EFFECT OF THE GRAVITY FRAMING SYSTEM ON THE OVERSTRENGTH AND COLLAPSE RISK OF STEEL SPECIAL MOMENT FRAMES DESIGNED IN NORTH AMERICA

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1. ABSTRACT

In the seismic-design of steel buildings in North America, the gravity framing system is considered ineffective in resisting lateral loads. However, past analytical and experimental studies showed its considerable flexural resistance that may contribute to the overall system overstrength. This paper discusses the effect of the gravity framing system on the overstrength and collapse risk of steel frame buildings designed with special moment frames (SMFs) in the United States. Numerical models are developed for five archetype buildings with heights ranging from 2 to 20 stories. The current code-based overstrength factor, specified for buildings with SMFs, is assessed. The collapse risk is also evaluated through nonlinear response history analysis. The results show that a static overstrength > 3.0 (code-based value) is only achieved when the composite action and the gravity framing are considered in the analytical model. The dynamic overstrength factor is > 3.0 for all buildings due to the dynamic amplification of story shear forces regardless of the gravity framing effect. This indicates that a different approach to define the overstrength may be used in future seismic provisions. Finally, it is shown that a probability of collapse less than 1% in 50 years can be achieved if SMFs are designed with strong-column-weak-beam ratios larger than 1.5.

2. INTRODUCTION

In the seismic design practice of steel buildings in North America, the lateral force resisting systems (LFRS) such as steel special moment frames (SMF) are typically placed at the perimeter of the building. The SMFs are designed to solely resist the lateral forces exerted on the building. On the other hand, the gravity loads are supported by an interior gravity framing system. This system is designed with beam-to-column shear-tab connections with theoretically negligible flexural strength. Past experimental studies [1] showed that simple shear-tab connections could have considerable flexural capacity reaching up to 50% of the plastic flexural capacity of the corresponding gravity beam. Prior studies [2, 3] have investigated the effect of the gravity framing system on the static and dynamic behaviour of steel frame buildings. These studies showed that considering the gravity framing in the analytical model can increase both the lateral stiffness and strength and mitigate the lateral drifts of the LFRS.

Prior studies of archetype buildings with perimeter SMFs [4] showed that the calculated static overstrength factors from pushover analysis can vary considerably from the overstrength specified by ASCE/SEI 7-10 [5], Ω_o=3.0. This requires a more comprehensive evaluation of the overstrength of SMFs considering the contribution of the gravity framing system, as well as, the collapse risk of the same frame buildings when they are subjected to extreme earthquakes. In this paper, a comprehensive analytical study is conducted on archetype steel buildings with perimeter SMFs with heights ranging from 2 to 20 stories. Nonlinear static and response history analysis through collapse are used to investigate the effect of the gravity framing system on the overstrength and the collapse risk of SMFs.

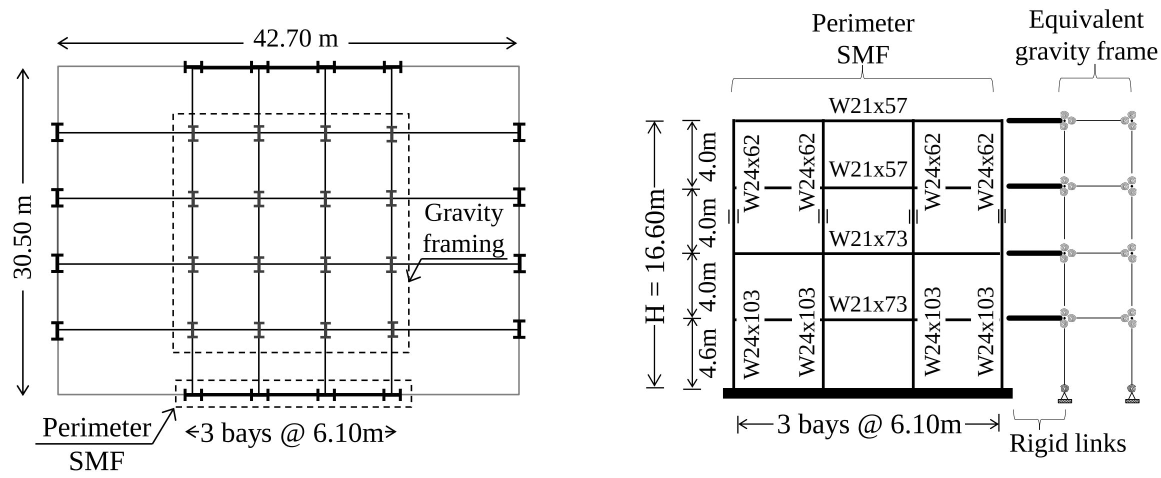
3. NUMERICAL MODELING OF ARCHETYPE BUILDINGS

Five archetype steel buildings with perimeter SMFs and heights ranging from 2, to 20 stories are utilized. The archetype buildings are located in urban California. The perimeter SMFs are designed with fully restrained reduced beam section (RBS) moment connections. The interior gravity framing system is designed with conventional single-plate shear tab connections. Details about the design aspects of the archetype buildings can be found in [6, 7]. Figures 1(a) and 1(b) show the typical plan view of the archetype building and the elevation of the perimeter SMF of the four-story for reference, respectively.

The perimeter SMF in the EW direction of each archetype building is modeled in OpenSEES [8] using a concentrated plasticity approach. The modified Ibarra-Medina-Krawinkler (IMK) hysteretic model [9] is used to simulate the hysteretic response of the SMF's fully restrained RBS connections with/without the presence of the concrete slab. This model is able to simulate the asymmetric behaviour of composite connections and the cyclic deterioration in strength and stiffness as shown in Figure 2(a). The input parameters for the backbone curve and the deterioration parameters of the modified IMK model are obtained based on [9] for bare steel components and based on [6] for composite beams.

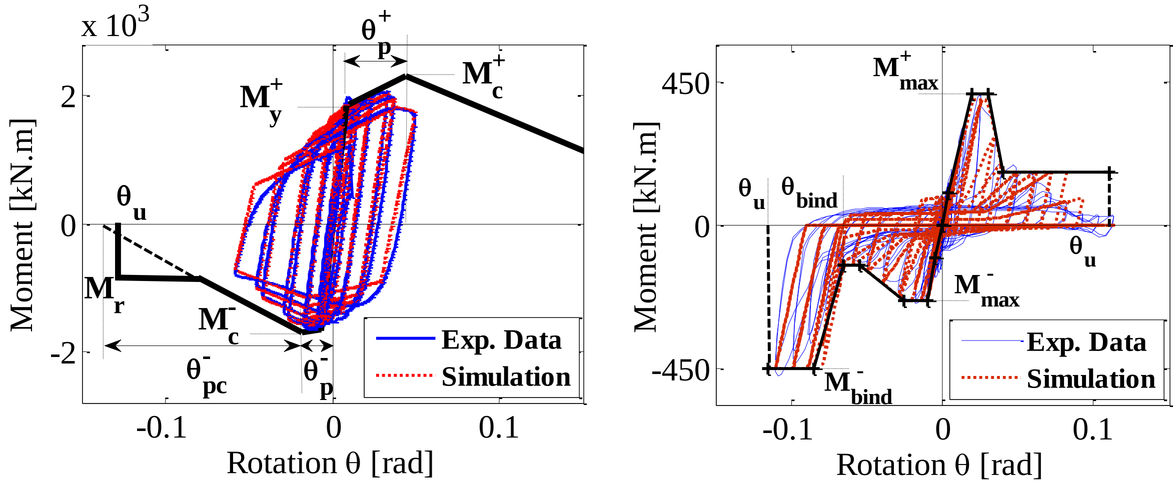
The interior gravity framing system shown in Figure 1(a) is modeled using an equivalent gravity frame [3], which is a one bay frame connected to the SMF by axially rigid truss links (see Figure 1(b)). This frame has strength and stiffness properties equivalent to those of the interior gravity framing system of the entire frame building. The beam-to-column connections of the equivalent gravity frame simulate the hysteretic behaviour of the simple shear-tab connections of the interior gravity frame. For this purpose, the Pinching4 hysteretic model [10] is employed. This model simulates a pinched force-deformation hysteretic response as expected from a typical shear-tab connection as shown in Figure 2(b). The input parameters of the Pinching4 model are obtained as proposed by [7].

For each archetype SMF, four different analytical model configurations are defined: (a) considering only the bare steel properties of the SMF (i.e., B model); (b) considering the composite slab effect when modelling the SMF (i.e., C model) (c) considering the bare gravity frame in the analytical model (i.e., BG-model) (d) considering both composite slab and the gravity framing in the analytical model (i.e., CG model).



(a) (b)

Fig. 1 (a) Plan view of the archetype building; (b) elevation view of the four-story perimeter SMF showing the equivalent gravity frame



(a) (b)

Fig. 2 (a) Modified IMK model calibrated with composite beam with RBS (experimental data from [11]); (b) Pinching4 model calibrated with composite shear-tab connection (experimental data from [1])

4 STATIC NONLINEAR ANALYSIS (PUSHOVER)

The first-mode period (T_1) of the four analytical models for each archetype building in the EW loading direction are summarized in Table 1 together with the static overstrength factors Ω_s for the frame buildings under consideration. A nonlinear static analysis (i.e., pushover analysis) is performed for this purpose using the first-mode lateral load pattern of each frame under consideration. Figure 3(a) shows the pushover curves for the four analytical models of the four-story buildings in the EW-loading direction. The pushover curve is plotted in terms of the base shear force V_1, normalized by the seismic weight W, versus the roof drift ratio $\theta_r=\delta_r/H$, where δ_r is the lateral displacement of the roof and H is the total height of the building. Figure 3(a) shows that the gravity framing increases the lateral force capacity of the frame buildings. The CG models develop an average base shear capacity nearly 50% higher than that of the B models.

No. of Stories	T_1 [sec]				Ω_s			
	B	C	BG	CG	B	C	BG	CG
2	0.88	0.81	0.82	0.76	2.98	3.48	3.51	4.66
4	1.51	1.37	1.38	1.25	1.75	2.00	2.21	2.95
8	2.00	1.82	1.89	1.72	2.63	3.13	2.89	3.71
12	2.70	2.46	2.58	2.35	2.09	2.52	2.25	2.92
20	3.44	3.17	3.35	3.08	1.89	2.27	2.02	2.59

Table 1. First-mode period and static overstrength factors for archetype steel buildings

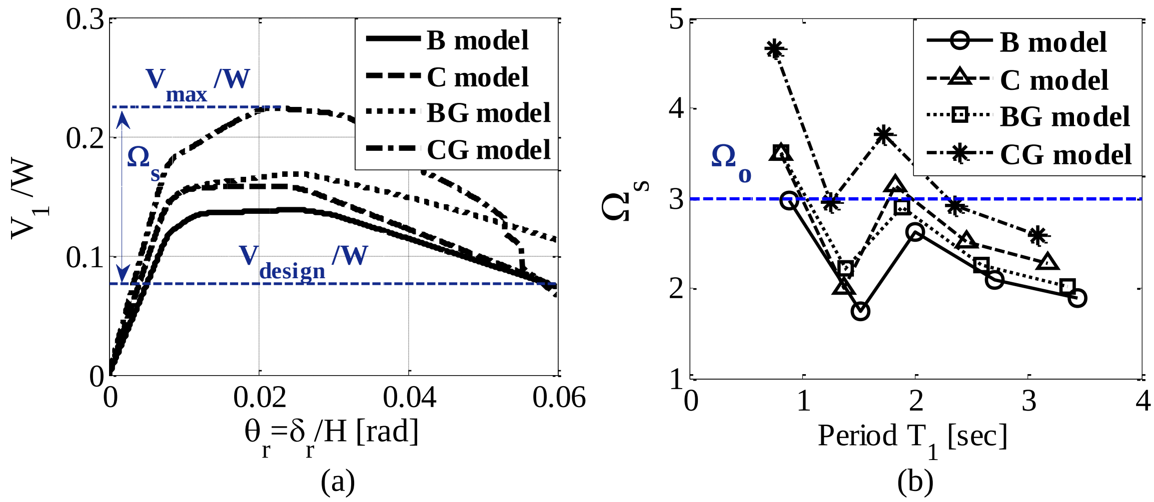


Figure 3. (a) *Comparison of pushover curves for the different analytical models of the four-story steel frame building in the EW-loading direction; (b) static overstrength factor versus first-mode period for the different analytical models of all archetype buildings*

This strength increase can be further evaluated using the static overstrength factor (Ω_s). This factor is defined as the ratio of the maximum base shear force V_{max} to the design based shear force V_{design} as illustrated in Figure 3(a) for the CG model. From Table 1, based on bare SMFs (i.e., B model), the computed overstrength factors are lower than the overstrength value, Ω_o=3.0, specified by the current seismic code provisions in the US [5] for frame buildings that utilize perimeter steel SMFs. The value of the overstrength factor increases when either the gravity framing or the composite slab is considered in the analytical model. Except for the twenty-story building, the steel frame buildings in the EW-loading direction achieve an overstrength ≥ 3.0 when both the composite slab and the gravity framing are considered. This is also shown in Figure 3(b) where the static overstrength factor is plotted versus the first-mode period for each analytical model. This figure demonstrates the static overstrength dependency with respect to the frame's first-mode period. As expected, lower overstrength factors are associated with taller buildings. The adequacy of the static overstrength factor as an accurate measure of the force demands that can develop in force-controlled components during earthquake shaking is questionable due to its large variability [4]. Consequently, the overstrength of the same archetypes is further evaluated in the next section based on nonlinear response history analysis.

5 NONLINEAR RESPONSE HISTORY ANALYSIS

Incremental dynamic analysis (IDA) [12] is conducted for each analytical model using the 44 Far-Field ground motion set and scaling procedures specified in FEMA P695 [13]. Each ground motion is scaled incrementally until dynamic collapse occurs. Collapse occurs when a number of stories displace significantly and the story shear capacity reaches zero due to increased P-Delta effects accelerated by structural component deterioration in strength and stiffness. In order to address the overstrength issue discussed in Section 4, the dynamic overstrength factor is employed. This factor is defined as the ratio of the maximum dynamic base shear to the design base shear for each frame. Figure 4(a) illustrates the definition of the dynamic overstrength for the CG model of the four-story steel frame buildings when subjected to the “SAHOP Casa Flores” record of the 1979 Imperial Valley earthquake scaled to collapse. Figure 4(b) shows the ratio of dynamic to static overstrength factor versus the first-mode period of the archetype buildings under consideration. This figure shows that the dynamic to static overstrength ratio is higher at larger periods (i.e., taller buildings) than that observed in shorter periods. This is attributed to the dynamic higher-mode effect that amplifies the story shear forces compared to pushover analysis that is based on a first-mode lateral load pattern.

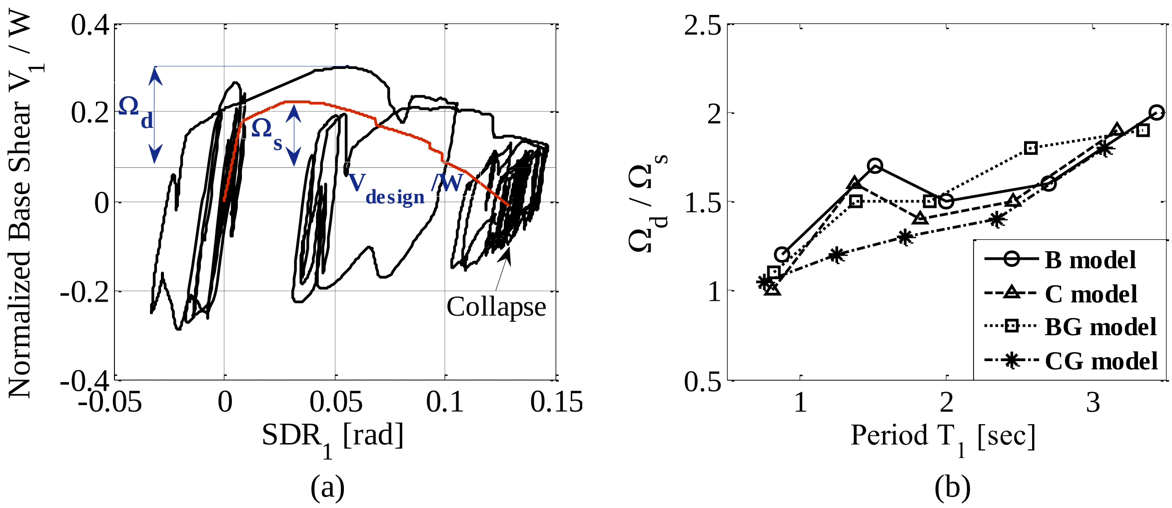


Figure 4. (a) *Normalized base shear force versus first story drift ratio at collapse intensity for the CG model of the four-story frame buildings in the EW-loading direction; (b) ratio of dynamic to static overstrength factor versus first-mode period for the different analytical models of all archetype buildings*

4.3 COLLAPSE RISK ASSESSMENT

The collapse risk of the archetype frame buildings is evaluated using the mean annual frequency of collapse (λ_c). The mean annual frequency of collapse is an accurate collapse metric that takes into consideration all the spectral intensities that contribute to collapse risk of a frame building. The mean annual frequency of collapse is calculated by integrating the fragility curve (obtained from the IDA) over the corresponding seismic hazard curve as discussed in [6, 14]. The seismic hazard curves are obtained from the USGS website.

Figure 5 shows a dual plot where the values of λ_c and the corresponding probability of collapse in 50 years P_c(50 years) are plotted against the strong-column-weak-beam (SCWB) ratio implemented in the design for the CG models of all archetype in the EW-loading direction. Figure 5 shows that mid-rise frame buildings (i.e., 4 to 12 story) designed according to the current seismic provisions in the US [15] with SCWB ratio > 1.0 achieve a probability of collapse in 50 years larger than the 1% limit specified by [5]. Furthermore, a recent study by the authors [6] demonstated that SMFs designed with SCWB ratio > 1.0 experience bottom story collapse mechanisms as well as excessive panel zone shear distortion that could lead to weld fractures in fully restrained beam-to-column connections, when the composite slab is considered. Based on the same study, a SCWB > 1.5 should be implemented in the seismic design of SMFs in order to avoid the aforementioned problems and to achieve an acceptable probability of collapse. This is demonstrated in Figure 5 where buildings with SMFs designed with SCWB ratio > 1.5 achieve a probability of collapse lower than 1% in 50 years, considering both the composite slab and gravity framing effect. Furthermore, SMFs designed with SCWB ratio > 2.0 achieve a uniform probability of collapse of about 0.25% in 50 years in average.

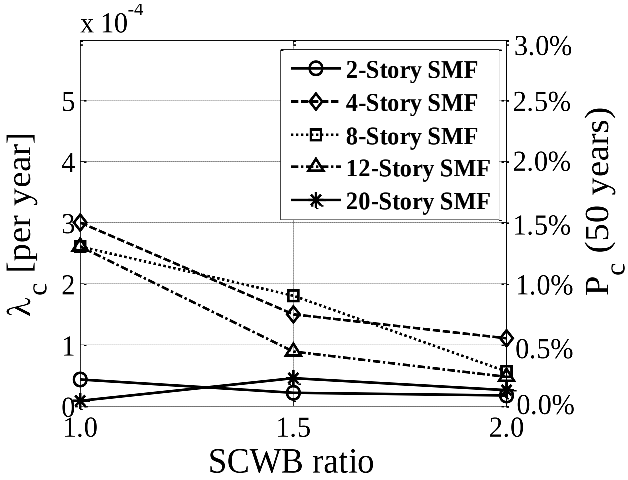


Figure 5. Mean annual frequency of collapse and the corresponding probability of collapse in 50 years versus SCWB ratio for the CG models of all archetype frame building in the EW loading direction

5. SUMMARY AND CONCLUSIONS

This paper discusses the effect of the gravity framing system on the overstrength and the collapse risk of steel buildings with perimeter SMFs designed in highly seismic regions in North America. Five archetype buildings with heights ranging from 2 to 20 stories are analysed using nonlinear static and response history analysis. The main conclusions from this study are summarized as follows:

- A static overstrength factor larger than 3.0 is only achieved for all archetype buildings when both the composite slab and the gravity framing system are considered as part of the analytical model.
- The dynamic overstrength factor for the B models is larger than 3.0. For the CG models, the dynamic overstrength is in average equal to 4.0 without any period dependency.

- Low to mid-rise SMFs designed with SCWB ratio > 1.0 achieve a probability of collapse in 50 years larger than the 1% limit specified by the current seismic provisions [15] even when the gravity framing system is considered as part of the analytical model. A SCWB ratio > 1.5 seem to be effective in terms of reducing the probability of collapse for such buildings over a period of 50 years less than 1%.

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ΣΥΝΕΙΣΦΟΡΑ ΤΟΥ ΣΥΣΤΗΜΑΤΟΣ ΜΕΤΑΛΛΙΚΩΝ ΠΛΑΙΣΙΩΝ ΒΑΡΥΤΗΤΑΣ ΣΤΗΝ ΥΠΕΡΑΝΤΟΧΗ ΚΑΙ ΣΤΗΝ ΑΠΟΦΥΓΗ ΤΗΣ ΣΕΙΣΜΙΚΗΣ ΚΑΤΑΡΡΕΥΣΗΣ ΚΤΙΡΙΩΝ ΜΕ ΜΕΤΑΛΛΙΚΑ ΠΛΑΙΣΙΑ ΚΑΜΨΗΣ

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ΠΕΡΙΛΗΨΗ

Κατά τον αντισεισμικό σχεδιασμό μεταλλικών κτιρίων στη βόρεια Αμερική, η συνεισφορά των μεταλλικών πλαισίων που φέρουν τα φορτία βαρύτητας στην παραλαβή σεισμικών δράσεων θεωρείται αμελητέα. Παλαιότερα πειραματικά δεδομένα έχουν δείξει ότι τα πλαίσια αυτά έχουν σημαντική αντοχή σε κάμψη με αποτέλεσμα να συνεισφέρουν σημαντικά στην υπεραντοχή ενός μεταλλικού κτιρίου. Η εργασία αυτή μελετά την συνεισφορά μεταλλικών πλαισίων βαρύτητας στην υπεραντοχή μεταλλικών κτιρίων με πλαίσια κάμψης όπως σχεδιάζονται στις Ηνωμένες Πολιτείες Αμερικής (ΗΠΑ). Η μελέτη βασίζεται σε αριθμητικά προσομοιώματα πρότυπων μεταλλικών κτιρίων από 2 εώς 20 ορόφους. Η προτεινόμενη υπεραντοχή με βάση τον αμερικάνικο αντισεισμικό κανονισμό για μεταλλικά κτίρια, όπως και η συνεισφορά των πλαισίων βαρύτητας στην αποφυγή της σεισμικής κατάρρευσης των κτιρίων υπό διερεύνηση αξιολογείται με τη χρήση μη γραμμικής στατικής και δυναμικής ανάλυσης με χρονική ολοκλήρωση. Με βάση τα αποτελέσματα, η υπεραντοχή που προκύπτει με χρήση στατικών μεθόδων ανάλυσης είναι μεγαλύτερη από την προτεινόμενη τιμή με βάση τον κανονισμό μόνο στην περίπτωση όπου τα πλαίσια βαρύτητας και η σύμμικτη δράση λαμβάνονται υπόψη στο αριθμητικό προσομοίωμα του εκάστοτε μεταλλικού κτιρίου. Η υπεραντοχή που προκύπτει με χρήση μη γραμμικών δυναμικών μεθόδων ανάλυσης είναι μεγαλύτερη από την προτεινόμενη τιμή του κανονισμού για όλα τα κτίρια που αναλύθηκαν ακόμα και στην περίπτωση όπου η συνεισφορά των πλαισίων βαρύτητας δεν λαμβάνεται υπόψη στην παραλαβή των σεισμικών δράσεων. Τέλος, η πιθανότητα σεισμικής κατάρρευσης των κτιρίων που εξετάστηκαν είναι μικρότερη από το επιτρεπτό όριο με βάση τον αμερικάνικο αντισεισμικό κανονισμό, μόνο στην περίπτωση όπου στον ικανοτικό έλεγχο κόμβων υποστυλωμάτων των μεταλλικών πλαίσιων κάμψης, ο λόγος αντοχής σε κάμψη των υποστυλωμάτων σε σχέση με την αντίστοιχη αντοχή σε κάμψη των δοκών είναι μεγαλύτερος από 1.5.