Performance of Steel Structures and Associated Lessons to be Learned from November 12, 2017, Sarpol-e Zahab - Ezgeleh Earthquake (M_W 7.3)

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ABSTRACT

Sarpol-e Zahab - Ezgeleh earthquake (M_W 7.3) occurred in Kermanshah province of Iran near the Iraq and Iran border region on November 12, 2017 at 18:18 UTC (21:48 local time). The epicenter was located about 5 km from Ezgeleh town with a focal depth of about 23 km. Sarpol-e Zahab - Ezgeleh earthquake is the most destructive seismic event in Iran in recent decade in terms of financial and human losses. Based on field observations, carried out by the authors between 25 and 30 November 2017, heavy non-structural and structural damages were occurred to all types of steel lateral load resisting systems, including concentrically and eccentrically braced frames and moment resisting frames. Early buckling of built-up brace members, excessive out-of-plane deformation in gusset plates, formation of plastic hinges at the column ends and lateral-torsional buckling of link beams were dominant failure modes in damaged steel buildings. Post-earthquake observations showed that damages in steel structures were mostly due to poor construction quality including lack of proper welding in connections, extent of irregularities of the structural system, false structural design, local site effects, and finally lack of enough supervision by "Iran Construction Engineering Organization" (IRCEO) and other responsible organizations. In this paper, observed damages to steel structures were examined and explaneed in detail.

Keywords:
Sarpol-e Zahab - Ezgeleh Earthquake; Steel structures; Failure types; Seismic code

1. Introduction

A few days after the main shock of the Sarpol-e Zahab - Ezgeleh earthquake, the first author visited the earthquake affected areas in Kermanshah province. All authors returned for a second investigation two weeks after the event for a period of about a week. This paper reports and comments on the observations made by reconnaissance team members of the International Institute of Earthquake Engineering and Seismology (IIEES), which visited the epicentral area of the earthquake. Contributions from local structural engineers and other members of IIEES reconnaissance team were also included in this paper for the sake of completeness. A total of five cities and adjacent villages were visited during field reconnaissance to study the damage patterns and their causes in the steel buildings, mainly in Sarpol-e Zahab city. The location of investigation sites are shown in Figure (1).

According to the formal reports by the Iranian legal medicine organization, the number of fatalities

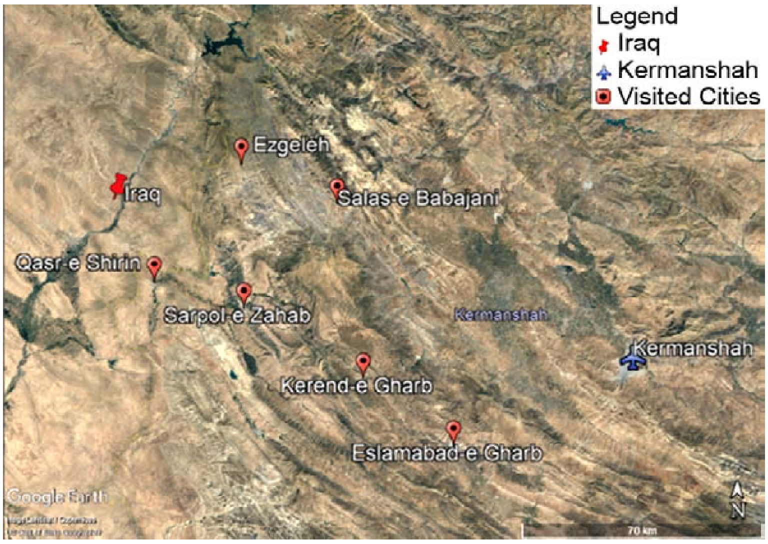


Figure 1. Locations of investigation sites that are referred in this report.

was over 620 in Iran, and injured was near 8100 due to Sarpol-e Zahab - Ezgeleh earthquake (Table 1). According to field observations, city of Sarpol-e Zahab suffered the most financial and human losses among earthquake affected cities. The structural damage density map for Sarpol-e Zahab city, provided by United Nations Institute for Training and Research (UNITAR) is shown in Figure (2). The structural damage density presented in Figure (2) is consistent with observed damage patterns by the authors in Sarpol-e Zahab city.

Table 1. Death toll after Sarpol-e Zahab - Ezgeleh earthquake.

Location	Deaths
Sarpol-e Zahab	560
Salas-e Babajani	23
Dalahu	19
Qasr-e Shirin	16
Kermanshah	1
Eslamabad-e Gharb	1
Iraq	10

Earthquake records and response spectra corresponding to the main shock event, recorded in city of Sarpol-e Zahab are plotted in Figure (3). As is shown in Figure (3a), the maximum PGA in case of N-S component was 0.68 g. Earthquake response spectra are compared with the recommended design spectra for various soil conditions as is mentioned in Iranian seismic code [1] (Figure 3b). The maximum recorded PGA for the Eslamabad-e

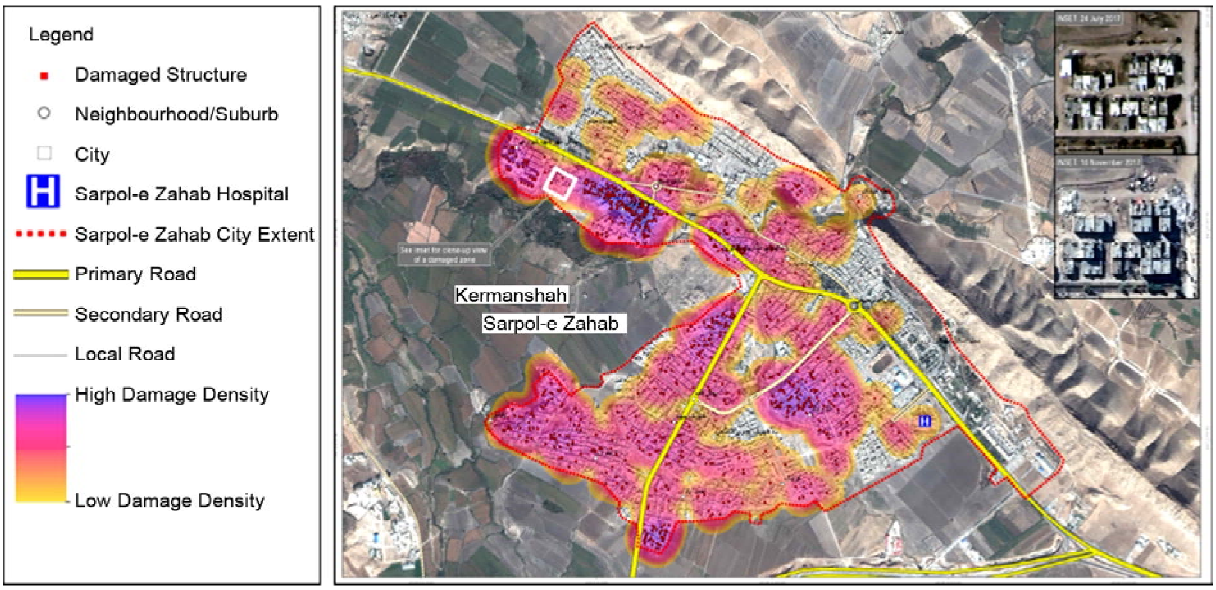


Figure 2. The structural damage density map for Sarpol-e Zahab city, provided by (UNITAR).

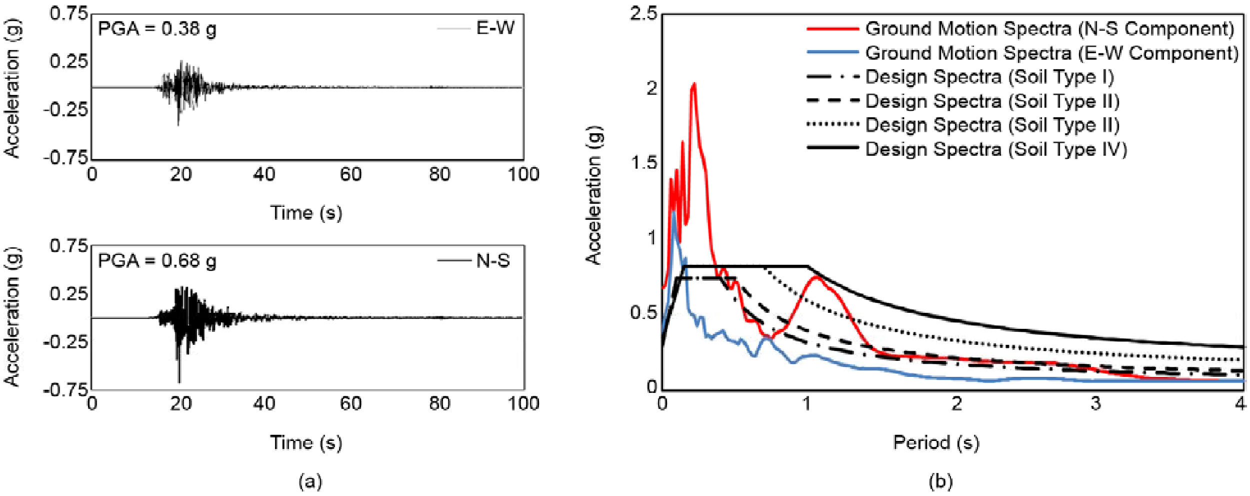


Figure 3. (a): Acceleration time histories recorded for the main shock event, (b): Elastic response spectra for the main shock recorded in Sarpol-e Zahab station and design spectra for various types of soils according to Iranian code of practice for seismic resistant design of buildings.

Gharb and Kerend-e Gharb are 0.123 g and 0.261 g respectively.

Although due to the high cost of steel as a construction material, owners of low-rise buildings tend to use concrete or masonary materials for construction purposes, during the site visit, considerable number of residential steel buildings with significant design, detailing and workmanship defects observed. Damaged steel structures are mainly concentrated in recently developed urban areas, mostly with loose and alluvial soil. Steel structures in earthquake affected areas often have two to five floors, with braced frame in one direction and moment resisting frame (MRF) in orthogonal direction. In many cases, the owner or the shareholder of the land is also the constructor of the building with no specific knowledge or experience on construction. The major causes of damages to steel structures were observed to be non-compliance with the current seismic design rules. Due to the fact that the majority of the observed structures are located in the urban areas, the lack of supervision of the organization of the engineering system is evident in the design and construction of damatged structures.

Steel ranks very high among structural materials suitable for earthquake resistance. It exhibits high strength and stiffness as well as good ductility and toughness with high strength-to-weight ratio. This makes the seismic performance of steel structures more predictable than that of other construction systems. However, building with steel is not sufficient by itself to warrant a proper performance during a strong earthquake induced ground shaking. Satisfactory performance can only be achieved if a sound structural arrangement is provided and if the structural elements and their connections are sized in such a manner that appropriate means of absorbing and dissipating energy exist and premature failures are avoided, especially within the gravity load resisting system. In spite of past earthquakes in which the seismic response of steel frames has been known to be tremendously reliable [2], due to Sarpol-e Zahab - Ezgeleh earthquake, considerable number of fatalities were attributed to unsatisfactory performance of steel structures. The performance of concentrically or eccentrically braced steel frames and moment resisting steel frames during the November 21, 2017, Sarpol-e Zahab - Ezgeleh earthquake, is examined herein. Evidences of significant inelastic response and several structural deficiencies were observed on steel-framed structures after the event.

2. Damages to Concentrically Braced Frames (CBFs)

For low and medium-rise structures, the concentrically braced frame (CBF) system is a common structural steel system in areas of any seismicity. It is simple to design and fabricate and provides required lateral strength and stiffness with a low material and fabrication cost. CBFs resisting lateral loads through a vertical concentric truss system.

The axes of the members aligning concentrically at the joints. Given the fact that axial force demand due to gravity loads are negligible in bracing members, these diagonal members are suitable candidates to act as fuse elements in concentrically braced frames to form the energy dissipating mechanism through yielding in tension and inelastic buckling in compression. Ductile and stable behavior of CBFs can be expected only if inelastic response is concentrated to properly detailed, bracing members and brittle failure modes are avoided in the other elements with force-controlled actions such as connections, columns and beams. According to Iranian seismic code, the response modification coefficient (Ra) and maximum permitted height for ordinary concentrically braced frames (OCBF) are considered 3.5 and 15 m respectively. Although OCBFs have minimal design requirements compared to other braced-frame systems, almost all of the damaged structures in earthquake affected areas with CBF, have not met the required provisions of OCBF system for which no attention was paid to ductile detailing or capacity design concepts. Although higher seismic loads are prescribed for OCBFs in comparison with SCBFs; however, some degree of inelastic response is still anticipated in ordinary braced frames and premature failure is probable if the weakest element does not exhibit enough ductility. Initial damage assessment of the structures indicated the CBFs had resisted the shaking with extensive inelastic response in brace elements as well as a significant number of brittle failure of the welded brace connections. The investigation demonstrated that the capacity of the welds was well below the actual strength of the bracing members and the forces that likely developed in these members during the shaking. In many cases, bracing members experienced significant inelastic out-of-plane buckling not only because of the axial seismic loads, but also because of premature failure and excessive out-of-plane rotations of gusset plates as shown in Figure (4).

A widespread failure mode, observed in braced frames, was the early buckling of built-up brace members with double channel section, due to the lack of connector plates or brace-to-connector welding as shown in Figure (5). According to AISC-360 [3], the longitudinal spacing of connectors, connecting components of built-up compression members must be such that the slenderness ratio of individual shapes does not exceed three-fourths of the slenderness ratio of the governing slenderness ratio of the built-up member. By ignoring the connector plates, the buckling response of built-up brace member, would be governed by single channel section characteristics with global buckling capacity, much lower than that of a double channel section.

As well as ignoring connector plates in brace elements, implementation of slender brace members and improper brace splices as shown in Figures (6) and (7), caused premature buckling and fracture of brace members.

(a) (b)

Figure 4. Premature buckling of brace members due to the excessive out-of-plane deformation in gusset plates.

(a) (b) (c)

Figure 5. Lack of connector plates or brace-to-connection plate welding in brace members with double channel section.

(a) (b)

Figure 6. Overall buckling of slender brace members.

(a) (b)

Figure 7. Inappropriate brace splices.

2.1. Damages of Connections in CBFs

According to AISC-341 [4], Bracing connections, including gusset plate and weldings in OCBFs are designed for forces corresponding to the over-strength seismic load with exceptions that allow for the force to be limited to the expected brace strength. The intent is to avoid brittle failure of connections prior to yielding or buckling of brace. As mentioned before, the braces as "fuse" elements are weakest element of frame in CBFs and all other elements (columns, beams, connections and diaphragms), are designed so that inelastic behavior is restricted to braces. Lack of proper seismic detailing as well as extensive fabrication deficiencies in bracing connections, are most common causes of failure of steel structures with CBF, observed in earthquake affected areas. Some of the most important deficiencies that caused overall or partial failure of the structures equipped with CBFs are shown in Figures (8) to (12). As shown in Figure (8), a large number of column web rupture, at gusset plate-to-column connection zone, observed due to direct welding of gusset plate to column web. Given that there is no evidence of the yielding or buckling in braces, it can be deduced that the connection failure have been occurred in early cycles of earthquake excitation in many cases. The premature brace connection failure has been deteriorated by the lack of welding of the gusset plate to upper beam as shown in Figure (9). Large story drifts that result from early brace connection ruptures can impose excessive ductility demands on the beams and columns, or their connections. Another common construction error, observed during the site visits, is the lack of welding or poor quality of brace-to-gusset plate welding as shown in Figures (10) and (11).

Finally, out-of-plane eccentricity in the connection, lack of consideration for a fold line in gusset plates and bending deformation caused by brace in-plane eccentricity can be considered as other widespread construction errors as shown in Figure (12).

(a) (b)

Figure 8. Premature failure of brace connection due to the direct welding of gusset plate to columns web leaded to column web rupture.

(a)

(b)

Figure 9. Ignoring gusset-to-beam welding.

(a) (b)

Figure 10. Ignoring brace-to-gusset plate welding.

Figure 11. Poor quality of welding in brace connections.

Figure 12. Inappropriate geometrical detailing of gusset plate. (a) Out-of-plane eccentricity in the brace connection. (b) Ignoring fold line in gusset plate, necessary to accommodate significant rotations corresponding to out-of-plane buckling of brace. (c) In-plane eccentricity of brace to beam-column joint.

3. Damages to Moment Resisting Frames (MRFs)

For low to medium-rise structures, moment frames are typically less economical than braced frames for resisting lateral loads. Columns, beams and moment connections in steel moment frames are proportioned to sustain actions that are result of MRF inelastic response during strong earthquake excitations. Although large inelastic responses are expected at targeted plastic hinge locations of beams (at the ends or at intentionally weakened parts of the beams), it is more desirable to keep inelastic responses, out of columns to avoid the formation of soft-story mechanisms. Moment frames generally exhibit higher redundancy and energy dissipating capabilities in comparison with concentrically braced frame.

According to seismic design philosophy for MRFs, the moment connections are anticipated to be able to transferring the moment and shear forces that can be developed at joints. As a result of material overstrength and strain hardening effects, these moment and shear forces can be considerably larger than the analysis forces, using code-specified seismic and gravitational loads. Different types of special and intermediate Moment connection are expected to be capable of developing at least 0.04 and 0.02 radians of inter story drift respectively without excessive strength loss, when subjected to the code-specified cyclic loading. According to testing difficulties, seismic codes permit the use of prequalified connections demonstrated by extensive testing and analysis to be capable of reliable service when used within specified application provisions.

As mentioned before, low-rise steel structures with braced frame in one direction and moment resisting frame system in orthogonal direction with built-up, double (or triple) I-shaped section as beams and columns are the most common type of steel structures in earthquake affected areas. The gravity loads generally carried by moment frame beams, thus the beam sections are generally stronger than columns in moment frames. An altered and inappropriate type of WFP moment connections with rectangular and trapezoidal shapes of flange plates are the most common moment connection type, implemented in the moment frame structures in the earthquake affected areas. WFP moment connections are permitted for Non-Special MRFs in Iranian seismic code [1] and are popular in current construction practice of Iran for ordinary and intermediate moment frames. Generally, different shapes of the top and bottom flange plates are implemented in WFP connection. The geometry of these plates is considered in a manner that site welding in a horizontal position is possible for connecting both flange plates to beam as shown in Figure (13).

Although the moment connection shown in Figure (13) meets the requirements of intermediate moment connections and can be considered as a rigid connection with satisfying ductility and dissipation capacity [5-6], the altered configuration WFP connections, widely used in damaged buildings, can hardly be considered as a moment connection. The most common construction defects, observed in moment connections are shown in Figure (14). Due to the implementation of moment connections, with insufficient stiffness, ductility and strength for the moment and shear actions corresponding to developing plastic hinge at the beams ends,

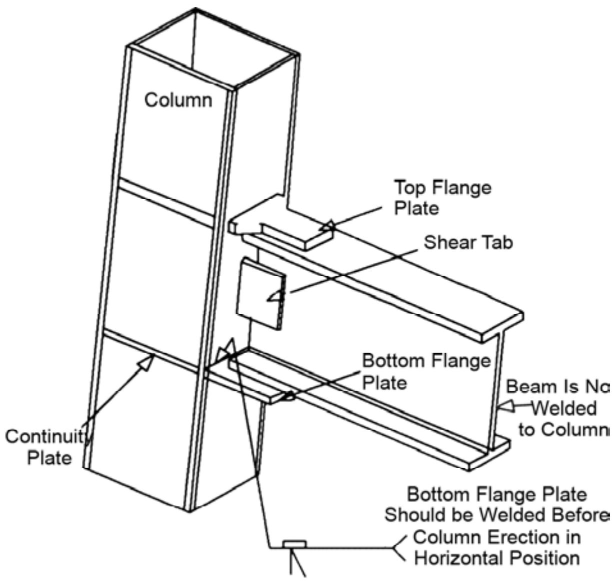


Figure 13. Configuration of WFP moment connection [5].

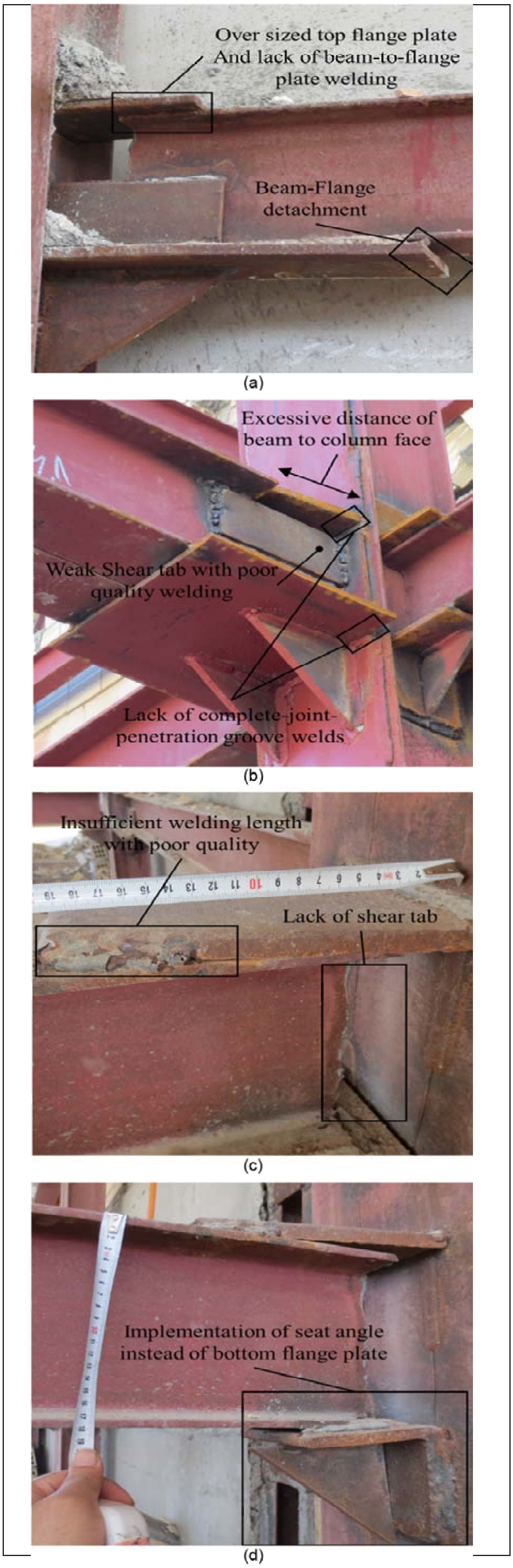


Figure 14. Configuration of WFP moment connection [5].

(a) (b)

Figure 15. Common Brittle failure modes in moment connections. (a) Weld failure at the flange plate-to-column. (b) Weld failure at the flange plate-to-beam.

(a) (b) (c) (d)

Figure 16. Formation of plastic hinges at the column ends.

formation of plastic hinges at the targeted location of beams, cannot be expected.

According to the observations, weld failure at the flange plate-to-column (Figure 15a) and flange plate-to-beam (Figure 15b) regions were the dominant failure mode in collapsed WFP connections.

As a result of incapability of moment connections to transfer moment and shear forces corresponding to the formation of plastic hinges at the beam ends and weak column / strong beams, all inelastic responses in moment frame were concentrated at the column ends. Formation of plastic hinges and local buckling at the column ends (Figure 16) leaded to the occurrence of widespread soft story mechanisms in damaged buildings.

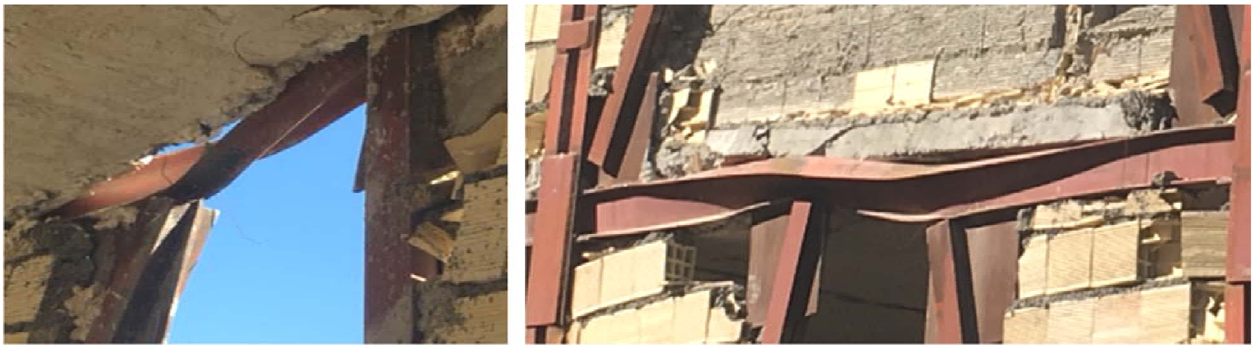
Although no damage was observed due to the failure of column base plates or yielding of anchor bolts, lack of capacity design for base plates is evident. Examples of common base plate construction practice in earthquake affected areas are shown in Figure (17). Asymmetric alignment of column in base plate especially in the side base plates, insufficient and improper bolt layout in base plate, lack base plate stiffeners and etc. are obvious according to Figure (17).

4. Damages to Eccentrically Braced Frame (EBFs)

Eccentrically braced frames (EBFs) can provide sufficient ductility and energy dissipation capacity in the inelastic range and satisfying amount of elastic stiffness particularly when short link lengths are implemented in the case of proper seismic datelining. Inelastic action in EBF under seismic excitations is limited primarily to the link members. Code based seismic provisions are intended to ensure that cyclic yielding in the links can occur in a stable manner while the other elements in EBF such

(a) (b)

Figure 17. Common base plate construction practice in earthquake affected areas.



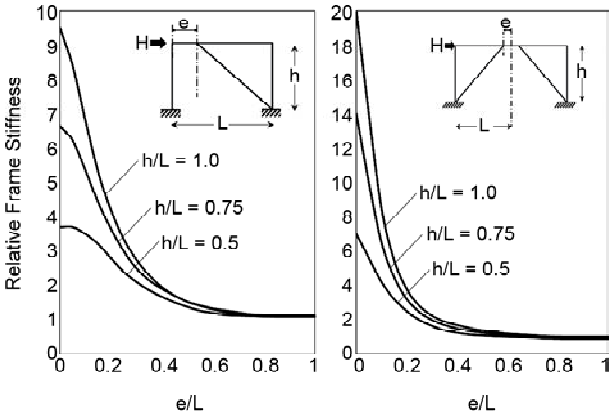
(a) (b)

Figure 18. Lateral-torsional buckling of link member due to the lack of lateral bracing of link member.

as diagonal braces, columns, and portions of the beam outside of the link remain essentially elastic under the forces that can be developed by fully yielded links. In contrast with CBFs, beam elements in EBFs are subjected to significant shear and bending actions and it is necessary to provide lateral bracing for link members to avoid lateral and lateral-torsional buckling especially at the ends of the link member to ensure stable cyclic behavior. Full-depth stiffeners at the ends of link member and intermediate web stiffeners are also required in link members to transfer the link shear forces to the reacting elements, restrain the link web against buckling and to delay the onset of inelastic shear buckling of the web. Although EBF with proper seismic detailing can provide a satisfying amount ductility, elastic stiffness and dissipation capacity, a significant number of damaged structures with inappropriate eccentrically braced frames, observed during the site visits. As shown in Figure (18), lack of lateral bracing and stiffeners in link members caused premature out-of-plane buckling of link member in early earthquake excitations for significant number of steel structures with EBF.

Hjelmstad and Popov [7] evaluated elastic lateral stiffness of the two configuration of EBF as a function of the eccentricity ratio (e/L) and frame height to span length ratio (h/L) as shown in Figure (19). The stiffness values for each value of the aspect ratio have been normalized by the stiffness value at $e = L$ that present the unbraced frame condition (moment frame). As shown in Figure (19), the elastic lateral stiffness of EBF significantly decrease by increasing the eccentricity ratio, and for the values of eccentricity ratio greater than 0.5, the EBF show little advantage over the unbraced frame.

Figure (20) shows the case study of a 2-story residential building in Sarpol-e Zahab city with eccentrically braced frame system in both direction. The semi-rigid beam-to-column connections like samples shown in Figure (15), in one direction and unstiffened Seat angle connection in orthogonal direction are implemented. As it is evident in Figure (20), due to the excessive eccentricity ratio implemented in the eccentrically braced bays ($e/L > 5$), the presence of EBFs showed little effect on the lateral stiffness of building. Removing the infills as well as the higher ceiling in the first story of building, intensified the lateral instability of building and formation of soft story mechanism. It is evident that, the corrugated steel panels, prevented the structural collapse through their in plane stiffness as shown in Figure (20c).



(a) EBF with Single Diagonal Brace (b) EBF with Split-K Brace

Figure 19. Variation of elastic stiffness with e/L for two frames employing different bracing arrangements [7].

5. Stiffness Irregularity (Soft Story)

The soft story phenomenon was widely observed among damaged buildings with different LFRS in earthquake affected areas as shown in Figure (21). In moment resisting frames, incapability of implemented moment connections to transfer moment and shear actions corresponding to developing of plastic hinges at the beam ends and strong beam/weak columns are the most important causes for


(a) (b) (c)

Figure 20. Case study: A 2-story eccentrically braced frame building with excessive eccentricity ratio.


(a) (b) (c) (d)

Figure 21. Soft story phenomenon.

the formation of soft story mechanism; while in CBFs, premature failure of brace connections and buckling of braces in early seismic excitations are the most common soft story causes. Finally, in EBFs, the implementation of large eccentricity ratio (Figure 20), as well as the lack of lateral bracing and stiffeners for link member, are the most important and widespread soft story causes. The Iranian seismic code, has not provided any mandatory regulations to avoid "soft story" for structures with three stories or less while, according to observations, low-rise steel buildings, are also vulnerable to soft story damages.

6. Damages to Non-Structural Elements (Partition Walls and Stair Cases)

A significant portion of the economic losses due to Sarpol-e Zahab - Ezgeleh earthquake and the subsequent aftershocks can be attributed to the losses from damage to non-structural components. According to the observations, in many buildings, the severity of damage to non-structural elements such as partition walls, staircases, windows and facades was more than that to the structural components. Although at this stage, it is not possible to provide a comprehensive figure on the exact number of buildings undergoing each type of non-structural failure, commonly observed damages to non-structural elements of steel structures are reported in this section. The observed damages to infill or partition walls are clearly the result of the low construction quality of the infill walls, infill walls inability in accommodating the drifts experienced by the structure and deficiencies in anchorage of the infill walls. In this study, the damages to non-structural components are organized in three general categories including in-plane failure of infill walls, out-of-plane failure of infill walls and failure of stair case as shown in Figures (22) to (24).

7. Conclusion and Lessons Learned That Should be Taken into Consideration in the Future

A few days after the main shock of the Sarpol-e Zahab - Ezgeleh earthquake, the first author visited the affected area. All authors returned to earthquake affected areas for a second investigation two

(a) (b) (c)

Figure 22. In-plane failure of infill walls.

(a) (b) (c)

Figure 23. Out-of-plane failure of infill walls.

(a) (b) (c)

Figure 24. Failure of staircase.

weeks after the event for a period of about a week. A total of five cities and adjacent towns were visited during field reconnaissance to study the damage patterns and their causes in the steel buildings, mainly in Sarpol-e Zahab city and other urban and rural regions.

In spite of previous divesting seismic events in Iran, happened in undeveloped cities or rural areas, in Sarpol-e Zahab - Ezgeleh earthquake, many so called "engineered" steel structures that are built in recent years and were supposed to be under effective supervisions in design and construction process, severely damaged, which could have been avoided by using code-specified structural provisions.

The main lessons to be learnt from this event are as follows:

- The need for more continuous supervision in the design and construction process by qualified engineers considering that poor quality construction done by uneducated workmanship was the most important cause of the widespread destruction in the earthquake affected region.
- Prohibition of implementation of detailing-sensitive lateral force resisting systems such as EBF for MRF in rural areas, which are not possible to be designed, fabricated or supervised by qualified engineers.
- Reconsideration of the design acceleration spectra provided in Iranian seismic code [1].
- Using proper infill walls anchorage systems to avoid in in-plane and out-of-plane infill collapse respectively, considering widespread human and economic losses caused by undesirable collapse of infill walls.
- Providing mandatory regulations to avoid "soft story" even in low rise, steel structures.

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