



Design Guidelines for Mitigating P-Delta Effects on the Seismic Response of Multi-Storey Steel Building Structures in Moderate and High Seismic Regions

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Abstract

This article introduces framing systems and design guidelines to mitigate P-delta effects on the seismic response of steel braced frames used in multi-storey building structures. The proposed systems represent practical and cost-effective solutions to prevent concentration of inelastic deformations along the building height and achieve stable inelastic response, with the objective of waiving the height limit currently specified in ASCE/SEI 7 provisions for buckling restrained braced frames (BRBFs) in Seismic Design Category D, E, and F. The proposed framing systems consist of elastic bracing members that induce unbalanced vertical loads on the floor beams during storey drifting, resulting in secondary lateral storey shear stiffness. The floor beams are proportioned to maintain this secondary stiffness upon the inelastic response of the braced frame, such that P-delta effects are neutralized, and a self-centering response can develop to prevent progressive drifting and avoid collapse by dynamic instability. The adequacy of the proposed framing systems and design guidelines is verified by means of nonlinear response history analysis on 10-, 20-, 30-, and 40-storey BRBFs located in Seattle, WA. Two designs are compared: Design A in which P-Delta effects are accounted for in design as prescribed in ASCE/SEI 7, i.e. by amplifying seismic induced member forces using the stability coefficient θ ; and Design B in which the proposed framing systems are introduced to counter P-Delta effects. The validation is accomplished by comparing the responses obtained using the two design approaches, with focus on peak and residual storey drifts. The study also includes a comparison of the responses from the two designs with the response obtained from analyses performed on the Design B frames when ignoring P-Delta effects. The results of the study show that amplifying seismic induced member forces as specified in ASCE 7 is not sufficient to mitigate soft-storey response and collapse by dynamic instability for the braced frames exceeding current height limits. In contrast, for all buildings examined, Design B in which secondary elastic storey shear stiffness is provided at every level could effectively ensure stable inelastic response, with uniform storey drifts and limited residual storey drifts over the full building height.

1. Introduction

The seismic performance of conventional buckling-restrained steel braced frames (BRBFs) during intense earthquakes is significantly influenced by second-order P-delta effects. This behavior is

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attributed to the limited kinematic strain hardening of the yielding braces. Despite the stable and buckling-free performance of buckling-restrained braces under cyclic loading, BRBFs exhibit a stability-critical reduction in storey shear stiffness after yielding when subjected to lateral loads in the presence of gravity loading, which can result in the development of a negative post-yield storey shear stiffness (Fig. 1a). Such an unfavourable response is replicated when the structure is subjected to strong ground motions, such as the Subduction Interface ground motions observed in the Seattle, WA., region. Consequently, this results in an irreversible concentration of drifts in particular storey(s) and, ultimately, may lead to excessive drifting (ratcheting) or global instability (Fahnestock, Sause et al. 2003, Kiggins and Uang 2006, Erochko, Christopoulos et al. 2011, Hariri and Tremblay 2022, De Francesco and Sullivan 2023, Hariri 2023). This failure mechanism is particularly accentuated in tall buildings due to their complex multi-mode response and the greater gravity loads they support.

In response to these considerations, ASCE 7-22 classifies structures primarily based on their geographical location using the Design Category and enforces stability-related stringent height limitations. These limitations are implemented to mitigate the risk of global instability. Specifically, for buckling-restrained steel braced frame structures with a normal risk factor in Seattle, WA, categorized under Seismic Design Category D and E, the maximum allowable height is set at 160 feet, while BRBFs in Seismic Design Category F are limited to 100 feet in height.

To address P-delta effects, ASCE 7-22 stipulates, first, assessing the structural sensitivity against P-delta effects using a stability coefficient θ calculated on a storey-by-storey basis using Eq. 1:

$$\theta = \frac{P_x/h_{sx}}{V_x/\Delta_{xe}} \quad (1)$$

Where V_x/Δ_{xe} represents the story's elastic stiffness at level x , calculated as the ratio of the seismic design shear (V_x) to the corresponding elastic story drift (Δ_{xe}), and P_x/h_{sx} is the negative stiffness due to P-delta effects, which is computed as the ratio of the total gravity design load at and above level x to the storey height (h_{sx}). The building code establishes a structural redesign threshold in case the stability coefficient surpasses the maximum allowed value, θ_{max} . However, for storeys with P-delta negative stiffness falling below 10% of the storey's elastic shear stiffness, no additional stability-design requirements are specified. For storeys with P-delta negative stiffness ranging from 10% of the story's elastic stiffness to θ_{max} , the code stipulates determining the P-delta induced effects on storey drifts and member forces utilizing rational analyses. Alternatively, the story drifts and member forces can be amplified by multiplying them by a factor of $1/(1 - \theta)$.

Despite the method, the code provides no explicit mechanism to resist P-delta induced storey shear, apart from amplifying the storey yielding strength, either directly or indirectly, to meet the prescribed drift limitations. Recent studies have demonstrated that solely amplifying yielding strength as a means to mitigate P-delta effects during the inelastic phase of the structural response is unreliable (Wei, Xu et al. 2012, Hariri 2023, Hariri and Tremblay 2023). Furthermore, ASCE 7-22 cautions against relying solely on the adopted stability coefficient as a dependable indicator for assessing the significance of P-delta effects, especially in cases involving substantial inelastic deformations.

Alternatively, an approach to mitigate P-delta effects by incorporating secondary (post-yield) storey shear stiffness is proposed in the literature (Jennings and Husid 1968, MacRae 1994, Gupta and Krawinkler 2000, Kiggins and Uang 2006, Tremblay 2018). The approach demonstrates a promising potential in mitigating P-delta effects and preventing global instability and producing resilient post-earthquake response with mitigated residual deformation (MacRae 1994, MacRae and Kawashima 1997, Borzi, Calvi et al. 2001, Pampanin, Christopoulos et al. 2003, Christopoulos and Pampanin 2004, Kiggins and Uang 2006, Pettinga, Christopoulos et al. 2007, Sahoo and Chao 2015, Deylami and Mahdavi-pour 2016, De Francesco and Sullivan 2023). This article introduces a novel Secondary Stiffness Frame (SSF) designed to induce positive post-yield storey shear stiffness and maintain it during the inelastic response of the primary braced frame. The article also presents SSF design guidelines adequate to ensure a P-delta-mitigated response. The effectiveness of the SSF system and the proposed guidelines are validated through nonlinear response history analyses on 10-, 20-, 30-, and 40-storey buckling restrained steel braced frames (BRBF) in Seattle, WA and considering the Subduction Interface ground motions. Although utilizing the BRBF solution for buildings exceeding 20 storeys might not be practical, the 30- and 40-storey prototype buildings are considered to prove the concept in the presence of higher modes of response. Two designs are considered: Design A follows ASCE 7-22 stability provisions, meeting stability coefficient limitations and amplifying member forces and storey drifts using $1/(1 - \theta)$. Design B couples BRBFs with the Secondary Stiffness Frame, developing positive storey shear stiffness to counteract the negative effects of P-delta. Residual and peak inter-storey drifts are then compared between the two designs to assess compliance with code-proposed limits and acceptance criteria. Additionally, the responses of Design B are furtherly compared with the case where P-delta analysis is ignored to assess the adequacy of the SSF in neutralizing P-delta effects.

2. P-delta Effects Mitigating Approach

Previous research has underscored the importance of the post-yield stiffness-to-initial elastic stiffness ratio in alleviating residual drifts caused by P-delta effects, commonly referred to as ratcheting, in Single Degree of Freedom systems (SDOF) (MacRae 1994, MacRae and Kawashima 1997, Borzi, Calvi et al. 2001, Pampanin, Christopoulos et al. 2003, Christopoulos and Pampanin 2004, Kiggins and Uang 2006, Pettinga, Christopoulos et al. 2007, Sahoo and Chao 2015, Deylami and Mahdavi-pour 2016, De Francesco and Sullivan 2023). Fig. 1b provides a visual representation of this concept using an SDOF system featuring an elastic perfectly plastic seismic force resisting system (SFERS) coupled with an elastic spring which develops a positive stiffness (k'_s) that counteracts the negative stiffness of P-delta (P/h_s).

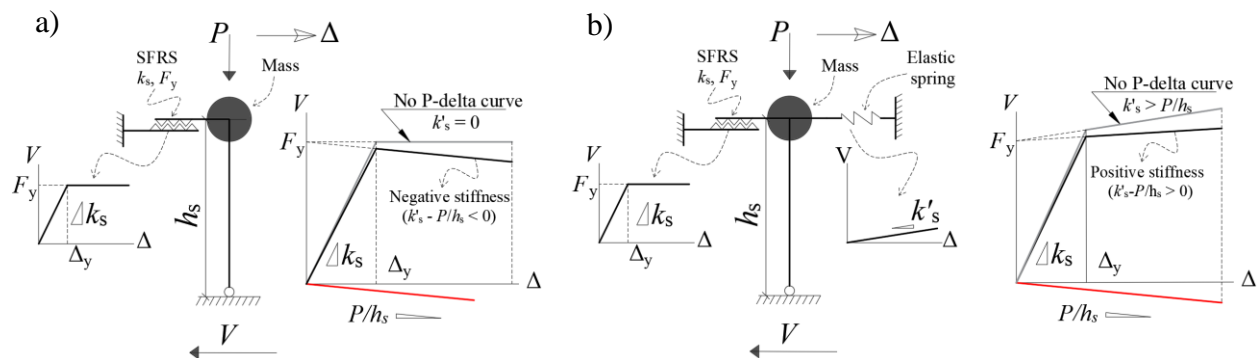


Figure 1: a) P-delta effects in SDOF system; b) Incorporating secondary stiffness approach.

Hariri (2023) conducted a comprehensive examination of the influence of lateral storey shear stiffness in multi-storey steel buildings exposed to seismic excitations arising from Crustal, In Slab, and Subduction Interface sources. The investigation specifically examined the relationship between secondary post-yield stiffness (k'_s) and both residual and peak inter-storey drift ratios. The study embraced 12 prototype buildings, with heights varying from 130 ft to 535 ft. The analysis covered a diverse range of structural configurations, including eccentrically braced frames, buckling-restrained braced frames, and friction braced frames. The key findings of the study demonstrated that incorporating secondary post-yield storey shear stiffness on a storey-by-storey basis, quantified at each storey to align with the negative stiffness of P-delta (i.e., $k'_s = P/h_s$) and sustained for storey drifts sufficient for energy dissipation, effectively mitigates P-delta effects. This approach resulted in structural responses closely resembling scenarios where no P-delta analyses were considered. Moreover, the incorporation of secondary stiffness slightly surpassing P/h_s contributed to a significant reduction in residual storey drifts.

This article follows the recommended guidelines proposed by Hariri (2023) and utilize a bracing configuration designed to establish and sustain the proposed secondary lateral storey shear stiffness. The Secondary Stiffness Frame (SSF), as designated here, incorporates a single-eccentric brace operating in conjunction with the corresponding beam. As lateral storey drifting occurs, the elastic brace/beam assembly utilizes the flexural stiffness of the beam to impart the intended secondary lateral storey shear stiffness. The concept of inducing post-yield storey shear stiffness employing the beam flexural stiffness was originally proposed by Tremblay (2018). Fig. 2 illustrates the utilization of the SSF system and provides a listing of the design equations.

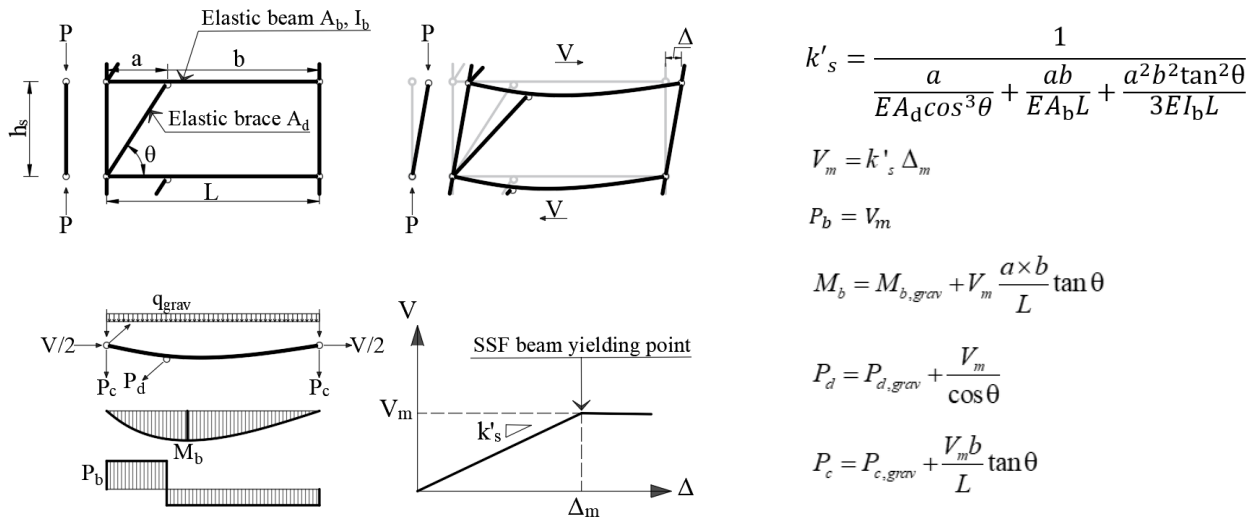


Figure 2: Secondary stiffness frame (SSF).

2. Description of Prototype Buildings

Two sets of prototype buildings (i.e., Design A and Design B) that vary with respect to the seismic stability considerations are considered in this study. The primary difference lies in addressing the seismic P-delta effects: Design A adheres to stability coefficient (θ) limitations and employs amplifying member design forces with $1/(1 - \theta)$. In contrast, BRBFs in Design B has no seismic

stability-related consideration, except that they incorporate a Secondary Stiffness Frame (SSF) designed to counteract P-delta effects.

Each set involves prototype steel office buildings located in Seattle, WA, ranging from 10 to 40 storeys, situated on soil Class C. The buildings share a symmetric layout, featuring five bays of 29.5 ft in each primary direction and a consistent storey height of 14 ft. In terms of seismic force resistance, both Design A and Design B incorporate low-strain hardening steel-restrained buckling-restrained steel braced frames (i.e., BRBFs) with a core yielding strength (F_y) of 50 ksi. The BRBF braced bays in Designs A and B are strategically positioned along the exterior column line for each principal direction. The number of braced bays varies based on building height, with a single braced bay assigned to the 10-storey structure, double-braced bays for the 20-storey buildings, and triple-braced bays for the 30- and 40-storey structures. As a source of secondary stiffness, BRBFs in Design B are coupled with SSFs. The SSF braced bays are centered-positioned along the interior axis backside the BRBF. Fig. 3 provides a visual representation of the building layout, bracing configurations, and the design gravity loads.

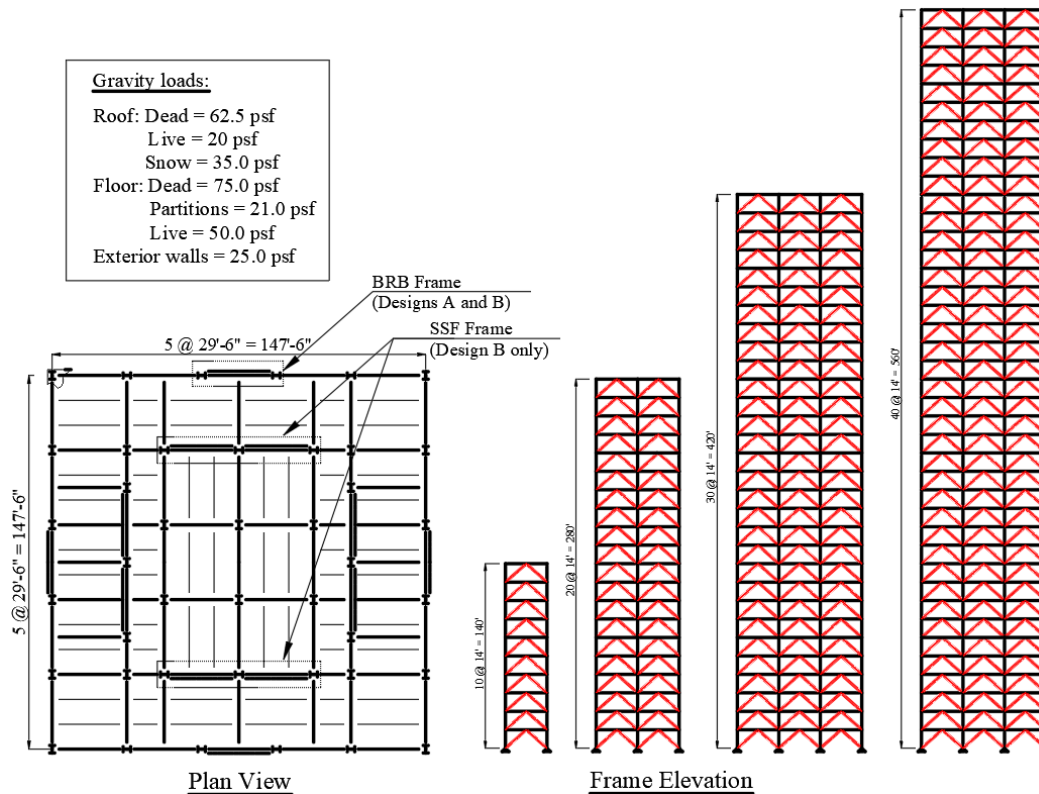


Figure 3: Prototype building.

3. Seismic Design Requirements and Design of Prototype Buildings

The seismic force resisting systems in the considered prototype buildings in both designs (i.e., the BRBFs in Designs A and B) adhere to the design standards outlined in ASCE 7-22, AISC 360-22, and AISC 341-22, for Seattle, WA. The key design parameters encompass a risk category of II, an importance factor I_e of 1, soil classification C, seismic category D, a ductility factor (R) of 8, a displacement amplification factor (C_d) of 5, with a maximum allowable height capped at 160 ft. It's noteworthy that, for the purpose of this study, the predetermined height limitation has

deliberately been overlooked. In addition, given the preliminary stage of this study and the symmetrical layouts of the buildings, torsional effects have not been considered. Moreover, Pettinga, Priestley et al. (2007) conducted a study, among the few, to evaluate permanent (residual) rotational drift in single-storey buildings with various torsional irregularities. It was noted that positioning the restraining elements orthogonally and in proximity to the plan's perimeter significantly reduces the rotational component of the drift and is likely to have a negligible effect on the permanent drift across the building plan.

The fundamental period (T) is determined through eigen vector analysis in SAP2000 and then verified with the upper limit (i.e., $C_u T_a$) for the base shear calculations, where C_u is the upper limit coefficient, set to 1.4, and T_a is the approximate period. Notably, coupling the BRBFs with the SSF frames in Design B has neglectable effects on the fundamental period and does not impact the calculation of the base shear, as design base shear is governed by the lower bound (Fig. 4).

The base shear and storey forces are calculated using the multi-mode response spectrum analysis using the Seattle, WA. design spectrum, factored with I_e/R . The storey shear is then adjusted, when smaller, to respect the requirement of the equivalent load force method $V=C_s W$, where W is the effective seismic weight and C_s is the seismic response coefficient, calculated respecting *Method 1* in the ASCE 7-22, with a lower bound equal to the largest of $0.5S_1/(R/I_e)$ and $0.044S_{DS}I_e$, where S_1 and S_{DS} are 0.66 and 1.17, respectively.

Storey drifts obtained from the multi-mode response spectrum analyses are scaled using $(1/(1-\theta)) C_d/I_e$ for Design A and using the C_d/I_e for Design B, then the amplified inelastic drifts are compared with the maximum permitted limit of 0.02 storey height. Fig. 4 presents the scaled Design spectrum (i.e., $S_a I_e/R$) along with the dynamic fundamental periods and design spectral accelerations for prototype buildings in Designs A and B.

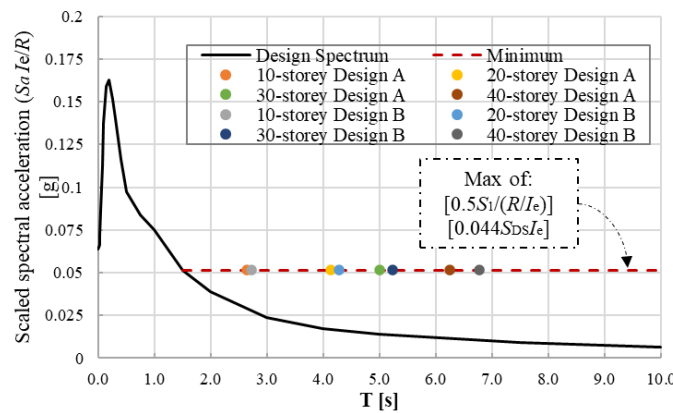


Figure 4: Seismic Design spectrum and design spectral accelerations.

The design for non-ductile frame elements incorporates capacity design principles outlined in AISC341-22 for buckling restrained steel braced frames. The earthquake forces on beams and columns are computed by employing the probable yielding strength of ductile members (BRBs), with an anticipated yielding-strength ratio (R_y) set at 1.1. Strain hardening and compression adjustment factors (ω and β) are applied at 1.4 and 1.1, respectively. P-delta effects are addressed

using the Direct Method, incorporating flexural and axial stiffness reduction factor of 0.8. Additionally, initial imperfections and P-delta effects are considered directly through modelling each member with a multi-element with a maximum initial imperfection corresponds to $L/500$.

Beams are designed assuming braces provide no support for the dead and live loads with out-of-plane lateral support provided at the beam-brace intersections. Columns are designed assuming continuous splicing every other storey with lateral supports in both principal buckling directions provided at storey levels. Plate-reinforced W-sections are used for the lower storeys in the case of 40-storey BRBFs, given the substantial axial loads that are carried by the columns. ASTM A992 steel with a yielding strength (F_y) of 50 ksi is used for beams and columns.

4. Design of the Secondary Stiffness Frames (SSFs)

The design forces for the elements of the Secondary Stiffness Frames in Design B are computed using the equations specified in Fig. 2, adhering to the parameter guidelines outlined by Hariri (2023). The storey-induced secondary stiffness is established as P/h_s , where P represents the total gravity load defined in Eq. 1, and h_s denotes the storey height. The drift (Δ_m) is regulated to align with the maximum allowable mean response drift according to ASCE 7-22, set at $0.03h_s$. Notably, this limit signifies the anticipated drift at which structures designed by ASCE 7-22 effectively dissipate the seismic energy.

To accommodate these design parameters, two SSFs with double-span braced bays are selected to couple the BRBFs for each principal direction in all prototype buildings (Fig. 3). The eccentricity (a) is set at 7 feet for the 10- and 20-storey buildings and 5 feet for the 30- and 40-storey buildings. The SSF beams are designed assuming a yielding strength of $R_y F_y$ and a ϕ factor of 1.0, as plastic hinging is expected at the selected drift (Δ_m) (Fig. 2). Beams are assumed supported out-of-plane, where top and bottom flanges are laterally supported with 10 feet spacing. Since the brace is designed to remain elastic, it was assumed that it would provide an in-plane buckling support to the beam. Fig. 5 presents the SSF bracing configurations.

Columns and braces are capacity-designed to remain elastic utilizing design forces respecting the formation of beam plastic hinging and are designed using the yielding strength (F_y) and ϕ factor of 50 ksi and 0.9, respectively. Braces and columns are considered pinned at every storey and laterally supported at storey levels. Table 1 presents a sample design for the 10-storey SSF, illustrating the beam, columns, and braces sections along with the ratio of the induced positive storey shear stiffness to the negative P-delta induced storey shear stiffness (i.e., $k'_s/(P/h_s)$).

Table 1: 10-storey SSF design details.

Storey	Beam	Brace	Column	$k'_s/(P/h_s)$
1	W12x252	W10x54	W12x120	1.05
2	W12x230	W10x49	W12x120	1.05
3	W12x210	W10x49	W12x96	1.06
4	W12x190	W8x48	W12x96	1.08
5	W12x152	W8x40	W12x72	0.97
6	W12x136	W8x35	W12x72	1.04
7	W12x106	W8x31	W10x49	1.00
8	W12x79	W8x24	W10x49	0.99
9	W12x50	W8x21	W8x31	0.99
10	W12x22	W8x15	W8x31	1.13

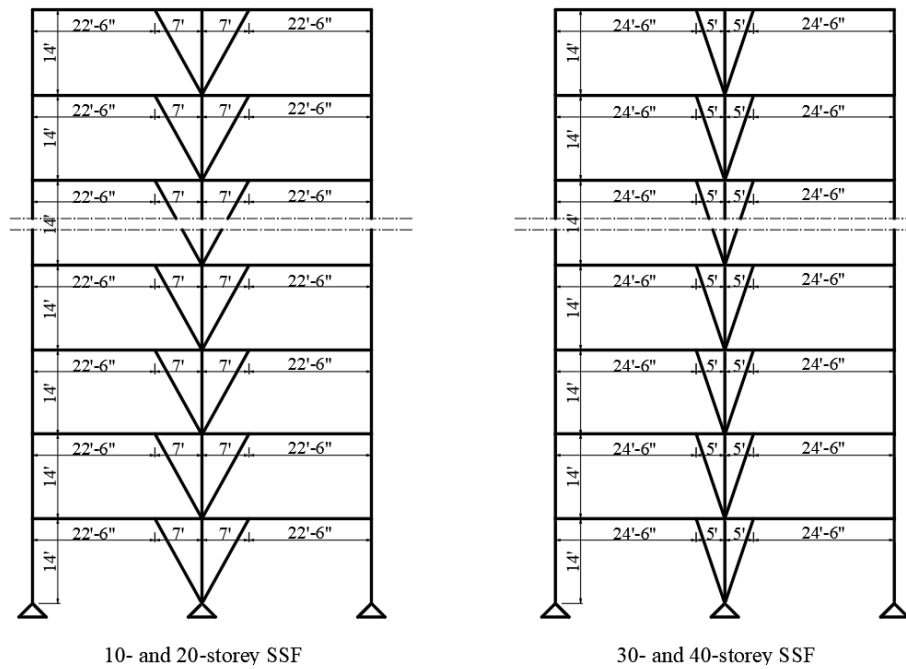


Figure 5: SSF bracing configurations.

5. Numerical Modelling

Prototype buildings of both Designs (i.e., A and B) are modelled in the OpenSees platform (Mazzoni, McKenna et al. 2006) using 2D frames, as permitted by ASCE 7-22 for preliminary performance investigations. The ductile elements (i.e., BRBs) are modelled using truss elements with an equivalent-to-actual axial stiffness ratio of 1.5 to represent the core element rigid extensions. Zsarnóczy (2013) *steel4* material is used with machine-learning calibrated parameters (Hariri and Tremblay 2021) based on tests performed at Polytechnique Montreal.

Beams, columns, and SSF braces are modelled using nonlinear beam-column elements with eight elements for each member and four integration points. The initial imperfections in beams and SSF braces are modelled using the Sine function with a maximum imperfection of $L/500$, assigned in the weak direction of their cross-sections. The columns' initial imperfections are assigned with a storey-alternating pattern. BRBF beams and SSF beams, as well as SSF braces, are all pinned-released when connected. BRBF columns are pinned-supported and continuously-spliced every other storey at 4 feet height. SSF columns are pinned-supported and moment-released at storey levels. Beams, columns, and SSF bracing members are modelled using fiber-section cross sections with initial residual stresses as defined by Galambos and Ketter (1959). Giuffré-Menegotto-Pinto *steel02* material with default parameters is assigned to the fiber-sections in beams, columns, and SSF braces. Gravity loads are represented using an axially-stiff leaning column modelled using vertical truss elements with an elastic material. The leaning column storey-level nodes are constrained to the braced frame mid-node and to the SSF mid-column node (Design B only) using a rigid diaphragm constrain. Masses are lumped at storey levels and assigned to column nodes at storey-levels. Fig. 6 represents the 10-storey frame developed models in Designs A and B.

The modelling incorporates current-step stiffness- and mass-proportional Raleigh damping with a

3% critical damping (Charney 2008). Specifically, stiffness-proportional damping is applied, excluding ductile elements to prevent over-damping in the system following ductile element yielding. The coefficients for mass and stiffness proportioning are computed with reference to the first and third modal periods (i.e., modes achieve 90% mass participation), as detailed in (Leger and Dussault 1992).

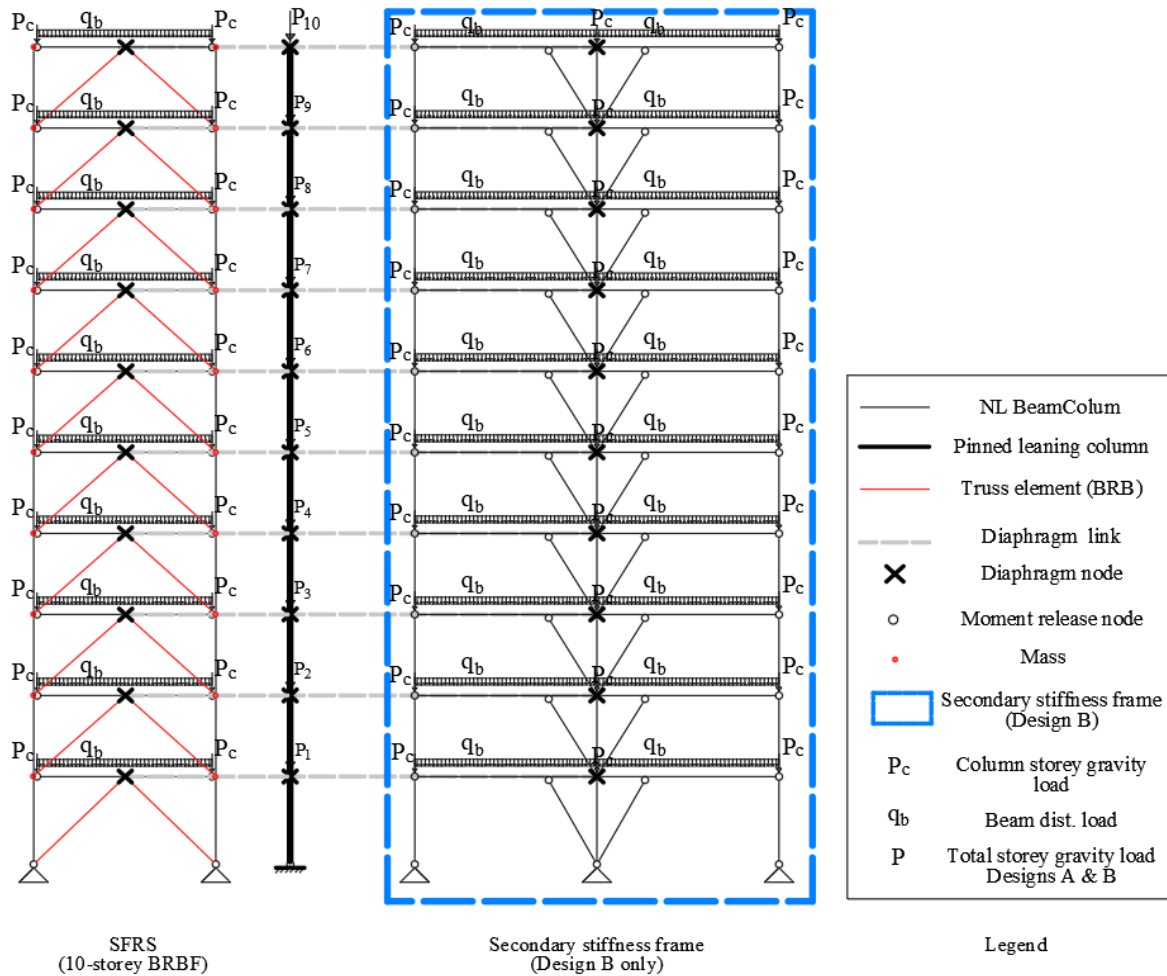


Figure 6: Numerical model (10-storey buildings).

A suite of 11 ground motions representative of Subduction Interface seismic hazard is chosen and scaled in accordance with ASCE 7-22, employing *Method 1*. A singular Target Spectrum is formulated to align with the multi-period MCE spectrum for Seattle, WA, considering Soil Class C. The selection of each ground motion is based on its maximum directional spectrum, derived from orthogonal components.

The decision to focus on the Subduction Interface is underpinned by its pivotal role as the predominant contributor to the seismic hazard spectrum in Seattle, particularly within the spectral periods relevant to the considered prototype buildings. This significance is obtained from the deaggregation analysis obtained from the *USGS Unified Hazard Tool*. Specifically, the Subduction Interface contributes 60% at the 3 s spectral period and 66.5% at the 5 s spectral period, while the remaining percentage at both periods is predominantly associated with shallow crustal ground

motions. Notably, the significance of P-delta effects in steel buildings under Crustal and In Slab motions is less influential, while P-delta effects gain heightened importance under the Subduction Interface seismic excitations, as validated by (Raghunandan, Liel et al. 2015, Tremblay 2018, Hariri 2023, Hariri and Tremblay 2023).

The deaggregation analysis further illustrates that a Magnitude-Distance ($M-R$) scenario, encompassing magnitudes ranging from 8 to 9.5 and distances from 80 km to 230 km, captures the deaggregated content. Fig. 7 comprehensively presents the selected and scaled ground motions, along with the scaling ranges pertinent to the prototype buildings under consideration.

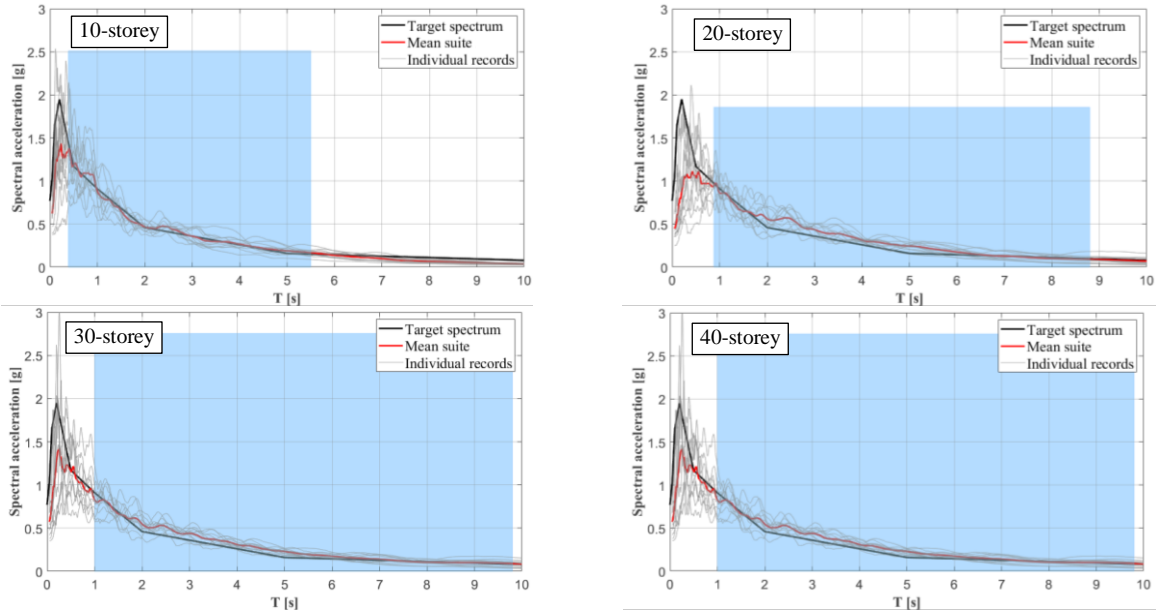


Figure 7: MCE spectrum and selected/scaled maximum-directional records.

6. Response History Analysis

Nonlinear response history analyses were performed on the developed frame models in Designs A and B using the OpenSees platform, incorporating selected ground motions. The monitoring focus was on peak inter-storey drift ratios and residual drifts along the height of the frames. Two evaluation criteria were employed for this analysis.

The first criterion adhered to the global acceptance criteria outlined in ASCE 7-22, which pertains to both the mean transient storey drift and the mean residual drift. Specifically, the mean transient storey drift needed to comply with the prescribed limit for buildings surpassing a height of 100 feet— $0.037h_s$ for the 10-storey configuration and $0.03h_s$ for the 20-, 30-, and 40-storey configurations. Additionally, the mean residual drift was required to meet the threshold of $0.01h_s$.

The second criterion involved a comparative analysis of the responses between Designs A and B, referencing the responses obtained from the frames of Design B in instances where P-delta analysis was omitted (i.e., no consideration of leaning columns). This criterion served the purpose of evaluating the effectiveness in mitigating P-delta effects.

Figures 8 and 9 present the inter-storey peak and residual drifts, accompanied by their respective mean responses, for the examined 10-, 20-, 30-, and 40-storey frames. The presentation sequence in these figures starts with the response of Design A in the top row, followed by Design B and the reference case without P-delta analysis.

The figures reveal that responses in frames designed with respect to the ASCE 7-22 seismic stability provisions, after ignoring the height limitations (Design A), did not meet the acceptance criteria outlined by the code. Notably, several collapses due to global instability were observed. When compared with the no P-delta analysis, Design A responses deviated significantly from replicating the no P-delta responses. In contrast, frames incorporating the SSFs (i.e., Design B), exhibited a uniform drift distribution along building heights with mean transient inter-storey drift response adhering to the specified limitations. Additionally, the residual drifts in Design B demonstrated reparability, displaying mean responses close to $0.005h_s$ across all tested buildings. Concerning the second evaluation criterion, frames in Design B exhibited responses closely mirroring those of the no P-delta effects case, underscoring the effectiveness of the P-delta mitigation independent of building height.

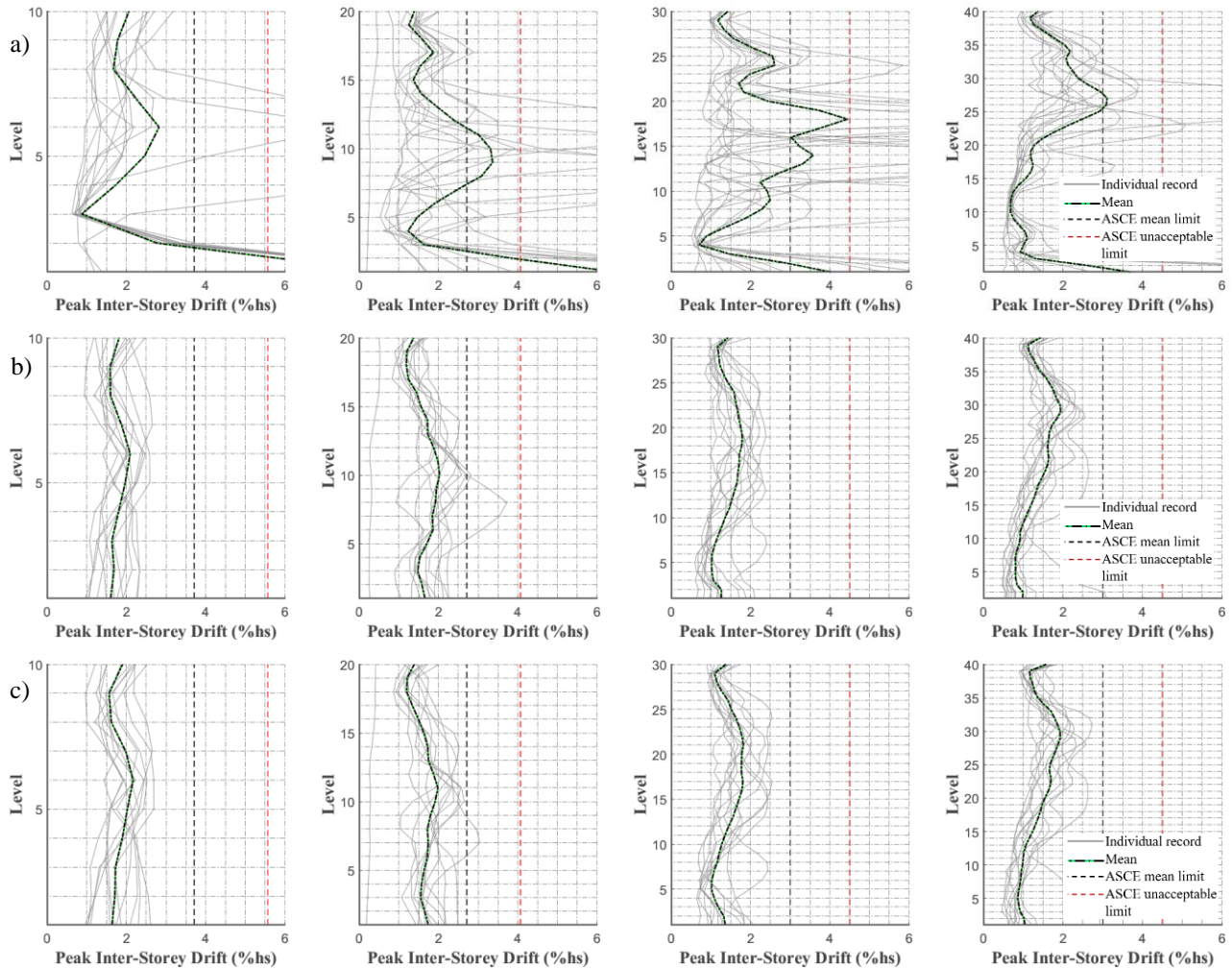


Figure 8: Peak inter-storey drift ratios. a) Design A; b) Design B; c) No P-delta analysis case (reference).

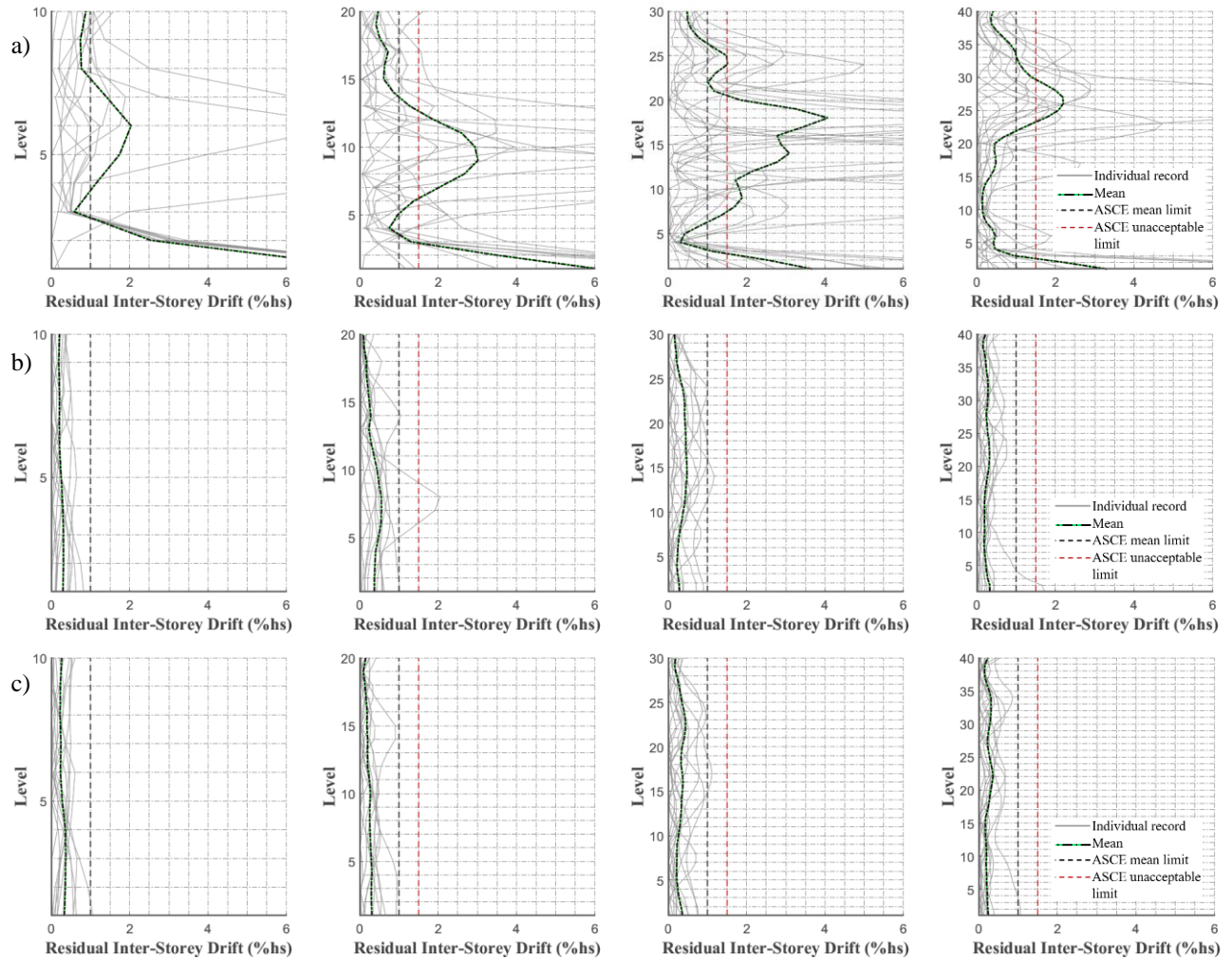


Figure 9: Residual inter-storey drift ratios. a) Design A; b) Design B; c) No P-delta analysis case (reference).

7. Design Guidelines

Based on extensive analysis conducted by Hariri (2023), including the outcome of this article, the stability-related height limitations imposed on the buckling-restrained steel braced frames (BRBFs) subjected to Subduction Interface ground motions can be relaxed when BRBFs are coupled with a Secondary Stiffness Frame (SSF) capable at every storey of producing positive lateral storey shear stiffness equal to at least the negative stiffness induced by P-delta effects. In addition, the SSF must be designed in every storey to maintain the induced positive stiffness for a storey drift of at least 0.03 of storey height. Consequently, no stability sensitivity analysis (i.e., $\theta \leq \theta_{\max}$) or stability-related drift/member force amplification is required for the BRBF structural members.

8. Conclusions

This article introduces innovative design guidelines and framing systems aimed at mitigating the second-order P-delta effects in conventional buckling-restrained steel braced frames (BRBFs) subjected to seismic loading, with a specific focus on Subduction Interface ground motions in

Seattle, WA. The study evaluates the efficacy of conventional measures outlined in the ASCE 7-22, such as amplifying member forces and storey drifts through stability coefficients and the stability-related height restrictions. Significantly, the emphasis is on minimizing the risk of global instability while adhering to peak inter-storey drift limits and repair-limit residual drift, especially for BRBFs falling under Seismic Design Category D.

The proposed guidelines stipulate that to mitigate P-delta effects, it is required to couple the BRBFs with a source of lateral storey shear stiffness system capable at every storey to develop positive lateral storey shear stiffness of at least the negative stiffness of P-delta (i.e., P/h_s) and maintain the developed stiffness for a drift of at least 0.03 storey height. No further stability-related design requirements are to be imposed on the BRBF (i.e., height limitation or stability-related force/drift amplification). Furthermore, the coupling system must be designed utilizing the capacity design principles respecting the outlined storey stiffness and drift.

As a recommended coupling system, this article introduces the Secondary Stiffness Frame (SSF), a bracing configuration deemed suitable for meeting the proposed design guidelines. The adequacy of the proposed system is verified through nonlinear response history analysis conducted on 10-, 20-, 30-, and 40-storey buckling-restrained steel braced frames situated on Soil Class C in Seattle, WA, under Subduction Interface seismic excitations. Residual and peak inter-storey drifts are monitored along the building heights. The P-delta effects were accounted for, first conventionally as stipulated by ASCE 7-22 after ignoring the height limits, and then by utilizing the SSFs.

The conventionally design BRBFs demonstrated global instability failures in all tested buildings. Conversely, coupling BRBFs with SSFs resulted in residual and peak inter-storey drifts within ASCE 7-22 limitations. Moreover, the buildings exhibited effectiveness in mitigating P-delta effects irrespective of the building's height, with seismic responses comparable to those obtained when P-delta analysis is neglected.

Acknowledgements

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