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SEISMIC PERFORMANCE OF LIGHTWEIGHT STEEL FRAMES IN MEDIUM DUCTILITY CLASS IN THE FRAMEWORK OF 2ND GENERATION EUROCODE

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Abstract

For the first time the European seismic building code, i.e. Eurocode 8 Part 1-2 (prEN 1998-1-2) explicitly covers lightweight steel structures and allows their design according to three possible alternative ductility classes, i.e. low (DC1), medium (DC2), or high (DC3) ductility class. Almost all available studies focused their attention on lightweight steel structures designed according to the capacity design and their response is affected by the specific designated energy-dissipating mechanism and thus corresponding to the high (DC3) ductility class according to prEN 1998-1-2 definition. In some cases, also lightweight steel structures designed without any capacity design were covered by specific study, making available even for the low (DC1) ductility class some background documents. The definition of medium (DC2) ductility class lightweight steel frame systems is a novelty introduced by prEN 1998-1-2 and consequently no solid research and extensive studies are available at the time of publication. Therefore, with the aim to try to overcome this gap, this study investigates the seismic response of lightweight steel structures designed according to the rules defined in prEN 1998-1-2 for medium (DC2) ductility class systems. In particular, the research involves the design of case study buildings according to prEN1998-1-2, which incorporates simplified design procedures to prevent brittle failure mechanisms. Then, the effectiveness of rules given by prEN1998-1-2 are investigates through non-linear analyses performed on the designed buildings, assessing their capacity to meet the intended seismic performance criteria.

Keywords: Seismic Performance, Medium Ductility Class (DC2), Lightweight Steel Frames, Nonlinear Analysis.

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1 INTRODUCTION

The ongoing revision of the Structural Eurocodes, mandated by the European Commission through M/515, represents a significant effort to address the criticalities identified in the past fifteen years and to incorporate the latest scientific advancements [1-8]. In this context, Eurocode 8 [9], which provides essential recommendations for the seismic design of structures, has undergone extensive modifications. The second-generation Eurocode 8 introduces a revised framework, particularly in prEN 1998-1-1:2024 [10], which defines seismic actions, and prEN 1998-1-2:2024, which focuses on buildings [11]. One of the most relevant innovations of this revision is the redefinition of ductility classes, replacing the previous classification (DCL, DCM, and DCH) with three new categories: DC1, DC2, and DC3. This modification aims to clarify the distinction between different levels of structural ductility and to resolve inconsistencies in the assignment of behavior factors (q) for various structural typologies.

Unlike in Europe, where lightweight steel structures were not explicitly covered in the seismic code before this revision, North America has long had seismic design provisions for cold-formed steel systems. The North American standard AISI S400 (2020) provides specific seismic design rules for lightweight steel buildings in the United States, Mexico, and Canada [12]. This standard covers several lateral force-resisting systems (LFRSs), such as cold-formed steel shear walls with different sheathing materials and strap-braced wall systems. AISI S400 defines a single level of ductility for most systems, except for conventional strap-braced walls, which follow a non-dissipative approach. This contrasts with the newly introduced prEN 1998-1-2, which distinguishes between three ductility classes and explicitly includes lightweight steel structures in the European seismic framework.

Among the notable advancements introduced in prEN 1998-1-2, the definition of an intermediate, medium-ductility (DC2) class for lightweight steel frames represents a novel concept. Until now, research on the seismic behavior of such systems has primarily focused on high-ductility (DC3) structures designed according to the capacity design approach, while studies addressing non-dissipative (DC1) systems remain relatively scarce. However, at the time of publication, no extensive research has been conducted on DC2 lightweight steel structures, leaving a significant gap in the literature.

This study aims to bridge this knowledge gap by investigating the seismic response of lightweight steel frames designed according to the new DC2 classification introduced in prEN 1998-1-2 [10]. The research involves the design of case study buildings following the revised code provisions, which incorporate simplified procedures to prevent brittle failure mechanisms. Subsequently, the effectiveness of these design rules is assessed through nonlinear analyses, evaluating the ability of the designed structures to meet the intended seismic performance criteria.

The study is structured as follows: the first part presents the design philosophies adopted in the new Eurocode, with a specific focus on the design rules for lightweight steel frames with gypsum sheathing designed in DC2 and DC3. Subsequently, a set of case studies is introduced, detailing the design approach adopted for the DC2 buildings. Finally, numerical models are developed, and the results of pushover analyses are presented to assess the seismic performance of the designed structures.

2 LIGHTWEIGHT STEEL SHEAR WALLS WITH GYPSUM SHEATHING IN THE FRAMEWORK OF 2TH GENERATION OF EUROCODE 8

2.1 Lightweight steel shear walls with gypsum sheathing

The revised Eurocode 8 introduces lightweight steel shear walls with gypsum sheathing as a new lateral force-resisting system (LFRS), addressing a gap in previous regulations. These walls rely on sheathing connections as the primary energy-dissipating components, ensuring ductile behavior through capacity design principles.

A typical gypsum-sheathed shear wall consists of a cold-formed steel (CFS) frame, gypsum panels, screw connections, and anchorages. The steel frame, made of CFS studs and tracks, provides the primary structural support. Gypsum panels are attached to the frame using screws, which act as the primary dissipative elements. The shear resistance of the wall depends on the weakest failure mode among sheathing connections, panel shear strength, stud buckling, and anchor failures. In particular, the design resistances of each component in a lightweight steel shear wall with gypsum sheathing can be derived as follows:

- Sheathing connections resistance ($R_{c,Rd}$): Determined based on the shear resistance of individual screw fasteners, which can be obtained experimentally and adjusted using partial safety factors as specified in Eurocode provisions [13].
- Steel frame resistance ($R_{f,Rd}$): Governed by the buckling resistance of the chord studs in compression, calculated according to EN 1993-1-3 for cold-formed steel members [14].
- Gypsum panel shear resistance ($R_{p,Rd}$): Based on the in-plane shear strength of the gypsum panel, considering factors such as thickness, material properties, and safety coefficients, as per EN 1995-1-1 [15].
- Tension anchor resistance ($R_{hd,Rd}$): Obtained from manufacturer data, ensuring compliance with design requirements for anchorage strength.
- Shear anchor resistance ($R_{sa,Rd}$): Derived from manufacturer-provided values, taking into account anchor spacing and shear load transfer requirements.

These resistances are used within the capacity design framework to ensure that energy dissipation occurs primarily through the intended failure mode (sheathing connections), while non-dissipative components exhibit adequate overstrength to prevent premature collapse.

2.2 General design requirements

According to prEN 1998-1-2, lightweight steel shear walls with gypsum sheathing must comply with specific geometrical and mechanical requirements to ensure adequate seismic performance for both DC2 and DC3 buildings. The main constraints relate to the gypsum panels, steel framing elements, and sheathing screws.

The gypsum panels used for sheathing must have a minimum thickness of 12.5 mm, and their width must not be less than 300 mm. The cold-formed steel (CFS) studs and tracks must have a thickness of at least 0.9 mm. Additionally, the yield strength of steel members varies based on their thickness: for elements with a thickness below 1.4 mm, the yield strength must be at least 220 MPa, whereas for members with a thickness of 1.4 mm or more, a minimum yield strength of 320 MPa is required. The spacing of the steel framing elements must not exceed 625 mm.

Regarding the sheathing screws, their nominal diameter must be at least 4.2 mm, with a minimum head diameter of 7.2 mm. The spacing of these fasteners along panel edges must range between 50 mm and 150 mm, while the distance from the screw center to the edge of the gypsum panel must be at least 12.5 mm.

These requirements ensure that the shear walls meet the capacity design principles outlined in the Eurocode, enabling a controlled energy dissipation mechanism while preventing premature failures in non-dissipative components.

2.3 Design of the dissipative component

For CFS shear walls with gypsum sheathing, the dissipative component is represented by the sheathing connections, meaning that their strength determines the overall design resistance of the wall. According to prEN 1998-1-2 (CEN 2024), the shear strength of these connections ($F_{c,Rd}$), must be determined exclusively through experimental testing, as no analytical methods are provided in the code. Once the shear connection resistance is obtained, the wall's design resistance, $R_{c,RD}$, can be derived using approaches originally developed for timber shear walls. However, it should be noted that these design rules are experience-based and have not undergone specific validation. Regarding the seismic design parameters, prEN 1998-1-2 assigns a behaviour factor (q) of 1.7 for DC2 structures and 2.0 for DC3 structures. While the DC3 value has been validated through extensive studies applying the FEMA P695 methodology [16], no specific studies have been conducted to confirm the applicability of the q value for DC2 buildings.

2.4 Overstrength requirements

The concept of overstrength in prEN 1998-1-2 [11] differs between DC2 and DC3 class structures, both in its application and in its physical meaning. While in DC3 structures the overstrength requirement is directly linked to the actual strength capacity of the dissipative elements, in DC2 structures it serves as a simplified method to amplify the seismic action without a direct link to the capacity of the LFRS dissipative component.

For DC2 class structures, the overstrength is implemented through a seismic action magnification factor (Ω) applied in Eq. (1):

$$E_{Ed} = E_{Ed,G} + \Omega E_{Ed,E} \tag{1}$$

Here, Ω is simply a scaling factor used to amplify the seismic action applied to the non-dissipative components of the LFRS. This approach does not derive from the actual strength of the dissipative zones but rather from a simplified assumption aimed at ensuring a sufficient margin of safety. The overstrength factor is set to 1.3 for CFS shear walls with gypsum sheathing.

In contrast, for DC3 class structures, the overstrength requirement is physically justified as it is based on the actual capacity of the dissipative elements. It is determined using Eq. (2):

$$E_{Ed} \geq E_{Ed,G} + 2E_{Rc,Ed} \tag{1}$$

where $E_{Rc,Ed}$ represents the action effect due to the design resistance of the sheathing connections. This means that in DC3, the overstrength factor of 2.0 is not just a simplification, but it directly reflects the real strength hierarchy between dissipative and non-dissipative elements.

In summary, while DC3 overstrength ensures that non-dissipative elements are designed based on the actual resistance of the dissipative components, in DC2 the overstrength factor is merely a simplified approach to increase the design action, without a strict correlation to the real strength of the system.

3 DESIGN OF THE CASE STUDY

3.1 Selected shear walls

The seismic design of Lightweight Steel (LWS) buildings involves selecting appropriate shear wall configurations to resist lateral forces, evaluating their design strength, and determining the total number of walls required. This is achieved by dividing the design base shear by the design strength of a single shear wall.

However, due to the lack of reference test data for shear walls meeting only DC2 Class requirements, it was necessary to derive a DC2-compliant wall by modifying an existing DC3 Class configuration. This was done by downgrading the DC3 Class walls through modifications to the chord studs and hold-down devices, ensuring that the resulting wall satisfies the DC2 Class criteria.

In this research, the reference DC3 shear walls were selected based on results from a series of experimental tests conducted at the University of Naples Federico II. From these tests, two shear wall configurations, named S-02 and S-03, were identified as meeting the DC3 Class requirements [17].

The downgrade process involved:

- Replacing the chord studs with single C-shaped sections of reduced cross-section size, selecting the minimum dimensions permitted by mechanical and geometrical constraints.
- Using hold-down devices with lower strength and stiffness with respect to the original DC3 configuration.

By implementing these modifications, new walls were developed and designated as S-02_mod and S-03_mod, which no longer fulfilled the DC3 design rules but complied with the DC2 Class requirements.

The original S-02 and S-03 walls, from which these modified versions were derived, had dimensions of 2700 mm in height and 2400 mm in width. Their sheathing connection design resistance ($R_{c,RD}$) remained unchanged in the modified versions, with values of 26.8 kN for S-02 and 53.6 kN for S-03. The primary difference between the two configurations was the screw spacing along the perimeter, which was 150 mm for S-02 and 75 mm for S-03.

Thus, the next section DC2 Class shear wall configurations used in the archetypes design were obtained by systematically reducing the strength and stiffness of key structural elements in an existing DC3 Class configuration, ensuring compliance with the requirements of the DC2 ductility class.

3.2 Case studies main features

The selection of archetype buildings was carried out by considering a set of key variables that influence the seismic response and design requirements. In this study, a single floor plan was adopted (see Figure 1), while variations were introduced in terms of seismic intensity, flooring system, and building height.

The seismic intensity was considered at two different levels, representing medium and high seismic hazard conditions. In particular, the new Eurocode 8 Part 1-1 [10] imposes a limitation on the use of LWS systems designed according to DC2 for values of S_δ (maximum acceleration of the elastic spectrum) equal to 7.5 m/s². This limit restricts the applicability of these systems in regions with higher seismic intensities. Therefore, one of the seismic intensity levels chosen for this study corresponds to this limit, with a value of S_δ =7.5 m/s^2, representing the high seismic hazard condition. The other intensity level is set lower, at S_δ =6.5 m/s^2, corresponding to a medium seismic hazard condition. These two levels were selected to evaluate the performance of the system under different seismic conditions and ensure a comprehensive analysis of its behavior across a range of expected seismic hazards.

Two flooring systems were investigated: type 1 (representative of a composite floor system) and type 2 (lightweight floor system). The choice of flooring system significantly affects the seismic mass and, consequently, the dynamic properties of the structure. In particular, the dead loads on floors vary based on the selected flooring system, with values of 1.30 kN/m² for type 2 solutions and 1.80 kN/m² for type 1. Additional loads include 0.10 kN/m² for ceilings and 0.80 kN/m² for vertical partitions. For the roof, the dead loads are 1.40 kN/m² for type 2and 2.30 kN/m² for type 1, with an additional 0.10 kN/m² due to ceilings. Finally, a distributed load equal to 2.70 kN/m was considered for the external façade. For the archetype buildings, residential occupancy is considered. The live load for residential buildings is set equal to 2.0 kN/m². Additionally, the roof is assumed to be accessible, with a live load equal to that of the floors, also set at 2.0 kN/m².

Typically, the total height of the LWS framed building was assumed to be between 3.0 and 12.0 metres. In this work, the selected archetypes correspond to two-storey configurations, considering a uniform height between floors (h_i) equal to 3 m.

By combining these variables, a set of eight archetype buildings was defined to comprehensively assess the influence of different design choices on the seismic performance of the considered structural system.

The seismic design of the archetype buildings follows the Lateral Force Method prescribed in EN 1998-1 [9], as all archetypes satisfy the regularity criteria in both plan and elevation. The behaviour factor q equal to 1.7 was used in the design process according to [11]. Moreover, to account for accidental torsional effects, the design includes an amplification of seismic forces, considering a a torsional amplification factor (δ) equal to 1.3.

The primary lateral force-resisting system consists of cold-formed steel (CFS) shear walls sheathed with gypsum fibreboard. The seismic design process involves determining the necessary wall configurations and lengths to resist the design base shear. Two previously defined shear wall configurations, S-02_mod and S-03_mod, each with a width of 2.4 meters, were adopted for the archetypes.

The archetypes matrix is summarized in Table 1.

Table 1: Archetypes main features

Building ID	S_δ [m/s^2]	Flooring system	Type of Shear walls	Shear wall for each direction
A-01	7.5	Type 1	S-03_mod	16
A-02	6.5	Type 1	S-03_mod	14
A-03	7.5	Type 2	S-03_mod	12
A-04	6.5	Type 2	S-02_mod	20

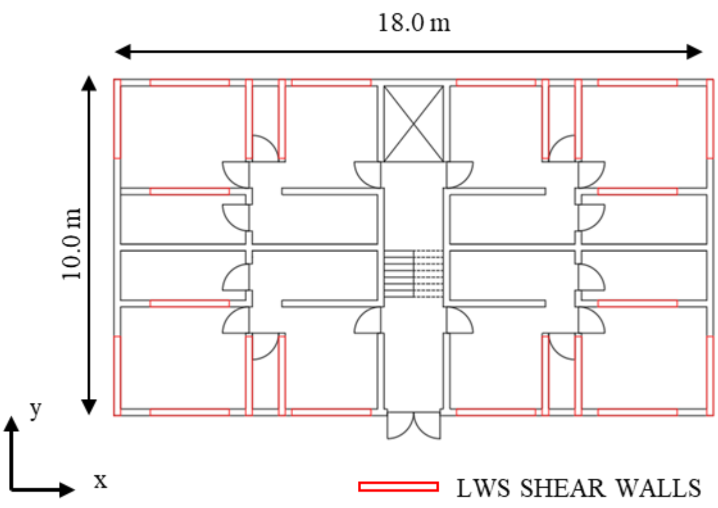

Figure 1: Archetype A-03 in-plane shear walls distribution

4 SEISMIC ASSESSMENT

4.1 Modelling assumptions

The modelling process began with the calibration of the numerically simulated shear walls tested experimentally, namely S-02 and S-03 [17]. Indeed, the performance of many steel systems is often evaluated experimentally due to the complexity of their behavior under seismic loading [18-23]. A simplified numerical model was adopted for individual walls, where equivalent diagonal truss elements were used to capture the nonlinear behavior of the shear walls, as observed in the experimental tests (see Figure 2). The chord studs were also modeled using truss elements, assigned an elastic uniaxial material with compression and tension strain limits derived from the design strengths according to Eurocode 3 [14]. Hold-downs were modeled using nonlinear zero-length links with rigid compression behavior, while the tensile stiffness and resistance were defined based on the test results.

The downgraded S-02_mod and S-03_mod numerical models were derived from the original ones by modifying the properties of the chord studs and hold-downs, replacing them with those obtained from the design process of the DC2 building archetypes. Accordingly, the strain limits were adjusted and monitored during pushover analyses to check for potential brittle failures.

More information on the simplified numerical model of the shear walls can be found in [16].

For each archetype, a three-dimensional numerical model was developed, including the most relevant structural components that significantly influence the global seismic response of the building. The structural framework of a typical cold-formed steel (CFS) building consists of wall elements, such as studs and wall tracks, connected to floor elements, including floor tracks and joists. Floor diaphragms were assumed to exhibit rigid in-plane behavior due to their high in-plane stiffness. In addition to seismic forces inducing lateral displacements, gravity loads contribute to global lateral deformations due to P-delta effects. These effects were also accounted for in the seismic force-resisting system. Finally, the cyclic hysteretic behavior of individual walls was not explicitly calibrated in this study, as the seismic assessment was conducted in a preliminary manner using only nonlinear static analyses.

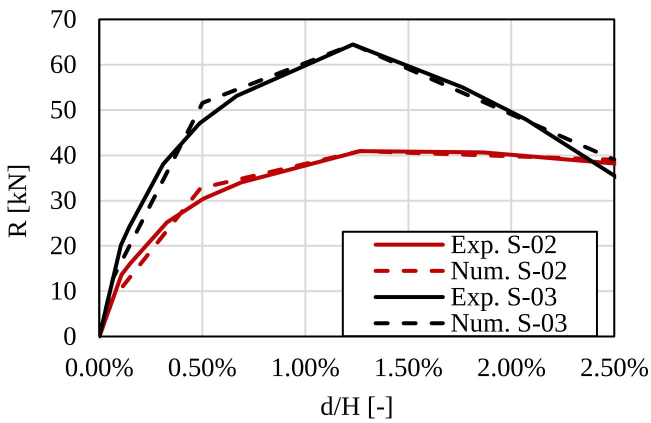

Figure 2: Numerical vs Experimental behaviour of the selected shear walls

4.2 Non-linear static analyses

Non-linear static analyses, commonly referred to as "pushover" analyses, were conducted along both the X and Y directions of the considered structures. These analyses were performed under a constant gravity load combination, specifically , while a monotonically increasing lateral load was applied. In accordance with the provisions outlined in the European standard [9-11], two distinct lateral load patterns were adopted: (i) the "1M" pattern, which approximates the inertial forces associated with the fundamental vibration mode in the direction under consideration, and (ii) the "UA" pattern, which represents uniform unidirectional lateral accelerations, aiming to capture the inertia forces that may develop in a potential soft-story failure mechanism.

The structural performance of the analyzed buildings was assessed in compliance with the Significant Damage (SD) limit state, as prescribed by [10]. The seismic hazard levels were defined based on the European code provisions, considering a design earthquake with a return period of 475 years, corresponding to a 10% probability of exceedance within a 50-year reference period for the SD limit state.

The compliance criteria for each limit state are summarized in Table 5.3. Specifically, for the Significant Damage limit state (SD), the global performance requirement dictates that the maximum interstory drift must not exceed 1.0% of the story height [10-11]. Additionally, local performance checks were conducted to monitor the potential brittle failure of non-dissipative elements. This evaluation entailed comparing the seismic demand with the corresponding capacity, particularly focusing on buckling resistance and tensile strength of chord studs, as well as the tensile capacity of anchor connections.

For the sake of brevity, only the X-direction pushover curves obtained using a model load pattern are shown in Figure 3.

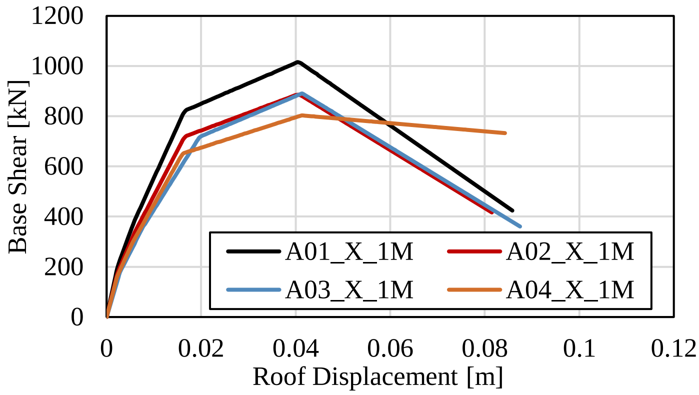

Figure 3: Archetypes pushover curves

5 RESULTS

The safety verification of the analyzed structures was performed following the N2 method [24]. This procedure involves deriving a bilinear capacity curve from the pushover analysis results, representing an equivalent single-degree-of-freedom (SDOF) system.

The extracted bilinear capacity curves were subsequently mapped onto the Acceleration-Displacement Response Spectrum (ADRS) format and compared with the elastic response spectrum. The intersection between the bilinear pushover curve and the elastic spectrum allows for the determination of the seismic demand within a displacement-based framework [25].

Tables 2 provide a summary of the computed safety ratios. These ratios are defined as the relationship between the demand displacement ($D_{Ed\text{-}SD}$) and the corresponding SDOF displacement associated with the prescribed compliance criteria, namely $D_{Rd\text{-}1.0\%}$ for global drift limitations and $D_{Rd\text{-}brittle}$ for local brittle failure checks.

Table 2: Seismic assessment results

Load pattern	Archetype ID	$D_{ed\text{-}SD}$	$D_{Rd\text{-}1\%}$	$D_{Ed\text{-}SD}/D_{Rd\text{-}1\%}$	$D_{Rd\text{-}Brit}$	$D_{Ed\text{-}SD}/D_{Rd\text{-}brit}$
-	-	[m]	[m]	-	[m]	-
1M_X	A-01	0.017	0.031	0.56	/	/
UA_X	A-01	0.020	0.036	0.57	/	/
1M_Y	A-01	0.016	0.031	0.54	/	/
UM_Y	A-01	0.020	0.036	0.57	/	/
1M_X	A-02	0.017	0.031	0.53	/	/
UA_X	A-02	0.020	0.036	0.56	/	/
1M_Y	A-02	0.017	0.031	0.55	/	/
UM_Y	A-02	0.020	0.036	0.56	/	/
1M_X	A-03	0.018	0.031	0.57	/	/
UA_X	A-03	0.023	0.037	0.61	/	/
1M_Y	A-03	0.022	0.033	0.69	/	/
UM_Y	A-03	0.029	0.036	0.78	/	/
1M_X	A-04	0.019	0.031	0.60	0.023	0.82
UA_X	A-04	0.022	0.036	0.61	0.026	0.84
1M_Y	A-04	0.021	0.030	0.69	0.024	0.88
UM_Y	A-04	0.025	0.036	0.69	0.028	0.87

The table provides an assessment of displacement demands relative to structural capacity, distinguishing between a global drift limit of 1% and brittle failure of the chord. In most cases, the demand remains well below the 1% drift threshold, indicating that the structures maintain sufficient capacity under the applied loading conditions. Brittle failure governs the capacity only in A-04, where the displacement demand approaches but does not exceed the failure threshold, leaving a safety margin. The data suggests that while brittle failure is a consideration, the overall structural response remains within acceptable limits at significant damage limit state

6 CONCLUSION

This study presents a preliminary assessment of the new design provisions introduced in the second generation of Eurocode 8 for DC2 buildings. The purpose of DC2 is to provide simplified design rules for structures with medium ductility. These provisions were evaluated by designing structural archetypes using a behavior factor of 1.7 and applying the simplified amplification rules for non-dissipative elements, such as chord studs and hold-downs.

The archetypes were designed with shear walls braced with gypsum panels, whose mechanical properties were taken from the literature. Since these panels were originally designed according to DC rules, a downgrading procedure was applied to ensure compatibility with DC2 requirements. Numerical models were calibrated, and nonlinear static analyses were conducted as part of the preliminary assessment.

The results indicate good seismic performance, with demand-to-capacity ratios remaining below 1, confirming that a behavior factor of 1.7 is appropriate. Regarding overstrength, in some cases, structural failure is governed by the brittle failure of the chords. However, the structures still exhibit displacement capacity before reaching failure, indicating the presence of safety margins.

For a more comprehensive assessment of the behavior factor, further studies are needed, considering a wider range of archetypes with variations in the number of stories and building layout. Additionally, more rigorous methodologies, such as the FEMA P695 procedure, should be applied to provide a more robust evaluation of the structural response and overstrength factors.

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