informes de la construcción

Informes de la Construcción 77 (577)
January-March 2025, 6931
ISSN-L: 0020-0883, eISSN: 1988-3234
https://doi.org/10.3989/ic.6931

Assessment of brace types of performance and optimal bracing configuration on steel frames structures

Evaluación del rendimiento de los tipos de arriostramiento y configuración óptima de arriostramiento en estructuras de pórticos de acero

David Dominguez-Santos Universidad de Talca. Chile. ddominguez@utalca.cl.
Pedro Muñoz Velasco Universidad Internacional de La Rioja. España. pedro.munoz@unir.net
Universidad Autónoma de Chile. Chile. pmunozv@uautonoma.cl

Autor de contacto: ddominguez@utalca.cl

ABSTRACT

The structural behavior of steel has deficiencies such as buckling, due to the thinness of the steel profiles in relation to their length. Commonly, this problem is solved by introducing bracing elements and shear walls. However, this effect also depends on the location of the braces in the structures. In this work, the optimal position of the braces in an eight-story frame is evaluated, analyzing 16 different situations in terms of resistance, ductility, and displacements, using static (Push-over) and dynamic (Time-history) methods with the registration from Lorca. The conclusions obtained in this work have been to determine the importance of ductile solutions in the structures and the use of bracing on all floors of the porch with symmetrical arrangements in height.

Keywords: Construction; braces; steel frame; building design; optimization; structure.

RESUMEN

El comportamiento estructural del acero tiene deficiencias como el pandeo, debido a la delgadez de los perfiles de acero en relación con su longitud. Comúnmente, este problema se soluciona introduciendo elementos de arriostramiento y muros de corte. Sin embargo, este efecto también depende de la ubicación de las riostras en las estructuras. En este trabajo, se evalúa la posición óptima de las riostras en un pórtico de ocho pisos, analizando 16 situaciones diferentes en términos de resistencia, ductilidad y desplazamientos, utilizando métodos estáticos (Push-over) y dinámicos (Tiempo-historia) con el registro de Lorca. Las conclusiones obtenidas en este trabajo han sido determinar la importancia de las soluciones dúctiles en las estructuras y el uso de arriostramientos en todos los pisos del pórtico con disposiciones simétricas en altura.

Palabras clave: *Construcción; riostras; pórtico de acero; diseño de edificio; optimización, estructura.*

Cómo citar este artículo/Citation: David Dominguez-Santos, Pedro M. Velasco (2025). Assessment of brace types of performance and optimal bracing configuration on steel frames structures. Informes de la Construcción, 77 (577): 6931. https://doi.org/10.3989/ic.6931

Recibido/*Received:* 13/03/2024
Aceptado/*Accepted*: 03/03/2025
Publicado on-line/*Published on-line:* 24/04/2025

1. INTRODUCCIÓN

Several catastrophes caused by earthquakes have recently taken place (e.g. in Haiti, Chile and Japan) and it has been revealed that both the building processes and construction materials are key points, which determine the success of the mechanical response, even in the case of low or moderate seismic forces (1, 2). By using lighter materials, the overall structural weight is reduced. This tends to increase the ductile behaviour of buildings, which is a desirable characteristic regarding the seismic forces. In contrast, throughout history, the optimisation of structures has been a fundamental issue in construction (3). The search for better mechanical response has led to the development of safe and economical structures that must be further improved with the aim of reducing both environmental impact and building costs. At this point, steel may be highlighted as one of the most used building materials due to its properties (e.g. ductile behaviour, good adherence and compatibility with other materials) and the feasibility for assembling and fitting to other building elements (4).

The bending and tensile strengths are also of great importance. For instance, horizontal structural elements (beams) made by steel profiles can achieve a larger distance between columns than other construction materials, even timber frames. However, steel structures possess two major issues.

First, steel is generally highly vulnerable to fire (5, 6) and corrosion (7, 8). This issue has been faced by improving the chemical formulation of steel, by improving construction techniques and by applying coating materials (9).

Second, the thickness of the steel profiles must be considered. Because of the high density and the aforementioned steel properties, the profile cross section tends to be reduced.

In spite of this, better performance is found in the case of beams, since columns are commonly highly affected by buckling. In addition, horizontal loads (e.g. seismic forces, wind, water and so on) highly increase this effect (10, 11).

Hence, in order to solve this issue, braces or infill walls are commonly used for complying with the technical codes in force. However, with the aim of choosing the most feasible and economical solution, this study only considers the assessment of braces. Furthermore, from the available braces, the most economical have been selected for assessing the most suitable location within the building frame structure, i.e., simple braces (SBs). Obviously, prior to such a study, a comparison between chevron braces (CBs), SBs, Saint Andrew's cross (SAC) and a bare frame (i.e. without braces) was conducted in order to determine the differences among such methods. It is concluded that no significant improvement can be highlighted when economical, scheduling and technical values are considered.

Despite several authors have improved building structure performance by paying attention to materials properties (12), there are few studies related to the assessment of the location of braces (13). Besides, these papers have been conducted for reinforced concrete rather than for steel structures. Among the studies related to steel structures, some authors have paying attention to the optimal number of braces and the most effective location (14). It was observed that the angle of about 45 degrees, which is typically used for tall buildings in practice does not guarantee minimum shear lag effect on structure. Besides, the structural stiffness, strength, and ductility can be enhanced simultaneously when the brace volume is appropriate but out of this value the ductility may be highly reduced (15, 16). Thus, this paper aims to provide a useful guide for steel building designers when lateral loads are of importance (17).

The paper is organised as follows. The methodology section shows the building model and the carried out assumptions and calculations. It also shows the different cases to be analysed. Afterward, within the structural analysis section, the results carried out by the non-linear static performance analysis (push-over) are given and discussed by considering the regulatory standards. Finally, the conclusions summarise the major findings.

2. METHODOLOGY

2.1. Frame design

The frame structure design is based on the most extended one across Latin America and Europe, i.e., regular geometries in elevation and plan (18-20). These structures are characterised by rapid execution times and few required auxiliary facilities for mounting. 8-stories frames with a height of 3 m were formed by four bays, each of 5 m. The dimensions of the columns and beams were designed by following the requirements established by the European Standard Eurocode (EC-3) (21) for structural steel and the design criteria established by the Seismic-Resistant Standard, i.e., Eurocode (EC-8) (22), since seismic loads are the most harmful horizontal loads regarding the slenderness of columns. In addition, it must be considered that these standards are accepted by most European members. Consequently, the obtained results can be considered as representative for a significant percentage of buildings in Europe and even parts of Latin America. The choice of this model in relation to this structural typology, the number of heights and the measurements between spans, is due to the most common construction characteristics of existing buildings in cities in places with medium and low seismicity.

The combination of loads was determined by EC-8. A value of 2 kNm^{-2} was set for life loads in all rooms in accordance with the residential, administrative buildings and small shops category (i.e. Type II). In addition, a 2 kNm^{-2} load was considered in the upper floor (roof) in order to consider maintenance uses. Due to the high brittle behaviour of glasses, the calculations do not consider the collaboration of windows.

The material used in the different structural sections (columns, beams and braces) is S-355 high strength steel (with an elastic limit of 3,550 Kg cm^{-2}). Regarding the profile sections, the beams are kept constant (i.e. IPE 400), while the columns are HEB profiles that change every two floors (i.e. HEB 400 on the two lower floors, HEB 360 on floors 3 and 4, HEB 340 on floors 5 and 6 and HEB 320 on the two upper floors) (Figure 1).

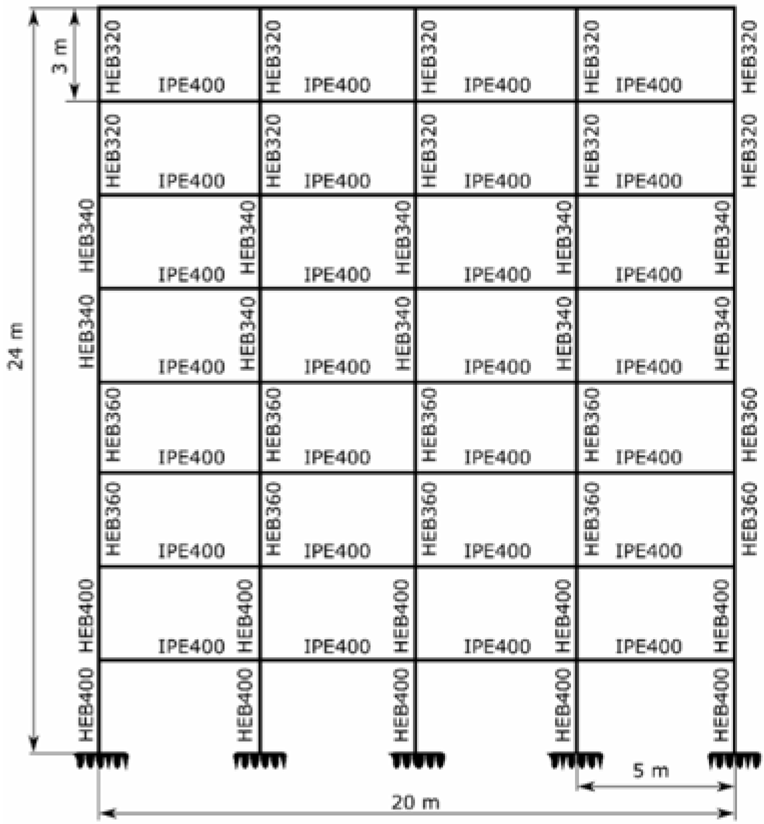


Figure 1. Building frame structure scheme

Although this scheme (Figure 1) is widely extended, it is also noticeable that it does not properly work under horizontal loads, mainly due to the slenderness of columns. Thus, as aforementioned, the use of braces is quite mandatory. The choice of brace profile (i.e. UPN 200) aimed to ease the assembly and its effectiveness to limit the buckling effect (23). The profiles of the braces used have been calculated considering that they were rigid enough so that they would not buckle. The maximum axial critical buckling load, considering the Euler critical load $P_{cr} = \pi^2 n^2 EI/(kL)^2$ for these braces it is 233T, considering the inertia of the x axis (I_x = 1910 cm^4) of the UPN 200 profile with embedded joints. In the dynamic analysis, the maximum axial values of the braces will be shown (Table 6).

Initially, the effect of each kind of brace (i.e. SB, CB and SAC) was assessed by solving the cases shown in Figure 2. The most suitable location for SB was then determined by comparing the push-over analysis results of each case shown in Figure 3. Hence, 16 types of SB locations were selected (Table 1), for a total of 16 diagonal bars between stories (Figure 3). In spite of several configurations being certainly feasible, with the aim of showing a realistic proposal, only one brace per story and bay is considered.

For the analysis carried out in this work, a bare frame model has been used (without any type of bracing), to which braces with different locations on the frames have been added. The purpose of this is to determine the optimal location of the braces, establishing a comparison of the structural behavior of the frames using different arrangements of the braces.

2.2. Structural analysis

For the analysis performance, two structural software were used, namely, Robot by Autodesk® (24) and Seismostruct v.18. by Seismosoft® (25). Both programs have been used to corroborate the results obtained in this research, not being relevant in the conclusions obtained in this research, due to the small differences obtained in the results. Hence, in order to compare the structural performance of different frames, the basal shear and the maximum displacements on the upper floor of the different frames were assessed. These values were compared with the displacements and basal shear obtained from the push-over analysis by increasing the horizontal loads.

 Informes de la Construcción, Vol. 77, 577, 6931, enero-marzo 2025. ISSN-L: 0020-0883. https://doi.org/10.3989/ic.6931

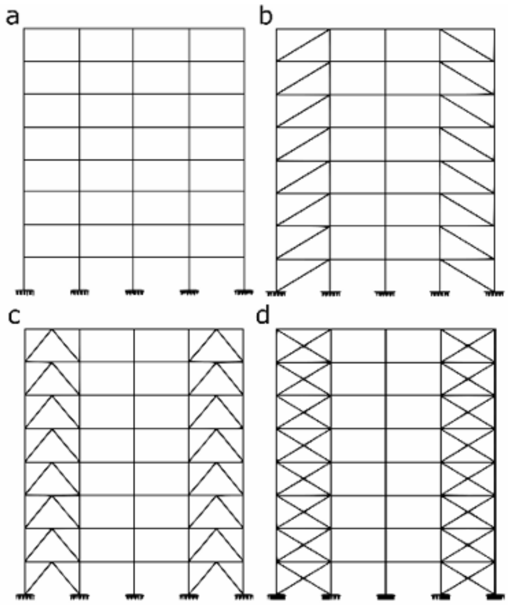


Figure 2. Type of braces frames: a) Bare frames; b) SB; c) CB and d) SAC

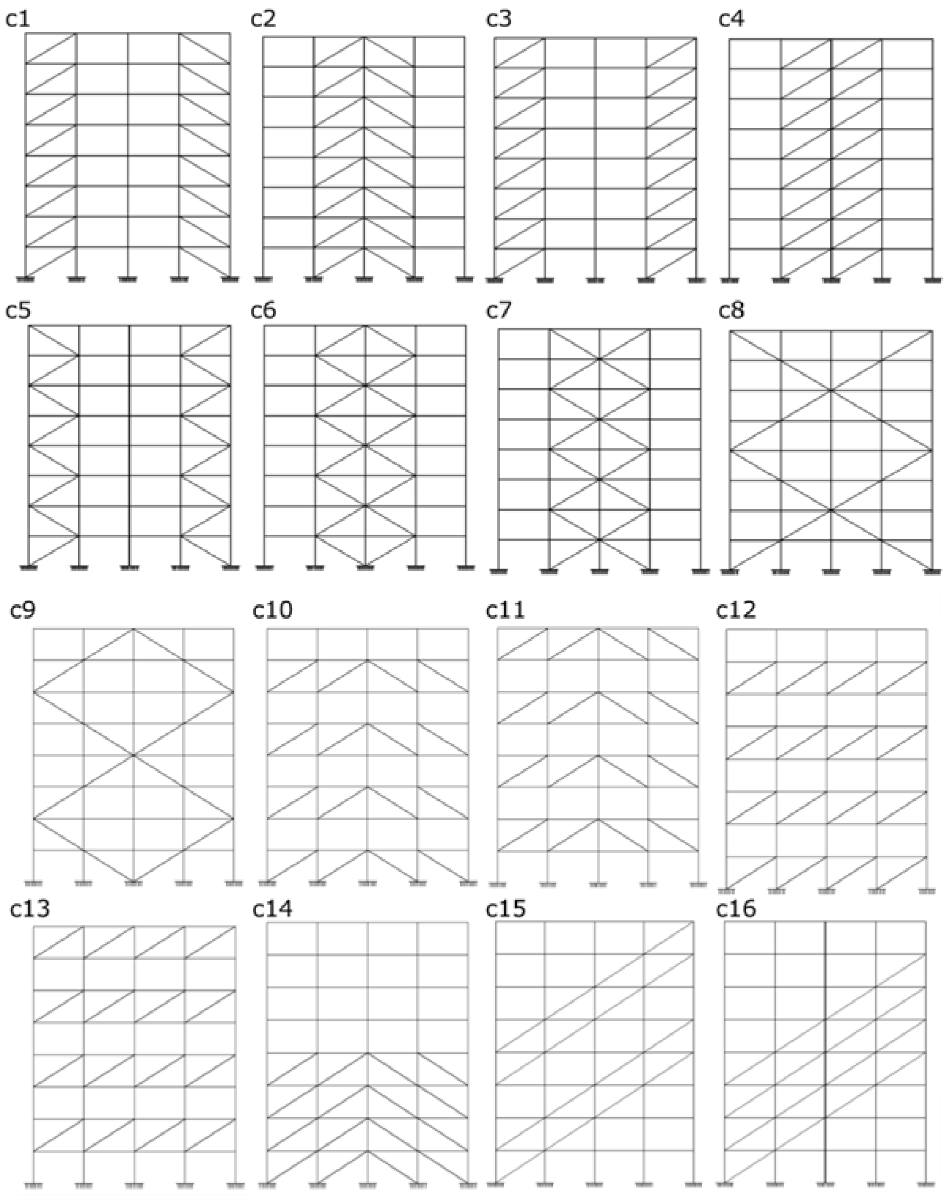


Figure 3. Analyzed cases by brace configuration, for simple bracing system

Table 1. Codes for considered cases

ID	Case description[1]
c1	Frame with braces on extremes “/\”
c2	Frame with alternating braces on extremes “/\”
c3	Frame with braces "/\" forming a double rhombus
c4	Frame with “//” braces on alternate floors last
c5	Frame with braces in center “/\”
c6	Frame with alternating braces on center “/\” forming 4 rhombuses
c7	Frame with “/\” braces on alternate lower floors
c8	Frame with “/\” braces on 4 lower floors
c9	Frame with braces on extremes “//”
c10	Frame with alternating braces on center “/\” forming 3 rhombuses
c11	Frame with “/\” braces on alternate floors last
c12	Frame with aligned “//” braces (a)
c13	Frame with braces in center “//”
c14	Frame with braces "/\" forming a double cross
c15	Frame with “//” braces on alternate lower floors
c16	Frame with aligned “//” braces (b)

[1] The “//” symbol is produced when the direction of the braces is the same along the floor, and "/\" occurs when the direction of the bars is alternated along the floor.

The analysis used in the frameworks is based on the finite bar element method (26-28). Each structural element (columns and beams) was modelled by following the prescriptions proposed in the bilinear model showed by (29) for structural steel. In particular, each element (i.e. columns, beams and braces), was represented as finite non-linear bar elements (30), where the non-linearities are concentrated in the plastic hinge (i.e. located near the joints between beams, columns and braces) at a distance equivalent to 15% of the total length of the element (31). The joints between the different structural elements are considered rigid (32). The tolerances used for displacements and rotations were of the order of 10^{-5} in both cases, with a maximum of 300 iterations.

The maximum base shear was the base value for discussing the results. This value was obtained from the sum of the seismic forces at each floor and the maximum displacements of the upper floor. For the analysis, the stress-strain curve provided by the software was considered (i.e. yield strength at 0.25% and fracture at 6%). Hence, deformations were modelled by following the classic laws of elasticity (33) and the shear capacity stated by EC-8. The results obtained by the push-over method were compared with the static analysis obtained from EC-8 and NCSE-02 (2002) (34). The value of q for the calculations in EC-8 is 3.9 corresponding to the case “Frame system, dual system, coupled wall system” for a mean ductility (DCM*) 3.0*α_u/α_1, and *“multistorey, multi-bay frames or frame equivalent dual structures"* $\alpha_u/\alpha_1 = 1.3$.

In order to compare the obtained results from EC-8 and those from NCSE-02, similar options were estimated. For this, the simplified method was also used since buildings account for less than 20-stories and the calculation options are similar to those used in EC-8. A D-type soil was selected (i.e. V_S = 30 m s^{-1}), typical of granular soils of medium compactness. The analysis was estimated with accelerations of 0.24 g and a level of normal importance in the constructions. This acceleration value has been used since it is representative of a medium seismic zone, which is the higher intensity one can find in Spain, for instance. These zones are characterized by buildings with fundamental periods of frames under *2 s or 4* T_c.

Finally, the selected spectrum was type 1 (i.e. M_s under 5.5).

In contrast, the push-over analysis was performed assuming triangular load distributions. This loading pattern has been used and selected in this research due to the efficiency and good results it has had in other research (18, 19, 35). This load pattern increases proportionally with a factor until structural collapse is reached. The yielding points were obtained by the area’s method, established by ATC 40 (36) and FEMA P-1050 (37).

The results obtained in the Push-over analysis are given in the form of response curves of the structure. These curves are represented in a Cartesian graph, where the X axis shows the displacements of the upper floor and the Y axis, the shear at the base of each frame.

The inelastic plastic hinge element “*infrmFBPH*” (25) is selected for the columns/beams/braces. The braces have been sized so that they are rigid enough so that they do not buckle for all cases. The length of these extreme elements was adjusted to around 10%-15% of the length of these elements. Non-linear static analysis is one of four analysis procedures embodied in FEMA 356/ASCE 41 and is commonly used in performance-based design approaches. For interested readers, a complete description of the method can be found in (38) and (39).

Informes de la Construcción, Vol. 77, 577, 6931, enero-marzo 2025. ISSN-L: 0020-0883. https://doi.org/10.3989/ic.6931

According to (31, 32), the joints/connections between the steel columns and beams were rigid, while the hysteretic behavior representing the stress distribution was calculated with fiber models based on the material properties and the geometry of the structural elements (discretized with 300 fibers). In the model, the loads were applied to the beams. The tolerances used for displacements and rotations were of the order of 10-5 in all cases, with a maximum number of 300 iterations. Simulating the mechanical behavior of each material in the frame elements required entering various data corresponding to the material properties. The experimental values of plasticization and breakage obtained from the capacity curve of each material. On the other hand, the unit strains corresponding to the steel failure processes used the standard values determined by Seismostruct (25): steel fracture (0.06). Furthermore, the curvature and rotations criteria were verified through the rotational capacity given in Mergos and Kappos (40) and the shear capacity was established in the EC-8. On the other hand, according to the type of material, the following values were taken:

Steel: Modulus of elasticity (2e8 kPa); Yield strength (500000 kPa); Strain hardening parameter (0.005); Transit on curve initial shape parameter (20); Fracture/buckling strain (0.20); Specific Weight (78 kN/m^3).

3. RESULTS AND DISCUSSION

The results obtained from the EC-8 approach (Table 2) and those carried out by considering NCSE-02 (Table 3) show similar values for the most unfavorable frames when no braces were installed (i.e. bare frames), which verifies the convergence of both methods.

Table 2. Results analysis with EC-8

Floor	Seismic forces (kN)	Shear strength (kN)	Displacement (m)
1st floor	2.70	97.20	0.010
2nd floor	5.40	94.50	0.030
3rd floor	8.10	89.10	0.051
4th floor	10.80	81.00	0.071
5th floor	13.50	70.20	0.090
6th floor	16.20	56.70	0.104
7th floor	18.90	40.50	0.115
8th floor	21.60	21.60	0.124

Table 3. Results analysis with NCSE-02

Floor	Seismic forces (kN)	Shear strength (kN)	Displacement (m)
1st floor	3.61	103.31	0.011
2nd floor	7.09	99.70	0.031
3rd floor	10.29	92.61	0.053
4th floor	13.10	82.32	0.074
5th floor	15.40	69.22	0.091
6th floor	17.12	53.82	0.106
7th floor	18.17	36.70	0.116
8th floor	18.53	18.53	0.124

Figure 4 shows the capacity curves of the buildings with the three types of bracing systems (i.e. CB, SAC and SB), including the frame without any bracing (i.e. bare frame). In contrast, Figure 5 shows the capacity curves of the frames with the 16 bracing locations. This Figure shows the yielding points (DY), the collapse points (DU) and the maximum displacements in the upper floor of the dynamic analyzes considering the Lorca record (Reg) and the scaled Lorca record (2º Reg). The scaled register (2nd Reg.) is the result of multiplying by 2 the accelerations of the Lorca Register (Reg.). The use of this amplification coefficient is due to the damage state of the structures, reaching the ultimate limit state in some cases.

The use of braces in the frames significantly improves the shear behavior, caused by the effects of wind or earthquakes among others. Therefore, it solves the possible buckling effects of the columns. The versatility of the use of steel in buildings with large lights and heights (industrial buildings) makes the use of these elements essential due to the significant slenderness of the columns. For example, the use of CB is a more effective solution in architectural design than the SAC. For instance, more space is available for glazing solutions. However, it is barely effective in structural behavior since SAC adds higher rigidity.

 Informes de la Construcción, Vol. 77, 577, 6931, enero-marzo 2025. ISSN-L: 0020-0883. https://doi.org/10.3989/ic.6931

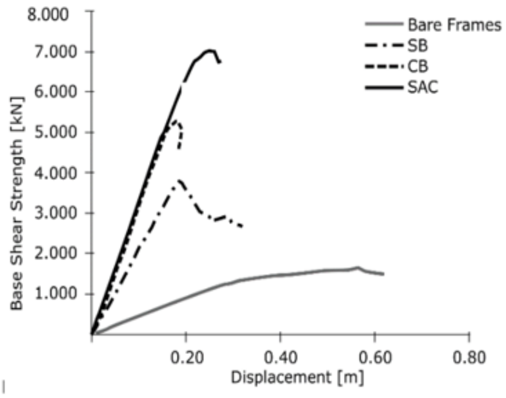
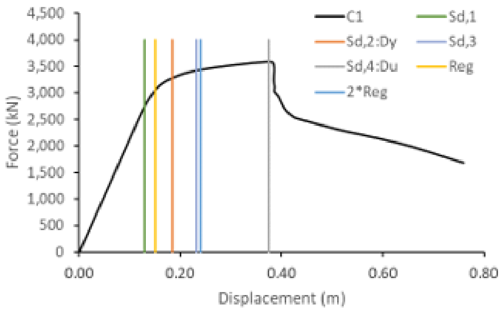
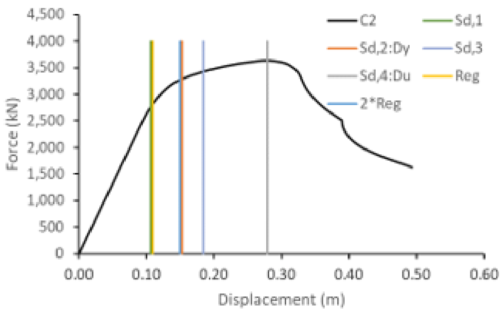


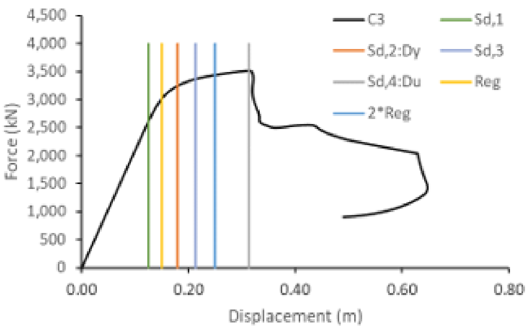
Figure 4. Frame capacity curves



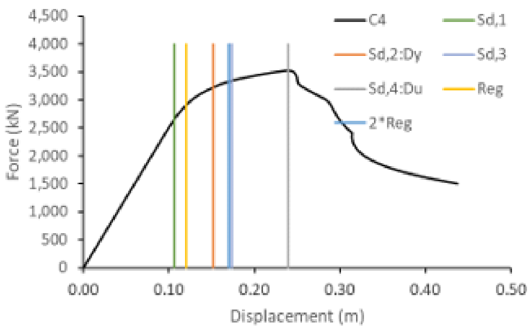
C1



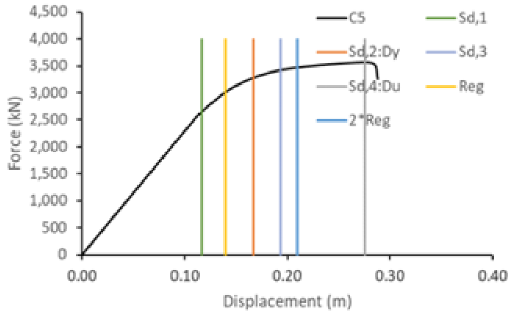
C2



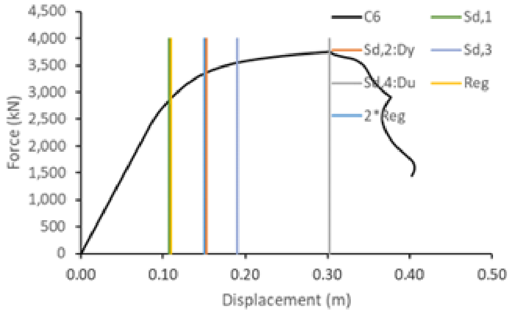
C3



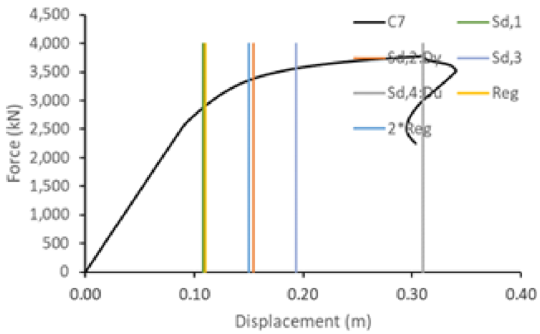
C4



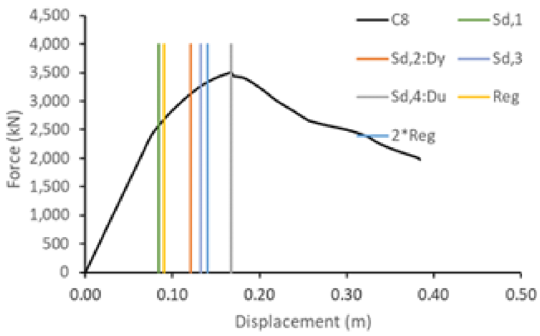
C5



C6



C7



C8

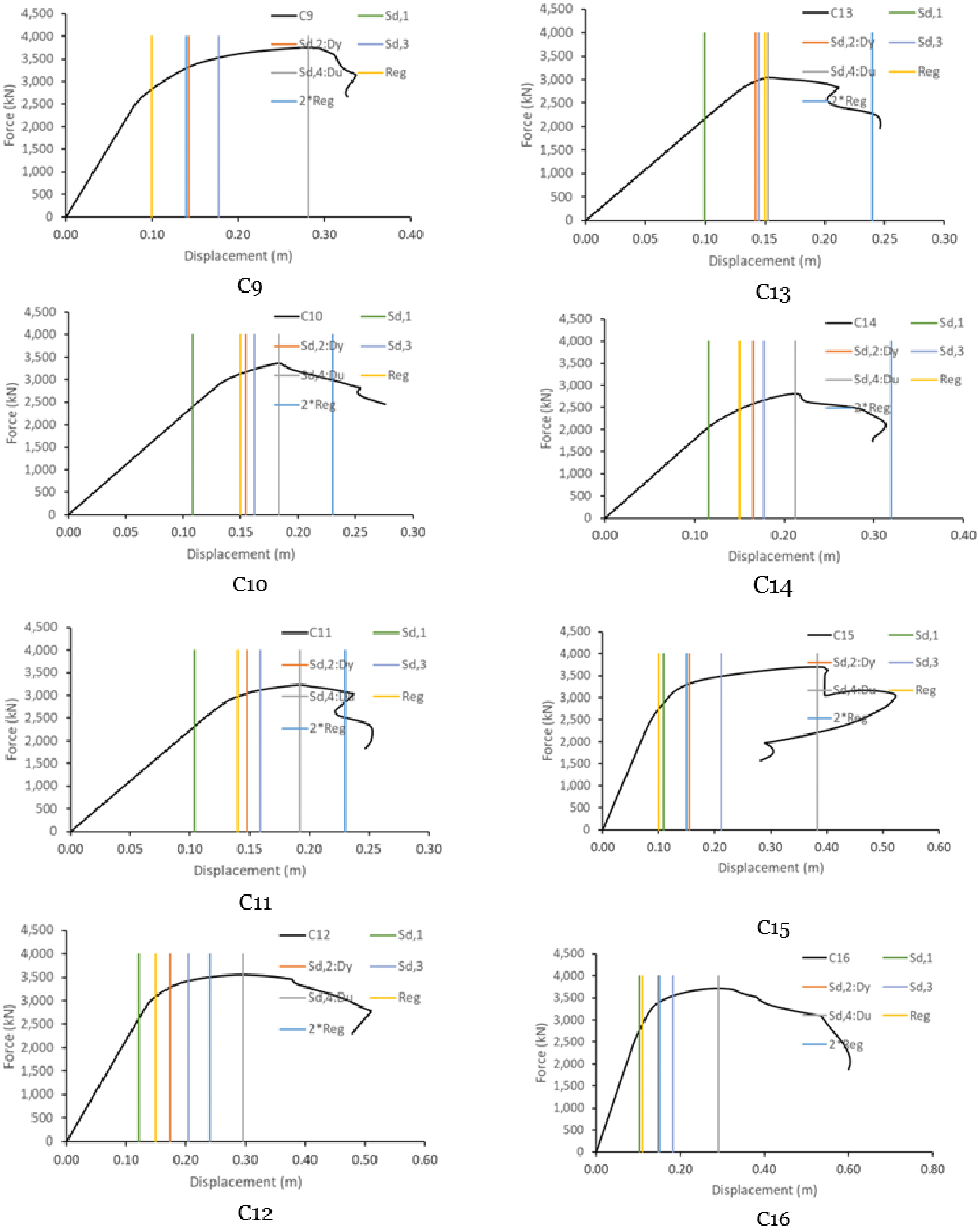


Figure 5. Capacity curves with alternating brace locations

The continuity of these bars significantly improves the shear behavior and the ductility of the frames. In contrast, the existence of the bare frame decreases the structural performance of the building.

The use of European codes leads to the application of the obtained results in many countries, which have incorporated such a standard.

The results obtained under the Spanish earthquake-resistant Standard (NCSE-02) corroborate this statement. Table 4 shows the most significant results from the capacity curves derived from the initial analysis, i.e., where different braces type devices (SB, CB and SAC) were compared to the bare frame.

As shown in the capacity curves, the damage thresholds were evaluated from the idealized bilinear capacity spectrum according to Lagomarsino and Penna (41), using the yielding displacement (dy) and the ultimate displacement (du). These four damage thresholds are:

Sd,1 = 0.7dy,
Sd,2 = dy,
Sd,3 = dy + 0.25(du-dy),
Sd,4 = du,
representing 'Slight', 'Moderate', 'Extensive', and 'Complete' damage states.

Table 4. Values of frames capacity curves

Prop.	No braces	SB	CB	SAC
Sd,4; D_u* [1] *(m)	0.557	0.184	0.182	0.259
Dif. without braces (%)	--	67%	67%	53%
Sd,2; D_y* [2] *(m)	0.404	0.179	0.164	0.215
Dif. without braces (%)	--	-44%	-41%	-53%
Base Shear (kN)	1630.9	3787.4	5284.9	6999.8
Dif. without braces (%)	--	132%	224%	329%
Ductility ***(D_u/D_y)***	1.377	1.026	1.111	1.202

[1] Maximum displacement. [2] Yielding displacement

The structural behavior of the frames with braces significantly improves the bare frame. The use of braces in the structures, improves by 132%, 224% and 329%, the base shear of the frames using SB, CB and SAC, respectively. The best ductility corresponds to SAC and the frames with aligned braces. Yielding displacements are reduced by 67% in frames with SB and with CB and 53% using SAC, compared to bare frames. Yield and ultimate displacement and the basal shear are similar in all the analyzed cases, with the exception of the frames that do not have braces in some of their plants, where the shear diminishes significantly. The increase in the rigidity of the structures is one of the factors causing the increase in the basal shear of the structures, as shown in table 5.

Table 5 shows the effect of different locations, in accordance with cases showed in Table 1. The higher strength occurred in frames c6, c7, c9, c15 and c16, and the highest ductility occurred in frames c1, c7 and c15. On the other hand, the stiffer frames were c8, c9 and c16. The D_u and D_y values have been set automatically by Seismostruct (25).

Table 5. Values of frame capacity curves

Code	***Sd,4: D_u (m)*** [1]	***Base shear (kN)***	***Sd,2: D_y (m)*** [2]	***Ductility (D_u/D_y)***	***Rigidity (kN/m)*** [3]	***Sd,1 (m)***	***Sd,3 (m)***
c1	0.38	3584	0.18	2.03	19386	0.13	0.23
c2	0.28	3628	0.15	1.83	23751	0.11	0.18
c3	0.31	3511	0.18	1.74	19545	0.12	0.21
c4	0.24	3533	0.15	1.58	23286	0.10	0.17
c5	0.28	3570	0.17	1.65	21414	0.11	0.19
c6	0.30	3746	0.15	1.98	24496	0.10	0.19
c7	0.31	3776	0.15	2.01	24447	0.11	0.19
c8	0.17	3512	0.12	1.39	29078	0.08	0.13
c9	0.28	3755	0.14	1.97	26305	0.10	0.18
c10	0.18	3371	0.15	1.19	21827	0.11	0.16
c11	0.19	3238	0.15	1.30	21892	0.10	0.16
c12	0.30	3552	0.17	1.70	20471	0.12	0.20
c13	0.15	3058	0.14	1.08	21549	0.10	0.14
c14	0.21	2825	0.17	1.28	17033	0.11	0.18
c15	0.38	3701	0.15	2.47	23924	0.11	0.21
c16	0.29	3708	0.15	1.97	25173	0.10	0.18

[1] Maximum displacement. [2] Yielding displacement. [3] Effective stiffness

 Informes de la Construcción, Vol. 77, 577, 6931, enero-marzo 2025. ISSN-L: 0020-0883. https://doi.org/10.3989/ic.6931

There was no collapse in any case by considering both the NCSE-02 and EC-8. All the results, (Tables 4 and 5) in terms of base shear and displacement were lower than the ones carried out by considering the push-over analysis, regardless of whether EC-8 or NCSE-02 are considered. Furthermore, collapse was not reached in any case and most of cases remained within the elastic zone.

All carried out results have been drawn in Figure 6 according to the two main key factors of structural design: ductility and basal shear. Hence it is possible to highlight the most preferred locations depending on the ductility behavior (c1, c7 and c15) or the base shear resistance criteria (c6, c7, c9, c15 and c16).

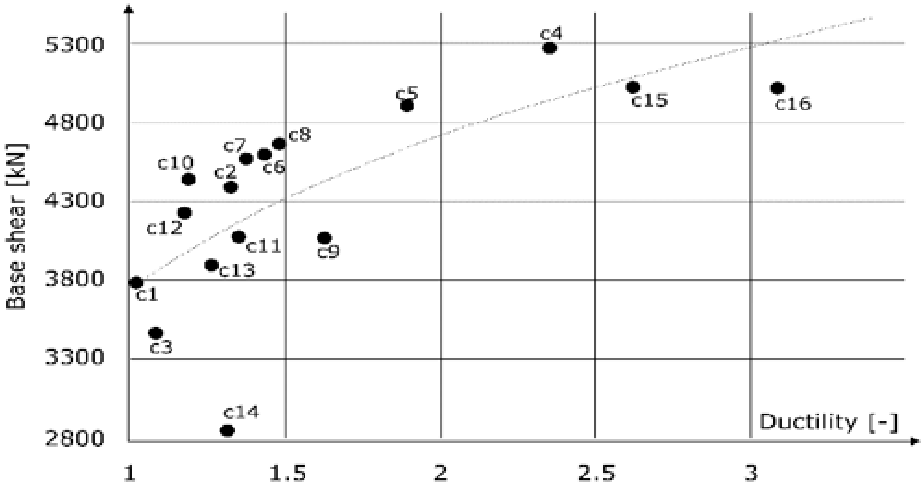


Figure 6. Ductility and base shear as function of simple bracing placement

4. DYNAMIC ANALYSIS

Non-linear dynamic analysis (42, 43) were performed in discrete time following (44) as discussed above. The time period (*Δt*) used in the analysis was 0.01 s to match the data from the Lorca earthquake register of May 11, 2011 (45, 46) for the direction N-S (Figure 8), the most unfavorable direction. The Lorca earthquake (35, 47, 48) has been one of the largest destructive earthquakes that occurred in Spain in recent years despite its moderate magnitude ($M_w = 5.1$); This severity is due to its shallow hypocentral depth (2 km) and the almost zero distance between the epicenter and the center of the city of Lorca. The Lorca earthquake has an impulsive character in the N-S component (49) considered in the dynamic analyzes of this research. The dimensionless Manfredi index takes the value I_D=2.57 (index that is defined as the integral of the square of the acceleration over the total duration and the product of the maximum values of acceleration and velocity). For comparison purposes with the Standard, Figure 7 shows the spectra of the record used for each direction with the spectrum of the Standard for a type I soil; In this image it can be seen that the spectral accelerations of the record clearly exceed the requirements of NCSE-02 (50) for almost all periods, especially for the N-S direction.

The choice of this record in this investigation has been chosen randomly considering the catastrophic effects it had in reality, in order to justify the behavior that buildings with steel frame structures with braces could have had, reducing the effects caused by earthquakes such as the one in Lorca (Spain). On the other hand, the choice of this structural typology is due to its abundance in areas that are not excessively seismic such as in Spain. This impulsive and superficial record (epicenter near the surface), was outside the scope of the Spanish Regulations (NCSE-02), as shown in the accelerations shown in Figure 7. On the other hand, the scaling used in this research was to demonstrate more conclusively the effectiveness of these devices in frame structures, because as has been shown in the dynamic results using the real record, the use of these devices would have significantly improved the structural behavior of existing buildings, reducing the damage and effects caused in reality.

The structure buffer (understood as the visual representation model used by the "Seismostruct" (40), software to represent the structure of the building) was represented by the Rayleigh model (Chopra) with a damping factor of 5%, an average value of our two modes damping values (4% and 6%) which has been used in diverse studies conducted in recent years for this type of frames.

Figure 9 shows the displacement of the upper floor from the time-history responses frames, corresponding to the North–South register acceleration data shown in Figure 8. This accelerogram was chosen as it was the most severe. The building structures were selected for this analysis as they were deemed to be the most representative of real-life settings. On the other hand, Lorca's record has been scaled up to collapse in the most unfavorable frames. Figure 9 shows the displacement of each frame, considering the log without scaling and scaling. The scale coefficient used in the records is 2.

 Informes de la Construcción, Vol. 77, 577, 6931, enero-marzo 2025. ISSN-L: 0020-0883. https://doi.org/10.3989/ic.6931

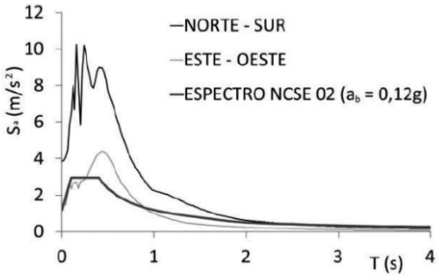


Figure 7. Response spectra of the accelerogram of the Lorca earthquake (35)

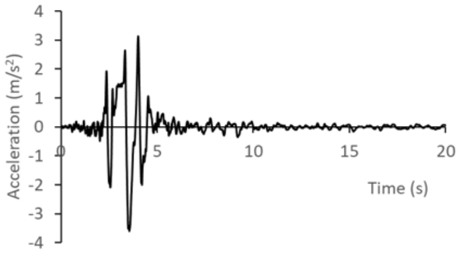
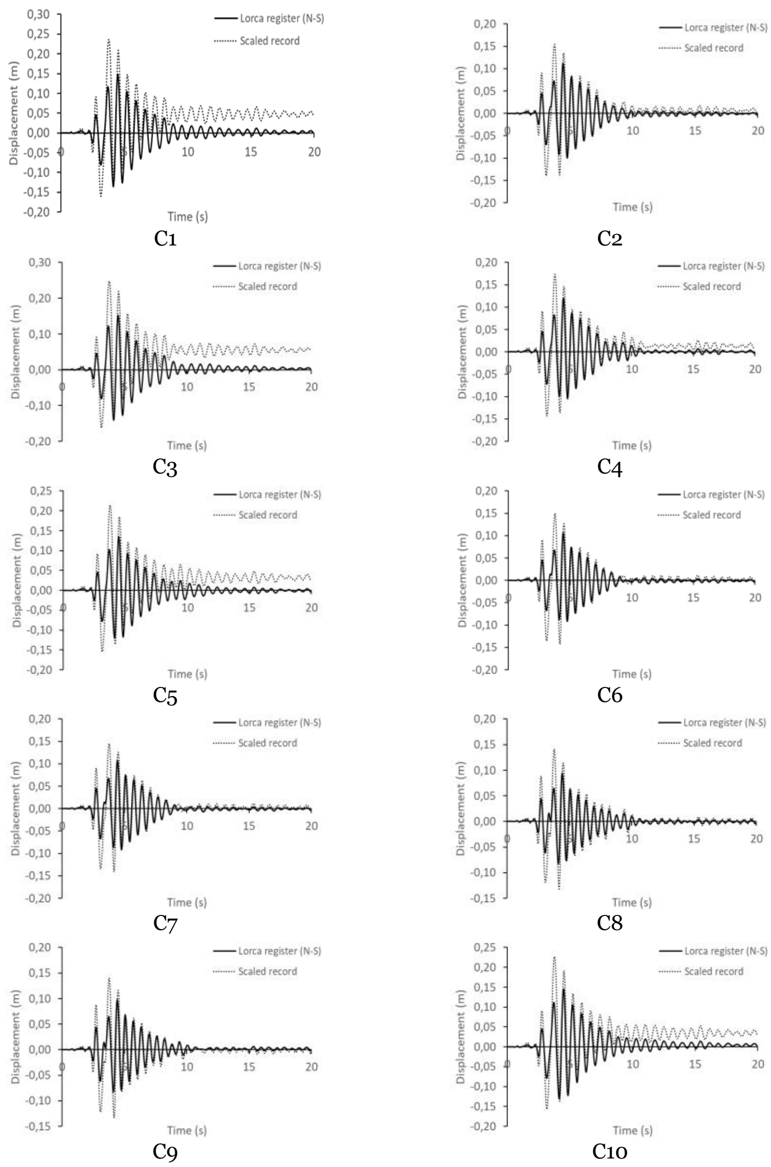


Figure 8. N-S acceleration (Lorca register 2011)



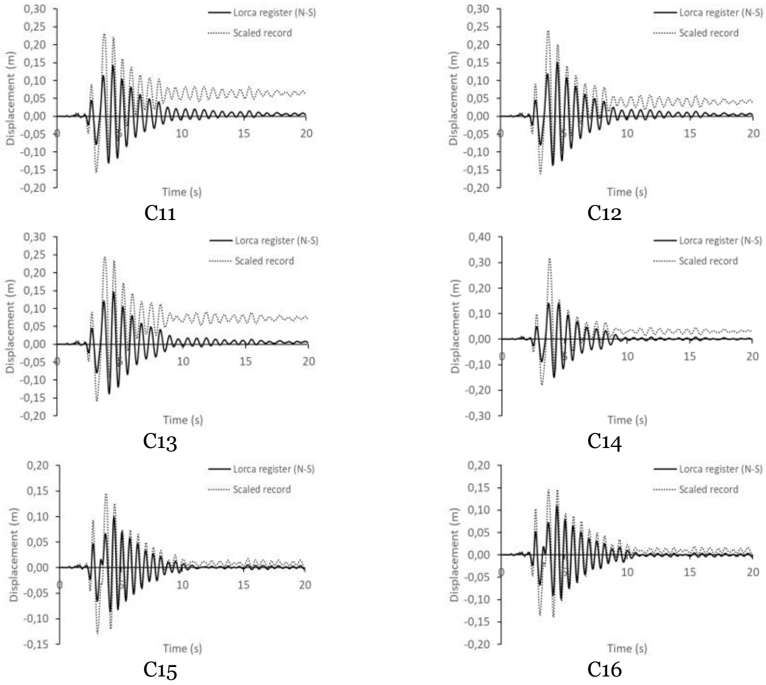


Figure 9. Displacement at the top of each frame

Table 6 shows the maximum displacements of the time-history analysis (dynamic analysis), belonging to the upper parts of the frames, considering the Lorca record of the year 2011 and the scaled Lorca record (the coefficient used is 2). On the other hand, the maximum axial compression of the braces in each case, considering the Lorca record (Reg) is lower than Euler's critical load, affirming what was previously mentioned that the braces do not buckle. On the other hand, in the displacements indicated in table 6, a series of gray colors of different shades are shown, related to the state in which the buildings are located considering the damage indices of Lagomarsino and Penna (41). The associated gray codes are as follows:

Code	*Sd,1 Slight damage (m)*	*Sd,2 Moderate damage: D_y (m)*	*Sd,3 Extensive damage (m)*	*Sd,4 Complete damage: D_u (m)*

Table 6. Maximum displacements in the upper part of the frames considering the Lorca record and Maximum axial braces.

Code	Max. Displacement Lorca register (*m*)	Max. Displacement Scale Lorca register (*m*)	Max. axial braces (*kN*)
c1	0.15	0.24	1025
c2	0.11	0.15	1015
c3	0.15	0.25	1017
c4	0.12	0.17	1012
c5	0.14	0.21	1036
c6	0.11	0.15	1035
c7	0.11	0.15	1037
c8	0.09	0.14	1038
c9	0.10	0.14	1036
c10	0.15	0.23	1037
c11	0.14	0.23	1030
c12	0.15	0.24	1036
c13	0.15	0.24	1015
c14	0.15	0.32	800
c15	0.10	0.15	1033
c16	0.11	0.15	1038

From the dynamic analysis, it is established that taking Lorca's record into account, no frame would enter the plastic regime. However, considering the scaled Lorca frame, only cases c2, c3, c6, c7, c8, c9, c12, c15 and c16 would not enter the plastic regime. On the other hand, frames c1, c4 and c5 would start to plastify, while frames c10, c11, c13 and c14 would collapse.

Finally, as shown in Figure 9, considering the scaled records, half of the frames (c1, c3, c4, c5, c10, c11, c12, c13 and c14) show significant permanent deformations at the end of the dynamic analysis.

5. DISCUSSION AND CONCLUSIONS

This study has demonstrated that SB may lead to a similar performance to CB or even SAC. This last case shows higher basal shear but equal ductility. Hence, more sustainable, and economical building structure may be obtained. The conclusions obtained from the non-linear static analysis (Push-over) and the static analysis carried out with the EC-8 and NCSE-02 Standards, using the seismic forces obtained, are similar. Detailing the results obtained with both standards, it can be observed that the analysis carried out with the European Standard (EC-8) are a little more restrictive than the analysis carried out with the Spanish Standard (NCSE-02). The design seismic forces vary between 25% and 15% as the height increases. These differences are practically insignificant in the displacements produced in the structures by carrying out the static calculations, due to the great rigidity that the structures have with the braces. Therefore, it is possible to obtain more sustainable and economical building which meets the mandatory requirements, as well.

Regarding the displacements of the damage states of the buildings, there are differences in the displacements, shears and ductilities with respect to the bare cases, evidencing the effect that these devices have on the structures of frame buildings. The smallest displacements occur in the CB case, but there are no major differences with respect to the SB case. However, the lowest base shears occur in the case of models without devices, considering their lower rigidity.

In spite of the most suitable locations for SB correspond to cases c2, c6, c7, c9, c15 and c16, the analysis determines that diagonals and central part of buildings lead to the better results regarding both base shear and ductility.

Considering the dynamic analysis, it is concluded that the frames that have floors without any brace are the most unfavorable and weak (c1, c4 and c10). While the frames with braces in the center (c2, c6, c7 and c8) and diagonals along the frames (c3, c9, c15 and c16) are the most resistant. After analyzing all the variables studied in this research, it could be concluded that the best structural behaviors occur in cases c8 and c15.

Half of the frames (c1, c3, c4, c5, c10, c11, c12, c13 and c14) show significant permanent deformations at the end of the dynamic analysis.

In general, cases that present greater ductility and effective stiffness are cases that have better dynamic behavior.

The structural behavior of the braced models determines that the maximum displacements occur in cases C1 and C15 and that the maximum plastic displacements occur in cases C1 and C3, cases in which the models have at least 2 braces on all floors, coinciding with a uniform arrangement of these devices along the heights of the models. The greatest shear forces occur in case C7, where the braces are concentrated symmetrically in the central part of the frame. The maximum ductility occurs in case C15, coinciding with the cases of greatest displacements and the lowest ductility in C13, where half of the floors of the model do not have braces, demonstrating the importance of having braces on all floors of the structural frame (eliminating the effects of “soft floors”), generating a solution that is too fragile. On the contrary, the maximum rigidity occurs in case C8, coinciding with a symmetrical arrangement on both axes of the structure in the form of a double St. Andrew's cross, and the lowest rigidity in case C14, coinciding with the case in which the upper half of the height of the frame has no braces, making a structure that is too weak.

As far as some researchers have explored new steel formulation, it is suggested to develop these manuscript findings by considering new material behaviors. In addition, it should be investigated higher building, i.e., from 15 stories in order to determine whether the obtained results are also valid.

The struts used in all cases never buckled considering the earthquake that occurred in Lorca.

The increase in the basal shear of the different cases studied is due to the significant increase in the rigidity of the frames.

c8, c12 c13, c14 and c15, are the cases that have no damage once the Lorca earthquake occurred. However, by doubling the accelerations, c13 and c14, they become severely damaged.

Regarding the dynamic analysis, although there are no major differences in the displacements (6 cm), there are differences in the structural behavior. Considering the accelerations of the Lorca record without scaling, the smallest displacements occur in cases C8 and C15, not reaching the point of plasticizing, as occurs in cases C12, C13, C14, although these are on the limit of beginning to plasticize. Doubling the accelerations of the record, changes in the structural damage are evident, reaching the point of plasticizing the models, even so, cases C8 and C9 are those with the smallest displacements in the structure, followed by cases C2, C6, C7, and C15, corresponding to structural models with braces on all floors. On the other hand, doubling the seismic accelerations of the record causes a significant worsening of some models, such as cases C13 and C14, curiously coinciding with the cases that do not have braces on half of the floors.

For future studies related to this research, it is advisable to conduct similar studies on frame buildings with different heights, with different structural configurations (structural design and different materials) and in different locations. What can be concluded is the structural improvement that these frame buildings have with the inclusion of braces and that ductility is beneficial for better structural behavior as stated in the earthquake-resistant regulations and in scientific theory.

DECLARACIÓN DE CONFLICTO DE INTERESES

Los/as autores/as de este artículo declaran no tener conflictos de intereses financieros, profesionales o personales que pudieran haber influido de manera inapropiada en este trabajo.

DECLARACIÓN DE CONTRIBUCIÓN DE AUTORÍA

David Dominguez-Santos: Conceptualización, Análisis formal, Investigación, Metodología, Administración de proyecto, Redacción – borrador original, Redacción – revisión y edición.

Pedro Muñoz Velazco: Conceptualización, Análisis formal, Redacción – revisión y edición.

REFERENCIAS

(1) Gardner, L. 2019. “Stability and design of stainless steel structures – Review and outlook”. *Thin-Walled Struct*, 141, 208-216. https://doi.org/10.1016/j.tws.2019.04.019.
(2) Poland, C.D., Mitchell, A.D. 2007. “A new seismic rehabilitation standard - ASCE/SEI 41-06”, Proceedings of Structures Congress, Long Beach, California, USA, May. https://doi.org/10.1061/40946(248)35.
(3) Dominguez-Santos, D., Pablo Ballesteros-Perez, Daniel Mora-Melia. 2017. “Structural resistance of reinforced concrete buildings in areas of moderate seismicity and assessment of strategies for structural improvement”. *Buildings*. 7(4):89. https://doi.org/10.3390/buildings7040089.
(4) Foraboschi, P. 2016. “Versatility of steel in correcting construction deficiencies and in seismic retrofitting of RC buildings”. *J. Build. Eng*. 8, 107-122. https://doi.org/10.1016/j.jobe.2016.10.003.
(5) Rakshith, B.D., Suneel Kumar, M. 2020. “Behaviour of steel columns with realistic boundary restraints under standard fire”, *Structures*, 28, 626-637. https://doi.org/10.1016/j.istruc.2020.08.028
(6) Safari, P., Broujerdian, V. 2020. Strategies to increase the survivability of steel connections in fire, *Structures*, 28, 2335-2354, https://doi.org/10.1016/j.istruc.2020.10.033.
(7) Rajput, A., Paik, J.K. 2020. “Effects of naturally-progressed corrosion on the chemical and mechanical properties of structural steels” *Structures*, article in press, https://doi.org/10.1016/j.istruc.2020.06.014
(8) Li, L., Li, C.-Q., Mahmoodian, M., Shi, W. 2020. “Corrosion induced degradation of fatigue strength of steel in service for 128 years”, *Structures*, 23,415-424, https://doi.org/10.1016/j.istruc.2019.11.013
(9) Wu, H., Zhang, L., Liu, C., Mai, Y., Zhang, Y., Jie, X. 2020. “Deposition of Zn-G/Al composite coating with excellent cathodic protection on low-carbon steel by low-pressure cold spraying”. *J Alloy Compd*. 821:153483. https://doi.org/10.1016/j.jallcom.2019.153483.
(10) Gao, X.Y., Balendra, T., Koh, C.G. 2013. “Buckling strength of slender circular tubular steel braces strengthened by CFRP”. *Eng. Struct*. 46, 547-556. https://doi.org/10.1016/j.engstruct.2012.08.010.
(11) Chen, J., Peng, W., Ma, R., He, M. 2012. “Strengthening of horizontal bracing on progressive collapse resistance of multistory steel moment frame”, *J. Perform. Constr. Facil*., 26 (5), 720-724. http://doi.org/10.1061/(ASCE)CF.1943-5509.0000261.
(12) Deng, K., Peng Pan, Xin Nie, Xu Xiaoguang, Peng Feng, Lieping Ye. 2015. “Study of GFRP Steel Buckling Restraint Braces”. *J Compos Constr*. 19(6):04015009. https://doi.org/10.1061/(ASCE)CC.1943-5614.0000567.
(13) Nazarimofrad, E., Shokrgozar, A. 2019. “Seismic performance of steel braced frames with self-centering buckling-restrained brace utilizing superelastic shape memory alloys” *Struct Des Tall Spec*. 28(16):e1666. https://doi.org/10.1002/tal.1666.
(14) Zahiri-Hashemi, R., Kheyroddi, A., Farhadi, B. 2013. “Effective number of mega-bracing, in order to minimize shear lag” *Struct Eng Mech*, 48(2), 173-193. http://dx.doi.org/10.12989/sem.2013.48.2.173.
(15) Qiao, S., Han, X., Zhou, K. 2017. “Bracing configuration and seismic performance of reinforced concrete frame with brace”. *Struct Des Tall Spec*. 26(14):e1381. https://doi.org/10.1002/tal.1381.
(16) Qiao, H., Luo, C., Wei, J., Chen, Y. 2020. “Progressive Collapse Analysis for Steel-Braced Frames Considering Vierendeel Action”. *J. Perform. Constr. Facil*. 34 (4):04020069, https://doi.org/10.1061/(ASCE)CF.1943-5509.0001475.
(17) Carrascal-Jiménez, M.C., Cifuentes-Tarquino, G., Nuñez-Moreno, F.A. 2017. “Lateral Bracing Effects on Slenderness, Stress Levels, and Lateral Displacements due to Wind and Seismic Events on a Steel Structure under Construction in Colombia” *J. Perform. Constr. Facil.* 31 (5):04017066. http://doi.org/10.1061/(ASCE)CF.1943-5509.0001050.
(18) López-Almansa, F., Domínguez, D., Benavent-Climent, A. 2013. “Vulnerability analysis of RC buildings with wide beams located in moderate seismicity regions”, *Eng. Struct*, 46, 687-702. https://doi.org/10.1016/j.engstruct.2012.08.033.
(19) Domínguez, D., López-Almansa, F., & Benavent-Climent, A. (2014). Comportamiento, para el terremoto de Lorca de 11-05-2011, de edificios de vigas planas proyectados sin tener en cuenta la acción sísmica. *Informes de la Construcción*, *66*(533), e008-e008. https://doi.org/10.3989/ic.12.092.
(20) Bruneau, M., Barbato, M., Padgett, J.E., Zaghi, A.E., Mitrani-Reiser, J., Li, Y. 2017. “State of the Art of Multihazard Design”, *J. Struct. Eng*. 143 (10):03117002. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001893.
(21) Hendy, C. R., Murphy, C.J. 2007. Designers' Guide To EN 1993-2: Eurocode 3: Design Of Steel Structures, Thomas Telford London, England, UK. https://doi.org/10.1680/dgte3.31609.
(22) Kumar, M. Stafford, P.J., Elghazouli, A.Y. 2013. “Seismic shear demands in multi-storey steel frames designed to Eurocode 8” *Eng. Struct*. 52, 69-87, https://doi.org/10.1016/j.engstruct.2013.02.004.
(23) Qu, Z., Xie, J., Cao, Y., Li, W., Wang, T. 2020. “Effects of Strain Rate on the Hysteretic Behavior of Buckling-Restrained Braces”. *J Struct Eng*. 146 (1):06019003, https://doi.org/10.1061/(ASCE)ST.1943-541X.0002486.
(24) ROBOT Structural Analysis Professional (2020), https://www.autodesk.com.
(25) SeismoSoft Inc. (2018), http://www.seismosoft.com.
(26) Li, S., Zhaiand, C-H., Xie L-L. 2012. “Evaluation of displacement-based, force-based and plastic hinge elements for structural non-linear static analysis”. *Adv Struct Eng*. 15(3), 477-488. https://doi.org/10.1260/1369-4332.15.3.477.
(27) Ghabussi, A., Marnani, J.A., Rohanimanesh, M.S. 2020. “Improving seismic performance of portal frame structures with steel curved dampers” *Structures*, 24, 27-40. https://doi.org/10.1016/j.istruc.2019.12.025.
(28) Wang, J., Uy, B., Li, D., Song, Y. 2020. “Fatigue behaviour of stainless steel bolts in tension and shear under constant-amplitude loading”. *Int J. Fatigue*, 133:105401. https://doi.org/10.1016/j.ijfatigue.2019.105401.
(29) Bosco, M., Ferrara, E., Ghersi, A., Marino, E.M., Rossi, P.P. 2016. “Improvement of the model proposed by Menegotto and Pinto for steel”. *Eng. Struct*. 124, 442-456. https://doi.org/10.1016/j.engstruct.2016.06.037.
(30) Spacone, E., Filippou, F.C., Taucer, F.F. 1996. “Fiber Beam-Column Model for Nonlinear Analysis of R/C Frames”, *Earthq Eng Struct D*. 25, 711-725. https://doi.org/10.1002/(SICI)1096-9845(199607)25:7<711::AID-EQE576>3.0.CO;2-9.
(31) Scott, M.H., Fenves, G.L. 2006. “Plastic hinge integration methods for force-based beam-column elements”. *J Struct Eng*. 132(2), 244-252. https://doi.org/10.1061/(ASCE)0733-9445(2006)132:2(244).
(32) Scott, M.H., Fenves, G.L., McKenna, F., Filippou, F.C. 2008. “Software patterns for nonlinear beam-column models”. *J Struct Eng*, 134 (4), 562-571. https://doi.org/10.1061/(ASCE)0733-9445(2008)134:4(562).
(33) Mergos, P.E., Kappos, A.J. 2015. “Estimating fixed-end rotations of reinforced concrete members at yielding and ultimate”. *Struct Concr*. 16(4), 537-545. https://doi.org/10.1002/suco.201400067.
(34) NCSE-02 (2002), Seismic Building Code (Construcción Sismo resistente: Parte general y edificación), Real Decreto 997/2002, Madrid, España.
(35) Domínguez, D (2012). Evaluación de la capacidad sismorresistente de edificios con vigas planas situados en zonas de España de sismicidad baja a moderada (Tesis doctoral). Barcelona. Universidad Politécnica de Cataluña.
(36) ATC 40 (1996), Seismic evaluation and retrofit of concrete buildings. Applied Technology Council. Redwood City, USA.

(37) FEMA P-1050-1 (2015), NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, Building Seismic Safety Council of the National Institute of Building Sciences. Washington, USA.
(38) López-Almansa, F.; Domínguez, D.; Benavent-Climent, A. (2013) Vulnerability analysis of RC buildings with wide beams located in moderate seismicity regions. Eng. Struct. 46, 687–702. https://doi.org/10.1016/j.engstruct.2012.08.033.
(39) Paulay, T.; Priestley, M.J.N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings; John Wiley: Hoboken, NJ, USA.
(40) Mergos, P.E.; Kappos, A.J. (2015) Estimating fixed-end rotations of reinforced concrete members at yielding and ultimate. Struct. Concr. 16, 537–545. https://doi.org/10.1002/suco.201400067.
(41) Lagomarsino, S., Penna, A. (2003), "Guidelines for the implementation of the II level vulnerability methodology. WP4: Table 4 Amount of materials required for each resistant system Resilient structures in the seismic retrofitting of RC frames: A case study Vulnerability assessment of current buildings", Technical Presentation RISK-UE Project: An Advanced Approach to Earthquake Risk Scenarios with Application to Different European Towns, https://cordis.europa.eu/project/rcn/54199/factsheet/en.
(42) Filippou, F.C.; Ambrisi, A.D.; Issa, A. Nonlinear Static and Dynamic Analysis of Reinforced Concrete Subassemblages; Earthquake Engineering Research Center, College of Engineering, University of California: Oakland, CS, USA, 1992.
(43) Magnusson, J.; Hallgren, M.; Ansell, A. Shear in concrete structures subjected to dynamic loads. Struct. Concr. 2014, 15, 55–65. https://doi.org/10.1002/suco.201300040.
(44) Newmark, N.M. A method of computation for structural dynamics. J. Eng. Mech. Div. 1959, 85, 67–94. https://doi.org/10.1061/JMCEA3.0000098.
(45) Escobedo, A. Damage assessment on building structures subjected to the recent near-fault earthquake in Lorca (Spain). In Proceedings of the 15th World Conference On Earthquake Engineering, Lisbon, Portugal, 24–28 September 2012.
(46) López-Comino, J.-A.; de Lis Mancnilla, F.; Morales, J.; Stich, D. Rupture directivity of the 2011, Mw 5.2 Lorca earthquake (Spain). Geophys. Res. Lett. 2012, 39.
(47) Murphy. P. (2011). Terremoto de Lorca 12 mayo 2011. Mesa redonda del 4° Congreso Nacional de Ingeniería sísmica. Granada.
(48) IGME. (2011). Informe geológico preliminar del terremoto de Lorca del 11 de mayo del año 2011, 5.1 Mw. Instituto Geológico y Minero de España.
(49) IGN. (2011). Serie terremoto NE Lorca (Murcia) 11/05/2011. Instituto Geográfico Nacional.
(50) NCSE-02 (2002). Norma de construcción sismorresistente. Ministerio de Fomento.