Seismic Overstrength on Special Steel Moment Frame due to drift limits

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Abstract*– This study investigates the nonlinear seismic behavior of steel Special Moment Frames (SMFs) designed in accordance with the Costa Rican Seismic Code of 2010 (CSCR 10). The main objective is to evaluate the overstrength acquired by the structure due to drift limits in the Code and whether the ductility assigned during seismic design is effectively developed under seismic demand. Two symmetric prototype structures—a three story and a five-story steel frame—were designed using W shaped members and analyzed through nonlinear static (pushover) analysis. The influence of interstory drift limits, building height, and second order P–Δ effects on overstrength and intrinsic ductility was examined. Results show that frames designed in strict compliance with CSCR 10 drift limits remain predominantly elastic, exhibiting significant overstrength and limited plastic hinge formation. Taller structures displayed higher stiffness and reduced ductile capacity compared to lower rise frames. When drift limits were relaxed and P–Δ effects were explicitly considered, increased inelastic behavior consistent with SMF design philosophy was observed. The findings suggest that current drift control requirements in CSCR 10 increase building overstrength and reduce inelastic behavior.

Keywords-- Steel Frames, Seismic Design, Steel structure Overstrength, Steel structures Overstrength.

I. Introduction

Steel Special Moment Frames (SMFs) are widely recognized as one of the most ductile seismic-resistant structural systems available for buildings located in high seismicity regions. Their performance relies on the development of stable plastic mechanisms that allow significant energy dissipation without collapse. Current seismic design philosophies assume that, under severe earthquake excitation, these structures will enter the inelastic range, forming plastic hinges primarily in beams while columns remain largely elastic, consistent with the strong-column–weak-beam design principle.

In Costa Rica, seismic design is regulated by the Costa Rican Seismic Code of 2010, Revision 2014 (CSCR-10), which establishes limits on lateral displacements, interstory drifts, and structural stability. While these limits aim to protect life safety and minimize damage to non-structural components, it can increase steel overstrength to buildings. This doesn´t mean that drift limits should be neglected or relaxed. Roeder [1] demonstrated that drift control is intrinsically linked to the slenderness of the elements. If drift limits are too permissive, deeper profiles are required, which increases susceptibility to local and lateral-torsional buckling, thereby reducing global ductility.

Gupta and Krawinkler [2] analysed steel frames of various heights and determined that strict drift control is fundamental to avoid dynamic instability collapse, especially at levels where the formation of "soft-story" mechanisms is critical.

Engelhardt and Sabol [3] investigated Reduced Beam Section (RBS) connections. Their findings suggest that by limiting elastic drift, the integrity of the panel zone is protected, allowing ductility to concentrate in a structural "fuse," at the beam location near the column and optimizing the strength-to-weight ratio of the system.

Elkady and Lignos [4] conducted an exhaustive study on wide-flange steel columns. They demonstrated that the ductility of SMFs is fundamentally limited by the local buckling of columns under high axial loads. Their findings suggest that drift limits should be more stringent when axial load ratios are significant to prevent sudden loss of lateral strength.

Flores et al. [5] analysed the influence of composite slabs on ductility. They found that while the slab increases initial lateral strength, it reduces the curvature ductility of the beam by constraining plastic rotation. This can lead to an unexpected concentration of drift in lower stories (soft-story mechanism) not predicted by bare-frame models.

Zhu and Zhang [6] proposed a probabilistic design framework demonstrating that optimized Reduced Beam Section (RBS) connections can maintain global stability (P-Δ) even at drifts as high as 0.05 rad, provided that cyclic stiffness degradation is strictly controlled.

According to all these studies, drift limits are an important consideration. On the other hand, a thesis conducted para Aguero [7] for the master's degree in the Costa Rica University. The thesis studies the seismic performance of regular steel moment-resisting frames designed under the Costa Rican Seismic Code (CSCR) 2002 and 2010. There were evaluated through non-linear finite element analysis (FEA) and alternative analytical methods. The results indicate that the seismic design of these structures is predominantly governed by serviceability drift limits rather than strength requirements, which inherently provides a significant degree of structural overstrength. FEA simulations of WUF-W connections demonstrated that their rotational capacity consistently exceeds the demands imposed by scaled seismic records, even when considering the more flexible designs resulting from the CSCR 2010 provisions. For low-rise buildings (3 levels), the overstrength is particularly pronounced, as connections exhibited minimal inelastic incursions under design-level events, suggesting that the application of special moment frame (SMF) requirements may lead to conservative and less economical designs compared to intermediate moment frames

(IMF). Furthermore, spectral capacity analyses showed a ductility demand of unity across all models, confirming that the overstrength derived from stiffness-based design ensures elastic behaviour under the code's elastic design spectrum.

The primary contribution of this study, in addition to validating Aguero's findings, is to is to assess whether the ductility assigned at the design stage is effectively developed under seismic demand, and to evaluate the influence of lateral drift control, building height, and second-order (P–Δ) effects on overstrength and intrinsic ductility. Two building configurations—three-story and five-story symmetric steel frames—are designed and analyzed to provide a comparative evaluation of code-based design assumptions versus actual nonlinear structural performance.

II. METHODOLOGY

A. *Structural Modeling and Desing*

Two prototype steel buildings were designed with identical plan configurations consisting of five bays in both the X and Y directions, each with a bay length of 6.0 m. The difference between the models lies exclusively in the number of stories: one structure contains five levels, while the second consists of three levels. The first-story height is 4.5 m, and all upper stories have a height of 3.5 m. Fig 1. shows the buildings' geometry. Lateral Force Resisting System and gravity frames are shown in Fig. 2 for the 3-story building and Fig. 3 for the 5-story building. The secondary beam direction is along the Y direction, as shown in Fig. 1, while the X frames serve as the load frames. The following results will focus on the X load frames.

All primary lateral-force-resisting elements were designed as Special Moment Frames (SMFs) using W-shaped steel sections fabricated from ASTM A992 steel. Gravity frames were modeled separately and did not contribute to lateral resistance. The bases of columns were assumed to be fully fixed, neglecting soil-structure interaction effects, in accordance with the scope limitations of the study.

B. *Seismic Design Procedure*

The seismic design followed the provisions of the CSCR-10, using the equivalent static force method (ELF) specified in Chapter 7 [9] for the initial design, with subsequent verification using the modal response spectral analysis (MRSA) method. The buildings were assumed to be in Seismic Zone III, on Soil Type S3, with normal importance and regular configuration in both plan and elevation.

Lateral loads were calculated using code-defined parameters, including:

- Effective peak ground acceleration, 0.36
- Importance factor, 1.00
- Assigned global ductility (μ), 6.0
- Overstrength factor (SR), 2.0

Interstory drift limits from Table 7.2 of CSCR-10 [9] were enforced for part of the study. For comparative purposes, additional structural models were created by relaxing these drift limits while explicitly accounting for P–Δ effects according to Section 7.8 of the code and Chapter C of the Specification for Structural Steel Building of the American Institute of Steel Construction (ANSI/AISC 360-22) [8]. As a result, four distinct structural models were analyzed:

1. Five-story building complying with drift limits
2. Three-story building complying with drift limits
3. Five-story building exceeding drift limits
4. Three-story building exceeding drift limits

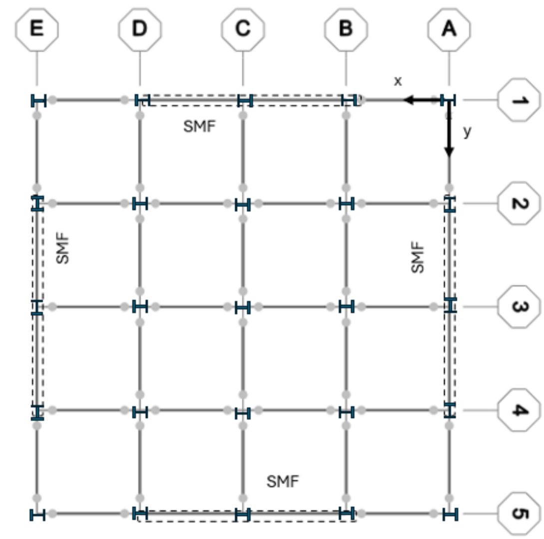

Fig. 1 Plan View for three-story and five-story building and Special Moment Frame (SMF) locations

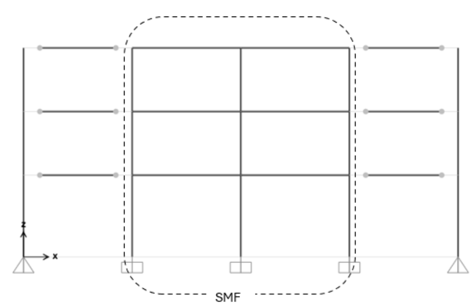

Fig. 2 Elevation for three-story and Special Moment Frame (SMF) locations

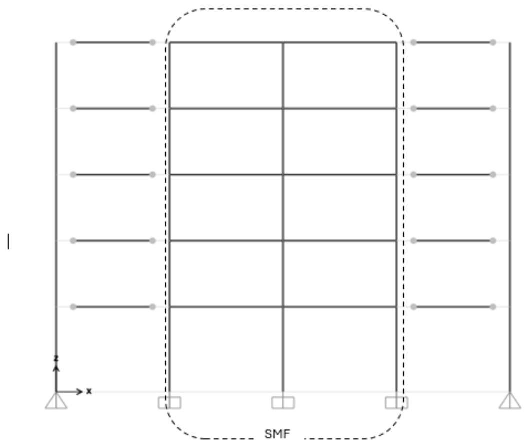

Fig. 3 Elevation for five-story and Special Moment Frame (SMF) locations

The design results for each case are presented in Tables I to IV. When drift limit control is utilized, the section size increases, resulting in a demand-capacity (D/C) ratio that is significantly lower than 1.0. This indicates that the structure has acquired a considerable amount of overstrength. When drift consideration is not considered, it is possible to obtain a D/C ratio closer to 1.0.

SECTIONS FOR FIVE STORY BUILDINGS WITH DRIFT LIMIT

Level	Beams	Columns
1st	W18x55	W40x215
2nd	W18x55	W40x215
3rd	W18x55	W40x215
4th	W18x55	W24x146
5th	W18x55	W24x146

TABLE II
SECTIONS FOR FIVE STORY BUILDINGS WITH NO DRIFT LIMIT

Level	Beams	Columns
1st	W12x22	W21x83
2nd	W12x22	W21x83
3rd	W12x22	W21x83
4th	W12x19	W12x50
5th	W10x17	W12x50

TABLE III
SECTIONS FOR FIVE STORY BUILDINGS WITH DRIFT LIMIT

Level	Beams	Columns
1st	W18x55	W40x215
2nd	W18x55	W40x215
3rd	W18x55	W24x146

Drift ratios for this first design are shown in Fig 7 for the 5-story building and in Fig 8 for the 3-story building. A drift ratio limit of 0.02 was used in compliance with table 7.2. of the CSCR-2010 for normal use buildings.

TABLE IV
SECTIONS FOR FIVE STORY BUILDINGS WITH NO DRIFT LIMIT

Level	Beams	Columns
1st	W12x22	W21x83
2nd	W12x22	W21x83
3rd	W10x17	W12x50

C. Nonlinear Analysis Procedure

Nonlinear static (pushover) analysis was carried out using SAP2000. Plastic hinges were assigned to beams and columns according to capacity design principles, ensuring that yielding was allowed where inelastic behavior was expected while preventing premature brittle failure modes.

The pushover analysis was conducted using a monotonically increasing ELF load pattern, representing the fundamental mode of vibration. The control point for displacement was located at the roof level. The analysis progressed until a performance level was achieved, in this case, collapse prevention, this was done for the constructions of the pushover curves. Nevertheless, Life Safety performance level was used as a work limit in this study.

Fig. 4 show unitary beam moment-rotation graphs. This graph represents the moment, and rotation divided by a scaling factor (SF).

The scaling factor is a numerical multiplier (typically between 0.5 and 2.0) applied to the properties of the hinge's deformation curve to adjust it to the specific conditions of the element or design. It is used to "scale" the entire load-deformation curve (or just parts of it, such as the yield moment or the ultimate rotation). By default, SAP2000 sets it to 1.0 for default hinges, but you can modify it to reflect factors such as overstrength, plastic hinge length, or shape factor.

Plastic hinges were assigned to the Moment Frame (MF) beams using the M3 property, following the SAP2000 conventions. Additionally, P-M2-M3 hinges were assigned to the columns to verify that no hinge formation occurs in those elements.

Location of the plastic hinges is assumed to form at column flanges. Welded unreinforced moment connection is used (WUF-F).

Acceptance criteria for plastic rotation are shown on Table V. The limits are set according to the Seismic Evaluation and Retrofit of Existing Buildings, ASCE 41-17 [10].

To evaluate seismic performance, capacity curves (base shear vs. roof displacement) were transformed into spectral acceleration vs. spectral displacement (Sa–Sd) format using the capacity spectrum method based on Seismic Design Procedure (ATC-40) [11]. These capacity curves were then superimposed with modified seismic demand spectra for direct comparison between demand and capacity, according to CSCR-2010, Chapter 7 [9].

The design spectrums used for de demand can be find in CSCR-2010, Chapter 5 [9], and includes de seismicity of Costa Rica´s ubication.

Second-order effects were incorporated through iterative P–Δ analyses, including stiffness reduction and the application of notional loads when required. Stability analysis was made to assess the significance of geometric nonlinearity on structural response.

III. RESULTS

A. Elastic vs Inelastic Response

The results reveal that the two structures designed in strict compliance with CSCR-10 drift limits remained predominantly elastic throughout the nonlinear analysis. Plastic hinge formation was minimal, and the global ductility developed by these structures was significantly lower than the ductility assigned during design. In several cases, less than 30% of the expected inelastic capacity was mobilized before reaching seismic demand levels. Fig. 5 shows the Spectral Capacity vrs Spectral Demand of the structures with drift limits.

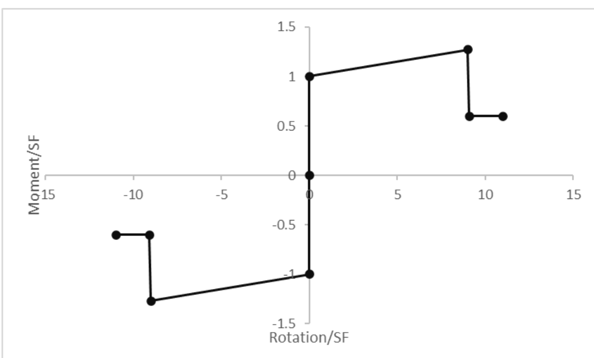

Fig. 4 Unitary Moment-Curvature graph for beams

TABLE V
ACCEPTANCE CRITERIA OF PLASTIC ROTATION /SF ACCORDING TO ASCE41-17

Level of Performance	Rotation/SF
Immediate Occupancy	2.25
Life Safety	9
Collapse Prevention	11

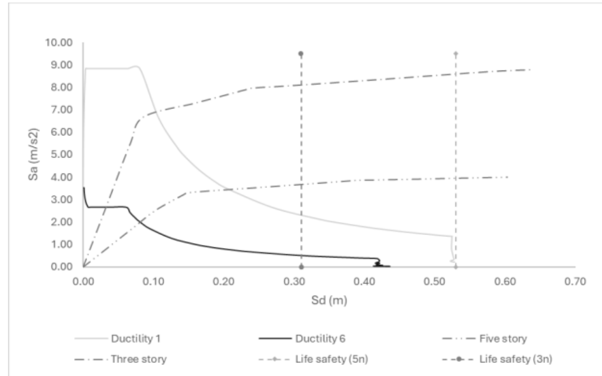

Fig. 5 Spectral Capacity versus Spectral Demand for buildings. With drift limits

This behavior indicates the presence of substantial overstrength, resulting from the displacement control requirements that limit lateral deformations and prevent full plastic mechanism development.

This can be obtained from the point where the demand for a ductility of 6 intersects the spectral capacity curve. At this point, no inelastic hinge has formed, although the spectral capacity curve reflects the inelastic capacity of the building.

This result is the same for both the 3-story and 5-story buildings. The buildings gain significant overstrength due to drift limits, preventing them from deforming enough to reach inelastic behavior.

When limit on drift is not taken into account, the spectral demand with a ductility of 6, cuts the spectral capacity curve at a point where inelastic behavior is achieved.

B. Influence of Building Height

Building height was found to be a key parameter influencing nonlinear behavior. Three-story frames exhibited higher overstrength and higher intrinsic ductility compared to five-story frames. Low rise structures demonstrated increased sensitivity to displacement restrictions, leading to stiffer designs that further suppressed inelastic response. The period for the first mode in X direction is 0.69 and in Y is 0.59 s.

In contrast, five-story buildings showed less capacity for ductile behavior, and the overstrength acquired due to drift limitations is smaller. The period for the first mode in X direction is 1.27 and in Y is 1.04 s.

Differences in the first mode periods indicate that the shorter building has gained more stiffness and overstrength compared to the taller building.

C. Effect of Drift Control Relaxation

When drift limits were relaxed and P–Δ effects explicitly considered, both structures exhibited substantially increased displacement capacity and plastic hinge development. In these cases:

- Plastic hinges formed primarily in beams, consistent with SMF design philosophy.

- Higher intrinsic ductility values were achieved.
- Structural response aligned more closely with the theoretical seismic energy dissipation mechanisms assumed during design.

This increase in ductility was accompanied by higher sensitivity to second-order effects, particularly in the five-story structure.

The results of the drift limit relaxation for both buildings are shown in Fig. 9 and Fig. 10, represented as drift ratios. In both cases, the drift ratio exceeds the intentional limit of 0.002, as specified in Table 7.7 of CSCR2010 [9]. The flexibility of the structure can be assessed by examining the period of the first mode in the X direction. For the 5-story building, this period is 3.33 seconds, while for the 3-story building, it is 1.84 seconds. Both values are significantly higher than those estimated for buildings that comply with the drift limitation.

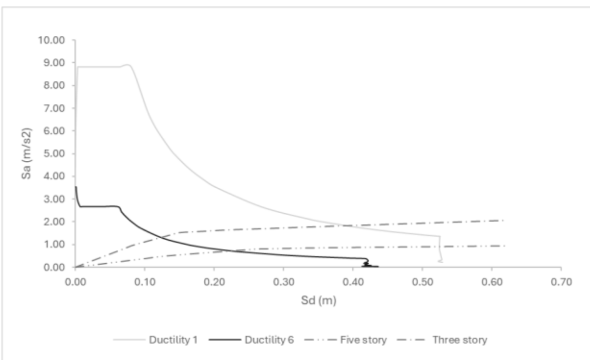

Fig. 6 Spectral Capacity versus Spectral Demand for buildings. No drift limits

D. P–Δ Effects and Stability

Second-order effects proved to be critical for models exceeding drift limits. As displacements increased, stability coefficients exceeded code thresholds, requiring stiffness reduction and the inclusion of notional loads. These effects notably increased interstory drifts and amplified internal forces, confirming the necessity of rigorous stability evaluation when displacement limits are relaxed.

Despite this, the structures did not exhibit global instability or collapse within the analyzed displacement ranges, suggesting that ductility-based performance may be achievable under controlled design refinement.

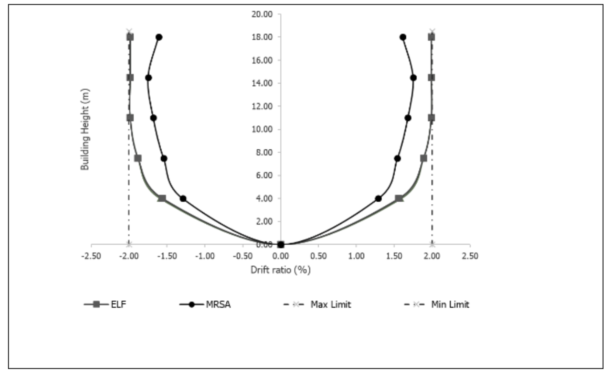

Fig. 7 Drift ratios for the 5-story building (with drift limits)

E. Calculation of Maximum Probable Ductility

This analysis is conducted for buildings that don´t satisfy the allowable **inter-story drift** limits. The structural ductility factor is calculated as the ratio between the ultimate displacement and the yield displacement, according to the following expression:

$$\mu = \frac{\Delta_u}{\Delta_y} \quad (2).$$

Where:

- Δ_u (Ultimate Displacement): Denotes the maximum inelastic displacement demand expected under the design seismic action.
- Δ_y (Yield Displacement)**:** Represents the displacement at the onset of structural yielding. For the purposes of this study, Δ_y is identified as the intersection point between the spectral capacity curve and the constant ductility line corresponding to $\mu = 6$.

The required parameters were derived from the numerical data and graphical representations provided in the preceding sections. The calculated ductility results are summarized as follows in table VI.

TABLE VI
MAXIMUM PROBABLE DUCTILITY

Building	Ductility	Condition
5-story	2.30	No drift limit
3-story	3.63	No drift limit
5-story	elastic	With drift limit
3-story	elastic	With drift limo

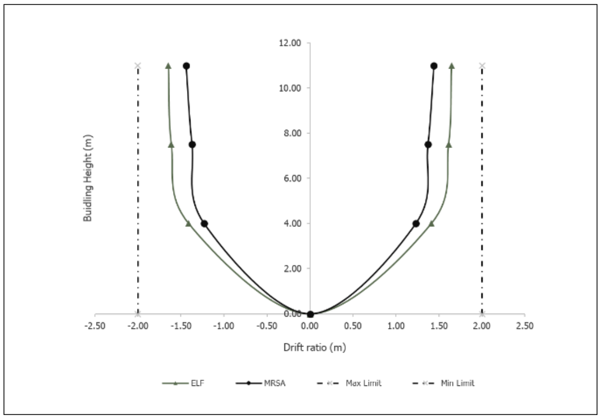

Fig. 8 Drif ratios for the 3-story building (with drift limits)

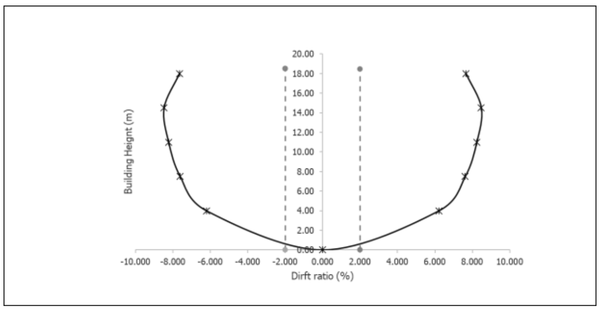

Fig. 9 Drif ratios for the 5-story building (with no drift limits)

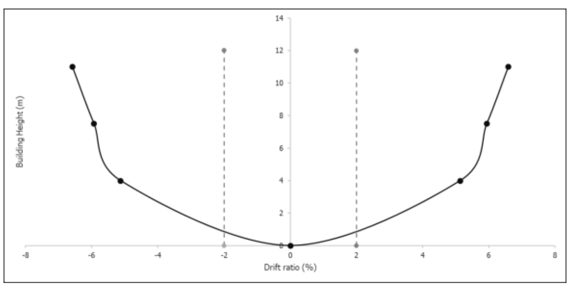

Fig. 10 Drif ratios for the 3-story building (with no drift limits)

IV. Conclusions

This study demonstrates that steel Special Moment Frames designed according to CSCR-10 exhibit significant overstrength, which prevents the effective development of ductility during seismic demand. The principal findings can be summarized as follows:

1. Displacement control requirements in CSCR-10 provide the structure of more resistance that the seismic demand required in steel SMFs, leading to elastic behavior even under severe seismic excitation.

2. The assigned ductility during design is not representative of actual structural performance, resulting in inefficient use of material and increased construction costs.

3. Building height amplifies overstrength effects, with low rise frames exhibiting more increment in overstrength taller ones-

4. Relaxation of drift limits, combined with explicit consideration of P–Δ effects, enables more realistic ductile behavior consistent with performance-based seismic design principles. Nevertheless, this doesn’t mean that a relaxation of drift should be recommended. Drif control are very important to control second order effects and damage on nonstructural elements.

5. Second-order effects must be carefully evaluated when allowing increased displacements, particularly in multi-story steel frames.

These findings suggest that future revisions of the Costa Rican Seismic Code could benefit from performance-based calibration of the overstrength factor for Steel SMFs, that is stated as 2 for all structures.

Nonlinear dynamic analysis is recommended for validation of these results as widening the cases to taller buildings.

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