whereas the shear wave velocity measured in the laboratory by transmitting ultrasonic waves through undisturbed samples shows a value of 365 m/sec, and the Vs-value estimated from the $q_c$-value renge between 300-400 m/sec, as indicated in Table 2. The values of shear wave velocity obtained from three different methods are summarized in Table 4 for comparison sake where it may be concluded that the shear wave velocity of the lake deposit in Leninakan does probably possess a value between 300 and 450 m/sec with a best estimated average of 350 m/sec.

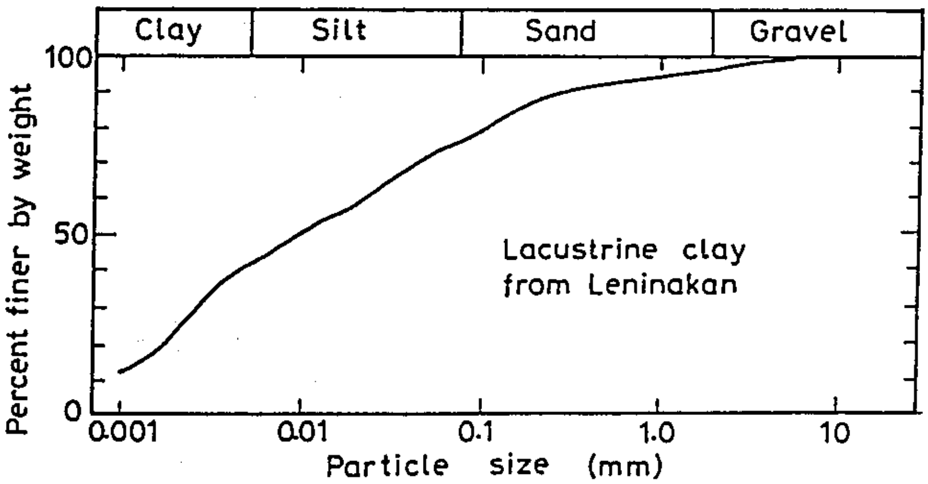



Fig. 8 Gradation curve of the lake clay

In addition to the above, a resonant column test was conducted on an undisturbed sample trimmed into a hollow cylindrical shape. The strain-dependent modulus and damping characteristics of the lake clay obtained in the test is demonstrated in Fig. 9, together with a typical average curve established thus far for clayey soils. In Fig. 9, the shear modulus, G, at a certain strain level is shown normalized to the shear modulus, Go, at a small strain of $10^{-5}$. It is to be noted

Table 3 Properties of two key soils

| Index property | Weathered basalt (black) | Lacustrine clay in Leninakan (Cream-colored) | |
|---|---|---|---|
| | Japan | USSR | Japan |
| Specific gravity, Gs | 2.65 | 2.62 | 2.74 |
| Liquid limit, $\omega_L$ | 51 | 75 | 64 |
| Plastic limit, $\omega_P$ | 24 | 41 | 30 |
| Plastic index, Ip | 27 | 34 | 34 |
| Unit weight $\gamma_t$ (kN/m³) | —— | 17.5 | 16.1 |
| Void ratio e | —— | 1.45 | 1.18 |
| Vs (m/sec) | —— | 450 ** | 365* |
| Vp (m/sec) | —— | 1440 ** ~1760 | 858* |
| Poisson's ratio | —— | 0.28** | 0.39 |

* By Ultrasonic method in the lab.
** By in-situ wave reflection method (Babayan et al.)

Table 4 Values of shear wave velocity for the lecustrine clay in Leninakan

| Test method | Shear wave velocity Vs (m/sec) |
|---|---|
| In-situ tests by elastic wave refraction | 450 |
| Laboratory tests by Ultrasonic wave transmission | 365 |
| Estimated from in-situ cone penetration tests | 300 ~ 400 |

that the shear wave velocity obtained at the lowest strain level in this test was 165 m/sec, a value much smaller than any value obtained by other methods listed in Table 4. This is probably because of the sample disturbance incurred during the trimming of the hollow cylindrical specimen. Thus, the data of shear wave velocity obtained in this test should be discarded. However, the strain dependency of modulus and damping as shown in Fig. 9 which coincide with the average curve from many other data on clays may represent actual behavior of the lacustrine clay undergoing the shaking during the earthquake

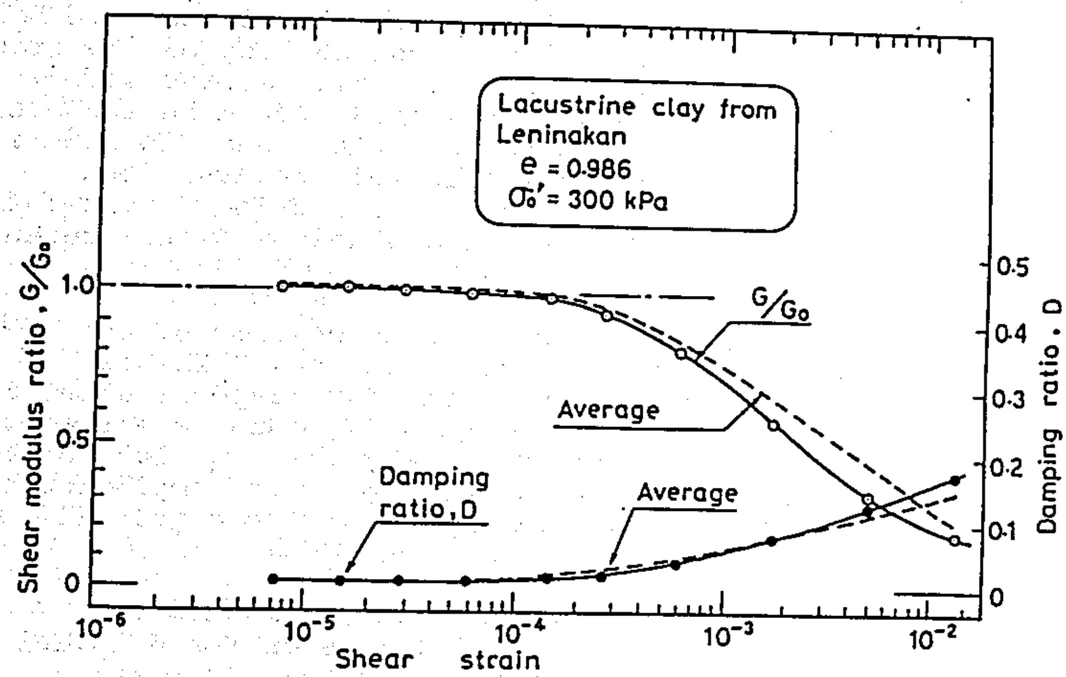



Fig. 9 Strain-dependent shear modulus and demping ratio for the clay in Leninakan

## 2.6 Ground Conditions and Damage Potential

It is generally recognized that the ground conditions are associated with the damage potential due to earthquakes in two ways as follows.

i) Ground failure-induced damage.

When the ground itself fails, accompanied by large deformations or flow of soils, the structures or facilities resting thereupon will be involved in fatal damage. Occurrence of liquefaction in the level ground and landslide on sloping grounds are well known examples of this type of damage.

ii) Strong shaking-induced damage.

If the predominant period of a structure matches a period of significant motions of the ground during earthquakes, the structures will experience intolerably large level of shaking leading to fatal damage. Thus, estimate of site-specific frequency-dependent amplification characteristics of the ground will constitute an important consideration for earthquake resistant design of structures at a given site.

In the case of 1988 Spitak earthquake, scarcely encountered was the type of damage caused by the failure of the ground. Most of the collapse or tumbling of buildings are likely to have been associated with a characteristically strong shaking of the ground during the earthquake. Thus, the amplification characteristics of soil strata specific to the affected areas would be the main issue to be addressed in relation to the extent and distribution of the damage in the affected areas. In this regards, the following remarks may be made for the investigation in the future although more or less conceptual and speculative at the present time.

(1) In the city of Leninakan, a thick deposit of the lacustrine clay exists on the basaltic base rock. In the southern part around the city center, the lake deposit about 300 m thick is overlaid by a sequence of gravelly sand layers about 20 m in total thickness. It appears likely that the lake deposit with a shear wave velocity of 350 m/sec might have amplified the incipient motions in conjunction with the overlying gravelly sand deposit.

(2) In the northern part of Leninakan, a mantle of tuffaceous deposit about 5 to 15 m thick lies on the diluvial gravelly sand deposit which is underlaid by the 300 m thick lacustrine deposit. Effects of the presence of the hard tuff (shear wave velocity is 780 m/sec) on rather soft deposits have not been fully understood. It appears likely that the presence of a stiff layer on a thick soft deposit might have acted towards eliminating high-frequency component and amplifying a long-period component of incipient motions. The coincidence of the long-period of the ground motions with the predominant period of 9-storey high buildings appears to have lead to the disastrous consequence in the apartment complex in Leninakan.

(3) The city of Leninakan lies on a burried lake having a distance of about 3 km in the east-west direction and a north-south distance of about 5 km. The western edge of the lake deposit outcrops on a hill just west of the Cherketz river (see Fig. 4). Therefore, the boundary between the lake deposit and the underlying rock at its western edge appears to dip sharply to the east. The presence of the inclined boundary as above might have acted toward concentrating the reflected waves in some particular section of the city.

(4) In the city of Spitak, the terrace area above the cliffs appears to have suffered by far greater level of damage to buildings as compared to the damage in other areas in its

neighborhood. Thus, the topographical prominence seems to have produced a stronger shaking, leading to the cataclysmic destruction in this area.

(5) In Stepanavan, while the damage in uphill area was much notable, the downhill area near the river canyon has experienced practically no damage. This is probably because of the amplification of earthquake motions which took place in the uphill area due to the presence of a thick soil stratum above the rock formation.

(6) In most of the 9-storey buildings, damaged or undamaged, the foundations consist of isolated square footings 2.5 m x 2.5 m in size connected poorly with underground beams. The pre-fabricated reinforced concrete blocks are carried to construction sites and placed on smoothed surface of gravelly soil deposits which have been exposed by removing the top soils to a depth of about 2 m. The gravelly soil deposits appear to provide a competent basis for supporting the overlying 9-storey buildings. In fact, there was no discernible evidence of building damage that is ascribable to excessive settlements or failure of the foundations of buildings in any city of the affected area during the Spitak earthquake. However, this observation should not be considered as an encouraging evidence to support the present-state-of-the-practice in the foundation design. When the design code for superstructures is modified so that they possess higher rigidity as a whole unit, the foundations will become a weakest point in the overall mode of deformation during seismic shaking and thus there is a possibility of buildings collapse being initiated from the foundations. In view of such situations, the present practice of foundation design should be revised in parallel with the modification of the design code in superstructures.

## 2.7 Concluding Remarks

The damage to buildings during the 1988 Spitak earthquake seems to be associated with unusually notable amplification of seismic motions which is dependent upon local soil conditions underlying the affected area. In Leninakan the presence of a thick deposit of lacustrine clay and a mantle of hard tuff appears to have generated a series of site-specific ground motions which had devastating effects on the performances of buildings. ,In Spitak the effects of topographical prominence were manifested as exemplified by the heavy damage concentrated on the area above the cliff in the southern part of the city. There was no visible signs of liquefaction and associated damage even in low-lying areas where the ground water table is generally high. The reason for non-occurrence of liquefaction is attributed to the highly cohesive and plastic nature of fine-grained soils prevailing over the large expanses of the affected area.

## 2.8 Acknowledgements

The field work in Armenia was conducted with the overall cooperation of the Institute of Geological Science in Yerevan and the institute of Geology and Engineering Seismology in Leninakan, Armenian Academy of Science. Mr. I. Zasursky from Presidium of Academy of Science of USSR offered kind assistance in all phases of investigation. Soil testing was performed by Mr. I. Morimoto of the Kisojiban Consultants in Japan. The author wishes to express his sincere gratitude to the organizations and individuals as mentioned above.

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## 3. Required Strength Ratio Due to Accelerograms Observed in Armenia

### 3.1 Abstract

Nonlinear response analyses of single-degree-of-freedom structural systems were carried out by using the idealized hysteretic models and the accelerograms observed in Armenia. The relationships among peak acceleration, strength level and ductility factor were obtaind in order to discuss the cause of damege to reinforced concrete buildings. The obtained results were considered to show that the insufficient strength and/or ductility capacities compared with the amplitude level of the accelerograms would have caused the collapse of many buildings.

### 3.2 Introduction

Performance and damage assessment of buildings is conducted by estimating their strength and ductility capacities and considering the intensity and characteristic of ground motions. If ground motion characteristics and hysteretic models are not changed, and the levels of peak acceleration and strength alone are variables, the peak acceleration, strength and ductility factor are connected each other so that one of them is obtained by determining the other on nonlinear response analysis for single-degree-of-freedom systems. The strength level normalized by the product of mass and peak acceleration is called herein the required strength ratio corresponding to ductility factor.

### 3.3 Structural Model

The basic model selected herein to illustrate structural response is the single-degree-of-freedom system in Fig. 1(a). This model has a linear viscous dashpot and a nonlinear hysteretic spring. The spring force is therefore prescribed with nonlinear function F(v) of relative displacement v(t). The principal quantities to characterize this function for monotonic loading are $p_c$, $p_y$, $v_c$ and $v_y$ as shown in Fig. 1(b). The loads $p_c$ and $p_y$ represent the spring restoring forces corresponding to concrete cracking and ultimate strengths, respectively, and $v_c$ and $v_y$ are corresponding displacements.

Hysteretic characteristics of the model are defined by the Degrading Trilinear model recommended for reinforced concrete structures by Umemura and his associates, where nonlinear deformations and failure characteristics are primarily controlled by flexure (ref.1). This model shown in Fig. 2 is characterized by four factors $p_{Bc}$, $p_{By}$, $v_{Bc}$ and $v_{By}$. They represent the load at concrete cracking, the load at which main reinforcing bars start to yield, both due to flexure, and the relative displacements produced by $p_{Bc}$, and $p_{By}$, respectively. Linear elastic behavior (without hysteretic loops) always takes place for oscillatory displacements where the corresponding loads are in the

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range $-p_{Bc}<p<p_{Bc}$, however, hysteretic behavior occurs with every cycle of deformation which has the load levels above $p_{Bc}$ or below $-p_{Bc}$. During that period of time between the initiation of loading and that instant at which the relative displacement first increases above $v_{By}$ or decreases below $-v_{By}$, the trilinear model behaves exactly like the standard bilinear hysteretic model having the stiffnesses $k_1$ and $k_2$ (QPOAB, Fig. 2(a)). However, as soon as the displacement increases above $v_{By}$ or decreases below $-v_{By}$, a new bilinear relation controls the response. For example, suppose the displacement for the first time increases above $v_{By}$ to level $v_{max}$ as represented by C in Fig. 2(a). Upon decreasing the displacement from this level, the corresponding load decreases along the path CD which has a slope equal to $\alpha k_1$, where

$$\alpha = \frac{2\, v_{By}}{v_{max} + v_{By}}$$

As soon as the load drops by the amount $2p_{Bc}$ reaching the point D in Fig. 2(a), any further drop in load will follow the continuing path having a slope $\alpha\, k_2$. It should be noted that the point D is located at the load level $p_{Bc}$ in Fig. 2(a) because the particular trilinear model in the figure is represented for the case $p_{By}/p_{Bc}=3.0$. If this ratio has been assigned a different numerical value, the load level at the point D would be different from $p_{Bc}$.

The new bilinear hysteretic model controlling continuing motion is shown in Fig. 2(b). Note that the origin of the skeleton curve is shifted from the point 0, the origin of the original bilinear hysteretic model, to the point 0′. This point is the intersection of the line QC and the abscissa axis in Fig. 2(a), therefore, the line 00′ is equal to BC/2. The stiffnesses of the new bilinear model are $\alpha\, k_1$ and $\alpha\, k_2$.

If during the period of response controlled by the second bilinear model (Fig. 2(b)), the displacement should increases beyond $v_{max}$ ( $v_{max}=v_{By}'$ ) as represented by the point B′ to a new level as represented by C′, the continuing response would be controlled by the third bilinear model whose characteristics could be obtained in exactly the same manner as that for the second model. Also if yielding of the trilinear model has taken place at the load level $-p_{By}$ rather than the load level $p_{By}$, the new bilinear model to control continuing motion would be obtained by a similar procedure.

The degrading trilinear model is completely characterized by any four of the seven parameters $k_1$, $k_2$, $k_y$, $p_{Bc}$, $p_{By}$, $v_{Bc}$ and $v_{By}$ shown in Fig. 2. It is convenient to use $k_1$, $k_y$, $p_{Bc}$ and $p_{By}$ substituting the period parameters $T_1$ and $T_2$, defined by $T_1=2\pi\sqrt{(m/k_1)}$ and $T_2=2\pi\sqrt{(m/k_y)}$, for $k_1$ and $k_y$, respectively. For reinforced concrete members, $k_1$ and $p_{By}$ usually fall in the ranges $2k_y<k_1<4k_y$ and $2p_{Bc}<p_{By}<3p_{Bc}$, respectively, therefore, for a given design one can specify the numerical values for ratios $k_1/k_y$ and $p_{By}/p_{Bc}$, which reduces the number of independent parameters to two. For the example case presented herein, the two parameters are $k_1/k_y=2$ and 4, and $p_{By}/p_{Bc}=3$ ($T_2=\sqrt{2T_1}$ and $2T_1$). The period $T_1$ and the strength $p_{By}$ can now be used as the two independent model parameters. It is convenient to normalize the strength parameter $p_{By}$ dividing by the force $mv_{go}$ where $v_{go}$ is the peak acceleration of ground motion, thus, the final results are independent of peak acceleration.

Viscous damping in the structural model is controlled by prescribing the numerical value of the damping ratio $\xi_1$ defined by $\xi_1=c/2\sqrt{m/k_1}$. A value of 2% of critical, $\xi_1=0.02$, has been selected for the example presented herein.

### 3.4 Accelerograms

In this investigation each of two horizontal components of the accelerograms of main shock and aftershock observed at Gukasian and one horizontal component of the record at Yerevan is used. The data of the accelerograms at Gukasian are digitalized by Professor Okada's Laboratory, Institute of Industrial Science, University of Tokyo and those at Yerevan by Public Works Research Institute, Ministry of Construction, Japanese Goverment. These accelerograms are shown in Fig. 3 in terms of the acceleration spectra.

### 3.5 Results and Discussions

The required strength ratio ($\beta=p_{By}/mv_{go}$) corresponding to ductility factor ($\mu=v_{max}/v_{By}$) is plotted in Figs. 5 through 9 as the functions of period $T_1$. The required strength ratios are defined as the normarized strength required to restrict ductility factors within a fixed value, just like that the acceleration response spectra are defined as the acceleration level required to restrict the acceleration response within linear conditions.

Significant figures of these results are that the required strength ratios become smaller with the higher ductility factors which accompany the large degradations of stiffness and that their curves shift toward the range of shorter periods with the higher ductility factors. Also the curves of required strength ratios become smoother with the higher ductility factors, because the influence of local predominant become little. These facts suggest that the ductility factors are very important upon considering the seismic capacity of buildings and that the ductility and strength capacities control the seismic capacity of buildings.

On the comparison between the cases of $T_2=\sqrt{2T_1}$ and $T_2=2T_1$ in Figs. 5 and 6, the required strength ratios for $T_2=2T_1$ are lower than corresponding ones for $T_2=\sqrt{2T_1}$, because the equivalent hysteretic dampings for $T_2=2T_1$ are larger than those for $T_2=\sqrt{2T_1}$ and the stiffness degradations for $T_2=2T_1$ are larger than those for $T_2=\sqrt{2T_1}$. It should be noted that the absolute relative response displacements for $T_2=2T_1$ are two times those for $T_2=\sqrt{2T_1}$, if the ductility factors of both cases are all the same.

One can easily find the relationship between the strength levels and ductility factors as the functions of the period $T_1$ by using Figs. 5 through 9 and assuming the level of peak accelerations. For example, by using Fig. 5(b) at Gukasian and assuming the level of peak acceleration to be 0.2g, where g is the acceleration of gravity, the results are,

$$T_1=0.6\text{sec},\quad \mu=1\quad \beta=1.4\quad C_B=0.28$$

$$\mu=2\quad \beta=0.8\quad C_B=0.16$$

$$\mu=4\quad \beta=0.5\quad C_B=0.10$$

and using Fig. 9(a) at Yerevan and assuming the peak accelerations 0.06g,

$$T_1=0.6\text{sec},\quad \mu=1\quad \beta=0.5\quad C_B=0.03$$

$$\mu=2\quad \beta=0.3\quad C_B=0.02$$

where $C_B$ is the base shear coefficient.

The influence of the characteristics and amplitude levels of the accelerograms brings up the large difference of the base shear coefficients at each site. These results might correspond to the degree of the damage for buildings at each site.

### 3.6 Conclusive Statements

Judging from the required strength ratio and amplitude levels of the accelerograms observed at Gukasian, the insufficient strength and ductility capacities would have caused the collapse of many buildings, considering that the predominant periods of accelerations would be possible to become longer and that the amplitude levels of accelerations higher in the affected areas. The joints connecting columns and beams in precast reinforced concrete frame buildings seemed to be very poor, and this would have accelerated the damage.

All parameters of this paper were assumed except that the accelerograms had been observed, but the results were resonable. Furthermore the effects of soil-structure interaction are neglected, but they are beyond a matter of this paper. It should be noted that the seismic capacity of a building depends on the strength and ductility capacities and that the strength and ductility capacities can make up for their insufficience each other.

### 3.7 References


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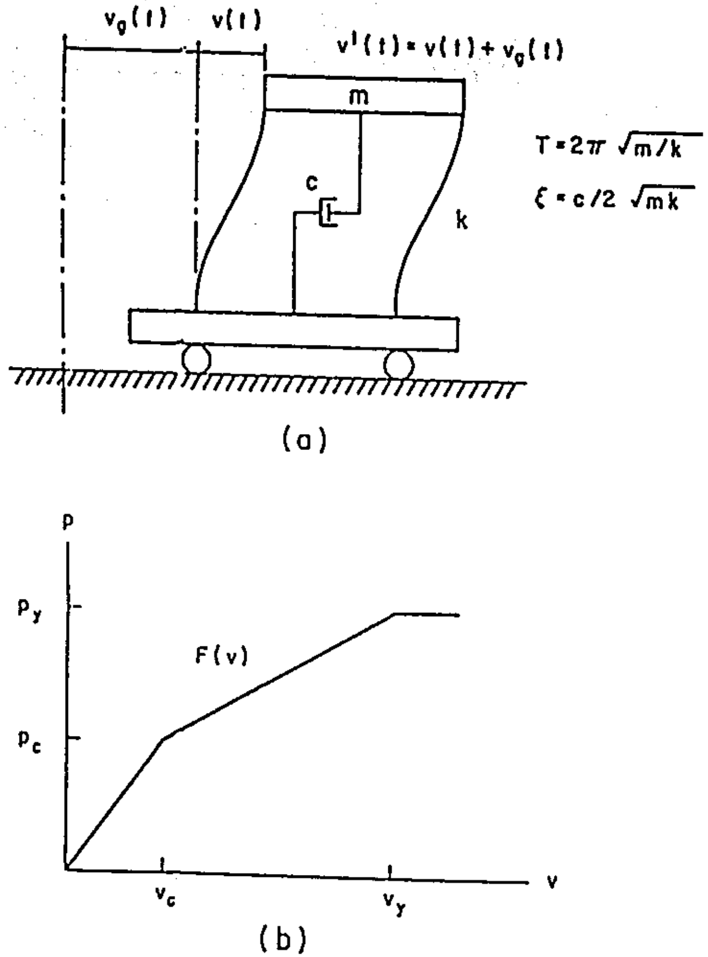



Fig. 1  Single degree of freedom model with force-displacement relationship.

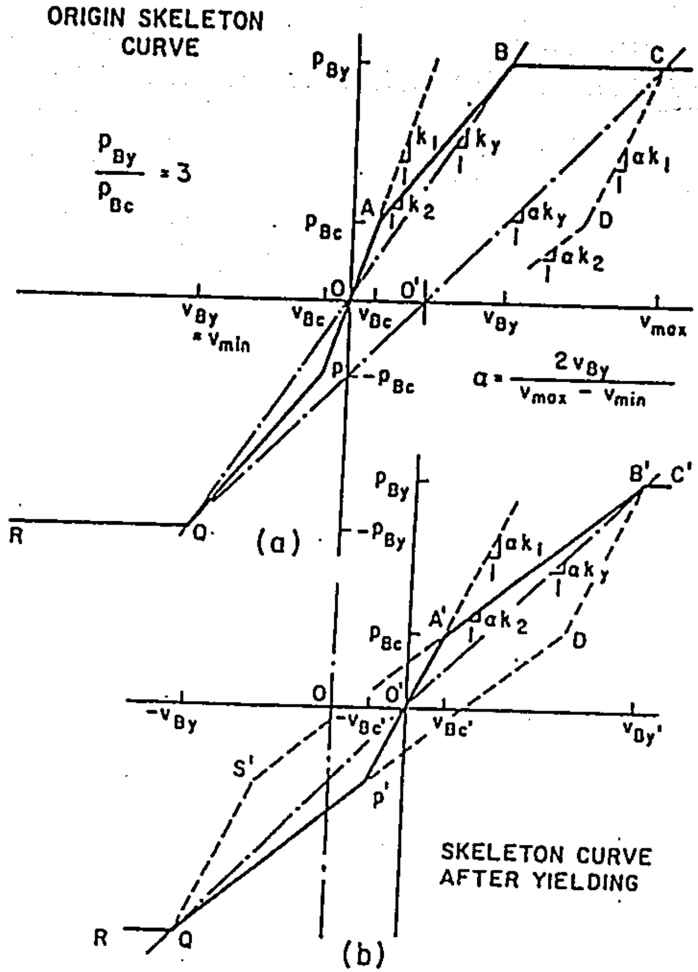



Fig. 2  Tri-linear stiffness degrading hysteretic model.

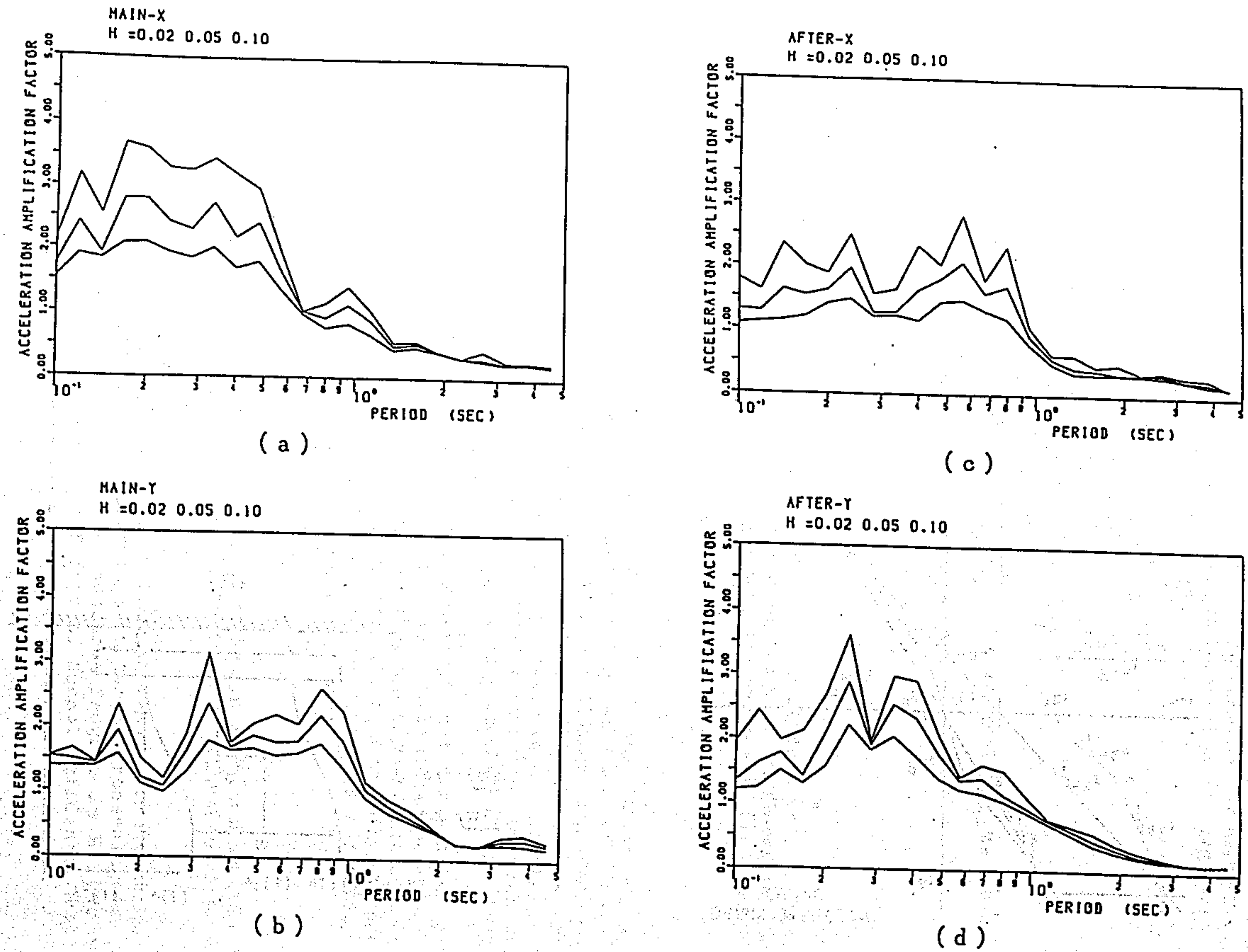



Fig. 3 Acceleration Response Spectra at Gukasian

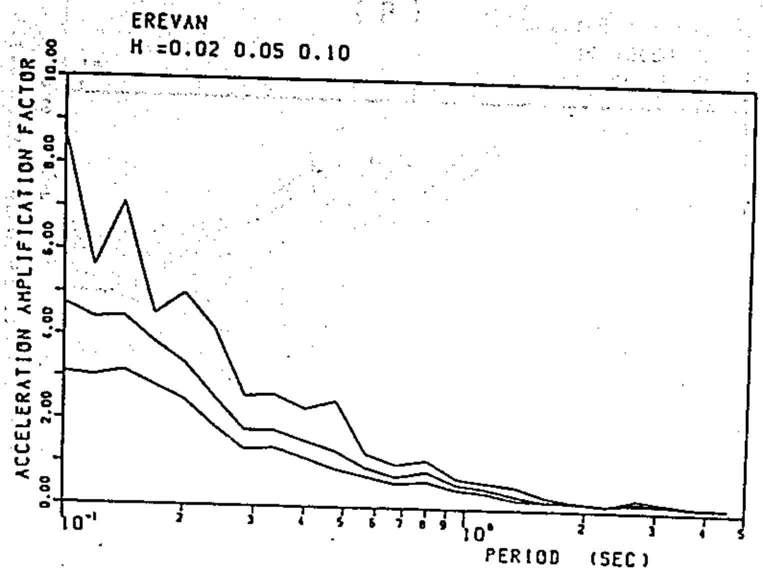
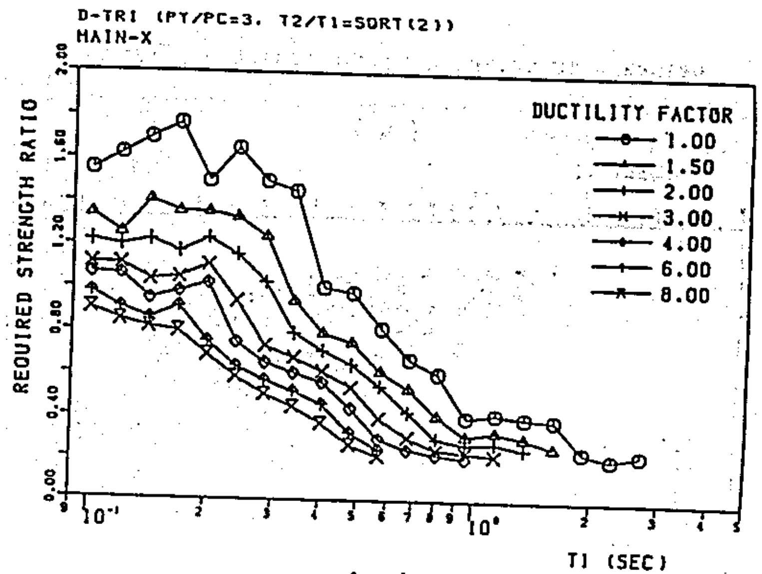
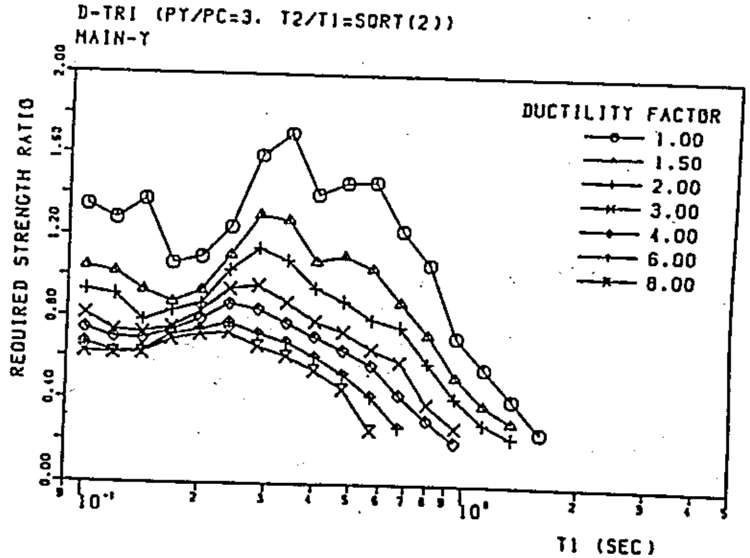



Fig. 4 Acceleration Response Spectra at Yerevan




( a )




( b )

Fig. 5 Required Strength Ratios at Gukasian

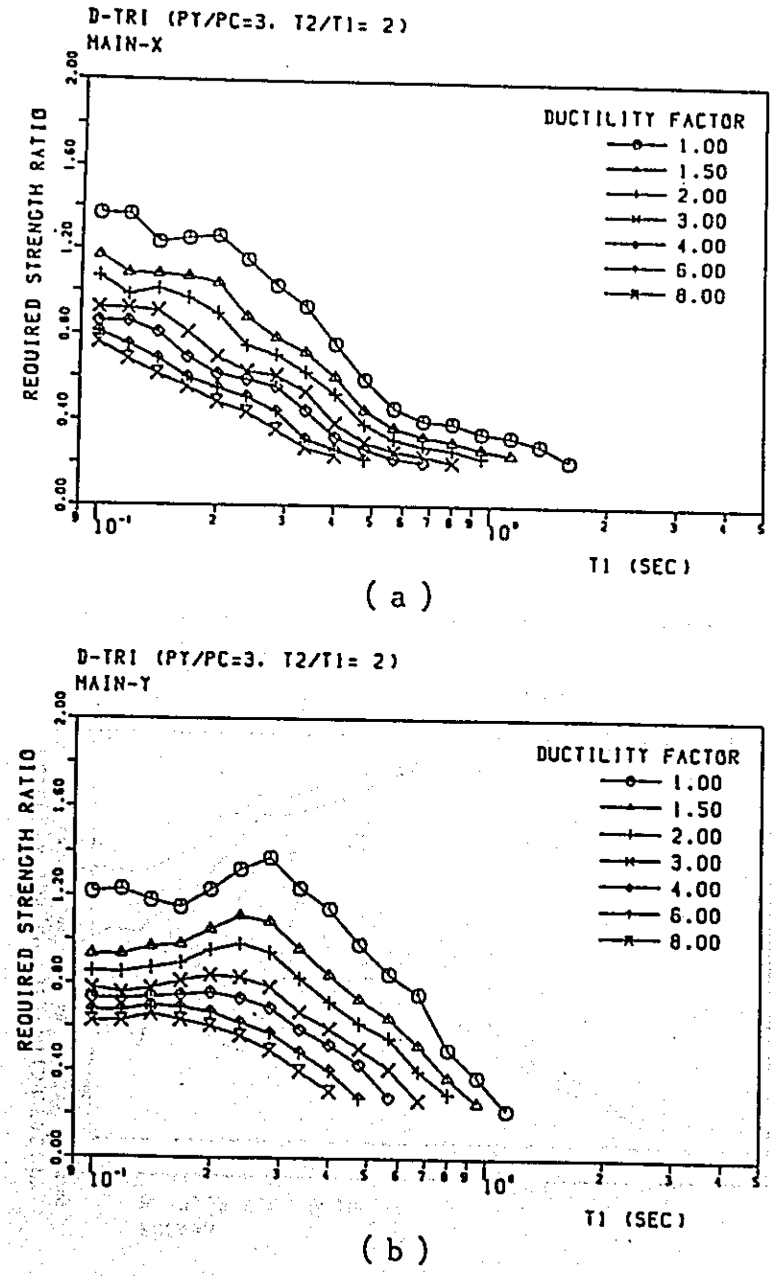



Fig. 6 Required Strength Ratios at Gukasian

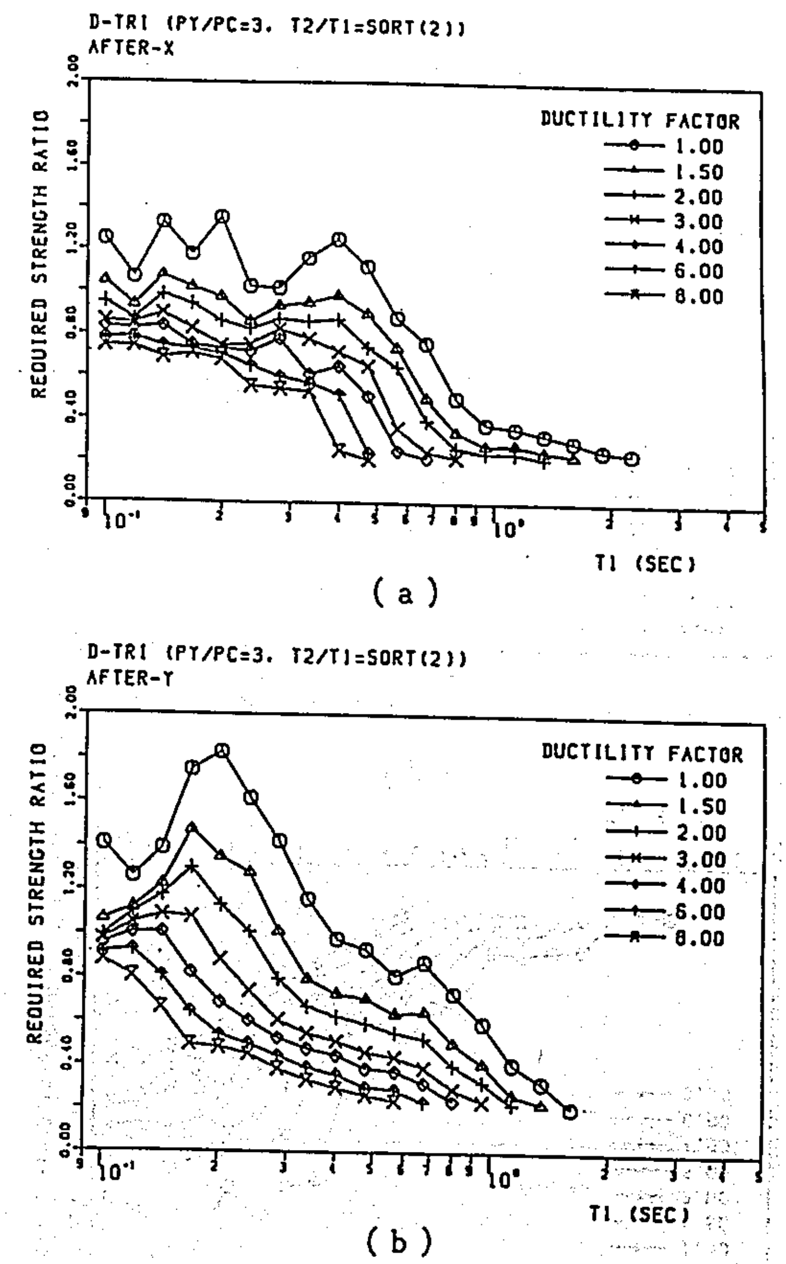



Fig. 7 Required Strength Ratios at Gukasian

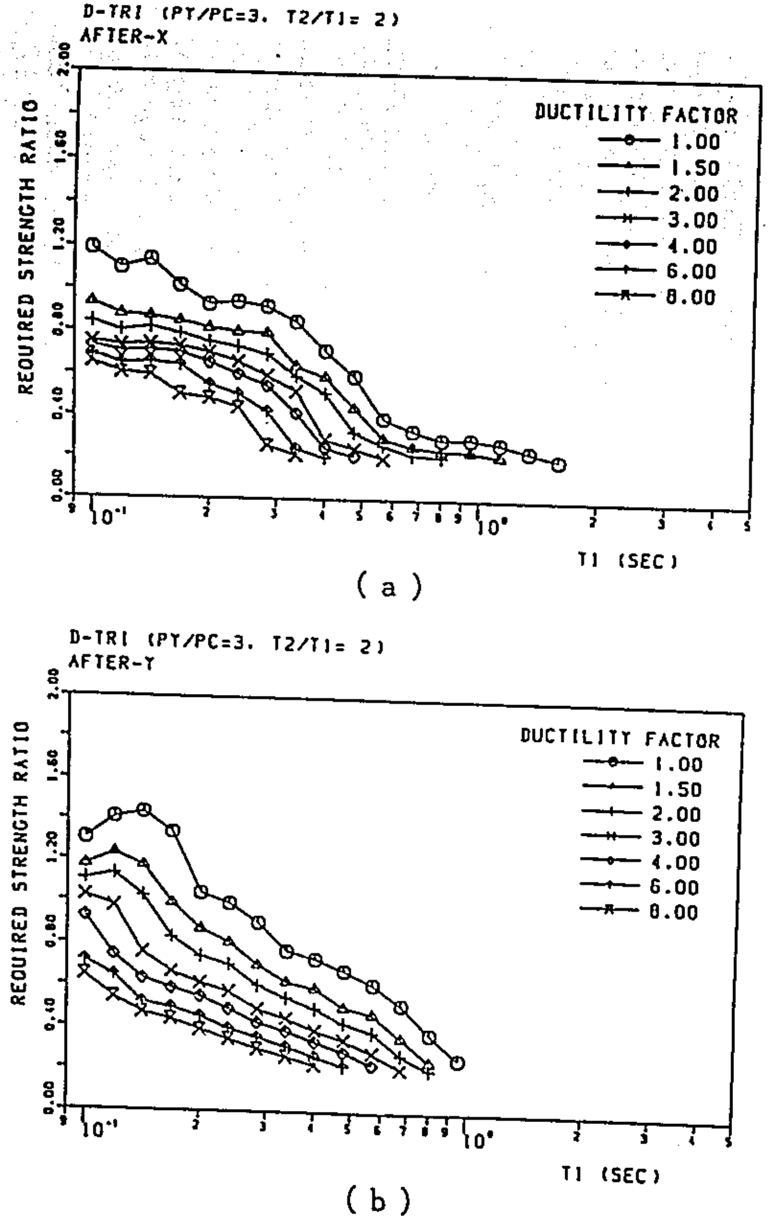



Fig. 8 Required Strength Ratios at Gukasian

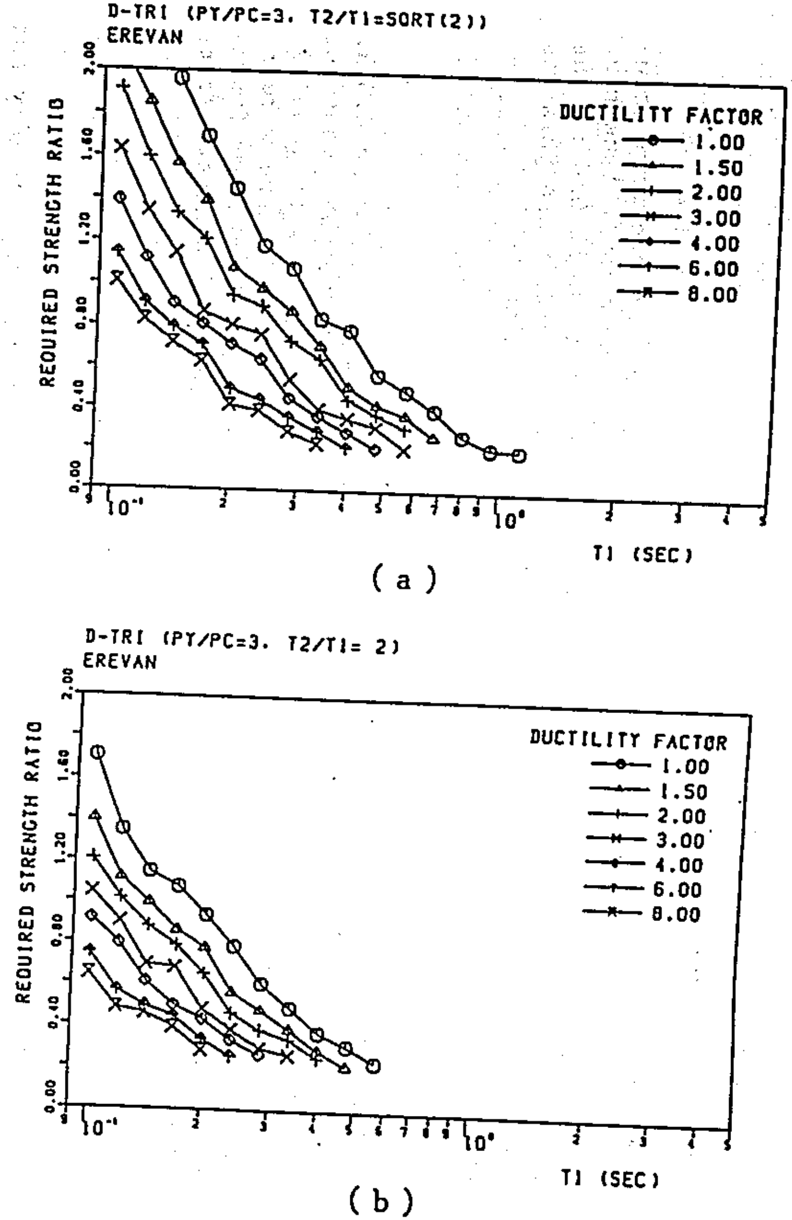



Fig. 9 Required Strength Ratios at Yerevan

## 4. Preliminary Analyses of a 9-story Precast Reinforced Concrete Frame Building

### 4.1 Abstract

Preliminary analyses of a 9-story precast reinforcement concrete frame building that was damaged severely due to Spitak Earthquake on 7 December, 1988, were carried out to investigate the process up to collapse of the building by the stepwise loading method. The earthquake response analysis was also carried out by using an accelerogram recorded at Gukasian.

### 4.2 Introduction

There were a lot of precast reinforced concrete frame building in S.S.R. of Armenia, which consisted of columns and beams of lightweight concrete, precast void slabs, shear walls and concrete curtain walls (wall panels). Shear walls were located in a part of transverse direction and there were no beams in this direction. The wall panels were attached on the exterior frames in the longitudinal direction. A 9-story precast reinforced concrete frame building was analyzed herein (Fig. 1).

### 4.3 Analytical Model and Method

The structural model was assumed that joints were connected rigidly and that wall panels did not contribute to the stiffness and the lateral load carrying capacity. The model was analyzed in the longitudinal direction considering the member-to-member characteristics. The torsional deformation was neglected. The model with 4 longitudinal bars in columns and that with 8 bars were studied, respectively.

Sectional properties and coefficients of materials shown in Table 1 were chosen based on the specifications in structural drawings. Beams and columns were assumed to have all the same sectional property from the 1st story through the 9th story. The modulus of elasticity of concrete was calculated by the equation (1) according to the Standards for Reinforced Concrete Structures by Architectural Institute of Japan (AIJ Standards).

$$E_c = 2.1 \times 10^5 \times \left\{ \frac{\gamma}{2.3} \right\}^{1.5} \times \sqrt{\frac{F_c}{200}} \quad , \quad (\mathrm{kg/cm^2}) \qquad (1)$$

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3) Professor, Institute of Industrial Science, University of Tokyo

$\gamma$ : weight of concrete per unit volume (tonf/m$^3$)
Fc : design strength of concrete (kgf/cm$^2$)

The weight of the structure was calculated from dimensions of members. The results were shown in Table 2. The axial force of each column was evaluated in accordance with the supporting floor area.

The stiffness and the strength of both beams and columns were calculated from the equations as shown in Table 3.

A lateral load distribution used in the stepwise loading analyses is Ai-distribution regulated in Japanese seismic codes (Fig. 2). The Ai-distribution was defined by the equation (2).

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right) \times \frac{2T}{1+3T} \qquad (2)$$

where, T : fundamental period (sec.)
$\alpha_i$: defined as follows;

$$\alpha_i = \sum_{j=i}^{N} W_j \Big/ \sum_{j=1}^{N} W_j$$

N : number of stories
Wi: weight of the i-th story (tonf)

The accelerogram recorded at Gukasian was used in the earthquake response analysis, that was digitized by the authors.

### 4.4 Results of Analyses

First, the stepwise loading analyses were carried out, and the process up to collapse of the structural model was investigated. The process up to collapse of the model with 4 longitudinal bars in columns was shown in Figs. 3a and 3b. Figs. 4a and 4b were shown in the case of 8 bars.

Figs. 3a and 4a showed plastic hinge formations at the base shear coefficient of 0.1. Several beams had already yielded while no yielding hinges were developed in any columns.

Figs. 3b and 4b showed plastic hinge formations at collapse hinge mechanism. Base shear coefficients of the models at the collapse hinge mechanism were 0.137 for 4 longitudinal bars, 0.151 for 8 longitudinal bars, respectively. The drift angle of the first story was 1/10 for 4 bars and 1/9 for 8 bars. The model was deformed extremely at the collapse mechanism stage, and it was considered that joints of precast elements might have been destroyed before collapse hinge mechanism was formed.

The relationship between the relative story displacement and the story shear force were shown in Figs. 5a and 5b. The stiffness of the model with 8 longitudinal bars was higher than that with 4 bars. The difference of the strength levels between the models, however, was not so large, because their levels were mainly controlled by the behavior of beams.

Secondly, the earthquake response analysis was carried out. The result was shown in Fig. 6. The main shock of which maximum acceleration was 187.9 gal observed at Gukasian was used in this analysis. The model with 8 longitudinal bars was analyzed in consideration of the seismic intensity at Gukasian. Damping factor of the model was assumed 0.04. The maximum response base shear coefficient was 0.061. The maximum drift angle was 1/165 in the second story and the overall drift angle was 1/300. No elements were beyond yielding. This result suggested that if the wall panels did not contribute to the lateral load carrying capacity and joints of precast elements were rigidly connected, the building might have survived the earthquake observed at Gukasian.

The calculated fundamental period was much longer than the observed one (approximately 0.61-0.63sec.) as shown in Fig. 7, because the stiffness of wall panels and the contribution of reinforcement to the stiffness of members were not considered. The response acceleration was, therefore, relatively small as shown in Fig. 8. If these contributions were considered, the natural periods were shorter and the response acceleration was expected much larger. Further analyses considering these contributions should be carried out.

## 4.5 Concluding Remarks

The stepwise loading analyses and the earthquake response analysis based on the member-to-member characteristics were carried out. The results were summarized as follows;

1) The precast reinforced concrete frame building was a weak beam type structure.
2) The collapse hinge mechanism was developed when the drift angle of the first story was about 1/10. Joints of precast elements, however, might have been destroyed before such an extreme deformation.
3) In the earthquake response analysis using the accelerogram observed at Gukasian, no beams and columns were beyond yielding.
4) The stiffness of wall panels and the contribution of reinforcement to the stiffness of members should be considered to simulate the damage level of the precast reinforced concrete frame building.

## 4.6 Acknowledgement

The authors are grateful to the Armenian Research Institute for Civil Engineering and Architecture at Yerevan for providing necessary data in carrying out these analyses.