THE BEHAVIOR OF THE STEEL STRUCTURES TO EARTHQUAKES – EXPERIENCE FEEDBACK

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Abstract

Using the reports of the post-seismic missions realized by AFPS for thirty years (for 1985 – Mexico) and articles and other publications concerning earthquakes that have not been the subject of site visits, an assessment on the behavior of steel structures to earthquakes is proposed.

The analysis takes into account reports and other written documents and pictures taken on-site.

The experience feedback (of 28 major earthquakes recorded worldwide) shows a globally satisfactory behavior of this kind of structures in earthquakes. It is also an opportunity to review the structures with dissipative behavior that have been used now for about twenty years and for which the data of real performance is available.

The characteristic damage identified during post-seismic mission is presented by type of steel structure.

Keywords: reports of the post-seismic missions, experience feedback, the characteristic damage, behavior of the steel structures to earthquakes.

1. Introduction

The Reports of post-seismic missions carried out by the AFPS for 30 years (from the earthquake in Mexico - September 1985) are a very interesting source of information about the behavior of buildings in 28 major earthquakes recorded all around the world.

We tried to make a first assessment, even though it is not exhaustive, of the behavior of steel constructions in response to these major events in regards to their resistance and integrity.

Besides the reports of AFPS, we also consulted other articles and publications, in particular for the earthquakes that have not been followed by on-site visits.

The characteristic damage identified during the post-seismic missions, is presented by type of steel structure.

2. Typology of steel structures

2.1 Moment resisting frames

All components of this type of structure (columns and beams) are involved in the recovery of vertical and horizontal efforts. Earthquake resistance is provided primarily by the bending of columns and beams. An example of three-dimensional frame is given in Fig. 1.

Fig. 1 - AFPS Kashiwasaki Mision, Japon 2007 (Photos Th. Lamadon)

2.2 Concentrically braced frames

These are structures in which resistance to horizontal forces is provided primarily by elements subjected to axial forces. Some examples of such structures are shown in Fig. 2 to 4.

Fig. 2 Parking Garage - Christchurch Earthquake 2011 – Canterbury University Report, New Zealand

Fig. 3 Firefighters Command Center – Los Angeles AFPS Mision Report – Northridge, USA 1994

Fig. 4 Parking Sendai – Tohoku Earthquake 2011, Japon –Hokkaido University Report

2.3 Eccentrically braced frames

These are structures in which resistance to horizontal forces is provided primarily by elements subjected to axial forces, but in which the eccentricity of the configuration is such that the energy can be dissipated in seismic sections or by cyclic bending, or by cyclic shear. An example of this type of structure is given in Fig. 5.

Fig. 5 Shopping Mall on Dilworth St. and Clarence St. - Christchurch Earthquake 2011 – Canterbury University Report, New Zealand (Photo by G. MacRae)

3. Behavior of steel structures

3.1 Moment resisting frames

The moment resisting frames (multi-storey and industrial) that were designed well (according to modern seismic codes) and built well generally performed very well, even during strong earthquakes. These structures rarely suffered either inelastic deformation or broken anchors fixings in the concrete foundations or in the connections. Though the damages remains very limited in the main structures, the damages are quite numerous and varied in non-structural elements.

Old buildings not designed for the loads caused to earthquakes, or designed according to out of date earthquake-resistant codes, suffered more or less significant damage, with up to a few recorded collapses.

The examples found in the AFPS reports and other documents reviewed are shown in Fig. 6 to 16.

3.1.1 Complete collapse

- Collapse Pino Suarez Complex (Mexico City, earthquake of 19/09/1985-magnitude 8.1)

It was five recent buildings, 3 central buildings of 20 floors and two buildings of 14 floors (Fig. 6 - left photo). The first 20 levels tower collapsed on the 14 levels, the other towers being severely damaged on their sides: the frames are deformed significantly and a column of the second tower had a localized buckling at the bottom connection (Fig. 6 – right photo).

Fig. 6 Pinô-Suarez Complex – Mexico City Earthquake, 1985 - AFPS Mision Report

- House supported by a steel structure (Earthquake of Kobé, Japan of 17/01/1995-magnitude 7.2)

The ruin of this structure (Fig. 7 – left photo) may be due partly to the lack of bracing and also to the poor quality of the welding work. Indeed, Fig. 7 – the right photo shows a detail of a plate onto which the columns were welded. The break was made at the weld.

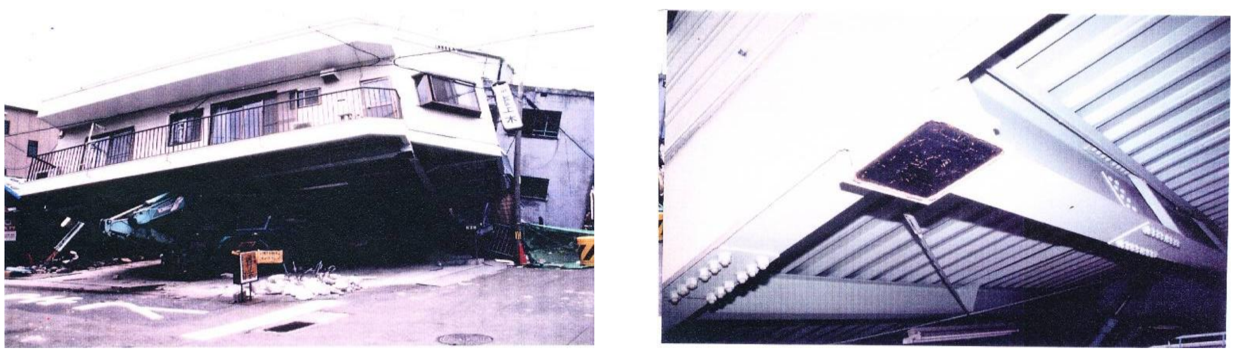

Fig. 7 Collapsed building - Kobe Earthquake 1995, Japan - AFPS Mission Report

- Collapse of two old steel structures (Chi-Chi Earthquake, Taiwan of 21/09/1999-magnitude 7.3)

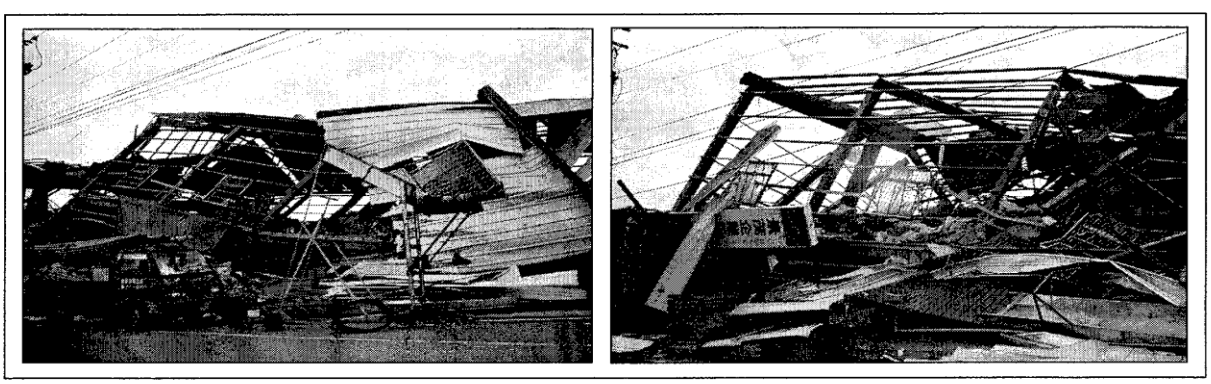

Fig. 8 Buildings collapsed - Chi-Chi Earthquake 1999, Taiwan - AFPS Mission Report

- Collapse of a steel industrial hall (Earthquake Chile, of 27/02/2010-magnitude 8.8)

The complete collapse of the roof occurred following the failure of the connections beam - column (Fig. 9). Observing the structure reveals that several key factors played a role in this collapse:

- Design fault of the truss beams and columns, of which the uprights and diagonals are made up of relatively slender angles, as well as strong eccentricities in the working drawing,

- Fragility of the connections beams - columns,

- Insufficient maintenance of the building, resulting in significant corrosion which can affect the strength of structures, especially of the welds.

Fig. 9 Collapsed of a steel industrial hall in Chillan - Chile Earthquake 2010 - AFPS Mission Report

3.1.2 <u>Partial destruction, major damage or collapse of a floor</u>

- Building having lost an intermediate floor - City hall of Kobé (Earthquake of Kobé, Japan of 17/01/1995-magnitude 7.2)

Fig. 10 City hall of Kobé - Earthquake of Kobé on 1995, Japan – AFPS Mission Report

The loss of an intermediate floor was a failure mode quite commonly encountered in Kobe. The lowest floors of some buildings of 8 to 10 floors are constructed using a steel structure coated in concrete for protection against fire. The upper levels are realized with classic reinforced concrete columns. The ruin of these buildings occurred in the connection area of the two types of structures. This is the case, in particular, of the old city hall of Kobé that contains 8 levels. On the photo of Fig. 10 we notice that this building lost its 6th floor.

- Major damage in steel structures in the port area (Earthquake of Cariaco, Venezuela of 09/07/1997 - magnitude 6.9)

Important disorders were observed. They are largely attributable to strong corrosion of metal profiles in marine and tropical environment. The earthquake only played a trigger role here on a structure in limit of stability (Fig. 11).

Fig. 11 Steel industrial halls - Earthquake of Cariaco 1997, Venezuela - AFPS Mission Report

- Significant damage to steel structures of the "Port of public trading" (Earthquake in Haiti, of 12/01/2010 - magnitude 7.1)

The port suffered the liquefaction phenomenon that caused the sliding of the platform, with consequences such as the movement of reinforced concrete foundations and the dislocation of the anchorage of the steel frame. The effects of liquefaction were added to a poor design: absence of deep foundations and the columns of the frames not resting on the ground but on the little concrete columns.

Fig. 12 Port of public trading - Haiti Earthquake of 2010 - AFPS Mission Report

3.1.3. Limited damage or no structural damage, but more or less important damage to non-structural elements

- Buildings for use in heavy industry in Armenia (Spitak Earthquake, Armenia of 07/12/1988-magnitude 6.9)

Constructions for heavy industrial use are produced in Armenia in frames with corbels or using heavy steel structures, with cross bracing, and provided with cladding masonry or precast reinforced concrete panels. Cladding generally fell apart, while the moment resisting frames remained intact, although some local buckling has been reported (Fig. 13).

Fig. 13 Industrial building – Spitak Earthquake, Armenia 1988 - AFPS Mission Report

- Industrial warehouses in Manjil, Iran (Earthquake Manjil, Iran of 20/06/1990-magnitude 7.3)

Some industrial warehouses with a range of about twenty meters, are built with large scale mechanic welded elements, assembled on site by well-kept plans of bolting (Fig. 14). Such buildings have not suffered any damage, except perhaps that due to the collapse of materials stored up high.

Fig. 14 Industrial building – Menjil Earthquake, 1990 Iran - AFPS Mission Report

- Sugar plant in Erzincan, Turkey (Earthquake Erzincan, Turkey of 13/03/1992-magnitude 6.9)

A number of buildings from 1 to 2 levels maximum in steel structure and masonry fillings performed well in the earthquake – Fig. 15 - left photo.

A more important building - a big warehouse with a steel structure - lost most of the windows and fills on its long sides, without presenting structural elements that reached a state of ruin – Fig. 15 - right photo.

Fig. 15 Sugar Factory - Erzincan Earthquake of 1992, Turkey - AFPS Mission Report

- "Carrefour" Store building in Izmit, Turkey (Kocaeli Earthquake-Izmit, Turkey of 17/08/1999 - magnitude 7.4)

The building has not been damaged in the earthquake (Fig. 16).

Fig. 16 The construction of the Carrefour store in Izmit - Kocaeli-Izmit earthquake of 1999, Turkey - AFPS Mission Report

3.1 Concentrically braced frames (CBF)

Buildings with concentrically braced frames had, with few exceptions, a good behavior to earthquakes. The examples found in the AFPS reports and other documents reviewed are shown in Figures 17 to 21.

- Steel frames building in the city of Rasht (Earthquake Menjil, Iran of 20/06/1990-magnitude 7.3)

Fig. 17 Steel frames building - Menjil Earthquake1990, Iran - AFPS Mission Report

In the city of Rasht, some buildings of 7 or 8 storeys were built shortly before 1980; one of them, that was under construction, remained in the state of a welded frame and with floors with some brick infills, bracing being provided by "X"-braces. All the connections are rather specific to a bolted construction, but the bolts are replaced by discontinuous welds. The building moved a lot in the earthquake. Significant damages are observed (see photos – Fig. 17), with inclinations of several centimeters per floor for various columns, breaking of bracing cross welds on the columns, with buckling of compressed braces.

- The Fire Command Center and Control Facility in Northridge (Los Angeles-Northridge Earthquake, USA of 17/01/1994-magnitude 6.7)

The Fire Command Center and Control Facility (Fig. 18) is an important element in the organization of emergency services in Los Angeles in particular following an earthquake. The building, steel frame braced by concentrically braced frames, consists of a ground floor and a floor. It rests on supports - 32 laminated elastomeric, evenly spaced under the structure. The behavior of the building in the Northridge earthquake was very good.

Fig. 18 Fire Command Center - Northridge Earthquake of 1994 - USA - AFPS Mission Report

- Fracture of brace member and fracture of its joint (Earthquake of Tohoku, Japan of 11/03/2011 - magnitude 9.0)

Buckling of brace member (Fig. 19 – left photo) and fracture of brace joint (net section fracture at bolt hole – Fig. 20) were observed at the steel Gymnasiums. The bucking of some diagonal member of latticed column was also observed. These latticed columns have the characteristic of being at simple diagonal working in traction and compression in the case of reversal of the sign of solicitation. The pronounced buckling is to be attributed to a high level of slenderness of the diagonal (Fig. 19 - photos of the center and left).

Fig. 19 Gymnasiums - Tohoku Earthquake 2011, Japan - Hokkaido University Report

Fig. 20 Gymnasium, Hall - Tohoku Earthquake 2011, Japan - Hokkaido University Report

Angle section was often used for many brace members, but circular hollow section steel was also used for brace members. For the Ibaraki Gymnasiums the fracture of brace welded connection was observed: fracture at column top (Fig. 21 – left photo) and fracture at brace crossing (Fig. 21 – photo of the center). Cracks were first propagated along the weld on one of the two faces of the pocket, causing a carry loads on the second half of the tube, which has broken by traction. The right photo of Fig. 21 shows bolts in shear fracture in another gym.

Figure 21. Gymnasium - Tohoku Earthquake 2011, Japan - Hokkaido University Report

3.1 Eccentrically braced frames (EBF)

The dissipative structures including structures with eccentrically braced frames, used now for twenty years, had a very satisfactory behavior to strong earthquakes that have suffered.

The behavior of this type of structures to earthquakes in Christchurch, New Zealand in 2010 and 2011 is very interesting. The lateral bracing of the active links, show their good dissipative behavior (photos of Fig. 22 and Fig. 23).

- The "Pacific Residential" Tower in Christchurch, New Zealand (Christchurch Earthquake, New Zealand of 22/02/201-magnitude 6.3)

The "Pacific Residential" tower of 22 floors is the tallest building in Christchurch.

The structure of this tower consists of perimeter EBFs up to the six floor (one on each building face), shifting to EBFs around the elevator core above that level, with a transfer slab were visible, as these levels housed a mechanical multilevel parking elevator system. A range of views of the structure are given in Fig. 22.

Paint flaking and residual link shear deformations were observed in the EBS links at the lower floors.

The EBF at all other levels were hidden in architectural finishes, and absence of damage to those finishes suggested limited inelastic deformation, except at level six where a few of the hotel room doors along the corridor could not be closed, which suggest greater residual deformations at that level.

A fractured link was discovered under the floor of this level (Fig. 22 e), in the northwest corner of the building. This element is visible behind the atrium in the corner of the nearest building in the photo (Fig. 22 a).

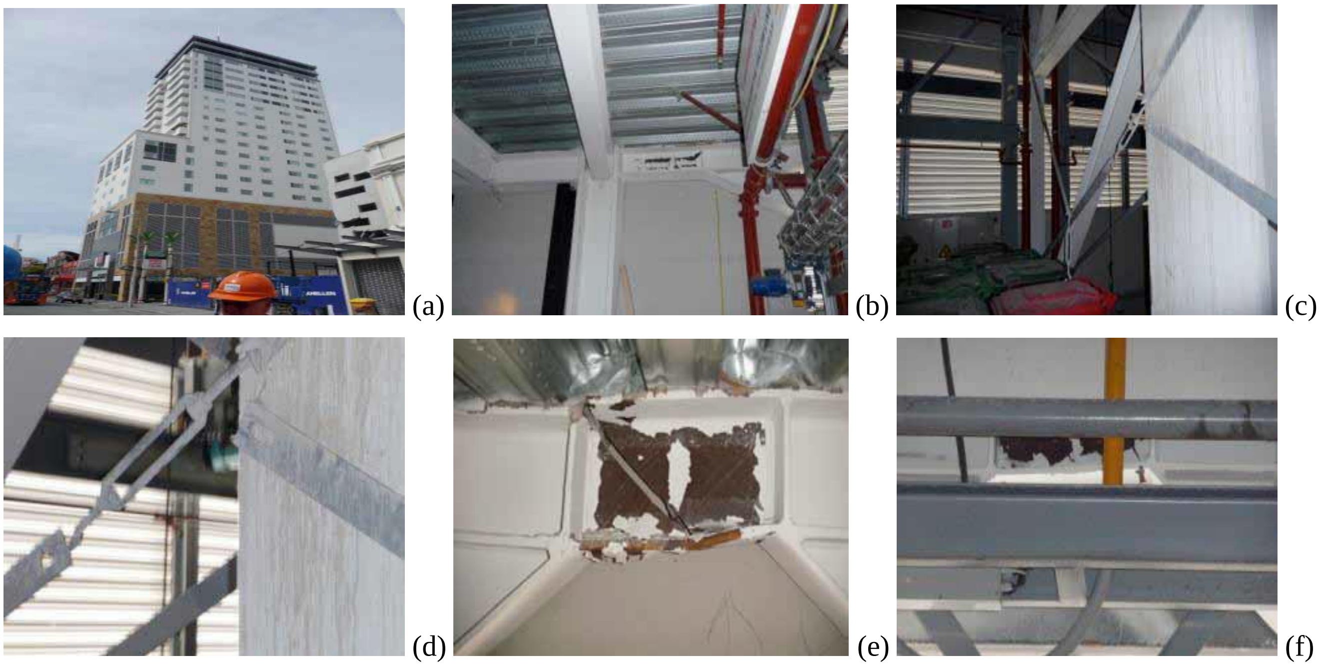

Fig. 22 The "Pacific Residential" tower, Christchurch - Canterbury University Report (Photos by M. Bruneau)

(a) Overview; (b) Paint flaking in the EBS links; (c) and (d) Fracture of a brace member at the level of the cars elevator; (e) EBF link fractured at the atrium; (f) Residual link shear deformations in the EBS links.

- Parking garage on St. Asaph St. and Antigua St. in Christchurch, New Zealand (Christchurch Earthquake, New Zealand of 22/02/2011 - magnitude 6.3)

The EBFs in this hospital parking garage closer to the epicenter performed well, although some link fractures were observed in two braced bays (Fig. 23).

The fractures, as shown in close-up in Fig. 23(c) were of particular concern as these were the first fractures recorded in EBFs worldwide. Further puzzlement was added by the fact that the fracture plane, shown in Fig. 23 (c), indicated a ductile overload failure rather than a brittle fracture. However, the likely explanation lies in the offset of the brace flange from the stiffener. This offset is shown in Fig. 23 (c) and means that, when the brace was loaded in tension, the axial tension force in the brace fed into the active link/collector beam panel zone through a flexible beam flange rather than directly into the stiffener. This meant that the junction between the unstiffened beam flange and the beam web was overloaded, leading to fracture between these two surfaces and this fracture spreading across the beam flange through the web.

The lateral bracing of the active links in the building shown in Fig. 23 was only in the form of a confining angle on each side of the top flange, as shown in Fig. 23 (d) and 23 (e). No lateral movement or twisting of the ends of the active links was observed, indicating that the lateral restraint provisions had been adequate despite only being applied to the top flange and for EBFs not integral with the slab above.

(a) (b) (c) (d) (e) (f)

Fig. 23 Parking garage on St Asaph St et Antigua St, Christchurch - Canterbury University Report (Photos by M. Bruneau)

(a) Redundancy provided by multiple EBF bays ; (b) Evidence of EBF link yielding ; (c) Fractured link at lower level EBF ; (d) and (e) Evidences of inelastic deformations at top level EBF ; (f) Close-up view of same.

4. Conclusions

The Reports of post-seismic missions carried out by the AFPS for thirty years is a very interesting and useful database for the study of the behavior in the earthquakes of different types of structures, designs and used materials.

The magnitudes, intensities, depths and types of earthquakes that were the subject of these analyses vary widely, which gives additional interest to this database. There are earthquakes of very high magnitude (between 7 and 9) and there are moderate earthquakes (between 5 and 7), closer to what has happened and what we expect to happen again in France.

Following analysis of these reports and several articles and other publications concerning earthquakes that have not been subject of site visits (eg earthquakes in Tohoku and Christchurch), an initial assessment has been established on the behavior of steel constructions. It is work that will be continued with other older earthquakes, not covered by post-seismic missions.

The experience feedback shows a globally satisfactory behavior of this kind of structures in earthquakes. This applies to buildings designed and made according to modern seismic regulations and even in part, for old buildings that have benefited from the natural ductility, characteristic to steel structures.

However, some examples of collapsed structures were found, or with very large structural damage, or with limited damage or no structural damage, but with more or less important damage to non-structural elements.

The characteristics damage noted most often by inspectors was:

- Residual link shear deformations and fracture of the brace members ;
- The dislocation of the anchorage, elongation or fracture of the anchor bolts ;
- Brace/beam/column connections showing out-of-plane yielding or fractured connections ;
- More or less important damage to non-structural elements.

The solution used in countries like Japan and New Zealand to ensure lateral stabilization of steel structures by the eccentrically braced frames (EBF) has a great deal of interest. This is a practice that is rarely used in France, but which showed its efficiency in cases where a dissipative functioning is required.

5. References

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