Lessons Learned from Steel Braced Buildings Damaged by the Bam Earthquake of 26 December 2003

N.A. Hosseinzadeh

Structural Engineering Research Center, International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, Iran, email: hosseinz@iiees.ac.ir

ABSTRACT: *Every earthquake provides new lessons for the earthquake engineering profession. The widespread damage to welded connections in steel structures was one of the major overall lessons of the Bam earthquake of December 26, 2003. The brittle nature of the fractures detected in weakly welded steel bracing connections, essentially invalidated the design approaches and code provisions based on "ductile" structural response. This paper reviews the performance of steel braced buildings during the Bam earthquake and the implications for design practice. The results can be used to develop and verify reliable and cost-effective methods for the inspection, evaluation, repair, and rehabilitation of similar existing steel buildings.*

Keywords: Bam earthquake; Steel buildings; Welded bracing connections; Brittle failure

1. Introduction

The Bam earthquake occured on Friday, December 26, 2003, southwest of the ancient city of Bam in Kerman province of southeast Iran [1, 2]. The earthquake struck in the early morning hours (5:26*am* local time) when most inhabitants were asleep, resulting in great loss of life. Tens of thousands of individuals were crushed by the collapsing walls and roofs of poorly-constructed dwellings made of non-reinforced mud bricks. Over 85% of the buildings and infrastructure in the area were damaged or destroyed.

An additional devastating blow was the destruction of Arg-e-Bam, a citadel on the historic Silk Road which is more than 2,000 years old. Arg-e Bam is the largest mud-brick complex in the world. This historical monument was destroyed in 12 seconds of strong ground motion duration of the Bam earthquake.

The total collapse of traditional mud or mud-brick construction is evidently the result of a lack of ductility and the poor quality of the materials. However, widespread damage to and the failure of welded connections in new steel buildings is of major concern. The brittle nature of the fractures detected in numerous welded steel connections, essentially invalidated design approaches and code provisions based on “ductile” structural response.

Several fundamental questions must be answered to develop effective and economical design procedures and construction standards, and to restore public and professional confidence in current forms of construction. These questions include:

- What caused the observed extensive damage to the steel buildings during the Bam earthquake?
- How can steel buildings that have sustained relatively minor damage be identified?
- How safe are these damaged steel buildings and do they need to be repaired? How can damaged buildings be reliably repaired and/or upgraded?
- How can be new buildings designed and constructed so that they will not sustain such damage?
- Can the vulnerability of existing steel buildings to future earthquakes be reliably determined and mitigated through effective rehabilitation procedures?
- What are the economic, social and political costs

of new design or construction practices?

Answering these questions requires consideration of factors including: metallurgy, welding, fracture mechanics, connection behavior, system performance, and practices related to design, fabrication, erection and inspection. Fortunately current theoretical knowledge and code requirements are adequate, but unfortunately, current technical and professional knowledge is inadequate.

2. Strong Ground Motion

The strong ground motion of Bam earthquake was recorded by digital *SSA*-2 accelerograph in Bam station. The geographic coordination of this record was (58.35*E*, 29.09*N*) and the direction of the longitudinal and transverse components to the North direction were 278 and 8 degrees, respectively [3].

The peak ground acceleration (*PGA*), velocity (*PGV*), and displacement (*PGD*) of the Bam earthquake in different directions are summarized in Table (1). The duration of strong ground motion based on *PGA*>0.05*g* was about 12 seconds. These show that the greatest *PGA* of 0.98*g* occurred in Vertical direction and the maximum horizontal *PGA* of 0.76*g* occured in the Longitudinal (East-West) direction. The *PGA* of the vertical component is about 30% greater than the *PGA* of the 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	060	53.3	18.8	0.20
Vertical	0.98	40.5	7.4	0.10

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

It is clear from spectral accelerations that the dominant period of ground motion in the horizontal and vertical directions are about 0.2*sec* and 0.1*sec*, respectively. The maximum spectral accelerations at these periods are 2.85*g* and 4.25*g* respectively. Also, the acceleration amplification in 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 (standard 2800) [4].

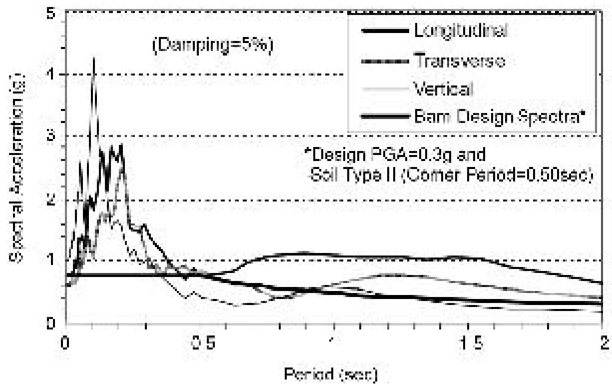

Figure 1. Spectral accelerations of Bam earthquake and Bam design spectra.

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

3. Intensity and Damage Distribution

The intensity level of the Bam earthquake is estimated to be I_o=*IX* (*EMS*98 scale), where the strong motions and damaging effects seems to have attenuated very fast especially in the fault-normal direction. The intensity levels were estimated to be *VIII* in Baravat, *VII* in Arg-e Jadid and the airport area. Also, the intensity level was estimated to be about *IV-V* in Kerman and Mahan [2].

A general view of the damaged areas of the Bam region based on the Arial photographs (1:10000 scale) is shown in Figure (2) [5]. The high level of destruction occured in old parts of the city with traditional adobe construction. However, damage distribution along the North-West to South-East direction is similar to the isoseismal map presented by Eshghi and Zare [2].

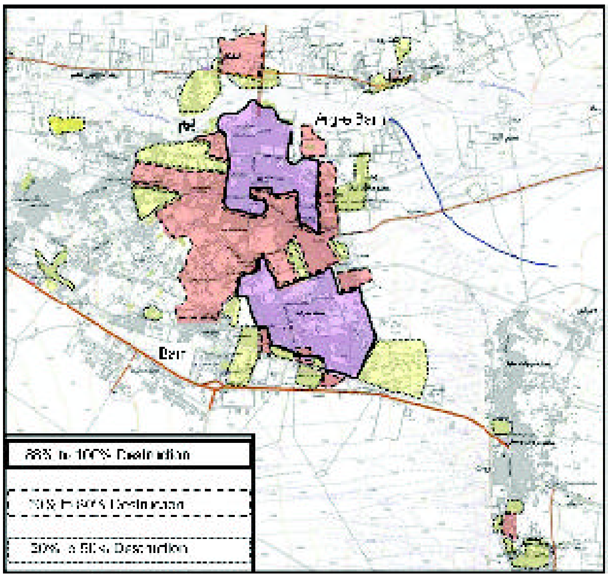

Figure 2. General view of damaged areas of Bam region [5].

Figure 3. Northern view of Kimia building.

Figure 4. Southern view of Kimia building

4. Damage to Structural Steel Construction

The use of structural steel in building construction in Iran is popular. The economy, and supposed earthquake resistance of braced steel framed buildings has led to their common usage. The same type of construction found in Bam has been used extensively throughout Iran. Thus, the effects of the Bam earthquake are an example of earthquake scenarios that have occurred in the past.

The 2003 Bam earthquake focused the attention of the earthquake engineering community in Iran on the probable seismic response of braced frames. Special attention was paid to the performance of new steel construction in the epicentral region to ascertain if the damage patterns observed had been repeated.

Five steel braced buildings have been selected by the author to investigate the effects of the Bam earthquake. These buildings are located near one an other in Bam, thus, the input earthquake motion of the selected buildings is similar. However, the earthquake *PGA* in East-Vest direction was about 27% greater than in the North-South direction, (see Table (1)), making damage in the East-Vest direction dominant. General damage patterns and failure modes of the sample buildings are described in the following sections.

4.1. Kimia Building

Northern and southern views of this five story residential steel braced building is shown in Figures (3) and (4). The lateral load resisting system of this building was diagonal bracing in the East-West direction and a simple frame in the North-South direction. As shown in these Figures, the bracings in the second and third stories fractured and a lateral movement of about 400 cm, occurred in these stories. The remaining fourth and fifth stories collapsed in aftershocks. Failure of the slender rod braces (Ø18) with very weak connections shown in Figures (5) and (6) led to a dramatic reduction in the lateral strength and stiffness of the building in second and third stories.

4.2. Insurance Bbuilding

Eastern and northern views of this four story braced building are shown in Figures (7) and (8). For architectural considerations, cross bracing of this building on the Eastern side (on the street) and also in

Figure 5. Rod bracings in stories.

Figure 7. Eastern view of the insurance building.

Figure 6. Rod bracing connections.

Figure 8. Northern view of the insurance building.

the second story was not considered. This resulted in horizontal and vertical irregularities. Weak cross bracings and very poor welded braced connections caused a sudden change in stiffness and strength of this story. Therefore, excessive lateral-torsional deformations of about 50*cm* occurred in the corner column of the second story. It should be mentioned that the height of this story was greater than for the other stories. This is a common problem occurance in buildings with shops in the ground floor. No damage occurred in the basement, surrounded by brick masonry infill walls.

A typical cross bracing member of this building (*L*80x80x8) spliced with a rod (Ø16) is shown in Figure (9). However, the failure occurred in bracing connection, not in the spliced section. In other words this kind of weak splice was stronger than the bracing connections. The masonry infill brick walls of the first story prevented the total collapse of this building. Because of excessive deformation and damage that occurred in the second story, the owner decided to demolish and reconstruct this building (Figure (10)).

4.3. Residential Building

The Northern view of this four story braced building is shown in Figure (11). Cross bracings in the second and third stories of this building failed during the earthquake. Lateral deformations of about 40*cm* occurred in the second and third stories. Fracture of tension brace member connections and buckling of compression members were the main modes of failure as shown in Figures (12) and (13). Fracture of connection plates between columns and gusset plates was another type of failure (Figure (14)).

4.4. Nursery Building

The Southern view of this three-story governmental building (a housing organization) is shown in Figure (15). This building was under construction during

Figure 9. Brace spliced with a rod.

Figure 10. Demolition of the insurance building.

the earthquake. A combination of cross and diagonal bracings was chosen as the lateral load resisting system.

Failure of cross bracing connections in the first story is shown in Figure (16). Excessive damage of bracing members is caused by the impact of compression bracing after connection failure. A compression buckling of diagonal bracings of the first story is shown in Figure (17). Built up cross sections similar to columns were used as diagonal bracings. Another type of failure in the cross bracing connections to the frame system is shown in Figure (18). This kind of connection was considered in some cross bracings that lacked sufficient length.

Figure 11. North view of the residential building.

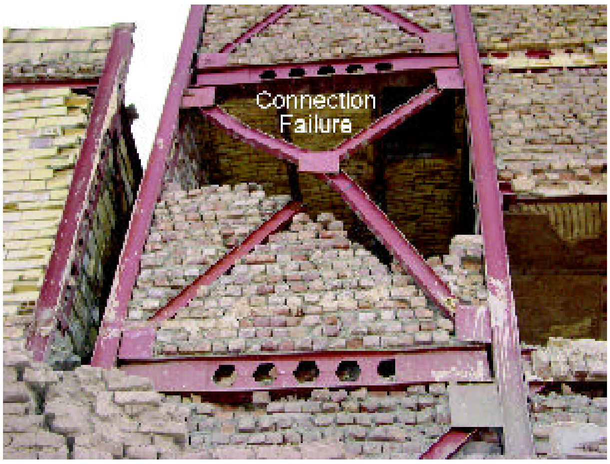

Figure 12. Fracture of bracing connections.

Figure 13. Fracture of bracing connections.

4.5. Bank Tejarat

The Southern view of this two-story braced building is shown in Figure (19). Very poor braced connections and braced member splices caused heavy damage. An example of bracing member fracture in a poor spliced section is shown in Figure (20). The small dimensions of the gusset plate, and poor quality and insufficient welding are clear in Figures (21) and (22).

Figure 14. Fracture of bracing-to-frame connections.

Figure 15. Southern view of nursery building.

Figure 16. Failure of cross bracing connections.

Figure 17. Buckling of diagonal bracings.

Figure 18. Failure of bracing connections to frame.

Figure 19. Southern view of bank Tejarat.

Figure 20. Fracture of spliced bracing members.

Figure 22. Fracture of bracing connections.

Figure 21. Very poor bracing connections.

As shown, extremely bad detailing has been used in the bracing connections of this building. It should be mentioned that all bank buildings were heavily damaged during the earthquake.

5. Seismic Demand and Capacity of Sample Buildings

The example buildings were investigated in accordance with the Iranian code provisions (standard 2800) [4]. Based on these provisions, the equivalent static method can be used for the analyses of the selected buildings. In this method, minimum base shear of a building in each direction can be determined from the following equations:

$$V = CW \tag{1}$$

$$C = {ABI}/{R} \tag{2}$$

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

$$T = 0.05H^{3/4} \tag{4}$$

In the above equations:
V = 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 (= 6.0 for concentric steel braced buildings)
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 meters).

In accordance with Eq. (2), code base shear coefficient, C, or design lateral strength, $V = F_{des}$, is determined by dividing the design lateral force required to keep the structure linear-elastic during an earthquake, F_{elas}, by a response modification coefficient, $R = R_{des}$, (Figure (23)). This force reduction is allowed provided that the resulting maximum nonlinear displacement demand, D_{nlin}, can be accommodated. The maximum displacement, Δ_{nlin}, depends on the R coefficient used in design. The R coefficient in current code provisions is based on a point of first "significant yield" in the lateral load resisting system. The term "significant yield" is defined as that level causing complete plastification of at least the most critical region of the structure (e.g., formation of a first plastic hinge in the structure). This procedure is a "force-based" design procedure [6].

Based on the Capacity spectrum procedure, inelastic demand spectra S_{ai} are constructed by

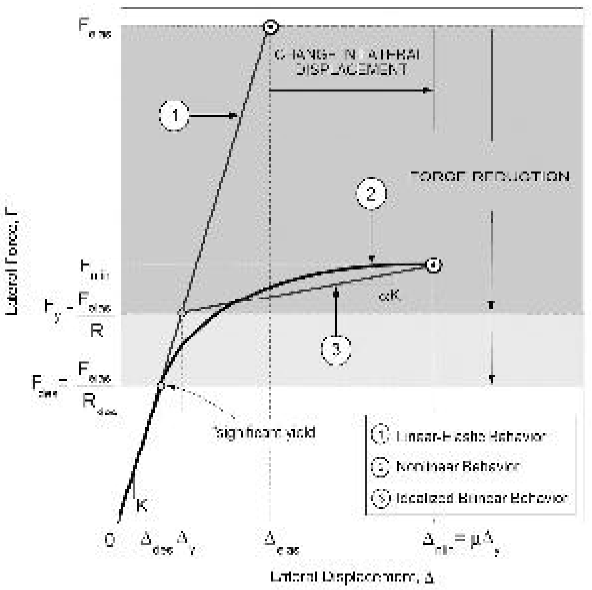

Figure 23. Lateral force-displacement relationships [6].

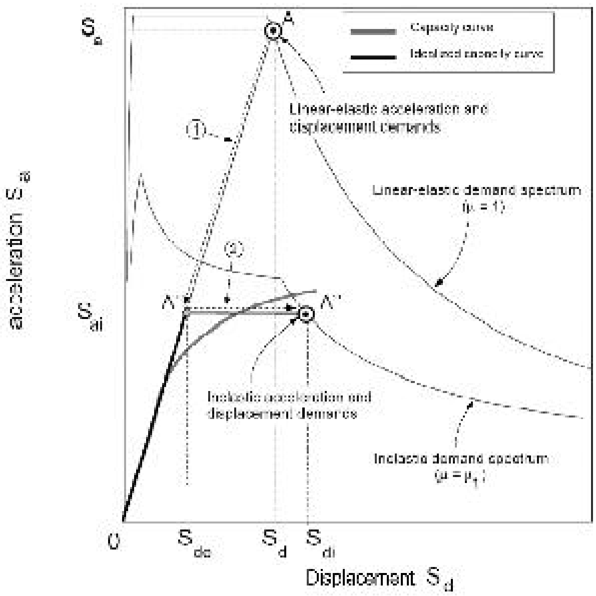

Figure 24. Inelastic demand spectra [6].

dividing the linear-elastic acceleration demand, S_a, with the R coefficient to determine the inelastic acceleration demand as:

$$S_{ai} = S_a / R \tag{5}$$

This equation can be explained in term of base shear coefficient as:

$$C_e = S_a / g \tag{6}$$

Then, the inelastic displacement demand, S_{di}, (Figure (24)) is determined by multiplying the linear-elastic displacement demand, S_{de} corresponding to S_{ai}, with the displacement ductility demand, μ (= Δ_{nlin}/Δ_y) as:

$$S_{di} = \mu S_{de} = \mu (T/2\pi)^2 S_{ai} \tag{7}$$

Either R or μ can be specified to construct the inelastic demand spectra for use within the framework of a "force-based" or "displacement-based" design procedure. In the displacement based design procedure, a target displacement ductility, $\mu = \mu_t$, is specified and the corresponding R coefficient is calculated using an assumed R-μ-T relationship. The inelastic demand spectrum is constructed using this procedure. Inversely, in the force-based procedure, an R coefficient is specified and the corresponding displacement ductility demand, μ, is calculated [6]. A "force-based" design procedure was investigated in this article.

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 $R = 6$. It should be noted that the assumed value for R parameter is considered for comparison between code and real earthquake demands and is not the real value of the sample buildings.

As noted in the previous sections, the brittle fractures detected in welded steel connections, essentially invalidated design approaches and code provisions based on "ductile" structural response. Therefore, in order to evaluate structural adequacy of sample buildings, base shear strength is compared to the demand base shears. The yield strength of a building, C_y, in terms of base shear coefficient (which can be considered as seismic capacity) is determined as:

$$C_y = V_y / W \tag{8}$$

In the above equations, V_y is the yield base shear force of a building. This parameter was evaluated based on the bracing cross section capacities or welded connection capacities in the example buildings using *AISC* code requirements [7]. All 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 the 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 the determination of lateral 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). In these calculations, an equivalent one mass system is considered by assuming the first mode shape and ignoring the other modes. Also, the weakest story is considered for calculation of C_y as story yield strength.

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

Name of building	No. of Stories	W (ton)	T (sec)	R	C	C_e	C_y
Kimia Building	5	570	0.40	6.0	0.125	0.145	0.033
Insurance Building	4	380	0.34	6.0	0.125	0.210	0.011
Residential Building	4	320	0.32	6.0	0.125	0.230	0.05
Nursery Building	3	1320	0.27	6.0	0.125	0.250	0.090
Bank Tejarat	2	220	0.20	6.0	0.125	0.460	0.032

One important conclusion can be obtained by comparing C_e and C, as determined for the sample buildings. In the case of the five-story Kimia Building with a fundamental period of $T = 0.4 sec$, the earthquake demand and code requirement base shears are similar ($C_e \cong C$). But for the two-story Bank Tejarat building with $T = 0.20 sec$, C_e is greater than C ($C_e \cong 3.7C$). Therefore, the seismicdemands of the Bam earthquake were greater than the Iranian code requirements in the case of low rise buildings with low dominant periods.

Another important conclusion can be obtained by comparing C (or C_e) and C_y. It is clear that in all sample buildings, the code required base shear coefficients (earthquake demands) are about 1.4 to 11.4 times greater than the yield capacity. Generally, this lack of lateral load capacity of steel braced buildings is generated from very weak welding in the 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 a ductility reduction factor of $R_\mu = 1$ should be considered for the example buildings. In fact, the brittle nature of the fractures invalidated design approaches and code provisions based on ductile structural response. Therefore, the failure of this kind of buildings was inevitable. A similar conclusion has been reported by Mahin [8]. However, determination of force reduction factor, R, by considering of structural overstrength factor, and allowable stress design factor in a global system (entire structure) is a difficult problem and needs more investigation.

The effect of infill walls (generally hollow clay units with poor quality mortar) or other nonstructural elements have been ignored in the determination of lateral capacities of the sample buildings. However, in some braced or unbraced steel buildings in Bam, the solid brick masonry infill walls with good quality cement mortar performed well and survived the earthquake forces. An example of this kind of good performance is shown in Figure (25). This figure shows a Southern view of a three-story building (with a basement) without any bracing in the first and second stories. The brick masonry infill is the only lateral load resisting system of this building in East-West direction. It experienced some cracking in the brick walls but the damage is much less than the adjacent braced buildings. However, where hollow clay units or poor quality mortar were used, heavy damage or total failure occurred in masonry infill walls.

It should be noted that for out-of-plain buckling in bracing members, failure of infill walls is inevitable. An example of this type of buckling in built-up braced members and failure of the infill walls is shown in Figure (26). As can be seen, no stitches have been used in the bracing members and out of plane buckling occurred in individual elements.

6. Evaluation of Seismic Code Provisions in Sample Buildings

Based on a detailed investigation of the sample buildings, the main drawbacks of steel braced frames with respect to Iranian seismic code requirements [4] and *AISC* requirements [7] can be summarized as follows:

6.1. Bracing Members

1. Slenderness: Based on Iranian code requirements, bracing members shall have slenderness ratio $Kl/r \leq 6025/\sqrt{F_y}$. Very slender rods and small size angles used as bracing membersin sample buildings do not meet this requirement.
2. Lateral force distribution: In order to prevent the use of non-redundant structural systems it is required that braces in a given line be deployed such that at least 30% of the total lateral force is resisted by tension braces and at least 30% of the total lateral force is resisted by compression

Figure 25. Masonry infill building.

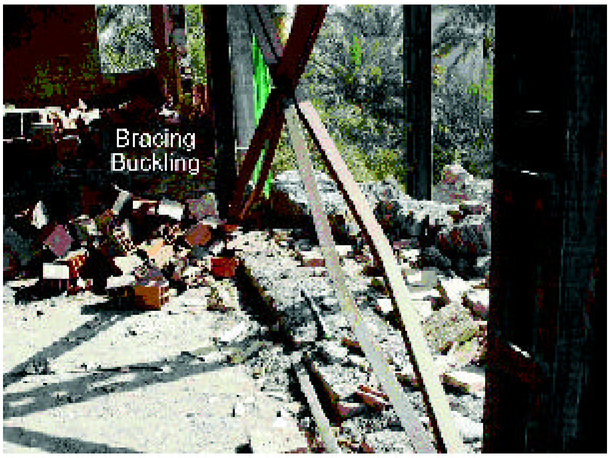

Figure 26. Out of plane buckling of built up bracing.

braces. This requirement had not been observed in the sample buildings. In fact, light rods and very slender braces experienced elastic buckling under very low compressive forces.

3. Built-up members: Based on the Iranian code requirements, the spacing of stitches shall be such that the slenderness ratio (*l/r*) of individual elements between the stitches does not exceed 0.7 times the governing slenderness ratio of the built-up members. Generally, this requirement was not observed in built-up bracing members. An example of this kind of bulking is shown in Figure (26).

6.2. Bracing Connections

1. Required strength: Based on the Iranian code requirements, the brace connections to beam and column (including beam-to-column connections if part of the bracing system) must be stronger than the braces themselves. Investigation showed that the strength of welded brace connections in sample buildings was about 10% of the bracing members. The extremely low strength of the connections caused brittle failure of the bracing connections. This lack of strength happened both in brace-to gusset plate and gusset plate-to-frame connections.
2. Eccentricity: The axes of bracing members should be aligned with beam and column axes. Highly eccentric brace connections used in residential building tend to fail prematurely due to the large secondary stresses induced by the eccentricities (Figures (11) to (14). These secondary stresses are generated both in bracing connections and in beam and column sections.
3. Tension Strength: Based on *AISC* requirements, the design tensile strength of bracing members and their connections, based on the limit states of tensile rupture on the net section and block shear rupture strength shall be based on expected yield strength of the brace $F_{ye} = R_y F_y$ which typically exceeds its specified minimum yield strength, F_y. In Iranian code it is assumed that $F_{ye} = F_y$. However, this requirement was not met in sample buildings.
4. Flexural strength: *AISC* requirements stipulate that the design flexural strength of the connections shall be equal to or greater than the expected nominal flexural strength 1.1 $R_y M_p$ of the brace about the critical buckling axes. This requirement is not included in the Iranian code. An sample of connection failure due to bracing buckling has been observed in sample buildings as shown in Figure (17).
5. Gusset plate: Based on *AISC* requirements, the design of gusset plates shall include consideration of buckling. Otherwise, the connection elements will themselves yield in flexure (such as gussets out of their plane). This requirement is also not included in the Iranian code. In the sample buildings, compression strength of bracing members and strength of welded connections were very low. Therefore, no buckling or failure occured in gusset plates. Astane-Asl, et al [9] suggested providing a clear distance of twice the plate thickness between the end of the brace and the assumed line of restraint for the gusset plate to permit plastic rotations and to preclude plate buckling.

7. Concluding Remarks

Hundreds of damaged and collapsed steel buildings in Bam 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 major role in the damage observed, 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. Based on the field investigations and detailed evaluations of sample steel braced buildings presented in this paper the following conclusions can be drawn:

- ❖ Current professional judgment is that the old or traditional practices used for the design and construction of bracing connections do not provide adequate reliability and safety and should be revised in the construction of new buildings intended to resist earthquake ground shaking through inelastic behavior.
- ❖ Common failure modes of bracing members include buckling of slender members, lack of compression strength, and weak spliced sections. Also, common failure modes of bracing connections include lack of strength in welded connections, brittle failure of nonductile connections, failure of brace-to-beam-column connections, failure of gusset plate connection due to out of plane buckling.
- ❖ Lack of strength in slender and weak bracing members and lack of strength in bracing welded connections limited the lateral strength of braced buildings to 0.10 to 0.5 times that of the code requirements and earthquake demands. Since the fracture of welded connections is brittle, there is no ductility in this kind of construction and the a ductility reduction factor $R_\mu = 1$ should be considered for these buildings.
- ❖ Sudden failure of brittle and weak welded connections of gusset plates introduced additional impact to earthquake forces in steel braced buildings. In accordance with second order effects (P-Δ effects), this caused large drifts in soft and weak stories. Damage was so severe in some buildings that all of bracing connections on one or more stories failed or significant permanent lateral displacement (between 40-400*cm*) occurred. From these observations in Bam, it can be concluded that no bracing is better than poor bracing.
- ❖ Damage to solid brick masonry infill walled buildings with good quality cement mortar was much less than to similar braced buildings. For out of plane buckling in brace members, failure occurred in infill walls.

References

1. Eshghi, S., et al (2003). "A Preliminary Reconnaissance Report on 26 December 2003 Bam Earthquake", IIEES Research Bulletin, **6**(3 and 4), Tehran, Iran.

2. Eshghi, S. and Zaré, M. (2003). "Bam (SE Iran) Earthquake of 26 December 2003, Mw6.5: A Preliminary Reconnaissance Report", IIEES, http://www.iiees.ac.ir.

3. BHRC (2003). "The History of Bam Accelerograph Station", http://www.bhrc.ac.ir.

4. BHRC (1999). "Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard 2800)",2nd ed.

5. Geological Survey of Iran (GSI) (2003), http://www.gsi.org.ir.

6. Farrow, K.T. and Kurama, Y.C. (2001). "Capacity-Demand Index Relationships for Performance-Based Seismic Design", Report # NDSE-01-02, University of Notre Dame, Notre Dame, Indiana.

7. AISC (2002). "Seismic Provisions for Structural Steel Buildings", ANSI/AISC 341-02.

8. Mahin, S.A. (1997). "Lessons from Steel Buildings in Northridge Earthquake", NISEE, University of California, Berkeley.

9. Astaneh-Asl, A., Goel, S.C., and Hanson, R.D. (1986). "Earthquake-Resistant Design of Double Angle Bracing", Engineering Jouranl, 23 (4), (4th Qtr.), AISC, Chicago, IL.