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A capacity-based design process for earthquake-resistant steel frames

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Abstract:
Capacity-based design ensures that the structure collapses according to a desired scenario in the event of a large earthquake to minimise loss of life. This design approach has been widely researched and incorporated into some building codes. However, current methods still have limitations, leading to potential uncontrolled collapses in certain scenarios. Many researchers have worked to address these shortcomings, but existing procedures remain complex and challenging to apply in practice. This paper introduces a new design method that focuses on optimising structures beyond the elastic range. First, the frame is calculated for optimal plasticity according to the earthquake load to determine the plastic moments. These plastic moments are used to redesign the cross-sections of beams and columns. The beam cross-section is designed according to the calculated plastic moments. Meanwhile, the moment value for designing the column cross-section is equal to the calculated plastic moment value multiplied by a factor > 1. The results show that the steel frames designed using the proposed method attain a global collapse mechanism. Additionally, the ductile behaviour of the frames has been controlled. A 3-story, 1-span 2D frame and a 6-story, 3-span 2D frame are analysed using this capacity design approach to demonstrate the effectiveness and performance of the proposed procedure and compared with other methods.

Keywords:
plastic design; capacity design; steel frames; seismic behaviour; global ductility; optimisation

1 Introduction

According to traditional design methods, structural collapse is random and beyond the designer's control. This collapse can occur instantaneously, without sufficient time for load redistribution among other components (due to poor energy dissipation capacity) and without the life safety provisions mandated by contemporary earthquake engineering. However, such a collapse can be controlled by providing the structure with special properties of stiffness, strength, and ductility to ensure that it can perform as desired under seismic loads. This can be achieved by appropriately selecting successive energy dissipation regions and ensuring them through a proper design process, so the structure has optimal energy dissipation capacity when subjected to dynamic loads. This concept is embodied in a design process called the capacity design process [1].

In this design approach, specific primary members or regions of the lateral force-resisting system are selected, designed, and detailed to dissipate seismic energy. The remaining parts of the structure are designed with adequate strength to ensure the selected energy-dissipating elements remain effective throughout the earthquake without significant degradation of their capacity. These energy-dissipating regions in structural members are often referred to as plastic hinges and are designed to undergo inelastic deformation. Thus, implementing capacity design ensures that plastic hinges form at the intended locations and in the desired manner in the load-bearing system. Capacity design allows engineers to control the failure mode and mechanism of the structure under seismic loading [2-5].

This design process ensures a controlled failure scenario, allowing the structure to have optimal energy dissipation capacity. This enables the structure to withstand seismic loads for an extended period before collapse, significantly minimising human casualties during major earthquakes.

Initial research on this design process has primarily focused on reinforced concrete structures [1; 6-7], particularly in New Zealand, where the capacity design methodology has been implemented since 1980 [6-8]. Currently, it is widely acknowledged that controlling the placement of dissipative regions is crucial regardless of the building material [2-5; 9; 10]. Consequently, the first phase in seismic design of dissipative structures involves selecting an appropriate location for dissipative zones, that is, a suitable collapse mechanism.

This is the general standard for designing seismic-resistant dissipative structures, known as the 'capacity design criterion'. This definition implies that non-dissipative components must be engineered to withstand the ‘capacity’ of the fully yielded and strain-hardened dissipative zones [11].

As mentioned above, regarding the placement of energy-dissipating zones, plastic hinges should be situated in the beams rather than the columns. The primary objective of capacity design for columns is to preclude the simultaneous formation of plastic hinges at both ends of all the columns within a story [12]. For moment-resisting steel frames, this straightforward design criterion is adequate to prevent a ‘soft-storey’ from forming, but it does not result in frames collapsing in a global mode [13; 14]. Structural design focused on controlling the failure mode is a comparatively recent issue that has emerged due to seismic design requirements, primarily addressed through simplified guidelines provided in seismic codes. As noted earlier, current seismic codes [2-5; 9] only require adherence to the member hierarchy criterion, which is enough to prevent soft-story mechanisms, but can't develop a global mechanism.

Based on nodal equilibrium, these criteria are effective only if plastic hinges form simultaneously and according to the expected distribution of internal actions. However, this is uncommon in actual structures.

Lee [14] suggested an enhancement to the code design procedure. This was derived from observing the development of bending moments in a structure under constant vertical loads and increasing horizontal forces until structural failure occurred. He underscored that, in the elastic stage, the inverted bending points are usually located near the middle of the column. When collapse is imminent, the moment distribution changes significantly, with the moment in the lower column generally about three times that of the upper column. He proposed a three-quarter rule, suggesting using a $M_{Rc} \geq 0{,}75\beta \sum_{beams} M_{Rb}$ for each column to compensate for the increased stiffness in beams due to steel hardening. Although this design criterion can be sufficient to prevent a 'soft-story mechanism,' it is not enough to cause frames to fail in a global mode.

Mazzolani [15] proposed another approach derived from the kinematic theorem of plastic collapse. In a particular structure, with known values of vertical and horizontal loads, this procedure ensures global collapse if the actual force distribution during the earthquake matches the force used in the numerical analysis.

Ghersi [16] introduced a simplified method to address the complexities of Mazzolani's procedure. Nevertheless, the practical application remains challenging.

Furthermore, existing methods often lead to suboptimal section design, failing to fully exploit material capacity or achieve efficient distribution. To address this, this study incorporates post-elastic structural optimisation to rationalise internal force distribution and maximise section utilisation before increasing member sizes. This paper proposes the following procedure.

2 Suggested process

Designing structures according to the above standards has certain limitations. After designing the components in the usual way and then increasing the column cross-sections by a coefficient to induce plastic hinges in the beams, the expected plastic hinges often do not form. This is because increasing the column cross-section raises the internal force in the columns while reducing it in the beams. Meanwhile, under the usual design method, the cross-sections do not reach their maximum working capacity, and their distribution is unreasonable, causing plastic hinges to appear at unwanted locations. Therefore, this paper considers the problem of optimising structures beyond the elastic stage, aiming to distribute internal force reasonably and bring the cross-sections to maximum working capacity before increasing the main component's cross-section. From the above ideas, the following process is presented:

- Step 1. Optimal design of the structure operating in the plastic range under the effect of horizontal loads. This will cause the structure to reach a limit state, with the bending moment at critical sections reaching the plastic moment Mp.
- Step 2: Use the plastic moment values calculated in Step 1 to determine the cross-sections for all beams. Simultaneously, the beams should be selected to have sufficient strength to resist the design vertical loads (or comply with any other specified requirements).
- Step 3: Determine the moment values of the columns that are kept elastic (columns from the 2nd floor) by multiplying the plastic moment values obtained in Step 1 by a factor.
- Step 4: Use the moment values from Step 3 to determine the cross-section of the columns that are kept elastic.
- Step 5: Use the plastic moment values from Step 1 to determine the cross-section of the 1st-floor column, considering the effect of axial force, while ensuring that the determined cross-section is not smaller than the cross-sections of the upper-floor columns.

The objective of this process is to achieve the optimal collapse mechanism.To evaluate the effectiveness of the above-mentioned process, the paper uses the push-over analysis method with ETABS software to build a collapse scenario of the frame designed according to the above process [17-20].

3 Optimal design of a 2D frame beyond the elastic range

The challenge of minimising structural weight can be formulated as a linear programming problem based on the following assumptions:

- Plastic hinges are placed at important cross sections where there are large moments.

- Equilibrium equations are based on the original, undistorted configuration.
- The structural loads are assumed to scale proportionally.
- Constraints are solely associated with bending moment yield criteria and design specifications. For each prismatic element within the structure, the bending moment magnitude cannot exceed the plastic moment. Linear relationships between plastic moments and their associated constraints can be incorporated.

In general, both the kinematic and the static approaches can be used in optimal design. The static approach, which has a definite advantage over the kinematic approach in terms of computer storage and time requirements, will be presented in this paper.

3.1 General formulation

Assume a linear relationship between the plastic moment capacity M_{pi} and the weight of a group of members *i*; then an effective objective function can be established [21].

$$W = \sum_{i=1}^{s} M_{pi} L_i \tag{1}$$

Where M_{pi}, L_i are the plastic moment and length of the *i*-th bar element, respectively, *s* is the number of elements in the system.
It is assumed that the critical section forms *j* groups, where all sections in a given group are required to have the same plastic moment M_{pi}. Equation (1) can be expressed as:

$$W = \{L\}^T \{M_p\} \tag{2}$$

Where $\{L\}$ and $\{M_p\}$ are the vectors of L_i and M_{pi}, respectively.
Using the static approach, assume the frame has *n* degrees of redundancy and *m* critical sections where plastic hinges may form. The vector of internal moments at *m* critical sections and the external loads are related as:

$$[B]\{M\} = \{P_u\} \tag{3}$$

where $\{M\} = \{M_1, M_2, ..., M_m\}$ is the vector of internal moments at m critical sections; $\{P_u\} = \{P_1, P_2, ..., P_p\}$ is a vector describing the ultimate load, including p loads; and $[B]_{i.m}$ is a matrix consisting of *i* rows and *m* columns. The elements of the matrix [*B*] depend only on the undeformed geometry of the frame. The number of rows *i* of the matrix [*B*] is the number of independent equations of equilibrium (1) $i = m – n$, where *n* is the degree of redundancy.
Thus Equation (3) represents a system of *i* equations with *m* unknowns. The moments {*M*} must satisfy the following yield conditions:

$$-[H]\{M_p\} \leq \{M\} \leq +[H]\{M_p\} \tag{4}$$

where [*H*] is the matrix consisting of elements 0 and 1, with *m* rows and *j* columns (where *j* is the number of plastic moments of the assumed members when calculating). If $H_{mj} = 0$, the *i*-th plastic moment does not govern section *m*. Equation (4) stipulates that the permissible moments must not surpass the plastic moment capacities of the structural elements. Ideally, these constraints should be enforced at every point within the structure. However, in practical applications, it is sufficient to restrict their consideration to a finite number *m* of potential hinge locations. This can be accomplished by focusing on moments only at the ends of bars and at the points of maximum moment within loaded components.
According to the static theorem of limit analysis, the equilibrium Equation (3) and yield criteria Equation (4) constitute a necessary and sufficient condition for the design {*Mp*} to support the specified loads. Consequently, designs that fulfil both equilibrium and yield criteria are secure against the required load factor. Based on Equations (2), (3), and (4), the linear programming problem can be formulated as follows: find {*Mp*} and {*M*} such that:

$$W = \{L\}^T \{M_p\} \rightarrow min \tag{5}$$

$$[B]\{M\} = \{P_u\} \tag{6}$$

$$-[H]\{M_p\} \leq \{M\} \leq +[H]\{M_p\} \tag{7}$$

The present linear programming problem has *j*+*m* variables, *i* equalities, and 2*m* inequality constraints.

3.2 Problem-solving steps

- Identify the critical sections of the structure (at the ends of bars and at the location of the peak moment in loaded elements subjected to concentrated loads).
- Establish the objective function based on Equation (5).
- Establish the relationship between plastic moments and loads based on Equation (6).
- Establish the inequality constraints based on Equation (7).
- Solve the linear programming problem.

4 Implementation of the suggested procedure and comparison with other techniques

In this section, two examples are given. Example 1 is a 3-story, 1-span frame structure used to illustrate the proposed process. The purpose is to verify whether the proposed design process results in plastic hinges appearing at the expected locations, in accordance with the desired scenario for the structural system. Specifically, in the frame structure, plastic hinges only appear at the two ends of the beams and at the base of the 1st-floor column. The appearance of plastic hinges as described above allows the system to dissipate large amounts of energy through the plastic deformation of the structure. Example 2 is a 6-story, 3-span frame structure, cited from [22], and is used to compare the results with those of other researchers.

4.1 Example 1

Given a project with the diagram shown in Figure 1, the calculated loads are as follows:

- Evenly distributed load on the floor (excluding self-weight): q = 140 kg/cm^2
- Self-weight of the floor (12 cm thick): g = 330 kg/cm^2
- Evenly distributed live load on the floor: p = 240 kg/cm^2
- Wall + partition load: q_1 = 150 kg/cm^2
- Earthquake load: Design and test according to EC8, assuming the soil class used for design is stiff soil (class D), peak ground acceleration a_{gR} = 0,1041g, behaviour factor q = 5,4.
- Calculation method: Equivalent horizontal static force.

After calculation, the horizontal earthquake force acting on the frame is shown in Table 1, where the value λ = 1,0 has been assumed for the importance factor.

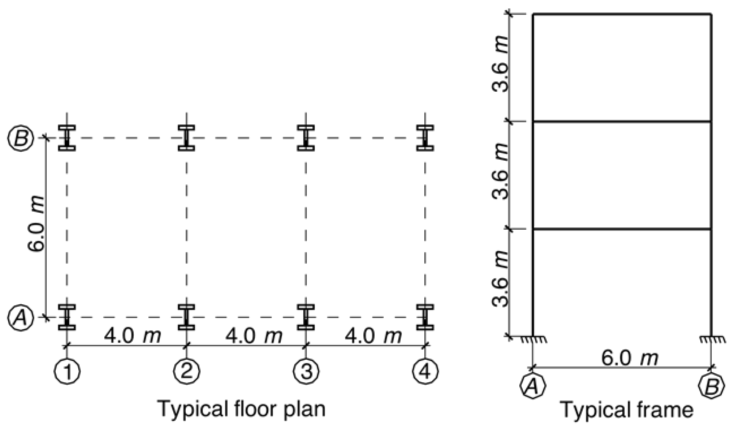

Figure 1. Diagram of example 1

Table 1. Distribution of horizontal force on building height

Storey	***h**i* (m)	***F**i* (kN)
1	3,6	21,78
2	7,2	43,56
3	10,8	65,34

4.1.1 Optimal design of the structure operating in the plastic range

The optimal design problem is to find $\{M_p\}^T = \{MP_1, MP_2, MP_3, MP_4, MP_5\}$ and $\{M\}^T = \{M_1, M_2, ..., M_{18}\}$ such that:

$$W = \{1.2l; 1.2l; 1.2l; 2l; l\} \begin{Bmatrix} M_{P1} \\ M_{P2} \\ M_{P3} \\ M_{P4} \\ M_{P5} \end{Bmatrix} \rightarrow min \quad (8)$$

The plastic moment M_{Pi} is unknown, and the 18 critical sections are shown in Figure 2.
The number of independent equations is given by:

- $i = m - n = 18 - 9 = 9$
- Matrix: $[B]_{9x18}$
- Matrix: $[P_u]_{9x1} = \{1{,}8F_1l; 3{,}0F_1l; 3{,}6F_1l; 0; 0; 0; ...; 0_{18}\}^T$

Equilibrium equations:

$$[B]_{9\times18}\{M\}_{18\times1} = \{P_u\}_{9\times1} \quad (9)$$

Matrix $[H]_{18x5}$, constraints:

$$-[H]_{18\times5}\{M_p\}_{5\times1} \leq \{M\}_{18\times1} \leq [H]_{18\times5}\{M_p\}_{5\times1} \quad (10)$$

To solve the optimisation problem, minimise the objective function (8), while satisfying the nine equilibrium equations (9) and 18 constraint conditions (10).
The optimal solution is:

- Minimum weight: $W_{min} = 5{,}91F_1l^2$
- Plastic moment: $\{M_p\}^T = F_1l\,\{1{,}35; 0{,}75; 0{,}45; 1{,}20; 0{,}45\}$
- Moments in critical sections: $\{M_i\}^T = F_1l\,\{1{,}35; 0{,}45; 1{,}35; 0{,}45; 1{,}20; -1{,}20; 0{,}75; 0{,}75; 0{,}75; 0{,}75; 1{,}20; -1{,}20; 0{,}45; 0{,}45; 0{,}45; 0{,}45; 0{,}45; -0{,}45\}$.

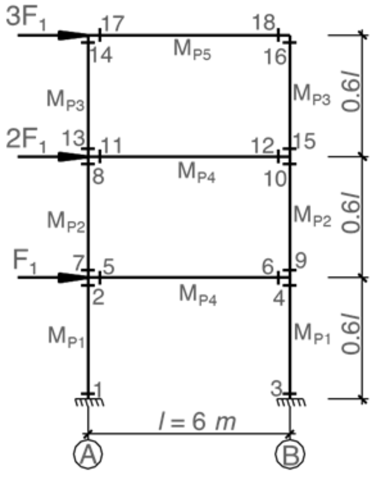

Figure 2. Critical sections of the frame and the group plastic moment M_{pi}

4.1.2 Selecting the cross-sectional size of the members

Cross-section of the 1st and 2nd floor beam:

- From $M_{P4} = F_1 \times l \times 1{,}2 = 21{,}78 \times 6 \times 1{,}2 = 156{,}82$ kNm

The plastic modulus of the beam is:

- $Z = \frac{M_{p,b} \times \gamma_{M_0}}{f_y} = \frac{156{,}82 \times 10^6 \times 1{,}1}{275 \times 10^3} = 627{,}26\ cm^3$

An IPE300 section in S275 steel (Z_{Rd} = 628 cm^3) is used.
Simultaneously, the design bending moment for the beam is calculated to resist vertical loads, determined by $(1{,}35G_k + 1{,}50Q_k)l^2/10$ (M_{sd} = 129,6 kNm). For this purpose, an IPE300 section in S275 steel (M_{Rd} =157 kNm) is adopted for the cross-section of the 1st and 2nd-floor beams.
Cross-section of 3rd floor beam:

- From $M_{P5} = F_1 \times l \times 0{,}45 = 21{,}78 \times 6 \times 0{,}45 = 58{,}8$ kNm

The plastic modulus of the beams is:

- $Z = \frac{M_{p,b} \times \gamma_{M_0}}{f_y} = \frac{58{,}8 \times 10^6 \times 1{,}1}{275 \times 10^3} = 235{,}22\ cm^3$

An IPE220 section in S275 steel (Z_{Rd} = 285 cm^3) is used.
Simultaneously, the design bending moment for the beam is calculated to resist vertical loads, determined by $(1{,}35G_k + 1{,}50Q_k)l^2/10$ (M_{sd} = 63,72 kNm). For this purpose, an IPE220 section in S275 steel (M_{Rd} = 71,25 kNm) is adopted for the cross-section of the 3rd-floor beam.
Column of the 2nd floor:

- From $M_{P2} = F_1 \times l \times 0{,}75 = 26{,}80 \times 6 \times 0{,}75 = 98{,}01$ kNm

To increase efficiency, the value of MP is multiplied by 1,7 (this accounts for the effect of axial forces):

- $M'_{P2} = M_{P2} \times 1{,}7 = 98{,}01 \times 1{,}7 = 166{,}62$ kNm

The plastic modulus of the columns is:

- $Z = \frac{M'_{p,c} \times \gamma_{M_0}}{f_y} = \frac{166{,}62 \times 10^6 \times 1{,}1}{275 \times 10^3} = 666{,}47\ cm^3$

A HEB220 section in S275 steel (Z_{Rd} = 827 cm^3) is used.
Similarly, the column cross-section of the 3rd floor can be chosen and is presented in Table 2.

Table 2. The column cross-section chosen from standard HEB shapes

Storey	Z_{min} (cm^3)	Section	Z (cm^3)
1	705,67	HEB220	827
2	666,47	HEB220	827
3	399,90	HEB180	481

4.1.3 Push over static inelastic analysis

Push-over analysis is based on the force-deformation relationship for sections that can form plastic hinges, as shown in Figure 3 in FEMA-356 [19]. This is represented as a curve with values at five points: A, B, C, D, and E. Point A is always the origin. Point B represents yielding. The displacement (rotation) at point B is subtracted from the deformations at points C, D, and E. Only the plastic deformation beyond point B is exhibited by the hinge. Point C represents the ultimate capacity for push-over analysis. Point D represents the residual strength for pushover analysis. Point E represents total failure. Beyond point E, the hinge will drop the load to point F (not shown), directly below point E on the horizontal axis. Additionally, deformation measures at points IO (immediate occupancy), LS (life safety), and CP (collapse prevention) are used for performance-based design, as they are informational measures reported in the analysis results. They do not have any effect on the behaviour of the structure.

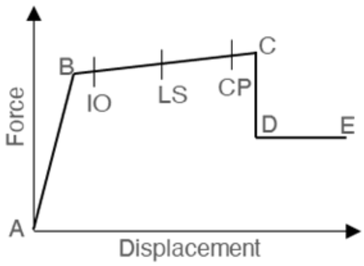

Figure 3. The curve for force vs. displacement

In this paper, push-over analysis is performed using ETABS software [23]. The *F-δ* diagrams (Figure 4) compare the behavioural curves of the proposed process's designed frame for different cases. In case 1, the plastic moments used to design the columns are not increased but remain unchanged from the optimal design value. In case 2, the plastic moments of the columns designed to work elastically are increased by 1,3 times compared to the initial value. In case 3, the plastic moments of the columns designed to work elastically are increased by 1,7 times compared to the initial value. The column and beam cross sections in all three cases are listed in Table 3.

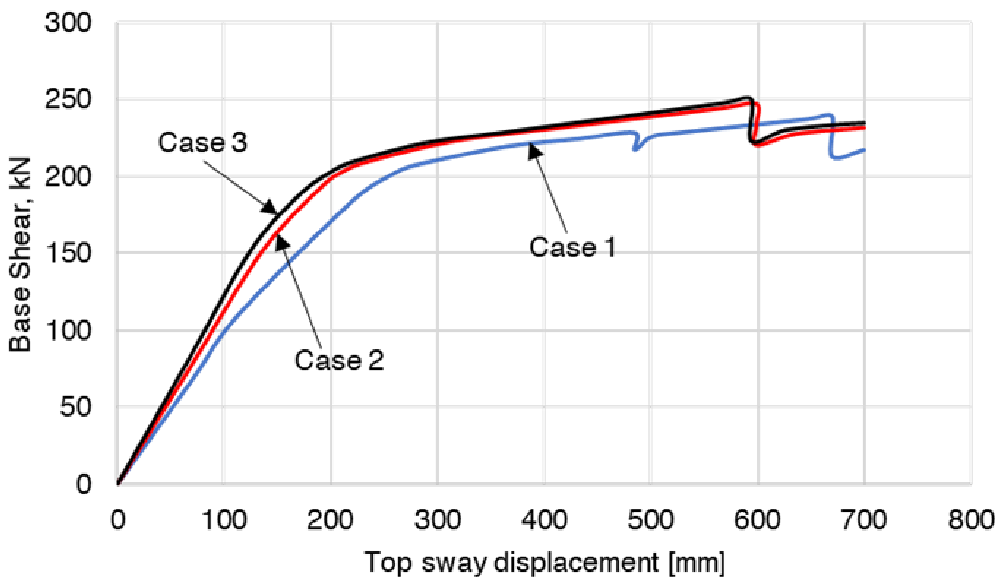

Figure 4. The behavioural curve of the frame for different cases

Table 3. Cross-sectional values of beams and columns for three cases

Section	Storey	Case 1	Case 2	Case 3
Column	1	HEB220	HEB220	HEB220
	2	HEB180	HEB200	HEB220
	3	HEB140	HEB160	HEB180
Beam	1, 2	IPE300	IPE300	IPE300
	3	IPE220	IPE220	IPE220

The formation of plastic hinges in the frame for the above cases is shown in Figure 5. From this result, it was observed that when increasing the coefficient (i.e., increasing the cross-sectional size of the elastic working members), the order of appearance of plastic hinges changes. The larger the coefficient, the earlier the plastic hinges appear in the beam. The larger the cross-sectional size of the elastic working members, the earlier the plastic hinges appear in the beam than in the column. That is, the location of the plastic hinges ensures more reliability. However, increasing the coefficient also increases the structure's weight, thus raising the cost. Therefore, an appropriate coefficient is selected depending on each specific case. In Example 1, selecting a coefficient of 1,70 (M'_P/M_P = 1,70) ensures that plastic hinges appear at both ends of the beam and at the base of the first-floor column.

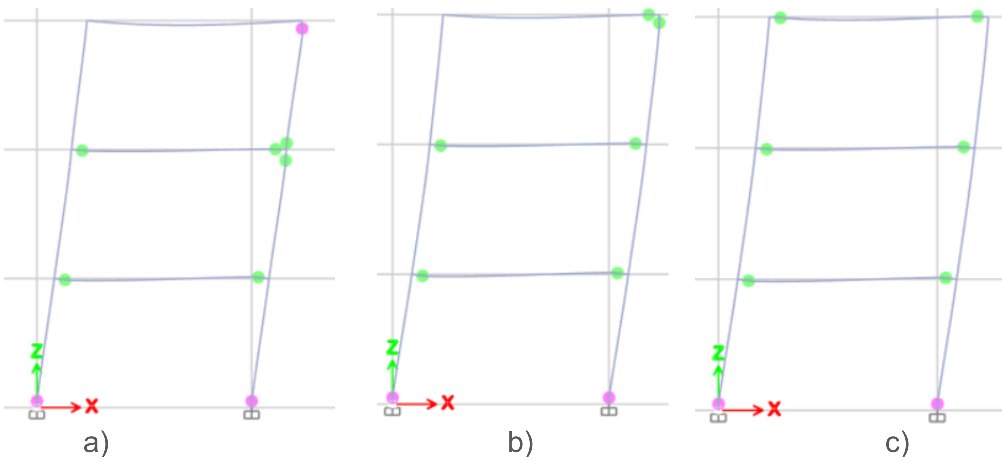

Figure 5. The formation of plastic hinges in the frame for different cases: a) case 1; b) case 2; c) case 3

4.1.4 Comparison with the member hierarchy criterion and Lee's three-quarter rule

“The member hierarchy criterion” is referenced in design standards [2-5]. After calculating the cross-sections of the beams and columns as given in Table 4, the results are compared using the member hierarchy criterion and Lee’s three-quarter rule. The formation of plastic hinges in the frame by the three methods is given in Figure 6. Additional criteria are presented in Table 5.

Table 4. Cross-sectional values of beams and columns for three methods

Section	Storey	The member hierarchy criterion	Lee's three-quarter rule	Proposed (Case 3)
Column	1	HEB180	HEB200	HEB220
	2	HEB180	HEB200	HEB220
	3	HEB180	HEB200	HEB180
Beam	1, 2	IPE300	IPE300	IPE300
	3	IPE220	IPE220	IPE220

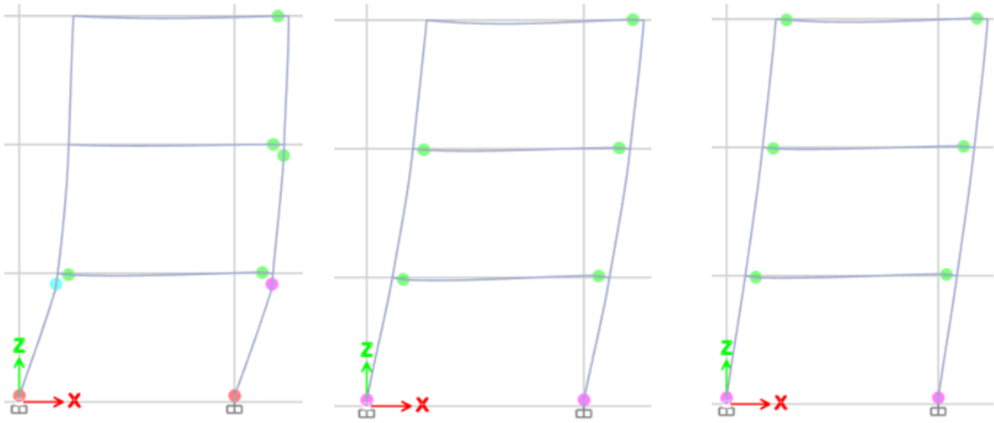

Figure 6. The formation of plastic hinges in the frame for different methods: a) the member hierarchy criterion; b) Lee's three-quarter rule; c) proposed method

Table 5. Some other criteria

Criteria	**Method**		
	The member hierarchy criterion	**Lee's three-quarter rule**	**Proposed**
Weight (tons)	1,771	1,988	1,989
Peak base shear (kN)	184,300	221,670	249,900
Ductility	4,560	5,410	5,620
Energy dissipation (kNm)	72,280	96,480	102,000

Figure 5 and Table 6 show that the proposed method is more effective than the member hierarchy criterion and Lee's three-quarter rule in terms of energy dissipation efficiency. Specifically, the proposed method outperforms the member hierarchy criterion and Lee's method by 29,1 % and 5,4 %, respectively. The weight of the proposed method is only 10,9 % larger than the member hierarchy criterion and approximately the same as Lee's method. Additionally, the plastic hinge formation scenario and bottom shear force are also better than these two methods.

4.2 Example 2

Example 2, cited in [21], compares the results with those of other researchers. The geometrical diagram of the design frame is shown in Figure 6. The uniformly distributed vertical loads acting on the beams are 15 and 10 kN/m for fixed action (G_k) and live action (Q_k), respectively. Therefore, the total vertical load acting on the beam is $1{,}35G_k + 1{,}50Q_k = 35{,}25$ kN/m.
Assuming the soil class used for design is stiff soil (class C) and $a_g = 0{,}35$g, the behaviour factor $q = 5$, based on Eurocode 8. The horizontal earthquake force acting on the frame is shown in Figure 7, where the value $\lambda = 1{,}0$ has been assumed for the importance factor.

4.2.1 Optimal design of the structure operating in the plastic range

The optimal design problem is to find $\{M_p\}^T = \{M_{p1}, M_{p2}, M_{p3}, M_{p4}, M_{p5}, M_{p6}\}$ and $\{M\}^T = \{M_1, M_2, ..., M_{84}\}$ such that:

$$W = \{4l, 8l, 8l, 4l, 22.5l, 4.5l\} \begin{Bmatrix} M_{p1} \\ M_{p2} \\ M_{p3} \\ M_{p4} \\ M_{p5} \\ M_{p6} \end{Bmatrix} \to min \tag{11}$$

The plastic moment MP is unknown. The 84 critical sections are shown in Figure 8. Similar to example 1, the optimal solution is:

- Minimum weight: W_{min} = 123,53$F_1 l^2$
- Plastic moment: $\{M_p\}^T$ = $F_1 l$ {3,74; 2,16; 1,74; 0,86; 1,95; 0,43}

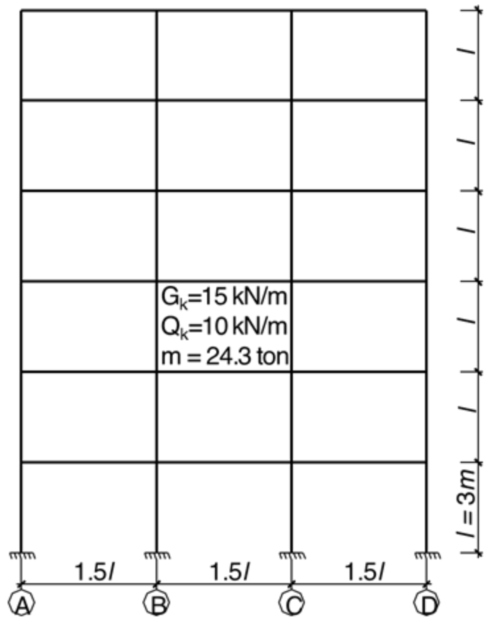

Figure 7. Geometric diagram of the example frame

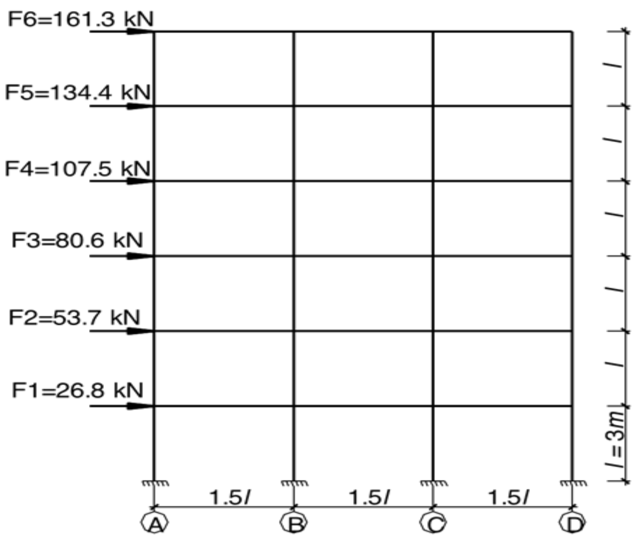

Figure 8. Seismic horizontal forces

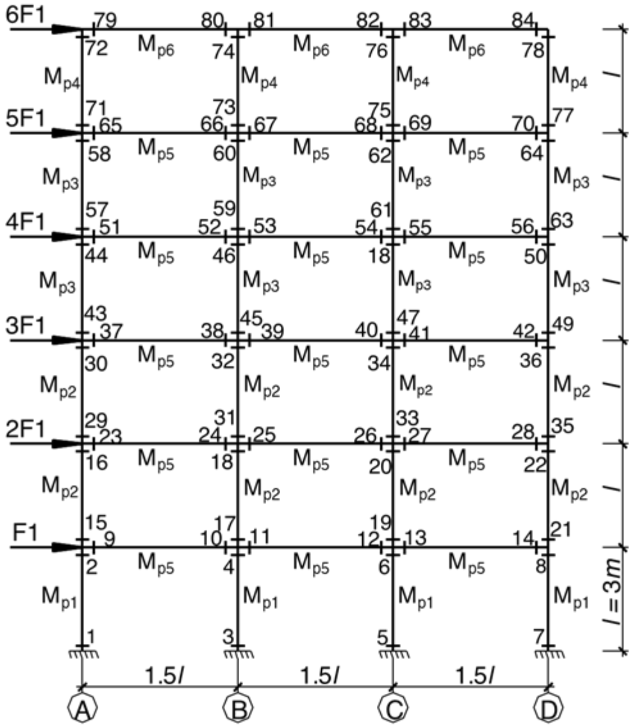

Figure 9. Critical sections of the frame, and the group plastic moment M_{pi}

4.2.2 Selecting the cross-sectional size of the members

Beams cross-section:

- An IPE300 section in S275 steel (Z_{Rd} = 628 cm^3) is used.
- Simultaneously, the design bending moment for the beam is calculated to resist vertical loads, determined by $(1{,}35G_k + 1{,}50Q_k)l^2/10$ (M_{sd} = 71,4 kNm). For this purpose, an IPE300 section in S275 steel (M_{Rd} = 157 kNm) is adopted for all the beams.

Columns cross-section, similarly, the column cross-sections are chosen and presented in Table 6.

Table 6. Design values of the column plastic modulus and corresponding sections chosen from standard HEB and HEA shapes

Storey	Z_{min} (cm^3)	Section	Z (cm^3)
1	1202,80	HEB300	1869
2, 3	1667,17	HEB300	1869
4, 5	1343,00	HEB280	1534
6	663,78	HEA240	745

4.2.3 Push over static inelastic analysis

To verify the advantages of the proposed design method, all design frames were subjected to pushover analysis until the collapse mechanism was formed.

The *F-δ* diagrams (Figure 9) compare the behavioural curves of the frame designed using the proposed method with those of frames designed using other techniques. According to Mazzolani's proposed process, the frame exhibits the highest durability and redistribution of ductility. The proposed method and Ghersi's method show equivalent performance; however, the proposed method has a larger dissipative capacity. Figures 10 through 14 illustrate the progression of plastic hinge formation and the ultimate collapse mode. Frame structures designed using the proposed method, Ghersi's, and Mazzolani's methods successfully attain a global collapse mechanism and exhibit significant overall ductility. In contrast, frames designed using the member hierarchy criterion and Lee criteria fail prematurely through localised mechanisms (involving 3 or 4 stories, respectively) and show reduced ductility. All structural configurations were successively examined using a push-over static inelastic analysis, considering the design spectrum for stiff soil conditions. Idealised plastic behaviour was assumed for the beam ends and the base sections of the first-story columns across all cases. In contrast, the other column sections were assumed to remain elastic.

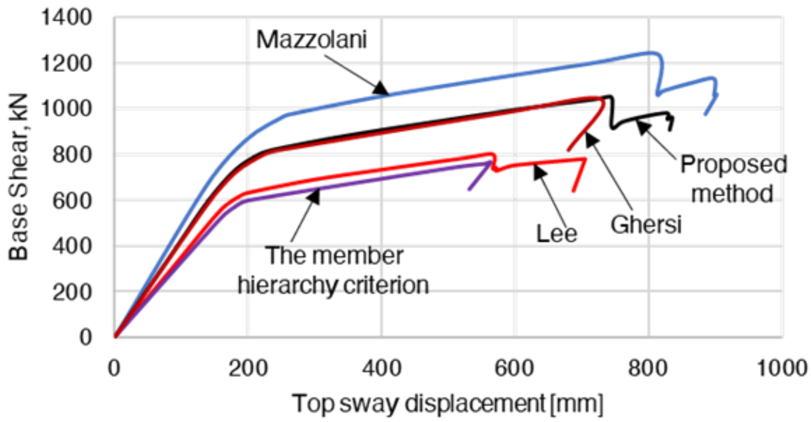

Figure 10. The behavioural curve of the frame for different methods

According to the member hierarchy criterion, the frame weighs 8,99 tons, and according to the three-quarter rule, it weighs 9,33 tons (Table 7). The weights for the frames designed using the Ghersi and Mazzolani processes are 10,76 tons and 12,15 tons, respectively, while the proposed method results in a weight of 10,82 tons. The total frame weight is minimal (8,99 tons) using the member hierarchy criterion and increases by 4 %, 20 %, and 35 % using the Lee, Ghersi, and Mazzolani processes. The proposed method results in a 20,4 % increase in weight. Therefore, to achieve a global collapse mechanism, the frame weight must be increased by 20 % compared to the member hierarchy criterion and by 16 % compared to the three-quarter rule. However, according to the proposed process, the frame weight is still less than that of Mazzolani's process.

Additionally, the frames designed using the proposed method and Mazzolani's method show significant improvements in strength and plasticity redistribution compared to those designed using other methods.

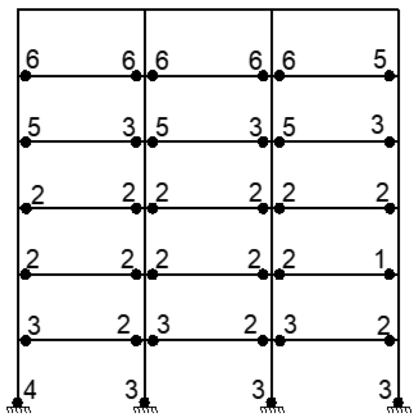

Figure 11. Order of appearance of plastic hinges in the frame according to Mazzolani's method

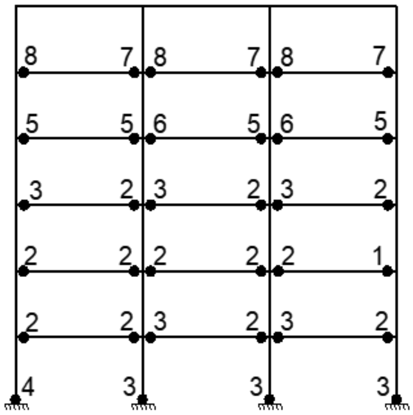

Figure 12. Order of appearance of plastic hinges in the frame according to the proposed method

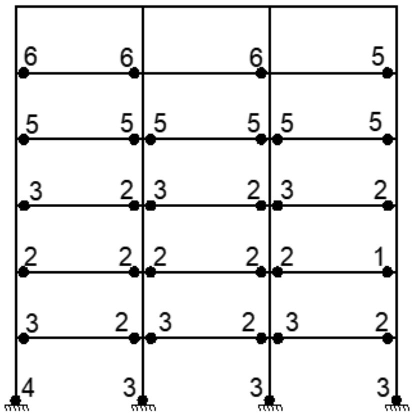

Figure 13. Order of appearance of plastic hinges in the frame according to Ghersi

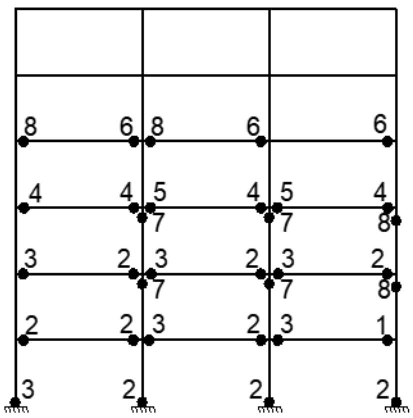

Figure 14. Order of appearance of plastic hinges in the frame according to Lee's three-quarter rule

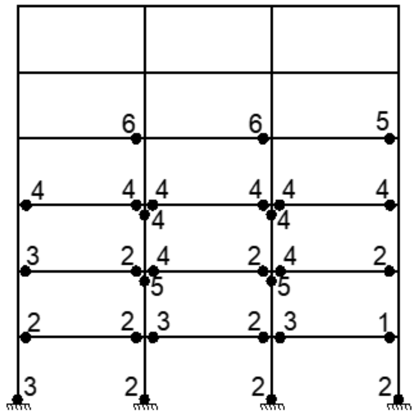

Figure 15. Order of appearance of plastic hinges in the frame according to the member hierarchy criterion

Table 7. Several other criteria compare the effectiveness of the methods

Criteria	Method				
	Mazzolani	Proposed method	Ghersi	Lee's three-quarter rule	The member hierarchy criterion
Weight (tons)	12,15	10,82	10,76	9,33	8,99
Peak base shear (kN)	1241,80	1047,90	1040,80	801,70	763,50
Maximum drift (cm)	90,20	83,40	72,90	70,60	56,20
Ductility	6,60	5,95	5,26	4,47	3,56
Energy dissipation (kNm)	601,50	594,10	408,60	352,20	247,30
Number of plastic hinges appearing on the beam	30	30	28	23	21

5 Conclusion

This paper outlines a new design methodology for steel frames designed to achieve global failure. The method is based on a post-elastic structural optimisation problem to rationally distribute internal forces and push the cross-sections to their ultimate capacity while working in the post-elastic range. This ensures that plastic hinges form as desired when the cross-sections of the main members are increased.

Furthermore, the reliability of the proposed design procedure was demonstrated through static pushover inelastic analysis of a 3-storey, 1-span frame and a 3-bay, 6-storey frame.

Lastly, the results of the proposed design procedure were compared with those of other methods. The performance of the proposed design approach was assessed through pushover and dynamic inelastic response analyses, confirming its capacity to induce a global collapse mechanism and substantial energy dissipation. The energy dissipation and ductility of the frame designed using the proposed method are significantly better than those designed using the member hierarchy criterion, Lee's three-quarter rule, and Ghersi’s procedure, and are comparable to those of the frame designed using the more complex and expensive procedure proposed by Mazzolani [15].

The proposed design method has achieved better energy dissipation capacity, ductility, and global collapse mechanism under seismic loads. However, this design method was only conducted on typical 2D frames. A major challenge is to verify the effectiveness and stability of the proposed method when applied to more complex 3D space structures, which involve interactions between load-bearing components in different directions.

In the future, research should expand the optimization objective to encompass factors beyond internal force distribution and material cost by integrating considerations such as fabricability and sustainability into the objective function. Furthermore, it is necessary to extend the research to other types of steel frames, such as braced frames, steel-concrete composite structures, or 3D space structures, to examine the adaptability of the method.

References

[1] Bertero V. V.; Popov, E. P. Seismic behaviour of ductile moment-resisting reinforced concrete frames. In: *SP-053: Reinforced Concrete Structures in Seismic Zones*. USA: American Concrete Institute; 1977, pp. 247-292.

[2] European Committee for Standardization. *Eurocode 8: Design of structures for earthquake resistance - Part 1-1: General rules and seismic action*. Brussels: CEN; 2024.

[3] European Committee for Standardization. *Eurocode 8: European Code for Seismic Regions - Design, Part 1.3: Buildings*. Brussels: CEN; 1994.

[4] International Conference of Building Officials. *Uniform Building Code*. Whittier, California; 1991.

[5] American Institute of Steel Construction. ANSI/AISC 341-10. *Seismic Provisions for Structural Steel Buildings, load and Resistance Factor Design*. USA: ANSI; 2010.

[6] Burns, R. J. Seismic design of ductile moment resisting reinforced concrete frames. *Bulletin of the New Zealand Society for Earthquake Engineering*,1978, 11 (2), pp. 121. https://doi.org/10.5459/bnzsee.11.2.121

[7] Paulay, T. Capacity design of earthquake resisting ductile multistorey reinforced concrete frames. In: *Proceeding of the 3rd Canadian Conference on Earthquake Engineering*. 1979, Montreal, Canada, Canadian Association for Earthquake Engineering and Seismology; 1979, pp. 917-948.

[8] New Zealand Standard Code of Practice for the Design of Concrete Structures. NZS 3101. *Part 1: Commentary NZS 3101, Part 2: Standard Association of New Zealand*. Wellington, New Zealand: NZS; 1982.

[9] Architectural Institute of Japan. *Standard for limit state design of steel structures*. Tokyo, Japan; 1992.

[10] Mazzolani, F. M.; Piluso, V. Design of Steel Structures in Seismic Zones. In: *European convention for Conventional Steelwork*. 1st Edition, 1994.

[11] Engelhardt, M. D.; Popov, E. P. *Behaviour of long links in eccentrically braced frames*. University of California, Berkeley: Earthquake Engineering Research Center, 1989.

[12] Paulay, T.; Priestley, M. J. N. *Seismic Design of Reinforced Concrete and Masonry Buildings*. USA: Wiley, 1992. https://doi.org/10.1002/9780470172841

[13] Bertero, V. V.; Zagajeski, S. W. Optimal inelastic design of seismic-resistant reinforced concrete framed structures. In: *Nonlinear Design of Concrete Structures, CSCE-ASCE-ACI-CEB International Symposium*. University of Waterloo, Ont. Canada; 1979, pp. 219-272.

[14] Lee, H.-S. Revised Rule for Concept of Strong-Column Weak-Girder Design. *Journal of Structural Engineering*, 1996, 122 (4), pp. 359-364. https://doi.org/10.1061/(ASCE)0733-9445(1996)122:4(359)

[15] Mazzolani, F. M.; Piluso, V. Plastic Design of Seismic Resistant Steel Frames. *Earthquake Engineering & Structural Dynamics*, 1997, 26 (2), pp. 167-191. https://doi.org/10.1002/(SICI)1096-9845(199702)26:2%3C167::AID-EQE630%3E3.0.CO;2-2

[16] Ghersi, A.; Neri, F.; Rossi, P. P. A Global Approach to the Design of Steel Frames. In: *Proceedings of the 6th International Colloquium, Stability and Ductility of Steel Structures (SDSS'99)*, Dubina, D.; Ivanyi, M. (eds.). September 9-11, 1999, Timisoara, Romania. Elsevier; 1999, pp. 377-384.

[17] Emad, A. E. Effect of Beam-Column Connection Types on the Response Modification Factors of Steel Frames. *International Journal of Steel Structures*, 2024, 24 (1), pp.132-143. https://doi.org/10.1007/s13296-023-00805-4
[18] Addessi, D. et al. Push ‘o ver: numerical simulation of the Castel di Lama pushover test through a force-based equivalent frame model. *Procedia Structural Integrity*, 2023, 44, pp. 536-543. https://doi.org/10.1016/j.prostr.2023.01.070
[19] American Society of Civil Engineers. FEMA 356. *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. USA: ASCE; 2000.
[20] American Society of Civil Engineers. FEMA P-2090. *Recommended Options for Improving the Built Environment for Post-Earthquake Reoccupancy and Functional Recovery Time*. USA: ASCE; 2021.
[21] Kirsch, U. *Structural Optimization*. 1st Edition, Springer Nature, 1993. https://doi.org/10.1007/978-3-642-84845-2
[22] Ghersi, A.; Marino, E.; Neri, F. A simple procedure to design steel frames to fail in global mode, In: *Proceedings of the 6th International Colloquium, Stability and Ductility of Steel Structures (SDSS'99)*, Dubina, D.; Ivanyi, M. (eds.). September 9-11, 1999, Timisoara, Romania. Elsevier; 1999, pp. 377-384.
[23] Computers and Structures, Inc. *CSI Analysis Reference Manual for SAP2000, ETABS, SAFE and CsiBridge*. Berkeley, California, USA; 2013.