An Analytical Study on the Impact of Beam Axial Forces on the Structural Response of Braced-Connected Columns in Steel Concentrically Braced Frames

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Abstract

In steel braced structures, different bracing configurations have a direct impact on the distribution of internal forces. Due to the presence of unbalanced forces in the bracing system, the axial forces developed in the beams of braced bays V and inverted V-(chevron) can significantly affect the seismic behavior of the structure. This axial force may increase the in-plane bending moments in the columns and compromise their stability. Accordingly, the present study investigates the effect of axial forces generated in the beams of special concentrically braced frames (SCBFs) and their impact on the connected columns. Structural models were developed for three building frames with 5, 10, and 15 stories using SAP2000 software, incorporating bracing configurations in bays V and inverted V-(chevron). Nonlinear pushover analysis has been employed as a method for evaluating the seismic behavior of these structures, and the results concerning axial force distribution in the beams and the resulting moments in the braced frame columns have been analyzed and compared.

The findings indicated that, at the target displacement, the axial forces in the braced bay beams can increase the moments in the columns by up to 5% of their flexural capacity, in addition to the existing axial loads.

Keywords: Special concentrically braced frame (SCBF), Chevron bracing, Beam unbalanced force, Pushover analysis.

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1- Introduction

The lateral resistance of buildings against external loads, including earthquakes, is provided through various structural systems. Among these, braced frames have been widely utilized since the early 20th century. Initially, these systems were implemented to resist wind loads; however, they were later developed to withstand seismic forces. Due to their high elastic stiffness and favorable load-bearing capacity, braced frames are considered effective solutions for resisting earthquake-induced lateral loads. These systems are typically classified into two categories: concentrically braced frames (CBFs), which exhibit rigid behavior, and eccentrically braced frames (EBFs), which provide more ductile performance. One commonly used configuration is the Chevron bracing system, which allows for more architectural flexibility, such as the inclusion of openings for doors, windows, and utility systems.
Despite these advantages, Chevron bracings are particularly vulnerable to buckling in compression members on the first floor, leading to damage concentration, reduced lateral stiffness, and strength degradation as nonlinear displacements increase. In general, owing to its inherently brittle response and vulnerability to compressive buckling, this system exhibits limited effectiveness in redistributing lateral forces. Empirical evidence from major seismic events (such as the 1994 Northridge and 1995 Kobe earthquakes)has underscored the inadequate performance of conventional bracing systems in mitigating nonlinear deformations and in averting the localization of structural damage [1–5]. In Chevron bracing systems, the compressive and tensile forces developed within the braces are directly transferred to the beam at their point of intersection. Under seismic design conditions, braces typically exhibit asymmetric behavior due to the disparity between their tensile and compressive strengths, the latter being significantly affected by brace buckling under compressive forces. In these bracing systems, under the application of lateral loads, and as long as both braces exhibit elastic behavior, the vertical components of the axial forces in the tensile and compressive braces counterbalance each other. However, once one of the members undergoes buckling under compression, the tensile force in the bracing system increases and surpasses the force in the compressive brace. In this case, the vertical components of these forces no longer counterbalance each other, resulting in significant vertical deformations in the beam. As lateral forces increase, bracing members act as fuses by entering nonlinear behavior through repeated buckling and yielding cycles, thus providing the required ductility. Meanwhile, other structural members such as beams, columns, and connections are ideally expected to remain in the elastic range. Nonetheless, due to the high axial forces resulting from brace buckling and yielding, the beam intersecting the brace may be subjected to substantial actual axial forces. If the beams and columns are not designed to accommodate such forces, they may enter nonlinear behavior prematurely, thereby compromising the intended performance of the system. Therefore, this requirement necessitates that a cross-braced beam must be capable of withstanding the substantial unbalanced force resulting from the difference between yield and buckling resistance, which typically accounts for 2 to 4 percent of the interstory drift. This often leads to the use of deep beam sections in braced spans. [6-11]. Extensive research on the behavior of concentrically braced frames has demonstrated that inadequate strength in linking beams can significantly affect the seismic performance of the system. Studies by Rai and Goel have clearly shown that mismatches between beam and brace capacities can lead to severe damage or even collapse. This issue becomes more critical at story drift levels of 2–4%, as observed by Chen et al., where substantial vertical deformations develop in the beams, significantly increasing the ductility demands at beam-column connections. Furthermore, findings by Uriz and Mahin indicate that such conditions can reduce the system's energy dissipation capacity by up to 40%. Sabelli et al. (2013) have also warned that deep beams in SCBF systems may induce critical flexural stresses in connections and trigger buckling in columns within braced bays. In response, several mitigation strategies such as the use of double-story X-braces or the implementation of a weak-brace/strong-beam design philosophy

have been proposed to enhance performance. Collectively, these findings underscore the necessity of precise design for link beams, considering all associated interaction effects [12–21].

2- Research Methodology

The seismic behavior of braced frames indicates that axial forces can have a significant impact on column stability. In bracing systems of types V and inverted V-(chevron), beams are responsible for transferring unbalanced vertical loads after brace buckling. In these systems, providing adequate torsional stiffness for the beams is of particular importance to ensure the lateral stability of the structure. Experimental studies have shown that the buckling and yielding process of braces propagates progressively along the height of the frame. As illustrated in Figure 1, this phenomenon leads to an asymmetric distribution of lateral displacements among the stories. In the stories where braces undergo tensile yielding, significantly larger lateral deformations are observed compared to adjacent stories. This deformation incompatibility results in considerable bending moments in the columns. The progressive collapse mechanism typically initiates from the weakest story and gradually spreads to the upper levels. The combination of these effects with existing axial forces in the columns can lead to premature instability of the columns before the system reaches its ultimate capacity. This phenomenon is especially pronounced in frames with variable story heights or non-uniform lateral stiffness. As shown in Figure 2, the yielding process first initiates in the braces of the first story (the lowest level). As tensile deformations in these braces increase, the lateral displacement of the story grows significantly, causing substantial bending moments in the adjacent columns. Concurrently, the compressive load-bearing capacity of the first-story braces gradually diminishes, reaching the post-buckling resistance level (C'_{exp}). This capacity reduction causes load redistribution to other structural elements. Under these conditions, structural equilibrium is maintained by the increased participation of columns in resisting lateral shear, which is a consequence of induced bending deformations. As loading progresses and the tensile braces of the second story reach their yield limit (T_{exp}), the system attains its maximum deformation capacity. At this stage, the compressive braces of the second story are still functioning within their initial buckling capacity (C_{exp}). This asymmetry in the behavior of adjacent braces leads to unbalanced shear forces within the system [22,23].

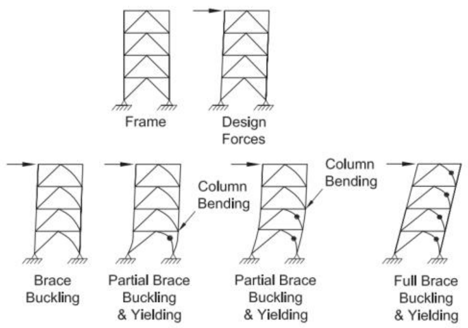

Figure 1. Progression of brace buckling and yielding in an SCBF frame [23]

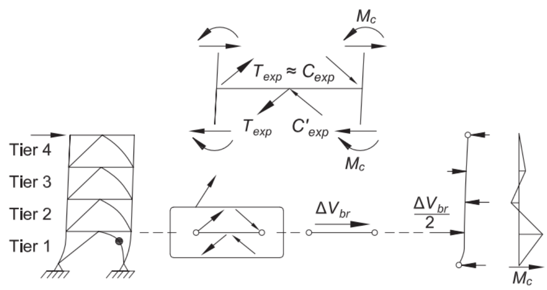

Figure 2. Unbalanced story shear resistance of braces in an SCBF frame [23]

3- Modeling

For the modeling, two-dimensional frames were employed using SAP2000 v19. The models under consideration are based on the assumption of constructing the structures in an area with a very high relative site amplification, situated on soil type II. Additionally, the structures are modeled in three building types, consisting of 5, 10, and 15 stories, with a seismic force-resisting system of special concentric braced frames, and two bracing types (V and inverted V-(chevron)), modeled separately. The structures feature four 5-meter spans, with a floor height of 3.2 meters. For the analysis and design of the mentioned structures, the fourth edition of Standard 2800, Chapter 10 of the Iranian National Building Code, and Publication 360 have been utilized. Figure 3 illustrates the schematic representation of the modeled structures.

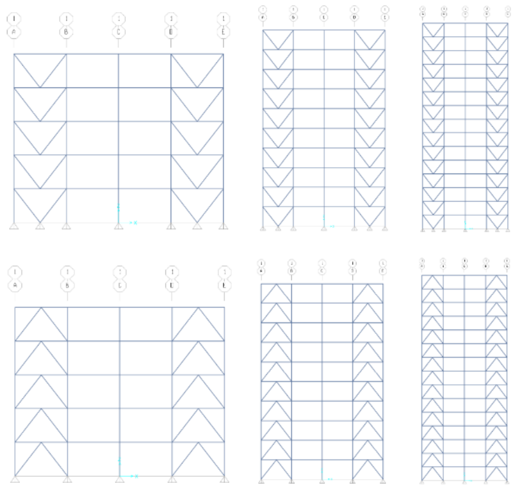

Figure 3. Schematic representation of the 5, 10, and 15-story structures modeled with bracing types V and inverted V-chevron

3-1-Structural Sections

The beams in the studied structures utilize IPE profiles and plate girders. Box sections fabricated from steel plates are used for columns, while double UNP channels form the bracing members. Table 1 presents the section types used for the 5, 10, and 15-story models.

3-2- Structural Loading

The gravity loading includes dead, live, and partition loads as follows:

- Floor dead load: 600 kg/m²
- Floor live load: 200 kg/m²
- Roof live load: 150 kg/m²
- Partition load: 100 kg/m²
- Parapet wall load: 300 kg/m²

3-3- Beam and Column Capacity Check at Bracing Spans

One of the critical checks in the design process is the evaluation of the capacity of the columns at the bracing spans. Since the fuse of the structure forms in the bracing during an earthquake, it is essential to ensure that the beams and columns at the bracing spans do not enter the nonlinear phase. Figure 4 illustrates the capacity check of the beams and columns at the bracing spans for the 5, 10, and 15-story structures with bracing types V and inverted V-(chevron), both in the linear state and according to the regulations of Chapter 10 of the National Building Code and Standard 2800.

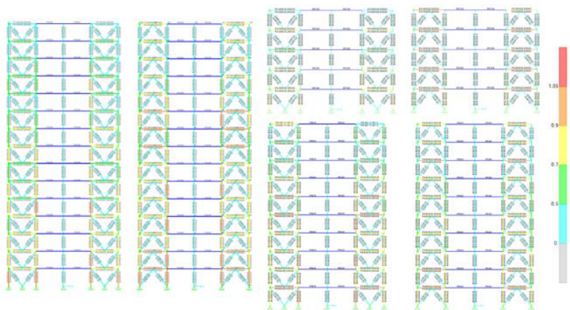

Figure 4. Capacity Control of Beams and Columns at the Bracing Bay in 5, 10, and 15-Story Structures with Bracing Types V and inverted V-(chevron)

3-4- Material and Section Properties

For modeling the mentioned structures, a yield stress of 2400 kg/cm² and an ultimate stress of 3700 kg/cm² were considered. Additionally, the expected yield and ultimate stresses were taken as 2880 and 4440 kg/cm², respectively. Table 1 presents the specifications of the structural sections used in the design of the 5, 10, and 15-story buildings, while Table 2 summarizes the total material tonnage. The obtained tonnages for structures with different bracing systems were found to be relatively close. Nevertheless, in the 5 and 15-story buildings, the structure with bracing type inverted V-chevron resulted in a lower total tonnage compared to that with bracing type V, whereas in the 10-story configuration, the system utilizing bracing type V demonstrated a lighter structural weight.

Table 1 – Structural Sections Utilized in the Modeling of 5-, 10-, and 15-Story Buildings

Simply Supported Beam	Brace	Column	Braced Bay Beam
IPE 400	2UNP180	BOX 550*30	PG 600*20-350*20
IPE 360	2UNP160	BOX 500*25	PG 600*20-350*25
IPE 330	2UNP140	BOX 450*25	PG 600*20-300*20
IPE 300	2UNP120	BOX 400*25	PG 600*20-250*25
	2UNP100	BOX 350*25	PG 600*20-250*20
		BOX 300*25	PG 550*20-250*25
		BOX 300*20	PG 450*12-150*15
		BOX 150*15	
		BOX 250*20	
		BOX 250*15	

Table 2 – Tonnage of Structural Sections Utilized in the Modeling of 5, 10, and 15-Story Buildings

Structure	Weight (ton)
5 Story – Bracing Type V	27.27
5 Story – Bracing Type inverted V	26.38
10 Story – Bracing Type V	65.75
10 Story – Bracing Type inverted V	67.80
15 Story – Bracing Type V	118.38
15 Story – Bracing Type inverted V	107.02

3-5- Plastic Hinges

Figure 5 illustrates the plastic hinges assigned to various structural elements. For the evaluation of the designed structures, plastic hinges were assigned to different members as follows:

a) In beams without bracing and structural connections, two shear force-controlled hinges (V2) were assigned at 95% and 5% of the beam length, and one moment force-controlled hinge (PM3) was assigned at the mid-span.

b) In beams within the braced bays, shear force-controlled hinges were assigned at 0.95, 0.05, 0.45, and 0.55 of the beam length, along with a moment force-controlled hinge (PM3) at mid-span.

c) In braces, a compressive plastic hinge was assigned at 10% of the brace length.

d) In columns, initially a compressive hinge (P) was assigned at mid-height, and subsequently, moment-controlled hinges (PM2) were assigned at 95% and 5% of the column height after hinge refinement.

e) In columns with $p/p_{cl} \leq 0.5$, two moment-controlled hinges (PM2) were assigned at 0.95 and 0.05 of the column height.

f) Additionally, in all columns, a tension deformation-controlled hinge was assigned at 20% of the column height.

4- Results Analysis

This section presents the results obtained from the modeling process. The models under consideration are designed with two types of braces, V and inverted V-(chevron), and are modeled as 5, 10, and 15-story structures. These models were analyzed statically using a push-over analysis method. The graphs examined include the outputs of axial forces in the beams of the braced spans and the bending moments induced by these axial forces on the columns of the braced spans under the first-mode load distribution at hazard level one. Figure 6 shows a schematic diagram of the axial forces, bending moments generated in the columns, and the plastic hinge formation in the 10-story structure. Additionally, Table 3 presents the target displacements obtained at the hazard level one for examining axial forces and moments in the existing structures.

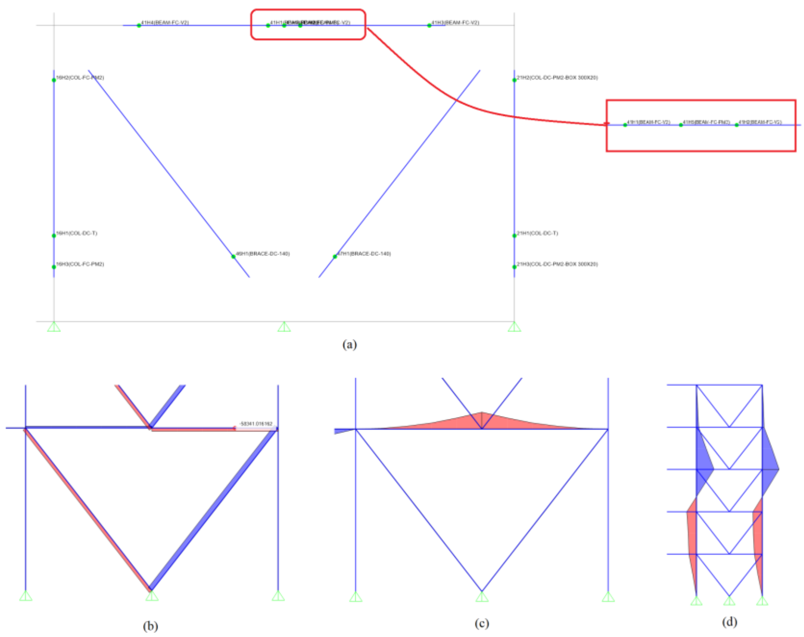

Figure 5 – Assigned Plastic Hinges to Structural Elements: (a) Force-Controlled Shear Hinges, (b) Axial Force Hinges, (c) Flexural Hinges about Axis 3, (d) Flexural Hinges about Axis 2 in Columns

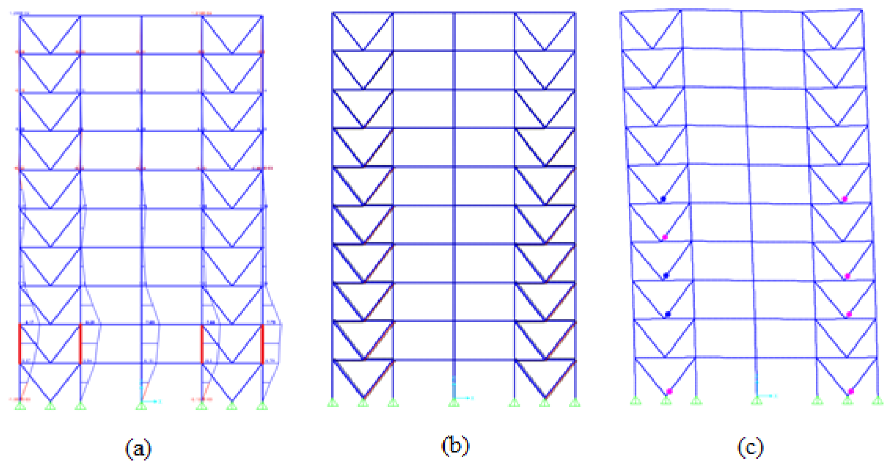

Figure 6. Bending Moment (a), Axial Force (b), Hinges at Target Displacement (c) in the 10-Story Structure with Brace Type V.

Table 3. Target Displacements of the Modeled Structures.

Structure	Target displacement (cm)
5 Story –Type V	6.5
5 Story –Type inverted V	5.2
10 Story –Type V	18.1
10 Story –Type inverted V	15
15 Story –Type V	35.2
15 Story –Type inverted V	31.3

4-1- Comparison of Axial Force Ratio at Target Displacement to Design Axial Force in Beams

In this section, the ratio of the axial force at the target displacement to the design axial force in beams located at the story where the connected columns experience maximum moments is investigated. These beams are located on the third floor in the 5-story structure, and on the second floor in the 10- and 15-story structures. According to Table 4, the axial forces developed in the beams at the target displacement are significantly lower than the design axial forces under the capacity-controlled condition of the braced span beams. In linear design, the force distribution is based on linear assumptions and member capacities; therefore, axial forces in braced span beams are typically high. However, the observed reduction in axial force in the braced span beams during nonlinear analysis is directly attributable to the buckling and subsequent loss of load-carrying capacity of the compression brace at higher displacement levels. As the structure undergoes increased displacement and enters the nonlinear range, the compression brace, due to its lack of local stability, buckles and ceases to participate effectively in axial load resistance. This phenomenon leads to a significant reduction in the lateral stiffness of the system and alters the force transfer mechanism at the beam-column joint. Consequently, despite the reduction in beam axial forces compared to linear analysis, secondary moments are introduced into the columns due to the resulting force imbalance at the joint, a condition that is absent in linear design. Furthermore, Table 5 presents the ratio of the developed tensile and compressive axial forces at the target displacement step to the beam capacity, which plays a critical role in assessing the system's stability and the force redistribution mechanism. Based on the results, all ratios of tensile and compressive axial forces to the beam capacity are found to be less than 10%, indicating that the beams, in terms of axial capacity, remain within a safe and non-critical range, and their primary behavior continues to be flexural. Nevertheless, the presence of these axial forces, within the framework of unbalanced brace behavior, plays a key role in transferring secondary moments to the columns.

4-2- Results of Axial Force and Bending Moment in Critical Columns of Braced Bays

Table 6 presents the results of the ratio of axial force and bending moment developed in the critical columns of braced bays at target displacement steps to the axial and bending capacities of the columns. The columns analyzed in the 5-story buildings are located on the third floor, and for the 10-story and 15-story buildings, the columns on the second floor exhibit the highest bending moment at the target displacement step. According to the obtained results, the ratio of the bending moment at target displacement to the column capacity for the different structures examined ranged from 3.9% to 5.3%. These values indicate the additional moment transferred to the column due to the imbalance of axial forces and the nonlinear behavior of the beams. Meanwhile, the ratio of the axial force in the column at target displacement to the column capacity varied from 4% to 54%, with the 10-story buildings having the highest share. Furthermore, the total of these ratios is below the limiting value of 1.0, in accordance with the code provisions, indicating a safe level.

4-3- Maximum Bending Moment in Columns Induced by Axial Force of Beams

Figure 7 illustrates the maximum bending moment induced by the axial force of beams in the A1 columns of 5, 10 , and 15-story structures with brace systems V and inverted V-(chevron). In the analyzed structures, individually modeled with brace systems V and inverted V-(chevron), this stress concentration in the lower regions is primarily caused by two key mechanisms: firstly, the accumulation of lateral displacement effects due to the overall frame behavior, and secondly, the bending moments induced by the axial forces in the beams of the braced bays. The axial forces developed in the beams connected to the braces are systematically transferred to the columns, resulting in significant bending moments. A comparative analysis of the bending moments in the columns of short- and mid-rise structures with brace systems V and inverted V-(chevron) reveals intriguing results. In short- and mid-rise structures (Figures 7-a and 7-b), brace system inverted V clearly exhibits larger maximum bending moments in the columns. However, in the analysis of tall buildings (Figure 7-c), this trend is reversed, and brace system V demonstrates higher bending moments. This interesting behavioral change that occurs with increasing building height could be attributed to the difference in lateral stiffness between the two brace systems and the alteration in the axial force distribution of the beams as the height increases. The columns in the lower floors, closer to the base, experience the highest bending moments. This stress concentration in the lower regions is caused by the accumulation of lateral displacement effects and the bending due to the overall frame behavior. As we move toward the middle floors, the distribution of bending moments becomes more balanced, with the changes in moment becoming smoother and more uniform compared to the lower floors. In the upper floors of the structure, the behavior of the columns changes noticeably. In these areas, the bending moments significantly decrease. This change in behavior indicates a transition from a predominantly bending-dominated behavior at the base to a more flexible behavior in the upper levels.

Table 4 – Ratio of Beam Axial Force at Target Displacement to Design Axial Force

Structure	Axial Force at Target Displacement (P) [ton]	Design Axial Force (Pr) [ton]	P/P_r
5 Story –Type V	-30.18	-60.30	0.5
5 Story –Type inverted V	-20.84	-63.92	0.33
10 Story –Type V	-56.92	-97.25	0.58
10 Story – Type inverted V	-58.13	-103.94	0.56
15 Story –Type V	-45.17	-96.14	0.47
15 Story – Type inverted V	-38.09	-100.33	0.38

Table 5 – Ratio of Axial Tension and Compression Forces at Target Displacement to the Critical Beam Capacity

Structure	Ratio of Tensile Axial Force to Tensile Capacity	Ratio of Compressive Axial Force to Compressive Capacity	Axial Force at Target Displacement (ton)		Beam Axial Force Capacity (ton)	
			Axial Tension Force	Axial Compression Force	Tension Capacity	Compression Capacity
5 Story –V	0.057	0.081	29.83	-30.18	522.72	371.85
5 Story –inverted V	0.070	0.065	35.33	-20.84	507.6	321.51
10 Story –V	0.085	0.098	49.83	-56.92	583.2	580.46
10 Story –inverted V	0.094	0.100	54.82	-58.13	583.2	580.46
15 Story –V	0.071	0.071	45.07	-45.17	637.2	634.31
15 Story –inverted V	0.077	0.060	48.81	-38.09	637.2	634.31

Table 6 - Ratio of Axial Force and Bending Moment in Column Bays at Target Displacement to Column Capacity

Structure	Axial Force at Target Displacement [ton.m]	Moment at Traget Dicplacement [ton.m]	Column Axial Compression Capacity (Pc) [ton.m]	Column Moment Capacity (Mc) [ton.m]	P/Pc	M/Mc
5 Story –V	65.4	1.98	465.01	50.88	0.14	0.039
5 Story –inverted V	16.74	2.3	465.01	50.88	0.04	0.045
10 Story –V	-547.19	8.17	1011.91	182.92	0.54	0.045
10 Story –inverted V	-452.8	9.83	1017.03	182.92	0.45	0.054
15 Story –V	-448.24	12.97	1332.39	263.12	0.34	0.049
15 Story –inverted V	350.02	11.91	1332.39	226.29	0.26	0.053

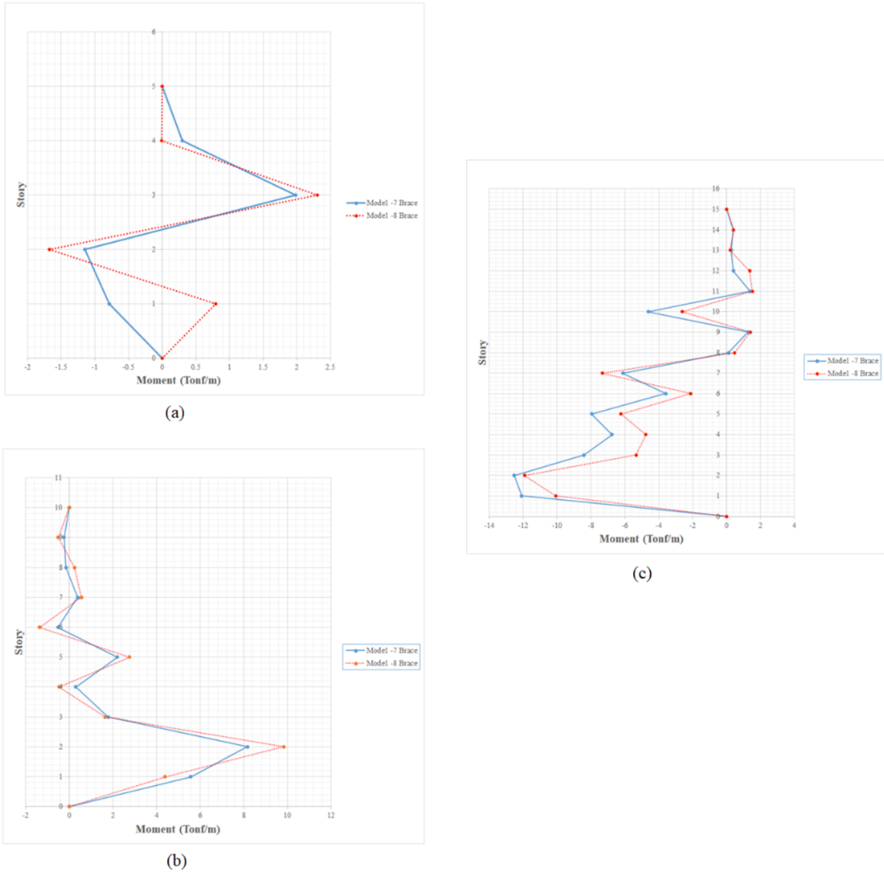

Figure 7 - Maximum moment generated in columns A1 of structures with 5, 10, and 15 floors with bracing systems V and inverted V-(chevron)

5- conclusion

Design codes based on prescriptive approaches are predominantly developed considering linear analysis, under the assumption that the bracing system fully sustains the axial forces in beams at the design level, and nonlinear redistribution of internal forces is not explicitly accounted for. However, in reality, and especially within performance-based design methodologies, variations in the axial force of beams can significantly influence column behavior, whereby an increase in beam axial force may lead to elevated column moments. The outcomes derived from the structural evaluations are summarized as follows:

1. Based on the analysis results, it can be concluded that, due to a more accurate representation of the nonlinear structural behavior, the axial forces in the braced bay beams at the target displacement level have decreased compared to those from linear analysis, primarily due to brace buckling effects. In high-rise structures, this reduction was found to be approximately 62%, while for mid-rise buildings, the reduction ranged between 42% and 48%.

2. The presence of axial forces in beams has been identified as a contributing factor to the generation of additional moments in the columns connected to these beams. Nonlinear analysis results reveal that with increasing building height, column moments induced by beam axial forces at the target displacement level have significantly intensified. A comparison between two bracing systems, namely Chevron (V-brace) and inverted V-brace configurations, indicates that despite differences in brace arrangements, the trends of column moments at critical stories are quite similar, reflecting aligned behavior in response to height increments.

3. A noticeable increase in critical column moments was observed when transitioning from 5-story to 10-story structures. In this phase, column moments increased by approximately 4.2 to 4.9 times for both

bracing systems. However, from 10 to 15 stories, the rate of moment growth significantly reduced. In the Chevron-braced system, this increase was approximately 1.58 times, whereas for the inverted V-braced system, it was about 1.2 times. This reduction in the growth rate of moments at greater heights can be attributed to factors such as changes in deformation distribution, the decreasing contribution of compressive braces to lateral load resistance, and the influence of post-buckling stiffness and behavior.

4. The analyses indicated that the ratio of column moment at the target displacement step to the column flexural capacity ranged between approximately 3.9% and 5.4%. Although these values might initially appear minor, they represent additional moments that are neglected in linear design assumptions, which idealize brace performance and pinned connections. As the structure height increases, these additional moments become more significant due to the complex redistribution of internal forces and the potential for partial brace degradation, underscoring the critical necessity to incorporate such effects into the design of taller buildings to prevent unanticipated performance behaviors.

5. As the building height increases, it can be observed that the structural behavior undergoes significant changes, with maximum column moments in taller buildings occurring in columns connected to the Chevron-braced bays (System V). In contrast, in low- and mid-rise buildings, maximum moments were found in critical columns of the inverted V-braced system. Additionally, with the increase in height, the column moments decrease, emphasizing the importance of plastic hinge formation and higher force demands in the lower stories of tall structures.

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