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Collapse risk assessment of a BRBF building considering the out-of-plane buckling of gusset plates and brace end-zones

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ABSTRACT

It is typical in research to adopt a 2D planar nonlinear modelling approach to simulate the response of buckling-restrained brace frames (BRBFs). This modelling approach, however, cannot simulate the out-of-plane buckling failure mode in BRBFs subjected to bi-directional earthquakes. This study develops a 3D modelling approach capable of addressing this shortcoming and applies it to a case study BRBF building to estimate its seismic collapse risk. The incorporation of gusset plate buckling in the BRBF modeling resulted in a 2.2-fold increase in the estimated collapse risk of the case study BRBF building compared to the estimates obtained from an analogous 2D planar model of the building. The results obtained from this study suggest that 2D planar models may not adequately capture the global seismic behavior of BRBF buildings. Moreover, the current design approach for gusset plates used in New Zealand may result in undersized gusset plates.

Keywords: Gusset plate buckling, BRBF modelling, and Collapse risk.

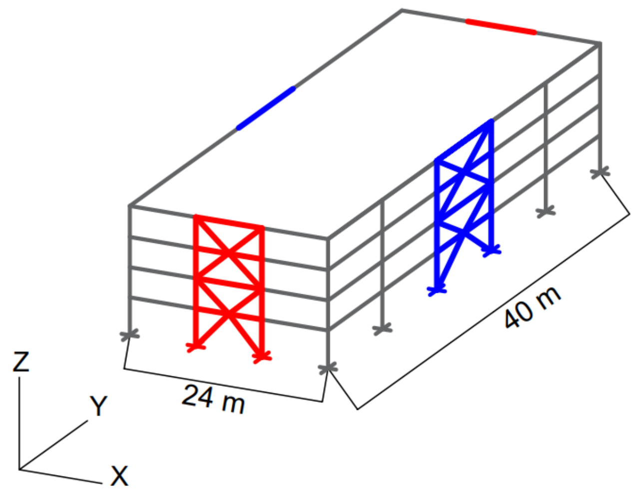
INTRODUCTION

Buildings with BRBFs are increasingly popular in New Zealand [1]. Several studies [2]–[6] have investigated the out-of-plane buckling failure mode of gusset plates and brace end-zones at the element and subassembly levels. They have highlighted the possibility of premature gusset plate buckling before the buckling-restrained braces (BRBs) can develop their full axial load carrying capacity. Vazquez et al. [4] developed 3D continuum finite element models of BRBFs at a subassembly level and demonstrated that bi-directional loading could exacerbate gusset plate buckling, resulting in a further 15% reduction in gusset plate strength compared to unidirectional loading. These models, however, would be computationally expensive for assessment of the global response of BRBF buildings and hence, the use of simplified 2D models is desired. However, the conventional 2D planar BRBF models cannot simulate the out-of-plane buckling failure mode [7]–[9]. These models generally ignore the continuous behavior of the gravity columns but consider the destabilizing effects of the gravity loads. This is accomplished through the modeling of pin-connected leaning columns, which lack lateral resistance but can effectively simulate the P-Δ effects. Furthermore, the conventional modelling approach cannot accurately simulate the response of BRBF buildings subjected to bi-directional ground motions.

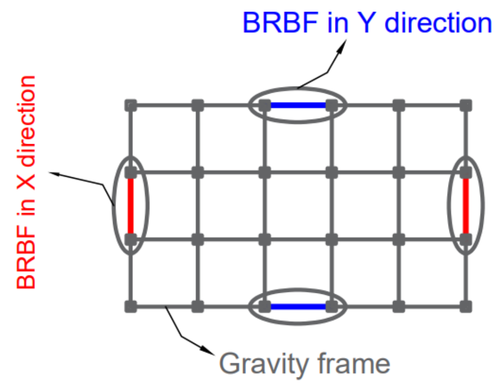
To address the aforementioned limitations, the authors developed a BRBF model in a previous study [10] that explicitly considers gusset plates and brace end-zones, enabling simulation of the out-of-plane buckling failure mode. Nevertheless, this model falls short in accurately simulating the global BRBF behavior under bi-directional loading, as it does not account for the continuous behavior of gravity columns and the participation of out-of-plane BRBF columns. Considering these shortcomings, this study develops an enhanced BRBF model that effectively addresses these issues and presents the resulting impacts on the assessed building response and collapse risk of a case study building.

DESIGN AND NON-LINEAR NUMERICAL MODELLING OF THE CASE STUDY BRBF BUILDING

The case study four-storey BRBF building (shown in Figure 1) adopted in this study was designed and analyzed previously in [10] and emulates the QuakeCoRE case study buildings examined by [11]. The rectangular floor plan of the building measures 24 m × 40 m, and is equipped with four BRBFs around its perimeter, as depicted in Figures 1a and 1b. The height of the first storey is 4.5 m, while all subsequent storeys are 3.6 m tall, with the bays spanning 8 m wide in both the X and Y directions. Remaining details and the final member sizes are summarized in Table 1, while the design process and calculations are covered elsewhere [10]. The strength of the building is governed by the serviceability limit state, while the peak storey drift ratio (SDR) is governed by the ultimate limit state (ULS).



(a) 3d view



(b) Plan view

Figure 1: 3D view and floor plan of the case study four-storey BRBF building

Table 1: Summary of design details for the case study BRBF building with reference to NZS1170.5 [12]

Method of Analysis		Equivalent Static
Site Subsoil Class		D
Importance Level		2
Design Life		50 years
Hazard Factor (Z)		0.3 (Christchurch)
Fundamental period (T_1)		0.97 sec
Ductility Factor (μ)		3.7
Structural performance factor (S_p)		0.7
Horizontal design action coefficient (C_d)		0.11
Peak storey drift from equivalent static analysis (storey 2)		1.65%
Summary of the final member sizes		
Member	Location	Section
Column	Storeys 1 and 2	350 WC 230
Column	Storeys 3 and 4	350 WC 197
Beam	Levels 1 and 2	530 UB 92.4
Beam	Levels 3 and 4	530 UB 92.4
Brace	Storeys 1 and 2	Rectangular section – 105 mm × 30 mm
Brace	Storeys 3 and 4	Rectangular section – 87 mm × 20 mm
Gusset plate	Storeys 1 and 2	Rectangular section – 435 mm × 425 mm × 16 mm
Gusset plate	Storeys 3 and 4	Rectangular section – 385 mm × 375 mm × 15 mm

As part of this study, two BRBF models were developed in OpenSees. The first model (model Planar) reflects the traditional approach to BRBF modelling, while the second model (model Non-planar-GP$_{buck}$) is an improved version that can address the limitations of the conventional modelling method. Typically, BRBFs are modelled as 2D planar frames [7]–[9]. One way to model the different parts of the frame is: i) represent the frame members using elastic beam column elements with plastic hinges using the Ibarra-Medina-Krawinkler (IMK) model to represent their hysteretic behavior, ii) represent BRBs and gusset plates using force-based fiber elements and stiff elastic elements, respectively, and iii) represent the gravity columns using a pin-connected leaning column to capture the destabilizing P-Δ effects due to the gravity loads. These leaning column elements are pin connected to the BRBF using rigid truss elements, ignoring their lateral resistance. The conventional modelling approach described above may not provide an accurate estimation of the case study building, as it lacks the capability to i) simulate the out-of-plane buckling of gusset plates, ii) account for bi-directional loading, iii) account for the continuous behavior of gravity columns, and iv) incorporate the involvement of out-of-plane BRBF columns.

In light of the limitations with a 2D modelling approach, an improved modelling approach referred here as model Non-planar-GP_{buck} is proposed to more accurately estimate the seismic behavior of BRBF buildings. The BRB core, BRB end-zones and the gusset plates are explicitly considered in this model, allowing for the simulation of out-of-plane buckling of gusset plates (as shown in Figure 2). Additional information specific to the BRB modelling can be found in a previous study by the authors [10]. Moreover, there are four leaning columns for each storey, whereas the case study building contains sixteen gravity columns (as seen in Figure 1b). In the numerical model, each leaning column represents four gravity columns in the building. Plastic hinges with elastic-perfectly plastic behavior were incorporated to account for yielding and limit the maximum forces borne by the gravity columns (as shown in Figure 2). To understand the behavior of the BRBF building subjected to bi-directional ground motions, all four BRBFs on the periphery of the building were modeled (as shown in Figures 3a and 3b). The slabs were assumed to have negligible post-cracking stiffness and were pin-connected to the frames to incorporate the in-plane slab effects only. Different modelling stages in the Non-planar-GP_{buck} model are highlighted in Figure 3.

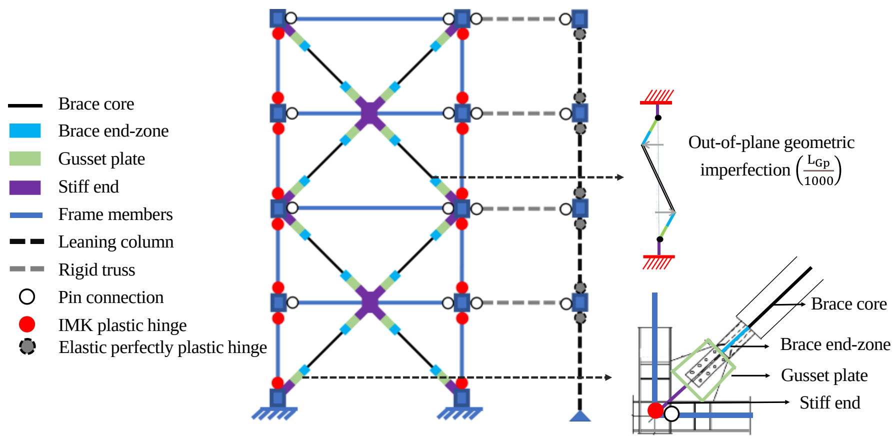


Figure 2 Detailed modelling schematic of a single BRBF within the model Non-planar-GP_{buck}

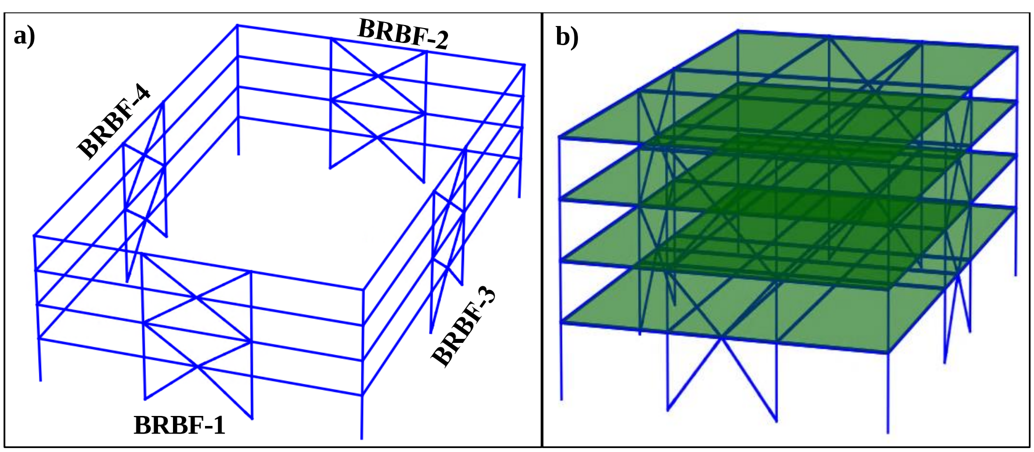


Figure 3 Building model schematic featuring BRBF frames, gravity system, and slabs in model Non-planar-GP_{buck}

NON-LINEAR DYNAMIC ANALYSIS OF THE CASE STUDY BUILDING

To assess the global structural performance of the case study building, two distinct models were developed. The first model, namely the "Planar" model is representative of a conventional 2D modeling approach and the " Non-planar-GP_{buck}" model adopts the proposed approach discussed in the previous section. The fundamental period of vibration for the Planar model is 0.954, while it is 0.945s for the Non-planar-GP_{buck} model. The lower period of the Non-planar-GP_{buck} model is attributed to its increased stiffness resulting from the inclusion of the out-of-plane BRBF columns. Multiple stripe non-linear response history analysis (MSA) is conducted by subjecting both models to 180 hazard-consistent ground motions at nine different seismic hazard levels. These levels are illustrated as cross marks on the hazard curve (Figure 4a). The ground motions were selected using the generalized conditional intensity measure approach [13], in a previous study conducted by Yeow et al [11]. The seismic hazard curve and the geometric mean of the pseudo-acceleration response spectra of the selected ground motion pairs at the intensity level 4 (IL-4) are shown in Figure 4b). $S_a(1s)$ of the selected ground motions is lower than the NZS 1170.5 ULS spectrum (IL-4). The conservatism observed in the NZS 1170.5 ULS spectrum mainly stems from two factors [11]. The first factor is the overestimation of the impact of small-to-moderate earthquakes by the ground motion prediction equation utilized in constructing the acceleration response spectrum of NZS 1170.5 [11]. The second factor is the adoption of a simplified spectral shape [11].

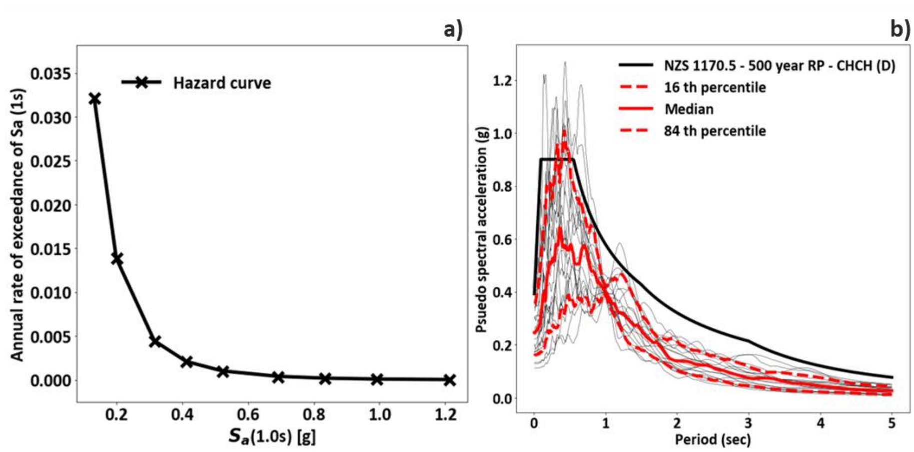


Figure 4 a) Seismic hazard curve with different intensity levels marked, and b) Geometric mean response spectra for the individual selected ground motion pairs at the ultimate limit state (ULS) level along with their median, 16th percentile and 84th percentiles

Although there are 180 ground motion pairs at nine different levels, the number of analyses required for the Planar model is twice as high as for the Non-planar-GP_{buck} model (360 vs. 180). It should be noted that for the Planar model, simulations in the X and Y directions are conducted independently as the planar model cannot simulate bi-directional response. Conversely, in the Non-planar-GP_{buck} model, the X and Y ground motion pairs of each ground motion are used together in a bi-directional simulation. The response of the Planar and Non-planar-GP_{buck} models is compared by calculating the maximum of the geometric mean of the peak response quantities obtained in the X and Y directions along the building height, as illustrated in Figure 5.

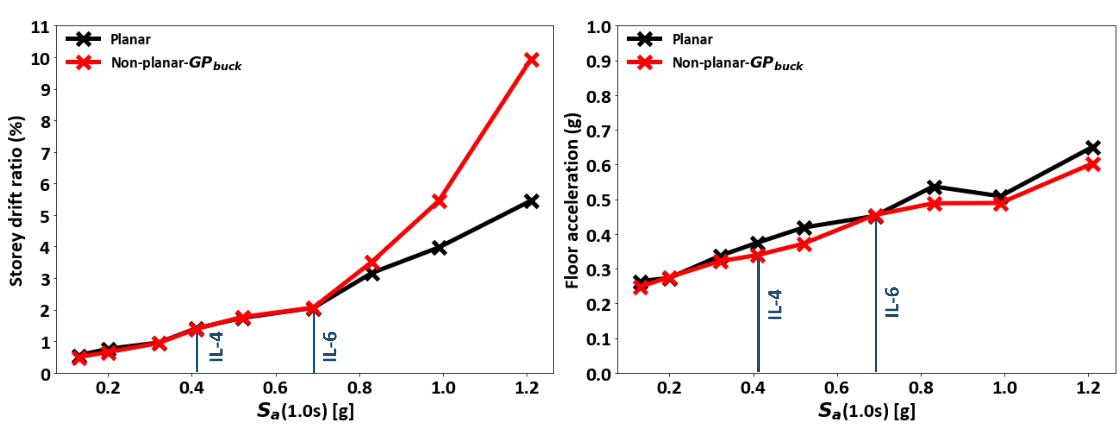


Figure 5 Maximum of the geometric mean of peak storey drift ratios and peak floor accelerations obtained in X and Y directions along the building height across various ground motion intensity levels

The peak SDRs and peak floor accelerations (FAs) obtained from MSA are shown in Figure 5, and there are no significant differences in the median up the building height between the Planar and Non-planar-GP_{buck} models up to IL-7 (2% POE in 50 years or 1 in 2475-year return period). This lack of difference is primarily because Non-planar-GP_{buck} model captures phenomena that occur after gusset plate buckling, and significant buckling of gusset plates was observed beyond IL-6. Buckling of gusset plates was primarily observed at the first floor, while buckling of at least one gusset plate in the X and Y direction BRBFs at this floor level was initially observed at IL-7, as illustrated in Figure 6. Furthermore, at IL-9, 10 out of 20 ground motions resulted in the buckling of all gusset plates at the first floor. It is noteworthy that at IL-9, only 5 collapse cases were estimated by the Planar model, while 11 cases were estimated by the Non-planar-GP_{buck} model. Hence, a major difference in peak SDR of both the models was observed at IL-9. Small differences in PFA at lower intensity levels are attributed to the out-of-plane imperfections in the gusset plate, which result in slight variations in building periods between the Planar and Non-planar-GP_{buck} models. In general, stiffer and stronger models demonstrate lower peak SDRs and higher peak FAs. Models capable of simulating the out-of-plane buckling failure mode tend to exhibit lower peak FAs, primarily attributed to the buckling of gusset plates, which results in reduced resistance and mitigates the inertia forces, subsequently leading to lower accelerations. This trend is clearly observed in Figure 5, where the Planar model, which is stiffer and stronger, exhibits lower peak SDRs and higher PFAs than the Non-planar-GP_{buck} model, which simulates out-of-plane buckling of gusset plates.

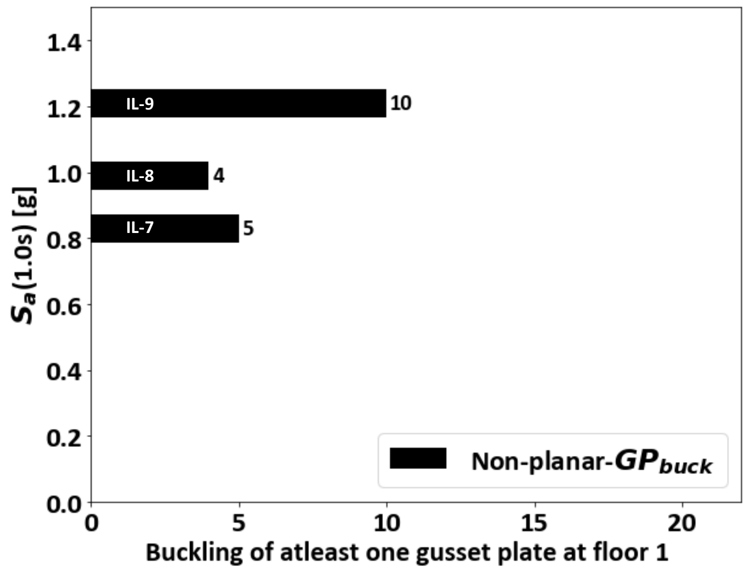


Figure 6 Number of ground motions causing buckling of all gusset plates at floor 1 as a function of earthquake intensity level

Figure 7 shows the maximum predicted peak residual SDR for the non-collapse cases across the entire building. According to FEMA 356 [14], a peak residual SDR limit of 1.0% is recommended for building demolition, as it indicates potential damage beyond feasible repair.

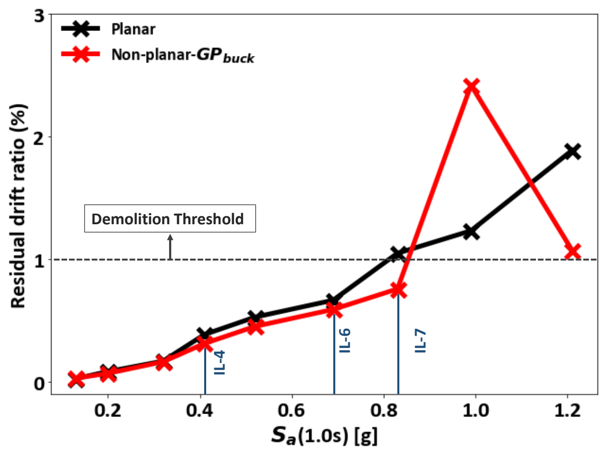


Figure 7 Comparison of peak residual storey drift ratio with ground motion intensity levels

Based on the analyses results, building demolition is unlikely to be necessary at the ULS level (IL-4). The peak residual SDR of the Non-planar-$\mathrm{GP_{buck}}$ model is typically equal to or lower than that of the Planar model until intensity levels below IL-7. This is mainly due to the presence of out-of-plane BRBF columns and continuous gravity columns in the Non-planar-$\mathrm{GP_{buck}}$ model, which assist in re-centering post-earthquake. Beyond IL-7, the dynamic behavior of the Non-planar-$\mathrm{GP_{buck}}$ model differs significantly from that of the Planar model due to gusset plate buckling, resulting in inconsistent peak residual SDR trends. This inconsistency may arise from a higher number of ground motions leading to collapse in the Non-planar-$\mathrm{GP_{buck}}$ model at IL-9, potentially distorting peak residual SDR statistics under non-collapse conditions. Both models predict that the peak residual SDRs exceed the 1% RSDR limit suggested by FEMA 356, indicating the need for building demolition beyond IL-7.

COLLAPSE RISK ASSESSMENT

The collapse risk of the case study building is quantified by estimating the mean annual rate of collapse (λ_C) (equation 1). $\mathbf{P(C|IM)}$ is the probability of collapse at a specific intensity level (or collapse fragility curve), with collapse defined as the attainment of a 10% storey drift ratio. Spectral acceleration at the fundamental period ($Sa(T_1)$) is the intensity measure (IM) adopted in this study, and $|\mathbf{d\lambda_{IM}}|$ is the absolute value of the hazard curve derivative.

$$\lambda_C = \int \mathbf{P(C|IM)}|\mathbf{d\lambda_{IM}}| \qquad (1)$$

The MSA analysis results of the Planar and Non-planar-$\mathrm{GP_{buck}}$ models were used to derive two collapse fragilities of the case study building, as shown in Figure 8. Figure 8a) illustrates the collapse fragilities derived directly from MSA analysis, including solely the record-to-record uncertainty (β_{rtr}). In contrast, Figure 8b) incorporates an additional modelling uncertainty (β_M) of 0.45, while maintaining the same median. The value of β_M was chosen based on the recommendations of Liel et al. [15]. The collapse fragility curve parameters and estimated λ_C values for both models are presented in Table 2.

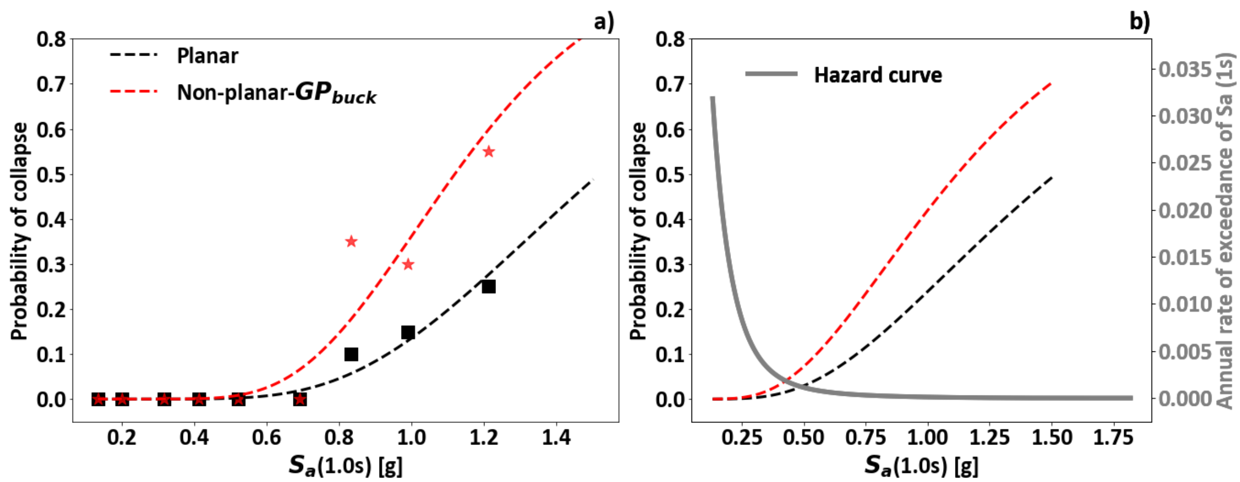


Figure 8 a) Comparison of collapse fragilities with record-to-record uncertainty only, and b) Comparison of collapse fragilities with revised standard deviation to include modelling uncertainty

Model Planar is assessed to possess an estimate of median collapse intensity that is 1.35 times larger than that of model Non-planar-$\mathrm{GP_{buck}}$, as it is unable to simulate gusset plate buckling. Moreover, the mean annual rate of collapse predicted by model Non-planar-$\mathrm{GP_{buck}}$ is almost 2.2 times that of model Planar. These findings can be attributed to the design criteria that resulted in gusset plates of lower strength than desired, as well as the enhanced modeling techniques employed.

Table 2: Summary of collapse fragility curve parameters and the mean annual rate of collapse

Model	$\theta(g)$	β_{rtr}	β_T	λ_c
Planar	1.52	0.38	0.58	1.62×10^{-4}
Non-planar-$\mathrm{GP_{buck}}$	1.12	0.32	0.55	3.54×10^{-4}
θ is the median collapse intensity of the collapse fragility curve				

COMPUTATIONAL COST

The differences in the modelling approach of the Planar and Non-planar-$\mathrm{GP_{buck}}$ models have been discussed earlier, and it is evident that the latter has a higher level of modelling complexity. Figure 9 displays the total computational time required in hours for both models at each intensity level. These run times were obtained using AMD EPYC 7F72 24-core processor. On average, the Non-planar-GP_{buck} model requires twice as much computational time compared to the Planar model.

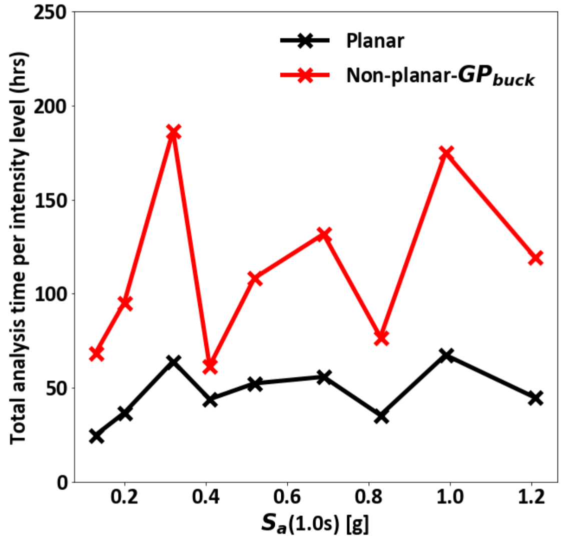


Figure 9 Comparison of total analysis time as a function of intensity level when analyses are run on an AMD EPYC 7F72 24-core processor

In the case of utilizing a single processor, the total computing time required to simulate 180 bi-directional ground motions across nine different intensity levels for the Planar and Non-planar-GP_{buck} models are approximately 17.2 days and 43 days, respectively. In this study, the master-slave architecture for parallel analysis execution [16] was adopted, resulting in a significant reduction in the total computation time. The parallel algorithm employed bears similarities to the MSA parallel algorithm discussed in Chapter 7 of [17]. A total of 40 processors were employed, with the first processor serving as the master and the remaining processors as the slaves. The adoption of parallel computing reduced the total computation time to approximately 12 hours and 25 hours for the Planar and Non-planar-GP_{buck}, respectively. Despite having twice the number of analyses, the average computation time for the Planar model is two times lower than that of the Non-planar-Gp_{buck} model.

If computational power is a constraint and the aim of the analysis is to understand the nonlinear response of BRBF buildings under moderate seismic events, the Planar model may be suitable. Nonetheless, for collapse risk assessment, the Planar model may not be appropriate if the gusset plates are poorly designed, as its behavior differs significantly from that of the Non-planar-GP_{buck} model at high intensity levels. If computing power is not a limitation, the Non-planar-GP_{buck} model should be adopted as it captures more features of BRBF buildings and ideally provides a better estimate when compared to the Planar model.

CONCLUSIONS

This study extends previous research by the authors, which demonstrated the influence of gusset plate buckling on the global seismic performance of BRBF buildings. While the conventional 2D planar BRBF model (referred to as model Planar in the study) is less complex and computationally more efficient than the improved model proposed in this study (referred to as Non-planar-GP_{buck}), it cannot simulate the buckling of gusset plates, the continuous gravity column effect, and the out-of-plane BRBF column behavior. The Planar model exhibits greater stiffness and strength in comparison to the Non-planar-GP_{buck} model due to its inability to simulate the out-of-plane buckling of gusset plates. Consequently, the former model displays lower peak SDRs and higher peak FAs than the latter model, as illustrated in Figure 5. Both models demonstrate the necessity of demolishing the building beyond the 1% POE in 50-year intensity level, as the residual storey drift ratios exceed a specified limit of 1%. Moreover, results obtained from the Non-planar-GP_{buck} model suggest that the gusset plates in the case study BRBF building are likely to be undersized by the current design method adopted in New Zealand. They are susceptible to buckling, starting from the design intensity level (ULS). The median collapse capacity/intensity of the case study building estimated by Non-planar-GP_{buck} model is 26% lower than that of the Planar model, while the mean annual rate of collapse was observed to be 2.2 times higher. These results demonstrate an increased seismic collapse risk due to gusset plate buckling. Further efforts are currently underway to improve the design method and mitigate the buckling of gusset plates.

ACKNOWLEDGMENTS

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