For more information, contact: EERI
6431 Fairmount Avenue, Suite 7
El Cerrito, CA USA 94530-3624
(415) 525-3668

EARTHQUAKE ENGINEERING RESEARCH INSTITUTE
COMMITTEE ON CONTINUING EDUCATION

SLIDES ON THE DECEMBER 7, 1988, SPITAK EARTHQUAKE

SET II: DAMAGE AND ENGINEERING FACTORS

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Created by:
Professor Armen Der Kiureghian, University of California, Berkeley, California, with contributions from Lloyd Cluff, Fred Krimgold, H. S. Lew, EQE Engineering, and the U.S. Geological Survey

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PREFACE

The magnitude 6.8 Spitak earthquake which struck Soviet Armenia at 11:41 a.m. local time on Wednesday, December 7, 1988, caused the following impacts:

- twenty thousand injured,
- an estimated 25,000 dead, (the exact number may never be known),
- five hundred ten thousand homeless,
- collapse and heavy damage to buildings (including hospitals, schools, apartment buildings and industrial facilities):
  - in Spitak: damage to 100% of the building stock, with at least 12,000 to 15,000 dead,
  - in Leninakan: damage to 80% of the building stock, with at least 10,000 to 12,000 dead, and
  - in Kirovakan: damage to 50% of the building stock, with at least 450 dead.
- extensive social disruption, and
- reconstruction costs that are estimated to reach $16 billion or more.

These impacts made this earthquake one of the worst natural disasters of the twentieth century. The Spitak earthquake was a disaster of modern precast-concrete-frame-panel buildings constructed in the 1970's and 1980's. In the Soviet Union, building construction is typically planned in Moscow where a limited number of basic general building designs are prepared for implementation repeatedly throughout the nation. Initially, the designs do <u>not</u> incorporate seismic loads and a local agency modifies the general design for seismic loads when they are applied in a region characterized by moderate-to-high seismicity. Both a building code prescription and a microzonation strategy are used.

In Armenia, the principal building types were:

- <u>stone-bearing wall buildings</u>, the traditional construction technique until 1970. These buildings were limited in height to five stories. The masonry walls are thick, lack steel reinforcement, and provide both lateral and vertical support for the hollow core concrete plank floors and roofs which were introduced in the 1950's and 1960's.

- <u>composite frame and stone wall buildings</u>, mostly 4- and 5-story buildings consisting of exterior stone shear walls and a framing system cast within the walls as well as the interior of the building.

- <u>precast concrete frame-panel buildings</u>*, which began in the 1970's and today are the predominant design for residential and industrial structures. In the affected area, the tallest of these buildings was nine stories with one-story penthouses. Floors and roofs are precast hollow-core concrete planks that bear on the walls but have no connections. The buildings have steel reinforcement.

- <u>precast concrete-panel buildings</u>, a contemporary building type in Armenia which was just beginning to be widely constructed for public and residential use. They ranged in height to nine stories. Floors and roofs are also precast hollow-core concrete planks. They are relatively stiff.

- <u>concrete lift-slab buildings</u>, which involve either one central core or double cores of cast-in-place concrete shear walls. Elevated floor and roof slabs are cast at grade, lifted into place, and supported by columns. The cores provide lateral stability for the structure. Building performance depends strongly on the quality of the attachments of the slabs to the cores. Only two buildings of this type--one of 10 stories and another of 16 stories--had been erected in Leninakan at the time of the Spitak earthquake. Both buildings were heavily damaged, requiring subsequent demolition.

In the 400 square kilometer epicentral region affected most severely by the Spitak earthquake, the damage statistics for the four principal types of buildings (see Table 1) stone bearing wall, composite frame and stone wall, precast concrete frame-panel, and precast concrete-panel) are:

*commonly called concrete frame buildings

Table 1: Statistics of the damage to multistory residential buildings as of January 24, 1989. Note: A = collapsed, B = heavily damaged to be demolished, C = damaged to be repaired or strengthened, and D = no significant damage, usable.

City or Town (Epicentral Distance, km)	Precast Panel				Precast Frame-Panel				Composite Frame-Stone				Stone			
	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
Spitak (9)	-	-	-	1	-	-	-	-	43	9	7	-	20	2	3	-
Stepanavan (23)	-	-	-	-	-	-	-	-	8	13	33	2	10	-	9	16
Kirovakan (25)	-	-	-	4	-	-	88	20	41	89	414	27	46	53	145	-
Akhourian (27)	-	-	-	-	-	-	-	-	18	4	17	2	1	2	5	3
Dzaghgahovia (30)	-	-	-	-	-	-	-	-	-	-	8	1	-	10	9	-
Kalinino (30)	-	-	-	-	-	2	11	5	-	-	4	-	-	3	-	-
Leninakan (32)	-	-	-	16	72	55	6	-	27	115	67	20	24	160	154	150
Aparan (33)	-	-	2	-	-	-	3	-	-	4	5	-	-	14	6	2
Artik (33)	-	-	-	-	-	-	-	-	-	2	47	25	-	14	2	-
Ghoukasian (34)	-	-	-	-	-	-	-	-	-	4	4	1	-	5	2	4
Amasia (37)	-	-	-	-	-	-	-	-	-	4	1	1	-	1	2	4
Pemzashen (38)	-	-	-	-	-	-	-	-	-	3	-	3	-	12	4	4
Maralik (44)	-	-	-	-	-	-	-	-	-	-	17	2	-	-	5	1
Alaverdi (47)	-	-	5	15	-	-	4	18	-	-	11	43	-	16	21	48
Dilijan (59)	-	-	-	-	-	-	3	5	-	19	8	47	-	7	5	2
Charentsavan (60)	-	-	6	21	-	-	8	7	-	-	19	79	-	-	-	-
Talin (62)	-	-	-	-	-	-	-	-	-	-	6	1	-	2	3	2
Razdan (63)	-	-	-	3	-	-	-	-	-	-	25	50	-	7	-	-
Ashtarak (65)	-	-	-	-	-	-	-	1	-	-	3	-	-	-	-	-
Sevan (73)	-	-	-	1	-	-	-	-	-	-	-	-	-	-	-	-
Noyemberian (74)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4
Abovian (78)	-	-	-	2	-	-	-	16	-	-	-	-	-	-	-	6
Idjevan (79)	-	-	-	2	-	-	7	5	-	-	23	3	-	-	9	13
Bert (100)	-	-	-	-	-	-	-	-	-	-	-	-	-	6	1	-
Krasnaselsk (102)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	5	-
Googark (??)	-	-	-	-	-	-	-	-	-	-	-	-	3	2	9	1
Baghramian (??)	-	-	-	-	-	-	-	-	-	-	-	-	-	1	3	3
Total	-	-	13	65	72	57	130	77	137	266*	719*	307	104	317	402*	263*
Percent in Each Category																
Spitak	-	-	-	100	-	-	-	-	73	15	12	-	80	8	12	-
Stepanavan	-	-	-	-	-	-	-	-	14	23	59	4	29	-	26	46
Kirovakan	-	-	-	100	-	-	81	19	7	16	72	5	19	22	59	-
Leninakan	-	-	-	100	54	41	5	-	12	50	29	9	5	33	31	31
All Cities	-	-	17	83	21	17	39	23	10	19	50	21	10	29	37	24

-- 314 buildings collapsed,
-- 641 needed to be demolished,
-- 1,264 needed repairs or strengthening, and
-- Only 712 (24%) remained habitable after the earthquake.

The Spitak earthquake produced two contrasts in performance:

-- the performance of precast concrete frame-panel buildings in Leninakan versus their performance in Kirovakan, and

-- the performance of precast concrete frame-panel versus the performance of precast concrete-panel buildings.

In Leninakan, 54% of the precast concrete frame-panel buildings collapsed, 41% will have to be demolished, 5% will need repairs, and none escaped damage. In contrast, in Kirovakan, none of the precast concrete frame-panel buildings collapsed or needed to be demolished and 19% escaped damage altogether. The explanation--site amplification in the 1.0 to 2.5 second period band by the deep (200-300 m; 660-1000 ft) lake bed deposits underlying Leninakan; soils in Kirovakan are thinner and stiffer. Also, the buildings in Kirovakan are limited in height to 5 stories.

The damage distribution is given in table 1 above. Armenian engineers rated the epicentral intensity as IX to X (MSK scale). They estimated that levels of horizontal peak ground acceleration may have reached 0.50 to 1.0g in Spitak, possibly with a large vertical component as well because of the thrust fault. The estimated level in Leninakan was about 0.40g, based on seismoscope records.

Recorded peak ground acceleration values are 0.21 g at Ghoukasian, located 27 km north of Leninakan, and 0.06 g at Yerevan, located 100 km from the epicenter.

In Armenia, most designs were for an intensity (MSK scale) of VII to VIII, with reductions being permitted for volcanic tuff foundation materials.

References

1) The Soviet Armenia Earthquake Disaster: Could a Similar Disaster Happen in the United States? Hearing of March 15, 1989, convened by the Subcommittee on Science, Research, and Technology of the Committees on Science, Space, and Technology of the U.S. House of Representatives; Witnesses: Frederick Krimgold, Peter Yenev, Loring Wyllie, Eric Noji, Henry Siegleson, Ronald Coleman, Larry Green, Christopher Rojahn, Jerome Iffland, Michael Heisler, and Richard Bail.

2) Cluff, Lloyd S., and Tobin, L. Thomas, The December 7, 1988, Earthquake in Armenia Soviet Socialist Republic, Report to the California Seismic Safety Commission, March 1989.

3) Filson, John R., Agbabian, Mihran S., and Noji, Eric R., Postearthquake Investigations of the December 7, 1988, Spitak Earthquake, Proceedings by the United States International Symposium on the Spitak Earthquake, May 23-26, 1989, Yerevan, Armenia.

COMMENTARY ON INDIVIDUAL SLIDES

<u>SLIDE 1: REGIONAL MAP OF ARMENIA</u> (Information provided by Armenia Geological Institute through Lloyd Cluff, California Seismic Safety Commission)

This slide shows a very generalized picture of the complex tectonics of the region. The North Sevan and Yerevan faults are strike-slip faults and are believed to be a major branch of the Anatolia fault in Turkey. The Spitak fault is a reverse fault that strikes northwest and dips about 55 degrees northeast. Both the North Sevan and Spitak faults were mapped prior to the earthquake and exhibit late Quaternary activity. They are tectonically related.

<u>SLIDE 2: RUPTURE PLANE OF THE SPITAK FAULT</u> (TAKEN AT 4:00 P.M. ON February 23, 1989)

This slide shows the rupture plane of the Spitak fault (a thrust fault) as viewed from inside a trench which was dug across the fault 5 km (3 miles) southwest of Spitak. The block on the right has moved 1.2 m upward relative to the block on the left.

<u>SLIDE 3: ISOSEISMAL MAP</u> (Source: EQE Engineering, San Francisco, California)

The isoseismal map for the Spitak earthquake is shown in this slide. In the Soviet Union, a 12-point intensity scale known as MSK-64 is used for seismic zoning and design. The description of each intensity level closely parallels that for the Modified Mercalli Intensity scale. Before the earthquake, Leninakan was specified as zone VIII, and Spitak and Kirovakan were specified as zone VII. The correlation of intensity with peak ground acceleration is:

-- intensity VI; 0.025 to 0.05 g
-- intensity VII; 0.05 to 0.10 g
-- intensity VIII; 0.10 to 0.20 g
-- intensity IX; 0.20 to 0.40 g
-- intensity X; 0.40 to 0.80 g.

The structures in Leninakan, Spitak, and Kirovakan had been designed for lateral forces approximately equal to 2.5 to 5 percent of their weights.

SLIDES 4, 5, 6, 7, AND 8: FAILURE MECHANISMS OF STONE-BEARING-WALL BUILDINGS

Damage to stone-bearing-wall buildings, which were the predominant construction type in Spitak, occurred in a variety of ways:

-- The onset of damage typically occurred at building corners with almost every surviving building showing visible cracks.

-- In some buildings, the walls tilted away from the concrete plank floors, resulting in the collapse of the planks.

-- In some buildings, the end walls collapsed; whereas, in others, the end walls remained upright and the middle collapsed as a consequence of the failure of the precast hollow-core concrete planks to act as an effective floor diaphram, causing the transfer of forces to the masonry walls.

SLIDE 4: (Taken at 10:00 a.m. on December 23) shows the failure of a typical 4-story stone bearing wall building in Leninakan. The failure has exposed both the exterior and interior walls, revealing the method of construction of the walls as well as the flooring system. The floor is of light weight concrete planks with simple, narrow supports on the walls. Note that the floor planks at the fourth level have also fallen.

SLIDE 5: (Taken at 9:00 a.m. on December 24) shows the failure of the stone bearing wall systems at the corners where the two walls meet. This failure often led to the failure of the end wall (see slides 4 and 6) or to the entire corner of the building. The fallen floor planks can be seen in the foreground.

SLIDE 6: (Photographed by Fred Krimgold, Virginia Polytechnic Institute and State University) shows the failure of the end wall. Across the street from this building in Leninakan were collapsed precast 9-story concrete frame-panel apartment buildings.

SLIDE 7: (Photographed by H. S. Lew, National institute of Standards and Technology) shows damage to a stone bearing wall building in Leninakan. The floor planks were not tied to the supporting bearing walls.

SLIDE 8: (Photographed by H. S. Lew, National Institute of Standards and Technology) shows damage to an old stone bearing wall building in Leninakan which was a hat and glove factory.

SLIDE 9: CORNER REINFORCEMENT OF STONE WALLS IN LENINAKAN
(Taken at 1 p.m. on December 22)

In their newer stone bearing wall buildings, Soviet engineers built a reinforced concrete member at the corner that provides continuity to both walls along the edges. In this slide, failure of the stone wall appears to have been arrested by this member. There were many cases, however, where this member many <u>not</u> have helped to stop the failure at the corner. Further study is required to evaluate the beneficial effects of this system.

SLIDE 10: COLLAPSE OF THE 19TH CENTURY CHURCH IN LENINAKAN
(Taken at 11:00 a.m. on December 24)

This is the main church of Leninakan--Amenaperkitch Vank--which was built over the period 1858-1873. It was modeled after the main cathedral in Ani; the old capital of Armenia, now located in Eastern Turkey, which was built by Terdat in the 10th century. The church is an important part of the Armenia culture, so its collapse has a significant social impact.

SLIDE 11: COLLAPSE OF TYPICAL VILLAGE HOUSE IN SHIRAGAMUT (FORMERLY NALBAND)

This slide (taken on December 22, 1988) shows the collapse of a village dwelling, typically consisting of stone walls and light timber roofing. Virtually all buildings in this village collapsed. The same happened in Spitak, leaving many dead and/or homeless.

SLIDES 12, 13, 14, 15, 16, AND 17: FAILURE MECHANISMS OF PRECAST CONCRETE FRAME-PANEL BUILDINGS

Precast concrete frame-panel buildings in Armenia were typically constructed in long rectangular configurations with columns and beams providing the vertical load carrying system. The floor and roof systems were hollow-core precast concrete planks, without topping slabs or positive connections to the building frame. Perimeter walls and selected interior walls of unreinforced masonry infill, precast fascia panels, and precast-concrete-shear panels were designed to provide lateral stability in the longitudinal direction; whereas, the frames were designed to provide the lateral-load resisting path in the transverse direction.

The most common failure patterns included:

-- Separation at wall, floor, and corner connections.
-- Loss of longitudinal stability due to infill masonry (typically volcanic tuff) falling out of the frames.
-- damage at column splices, which consisted of lap welds of reinforcing steel bars extending from the upper and lower column sections. Due to poor quality control in the field, these splices were often eccentric.
-- loss of containment due to minimal hoop reinforcement.
-- buckling of columns at reinforcing splices.
-- failure of frames due to the rigid, heavy, precast infill panels

Slide 12, taken at 10:00 a.m. on December 24 in Leninakan, shows the failure of floor planks of a building under construction which was to become the new building of the Polytechnic Institute. The floor planks, of lightweight concrete, hollow-core, and approximately 4 ft x 6 ft x 8 inches are simply supported on the beams over a 2 to 3 inch seat. There are no ties between the planks or between the planks and the beam reinforcement.

SLIDE 13, (Photographed by Fred Krimgold, Virginia Polytechnic Institute and State University) shows classic failure of infill walls and the floor planks in a furniture factory just outside of Leninakan.

SLIDE 14, taken on December 23 in Leninakan, shows column failure. The failed column striking out from the rubble of a 9-story precast frame-panel apartment building shows that failures occurred at the face of the cast-in-place beam column joint and at the splice point of the column. Close inspection showed that the rebars fractured in a brittle manner at the end points of the welding.

SLIDES 15, 16, AND 17, show different views of the failure of a column at the joint between two precast elements. Brittle failure of welded rebars can be observed. Some rebars from the column above are not contained in the column below. This kind of failure at the connection between the precast column elements was very common.

SLIDE 18: METHOD OF CONSTRUCTION

This slide, taken at 10:00 a.m. on December 24 in Leninakan, shows clearly the method of construction of the framing system in one direction (perpendicular to the slide) and the shear panels in the other direction (parallel to the slide).

SLIDE 19: PRECAST CONCRETE PANEL BUILDINGS

In the Soviet design of precast concrete panel buildings, virtually every precast interior wall is used as a load-bearing element having shear capacity. This design gives a stiff, redundant structure. Floors and roofs are precast concrete planks, but with positive intrastructural connections between the various elements.

In this slide, taken at noon in Leninakan on December 27, the performance of precast frame-panel and precast panel buildings can be compared. Many precast frame-panel buildings in Leninakan collapsed and are shown in the foreground, including one under construction. Precast panel buildings, in contrast, performed very well and are shown standing in the background. The difference in performance is due to the basic differences in their design as well as possibly to specific characteristics of the ground motion. Site amplification in the 1.0 to 2.5 second band was found in the strong motion records of the aftershock sequence recorded in Leninakan may have generated a greater load on the concrete frame-panel buildings.

SLIDE 20: PARTIAL COLLAPSE OF 9-STORY APARTMENT BUILDING IN LENINAKAN

This slide, taken at noon on December 23, shows the still standing half of a 9-story apartment building. The other half had collapsed. The building is of precast beam, column, and shear panel elements, with framing in the longitudinal direction and shear panels in the transverse direction. Floors are made of lightweight concrete planks with no ties to each other or to the beams upon which they are simply supported. The failure was probably due to the lack of horizontal diaphragms at the floor levels and/or due to insufficient shear resistance in the transverse direction. Note the precast beam elements hanging from the connection points to the columns.

Reconstruction in Armenia must take into account all of the factors discussed and illustrated in this slide set.