Review Article

Inelastic Behavior of Steel Buildings in Seismic Zones

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Abstract: This paper provides general overview on the design principles of steel structures in *Seismic Zones*. In particular, seismic design of multi-storey steel structures using limit states (ultimate, serviceability and damageability) and performance based design approach is firstly discussed and the importance of steel structures is consequently highlighted; then, general concepts to be incorporated in the structural design are provided. The well-known adopted lateral load resisting systems (moment resisting and braced frames) are also criticized to highlight the *pros* and *cons* of each system. The concept of *dissipative* and *non-dissipative* Zones is given for each lateral load resisting system; and therefore aims to give useful information for the engineers and technicians involved in the design of steel structures in the seismic zones.

Keywords: Steel Structures, Seismic Design, Moment Resisting Frames, Concentric Braced Frames, Eccentric Braced Frames

1. Introduction

Structures are designed to sustain safely the applied actions that occur during the life of the building and in service with the aim to have adequate durability and sustainability. Hence, the structural design methods provided by modern building Codes endeavour for guaranteeing the acceptable safety level. These depend on the probability of occurrence of the event to be considered, such as earthquake, wind, snow, and thermal actions. In order to transfer vertical and lateral loads, resisting systems need to be defined in a structural system. Vertical elements transfer load from roof to foundation, these element might transfer only gravity loads or both gravity and lateral loads. *Vertical loading* include, the self-weight of the structural and non-structural elements, live loads (imposed) and snow loads etc. While *lateral loads* are attributed to either wind or seismic actions. Wind actions are not the subject of discussion in this paper; nevertheless, these should be evaluated following building Codes provisions for the desire locality. Earthquake is a natural phenomenon and can represent a huge threat for any building which has not been properly designed and erected. With regard to the seismic loadings, these are evaluated according to the Code prescribe rules.

Structural requirements are associated to the way in which the acting forces are resisted and transferred. Two Limit States are normally considered for building design, which are: i) the Ultimate Limit State (ULS) for strength design and, ii) the Serviceability Limit State (SLS) for excessive deflection, cracking, vibration and comfort to the occupants under service loads.

Most importantly, the ULS are satisfied when the structural system has the resistance, stability and energy dissipation capacities to support the actions in which be considered according to the building codes.

The concept of designing sacrificial members is to dissipate the seismic energy "*dissipative members*"; while preserving the integrity of other main components "*non-dissipative members*" is known as the *structural fuse concept* [1, 2]. The term "structural fuse" has been defined by Roeder and Popov [3], where they used it as a part of their proposed eccentrically braced frame concept for steel frames. In much subsequent research such as by Derecho et al. [4], a similar capacity design concept was used with designated plastic hinging of the beams to be structural fuses [5, 6]. For achieving global ductility and avoiding soft story mechanisms "*weak energy dissipation*" of a structural system, dissipative and non-dissipative zones are generally defined by the Codes.

These are in which the non-dissipative zones should remain in the elastic field and the dissipative ones should experience large inelastic deformation. To control such a global structural behaviour, Codes give the so-called criterion of capacity design (firstly initiated in 1980's in New Zealand); where non-dissipative members are design for comparatively higher seismic forces than dissipative members that are kept at such locations and will fail before the brittle members and subsequently will protect non-ductile elements by overstressing.

In order to satisfy the damageability limit state (deformability criteria), it is necessary to restrict:

- The deflection of the beams under vertical loads; and

- The storey displacements under horizontal actions. The latter aims to satisfy two deformation criteria, which are i.e.: second order effects and interstorey drifts limitations given by the codes, especially in presence of seismic action.

In fact, if we focus the attention on seismic design, damage limitation is achieved by limiting the overall deformations (lateral displacements) of the building to levels acceptable for the integrity of all its parts (including non-structural elements). More specifically, *interstorey drift ratios* (defined as the difference between the mean lateral displacements of adjacent storeys divided by the interstorey height) are limited for the structural design in seismic zones.

The most advanced building Codes have adopted the use of the *performance based design* (PBD) approach, being a useful tool for designing structures. PBD takes into account different performance levels as shown in Table 1, accomplished for seismic demands having different return periods. Depending on the importance of the structure, performance levels are related to earthquakes characterized by: i) frequent, ii) occasional, iii) rare, and iv) very rare occurrences. Further, for the evaluation of seismic loads, building Codes give the response spectrum (as shown in Figure 1), which is a plot of the steady-state response (displacement, velocity or acceleration) of a structure on a specific soil of varying natural period that is forced into motion by the seismic excitation.
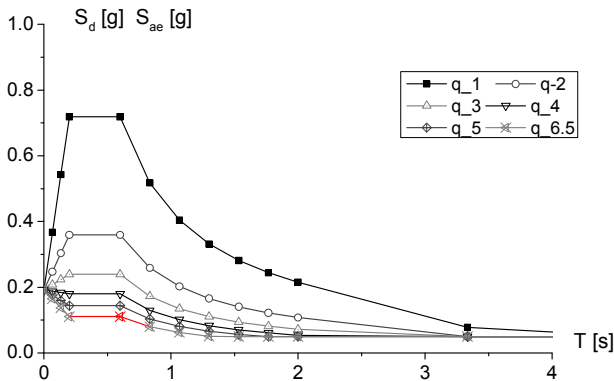

Figure 1. *Eurocode 8 design spectra for various q factors.*

Normally, elastic spectrum is provided by Codes [7-9]; whereas, the design spectrum is then obtained by reducing it with a specified value (as shown in Figure 1 by their values equal to 1, 2, 3, 4, 5 and 6.5) that depends on the lateral load

resisting system known as response modification factor or behaviour factor. Such factor is used to account for the inelastic behaviour of the structural system. It is defined as the ratio of the "*elastic strength demand*" to the "*design strength demand*"; or simply, it is the ratio of the "*elastically induced forces*" to the "*prescribed design forces*" at the ultimate limit state under the specified ground motion during an earthquake. Further, it is used to reduce the ultimate seismic event and the amount of reduction depends on the overstrength and ductility of the structure. In this way, the *design peak ground acceleration* (PGA), denoted as S_d, is normally obtained, which is used for evaluating the base shear of the structure by multiplying it with the *mass of the structure* (m). Hence, the horizontal seismic action is obtained and is basically described by two orthogonal components, namely in the transversal and longitudinal directions. The third component which is usually the vertical one is described by a different elastic response spectrum when the structure is irregular, or the vertical loads are amplified by certain factors.

$$V = m. \, S_d \qquad (1)$$

The design base shear (V), as shown by the basic equation (1) which is derived from the newton second law of motion, is used for estimating the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. The `calculations of base shear (V) depend on: soil conditions at the site, proximity to potential sources of seismic activity (such as geological faults), probability of significant seismic ground motion, the level of ductility and over-strength associated with various structural configurations, the total weight of the structure, and the fundamental (natural) period of vibration of the structure when subjected to dynamic loading.

In seismic design, it is very important to assess the ability of a structure to develop and maintain bearing resistance in the inelastic range. A measure of this ability is ductility, which can be observed in the steel itself, in a structural element, or to a whole building structure. In modern Codes, the concept of energy dissipation capacity of steel structures is normally prescribed by means of *three ductility classes*, which are:

i) Fully Ductile Structures or High Ductility Demands: These are capable of sustaining or required to sustain large inelastic deformations without a significant loss of strength or reduction in energy dissipating capacity. Capacity design procedures shall be used.

ii) Limited Ductility Structures or Medium Ductility Demand: These are capable of sustaining small to moderate inelastic deformations without a significant loss of strength or stiffness. Capacity design procedures shall be used with modifications to design and detailing requirements.

iii) Elastic Structures: These structures are expected to respond elastically to large earthquake motions and do not require any special detailing requirements. Elastic detailing based on the Code requirements will ensure that most steel structures will not fail even with minor excursions into the post elastic range.

Table 1. Matrix of performance objectives.

Occurrence probability	Fully Operation	Operational	Life Safety	Near Collapse
Frequent earthquake	Basic facilities		Unacceptable Performance	
Occasional earthquake	Essential facilities	Basic facilities		
Rare earthquake	Critical facilities	Essential facilities	Basic facilities	
Very Rare earthquake	Critical facilities	Critical facilities	Essential facilities	Basic facilities

2. Lateral Load Resisting Systems in Steel

In many ways structural steel represent itself as an ideal material for the design of earthquake-resistant structures. It has distinct capabilities compared to other construction materials such as reinforced and pre-stressed concrete, timber, brickwork etc. It is strong, lightweight, ductile, and tough, capable of dissipating extensive amount of energy through yielding when stressed into the inelastic range. Given the seismic design philosophy of present building Codes, which is to rely on the inherent capability of structures to undergo inelastic deformation without failure, these are exactly the properties of steel as desired for seismic resistance.

For a building, the gravity loads are normally fixed and estimated easily, their transfer can be made through the foundations with the help of slab having sheeting etc., to the beams and then to columns. The consideration, the estimation, and the so-called lateral load resisting are of extreme importance in a structure to resist the horizontal acting forces, for examples, seismic and wind actions. In low rise steel multi-storey buildings when are constructed in seismic zones, during the initial planning stage, the type of lateral load resisting system(s) to be used in the building should be decided. In the case of steel structures, the capability to resist the lateral actions (like seismic) are achieved with the help of several *Lateral Load Resisting Systems* (LLRS). These LLRS correspond to diverse combinations of the deformability and strength and can be conclude with the required ductility characteristics of the system. Therefore, different types of earthquake resistant steel structures may be conceived depending on the selected load carrying mechanism. The most conventional types of earthquake resistant steel structures are: 1) The Moment Resisting Frames (MRFs) and is called Un-braced Frames; and 2) Braced Frames: a) The Concentrically Braced Frames (CBFs) and b) The Eccentrically Braced Frames (EBFs).

2.1. Unbraced Frames

Moment Resisting Frames (MRFs) represent itself as an ideal case for steel *Unbraced Frames*. One of the main concern of steel MRFs is their high susceptibility to large lateral displacements (lateral stiffness) during severe earthquakes, therefore needs special attention while designing, in order to limit interstorey drift so that the issues due to geometric nonlinearities and brittle fracture of beam-to-column connections are mitigated and therefore excessive damage to non-structural elements is avoided. Therefore as an alternative many practical and economic

issues, engineers are increasingly turning to the use of concentrically braced steel frames as a lateral load resisting system. However, frequent damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico, 1989 Loma Prieta, 1994 Northridge, and 1995 Hyogo-ken Nanbu earthquakes, has raised concerns about the ultimate deformation capacity of this class of structure.

In order to resist seismic events, steel moment resisting frame (MRF) may behave elastically or in elastically depending on the intensity and severity of the seismic event. For instance, in the case of a moderate to low seismic event, MRF should be undamaged without permanent deformation and thus remain in the elastic state. Contrary, in the case of a heavy seismic event, MRF is expected to exceed the elastic limit thus deform in elastically allowing for the dissipation of energy hysterically. Once the MRF has become inelastic, the lateral stiffness is reduced. If yielding has not occurred, the lateral force resisted by the MRF will continue to increase proportional to the ground acceleration. Moment resisting frames (MRFs), Figure 2, are made of members that are rigidly connected each other. This guarantees evident architectural advantages, since the meshes of the frame are not occupied by structural elements, leading to extreme functional flexibility. The capability of resisting to the horizontal actions is essentially based on the flexural regime occurring in the beams and the columns. In general, the initial lateral stiffness of steel MRFs is quite weak, and therefore they have excessive deformability, with consequent potential damage to non-structural elements in case of low-intensity earthquakes. On the other side, with regard to the energy dissipation capacity and thus due to their high inelastic capabilities, steel MRFs are in principle able to exhibit very good performances under severe earthquakes. Ductile collapse behaviour in MRFs is achieved if the energy dissipation takes place through cycles of plastic deformations in bending at the ends of the beams. These areas are interested by the so called *plastic hinges*, and the most convenient collapse distribution in a steel MRF foresees plastic hinges at the beam ends and at the column bases. This leads to a global mechanism, which maximizes the amount of dissipated energy and, in addition, causes a local ductility demand lower than that corresponding to other collapse mechanisms, such as those with plastic hinges at the ends of the columns. In the perspective of the capacity design, it is clear that the *beams* are the *dissipative elements*, whereas the *connections* and the *columns* are the *non-dissipative elements*, and they must be designed accordingly. In particular, the conception and realization of the *beam-to-column* nodes is rather delicate, since they must be over-resistant with respect to the connected beams and must, at least in principle, prevent any relative rotation

between the connected members [10, 11].

Rigid frames are used when the architectural design will not allow a braced frame to be used. This type of lateral resisting system generally does not have the initial cost savings as a braced frame system but may be better suited for specific types of buildings. Figures 2a and 2b show a floor plan and building line elevation of a rigid frame system. Connections between the beam/girder and column typically consist of a shear connection for the gravity loads on the member in combination with a field welded flange to column flange connection. Column stiffener plates may be required based on the forces transferred and column size. It must be noted that this type of joint requires all vertical utility ductwork and piping to be free and clear of the column and beam/girder flanges. Coping of the beam/girder flanges to allow passage of piping or other utilities is usually not acceptable and must be brought to the attention of the structural engineer as soon as possible [12-14].
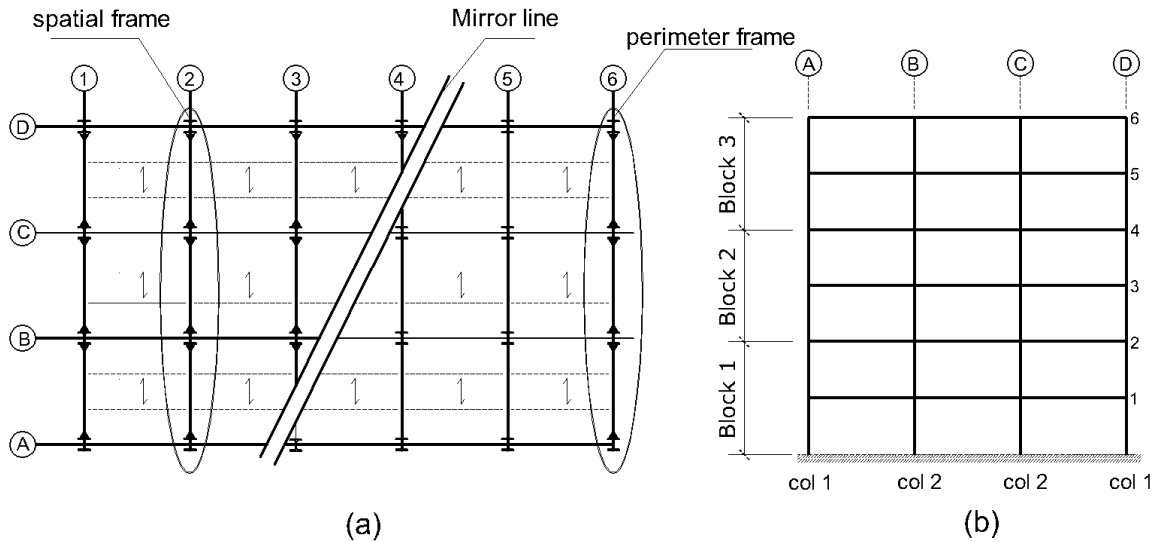

Figure 2. MRFs configurations, (a) Spatial and perimeter configurations in plan, (b) elevation representations.

A frame is considered rigid where its beam to column connections have sufficient rigidity to hold virtually unchanged the original angles between intersecting members. In other words, a *moment resisting frame* is a structure that utilizes moment resisting connections between columns and girders throughout its perimeter to resist the lateral loads applied. In this system, lateral loads are resisted primarily by the rigid frame action that is, by the development of shear forces and bending moments in the frame members and joints. In contrary, *rigid frames* are identified by the lack of pinned joints within the frame. The joints are rigid as shown in the Figure 2 and resist rotation. They may be supported by pins or fixed supports and they are typically statically indeterminate.

Moment resisting frame structures are characteristic of early skyscrapers where 3 dimensional structural analyses were still in its infancy. The repetitive pattern with small cross sectional changes from floor to floor allows simple construction. Moment frames also allow unobstructed bays that allows for flexibility in spatial programming and locations of openings. This feature is much desired by architects seeking flexibility in their design and also helps to introduce as much natural light into the space as possible.

As far as the behaviour of rigid frames is concerned, the relation between the joints has to be maintained, but the whole joint can rotate as shown in the Figure 3.

The amount of rotation and distribution of moment depend on the stiffness (EI/L) of the members connected in the joint. End restraints on columns reduce the effective length, allowing columns to be more slender. Because of the rigid joints, deflections and moments in beams are reduced as well. Since *moment resisting frame* resists lateral loads by bending, it is the most ductile lateral load resisting system used in tall buildings.

For dissipation of seismic events, modern seismic Codes give the criteria of reducing the seismic forces by a specific amount with the use of a reducing factor, commonly termed as *response modification factor*. The dissipative zones are therefore designed with the reduced forces and allow them to remain in the inelastic state during a seismic excitation. Contrary to the dissipative zones, such as, beams in MRFs, the internal forces of the non-dissipative zones are increased by over-strength factor which represent the ratio of the elastic base shear to the design base shear.
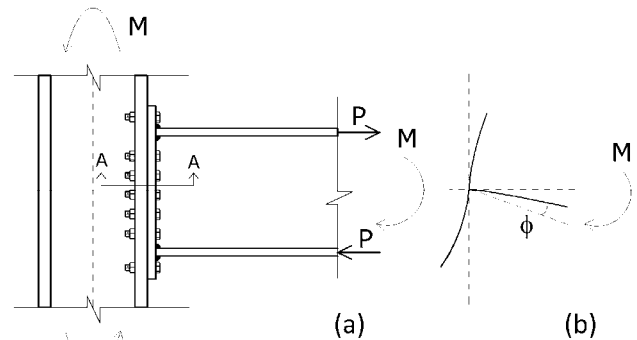

Figure 3. Rotation of a joint in moment resisting frame.

2.2. Braced Frames

Structural Steel has the property where braces can be used very efficiently compared to other structural materials. Braced Frame is a common system employed to resist the significant lateral loads where when tall structures are exceptionally subjected to brace the frames, bracing can occur within a single bay inside the internal bays or along the external bays or it can span the entire face of a structure on perimeter.

The advantages of braced frames from a structural engineering standpoint are enormous. Braced frames carry the lateral forces in an axial manner with tension and compression (through the diagonal elements) rather than through the bending of elements which is quite inefficient from flexibility point of view. The separation of the lateral system from the gravity system being concentratic at some points gives further advantages during the design phase. This allows the lateral system to be designed separately from the gravity system therefore permits for repetition in floor systems and column sections. With minimal frame action and mostly axial deformation, minimal moments in the columns and girders result from the applied lateral loads compared to a moment frame. This in turn leads to cheaper girder-column connections. Among all *Bracing Systems*, two of them are normally widely used by the designers, namely, the *cross Concentric Bracings System* and the *Eccentric Bracing System*. These are explained further in detail as follows:

Concentrically Braced Frames

Steel Concentrically Braced Frames (CBFs) are assumed and recommended to be strong, stiff and ductile. The quality of the seismic response of CBF is determined by the performance of the brace. For achieving a good performance from a CBF, the brace must behave as a structural fuse thus should fail prior to any other component of the frame. This is important because although the frame may sustain significant damage during an earthquake, it is expected to remain stable and the building must be capable of resisting gravity loads and of withstanding aftershocks without collapse.

Concentrically Braced Frames (CBFs) are made of structural members which from a theoretical point of view, may be connected each other by means of simple flexural hinges. The resistance to horizontal forces, such as wind or seismic, is achieved by means of braces, which essentially work in tension or compression.
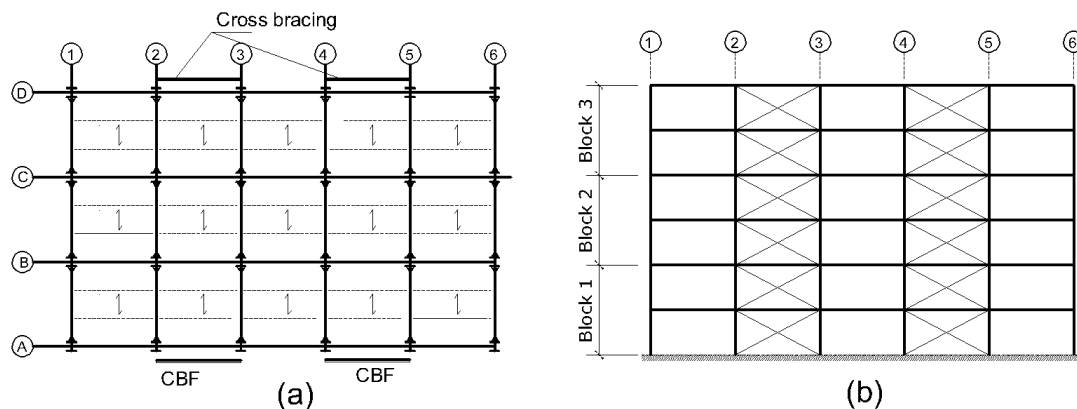

Figure 4. Concentric bracing configurations, (a) bracing positions in plan, (b) elevation representations.

From an architectural point of view the meshes of the frame which are occupied by the braces cannot be used for openings with consequent functional flexibility reduction. The initial lateral stiffness of CBFs is generally high due to the axial stiffness of the braces. On the other hand, the capacity of dissipating the input seismic energy is quite poor; it is being based on the plasticization of braces in tension. The effectiveness of this dissipative mechanism is reduced cycle by cycle, due to the degradation caused by the repeated buckling undergone when braces are subjected to compression. From the capacity design point of view, the dissipative elements are the braces, whereas the connections (the beams and the columns) must be over-resistant, behaving in the elastic field up to the failure of the braces, therefore are considered as non-dissipative zones [15].

Perhaps, the most common type of braced frame is the concentric cross brace. It is important to establish early on in the development of any project the location of braced bays. Connections for this type of bracing are concentrated at the beam to column joints. Figure 4 illustrates a typical beam-to-column joint for a cross-braced frame. For taller buildings, usually over two or three stories, these connections could become large enough to minimize the available space directly adjacent to the column and below the beam. This restricted space may have an effect on the mechanical and plumbing distribution as well as any architectural soffit details. The structural engineer needs to be able to provide this type of information to the architect to avoid potentially costly field revisions during construction. Bracing members are typically designed as tension only members. With this design approach, only half of the members are active when the lateral loads are applied. The adjacent member within the same panel is considered to contribute no compressive strength. Utilizing tension only members makes very efficient use of the structural steel shape and will result in using the smallest members. Without full consideration of a specific bay size and amount and location of the bracing, a generalized range of sizes cannot be determined.

Cross-braced frames are composed of single span, simply connected beams and girders. Columns that are not engaged by the braced frame can be designed as gravity load only column.

Most of the bracing systems are concentric, this means that the members intersect at a node where the centroid of each member passes through the same point.

The purpose of designing such bracings requires ensuring adequate ductility (i.e. to stretch without breaking suddenly).
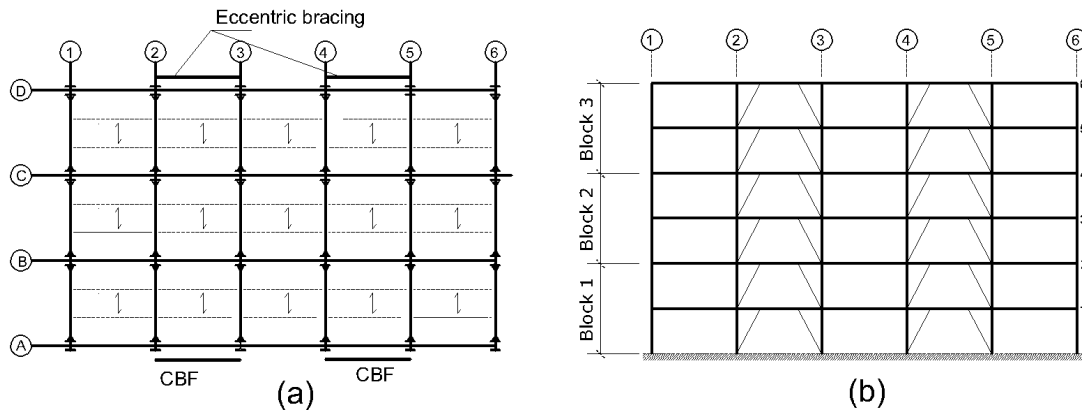
Eccentrically Braced Frames

Eccentrically Braced Frames (EBFs) are well known for their attractive combination of high elastic stiffness and superior inelastic performance characteristics. Eccentrically braced frames are very similar to frames with Chevron bracing. In both systems, the general configuration is an inverted V shape with a connection between the brace and the column and a connection at the beam/girder at the next level up. However, unlike the Chevron-braced frame, the brace members work points intersecting at the same point on the beam/girder for the brace elements. The condition is shown in Figure 5.

Figure 5. Eccentric bracing configuration, (a) bracing positions in plan, (b) elevation representations.
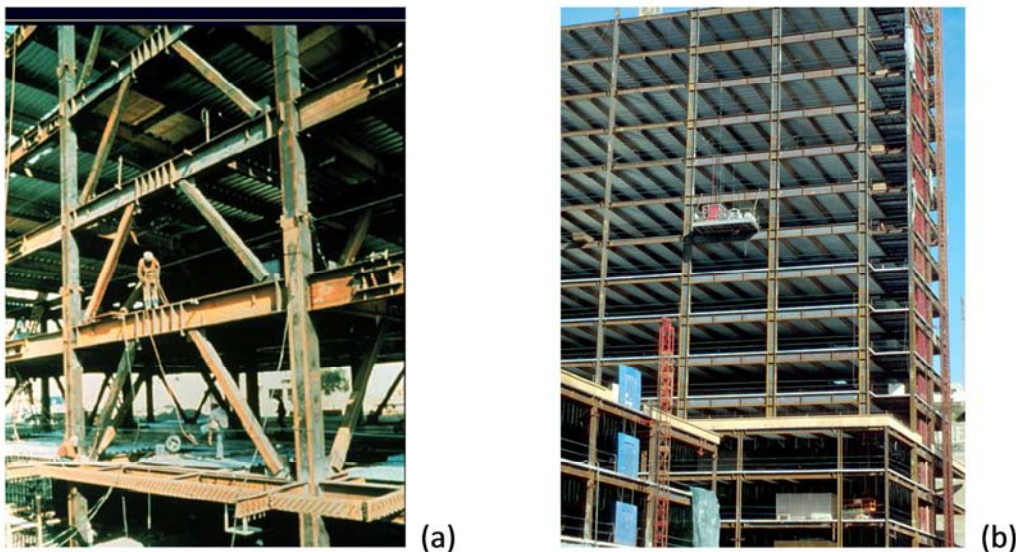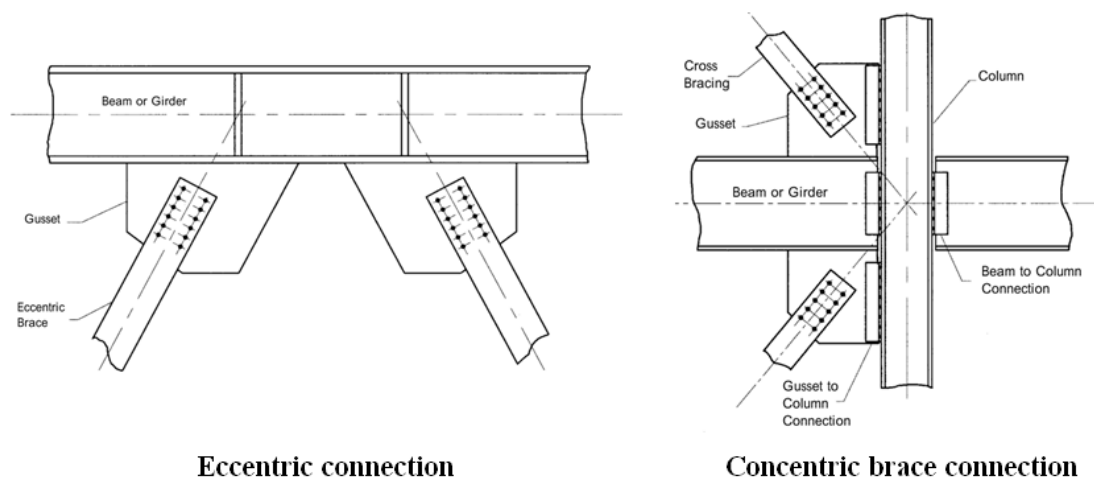

Figure 6. Eccentrically Braced Frame (a) and Moment Resisting Frame (b).

This type of bracing is commonly used in seismic regions requiring a significant amount of ductility or energy absorption characteristics within the structure. The beam/girder element between the work points of the bracing member is designed as link element and assists the system in resisting lateral loads caused by seismic action.

Eccentrically braced frames are a sort of "compromise" between *moment resisting frames* (MRFs) and *concentrically braced frames* (CBFs). They are conceived for combining the advantages of MRFs, in terms of ductility and energy dissipation capacity, with those of CBFs, in terms of lateral

stiffness. Also in terms of architectural flexibility, the *eccentrically braced frames* (EBFs) solution shows intermediate peculiarities. Therefore, the most attractive feature of EBFs for seismic-resistant design is their high stiffness combined with excellent ductility and energy-dissipation capacity. The braces in EBFs deliver the high elastic stiffness characteristic of CBFs, permitting Code drift requirements to be met economically, and in addition, under severe earthquake excitation, properly designed and detailed EBFs provide the ductility and energy dissipation capacity characteristic of MRFs [3, 16, 17].

Figure 7. *Eccentric and concentric braces with typical brace to beam connection.*

The basic idea is to endow moment resisting frames with appropriate braced members and thus reduce the lateral deformability of the frame. At the same time, since at least one end of the braces is connected to the beams, a part of these, usually called "*link*", is devoted to the dissipation of the input energy, by yielding in shear and/or in flexure. In this way, the stiffness and ductility properties can be in principle adequately calibrated, so leading towards optimal structural solutions. The performances of the structure are strongly dependent on the behaviour of the links, which require particular care in phase of design. The connections dealt with in this work are conceived for steel moment resisting frames, due to their unexpected non-ductile behaviour shown during the 1994 Northridge in California, USA earthquake. Consequently, focus was made on steel MRFs with particular care to the aspects related to the nodal constructional details and to the research developed subsequently [18].

3. Conclusions

The paper has dealt with the examination of the common lateral load resisting systems employed in the design of steel structures. Three conventional seismic load resisting systems were discussed, which are: steel moment resisting frames, eccentric braced frames and cross-concentric braced frames, of which braced frames are much rigid compared to the moment resisting frames and therefore are mostly used in steel structures. Braced frames has the further advantage of simple connections compared to moment resisting frames where connection challenge its design and its fabrication when bolted and welded. The concept of dissipative and non-dissipative zones were given as relied by the modern building Codes. The use of capacity design approach for steel building structures are believed to be of significant importance with regard to the use of the seismic links for fulfilling the ductility requirements of the structures to conclude with an acceptable performance. The paper gives useful basic information for the Technicians involved in the design of steel structures.

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