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ORIGINAL ARTICLE

Seismic performance of post-and-beam timber buildings II: reliability evaluations

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Abstract This paper presents an approach to evaluate the performance reliability of post-and-beam timber buildings under seismic excitation. The uncertainties considered include those associated with the earthquake ground motions, the structural mass and shear wall characteristics. The approach uses a verified structural model called "PB3D" for the creation of a database of seismic responses, which are then represented by appropriate response surfaces. These, in turn, are used to formulate explicit performance functions for the reliability analysis. Performance is studied in terms of peak inter-story drift, and polynomial functions are used to represent the seismic response surfaces. Non-performance probabilities are evaluated with respect to different performance expectations, using FORM and importance sampling methods. Case studies for two multi-story post-and-beam buildings are also presented.

Keywords Seismic performance · Timber buildings · Reliability analysis · Response surface method

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Introduction

Reliability evaluations of the seismic performance of timber buildings must consider the uncertainties inherent in the ground motions and in the structural capacity. The evaluation of the performance functions requires a robust structural analysis computer model, with good predictive accuracy and computational efficiency, as well as reliability evaluation software to estimate probabilities of non-performance. Seismic reliability of timber structures has been the subject of study by several researchers. A common approach is the use of fragility analysis, estimating first the probability of non-performance conditional on a given seismic hazard level. This conditional estimate is then integrated over the range of hazards to produce a final probability. This method of uncoupling the calculation has been widely used by researchers [1–4] to study the seismic reliabilities of wood-frame shear walls and buildings. Other studies include Ceccotti and Foschi [5], who calibrated a seismic design factor for wood shear walls in the National Building Code of Canada. Random variables used were the peak ground accelerations (PGA) and wall hysteretic parameters. The structural analysis must incorporate a good representation for the hysteretic behavior of connections. Different types of hysteresis models have been used, usually calibrated to experimental data for a specific cyclic loading history. For example, Foliente [6] used Monte Carlo simulations (MCS) and the modified BWBN hysteretic model to estimate the seismic reliability of a series of Japanese walls. Paevere and Foliente [7] also used MCS and the BWBN model to study the influence of the pinching effect and stiffness degradation on wood shear wall capacity and seismic reliabilities.

Fragility estimations usually correspond to a particular structural configuration. Should any characteristic change, for example, the nail spacing in a wall or the amount of shear walls in a building, a new fragility estimate must be developed. The approach presented in this paper shows an alternative by means of which a structural response database is developed for a range of both strength characteristics and design variables. Such database, obtained from deterministic dynamic analyses, can then be represented by explicit functions or response surfaces. Once these functions are obtained, they provide a very effective tool for reliability estimation and optimization of design variables for performance-based design.

This database representation can use polynomial functions or neural networks. Zhang and Foschi [8] used the neural network method to study the seismic reliability of light-frame walls considering the randomness involved in ground motions, structural mass and nailing spacing. Foschi [9] also developed a framework to estimate the seismic reliabilities of post-and-beam shear walls using the neural network method. Li et al. [10] studied the seismic reliabilities of eight types of post-and-beam walls using the response surface method (RSM) with polynomial functions and importance sampling (IS) for probability estimation. The uncertainties involved were those from the ground motion and the structural mass.

Crude MCS might not be suitable when it requires nonlinear time history analyses of complicated systems, because this is very computational intensive. Alternatively, a RSM database can be developed by carrying out a reduced number of simulations at discrete sampling points of the random variables. A structural response surface can be fitted using either mathematical functions or training a neural network. Reliability evaluation tools, such as first-order or second-order reliability methods (FORM/SORM), or reduced-variance simulation techniques like importance sampling, can then be used efficiently to estimate failure probabilities. This paper presents an application of this RSM reliability methodology to the seismic reliability of post-and-beam timber buildings. A verified computer structural model called "PB3D" was used to establish a seismic response database for buildings including the variables such as the seismic ground motions, the structural mass and the design characteristics of shear walls in the buildings. The introduction and verification of the "PB3D" model has been presented in a companion paper [11].

Theory

The differences in observed damages in wood buildings subjected to earthquake ground motions must be related to differences in their structural deformations, which are controlled by the characteristics of ground motions including PGA or peak ground velocity (PGV), frequency contents, and duration of shaking as well as structural dynamic characteristics and local site conditions. The variability in the structural deformations is thus related to the variability in the entire earthquake record. Studies of eight significant earthquakes in California [12] indicated that for moderate earthquakes, PGA might be a good damage indicator and for severe earthquakes, PGV was more often correlated with structural damages. Based on the survey statistics after the 1995 Kobe earthquake, Onishi and Hayashi [13] also established fragility curves between PGV or peak velocity response and the damage levels of wood houses considering structural deteriorations due to aging. Therefore, to estimate seismic reliability, the description of ground motions is essential. The seismic analysis yields the structural deformation including the inter-story drift. The structural deformation response should be linked to the damage experienced by the structure, that is, there should also be a "damage function" relating the structural deformation to the damage level. This function is, however, not easy to determine, as it may be quite subjective, requiring extensive experimental data and evaluation by damage or insurance engineers. The relationship between the damage and the structural deformation would always be an increasing function with respect to the deformation. In this paper, however, a simplifying assumption is made that the damage in a wood building could be equated to the peak inter-story drift (PID) and the PID response of the entire building is therefore selected to formulate the performance functions. Of course, the method described in the paper would be entirely valid should such a relationship between the damage and the PID be available. Other damage criteria, such as the maximum base shear force, might also be used, as long as their relationship to the damage could be established.

The performance function is given as

$$G = \delta - \Delta(a_{\mathrm{G}}, r, M, F_{\mathrm{d}}, \varepsilon) \tag{1}$$

in which δ is the inter-story drift capacity of the building; Δ is the PID demand which involves the characteristics of the ground motion r, structural mass M; design factors of interest F_{d} (e.g., total length of shear walls or others) and the response surface fitting error ε. The intensity measure a_{G} of ground motions can be represented by PGA, PGV, pseudo-spectral acceleration or pseudo-spectral velocity. r represents other characteristics of ground motions and considers the record-to-record variability. Normally, a group of earthquake records should be used to consider such variability in accelerograms.

A seismic response database needs to be established first by running time history analyses for the selected combinations of random variables over their defined domain. For each combination of random variables, and over a suite of earthquakes, it is possible to obtain the mean $\overline{\Delta}_{\mathrm{sm}}$ and standard deviation $\sigma_{\Delta \mathrm{sm}}$ of the PID responses over the different ground motions. For all the selected variable combinations, a discrete set of $\overline{\Delta}_{\mathrm{sm}}$ and a discrete set of $\sigma_{\Delta \mathrm{sm}}$ can be obtained, respectively. Then, polynomial functions, Eqs. 2a and b, are used to fit or represent these sets of PIDs over the domain of random variables, respectively. A boundary condition has been applied here in which the building response vanishes when structural mass or PGA is equal to zero [14].

$$\overline{\Delta}_{\mathrm{rs}} = \sum a_{ijk} a_{\mathrm{G}}^{i} M^{j} F_{\mathrm{d}}^{k} \tag{2a}$$

$$\sigma_{\Delta \mathrm{rs}} = \sum b_{ijk} a_{\mathrm{G}}^{i} M^{j} F_{\mathrm{d}}^{k} \tag{2b}$$

where a_{ijk} and b_{ijk} are coefficients evaluated by minimizing the squared error between the polynomial fitting and the model simulation results; and superscripts i, j, and k are the orders of polynomials.

Now taking into account the RS fitting errors, the mean and standard deviation of the peak responses can be adjusted to

$$\overline{\Delta} = \overline{\Delta}_{\mathrm{rs}} \left(1 - \varepsilon_{\overline{\Delta}}\right) \tag{3a}$$

$$\sigma_{\Delta} = \sigma_{\Delta \mathrm{rs}} \left(1 - \varepsilon_{\sigma_{\Delta}}\right) \tag{3b}$$

where $\varepsilon_{\overline{\Delta}}$ and $\varepsilon_{\sigma_{\Delta}}$ are random variables representing RS fitting errors and assumed to follow normal distributions. The errors of the generic ith combination of the random variables are calculated by Eqs. 4a and b.

$$\varepsilon_{\overline{\Delta}}^{i} = \frac{\overline{\Delta}_{\mathrm{rs}}^{i} - \overline{\Delta}_{\mathrm{sm}}^{i}}{\overline{\Delta}_{\mathrm{rs}}^{i}} \tag{4a}$$

$$\varepsilon_{\sigma\Delta}^{i} = \frac{\sigma_{\Delta \mathrm{rs}}^{i} - \sigma_{\Delta \mathrm{sm}}^{i}}{\sigma_{\Delta \mathrm{rs}}^{i}} \tag{4b}$$

The mean and standard deviation of the overall fitting errors can be obtained when all combinations are considered.

Using the assumption that PID responses follow a log-normal distribution, the performance function Eq. 1 can be rewritten as

$$G = \alpha H - \frac{\overline{\Delta}}{\sqrt{1 + v_{\Delta}^{2}}} \exp\left(R_{\mathrm{N}} \sqrt{\ln\left(1 + v_{\Delta}^{2}\right)}\right) \tag{5}$$

where αH is the specified inter-story drift capacity (α is drift ratio limit and H is the story height); $\overline{\Delta}$ is the mean of PID demand; v_{Δ} is the coefficient of variation $\sigma_{\Delta}/\overline{\Delta}$; and R_{N} is the standard normal variate $R_{\mathrm{N}}(0,1)$. $\overline{\Delta}$ and σ_{Δ} are given by Eqs. 3a and b, respectively.

Once the explicit performance function Eq. 5 is obtained, and probability distributions for the random variables are given, the reliability index β can be estimated by FORM or importance sampling (IS) simulations.

Considering the high nonlinearity in the performance function, IS can be further used by centering a sampling distribution near the design point, i.e., a region of most importance or likelihood of non-performance [15–17]. Rewriting the integration function of failure probability following the IS method, one has

$$P_{\mathrm{f}} = \int_{D} I(x)[f(x)/h(x)]h(x)\mathrm{d}x \tag{6}$$

where x is the vector of random variables, the index $I(x)$ is such that $I(x) = 0$ defines non-failure and $I(x) = 1$ implies failure; $f(x)$ is the joint density function of random variables x, centered at the means; $h(x)$ is the sampling joint density function centered at the design point x_{d}. Thus, the failure probability can be estimated by the average of the function $I(x)[f(x)/h(x)]$ over the sampling domain around the design point:

$$P_{\mathrm{f}} = \frac{1}{N} \sum I(x)[f(x)/h(x)] \tag{7}$$

Case studies

Case studies included a two-story post-and-beam building and a three-story post-and-beam building tested on shake tables in Japan. For the two cases, a total of ten historical earthquake records in Japan (Table 1) were used to consider the influence of record-to-record variations. These records were obtained from Building Research Institute of

Table 1 Historical earthquake records used in case study

Event	Year	Comp	PGA (g)	Station
Tokyo	1956	NS	0.0755	Tokyo 101
Northern Miyagi-oki	1962	EW	0.0485	Sendai 501
Head land of Echizen	1963	EW	0.0255	Osaka 205
Tokachi-oki	1968	EW	0.1866	Hachinohe
Miyaki-oki	1978	NS	0.2634	Tohoku
Hyogo-ken Nanbu	1995	NS	0.8365	JMA Kobe
Hyogo-ken Nanbu	1995	NS	0.2432	Shin Osaka
Hyogo-ken Nanbu	1995	EW	0.6155	Takatori
Chuetsu	2004	NS	0.4736	NIT
Niigata	2007	EW	0.2522	OJP2

Japan [18]. Since only one horizontal component is available for the first five records, the seismic simulations used one-directional ground motions along the weaker direction of the buildings with fewer shear walls.

A two-story building

The building had a plan size of 7.28 m × 7.28 m and a story height of 2.73 m. The timber frame was constructed using European whitewood glulam. The exterior shear walls were sheathed by 7.5 mm thick plywood panels and 12.5 mm thick gypsum wall boards (GWB). JIS N50 nails were used and nail spacing was 150 mm. The interior shear walls were constructed by 45 mm × 90 mm diagonal braces. The total effective length of existing shear walls in the first story just satisfied the minimum requirement stipulated by the Building Standard Law (BSL) in Japan. The maximum eccentricity ratio was 0.19, also satisfying the code requirement. In the shake table test, the building was excited by 100% Kobe JMA ground motions. The building experienced a lot of structural damage in the first story and was at a near collapse state. Figure 1 shows the tested building as well as the corresponding "PB3D" model. The model predictions agreed very well with the test results. Li [19] presented detailed information about the model verification using this building.

For this building, the performance function considered uncertainties of seismic intensity represented by PGA, structural mass M and the RS fitting errors. It is written as

$$G = \delta - \Delta(a_{\mathrm{G}}, M, \varepsilon) \tag{8}$$

Ten PGA levels (0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, and 1.0 g) and five structural mass levels (130, 150, 170, 190, and 210 kg/m^2) were used to generate the seismic response database. Table 2 gives the statistics in terms of mean and standard deviations of the PIDs for all selected combinations of PGA and structural mass.

Third-order polynomials with nine coefficients were used to fit $\overline{\Delta}_{\mathrm{sm}}$ and $\sigma_{\Delta\mathrm{sm}}$ of PIDs from all the combinations of random variables:

$$\begin{aligned}\overline{\Delta}_{\mathrm{rs}} = {} & a_1 a_{\mathrm{G}} M + a_2 a_{\mathrm{G}} M^2 + a_3 a_{\mathrm{G}}^2 M \\ & + a_4 a_{\mathrm{G}}^2 M^2 + a_5 a_{\mathrm{G}} M^3 + a_6 a_{\mathrm{G}}^3 M \\ & + a_7 a_{\mathrm{G}}^2 M^3 + a_8 a_{\mathrm{G}}^3 M^2 + a_9 a_{\mathrm{G}}^3 M^3\end{aligned} \tag{9a}$$

$$\begin{aligned}\sigma_{\Delta\mathrm{rs}} = {} & b_1 a_{\mathrm{G}} M + b_2 a_{\mathrm{G}} M^2 + b_3 a_{\mathrm{G}}^2 M \\ & + b_4 a_{\mathrm{G}}^2 M^2 + b_5 a_{\mathrm{G}} M^3 + b_6 a_{\mathrm{G}}^3 M \\ & + b_7 a_{\mathrm{G}}^2 M^3 + b_8 a_{\mathrm{G}}^3 M^2 + b_9 a_{\mathrm{G}}^3 M^3\end{aligned} \tag{9b}$$

Figure 2 shows the polynomial fitting errors for all sets of $\overline{\Delta}_{\mathrm{sm}}$ and $\sigma_{\Delta\mathrm{sm}}$. Good fitting can be observed, with small fitting errors and most of the data points located near the 45°, i.e., the perfect agreement line. Figure 3 shows the polynomial response surfaces with respect to PGA and structural mass as well as the fitted polynomial coefficients.

The seismic reliability of the building can then be estimated based on assumptions of seismic hazards and structural mass. It was assumed that this building was constructed in a high seismicity zone. In a 50-year period, the PGAs of earthquakes follow a lognormal distribution with mean of 0.25 g and COV of 0.60. This, coupled with the assumption of an annual Poisson arrival rate 0.1/year, results in a design earthquake with PGA of 0.66 g with exceedance probability of 10% in 50 years or a design earthquake with PGA of 0.93 g with exceedance probability of 2% in 50 years. The structural mass was assumed to follow a lognormal distribution with COV of 0.1. The seismic reliabilities were estimated with respect to different structural mass levels and three performance expectations: inter-story drift ratio limit of 1% for immediate occupancy (IO); 2% for life safety (LS); and 3% for collapse prevention (CP) according to FEMA [20].

Figure 4 gives the FORM and IS results of the reliability indices β_{e} with respect to different mass levels and performance expectations. It can be observed that FORM and IS gave very close failure probabilities and IS results were slightly higher. Based on these results, when the structural mass increased by 20 kg/m^2, the reliability index β decreased by 0.15–0.20 with respect to each performance expectation. Given the structural mass, the difference

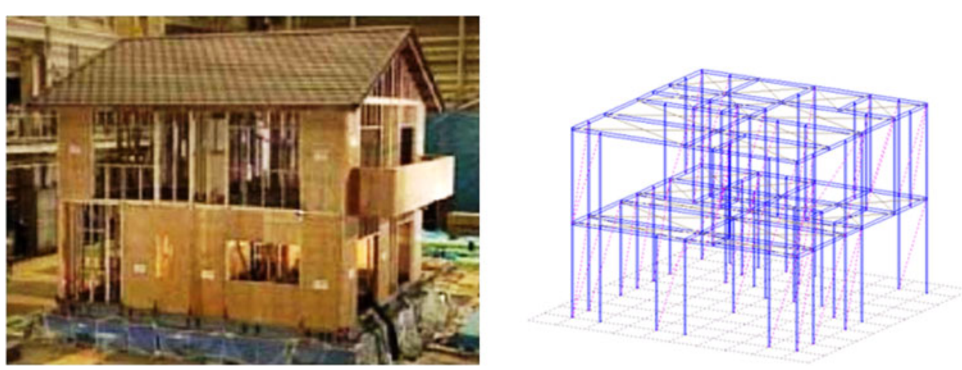


Fig. 1 Two-story building and the "PB3D" model

Table 2 Statistical data of PIDs (mm) from seismic simulations of two-story building

PGA (g)	Structural mass level									
	130 kg/m^2		150 kg/m^2		170 kg/m^2		190 kg/m^2		210 kg/m^2	
	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$
0.1	2.6	0.5	3.1	0.7	3.6	0.7	4.1	0.7	4.7	0.8
0.2	7.4	0.8	8.8	1.2	10.7	2	12.5	2.8	14.4	3.7
0.3	14.4	2.5	17.8	3.9	21.2	5.7	25	8.1	28.9	10.6
0.4	23.6	5.4	29.1	8.5	35	12.4	41.1	16.3	46.6	20.1
0.5	35.1	10.2	43.5	15.7	52.1	21.4	60	26.9	69.4	35.7
0.6	49.1	17.4	61.1	25.6	73.4	34.7	86.2	46.7	95.9	54.2
0.7	66.4	27.7	85.3	42.5	104.4	58.8	113.9	65.9	125.6	77.2
0.8	92.7	47.5	115.6	64.8	136.5	84.3	149.1	93.5	160.3	102.7
0.9	125	74.3	155.9	93.1	176.6	113.8	192.3	121.2	201	131.3
1.0	173.4	102.6	215.4	138.4	227.9	157.4	245.4	160.6	260.7	164.6

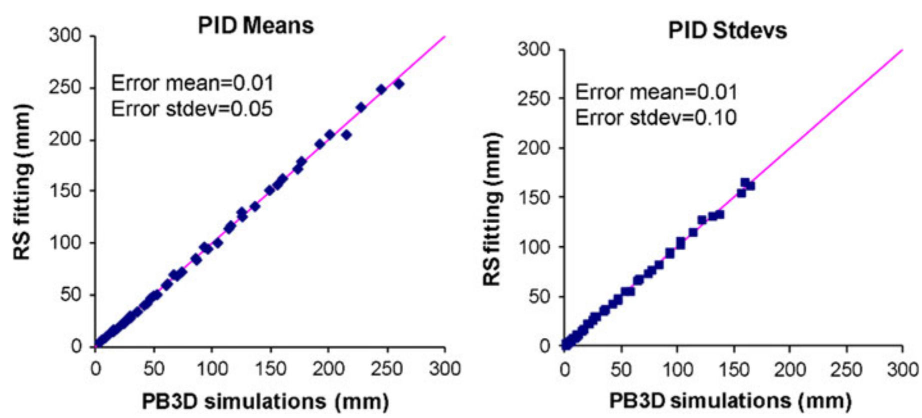


Fig. 2 Polynomial RS fitting errors (two-story building)

between the β values with respect to the LS performance expectation and the β values with respect to the IO performance expectation was about 0.6–0.7. And the β difference between CP performance expectation and LS performance expectation was about 0.3–0.35. Interestingly, for each performance expectation, the relationship between the structural mass and reliability indices is fairly linear, as shown in Fig. 7.

A three-story building

Figure 5 shows the three-story building and the "PB3D" model. The model verification by this building has been introduced in the companion paper [11]. This building was designed according to the allowable stress design (ASD) method for wooden post-and-beam construction [21]. The total effective length of shear walls in each story and the eccentricity ratios of the building also satisfied the BSL. The seismic zone factor Z and the vibration characteristic factor R_t were both 1.0. The basic base shear coefficient was 0.2. The design of framing members and joints in shear walls, such as hold-down connections, all followed the guidelines in the ASD design method. And the shear capacity of the floors was also calculated according to the guidelines.

For this building, the performance function considered PGA, the RS fitting errors and a design factor F_d called lateral force resisting factor (LFRF). This factor is defined as the ratio between the total allowable shear forces of designed shear walls and the minimum requirement by seismic loads stipulated by the BSL. The structural masses followed the design code and were the same as used in the shake table test (228 kg/m^2 for the 2nd floor, 225 kg/m^2 for the 3rd floor and 159 kg/m^2 for the roof).

The performance function was then given as

$$G = \delta - \Delta(a_G, F_d, \varepsilon) \qquad (10)$$

Ten PGA levels (0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, and 1.0 g) and five LFRF values (2.0, 2.5, 3.0, 3.5, and 4.0) were used to generate the building response database. The original design of shear walls in this building was governed by wind loads rather than seismic loads. It turned out that, for the first story, the required minimum amount of shear

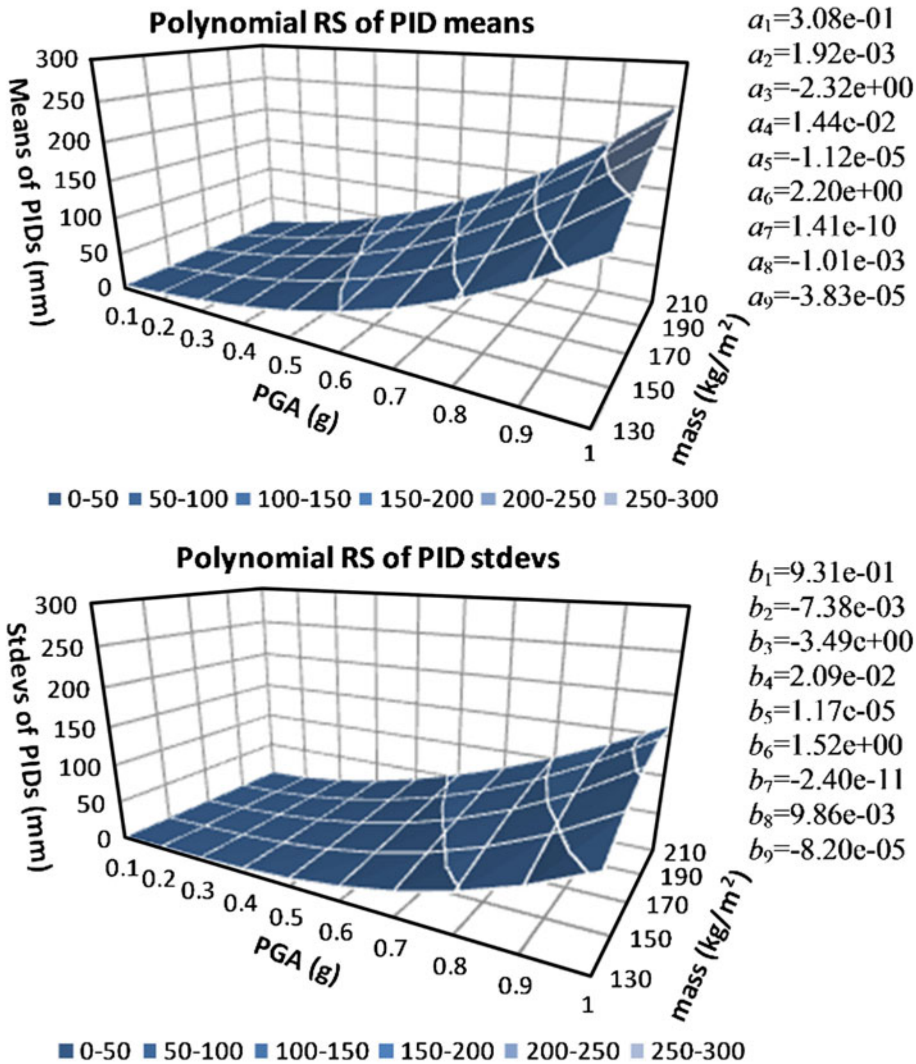


Fig. 3 Polynomial RS for PID mean and SD (two-story building)

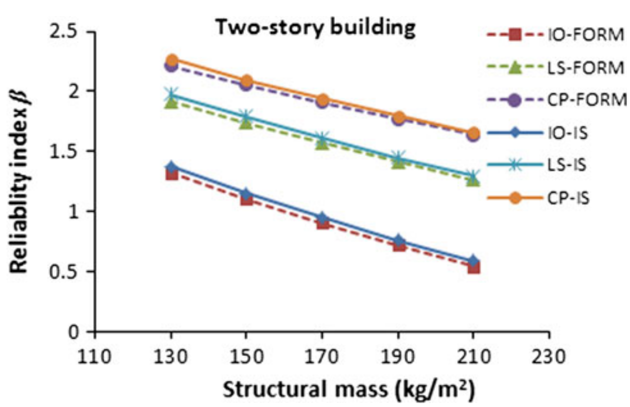


Fig. 4 Reliability indices of the two-story building

walls by the wind loads is 1.75 times of the minimum requirement by the earthquake loads. Therefore, in this case study, the LFRF values were chosen to start from 2.0 in the seismic simulations. Table 3 gives the statistics in terms of mean and standard deviations of the PIDs with respect to all combinations of PGAs and LFRFs.

Now, the RS polynomials for the mean and standard deviations of the PIDs become

$$\overline{\Delta}_{\mathrm{rs}} = a_1 a_{\mathrm{G}} F_{\mathrm{d}} + a_2 a_{\mathrm{G}} F_{\mathrm{d}}^2 + a_3 a_{\mathrm{G}}^2 F_{\mathrm{d}} + a_4 a_{\mathrm{G}}^2 F_{\mathrm{d}}^2 + a_5 a_{\mathrm{G}} F_{\mathrm{d}}^3 + a_6 a_{\mathrm{G}}^3 F_{\mathrm{d}} + a_7 a_{\mathrm{G}}^2 F_{\mathrm{d}}^3 + a_8 a_{\mathrm{G}}^3 F_{\mathrm{d}}^2 + a_9 a_{\mathrm{G}}^3 F_{\mathrm{d}}^3 \tag{11a}$$

$$\sigma_{\Delta \mathrm{rs}} = b_1 a_{\mathrm{G}} F_{\mathrm{d}} + b_2 a_{\mathrm{G}} F_{\mathrm{d}}^2 + b_3 a_{\mathrm{G}}^2 F_{\mathrm{d}} + b_4 a_{\mathrm{G}}^2 F_{\mathrm{d}}^2 + b_5 a_{\mathrm{G}} F_{\mathrm{d}}^3 + b_6 a_{\mathrm{G}}^3 F_{\mathrm{d}} + b_7 a_{\mathrm{G}}^2 F_{\mathrm{d}}^3 + b_8 a_{\mathrm{G}}^3 F_{\mathrm{d}}^2 + b_9 a_{\mathrm{G}}^3 F_{\mathrm{d}}^3 \tag{11b}$$

Figure 6 shows the accuracy of the fit for the response surfaces. Figure 7 shows the polynomial response surfaces with respect to PGA and LFRF.

The same assumption on the seismic hazard was used to estimate the seismic reliability for this building. The LFRF was assumed to follow a normal distribution, with a mean equal to the given value and with COV of 0.05, considering

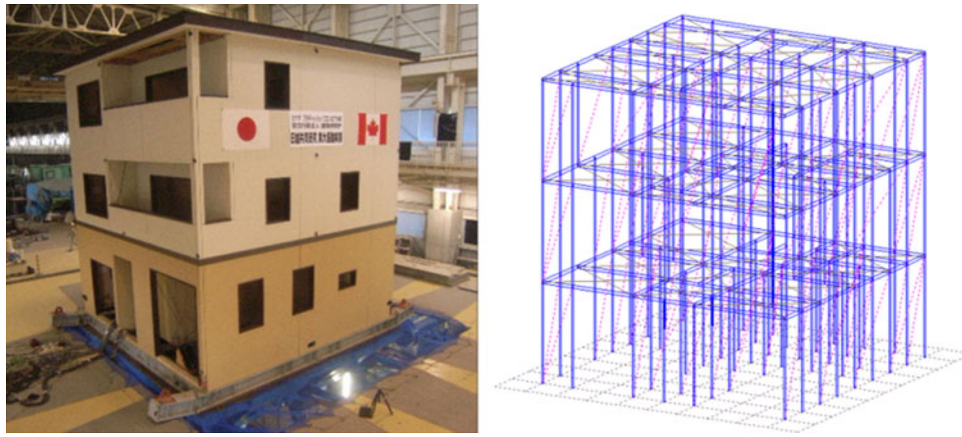


Fig. 5 Three-story building and the “PB3D” model

Table 3 Statistic data of PIDs from seismic simulations of three-story building

PGA (g)	LFRD									
	4.0		3.5		3.0		2.5		2.0	
	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$	$\overline{\Delta}_{sm}$	$\sigma_{\Delta sm}$
0.1	1.6	0.4	1.9	0.4	2.4	0.7	2.8	0.8	3.2	0.8
0.2	3.8	1.1	4.4	1.2	6.1	2.0	7.0	2.1	7.9	2.0
0.3	6.9	1.9	7.7	2.0	11.0	3.2	12.3	3.0	14.2	3.1
0.4	10.7	3.2	12.3	3.5	16.9	3.8	18.7	3.8	22.0	4.2
0.5	15.4	4.2	17.6	4.4	23.4	4.4	26.6	3.7	33.7	5.1
0.6	21.3	5.5	24.0	5.7	31.9	7.0	39.3	7.2	47.5	8.8
0.7	27.9	7.3	31.7	7.0	44.1	12.7	56.1	15.5	67.4	18.6
0.8	35.7	9.9	41.6	9.3	61.0	22.0	77.4	28.3	94.9	33.1
0.9	44.7	13.1	54.5	12.5	81.8	35.3	112.3	54.8	131.6	56.7
1.0	56.1	17.1	73.1	19.4	105.7	51.1	132.6	65.0	146.5	57.3

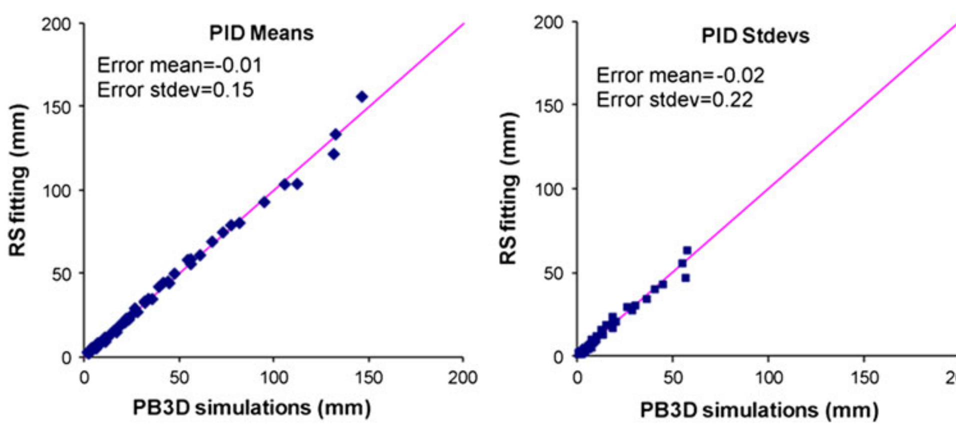
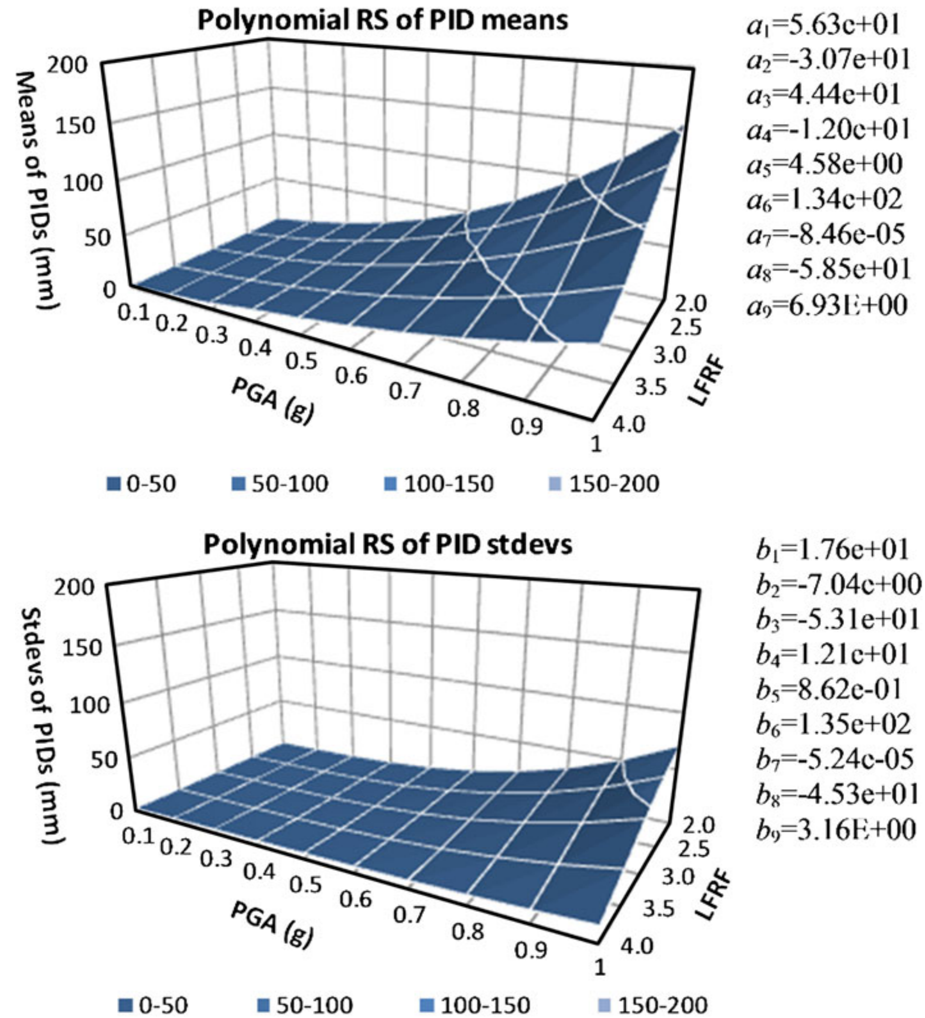


Fig. 6 Polynomial RS fitting errors (three-story building)

the variability in construction quality. Figure 8 shows the FORM and IS results of the event reliability indices β_e with respect to five LFRF values and three performance expectations.

Given the structural mass and seismic hazard assumptions, the results imply that, for this building, should the lateral force resistance provided by shear walls be increased by 50% of the seismic code minimum requirement, without significantly changing the building eccentricity, the reliability index β would increase by 0.20–0.30 in each performance expectation. The relationship between desired safety margins in each performance level, and the code design requirements, can thus be quantified by the proposed reliability analysis. If target reliabilities are set, an optimized shear wall design can thus be achieved.

Fig. 7 Polynomial RS for PID mean and SD (three-story building)



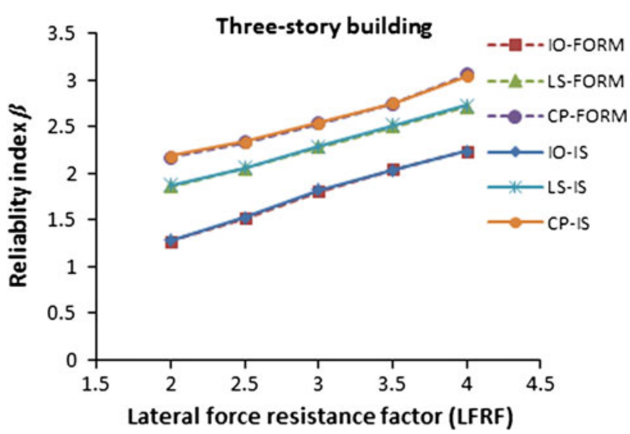


Fig. 8 Reliability indices of the three-story building

Conclusions

This paper presented an efficient approach to study the seismic reliability of post-and-beam timber buildings using the response surface method. The seismic simulations of the buildings were carried out by a verified computer model “PB3D”. The calculated building seismic responses were fitted by third-order polynomial functions, with the fitting errors taken into account as additional random variables. Peak inter-story drift response was used as the criteria to formulate the performance functions. Given statistics for seismic hazard, FORM and importance sampling simulation were then used to estimate the failure probabilities with respect to three different performance expectations. Case studies on two post-and-beam buildings were presented. For the two-story building, the formulation of the performance functions considered the randomness in ground motions, structural mass and response surface fitting errors. For case for the three-story building considered the randomness in ground motions, the amount of designed shear walls and the response surface fitting errors.

The procedure presented in this paper provides a very useful tool to evaluate the seismic performance of post-and-beam timber buildings in a probabilistic-based manner. The performance function can take into account the uncertainties of seismic ground motions and any important design parameters such as mass, shear walls and building eccentricity. By doing so, the optimization of seismic design can be achieved by satisfying specified target reliabilities.

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