

**INVESTIGATION OF THE EFFECTIVENESS OF CAPACITY DESIGN  
GUIDELINES IN MULTI-STOREY STEEL BUILDINGS WITH  
CONCENTRICALLY BRACED FRAMES**

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**1. ABSTRACT**

The present paper deals with the capacity design of regular multi-storey steel buildings employing concentrically braced framing systems. A detailed investigation of their behaviour and load-bearing capacity under seismic loads using advanced non-linear analyses is presented. In particular, an extensive research is carried out regarding the influence of Eurocode 8 capacity design guidelines, and especially §6.7.2(2) which stipulates that the contribution of the compressed diagonal should be ignored during the analysis and design of a structure. In order to reach realistic conclusions, three possible design scenarios of a regular three-storey steel building are introduced, where the structure is redesigned in each case so as to fully comply with all Eurocode 3 and 8 guidelines. The behaviour of this steel building is then extensively investigated using material and geometry non-linear static analyses, for seismic action in the direction of the bracing systems and for all design scenarios. The main purpose is to acquire a more realistic estimation of the structure's ultimate load when the compressed diagonal, including its post-buckling behaviour, is taken into account. Furthermore, the necessity of capacity design procedures is highlighted by introducing a separate design scenario where the structure under investigation is designed ignoring the requirements of the seismic code. Valuable conclusions are reached through this approach, especially in cases where the nominal values of the imposed seismic loads exceed the design ones.

## **2. INTRODUCTION**

Concentrically braced frames (CBF) are cost-effective and, thus, popular forms of providing lateral resistance to multi-storey buildings. A typical CBF consists of diagonal braces attached to beams and columns using gusset plate connections. The two bracings usually intersect in the middle in order to reduce the compressed diagonal's buckling length. Due to their geometric configuration, the lateral forces are resisted by developing truss action; tension and compression. Their popularity and widely applicable use render the examination of their behavior under seismic loads extremely important. Even though they have been extensively investigated independently, it is rather important to examine their behavior when they are integral parts of a multi-storey structure.

Of particular interest is §6.7.2(2) of EN1998-1, which explicitly stipulates that in concentrically braced frames with diagonal bracings, only the tension diagonal should be taken into account. In order to approach this matter of investigation under realistic conditions, two design scenarios are introduced. More specifically, in Scenario 1 the three-storey building is designed ignoring the contribution of the compressed bracings in the structural configuration, while in Scenario 2 both diagonals are taken into account. The structure is designed through elastic analyses in the commercial software ETABS, so as to fully comply with the guidelines of capacity design guidelines. Finally, in an additional Scenario 3 the building is designed ignoring the capacity requirements of the seismic code, thus aiming to highlight the necessity of capacity design in the seismic resistance.

After dimensioning of the structure in Scenarios 1 and 2 is completed on the basis of linear analyses, we proceed to investigate its load-bearing capacity in the direction of the bracing systems. In order to acquire a better understanding regarding its actual response, which is primarily dominated by the behavior of the braced members, the behaviour of both diagonals should be taken into consideration. As a consequence, detailed geometry and material non-linear analyses are performed, so as to approach the post-buckling behavior of the compressed bracings. For ease of reference, only the results of the non-linear analyses carried out in Scenario 1 and Scenario 3 are presented and compared in detail.

## **3. DESCRIPTION OF THE STRUCTURE**

### **3.1 Geometry**

The steel building under investigation comprises of three storeys with a total height of 12 m, along with two structural systems in the two global directions of the building; an MRF system with multiple bays in the X direction and a CBF system in the middle frame in the Y direction (Figure 1). The building is symmetrical along its two global axes and is, therefore, regular both in plan and elevation (EN 1998-1: 2004, §4.2.3.2). Columns are fully fixed at their base in the direction of the MRF system, whereas in the direction of the CBF system they are assumed to be pinned. The thickness of each storey's concrete slab is assumed to be 0.20 m, while the centre of mass of each diaphragm coincides with its respective centre of rigidity due to the buildings symmetry in plan in both axes.

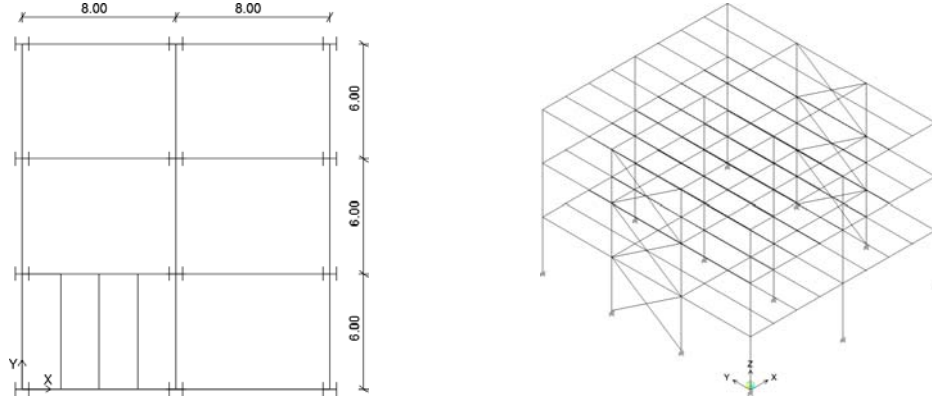


Fig. 1 Plan-view and 3D demonstration of structural system

### 3.2 Imposed loads

The vertical loads imposed in the structure are designated by the seismic combination and comprise of the following; the members' dead load, a uniformly distributed load owed to the slab's dead load in each storey which is then applied to the secondary beams with regard to their influence area, a superimposed dead load in the perimeter of the building due to peripheral walls and, finally, a uniformly distributed variable load on the level of each slab.

As for the horizontal loads, only the seismic force is taken into account. According to EN 1998-1: 2004, §3.2.2.1 the horizontal earthquake motion can be represented by the inelastic response spectrum in conjunction with the respective period of vibration of the structure. The period of vibration is a combination of the existing rigidity of the members, activated in the direction of the seismic motion (X or Y), and the total mass acquired from all imposed loads in the seismic design situation (G+0.3Q). These earthquake induced actions are automatically calculated using ETABS and then applied to the center of mass of the structure in each storey. Due to the building's symmetry in plan, its response is dominated by the fundamental mode of vibration of each direction and, therefore, the lateral force method of analysis can be applied to distribute the seismic actions in each storey. The design base shear is determined using the following equation:

$$F_b = \lambda \cdot m \cdot S_d(T_1, \xi) \quad (1)$$

$\lambda$  is the correction factor equal to 0.85 if  $T_1 \leq 2T_g$ , and the building has more than two storeys; otherwise  $\lambda=1.00$

$m$  is the total mass of the building in the seismic design situation (G+0.3Q)

$S_d(T_1, \xi)$  is the ordinate of the design spectrum for the fundamental period of vibration in the examined direction  $T_1$

While the distribution of the seismic force at the centre of mass in each storey is estimated as:

$$F_j = F_b \cdot \frac{m_j \cdot z_j}{\sum_i m_i \cdot z_i} \quad (2)$$

$z_j, m_j$  are the respective height and mass of each storey

The building under investigation should demonstrate its ability to resist the vertically and horizontally imposed loads in the most unfavourable combination in the seismic design situation and the Ultimate Limit State (ULS), as well as satisfy the deflection limitations in the Serviceability Limit State (SLS). The seismic design combination prescribed in EN 1998-1: 2004, §6.4.3.4 where  $\psi_{2,i}=0.30$  assuming office areas, is:

$$\sum_{j \geq 1} G_{k,j} + \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \quad (3)$$

An additional 30% of the seismic action in the transverse direction is also considered for all seismic combinations according to EN 1998-1: 2004, §4.3.3.5.2(4).

$$E_{Ed,X} + 0.30E_{Ed,Y} \text{ and } E_{Ed,Y} + 0.30E_{Ed,X} \quad (4)$$

Finally, according to EN 1998-1: 2004, §4.3.3.5.2, the vertical component of the seismic action is not required to be taken into account. The effects of accidental eccentricity are also taken into account.

#### 4. DESCRIPTION OF DESIGN SCENARIOS

In order to investigate the effects of the post-buckling and yielding behavior of the diagonals in the seismic load-bearing capacity of the structure, possible elastic scenarios that account for the different approaches of guideline EN 1998-1, §6.7.2(2) are introduced. The speculation on whether or not to include the compressed bracing in the elastic analysis, does not have an immediate and conclusive answer and is examined through two possible scenarios. In Scenario 1, the structure is modelled by taking into account only the tension diagonal in the structural system, thus completely ignoring the contribution of the compressed bracing in the lateral rigidity of the structure during the static analysis. On the other hand, in Scenario 2, the modelling of the structure is carried out considering both bracings during the dimensioning of the structure.

##### 4.1 Numerical Simulation

For the analysis of the multi-storey steel building, a three-dimensional (spatial) numerical model is created using the commercial analysis software ETABS. All elements are modelled as beam-type finite elements, whereas no eccentricity is assumed in the connections of the intersecting members. Columns are set to be fully fixed in the level of the foundations in the direction of the MRF system, whereas in the direction of the CBF system to be pins. The concrete slab of each storey is not modelled, although the equivalent vertical loads are introduced to the secondary beams, assuming a uniformly distributed load in the level of the slab. Rigid diaphragms are introduced in each storey due to the existence of the concrete slab. The design of the members is primarily executed manually in order to acquire more trustworthy results. For the dimensioning of the structure in each scenario, only linear elastic analyses are carried out. Both material and geometry non-linearity are ignored at this point.

##### 4.2 Dimensioning in Scenario 1

The most definitive requirement in the design of the diagonal members is the upper limit of the non-dimensional slenderness limitation in conjunction with the  $\Omega$  criterion for uniform distribution of ductility in height. Regarding the columns in both CBF and MRF systems, the

damage limitation and P-Δ effects checks dominate their design. As a result, the resistance checks are adequately satisfied, since it is the capacity design guidelines that govern the design of the members in Scenarios 1 and 2. On the other hand, the design of the main and secondary beams is dominated by the ULS and SLS states which are more unfavorable compared to the seismic situations. The following tables summarize the required checks according to EN 1998-1 for the diagonals and the columns which define the CBF system.

Capacity design requirements	Checks
Cross-section classification	Class 1 or 2
Internal force $N_{Ed}$	$N_{Ed}/N_{pl,Rd} \leq 1$
Non-dimensional slenderness limitation	$1.3 \leq \bar{\lambda} \leq 2.0$
Uniform distribution of ductility in height	$(\Omega_{max} - \Omega_{min})/\Omega_{min} \leq 0.25$

Table 1. Summary of capacity design requirements and checks for diagonals

Capacity design requirements	Checks
Cross-section classification	Class 1 or 2
$E_d = E_{d,G+0.3Q} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot E_{d,E}$	Section and member checks in bending and compression (EN 1993-1-1, §6.3.2.1(1), Eq. 6.61 & 6.62)
Local ductility condition	$\sum MR_c \geq 1.3 \sum MR_b$
P-Δ effects	$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} < 0.2$
Interstorey drifts check	$d_r \cdot v \leq \psi \cdot h$

Table 2. Summary of capacity design requirements and checks for columns in MRF system

Capacity design requirements	Checks
$N_{b,Rd}(M_{Ed}) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E}$	Section and member checks in bending and compression (EN 1993-1-1, §6.3.2.1(1), Eq. 6.61 & 6.62)
P-Δ effects	$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} < 0.2$
Interstorey drifts check	$d_r \cdot v \leq \psi \cdot h$

Table 3. Summary of capacity design requirements and checks for columns in CBF system

Scenario	Storey	Cross-section	A (cm <sup>2</sup> )	$\bar{\lambda}_z$	$N_{b,Rd}$ (kN)	$N_{cr}$ (kN)
1	1	RHS100X60X5	14.7	1.99	117	133
	2	RHS100X60X4	12	1.95	99	112
	3	SHS70X3	7.94	1.74	81	94

Table 4. Summary of diagonals' characteristics in Scenario 1

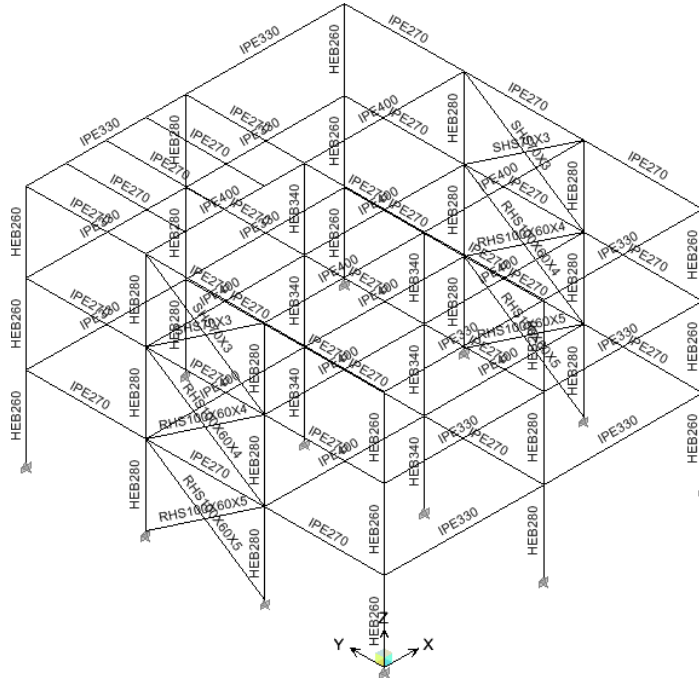


Fig. 2 Design of structure in Scenario 1

### 4.3 Dimensioning in Scenario 2

This scenario approaches the matter under investigation by considering the compressed bracing's contribution in the structural system during the dimensioning of the structure. Since the lateral rigidity of the CBF system is doubled when both bracings are taken into account, the period of vibration and, therefore, the imposed seismic loads are increased compared to Scenario 1. The same checks and requirements are applied here as well.

Scenario	Mass [m]	Rigidity [k]	Period of Vibration [T]	Base shear [ $F_b$ ]
1	m	k	T	$F_b$
2	m	2k	$\approx \sqrt{2}T$	$\approx \sqrt{2}F_b$

Table 5. Relative demand for base shear in Scenarios 1 and 2

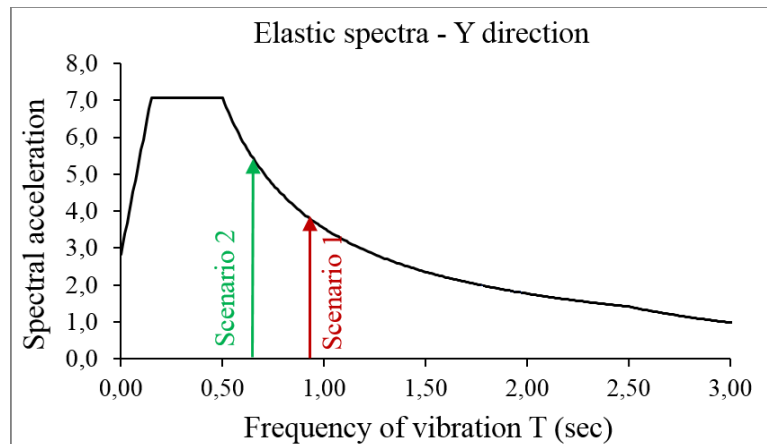


Fig. 3 Structure's response in a GMNIA analysis considering both diagonals in Scenario 1

The final design is quite similar in the two scenarios. This is an expected outcome when the following are considered. Firstly, according to Table 5, the increased base shear is resisted through a structural system comprising of two diagonals instead of one. Secondly, the design of the diagonals is dominated by the non-dimensional slenderness's upper limit in combination with the  $\Omega$  criterion instead of the resistance checks, which take into account the value of the imposed seismic loads. As a result, slightly different cross-sections are selected for the diagonals in the two cases.

Scenario	Storey	Cross-section	A (cm <sup>2</sup> )	$\bar{\lambda}_z$	N <sub>b,Rd</sub> (kN)	N <sub>cr</sub> (kN)
2	1	SHS80X5	14.7	1.56	183	218
	2	SHS70X4	10.4	1.77	103	119
	3	SHS70X3	7.94	1.74	82	94

Table 6. Summary of diagonals' characteristics in Scenario 2

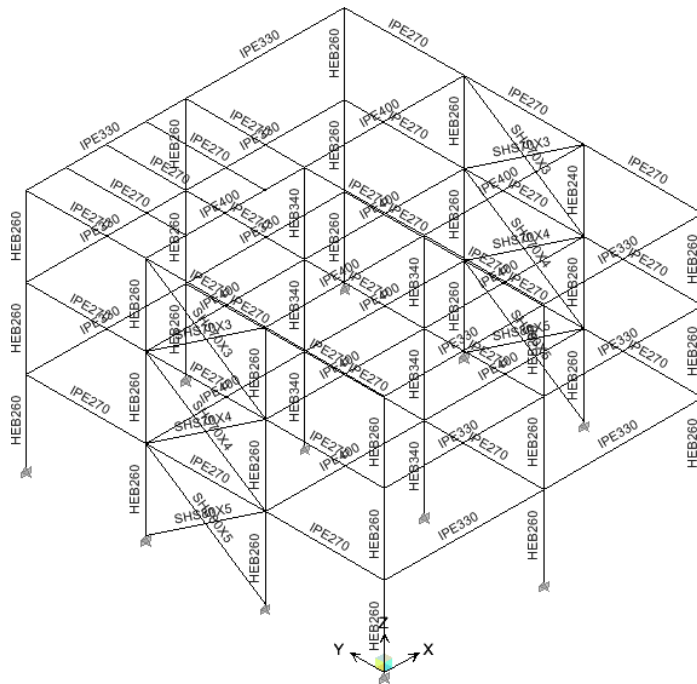


Fig. 4 Design of structure in Scenario 2

#### 4.4 Dimensioning in Scenario 3

In this scenario the bracings are designed only for the maximum developed tension axial force according to all seismic combinations. The columns are designed so as to be able to resist adequately the most unfavourable combination of axial force and bending moment according to Eq. (6.61) and (6.62) stated in EN 1998-1: 2004, §6.3.3.(4) without any capacity design requirements being applied. The beams in the MRF system are not checked for the capacity increased internal forces and should only present adequate resistance in both ULS and SLS states. The definitive requirement of the slenderness upper limit for the design of the diagonals as well as the strict damage limitation checks are also not applied in this scenario. Since resistance checks are only allowed to define the cross-sections of the members, it is

expected that the selected cross-sections of the diagonals are significantly smaller and therefore provide a more economic solution compared to Scenarios 1 and 2.

Scenario	Storey	Cross-section	A (cm <sup>2</sup> )	$\bar{\lambda}_z$	N <sub>b,Rd</sub> (kN)	N <sub>cr</sub> (kN)
3	1	CHS60.3X3.2	5.74	2.35	34	38
	2	CH48.3X4	5.57	3.02	20	22
	3	CHS33.7X3.2	3.07	4.39	5	6

Table 7. Summary table of diagonals' characteristics in Scenario 3

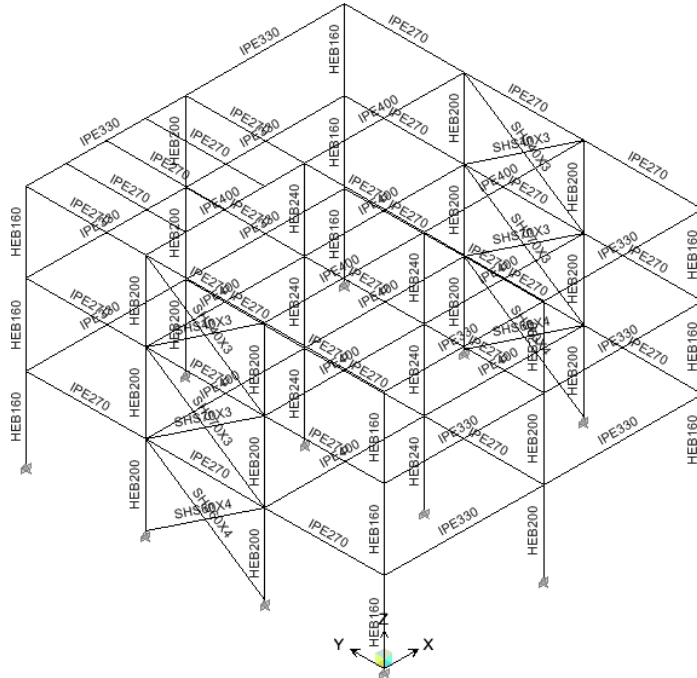


Fig. 5 Design of structure in Scenario 3

## 5. ASSESSMENT OF DESIGN SCENARIOS USING NON-LINEAR ANALYSES

### 5.1 Numerical Simulation

The simulation of the structure for the required non-linear analyses is carried out with the finite element analysis software ADINA. Eurocode 8 limits the cross-section classification for braced members to either Class 1 or 2, as in frames with concentric diagonal bracings the dissipative zones are located in the tension diagonals. This limitation eliminates the possibility of local buckling in the compressed braced members. For this reason, they are simulated with beam-type finite elements, as the only possible form of buckling is the flexural one. As for the simulation of columns, Eurocode 8 limits the cross-section classification to either Class 1 or 2 as well, whereas capacity design was applied to both directions. Therefore, beam-type finite elements are also introduced for the simulation of the columns.

### 5.2 Material

The actual behaviour of the structure is largely affected by material non-linearity, as its load-bearing capacity in the direction of the bracings relies mainly on the diagonal members. A



bilinear elastic-plastic material of hot rolled S355 steel is introduced in the non-linear model, with a realistic maximum allowable effective plastic strain of 20% and an ultimate tensile strength that reaches the value of 490 MPa.

### 5.3 Geometry

As illustrated in the following figures, the moment-resisting frames which are not directly connected to the bracing systems are removed from the non-linear model. This is due to the fact that they do not contribute to the rigidity of the structure in the Y direction since they are pinned at their base.

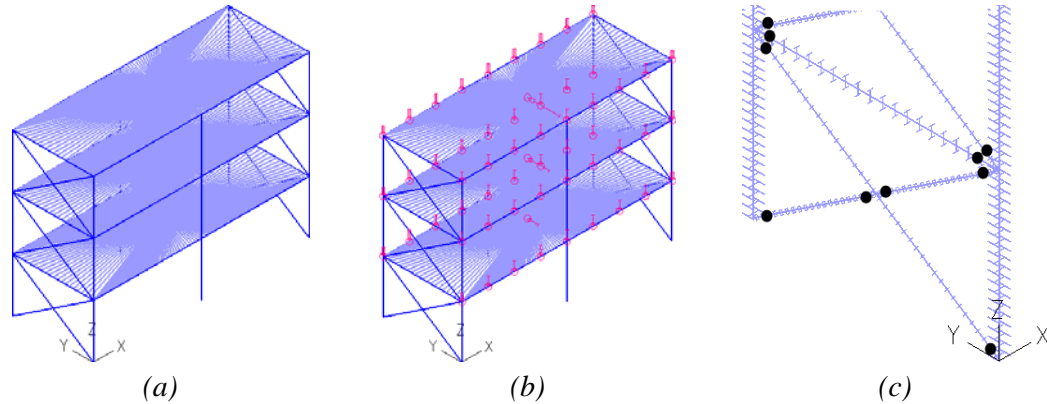


Fig. 6 Simplified non-linear model (a) geometry (b) rigid diaphragm (c) moment end-releases

### 5.4 Scenario 1 - Geometry and material non-linear analysis with initial imperfections

While geometric non-linearity is crucial for the development of the compressed bracing's global buckling, the tension bracing's main type of failure depends on yielding. Only out of plane initial imperfections are introduced to the RHS bracing systems, while both in and out of plane to the upper SHS as a result of the interaction between the major and minor principal axis of these diagonals. The required base shear includes only the tension diagonal, as the compressed bracings have lost their axial rigidity due to buckling.

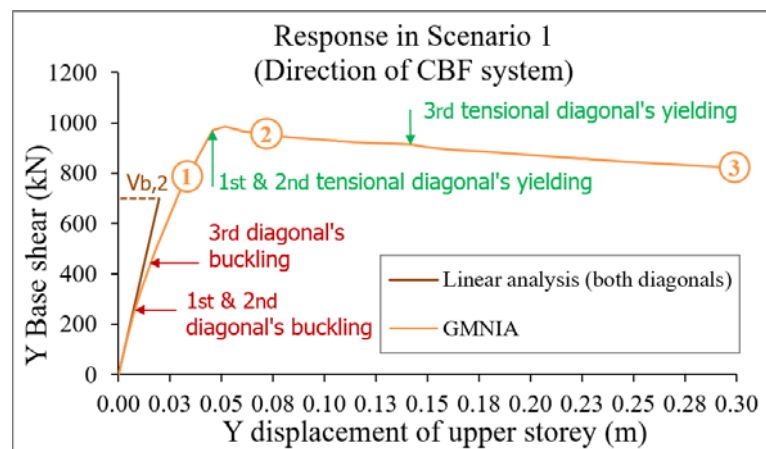


Fig. 7 Structure's response in a GMNIA analysis considering both diagonals in Scenario 1

As illustrated in Fig. 7, the elastic and GMNIA analyses present the same tangent and, therefore, rigidity for small base shear values. After the first buckling takes place for a base

shear of approximately 200 kN, the structure's response deviates from the elastic behavior and the two curves begin to separate from each other. Despite the fact that all compressed bracings lose their rigidity due to global buckling in the seismic design situation, the structure continues to resist horizontal loads as tension diagonals retain theirs. Considering that the buckling of the compressed bracings precedes the yielding, the structure's ultimate load-bearing capacity depends entirely on the tension diagonals after all bracings have buckled.

At this point, it is rather crucial that the behaviour of the CBF columns be investigated so as to detect whether any lateral torsional buckling, global buckling or yielding takes place. Such phenomena could influence the behaviour of the columns greatly and, consequently, the structure's behaviour as a whole. As a consequence of the stern drift and stability requirements and the relative sensitivity of the steel moment frames in the X direction that govern the design, CBF columns lead to considerable over-strength. Consequently, the CBF columns demonstrate a perfectly linear behavior.

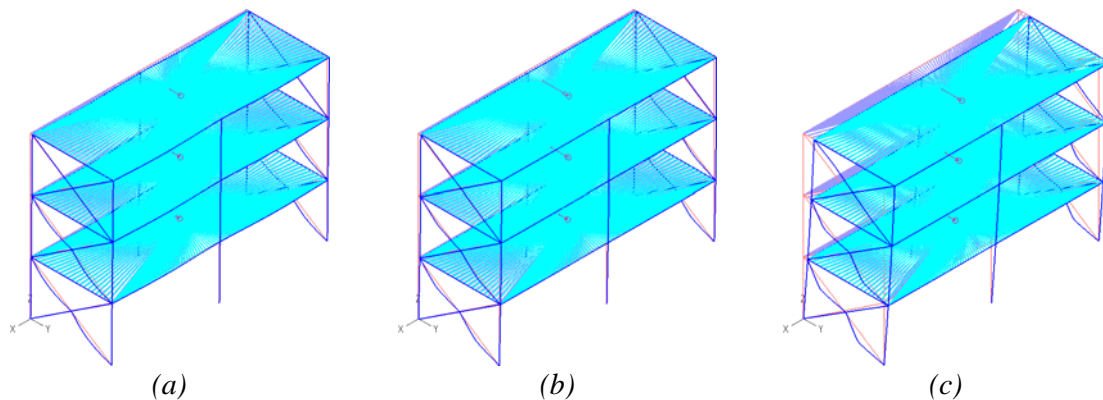


Fig. 8 Deformed shape of the structure in characteristic points of the GMNIA (a) Point 1 (b) Point 2 (c) Point 3

### 5.5 Scenario 2 - Geometry and material non-linear analysis with initial imperfections

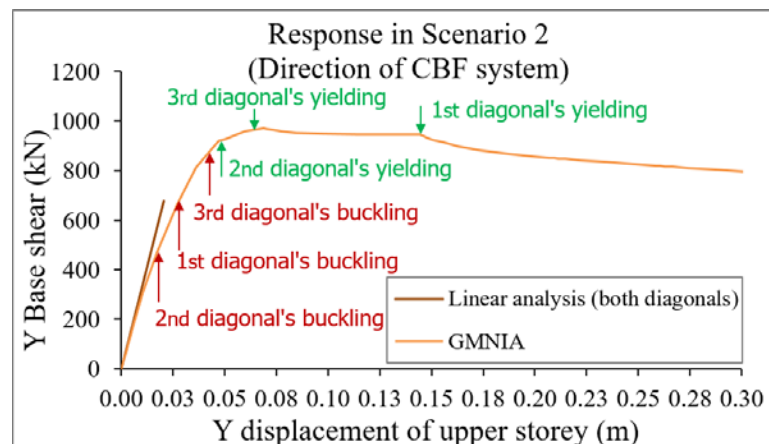


Fig. 9 Structure's response in a GMNIA analysis considering both diagonals in Scenario 2

According to

Fig. 9, the structure displays an almost identical ultimate load and overall response compared with Scenario 1, since the final design differs slightly in the two cases. Finally, since the GMNIA curves in the two scenarios are quite similar, it is safe to conclude that the guideline

under investigation is a safe approach to the structure's actual response. This is attributed to the rather definitive non-dimensional slenderness upper limit requirement.

### 5.6 Scenario 3 - Geometry and material non-linear analysis with initial imperfections

One of the main principles of capacity design is to avoid the possibility of partial or total collapse and, thus, the following two factors should be taken into consideration. Firstly, the structure should be able to resist seismic loads for the design earthquake or in case the seismic actions exceed the design earthquake's, without any danger of developing collapse mechanisms. Secondly, the structure should be able to resist the required seismic loads in case the members' nominal resistance is lower compared to that calculated during the design. The first point is satisfied by implementing the capacity design guidelines, while the second requirement is immediately satisfied through the extremely high standards employed in the production of structural steel by the industry nowadays.

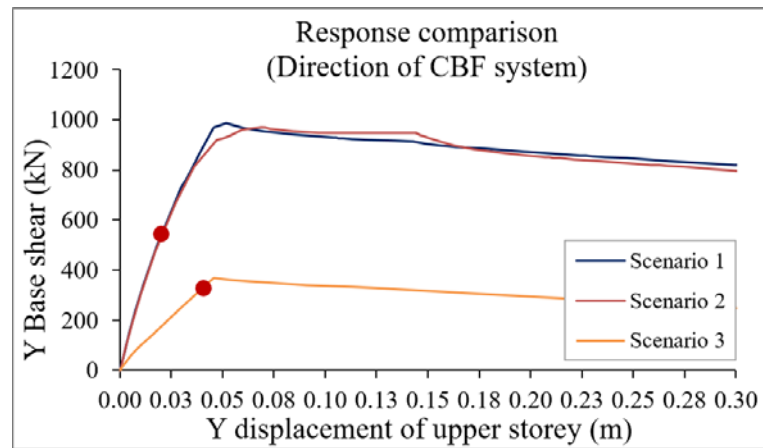


Fig. 10 Comparison of GMNIA curves for Scenarios 1,2 and 3 along with their respective demands for seismic shear (red)

Scenario	Load-bearing capacity (kN)	Base shear demand (kN)	Mass of steel members (tn)
1	1000	509	46
2	980	495	43
3	395	327	35

Table 8. Comparative demonstration of ultimate loads, seismic shear demand and steel mass

According to the GMNIA analysis, the buckling of the compressed bracings occurs for extremely small values of the seismic shear, as they do not comply with the non-dimensional slenderness limitation. After evaluation of the columns' behaviour, even in the case of Scenario 3 where columns do not comply with capacity design concepts, both buckling and yielding behaviors are prevented and, thus, do not influence the overall response of the structure. The selected diagonal cross-sections in this scenario, compared to Scenarios 1 and 2, result in a more economic although considerably less safer design. It is obvious that in Scenario 3, where capacity design requirements are ignored, the margin between the ultimate load and the required base shear is not considered adequate in case the nominal seismic values calculated in the design are exceeded.

## 6. CONCLUSIONS

An extensive research is carried out regarding the influence of Eurocode 8 capacity design guidelines, and especially §6.7.2(2) which stipulates that the contribution of the compressed diagonal should be ignored during the analysis and design of a structure. The load-bearing capacity of a three-storey steel building with concentrically braced frames is extensively investigated in the seismic design situation using non-linear analyses for two design scenarios. In Scenario 1, the structure is modelled by taking into account only the tension diagonal in the structural system, thus completely ignoring the contribution of the compressed bracing in the lateral rigidity of the structure during the static analysis. On the other hand, in Scenario 2, the modelling of the structure is carried out considering both bracings during the dimensioning of the structure. The structure is designed in each case separately using linear elastic analyses, so as to fully comply with Eurocode 3 and 8 guidelines. The resistance checks are adequately satisfied, since it is the capacity design guidelines that govern the design of the members in Scenarios 1 and 2. Afterwards, material and geometry non-linear analyses with initial imperfections are carried out in order to assess the load-bearing capacity of the structure in each scenario. After assessment of their response, it is safe to conclude that the guideline under investigation can be considered not only a safe, but also a quite realistic approach of the CBF system's response. According to the GMNIA curves, the non-dimensional slenderness limitation coupled with the 25% requirement of  $\Omega$  for the uniform distribution of ductility in height, increase the demand for cross-sections and, therefore, result in a higher load-bearing capacity compared to the respective demand for seismic shear. Furthermore, the stern drift and stability requirements lead to considerable overstrength of the CBF columns and eliminate the possibility of failure due to global buckling or yielding, while the classification requirements limit the possibility of local buckling. Finally, Scenario 3, where the seismic code is completely ignored, demonstrates the detrimental effects of the non-compliance to capacity design requirements in the overall load-bearing capacity of the structure, in case the nominal value of the calculated seismic force during the design is exceeded.

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**ΔΙΕΡΕΥΝΗΣΗ ΤΗΣ ΑΠΟΔΟΤΙΚΟΤΗΤΑΣ ΤΟΥ ΙΚΑΝΟΤΙΚΟΥ ΣΧΕΔΙΑΣΜΟΥ  
ΣΕ ΠΟΛΥΩΡΟΦΑ ΜΕΤΑΛΛΙΚΑ ΚΤΙΡΙΑ ΜΕ ΚΕΝΤΡΙΚΟΥΣ ΧΙΑΣΤΙ  
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## **ΠΕΡΙΛΗΨΗ**

Η παρούσα εργασία πραγματεύεται τον ικανοτικό σχεδιασμό κανονικών πολυώροφων μεταλλικών κτιρίων με κεντρικούς χιαστί συνδέσμους δυσκαμψίας, παρουσιάζοντας μία λεπτομερή διερεύνηση της φέρουσας ικανότητάς τους με χρήση μη γραμμικών αναλύσεων. Συγκεκριμένα, μελετάται αναλυτικά η επιρροή της διάταξης §6.7.2(2) του Ευρωκώδικα 8 στη φέρουσα ικανότητα της κατασκευής η οποία διευκρινίζει πως η συνεισφορά του θλιβόμενου συνδέσμου πρέπει να αγνοείται στη διαστασιολόγηση. Για την εξαγωγή ρεαλιστικών συμπερασμάτων διατυπώνονται 3 σενάρια διαστασιολόγησης ενός τριώροφου μεταλλικού κτιρίου, στα οποία η κατασκευή επαναδιαστασιολογείται σε κάθε περίπτωση ώστε να υπακούει σε όλες τις κανονιστικές διατάξεις. Ύστερα μελετάται η συμπεριφορά του τριώροφου μεταλλικού κτιρίου με χρήση μη γραμμικών στατικών αναλύσεων γεωμετρίας και υλικού για σεισμική διέγερση στη διεύθυνση των συνδέσμων δυσκαμψίας για κάθε σενάριο. Ο κύριος σκοπός είναι να προσεγγισθεί ικανοποιητικά το πραγματικό οριακό φορτίο της κατασκευής όταν οι θλιβόμενοι σύνδεσμοι και η μεταλυσισμική τους συμπεριφορά λαμβάνονται υπόψη. Αναδεικνύεται επίσης η αποδοτικότητα του ικανοτικού σχεδιασμού θεωρώντας ένα τέταρτο σενάριο όπου το υπό μελέτη κτίριο διαστασιολογείται αποκλειστικά ως προς τα αναπτυσσόμενα εντατικά μεγέθη, αγνοώντας πλήρως τις απαιτήσεις του σεισμικού κώδικα. Από τη διερεύνηση αυτή εξάγονται πολύτιμα συμπεράσματα στην περίπτωση όπου οι χαρακτηριστικές τιμές, κυρίως της σεισμικής έντασης, υπερβούν αυτές που υπολογίστηκαν κατά το σχεδιασμό.