論文 Damage of Reinforced Concrete Frame Structures during the Iran-Qayen (Ardakul) Earthquake of May 10, 1997

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ABSTRACT: A large number of engineered as well as non-engineered buildings were severely damaged and collapsed during a magnitude 7.1 earthquake in Iran. This paper reports the damage on R/C frame structures and describes their possible causes. It is revealed that most of the R/C framed structures suffered damages due to the lack of seismic resistance design practices and poor reinforcement detailing. Results of nonlinear earthquake response analysis of a typical damaged building showed that the degree of damages could greatly decrease with a proper earthquake resistance design and reinforcement detailing.
KEYWORDS: investigation, earthquake response, nonlinear analysis, damage, collapse

1. INTRODUCTION

All 66 single-story reinforced concrete buildings were entirely collapsed in Ardakul, when an earthquake with a surface wave magnitude Ms of 7.1 struck the Qayenat Township in northeastern parts of Iran on May 10, 1997 (**Fig. 1**). A large number of RC buildings in other villages also suffered severely damages in the region. The epicenter was located between Hajiabad and Ardakul (33.55°E, 59.98°N) at a focal depth of 10-12 km [1]. The nearest available uncorrected acceleration time histories recorded in Marak [2], and related pseudo elastic velocity response spectra are shown in **Fig. 2**. A reconnaissance report revealed that most of the R/C frame structures suffered damage due to lack of seismic resistance, especially due to poor reinforcement detailing [1]. However, the rational cause of collapse, based on analytical results, was not studied. The objective of this paper is to study the real cause of the collapse of buildings using nonlinear earthquake response analysis.

2. REINFORCED CONCRETE FRAME STRUCTURES

Traditionally, single-story adobe buildings have been used for construction in majority of the region. The 1979 Karizan earthquake (**Fig. 1**) destroyed traditional adobe buildings in some villages. A number of villages were completely moved to other places and some were re-constructed in the same location by the government. The predominant structural system used for buildings in re-constructed villages consists of reinforced concrete frames with unreinforced masonry infills (mostly brick, rarely concrete masonry). This type of construction is used for most of residential units with a nearly identical plan. A typical reinforced concrete frame building in the region consists of a regular, symmetrical floor plan, with square or rectangular columns, connecting beams and girders (**Fig. 3**). The roof consists of a relatively thick reinforced concrete slab covered by an additional layer of mud or concrete in many cases to insulate the slab. The exterior enclosures

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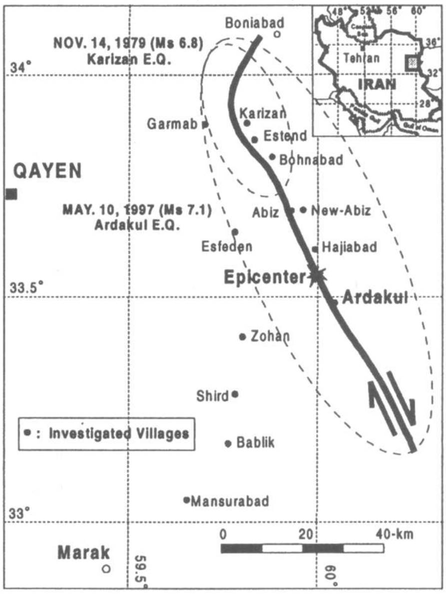


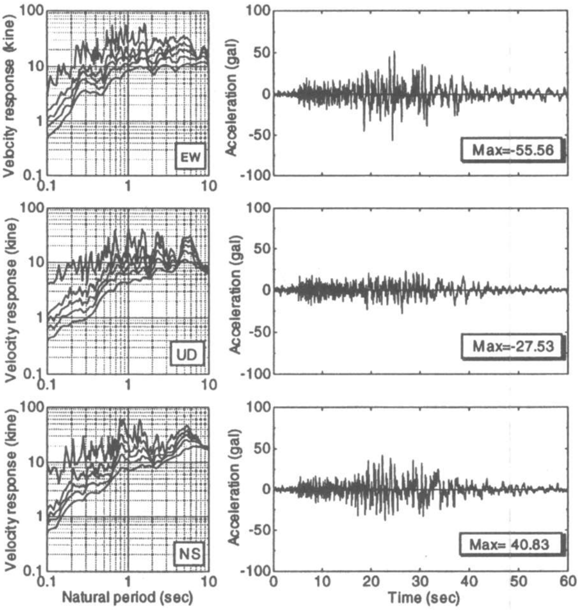
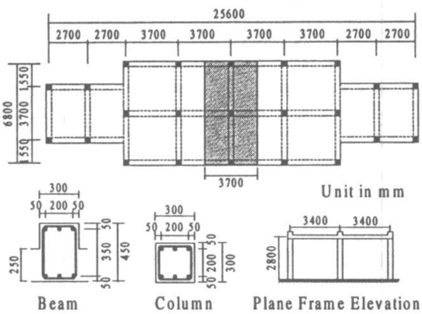


Fig. 1. Affected areas, investigated villages, and ruptured surface faulting (110 km)

Fig. 2. Acceleration time history and response spectra (Damping factors 0.0, 0.02, 0.05, 0.10, and 0.20)

as well as interior partitions are constructed with non-bearing unreinforced brick masonry infill. These masonry walls contributed to the lateral stiffness of the buildings during the earthquake and, in many instances, controlled the lateral drift and resisted seismic forces elastically. This was only true when the bricks and grout were bonded sufficiently with good construction practice. When structural response and deformation demands were very high, the masonry walls were not able to remain elastic. In such buildings, typical diagonal cracks of exterior as well as interior brick walls were observed. In most cases, the out-of-plane failure of infill walls occurred. Once the brick infills failed, the lateral strength and stiffness had to be provided by the frame alone, which then experienced significant inelastic response in the critical region of the members. Most reinforced concrete frame buildings suffered damage due to the lack of seismic resistant design practices and poor reinforcement detailing. The following sections are devoted to the discussion of design and detailing observed, and the consequential structural damage, with relevant examples.



Reinforcement	Beams	Columns	Slab(2-layer)
Longitudinal	6 - D12	6 - Φ12	Φ12@250
Transverse	Φ6@250	Φ6@250	Φ12@250

Fig.3. Typical single-story R/C frame structure

2.1 STRONG BEAMS, WEAK COLUMNS, AND HUGE MASS

The observed behavior of frame structures indicated that strong beam remained elastic, and weak columns suffered compressive crushing or bending failure but rarely shear failure. In many cases relatively deep beams were used with slender columns, contributing to the strong-beam weak-column behavior. Moreover, the relatively thick slab compounded this problem with enhancing the beam stiffness. **Photo 1** shows a typical R/C frame structure, in which strong beams remained elastic while weak columns experienced flexural hinging. Traditionally, to prevent the water penetration a layer of mud-straw is used for thatching the roof of adobe buildings. As long as water penetrating continued, additional mud-straw layer is used to cover existing one. This method, applied in some R/C frame structures, lead to a heavy slab. Some owners used a layer of concrete for thatching the roof of R/C buildings to prevent water penetrating as well as strengthening the structure. Absolutely, this additional mass was not considered in earthquake resistant design process and can cause some damages. It is worth mentioning that as a typical building in Iran, asphalt was used to cover the roofs in some cases.

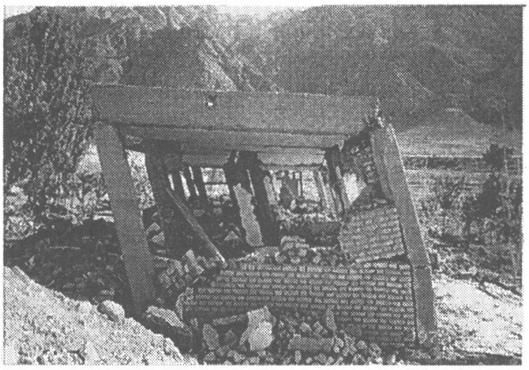


Photo 1. A typical R/C building (Ardakul)

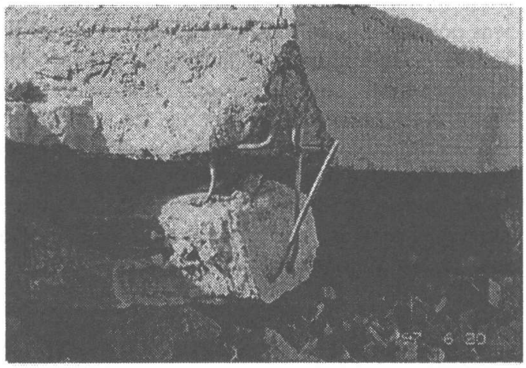


Photo 2. Joint failure and slip out of plain bars

2.2 LACK OF COLUMN CONFINEMENT

Most of the structural damage observed in frame buildings was concentrated at column ends. Measurements taken from some of the columns indicated a widespread use of 6 mm plain steel bars as hoop reinforcement with 90° bent hook at their ends, placed at a spacing of approximately 200 mm to as wide as 400 mm. A number of columns failed by premature crushing of concrete due to the lack of confining reinforcement, followed by buckling of longitudinal reinforcement. It should be noted that the Iranian Seismic Code revised in 1988 [3] called for confinement reinforcement at the both ends of columns in moment resisting frames with intermediate ductility by means of limiting the maximum spacing of closed hoops (ties). Nevertheless, the minimum bend of hooks as well as minimum bar diameter used for hoops are not specified explicitly in the revised Iranian Seismic Code. The use of extremely slender columns caused bending failure prior to shear failure. No diagonal tension failure was observed under any circumstances. Use of masonry walls, providing unintended supports, results in captive columns in many buildings by increasing shear vulnerability of these members. However, author observed no evidence of short column behavior including shear failure.

2.3 POOR JOINT PERFORMANCE & LACK OF GOOD REINFORCEMENT DETAILING

Joint shear failure was observed widely in the damaged buildings. Although the current Iranian Seismic Code [3] calls for continuing the column hoops into beam-column joints, virtually no joint reinforcement was observed in the buildings (**Photo 2**). One of major reasons of damage was the lack of good detailing practice. One common inadequate point was the use of longitudinal round bars in column without any anchorage hook at the end. This caused the bars to slip out at the critical regions, leading to unintended premature failure at the column ends, which resulted in collapsing of the buildings. An example of this fact is illustrated in **Photo 2**.

2.4 BEHAVIOR OF MASONRY INFILLS & CONCRETE QUALITY

The masonry infills were generally constructed with non-reinforced solid mud-brick jointed with cement mortar. Although the masonry infills used in reinforced concrete frames were initially employed as non-bearing partitions; they suffered heavy damages during the earthquake. As it was mentioned earlier, extremely slender columns used in construction lead to a high lateral sway. To resist this lateral drift, well-constructed infills carried earthquake force in an elastic range. However, when the intensity was high in some regions, the diagonal shear failure occurred in masonry infills causing structure to collapse. This mechanism was not effective in most cases because the infills were broken-down completely due to out-of-plane failure. The absence of anchorage between infill and frame and the use of poor mortar were the reasons of out-of-plane failure. It is worth noting that mud also used widely as the mortar in many cases. As expected, mud mortar did not show good performances during the earthquake. Revised Iranian Seismic Code [3] requires a minimum specified cylinder strength of 20 MPa for concrete used in reinforced concrete frames with intermediate ductility. In some buildings, test of Schmidt Hammer was conducted [1] to estimate the compressive strength of the concrete(**Table 1**). In most cases, the cylinder strength was over 30 MPa. Consequently, no evidence of structural failure due to low quality of concrete was observed.

Table 1. Average compressive strength of concrete f'_c (MPa)

Village Name	Position	Strength
Bohnabad (School)	Column	24.5
	Column	35.0
	Beam	23.5
Ardakul (Residence)	Column	35.0
	Column	42.5

3. DESIGN EARTHQUAKE LOADS IN IRAN

The Iranian revised code for earthquake resistant design of buildings was issued by Building and Housing Research Center of Iran in 1988. The design base shear coefficient C was defined as a function of fundamental period T, design base acceleration A for three zones, behavior coefficient of buildings R, importance factor of buildings I, and spectral parameter B, T_0 and T_1 for the four soil types.

$$C = \frac{ABI}{R} \qquad \begin{array}{ll} B=2.0 & \text{for } T<T_0 \\ B=2.0\,[T_0/T]^{2/3} & \text{for } T_0<T<T_1 \\ B=0.6 & \text{for } T_1<T \end{array}$$

where ***A*** is 0.35, 0.25, and 0.20 for high, intermediate, and low seismic relative hazard zones of Iran, respectively; ***I*** is 0.8 for not important buildings, 1.0 for normal buildings, or 1.2 for important buildings. The factor ***R*** represents the building capacity to dissipate energy, and includes the effects of material, damping, ductility capacity, and so on, as shown in **Table 2**. ***B*** is response coefficient of the building, which shows the response of building in relation to the design base acceleration. T is fundamental period of building, and T_0 and T_1 are corner periods based on soil types that indicate in **Table 3**. The values of the design response spectrum for the four types of soils are shown in **Fig. 4**. The affected area falls in high seismic relative hazard zone of Iran with A of 0.35. Considering I=1 and R=5 for a residential R/C framed structure building, the base shear coefficient C becomes 0.07B as illustrated in the right side of **Fig. 4**.

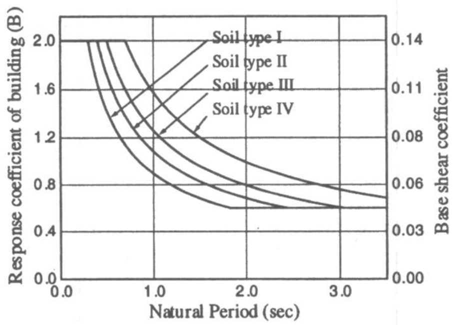


Fig. 4. Design Seismic Load and Base Shear Coefficent for Iranian Code [3]

Table 2. Behavior coefficient, R

Structure System	Lateral force resisting system	R
Bearing Wall	R/C shear wall	5
	Reinforced Masonry shear wall	4
Space Frame	R/C shear wall	7
	Bracing	7
	Reinforced Masonry shear wall	5
Space Frame	R/C frame	5
	Steel frame	6
Space Frame	Frame+ R/C shear wall	8
	Frame+ Bracing	8

4. NONLINEAR EARTHQUAKE RESPONSE ANALYSIS

Table 3. Design spectral parameter of Iranian code

Soil Type	Description	T_0	T_1
I	Igneous rocks. Conglomerate beds up to 60 meters from bedrock.	0.3	1.83
II	Loose igneous rocks. Conglomerate beds exceed 60 meters from bedrock.	0.4	2.43
III	Disintegrated rock. Gravely and sandy beds up to 10m from bedrock.	0.5	3.04
IV	Soft and wet deposit. Gravely and sandy beds exceed 10m from bedrock.	0.7	4.26

The structure was idealized as a 2-bay 1-story plane frame and then was subjected to the ground motion acceleration records recommended by Iranian code and the El Centro NS component. To evaluate the seismic resistant capacity of the structure but with a proper reinforcement detailing at joints, the effect of joint damage and slip out of the reinforcement was ignored. The maximum velocity of a record was computed and PGA of the input acceleration records was scaled for a PGV of 50 cm/s to simulate a large earthquake. A nonlinear frame response analysis program CANNY [5] was used to simulate the inelastic behavior of the structure during an earthquake. The foundation was assumed to be rigid and with no rocking. The analysis was carried out only in the direction of collapse of structure, i.e. **Photo 1**. Total weight of frame was estimated equal to 168 kN. The mass assumed to be lumped at the nodes of floor level of frame. Beams and columns were modeled with one component model [4] that assumes the moment contra-flexural point at mid-span of the members. The effect of shear deformation and moment-axial force interaction were neglected in the analysis. A constant axial force was considered in estimating section strength. Takeda model [6] was used to model hysteric behavior of moment-rotation relationship. Unloading stiffness degradation and reloading stiffness deterioration was ignored to obtain high-energy dissipation. Newmark method with a β value of 0.25 and a damping constant of 0.05 proportional to instantaneous stiffness was used to solve the equation of motion.

4.1 MATERIAL PROPERTIES AND SECTION ANALYSIS

Table 4. Steel bars used in Iran for R/C structures

Type	Expression	Elasticity Modulus (MPa)	Yeild Strength (MPa)	Tensile Strenght (MPa)	Fracture Strain
A1	Plain round bar	2.1×10^5	240	380	0.25
A2	Deformed bar	2.1×10^5	300	500	0.19
A3	Deformed bar	2.0×10^5	400	600	0.14

The typical steel bars used in construction of reinforced concrete buildings in Iran are indicated in **Table 4**. The plain round bar (type A1) and deformed bar (type A2) were used in columns and beams, respectively. A compressive strength f'_c of 30 MPa was assumed for concrete based on Schmidt Hammer test (**Table 1**). Tensile strength of concrete was assumed to be 10 percent of compressive strength. Initial elasticity Young's modulus of concrete E_c was 2.92×10^4 MPa based on a secant value recommended by AIJ (Architectural Institute of Japan).

Results of section analysis are shown in **Table 5**. A parabolic stress-strain relation was assumed for concrete in compression zone at cross section of members, and moment-curvature relation was calculated for a given constant axial force. The stiffness reduction factors α, and β of moment-rotation relations were calculated based on the integration of moment-curvature along a member axis. Beams were modeled as an inverted T-beam shape by assigning an effective flange length of 37 cm in each side as recommended by AIJ.

4.2 RESULTS OF ANALYSIS

The fundamental period T of the undamaged building was calculated as 0.157 sec. The maximum rotational ductility responses of member ends and base shear responses of the building are shown in **Table 6**. The ultimate moment resistance M_u of column ends leads to a base shear capacity of 0.38; which is 2.71 times of base shear load required by Iranian code (**Fig. 4**). While exterior

Table 5. Input data for nonlinear analysis of a typical damaged building

Member	Axial Force (kN)	Cracking Moment (kN.m) $+M_{cr}$	$-M_{cr}$	Yielding Moment (kN.m) $+M_y$	$-M_y$	Ultimate Moment (kN.m) $+M_u$	$-M_u$	Stiffness reduction factor after cracking $+\alpha$	$-\alpha$	Stiffness reduction factor after yielding $+\beta$	$-\beta$
Ext. Columns	42	11.24		23.97		28.22		0.1119		0.0113	
Int. Column	84	15.18		28.75		33.37		0.1171		0.0118	
T-Beam	0	25.25	30.07	51.10	87.57	69.90	105.29	0.0551	0.1090	0.0127	0.0106

Table 6. Rotational ductility demand of members, and base shear response of structure

Earthquake Name	Maximum Acceleration PGA(gal)	Scaled PGA(gal) V_{max}=50kine	Exterior Columns Btm.	Top	Interior Column Btm.	Top	Beams Corner	Inside	Base Shear Factor
Marak (EW)	-55.6	-371	2.24	2.09	2.11	2.02	0.10	0.03	0.37
Marak (NS)	40.8	189	1.02	0.99	0.99	0.99	0.07	0.03	0.35
Elcentro (NS)	341.7	511	4.76	4.47	4.52	4.52	0.26	0.03	0.43
Abbar [3]	554.2	986	6.85	6.78	6.62	6.62	0.35	0.04	0.47
Tabas [3]	915.4	755	7.04	6.74	6.64	6.63	0.36	0.04	0.47
Naghan [3]	-709.5	-418	2.35	2.20	2.26	2.18	0.21	0.03	0.38

and interior columns were yielded in almost all cases, the base shear capacity of structure seems to be enough to prevent the collapse of building in case of Marak and Naghan earthquakes. However, in the real structure, the use of longitudinal plain bars without hooks, leaded to a premature failure at the column ends, which resulted in premature collapse of buildings. On the other hand, if the premature failures at column ends due to slip out of bars were prevented, the structure could resist not only the Iranian earthquake-loading requirement [3], but also some other large scale strong ground motions.

5. CONCLUSIONS

The use of non-ductile unreinforced masonry infills in R/C buildings resulted in the catastrophic damage. The absence of anchorage as well as the use of poor mortar was the cause of out-of-plane failure of unreinforced masonry infills. Beyond the elastic limit, seismic survival of the building depended heavily on ductility of the structural components. In reinforced concrete frames, the damage could be attributed to the failure of column and beam-column joints. Lack of column confinement, inadequate splice length and bond slip of longitudinal bars, using plain bars, and poor reinforcement detailing can be listed as the primary causes of the damage, especially since the column ductility demand was high due to the presence of strong beams.

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