Seismic Risk Assessment and Mitigation Strategy in Armenia

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Introduction

Armenia is situated in a seismically active zone. Earthquake magnitudes reach M = 7.1 (according to historical and paleoseismic estimations). Focal depth is, on average, ten kilometers. All sources are located on active faults, with an average slip rate of about one centimeter per year. The duration of destructive earthquakes may last one minute under adverse ground conditions. The average recurrence interval of large earthquakes ($M \geq 5.5$) is about 30 to 40 years. The above-mentioned characteristics of the seismic regime indicate a high-level seismic hazard in Armenia (Balassanian, Martirosian, Nazaretian, Arakelian, Avanessian, Igumnov, Ruttener 1999: 227-236).

Since the hazard has been considerably underestimated, the seismic resistance of buildings and structures is well below the level required for this high magnitude of seismic risk. The absence of state policy in the field of seismic risk reduction led to the disaster in Spitak in 1988 (Balassanian, Melkumyan, Arakelyan & Azaryan 1999:43-45).

In 1999, the Government of Armenia adopted the strategy of seismic risk reduction. Several institutions under the coordination of the National Survey for Seismic Protection (NSSP) developed a uniquely structured program.

Seismic Mitigation Scheme

After the 1988 Spitak earthquake, the seismic risk assessment for Armenia was revised. According to this new assessment, the expected acceleration of earthquakes is 0.4g almost everywhere in Armenia (Seismic Code of the Republic of Armenia ÑÍÐÀ II-2.02-94). At the same time, the majority of buildings constructed before the Spitak earthquake were designed for seismic action of about 0.1-0.2g, according to the design codes of the former Soviet Union.

There is a pressing need to strengthen existing buildings. However, large-scale building strengthening is difficult even in developed countries. Based on feasibility, mitigation strategies envisage strengthening the most vulnerable buildings in Armenia. Since not all buildings will need strengthening, cities should:

- Development a seismic risk map
- Identify high risk districts and sequence them for seismic risk reduction
- Classify buildings in high risk districts
- Assess the maximum possible accelerations and prepare synthesized accelerograms
- Analyze non-elastic earthquake response and estimate potential building damage
- Prioritize buildings according to retrofit urgency
- Design retrofit methods for each type of building using conventional and/or new non-conventional structural concepts
- Retrofit the most vulnerable buildings, according to the priority list.

Seismic Risk Assessment

Based on the seismic risk map of Armenia, maximum seismic risk is concentrated around the capital city of Yerevan, where 40 percent of the republic's population resides. The seismic hazard risk for Yerevan is based on active faults and historical earthquake data. According to available historical data, the 1679 Garni earthquake was the most destructive in the centuries-old history of Yerevan.

By reproducing the 1679 Garni earthquake scenario, a general pattern of seismic risk was obtained (Balassanian, Melkumyan 1996). The following factors were considered:

- Seismic hazard level in Yerevan based on data obtained from time series
- Earthquake resistance of existing buildings and structures estimated from soil conditions and design types.
- The principle of seismic risk assessment is based on the definition of the seismic risk coefficient: $K_r=A_{exp}/A_{design}$, where A_{exp} is expected acceleration at the site of the building, and A_{design} is the design

> acceleration of the ground at the same site (Azaryan, Goroyan, Melkumyan, Voskanyan 2001:49-62). Analysis of the 1988 Spitak earthquake, as well as non-linear earthquake response analyses, has revealed that buildings and structures are able to resist seismic impact at the aggravation twice as much as the design value. (Inoue, Melkumian, Kumazawa, Nakano, Okada, 1991:57-64) Therefore, if $K_r > 2$, the seismic risk is high. If $1.2 < K_r < 2$, seismic risk is moderate. If $K_r < 1.2$, there is no risk.

Depending on soil conditions, expected accelerations are larger than the design values by 3.2, 2.0 and 1.2 in specific zones of the city. For each building, the seismic risk coefficient was calculated using GIS technology.

Data analysis has shown that about 26 square kilometers of the city are in a high seismic risk area. This represents 15 percent of the total city territory and 5,389 buildings. In addition, 44 square kilometers of the city have 34,143 low-storied, private stone houses that should be considered to be in seismic risk areas. The remaining 2,185 buildings have moderate risk.

Using existing design and archive data, as well as normative documents, a computer map of seismic risk damage (MSRDBS) was generated. According to the damage map, in the event of a strong earthquake with magnitude M=7.0, similar to the Garni Earthquake in 1679, most buildings would collapse and about 300,000 people could die in Yerevan.

The seismic risk map of Yerevan is the most important component of the strategy development. As a result, there is now a base file that provides an opportunity to identify further details in the MSRDBS. As a first step, a database for 20 buildings was created in order to estimate seismic resistance.

Establishing the Database for 20 Buildings

Building documentation from the a1 and a2 micro districts of the southwestern part of Yerevan was used to select 20 buildings for a pilot study. In addition to the methods described (see attachment one for more details and results), a visual survey of the buildings and estimates of the seismic resistance of reinforced concrete buildings were used to determine potential seismic risk (FEMA 1998 and Melkumyan 1996; Melkumyan 1998:73-79). As a result, a database for 20 buildings exists. As an example, data on a specific building is presented in table 1. It is necessary to note that in reality, the database on each building is comprehensive and includes information on the physical strength of all construction elements, possible structural damage and building alterations.

Table 1. Data on building N141 in the A1 Micro district in southwestern Yerevan

Design acceleration		0.2g
Type of the building		apartment building
Structural design		reinforced concrete frame
Stories		9
Series		111
Vibration period T_x, sec in transverse direction	design	1.013
	experimental	0.56
Vibration period T_y, sec in longitudinal direction	design	1.631
	experimental	0.62
Average strength of concrete by stories, kgf/cm^2	1	240
	2	255
	3	260
	4	246
	5	255
	6	245
	7	265
	8	250
	9	234

Mitigation Strategy

Taking seismic risk to Yerevan and other Armenian cities and towns into account, a long term state program for seismic risk mitigation was initiated with the involvement of ministries, departments and public and private organizations. NSSP coordinated the work. The government and president of Armenia adopted the program in 1999.

The program includes a number of issues including:

- Seismic codes and standards
- Seismic strengthening and upgrading of existing buildings
- Earthquake resistant construction
- Education of population
- Emergency response and rescue operations.

The section on seismic codes includes the first National Standards of Seismic-Resistant Construction. An interdepartmental joint committee with experts from several institutions designed the standards and the Ministry of Urban Development confirmed them on April 1, 1995.

Seismic strengthening and upgrading of existing buildings incorporates new innovative methods of retrofitting. Two of them are described below.

In the area of earthquake-resistant construction, the Ministry of Urban Development has adopted a new construction policy. Educational materials for all population age-groups are under preparation to improve their preparedness for seismic hazards; special training materials for preschool-age children have been designed; and presentations in schools, universities, and TV and radio broadcasts aim to increase knowledge and awareness of seismic risk.

The Emergency Management Administration (EMA) under the Armenian government with support from UNDP launched the national disaster management training program in “emergency response and rescue operations.”

Within the programs listed above, the seismic strengthening and upgrading of existing buildings is the most expensive, long term and difficult task for Armenia.

Using New Technology in Retrofitting Existing Buildings

As indicated earlier, one of the tasks of mitigation is to reduce the seismic risk of buildings. But in Armenia, where people are suffering the difficulties of the post-Soviet transition period, the implementation of conventional retrofitting techniques creates difficult conditions. Non-conventional approaches, that allow the retrofitting of existing buildings without interruption in use, would be preferable. To support this approach, new retrofitting techniques were introduced based on the application of seismic isolation structures.

New technology was used in retrofitting a nine and a five-storey building in Yerevan. The nine-storey reinforced concrete (R/C) buildings (series 111) are designed as precast framed systems. Horizontal stiffness is provided in the longitudinal direction by the frames with strong beams and in the transverse direction by the frames with weak beams and shear walls (Melkumian, 1996). Precast columns with a cross-section of 40 by 40 centimeters are designed with the length of three-storey, precast strong beams with a cross-section of 40 by 52 centimeters (including thickness of slabs). They are designed with the length of one span. Weak beams with a cross-section of 120 by 25 centimeters are also designed with the length of one span. The slabs of these buildings include precast void 22 centimeter-thick floor panels of different widths. Shear walls are also prefabricated and 14 centimeters thick. All joints of factory-produced precast elements such as column joints, columns, beams and shear walls have used on-site welding. The joints of floor panels, shear walls and weak beams use connections of reinforcement. Exterior walls of precast concrete are attached to the outer face of the frames. Buildings designed in the above-described methods were widely used in Armenia, particularly in Gumri and Vanadzor. These types of buildings were heavily damaged during the 1988 Spitak earthquake.

This project aimed at upgrading seismic resistance of a nine-storey frame R/C building with shear walls using new technologies (seismic isolation or AIUF=Additional Isolated Upper Floor method). As an innovation, it applied seismic isolation structures in the upper part of the building rather than in its base. To improve the building's earthquake resistance, various laminated rubber bearings (HDRBs) were used.

The building had a square plan with 16 columns having 6 meters between them. All columns passed through the slab of the ninth floor, one meter into the attic space. For the project, 16 bearings were used. The retrofitting methodology required that HDRBs be installed in each column (jacket). The AIUF represents a steel frame structure with the same number of columns as the building. The base of each column is a steel plate bolted to the upper recess rings of HDRBs. All steel columns of the AIUF were connected to each other by steel trusses. On the level of upper belts of trusses, a 22-centimeter R/C slab is designed using precast panels. The plane roof and exterior walls of AIUF were designed using light "sandwich" type elements. In essence, the additional floor itself represents a rigid structure that, during earthquakes, is supported by HDRBs.

During an earthquake, AIUF acts as a vibration damper, reducing stress-deformation and doubling a building's earthquake resistance rate (Melkumian, 1996). AIUF reduces the shear forces on the building by increasing the time period of vibration and decreasing the first mode vibration coefficients. It also changes the second mode vibration and as a result, AIUF oscillates the building in anti-phase. All these factors contribute to the reduction of shear forces and horizontal displacements.

It is worth noting that the isolated upper floor allows not only upgrading a building's earthquake resistance, but also enlarges its useful space. The most distinctive feature of the new earthquake resistance upgrading method, however, is that tenants can stay in the building during construction works. Opponents may argue that the method improves the earthquake resistance only by a coefficient of 1.6. Nevertheless, the seriousness of the housing problem and the high seismic risk justifies any, even the slightest upgrading of earthquake resistance for existing buildings.

The pilot project included the retrofitting of a five-storey stone building as well. Buildings of this type (series 1A-450) can be found in all regions of Armenia. They have 45 to 50-centimeter thick bearing walls located mainly in a transverse direction. The horizontal stiffness in the longitudinal direction is provided partly by R/C frames with strong beams and columns, made inside the body of walls, and by longitudinal walls at the edge of buildings. The evaluation of the Spitak earthquake has shown that the most vulnerable parts of these buildings are the edges where the direction of bearing walls changes. In these areas, plastic deformations resulted in building resistance failure due to weak connections between longitudinal and transverse walls.

The newly developed structural concept aims to address the above weakness of the buildings. (Melkumyan, 1994; Melkumyan, 2002:344-352). The idea is to provide seismic isolation in the building's foundation. Using a two-stage system of R/C beams, the technology sets isolators into the walls at the level of the foundation's upper edge. During construction, tenants can also stay in the building.

With this new technology, openings are made in the basement and the bearing walls to accommodate lower reinforcement frames with seismic isolator sockets. Binding reinforcement lower frames are passed through both sides of bearing walls and set in concrete. The next step is to place seismic isolators in the lower sockets. Here again, a simple recess connection detail was chosen. Upper sockets and upper reinforcement frames are placed on the isolators, passing upper binding reinforcement frames along both sides of bearing walls and set in concrete. In setting concrete frames, ends of binding reinforcement frames are left free beneath and above seismic isolators. In the parts of walls between seismic isolators, openings are made where binding reinforcement frames are placed, providing additional reinforcement frames. The latter ties binding reinforcement frames of neighboring seismic isolators. Thus, continuous upper and lower beams are formed along all bearing walls of the building. Parts of walls between seismic isolators are removed and the building is separated from its original foundation and linked to it only with seismic isolators.

It is important that openings in walls are made with single-spacing. Two adjacent openings should not be made simultaneously; parts of walls between seismic isolators should be cut out beginning with the middle of the building. Currently, retrofitting of one building using base isolation has been completed in the city of Vanadzor, and the residents were able to remain in the building during retrofitting.

Summary

A precise assessment of seismic hazard (SHA) is important and the success of seismic risk reduction activities depends upon this assessment. The assessment involves several uncertainties and requires a probabilistic approach. This must be complemented with hazard monitoring. Monitoring of the current seismic hazard represents essentially a short-term prediction of seismic events, one of the most complicated tasks of seismology.

In several developing countries where seismic risk is high and hazard risk is increasing due to urbanization, seismic risk reduction requires state policy and a government agency with responsibility for developing and implementing such a policy. The twenty-first century strategy of reducing seismic risk should give priority to mitigation and preparedness over recovery.

Armenia has taken initial steps in this direction. A seismic mitigation strategy has been developed and endorsed. The seismic risk map of Yerevan has been constructed. Based on this map and by using the newly developed assessment method of seismic resistance in a pilot project, a comprehensive building stock database was created for a district in Yerevan. Data analysis has revealed that for most buildings, seismic resistance is inadequate. Therefore, special attention should be given to seismic strengthening and upgrading existing buildings.

Within a short period of time, Armenian researchers developed unique and effective seismic protection methods. The paper describes the structural concepts of these technologies.

The proposed non-conventional methods are based on the application of different types of seismic isolators. Two projects were developed and implemented in Vanadzor, Armenia. The first upgraded the earthquake resistance of a nine-storey apartment building using AIUF and the second retrofit a five-storey apartment building using seismic isolation. In both cases, tenants could stay in their apartments during construction.

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Attachment

Seismic Resistance Assessment for Reinforced Concrete Buildings: Methodology

Quantitative assessment of seismic resistance compares the actual resistance (**A**) with the required one (**R**). It is clear that the ratio **A**/**R** must be greater than or equal to one for safe buildings. This ratio must be computed for each storey of the building.

A can be identified from vertical load bearing structures, based on their actual technical condition and the actual strength of materials. **R** is the value of the total shear force on each storey of the building during the scenario earthquake. **R** is calculated based on the seismic code effective in the Republic of Armenia.

To calculate "**A,**" characteristics of the construction materials are used. They can be obtained from building design schetches. In the absence of design sketches, design characteristics of similar buildings should be used. When the design sketches of a building are missing, it is difficult to establish with certainty the quantity and diameters of the reinforcing bars in the cross sections of the load-bearing structures and the diameter of the transverse reinforcement. It makes the calculation of **A** especially difficult and, in a number of cases, impossible. In cases where basic information is missing on a building's load-bearing structures, the method described below can be used.

First of all, it is necessary to determine the actual strength of all vertical loadbearing structures for the building in question. Further, based on the known values of actual strength of various structures from the effective normative documents, the values of elasticity module can be determined. After collecting the physical and mechanical characteristics of all bearing structures of a building, it is necessary to find values for the horizontal stiffness and geometrical dimensions of the bearing structures.

If horizontal stiffness of all structures is defined, **A** for the given storey can be calculated using the following formula:

$$A = \Delta \cdot K \cdot \sum_{i=1}^{n} C_i \quad (1)$$

where C_i – horizontal stiffness of the i vertical bearing element of the given storey; Δ - ultimate displacement for the elastic stage, given in table 2; K –

stretch coefficient, given in table 2 as well; n – number of the vertical bearing structures.

Table 2. Coeeficients to Identify "A" for Reinforced Concrete Buildings

Building's structural system	Δ	K
Frame with weak beams	h/1250	4,5
Frame with strong beams	h/1500	4,2
Braced frame with solid reinforced concrete shear walls or a system with solid bearing walls	h/2000	4,0
Braced frame with shear walls or bearing walls, having openings	h/3150	3,7

h – ceiling's height

The values of Δ and K, listed in table 2, have been calculated using experimental research on the seismic resistance of various types of reinforced concrete structures (Melkumyan, 1993). As new information emerges on the behavior of reinforced concrete structures during the seismic actions, these coefficients can be fine-tuned.

Seismic Resistance Assessment for Reinforced Concrete Buildings: Results

Following the above-mentioned method, the seismic resistance for all 20 buildings, selected in the A1 and A2 microdistricts in Yerevan, have been estimated. We can demonstrate it using the example of the nine-storey building N141. To determine the requested load-bearing capacity **R**, it is necessary to quantify the seismic forces as well.

The following initial assumptions were used. The masses of the building's stories are equal to: $m_1 = \ldots = m_8 = 218$t, $m_9 = 260$t, and the module of elasticity of the column is E_K=19000 MPa and the module of the shear walls is E_d=15000MPa. (Melkumyan, 1996)

For calculating the seismic forces, the design complex "Mirage" has been used. For 0.2g acceleration, the following values of the total shear seismic forces, at each storey of the building were found: Q_1^R=118.3t, Q_2^R=117.6t, Q_3^R=115t, Q_4^R=110.0t, Q_5^R=101.6t, Q_6^R=89.5t, Q_7^R=73.4t, Q_8^R=53.1t, Q_9^R=18.6t in transverse direction, and Q_1^R=109.1t, Q_2^R=106.7t, Q_3^R=101.3t, Q_4^R=93.0t, Q_5^R=82.1t, Q_6^R=68.8t, Q_7^R=53.5t, Q_8^R=36.6t, Q_9^R=18.6t in longitudinal direction of the building. The design values of the periods of the first vibration mode of the building is equal to: T_x=1,013sec in the transverse direction and T_y=1,631sec in longitudinal direction.

To determine the actual load-bearing capacity, the values of the actual concrete strength from table 1 of the database were used. With the purpose of simplifying the computation, the average value of the concrete strength was equalled to 250kgf/cm^2 for the columns, beams and strength diaphragms on all stories. Therefore, the elasticity module of the columns, beams and the shear walls will be equal to 15800 MPa. Now we are able to determine the total actual strength of each building storey. Omitting the intermediate computations, we will give the final values of each storey's stiffness: C_K=31.0 t/mm in a transverse direction and C_K=31.0 t/mm in the longitudinal direction.

The ultimate displacement for the elastic stage Δ is from table 2. In a transverse direction Δ_x=3000/2000=1,5mm, as the building has a weak scheme with solid shear walls in this direction. In the longitudinal direction - Δ_y=3000/1500=2,0mm, as in this direction the building has a frame scheme. Accordingly, K_x=4.0 and K_y=4.2. With the above values, the total shear forces can be calculated from all vertical load-bearing structures for each level. These values are: $Q_1^A = Q_2^A = \ldots = Q_9^A$ = 186t in transverse direction and $Q_1^A = Q_2^A = \ldots = Q_9^A$= 111.5t in longitudinal direction.

Comparing the actual and required force values for each level and each direction we found that the ratios are above one. Therefore, the seismic resistance of the building is acceptable. However, a very important detail must be noted. The conclusion is derived, based on the Code of the USSR ÑÍèÏ II-7-81 which was in effect when the building was built. Currently, the ÑÍÐÀ II-2.02-94 Code is in force in Armenia. According to the new norms, the forces, corresponding to the requested load-bearing capacity will be equal to: Q_1^R=459.3t, Q_2^R=457.3t, Q_3^R=447.6t, Q_4^R=427.6t, Q_5^R=394.9t, Q_6^R=347.7t, Q_7^R=284.9t, Q_8^R=206.1t, Q_9^R=111.6t in transverse direction and Q_1^R=369.1t, Q_2^R=361.5t, Q_3^R=343.2t, Q_4^R=315.2t, Q_5^R=278.2t, Q_6^R=233.2t, Q_7^R=181.4t, Q_8^R=124.2t, Q_9^R=63.2t in a longitudinal direction.

The values of the required load-bearing capacity, according to the ÑÍèÏ II-7-81 and to the CHPA II-2.02-94, as well as the values of the actual load-bearing capacity, are listed in table 3.

Table 3. Values of the Required and Actual Load-bearing Capacities

Required bearing capacity (ÑÍèÏ II-7-81)		Required bearing capacity (ÑÍÐÀ II-2.02-94)		Actual bearing capacity	
X	y	X	y	x	y
$Q_1^R=118.3$	$Q_1^R=109.1$	$Q_1^R=459.3$	$Q_1^R=369.1$	$Q_K^A=186$	$Q_K^A=111.5$
$Q_2^R=117.6$	$Q_2^R=106.7$	$Q_2^R=457.3$	$Q_2^R=361.5$		
$Q_3^R=115.0$	$Q_3^R=101.3$	$Q_3^R=447.6$	$Q_3^R=343.2$		
$Q_4^R=110.0$	$Q_4^R=93.0$	$Q_4^R=427.6$	$Q_4^R=315.2$		
$Q_5^R=101.6$	$Q_5^R=82.1$	$Q_5^R=394.9$	$Q_5^R=278.2$		
$Q_6^R=89.5$	$Q_6^R=68.8$	$Q_6^R=347.7$	$Q_6^R=233.2$		
$Q_7^R=73.4$	$Q_7^R=53.5$	$Q_7^R=284.9$	$Q_7^R=181.4$		
$Q_8^R=53.1$	$Q_8^R=36.6$	$Q_8^R=206.1$	$Q_8^R=124.2$		
$Q_9^R=18.6$	$Q_9^R=18.6$	$Q_9^R=111.6$	$Q_9^R=63.2$		

The actual seismic forces remain unchanged. Comparing them with the new values of the required seismic load-bearing capacity, it can be seen that the ratios in general are below one. Therefore, considering the new building codes in Armenia, it can be concluded that the seismic resistance of the buildings studied is not satisfactory. A similar conclusion has been reached for the remaining 19 buildings of the A1 and A2 microdistricts of the southwestern region.