

DETAILED ANALYSES OF STEEL AND REINFORCED CONCRETE BUILDINGS DAMAGED BY THE 2003 BAM EARTHQUAKE IN IRAN

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ABSTRACT :

Two steel and two reinforced concrete (RC) buildings damaged by the Bam earthquake on December 26, 2003 in Iran with magnitude $M_w=6.5$ is selected for detailed damage evaluation using linear and nonlinear analysis procedures. The recorded ground motion of the Bam station with Peak Ground Acceleration (PGA) of 0.76g in longitudinal (East-West) direction is considered for nonlinear dynamic analyses. Studied buildings had 3, 4 and 5 stories in height, and were located about 5 km in West side of the epicenter. In the case studied RC buildings, the analysis results and also observed damages during the earthquake indicate moderate damages. More detailed evaluations indicate that the RC buildings satisfy the current Iranian building code requirements. However, in the case of steel buildings, severe damage and collapse has been observed. This kind of buildings not satisfies the current Iranian building code requirements. Field investigations show that the main reasons for damages of steel buildings appeared due to the very poor quality of welded construction in seismic resisting elements. Comparison of the analyses and field observations were intended to help identify the causes of the damage, as well as the ability of analytical methods and modeling assumptions to predict damages.

KEYWORDS: Bam earthquake, RC buildings, Steel buildings, Nonlinear analysis, Field observations

1. INTRODUCTION

The Bam strong earthquake occurred at 01:56:52 UTC on Friday, December 26, 2003, southwest of the city of Bam in the Kerman province in the southeast of Iran [1, 2]. In terms of life loss and casualties, the Bam earthquake was the worst in 2003. The earthquake struck near the ancient city of Bam in the early morning hours (at 5:26 AM local time) when most of the inhabitants were in sleep, which is one of the causes of the great life loss. Tens of thousands of individuals were crushed by collapsing walls and roofs of poorly-constructed dwellings of unreinforced mud bricks. Over 85 percent of the buildings and other infrastructure were damaged or destroyed in the Bam area.

The form of construction in Bam was predominantly (90%) adobe and masonry, with 8% steel and 2% concrete construction. Site surveys indicate that 62% were beyond repair, 35% could be retrofitted and 3 % were safe. Total collapse of traditional mud or mud-brick construction is evidently due to lack of ductility and poor quality of the materials. However, widespread damages and total failure to welded connections in new steel buildings is one of the major lessons taught by the Bam earthquake. The brittle nature of the fractures detected in numerous welded steel connections, essentially invalidated design approaches and code provisions based on "ductile" structural response [3]. On the other hand, limited damage observed in reinforced concrete frames in comparison with total collapse of steel braced frames is an important observation. This observation is particularly appropriate in earthquake-resistant design where the intensity and orientation of loading are highly uncertain. Buildings having simple, regular, and compact layouts incorporating a continuous and redundant lateral force resisting system tend to perform well and thus are desirable[4].

In this paper, two sample reinforced concrete buildings and two sample steel buildings damaged during Bam earthquake have been studied using nonlinear static (Push over) and dynamic analyses. As built details have been used in analytical modeling and a comparison between observed, analyses results, and code requirements

was performed.

2. STRONG GROUND MOTION OF BAM EARTHQUAKE

The strong ground motion of Bam earthquake has been recorded by digital SSA-2 accelerograph in Bam station. The geographic coordination of this record was (58.35E, 29.09N) and the direction of longitudinal and transverse components with respect to North direction was 278 and 8 degrees, respectively [5]. The peak ground acceleration, velocity, and displacement of the Bam earthquake (PGA, PGV, and PGD, respectively) in different directions are summarized in Table 1. The duration of strong ground motion based on $PGA > 0.05g$ was about 12 seconds. These show that the greatest PGA of 0.98g occurred in Vertical direction and the maximum horizontal PGA of 0.76g belongs to the Longitudinal (East-West) direction. The PGA of vertical component is about 30% greater than the PGA of longitudinal component.

Table 1 Strong Ground Motion parameters of Bam earthquake

Direction	PGA (g)	PGV (cm/sec)	PGD (cm)	Dominant period (sec)
Longitudinal	0.76	118.1	30.4	0.20
Transverse	0.60	53.3	18.8	0.20
Vertical	0.98	40.5	7.4	0.10

The spectral accelerations of Bam earthquake and also Bam design spectra for 5% damping ratio are shown in Figure 1. The Bam design spectra is determined based on Iranian code of practice for seismic resistant design of buildings by considering design base acceleration of 0.3g (high seismicity region) and soil type II with $T_0 = 0.50$ sec. the parameter T_0 is the period at which the constant acceleration and constant velocity regions of the design spectrum intersect (corner period).

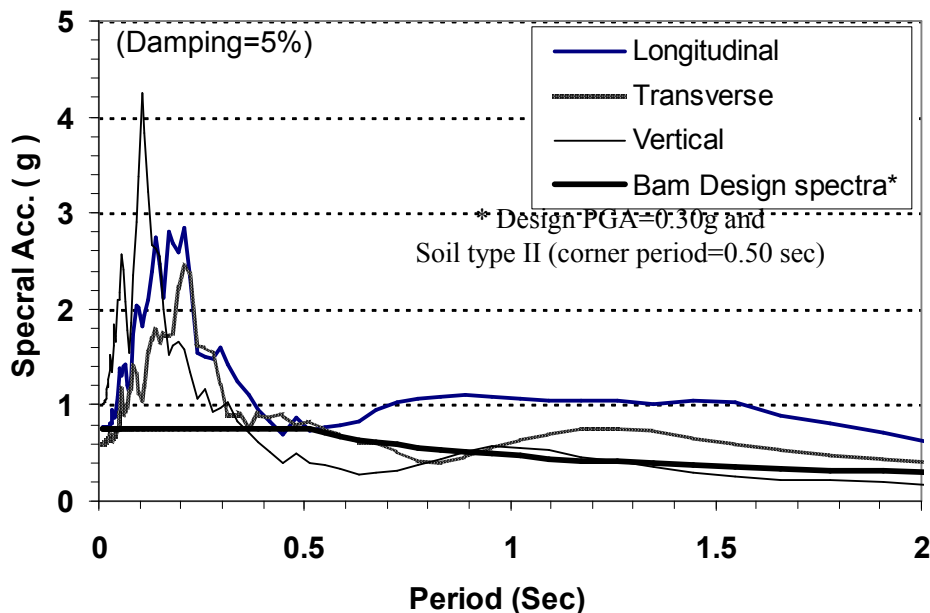


Figure 1 Spectral accelerations of Bam earthquake and Bam design spectra

It is clear from spectral accelerations that the dominant period of the ground motion in horizontal and vertical directions are about 0.2 sec and 0.1 sec, respectively. The maximum spectral accelerations at these periods are 2.85g and 4.25g respectively. Also, the acceleration amplifications at these periods are about 4.0 for both horizontal and vertical components. This value is 1.6 times greater than the Iranian seismic code design requirements [6].

Comparison between response spectra of Bam earthquake and Bam design spectra shows that the recorded event is greater than the design values (see Figure 1). For example, the spectral acceleration of horizontal component in dominant period (0.2 sec.) is about four times greater than the design value. This means that the lateral seismic loads in elastic range based on Iranian seismic code is lower than those implied by Bam earthquake, especially in low rise buildings.

3. SITE INVESTIGATIONS

The location of two sample reinforced concrete buildings, RC1 and RC2 and two selected steel buildings, ST1 and ST2 in the region is shown in Figure 2. These buildings are located in the city of Bam with about 5km distance from the earthquake epicenter. The earthquake PGA in East-Vest direction was about 27% greater than in North-South direction. Therefore, building damages was dominant in East-Vest direction. General damage patterns and failure modes of the example buildings presented in the following sections.

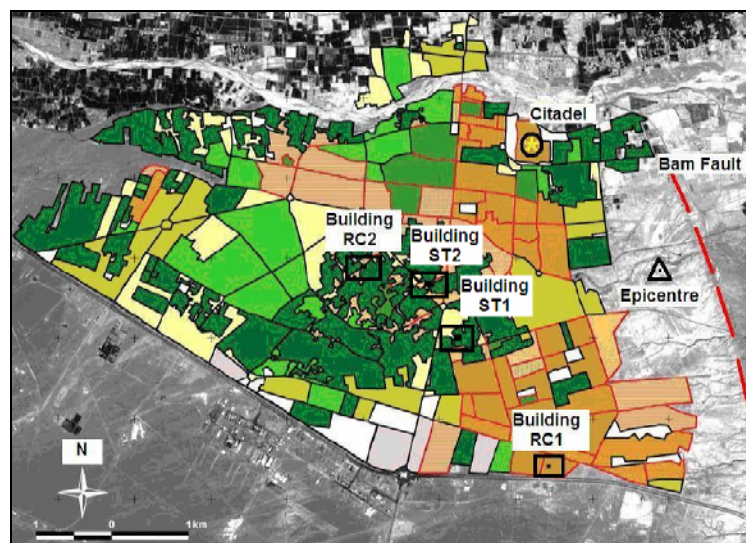


Figure 2 Location of example buildings ST1, ST2, RC1 and RC2 in the Bam city

3.1. Building ST1

The North and South view of this 5 story residential steel braced building (Kimia building) is shown in Figure 3. The lateral load resisting system of this building was diagonal bracing in East-West direction and a simple frame was considered in North-south direction. As shown in these Figures, the bracings in second and third floors fractured and a lateral movement of about 400 cm, occurred in these floors. The remaining upper floors (4 and 5) collapsed by aftershocks. Failure of the slender rod braces ($\varnothing 18$) with very weak connections led to a dramatic reduction in lateral strength and stiffness of the building in floors 2 and 3.



Figure 3 North view of building ST1 and failure of bracing connections

3.2. Building ST2

The North view of this 4 story braced building (Residential Building) is shown in Figure 4. Cross bracings in the second and third floors of this building failed during the earthquake. Lateral deformations of about 40 cm occurred in the second and third floors. Fracture of tension brace member connections and buckling of compression members were the main modes of failure of this building. Fracture of connection plate between column and gusset plate was another type of failure.



Figure 4 North view of building ST2 and failure of bracing connections

3.3. Building RC1

General view of this four-story reinforced concrete (RC) building is shown in Figure 5. This building was under construction, and about 90% of total gravity loads applied during the earthquake. Typical column sections is 35×35 cm with longitudinal bars of 8 ϕ 18, and typical beam sections is 30×30 cm with 6 ϕ 18 reinforcement. A shear reinforcement of ϕ 6@30cm has been used in all beam and column sections. Insufficient development length of the stair case bars in connection to the frame caused the collapse of the staircases in top stories. Diagonal shear cracking of beams is another damage pattern. However, limited and repairable seismic damage has been observed in this building during the Bam earthquake.



Figure 5 General view and diagonal cracks in beam elements of building RC1

3.4. Building RC2

General view of this three-story reinforced concrete (RC) building is shown in Figure 6. This building was under construction during the earthquake. Beam, column and reinforcement details of this building are similar to RC1. Cracking in beam-column connection zone was the only damage pattern observed in this building.



Figure 6 General view and cracking in beam-column connection zone of building RC2

4. SEISMIC DEMAND AND CAPACITY OF EXAMPLE BUILDINGS

Based on the Iranian code provisions, the equivalent static method can be used for the analyses of the selected buildings [6]. In this code, minimum base shear force of building structures in each direction can be determined from the following equations:

$$V = CW \quad (1)$$

$$C = \frac{ABI}{R} \quad (2)$$

$$B = 2.5(T_0/T)^{2/3} \leq 2.5 \quad (3)$$

$$T = 0.05H^{3/4} \quad (\text{For steel braced systems}) \quad (4)$$

$$T = 0.07H^{3/4} \quad (\text{For R.C. framed systems}) \quad (5)$$

Where in the above equations:

V = seismic base shear

W= total weight of the building (dead load + 20% live load)

C= base shear coefficient

A= design base acceleration (=0.30g for Bam region)

I = importance factor of building (=1 for ordinary buildings)

R =reduction factor (R=6.0 for concentric steel braced buildings, and R=5 for low ductility R.C. frames)

B =amplification factor

T_o =Corner period of the acceleration response spectra (=0.5 sec for soil type II)

T = fundamental period of the building

H =height of the building from the base (in m).

Equations 1 to 4 are used to determine the code required base shear. Based on the capacity spectrum procedure, the inelastic demand spectra S_{ai} is constructed by dividing the linear-elastic acceleration demand, S_a , with the R coefficient to determine the inelastic acceleration demand using equation 6.

$$S_{ai} = S_a / R \quad (6)$$

This equation can be explained in term of base shear coefficient using equation 7.

$$C_e = S_{ai} / g \quad (7)$$

Comparison of base shear strength and demand base shear is important to evaluation of the strength adequacy. The yield strength of a building, C_y , in terms of base shear coefficient which can be considered as seismic capacity is determined from equation 8.

$$C_y = V_y / W \quad (8)$$

Parameters C , and C_e in the above equations are the code required and earthquake demand base shear coefficients, respectively. These parameters are determined for sample buildings by assuming $R=6$ for steel buildings and $R=5$ for R.C. buildings. It should be noted that the assumed value for R parameter is considered based on code requirements. As-built structural details include tension and compression strength of brace members, strength of spliced sections, quality and quantity of welded brace connection considered in strength evaluation. Torsional effects are ignored and only the responses of example buildings in East-West direction (critical direction) are determined. Also, the effect of infill walls (generally hollow clay units with poor quality mortar) or other nonstructural elements have been neglected in determination of lateral load capacities.

The main seismic parameters and the results of base shear coefficients (C , C_e , and C_y) determined for the example buildings are summarized in Table 2. The results of nonlinear pushover analyses are shown in Figure 7 for R.C. buildings for example.

Table 2 seismic demand and capacity of sample buildings in East-West direction

Build. name	No. of stories	H (m)	W (ton)	T (sec)	R	C	C_e	C_y
ST1	5	16	570	0.40	6.0	0.125	0.145	0.033
ST2	4	12	320	0.32	6.0	0.125	0.230	0.05
RC1	4	13	340	0.48	5.0	0.15	0.174	0.20
RC2	3	10.4	454	0.41	5.0	0.15	0.168	0.24

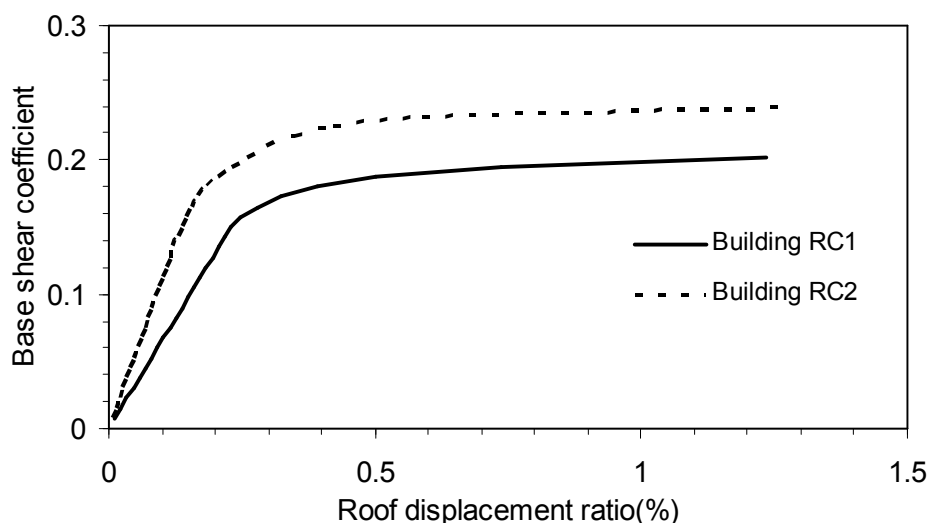


Figure 7 Push over curves of R.C. sample buildings

One important result has been obtained by comparing C and C_e , as determined for the example buildings. Generally, the earthquake demand is greater than the code required base shears in the case of ordinary studied buildings with low dominant periods ($C_e \geq C$). This result is similar for R.C. and steel buildings.

Another important result has been obtained by comparing C and C_y . In the case of steel buildings, the code required base shear coefficients (earthquake demands) is very greater than the yield capacity ($C = 2.5C_y \sim 3.8C_y$). But, in the case of R.C. buildings, the code required base shear coefficients is lower than the yield capacity ($C \approx 0.7C_y$). This results show that the base shear demands of sample RC buildings designed based on the Iranian seismic design code requirements is almost adequate. But, in the case of sample steel braced buildings, the base shear capacity is dramatically low. Generally, the lack of lateral load capacity in steel braced buildings is generated from very weak welding in gusset plate connections or bracing members themselves. Since the fracture of welded connections is brittle, there is no ductility in this kind of construction and the reduction factor $R=1$ should be considered for the example buildings. In fact, the brittle nature of the fractures detected in numerous welded steel connections, essentially invalidated design approaches and code provisions based on "ductile" structural response. Therefore, the failure of this kind of buildings was inevitable. Similar results have been reported in reference [7]

5. CONCLUDING REMARKS

Hundreds of damaged and collapsed steel buildings in Bam region have been identified, including hospitals and health care facilities, government, civic and private offices, cultural and educational facilities, residential structures, and commercial buildings. Damage occurred in new as well as old low rise (generally lower than 5 stories) buildings. While inadequate workmanship was believed to play the main role in the damage observed in the majority of the structures, very few damaged buildings are believed to have been constructed according to new codes and standards of practice. The effect of these observations has been a loss of confidence in the procedures used in the past to design and construct welded connections in steel braced frames and a concern that existing structures incorporating these connections may not be sufficiently safe. Analytical results show that the base shear demands of steel braced buildings are adequate. Based on the field investigations and detailed evaluations of example braced steel buildings the main failure modes of this kind of construction are: buckling and lack of compression strength of slender bracing members, weak spliced bracing members, very weak welded connections and brittle failure of bracing elements.

There were comparatively few RC framed buildings in the Bam region. Despite poor detailing at connections

and relatively poor quality of concrete, reinforced concrete framed buildings performed much better than the other types of buildings such as steel braced buildings during the strong Bam earthquake. No collapse observed in RC buildings and generally damages was in repairable limit. Analytical results show that the base shear demands of RC buildings are almost adequate. However, deformations, ductility demands and damage indexes determined from analytical models are greater than the observed values.

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