Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Analysis of the collapsed Armenian precast concrete frame buildings

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ABSTRACT: The Soviet Armenia Earthquake of December 7, 1988, caused extensive damage and loss of life in eastern Armenia. Over 25,000 people were killed and hundreds of thousands were left homeless. A major contributor to the fatality count and number of homeless was the poor performance of modern, nine-story, precast reinforced concrete frame buildings located in the city of Leninakan. This paper discusses a detailed computer analysis of these buildings correlated with damage observations following the earthquake.

1 INTRODUCTION

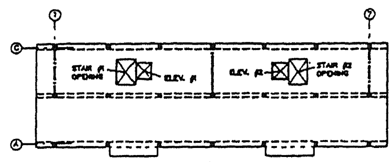
A devastating earthquake occurred in Soviet Armenia on December 7, 1988, causing a loss of life in excess of 25,000. The fault break was near the city of Spitak, but extensive damage occurred in the City of Leninakan with a population of about 300,000 located 30 kilometers west of Spitak. The most prevalent modern building in Leninakan was the nine-story precast concrete frame building. Of the 133 buildings of this type, 72 collapsed and 55 were heavily damaged and had to be demolished. Obviously, there was a tremendous loss of life associated with these buildings.

The author visited Armenia shortly after the earthquake and on two additional trips in 1989. During these visits information was gathered regarding the design and construction of these buildings. Observations of damage and failure tendencies were made shortly after the earthquake in the heavily damaged buildings before they were demolished.

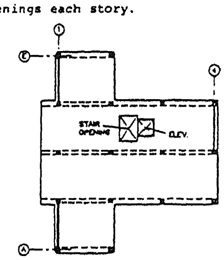
2 BUILDING DESCRIPTION

The buildings, known as the Soviet Series 111, came in several different configurations built with similar details. Figure 1 shows the basic building shapes observed in Leninakan. Column spacing was typically 6.0 meter or 5.4 meters and story height was 3.0 meters. The buildings were apparently designed in the 1960s and relied on moment resisting frames in one direction and shear walls in the other direction to resist lateral forces. The buildings were designed to the seismic provisions of the Soviet code, SNiP 11-A 12.62.

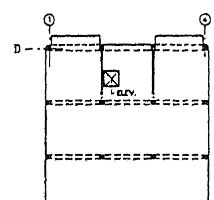
The moment resisting frames consisted of columns 400mm by 400mm and beams 400mm wide and 520mm deep. The beams were precast for their lower 300mm to provide a seat for the 220mm hollow core floor panels which spanned between beams. The bottom reinforcement of the precast portion of the beams extended out of the end of the precast beam section and sat on two steel angles that had been cast into the precast column. This provided for easy beam erection and the bottom bars were field welded to the angle for



a. Rectangular plan with three shear walls. Walls on line 1 and 7 each have two door openings each story.



b. T-shaped floor plan with three solid shear walls.



c. Square floor plan with two shear walls each with a single door openings

Figure 1 Typical plans of three versions of frame buildings (Soviet series 111) as found in Leninakan (From Reference 1) seismic frame moments. Likewise, portions of top bars were cast into the precast column which were field welded to long top bars which were cast into the upper beam in the field at the same time the joint at the column face was cast.

The precast columns were cast in two story lengths and spliced near midheight. The detail on the drawings called for the four or six bars of the columns to align and be butt welded with full penetration welds. However, in the dozens of buildings damaged or under construction which were observed by the author, the condition shown on the drawings was never observed. Instead, a third shorter piece of reinforcing steel of the size of the longitudinal reinforcement was side welded to the bars in each precast section. This resulted in an eccentricity between the longitudinal column reinforcement. In the worst case, which was frequently observed, the splice bar was between the two column bars, making the eccentricity between bar centerlines two bar diameters. This construction modification to the design detail had considerable impact on building performance, which will be discussed later.

The hollow core floor slab was grouted between units but had no interconnection. Thus, along the place between two hollow core floor panels, the only reinforcement or steel to cross that plane was the beam reinforcement at each column line.

Shear walls were provided for seismic bracing in the direction perpendicular to the moment frames. As can be seen in the framing plans of Figure 1, each building had two or three shear walls. The shear walls were 140mm thick precast and had embedded steel that was field welded to similar steel in the columns at each end of the wall. The precast wall panels were precast in one-story heights except for the depth of the hollow core floor panels. That depth was poured in place with the vertical wall reinforcement from below and above extending into the field cast joint. The walls were lightly reinforced and, in two of the plans, contained doorways. The spandrel or coupling beam between the vertically stacked doorways had heavier shear reinforcement and bottom steel cast in the precast section. A top bar for the coupling beam was shown on the drawings to be field cast in the horizontal joint at the floor.

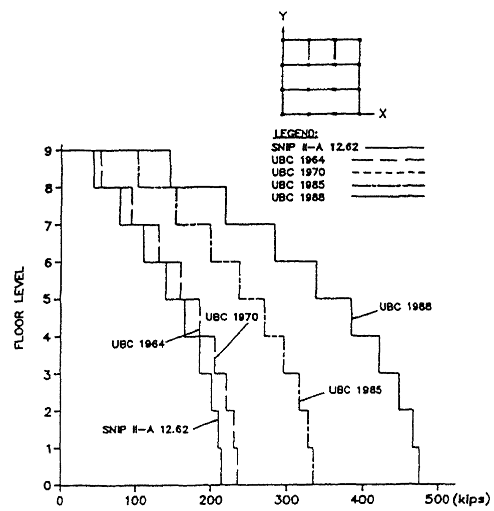
These buildings were typically nine stories high in Leninakan, although a few of twelve stories height appear to have been present. The buildings appear to have been built over a twenty or so year period to the same standard design, apparently the precast elements manufactured in the same precasting plant.

3 DESCRIPTION OF ANALYSIS

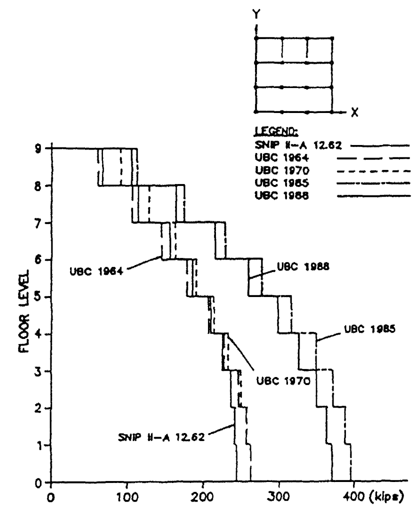
Two of the building configurations were analyzed by the computer program ETABS. The buildings were analyzed in each direction by dynamic analysis to determine periods of vibration and mode shapes. Since there were no recorded strong motion records for dynamic input, it was decided to use the equivalent static earthquake load of various building codes applied to the dynamic model. This was considered reasonable because of the regular nature of the buildings. The codes considered in the analysis were the SNiP 62 and the Uniform Building Code (UBC) of the United States, the 1964, 1970, 1985 and 1988 editions.

4 ANALYSIS RESULTS AND PERFORMANCE CORRELATION

The analysis indicated fundamental periods of vibration for the nine-story building of 1.31 to 1.38 seconds in



a. Seismic design shears in X-direction or moment frame direction.



b. Seismic design shear in Y-direction or shear wall direction

Figure 2 Seismic Shear for Various Disign Codes the moment frame direction and 0.99 to 1.07 seconds in the shear wall direction.

The analysis indicated that the design of the buildings did comply with the Soviet SNiP 62 code. Figure 2 shows the story shears for the square building configuration for the various codes studied. It is very interesting to note how close the Soviet code parallels the UBC of the United States at that time. However, it should be pointed out that the story shears calculated by the Soviet code contain a 1.4 factor which the author believes to be similar to a load factor in United States codes. The UBC story shears shown in the figure are unfactored shears.

The buildings did not meet many minimum reinforcement requirements of the 1964 UBC. However, the design of these buildings was not substantially non-conforming to that Code. Since 1973, seismic codes for concrete structures in the United States have required considerable special reinforcing details to provide ductile members and ductile performance. The Armenian Series 111 buildings do not meet those more recent U.S. code detailing requirements.

A review of the analysis and the damage suggests that the most probable cause of damage leading to collapse was the failure of the column splices with the eccentrically welded column reinforcement. Assuming some creep in the columns had transferred axial gravity loads to the column reinforcement, the eccentricity of the column reinforcement overstressed the column ties by factors of 3 to 16 times, depending on the actual configuration. These bars must have been on the verge of buckling prior to the earthquake. Figure 3 shows one of these splices in a damaged but sill standing building.

The next issue is that of the moment resisting frames. Beyond the column splices, the frames were analyzed to find the weak link under seismic loads. The analysis revealed the weak link was the beam shear capacity. The beams were not strong enough in shear to develop the reverse plastic hinge capacity of the beam at its ends. This was confirmed in field observations where columns in the vicinity of the beams and the beam-column joint were always uncracked, where as shear cracks were often evident in the beams. Most dramatic were conditions like Figure 4 with wide shear cracking at the end of the angle embedded in the column. In a few cases observed, the only thing holding up the beam was the bottom reinforcement welded to the embedded angle. Many modern codes have detailing requirements for seismic frames to prevent these shear failures.

The shear walls were analyzed and their weak link was the spandrel or coupling beams between the stacked doorways. This was consistent with observed damage, as in Figure 5, where these beams were completely shattered. This was consistent where these openings were observed in shear walls. In the building plan with solid walls, working of the connection to the precast column and cracking along the floor joint were also observed.

Another observation were offsets in the floor diaphragms. Although the analysis did not model the unitized format of the floor construction, cracking and offsets were observed in several diaphragms, especially adjacent to edge units which connected to the columns to provide column bracing stability.

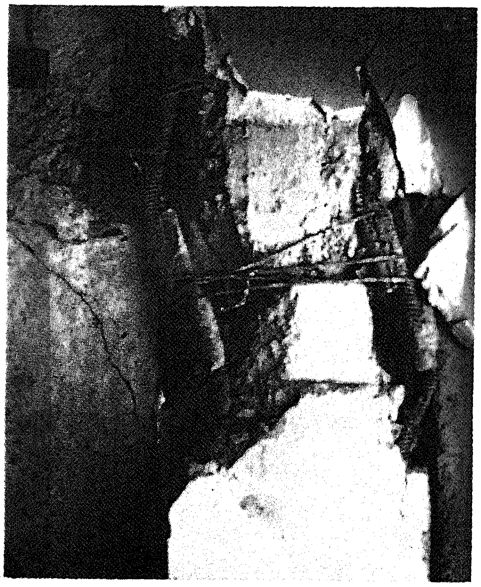


Figure 3 Column splice with buckled reinforcement at eccentric bar splice.

Figure 4 Beam column connection showing severe shear cracking in beams beyond embedded steel angle.

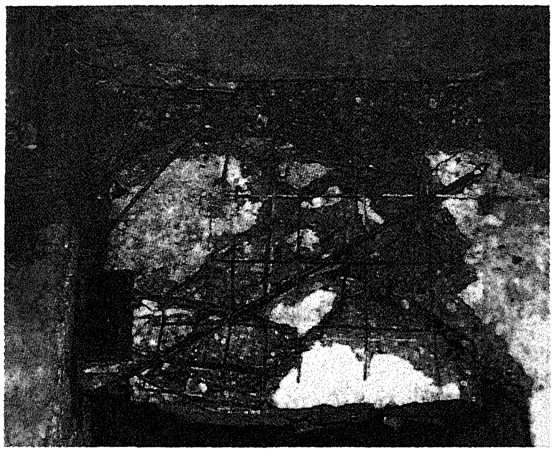


Figure 5 Shear wall coupling beam between wall at right and precast column at left between door openings.

5 CONCLUSIONS

The following conclusions are based on site inspections of collapsed and damaged Series 111 precast frame building in Leninakan following the December 7, 1988 earthquake and analysis of the structural systems:

1. The buildings were designed for seismic forces in accordance with the Soviet Code (SNiP 11-A 12.61). An analysis of the structural members indicates that they were designed for strength consistent with that code. It should be pointed out, however, that some members were reinforced only to meet the design forces, i.e., they had no overstrength or excessive capacity. Essentially, these buildings met California seismic design requirements of the 1960s.
2. The weakest link of the Series 111 buildings as constructed was the column splices with eccentricities between the weld spliced column vertical reinforcing. The eccentricities in these bars, combined with weak concrete in the splice region, led to bar buckling and a failure of the columns, leading to collapse.
3. Shear walls consisting of precast panels welded to columns had stacked door openings. The spandrel beams above or between these openings were lightly reinforced and they sustained substantial damage. Links like these beams which are weak in shear should be substantially reinforced to allow the rest of the wall to work in resisting seismic forces.
4. An analysis of the moment resisting frames of the Series 111 buildings revealed that the weakest link in these frames was the beams in shear. This is consistent with the field observations of damaged buildings where extensive shear cracking was observed in the beams at the end of the steel angle embedded in the column.
5. The diaphragms consisted of precast hollow core planks without a reinforced topping slab nor connections to adjacent planks. The only connections were provided at the beams. In damaged buildings, cracks were observed along panel joints. The weakness of the floor diaphragms prevented forces from being redistributed once a failure occurred in the building and contributed to the overall performance.

ACKNOWLEDGEMENT

The author wishes to express thanks to the Earthquake Engineering Research Institute, the National Science Foundation, the National Academy of Sciences and the United States Geological Survey for arrangements and support of the author's trips to Armenia. The assistance and cooperation of many Armenian and Soviet engineers is also appreciated. A portion of this work has been supported by National Science Foundation Grant BCS-8912147. The opinions expressed are those of the author and not necessarily those of the agencies mentioned.

REFERENCE

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