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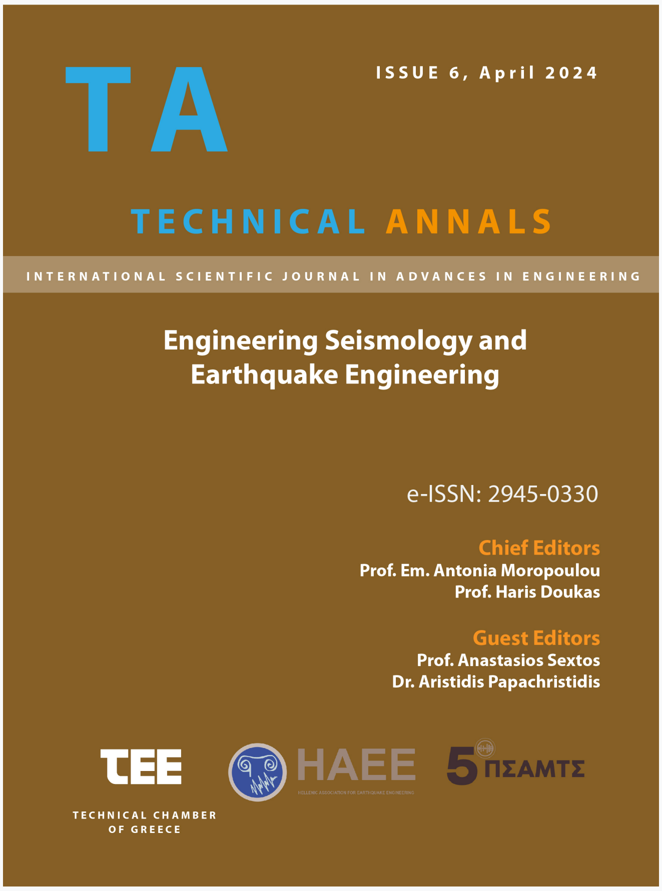

Behavior of steel building structures subjected to strong and benchmark seismic actions: An overview of damage from the observations of the last 40 years

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Behavior of steel building structures subjected to strong and benchmark seismic actions: An overview of damage from the observations of the last 40 years

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Abstract. It is well known that steel structures have a high ductility capacity and a high strength-to-weight ratio, which theoretically, by nature, makes them one of the most efficient seismic structural systems against strong earthquakes. However, the recorded experience of failures that have befallen over the last 40 years as a result of strong seismic actions suggests that this by itself isn't always sufficient. Generally, it is essential that an appropriate and preferred conformation and configuration of the structural system, and in particular of its joints, be adopted. In any case, the steel building structures showed local failures without general or complete collapses. The work in this paper presents the seismic performance focused on steel building structures, as revealed by strong earthquakes such as those of Mexico (1985), Northridge (1994), USA, Kobe (1995), Japan, Christchurch (2010–2011), and New Zealand, which affected and changed the design of metal structures, as well as other earthquakes like Maule (2010), Chile, Emilia (2012), Amatrice (2016), and Italy, which completed the picture in the better understanding of failures and their reasons. On the basis of the lessons learned, a discussion on avoiding such situations is commented on and provided in this work.

Keywords: Steel Buildings, Seismic Performance, Benchmark Earthquakes

1 Introduction

Traditionally, steel structures are considered one of the most efficient earthquake-resistant systems due to their enhanced ductile capacity and high strength-to-weight ratio. However, as revealed by past earthquakes, these two very important mechanical parameters are not sufficient to avoid failures. Although it has been observed that no global collapses have occurred so far, except in a few special cases, only certain local failures are registered at joints and connections. This real fact was recognized by the engineering community through the reconstruction of Christchurch in New Zealand [1].

In fact, the way to learn and improve our design, detailing, and construction practices comes from two aspects: the first is to learn from failures, and the second is from successes. The first one reveals the level of vulnerability, while the second one reveals the level of capacity of any type of structural system. Both are very educational and form the dipole of engineering knowledge and judgment. In this direction, the seismic performance is also defined by the aforementioned. The engineering community is mainly focused on failure, with success being self-evidenced. Associated with steel building structures, the avoidance of global collapse is considered a success of steel structures. Thus, it is of paramount importance to look at and provide a concise presentation of failures from past earthquakes that influenced the design and construction, and at the same time, a scope of discussion as well. Literally speaking, "*the light failing through the crack...*" as wrote Leo Tolstoy, 1869, in the book War and Peace (as in our cases is Failure and Success). The Northridge (1994), USA, Kobe (1995), Japan, and Christchurch (2010–2011), New Zealand, could be considered seminal earthquakes for steel building design practice, while other important earthquake events, such as Mexico (1985), Maule (2010), Chile, Emilia (2012), Amatrice (2016), and Italy, also contributed to providing information on the seismic performance of steel building structures.

Typically, the main structural systems used for steel buildings are the following: (i) moment-resisting frames, MRF, where the seismic resistance is provided mainly by the cyclic bending action of beams and columns, targeting through the capacity design to concentrate the inelastic action only in the beams; (ii) concentrically braced frames, CBF, where the seismic resistance is provided by the cyclic axial action of the braces; (iii) eccentrically braced frames, EBF, where the seismic resistance is provided by the axial, shear, and bending cyclic action of the eccentric region between the braces; and finally (iv) frames with buckling restrained braces, shear wall, and rocking systems [2]. From the above-mentioned structural systems, only (i), (ii), and (iii) are subjected to strong earthquakes, while the systems of (iv) have mainly progressed after the Northridge and Kobe earthquakes, and we have no signs of their behavior under real strong cyclic actions. Therefore, in the following discussion, only (i), (ii), and (iii) will be commented on. Moreover, in the steel building sector, due to its different structural requirements, we can distinguish between one-story buildings primarily for industrial applications and multi-story buildings for residential, office, retail, and hotel uses.

This paper provides an overview of the failure observed from the occurrence of strong earthquakes over the past 40 years that influenced the steel building industry, followed by a brief discussion related to the avoidance of such situations in European engineering design practice.

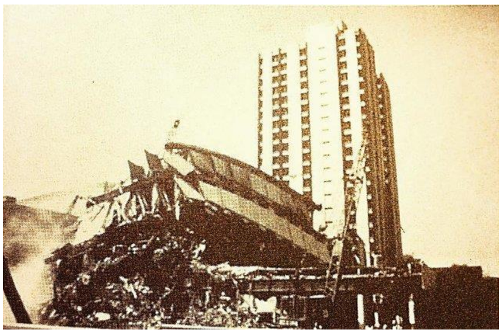
2 Seismic performance of steel building structures

2.1 Mexico City, Mexico, 1985

On September 19, 1985, a strong earthquake of Ms = 8.1 magnitude, with its epicenter 400 km from Mexico City, affected Mexico City, which is situated in a highly compressible clay of an old lake bed [3]. High amplitude, along with a large number of strong cycles and a long duration of about 30 seconds, led to the collapse of many high-rise resilient buildings. The specific characteristic of this earthquake was that the epicenter, localized in Michoacán State, was about 400 km away from Mexico City, where the severe damage occurred; the soft clay geotechnical conditions led to the amplification of the period at around 2.0 seconds, which coincided with the period of the collapsed buildings [4,5]. The Michoacán earthquake, due to the unique soil conditions of the Mexico City basin, mainly unveiled the site effects and their influence on period amplification, long duration, large amplitude of ground motion, and high accelerations as well. Generally, structures of six to twenty stories, interestingly built between 1956 and 1976 and made of reinforced concrete, were severely damaged, while buildings with less than six and more than twenty stories also sustained significant damage [3,6,7].

Steel buildings in Mexico date from the '20s, they are generally considered more expensive than reinforced concrete structures. Mainly starting in the '40s, many tall buildings, ranging from 25 stories to 43 stories, were constructed [7]. In any case, steel buildings behaved very well, except the Pino Suarez building complex, and especially those with a natural period of 2.0 seconds, which was critical for resonance [7,8].

The Pino Suarez complex was constructed in early 1970 and consisted of five steel moment resisting frames, (three central buildings of 21-stories and two of 14-stories). The structural system of the 21-story collapsed building, which failed onto the 14-story building, was a moment-resisting frame consisting of welded plates forming box columns with truss beams and moment-resisting connections. The collapse was attributed to the buckling of the exterior box columns, in the fourth storey [8,9], Fig. 1.



(a)

(b)

Fig. 1. (a) Collapsed building and (b) buckled column of the Pino Suarez building [3].

The lessons learned from the 1985 Mexico earthquake were not focused on steel structures that behaved well. The geological setting, the effect of local geotechnical site conditions, and the soil-structure interaction that strongly alters the strong ground motion, affecting the inelastic behavior of structures, were the main aftermaths. Nevertheless, this earthquake marked the first notable collapse of a steel structure and, moreover, begged questions about steel conformation practices and redundancy, the level of available ductility to withstand severe earthquakes, and the slenderness of box sections as well.

2.2 Northridge earthquake, USA, 1994

On January 17, 1994, a strong earthquake of Mw = 6.7 magnitude, with the epicenter near Northridge, about 30 km northwest of downtown Los Angeles, California, damaged more than 40.000 buildings of all structural systems. More than 150 tall and short (from one to 26-stories), new and old, steel moment-resisting frames suffered widespread brittle damage. It should be underlined that no steel building collapsed and no loss of life was registered. The typical US pre-Northridge flange welded-web bolted with shear tab connection is presented in Fig 2. The failure is concentrated at the joint region, especially at the beam-column connection and mainly at the bottom flanges [10,11], Fig. 3. The damage was observed in the welded connections, with complete penetration welds between the beam and column flanges. In some cases, cracks were propagated into the column's web and, as such, developed a column fracture or cracks into the beam's flanges. Prior to the Northridge earthquake, this connection type was believed to have adequate ductility capacity to withstand high seismic forces. Despite this, experimental testing of a large scale of such connections between 1970 and 1992 revealed relatively low beam plastic rotations between 0.010 and 0.030 rad [12,13,14]. Moreover, for economic reasons, a lot of buildings were constructed only with perimeter moment-resisting connections, while the rest were bolted with simple supported connections, and in many cases, the moment-resisting connections had no continuity and/or doubler plates or lapped plating. The brittle failure was attributed to the mechanical properties of materials, insufficient practices of constructional conformation, detailing, welding, and design, and poor workmanship [15,16], although an additional important contributing parameter was the near-field ground motion, which introduced a high strain rate leading to brittle mode failures [17,18,19].

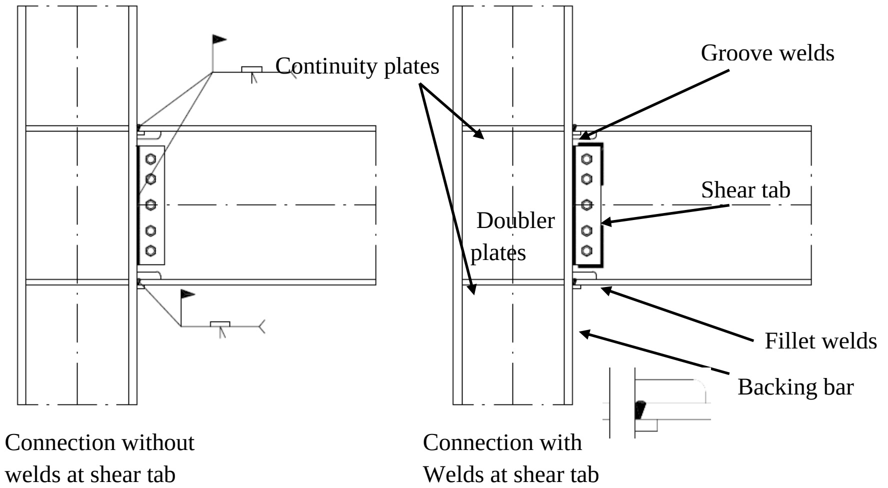


Fig. 2. Typical US pre-Northridge flange welded-web bolted beam-column connection, with and without weld at shear plate, for welded steel moment frames.

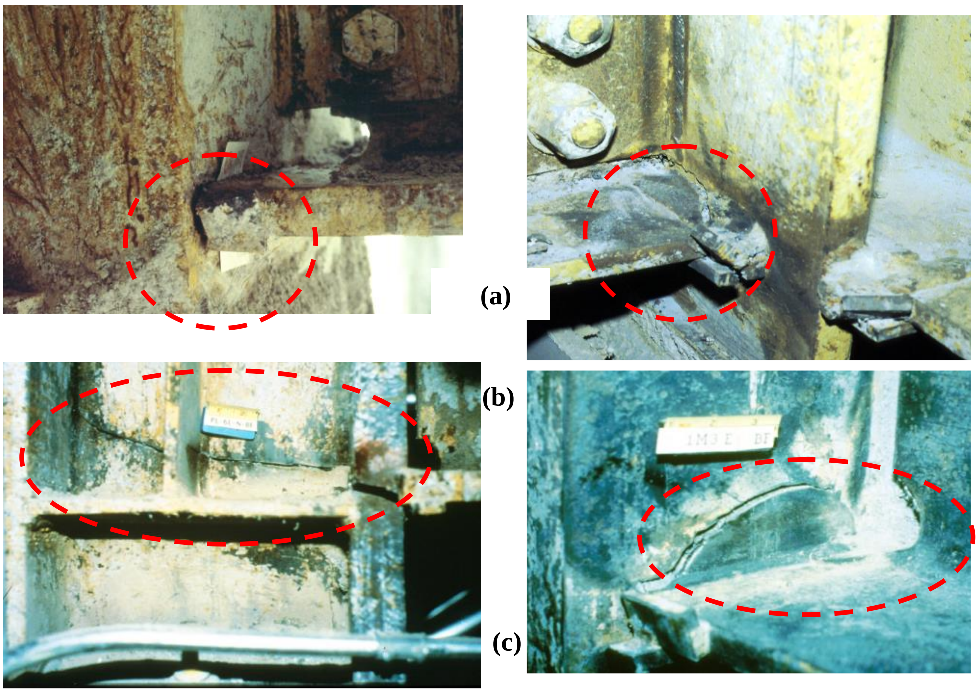


Fig. 3. Typical damage observed after the Northridge earthquake, (a) Fracture at fuse zone, (b) column fracture, and (c) column flange "divot" fracture [10].

The Northridge earthquake, 1994, USA, is a benchmark point in the history of seismic design of steel structures, especially steel welded moment-resisting frames. The unexpected damage greatly surprised the US structural engineering community [20]. The response was immediate and radical; a great research program was initiated, starting in 1994 and ending in 2000, through the cooperation of The Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC), and California Universities for Earthquake Engineering (CUREE) in cooperation with the Federal Emergency Management Agency (FEMA), forming the SAC Joint Venture with the main goal of "developing reliable, practical, and cost-effective guidelines and standards of practice for the repair or upgrading of damaged steel moment frame buildings, the design of new steel buildings, and the identification and rehabilitation of at-risk steel buildings" [21, 22]. It was an exemplary program; the result was design-oriented, namely, the development of guidelines, which were the basis for the further processing of new standards and codes not only for new structures and for the rehabilitation of existing ones [23, 24,25], but also sound information related to the metallurgy of structural steel, welding, inspection, and quality control [26,27,28].

It was realized that the column face, at the connection, is a highly stressed zone; therefore, constructional detailing using the concept of weakening (reducing the upper and lower flanges at a selected distance from the column face) [29,30] or strengthening (adding ribs, cover plates, and haunches) could be applied [31]; the plastic hinge must be removed from the column face to a zone of lesser stress. In addition, the detrimental effect of the slab, when it is coupled compositely with the beam, which converts the strong column-weak beam concept into a weak column-strong beam mechanism, was also evaluated [32,33].

Related to moment-resisting frames, among the "constructional novelties," the reduced beam section was introduced in the current practice of designing welded moment connections. Further on, the welding details of the flange-welded-web bolted connections are strongly improved. Additionally, new lateral load-resisting systems were introduced in practice, such as the buckling restrained braces [34] and steel plate shear walls systems [35].

Associated with the improvement of the seismic design, all the traditional steel structural systems (moment-resisting frames, concentrically and eccentrically braced frames) were scrutinized, and improvements related to ductility and capacity design were provided [36]. Another important step was the development of loading protocols for the assessment of the inelastic behavior of the steel subassemblies and components [37,38]; moreover, understanding the differences in near-field ground motion-related protocols was also developed and used [39]. Nevertheless, the most important issue was the implementation of performance-based design, where a qualitative concept was transformed into a quantitative methodology based on reliability engineering [22,40,41].

In the USA, the codification is rather complex and is not harmonized as in Europe or other countries; the seismic design of steel structures was fragmented between different codes according to region and jurisdiction [42]. However, the impact of the Northridge earthquake was crucial. The American Institute of Steel Constructions (AISC), representing the US steel industry, took initiatives and, exploiting the results of the SAC Joint Venture research program, starting in 1997 and continually revising the standards every five to six years, developed a complete design framework. Nowadays, there exist three major standards related to (i) Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341/22), (ii) Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (ANSI/AISC 358/22), (iii) Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings (ANSI/AISC 342/22) [43]. Three issues are of particular consideration. The first is connected to prequalified connections, where the prescribed eleven types of moment connections, in ANSI/AISC 358/22, are prequalified and no testing is required; this means that the corresponding moment connection is sufficiently examined (testing, analysis, evaluation, review), providing a sufficient level of confidence. The second issue is that among the prequalified connections, there are seven that are non-proprietary (reduced beam section (RBS), bolted unstiffened and stiffened extended end-plates, bolted flange plate (BFP), welded unreinforced flange-welded web (WUF-W), cast bolted bracket (CBB), double-tee moment connection, slotted web (SW) moment connection) and four that are proprietary connections (ConXtech CONXL connection, SidePlate connection, Simpson strong-tie strong frame, DuraFuse Frames moment connection). The third issue prescribed in ANSI/AISC 341/22 is the protected zone; this means that in locations where large strains are expected (i.e., plastic hinges in beams), there are no attachments, discontinuities, or welded shear studs. Finally, the ANS/AISC 341 cancels the prequalification in the case of composite slabs that are present. All the aforementioned provisions follow the concept to ensure a reliable capacity design (i.e., the formation of plastic hinges in beams and not in columns, as well as to not cancel the predetermined designed plastic mechanism).

2.3 Kobe earthquake, Japan, 1995

On January 17, 1995, a powerful earthquake of Mw = 6.9 magnitude hit Kobe city. The severely damaged area was a narrow, concentrated band of approximately one kilometer in width and extending, in length, for about 25 km. The epicenter region was located on Awaji Island, 20 km from the city of Kobe. It was a near-field ground motion with impulsive characteristics and a short duration of about 10–15 seconds. Furthermore, close to the epicenter area, large vertical accelerations were also recorded, with a vertical to horizontal peak ground acceleration ratio (V/H of more than 1.5 and a mean value close to 0.90 [44]. For instance, in the case of the Northridge earthquake, a V/H ratio of 1.79 was recorded [45]. Ground motions varied significantly at the different sites due to local soil and geological conditions (i.e., infill-reclaimed land, alluvium, soft rock, and variations in the thickness of the soil at different sites).

Steel is the second most popular structural material after wood in Japan [46]. The typical beam-column connection for moment resting frames used in Japan is depicted in Fig. 4. Generally, a box column (cold-formed or built-up) is connected with the use of groove welds, by the aide of the steel diaphragms, with the beam; the column is divided in three parts, one for the lower storey, one for the upper storey, and the middle segment to form the rigid node of the beam-column connection. A shear plate welded at the shop is used to facilitate the easy on-site welding of the beam to the column. This is the through-diaphragm connection, which was the most popular before the Kobe earthquake. There was also a solution with interior and exterior diaphragms, which are the most costly.

It was surprising that the same brittle damage at the beam-column connection, as it was in the case of the Northridge earthquake exactly a year ago, was observed. The same failure at the welds, heat-affected zone, and base material fracture is recorded, although the general conformation of a typical joint between US and Japanese practice is different, Fig. 4. Moreover, brittle failure also occurred at: (i) columns (of square hollow section mainly cold formed), with fracture at the base material of approximately 50–55 mm thickness, at welded column splices, and beam to brace connections; (ii) column bases at anchor bolts; and (iii) braces of small cross section and large slenderness like rods, angles, and flat plates for older buildings, while for modern buildings with larger cross sections, the damage was concentrated at the connection zone [46,47,48,49], Fig. 5,6.

The causes of damage attributed to the fracture toughness of steel material, general configuration and detailing of the beam to column connection, low toughness of weld metal and the severity of the strong ground motion having strong pulses and high V/H PGA ratio as well. The repeatability of the damage observed for both Northridge, 1994, USA, and Kobe 1995, Japan, unveiled the enhanced vulnerability of the welded beam-to column connection of the steel moment resisting frames. Moreover, the practice was to execute the welding on site. Hence, considering that joints are severely stressed, then the weldments that characterized by a potential brittleness, must be very well executed, inspected and assured the quality control; this was not the case. It was a failure of the seismic design and construction practice for both countries.

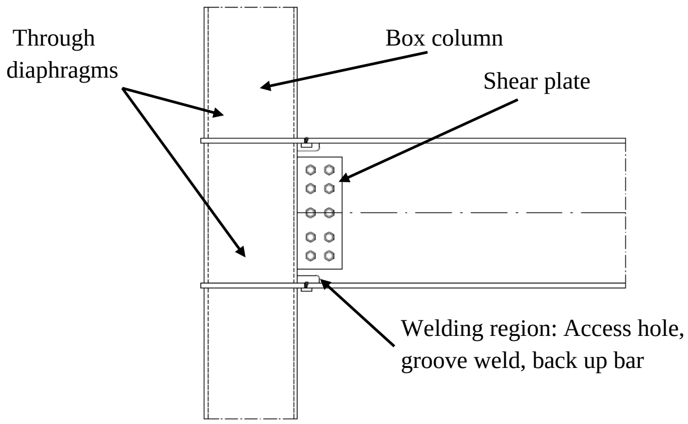


Fig. 4. Typical beam-column through diaphragm connection before Kobe earthquake.

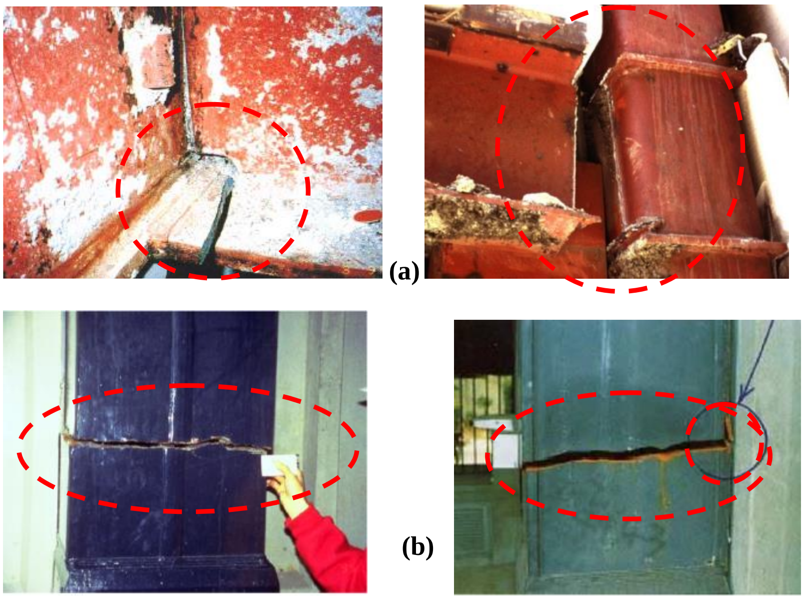


Fig. 5. Brittle damage from Kobe earthquake, (a) beam to column fracture, (b) column fractures [48,49].

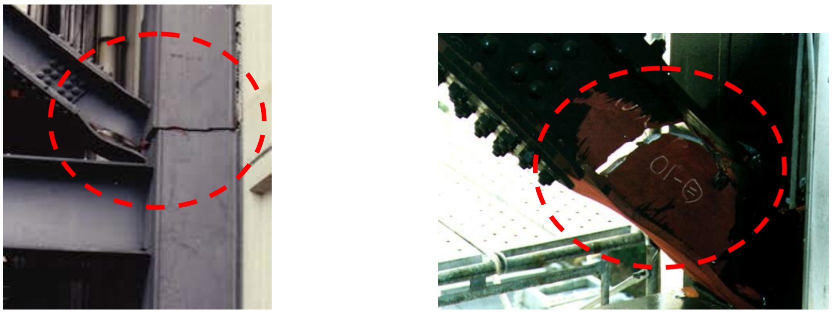


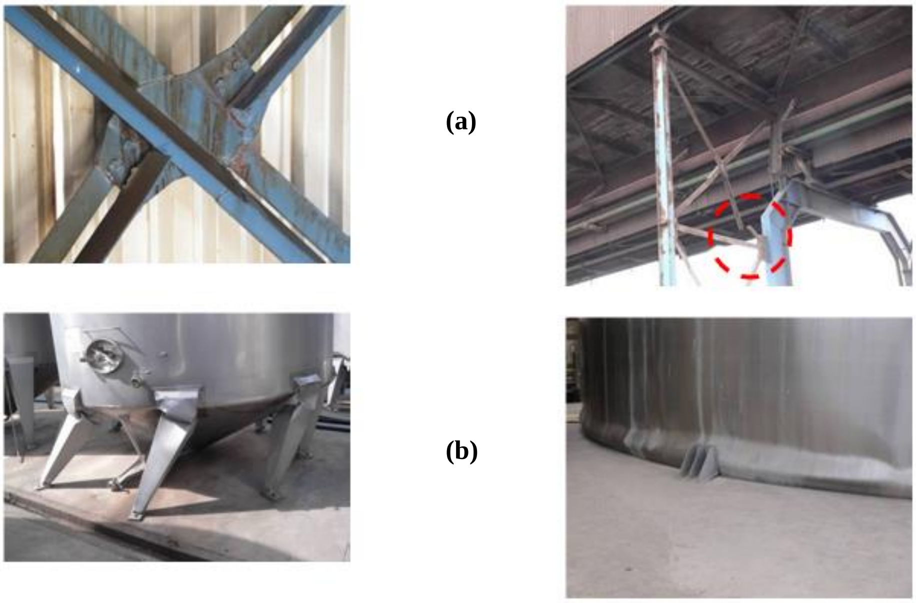
Fig. 6. Brittle damage from Kobe earthquake, brace fracture [48,49].

As was the case for the USA after the Northridge, Japan also funded many research programs targeting to investigate the material and welding practices, evaluate the inelastic behavior and the ductility capacity of moment-resisting connections, and improve the welded connections [50]. Certainly, the design code was revised towards performance-based design [51]; however, the main progress was the development of high-strength steels, fire-resistant steels, low-yield steels (with yield limits between 100 and 200 MPa for use in steel hysteretic dampers), the introduction of buckling restrained braces, as well as the use of shear panel dampers [50]. The Kobe earthquake triggered the construction of the world's largest shaking table 3-D Full-Scale Earthquake Testing Facility, nicknamed "E-Defense" [52].

2.4 Maule earthquake, Chile, 2010

On February 27, 2010, a strong earthquake of magnitude Mw=8.8 struck Chile. The epicenter was located 8 km from the town of Curanipe and 115 km from the second-largest Chilean city of Conception, with peak ground acceleration in that city of the order of 0.65g. This ground motion was the second largest in Chilean earthquake history, after the Valdivia earthquake (1960). The most popular structural material in Chile is reinforced concrete; however, steel is used for the construction of industrial facilities, and therefore damage was observed in such structures [53, 54]. Generally, the steel structures performed well in that severe ground motion. Brace buckling, anchorage failure, roof truss failure, and buckling of the leg and wall tank are observed in Fig. 7 [53]. In addition, extended damage was observed to non-structural elements, such as the unreinforced masonry used as a façade, infill and interior partition masonry (mainly due to out-of-plane action), ceilings, and other architectural elements.

This positive performance of steel structures was correlated to the overstrength provided by the seismic design code (which is based on US codes) rather than the ductility capacity [53]. It is important to mention that for many industrial facilities, the snow or wind load is the predominant one, providing the design of the building and not the seismic actions. One can observe that this powerful ground motion unveiled the following: (i) the resilience of steel structures used in the industrial sector; (ii) the importance of anchorage and bracing the equipment in order to avoid production and business interruption.



(a)

(b)

Fig. 7. Maule earthquake failure, (a) brace buckling and fracture, (b) buckling at the base of tanks [53].

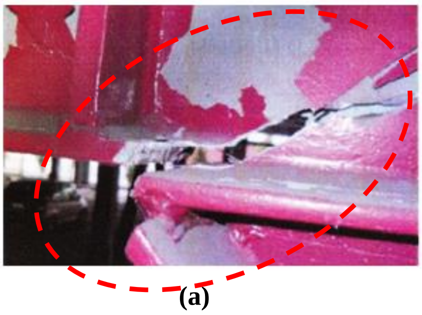
The Maule earthquake has not had the same global impact as the Northridge, 1994, USA, and Kobe, 1995, Japan earthquakes; it was a strong earthquake that subjected steel structures beyond their limits in one of the highest seismicity countries in the world. In fact, it was a test of industrial facilities built with structural steel, largely used all over the world. Chile has a special design code for the seismic design of industrial facilities, Ch2369.Of2003 [55], which also includes provisions coming from the US seismic design codes (AISC) [56, 57]. It is important to notice that, concerning industrial facilities, except for life safety, it is of paramount importance to be functional after severe earthquakes; thus, they are designed to remain in the elastic field of behavior. Concentrically braced frames respect such conditions. Studies revealed the effectiveness of the existing code; however, in order to satisfy continuity and operational functionality after a strong earthquake, the drift is the controlling parameter [58]. Moreover, there are many situations where sensitive industrial equipment is also controlled by floor accelerations [59].

2.5 Christchurch earthquake, New Zealand, 2010-2011

On September 3, 2010, and on February 22, 2011, strong earthquakes struck the city of Christchurch; the second one was more catastrophic for the built environment. The focal depth was 5 km, and the epicenter was 10 km from Christchurch's Central Business District, causing collapses and widespread damage to reinforced concrete and unreinforced masonry buildings.

With regard to steel structures, they generally performed well [60]. A specific fracture was observed in the active links of the eccentrically braced frames at Pacific Residential Tower, completed in 2010, with a perimeter EBF as part of a lateral load resisting system, and at a hospital parking garage designed to accommodate two additional floors [60], the first known recording of such failure worldwide, Fig. 8. Related to the EBF links, many studies were performed in order to explain this unexpected fracture, supporting the lack of redundancy, constructional inefficiency, and material inefficiency. However, this type of failure was concentrated on the aforementioned cases and was not a systematic observed fracture. The other types of failure are the classical ones related to steel structures when subjected to severe shaking [46].

Of paramount importance to be underlined is that the steel structures were the preferred material for the reconstruction of Christchurch [1, 60], due to their repairability as compared to the reinforced concrete structures. New types of steel frames were proposed and constructed, namely low-damage systems, shifting the current seismic design philosophy from ductility to repairability and performance [62].




(a) **(b)**

Fig. 8. Fracture of EBF's links, a) at hospital parking garage, b) at Pacific Residential Tower, [60].

Moreover, typical fractures in braces and local buckling in braces and columns of storage rack pallets were also observed [60], Fig. 9. However, the same damage was also reported in the USA in a series of medium and strong ground motions, such as the Whittier earthquake in 1987, the Loma Prieta earthquake in 1989, the Landers earthquake in 1992, the Northridge earthquake in 1994, the Nisqually earthquake in 2001, and San Simenone in 2003. Storage rack pallets are light, although they carry heavier live loads than dead loads. These structures have complex inelastic behavior due to thin-walled and unsymmetrical cross sections and asymmetry of connections as well, making them prone to buckling. For instance, in the USA, after the repeated and extended damage to the steel storage rack pallets, the FEMA 460 document was published [64] in order to guide the industry.


Fig. 9. Failures of steel rack pallets in Christchurch earthquake, 2011 [60].

This New Zealand earthquake of 2010–2011 was once again an alarming sign that not only the life safety but also the functionality of buildings (and also the infrastructure) must be ensured after a strong earthquake. The structural engineering community realized that ductility is a property that saves lives; however, this one should be balanced with sufficient stiffness in order to avoid extended non-structural damage that interrupts the operation of a building facility. New types of structural systems are proposed (such as self centering steel systems, eccentrically braced frames with replaceable links, braced frames with controlled rocking [65,66], dissipative fuses for steel moment and braced frames [67,68,69] in order to ensure a controlled inelastic behavior, with predetermined collapsed mechanisms, however, easily repairable after a strong earthquake.

2.6 Emiglia Romagna earthquake, Italy, 2012

On May 20 and 29, 2012, two strong ground motions of magnitudes of Mw = 6.1 and 5.8 hit the Emilia Romagna region in the northern part of Italy. A special characteristic of this earthquake was the high peak vertical acceleration of the order of 1.0g [63,70].

This earthquake affected the urban and industrial areas, where, for the second one, many reinforced concrete one-story precast buildings were severely damaged or collapsed. With regard to steel structures, this quake could be associated with the damage to cold-formed steel racks that are or are not part of the lateral load-resisting system of the building (Fig. 10). Column local buckling, plastification of connections, anchor failure, and buckling of braces are the main types of damage [63,70]. This earthquake unveiled, once again, the lack of seismic design for such structures.

In response to the aforementioned failures, in Europe, after intensive research, [71,72], the EN 16681 design code for "Steel static storage systems. Adjustable pallet racking systems. Principles for seismic design" was developed and implemented as a European regulation, [73].

Fig. 10. Failure of steel racks making part, or not of the lateral load resisting system, Emilia earthquake, 2012 [70].

2.7 Amatrice earthquake, Italy, 2012

A sequence of earthquakes on August 24, October 26, and 30 hit the region of Central Italy, causing severe damage mainly to buildings and the architectural heritage.

Related to the steel buildings, a characteristic failure was pointed out in the case of an moment resisting frame interacting with masonry infill, Fig. 11, [74]. There are many situations where infill masonry is used instead of braces to limit lateral deformation. However, for such cases, special detailing should be used; for instance, we can distinguish between two main ways: (i) detaching the infill masonry from the main frame and filling the gap with a compressible material, or (ii) connecting it with steel angles or other steel elements anchored to the main frame. In both cases, out-of-plan bending of the infill masonry must be ensured.

Fig. 11. Steel frame with infill masonry interaction, Amatrice earthquake, 2016, [74].

3 Discussion and aftermaths

A brief overview of the seismic performance of steel building structures reveals the vulnerable points and defines the limits of capacity. Principally, the accumulated experience from past earthquakes revealed that a steel structure must be detailed ensuring: (i) a continue flow of forces, without concentrations, (for instance using continuity plates, diaphragms), (ii) a compatibility of connecting systems, (i.e. in case of a combination of bolts and welds employed in a connection), (iii) alternative load paths when a structural component buckles, through the use of high redundant systems, (iv) proper bracing in all directions of action and (v) suitable anchoring not permitting undesirable sliding or movements, (vi) the differences between near field, (predominant strain rate effect and vertical acceleration component), vs. far field, (predominant cyclic effect and influence of geotechnical conditions), earthquake actions [2,75,76,77].

It is well known that old structures designed without using the capacity design philosophy are more prone to damage and have a higher probability of collapse. Even the newer structures dimensioned according to capacity design must be detailed in such a way to move the plastic hinge away from the column face; this is done because, by definition, in moment-resisting frames, the beam-column connection is the most stressed region. Therefore, over-stress mitigation techniques (by strengthening, adding cover plates and haunches to the beam's flanges, or by weakening and reducing the beam's flanges, namely the reduced beam section concept) should be applied [75]. In the case of MRFs governed by drift design, it is proper to use the strengthening solution to improve the rigidity of the joints, while in the case of prevalent seismic design, the weakening solution is preferred to relocate the plastic hinge in the lower stressed zone [31,33,78]. Furthermore, the application of the “column tree concept” is also a suitable solution, where shop welded and field bolted connections are employed (beam-column shop welded connection and a field bolted splice of beams, applied at a point of low bending moment, within the beam span). Finally, a simple conforming rule is to use higher steel quality for columns than the beams, forcing the formation of the plastic hinge in a region of lower strength (i.e., columns made from steel quality S355 or S460 and beams from steel quality S235 or S275, or even S355 when the steel quality of the column is S460) [79,80,81]. Generalizing this rule, for elements that by definition are preferred to remain in the elastic region, a higher steel quality should be employed. For steel moment-resisting frames subjected to near-field actions, some measures would be to use thinner plates, high steel ultimate to yield stress ratio, sections with wider flanges, forcing longer flange buckling lengths [82].

The aforementioned discussion is associated with the ductility concept, which accepts a high level of inelastic deformation, namely a high level of damage. Nowadays, we can distinguish two new design trends: (i) a new concept of low-damage beam-to-column connections, as accepted and widely used in the Christchurch reconstruction [1, 62], and (ii) a design philosophy of connections free of damage [83] and prequalification of connections (applicable to Europe [84,85]). In practice, the seismic design should balance stiffness, strength, ductility, and repairability according to seismicity, geotechnical conditions, building importance, cost of business interruption, and construction budget.

Globally, both in academia and industry, a shift from the traditional forced base design, considering ductility as the only vital mechanical property, to a resilient base seismic design is under debate and investigation. In fact, ductility is connected with life safety and collapse prevention. Currently, the structural engineering community is striving with issues of sustainability (to reduce material consumption, to reuse the structural elements and structures, and to recycle the steel material) and structural resiliency (predictable and timely functional recovery after a strong earthquake [86]). Such examples are presented in [87, 88]. Nevertheless, any type of action requires collaboration, combination, and the development of strong relationships between research, construction practices, and policies.

4 Conclusions

The present work, through a qualitative analysis, records the failures in steel building structures observed in the last 40 years after strong earthquakes. Such review studies are very useful; in the evolution of time and in a centralized way, one can monitor the seismic performance of steel building structures, unveiling the vulnerabilities and capacities. Learning from failures is part of education and development. From failures, we mainly understand the system vulnerabilities. Nevertheless, we also learn from success; this reveals the system's capacities. To this end, we must remark that unfortunately, in academic studies and also in university curricula, there are not

such lectures to educate young students or professionals about the successes and failures of the different structural systems.
Focused on the current analysis, the following conclusions could be drawn:

- Usually steel building structures, except in the case of the Pino Suarez building in Mexico City, present only local failures that are repairable and not global failures or collapses. They ensure the criteria of life safety.
- In the case of multistory buildings, care should be given, after strong earthquakes, to the inspection of the welded or bolted connections, which are covered in fire-proof protective intumescent coating, gypsum boards, or ceilings. In these cases, the architectural and fire-protective elements should be removed in order to make the connections visible. This was the aftermath, especially from the Northridge earthquake in 1994, where damage was found in welds probable from previous earthquakes (e.g., Loma Prieta, 1989).
- The repeated damage observed from Northridge, 1995, USA, and Kobe, 1995, Japan, unveiled the vulnerability of the improperly detailed welded connections. If this zone, especially at the column face, is highly stressed, then special care should be given to the onsite executed welds, and a thorough inspection must be performed as well.
- Under certain circumstances, steel buildings provide a viable solution for rapid and safe mass construction after a devastating earthquake. This was the case with the Christchurch reconstruction in New Zealand after the strong earthquakes of 2010–2011.
- Steel structures are the leading material and structural bearing systems for one- or two-story industrial building facilities and platforms. Mainly, this is due to the reduced dead load influencing, positively, not only the seismic action but also the foundation system. However, for low-rise, medium-rise, and tall buildings, there is a choice mainly in the USA and Japan, and at a lower level in Europe and other countries. In any case, this is a viable solution respecting the sustainability and resilient mode of constructing buildings.
- The long list of vulnerabilities of steel storage pallet racking systems is unveiled, and currently, in both the USA and Europe, there is a framework to design such structures subjected to earthquakes. The research projects SEISRACKS1 and SEISRACKS2 provided proposals for the revision of the existing code; however, they are still not implemented. Open issues remain: loading protocols, experimental tests to simulate seismic actions, and more tests with regard to full racks, base connections, and beam-to-inside connections.
- Certainly, from the accumulated experience of the past 40 years, it has been demonstrated that steel structures, when properly designed and constructed, respect the performance levels of life safety and collapse prevention. However, the next challenge is to ensure the performance of the operation and immediate occupancy. Due to their inherent flexibility, this task would be addressed in the near future in order to conform resilient steel buildings to a capacity for easy repair not only of the bearing structure (this is done) but also of the non-structural components, through design, and proper constructional detailing.

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