Seismic Reliability Assessment of Mid- and High-rise Post-tensioned CLT Shear Wall Structures

Xiaofeng Sun, Zheng Li, and Minjuan He†

Department of Structural Engineering, Tongji University, Shanghai, 200092, China

Abstract

Currently, few studies have been conducted to comprehend the seismic reliability of post-tensioned (PT) CLT shear wall structures, due to the complexity of this kind of structural system as well as due to lack of a reliable structural model. In this paper, a set of 4-, 8-, 12-, and 16-storey benchmark PT CLT shear wall structures (PT-CLTstrs) were designed using the direct displacement-based design method, and their calibrated structural models were developed. The seismic reliability of each PT-CLTstr was assessed based on the fragility analysis and based on the response surface method (RSM), respectively. The fragility-based reliability index and the RSM-based reliability index were then compared, for each PT-CLTstr and for each seismic hazard level. Results show that the RSM-based reliabilities are slightly less than the fragility-based reliabilities. Overall, both the RSM and the fragility-based reliability method can be used as efficient approaches for assessing the seismic reliabilities of the PT-CLTstrs. For these studied mid- and high-rise benchmark PT-CLTstrs, following their fragility-based reliabilities, the 8-storey PT-CLTstr is subjected to the least seismic vulnerability; while, following their RSM-based reliabilities, the 4-storey PT-CLTstr is subjected to the least seismic vulnerability.

Keywords: Cross-laminated timber, post-tensioned timber structure, seismic reliability, fragility analysis, response surface method.

1. Introduction

Cross-laminated timber (CLT) is suitable for shear wall applications in mid- and high-rise timber structures, due to its positive characteristics including high in-plane shear strength, ideal integrity, good thermal insulation performance, etc. A series of tests were conducted on CLT shear walls (Gavric et al. 2015; Pai et al. 2017; Deng et al. 2019; Shahnewaz et al. 2019) and on CLT structural systems (Cecotti et al. 2013; Gavric et al. 2015; Porcu et al. 2018; van de Lindt et al. 2019), in which the CLT wall panels were connected to the foundation or the floor diaphragm using metal connections (e.g., hold-downs and angle brackets). It was attested that the CLT shear walls were always deforming as rigid bodies in the tests, and the areas of the metal connections were prone to premature damages. For overcoming this damage mode commonly occurring in the CLT shear walls and for enhancing their lateral performance, the concept of self-centering massive timber (i.e., CLT or laminated veneer lumber (LVL)) shear walls was proposed (Palermo et al. 2006; Smith et al. 2007). In these massive timber shear walls, post-tensioning strands or bars were incorporated in the wall panels for providing the self-centering force, therefore, forming the post-tensioned (PT) CLT or LVL shear walls. Sometimes, energy dissipaters (e.g., mild steel dissipater and friction dissipater) were mounted in the PT massive timber shear walls, for increasing their ductility and energy-dissipating abilities. Such a hybrid massive timber system combining the post-tensioning strands or bars and the energy dissipaters has attracted the interests from the fields of both engineering and scientific research. For the PT LVL shear walls, Buchanan et al. (2007) tested the lateral performance of coupled PT LVL shear walls with steel dissipaters; Newcombe et al. (2010) tested the lateral performance of one two-thirds-scale two-storey PT LVL shear wall structure. For both the dissipative PT LVL shear wall and the PT LVL shear wall structure, a good self-centering performance with ideal energy-dissipating abilities under seismic loads was attested. Furthermore, Sarti et al. (2015; 2017) investigated the seismic design factors for the PT LVL shear wall structures based on both experimental and analytical researches. Currently, the concept of this hybrid massive timber system has been extended to the field of CLT shear walls. A series of studies have focused on the lateral performance of PT CLT shear walls and on the seismic performance of PT CLT shear wall structures (PT-CLTstrs). Ganey et al. (2017) tested the lateral performance of the PT CLT shear walls with or without dissipaters; following Ganey’s work, Akbas et al. (2017) developed the shear wall model and defined several typical limit states for the PT CLT shear walls. Wilson et al. (2019) developed one elastic-plastic predictive model for PT CLT shear walls using the 8-node solid elements. Chen et al. (2020) tested
full-scale PT-CLT shear walls with four configurations amounted with the axial energy dissipaters. It was observed that localized crushing of wood occurred in the CLT wall toe, when the drift reached or exceeded 2.5%. As for the structural seismic performance, Pei et al. (2019) and Blomgren et al. (2019) studied the dynamic performance of one full-scale 2-story building that utilized a lateral load-resisting system consisting of two PT-CLT shear walls. It was attested that the structural system could remain stable self-centering performance even in case of a roof drift pushed to 5%; besides, this structural system can satisfy the life-safety requirement in maximum considered earthquakes (MCEs). Sun et al. (2019; 2020a; 2020b; 2020c) tested the lateral performance of 0.5-scale two-storey PT CLT shear walls constructed by the platform method, and then analyzed the seismic response of the PT-CLT strs with or without dissipaters. For the immediate occupancy (IO), life safety (LS), and collapse prevention (CP) hazard levels, the structural drift limitations were recommended as 0.7%, 1.4%, and 2.2%, respectively.

The structural seismic performance can be quantified using probabilistic methods that combine reasonable performance-based criteria; therefore, several researchers have focused on the seismic reliability analysis on different types of timber structures. van de Lindt and Walz (2003) analyzed the seismic reliability of light-frame wood shear walls based on a developed model for dynamic performance. Li et al. (2012) investigated the seismic reliability of post-and-beam timber buildings, considering the uncertainties from earthquakes, structural mass and shear wall characteristics. Zhang et al. (2018) assessed the seismic reliability of timber-steel hybrid system and found that the earthquake ground motion was the most significant factor for the structural seismic reliability. Hong and Yang (2019) estimated the seismic reliability for mid- and high-rise wood buildings constructed with glulam for beams or columns, and with CLT for shear walls or floor diaphragms. Stellacci et al. (2018) assessed the currently adopted rehabilitation techniques for traditional timber frame walls by analyzing their structural reliabilities. However, currently few studies have been conducted to comprehend the seismic reliability of PT-CLTstrs, due to the complexity of this kind of structural system as well as due to lack of a reliable structural model.

In this paper, for considering the effect of total floor number, a set of 4-, 8-, 12-, and 16-storey PT CLT shear wall structures (PT-CLTstrs) were designed using the direct displacement-based design (DDD) method. The model of each PT-CLTstr was developed and calibrated. The seismic reliability of each PT-CLTstr was assessed based on the fragility analysis and based on the response surface method (RSM), respectively. The fragility-based reliability index and the RSM-based reliability index were obtained and then compared, for each PT-CLTstr and for each seismic hazard level (i.e., IO, LS, and CP hazard level). The study can provide a technical basis for the PT-CLTstrs in terms of the seismic performance quantification and the floor number optimization.

2. Structural Model Calibration

Three 0.5-scale two-storey PT CLT shear walls with different initial post-tensioning stress ratios were tested; besides, their numerical models that could accurately predict the lateral performance were also developed, as shown in Figure 1. The floor diaphragm incorporated in

![Figure 1. Post-tensioned CLT Shear Wall: (a) Test setup; (b) Model (Sun et al. 2020a)]
the shear wall specimen was structurally designed for mitigating its creep deformation under the perpendicular-to-grain compression. More details of the shear wall specimens and of the predictive shear wall model are introduced by Sun et al. (2019; 2020a). The CLT for the shear wall specimens was fabricated with No.2-grade Canadian hemlock. He et al. (2018) tested the mechanical properties of this Canadian hemlock CLT, and found that the CLT can provide ideal compressive or bending properties for engineering applications. Then, 4-, 8-, 12-, and 16-storey benchmark PT-CLTstrs (Figure 2) were respectively designed using the DDD method, which was proven to be suitable for designing the PT-CLTstrs with or without the dissipaters (Sun et al. 2019). The storey height of the PT-CLTstrs ($H$) was designed as 2600 mm. For the DDD-based PT-CLTstrs with dissipaters and those without dissipaters, their response of inter-storey drift was similar under identical seismic hazard level (Sun et al. 2019 and 2020c). Since the inter-storey drift was used as the measurement of the structural seismic damages, therefore, for increasing the efficiency of numerical calculation, it was determined that no dissipaters were amounted in the designed benchmark PT-CLTstrs. These benchmark PT-CLTstrs designed with the wall panels of every two floors being post-tensioned into one single rocking segment were assumed located in Sichuan, China; the floor weight and the roof weight was respectively obtained as $2.466 \times 10^5$ kg and $2.041 \times 10^5$ kg based on the gravity analysis. The detailed DDD procedure for the PT-CLTstrs was described by Sun et al. (2020c). A summary of the shear wall Y-direction layout for the 4-storey PT-CLTstr is listed in Table 1 as an example; the shear wall layouts of the 8-, 12-, and 16-storey PT-CLTstrs were provided by Sun et al. (2019). Based on the design results of the shear wall layout for each PT-CLTstr, the simplified two-dimensional structural model was developed using OpenSees, as shown in Figure 3. The fiber-based DispBeamColumn element was used for simulating the post-tensioning

Figure 2. Floor Layout of the Benchmark PT CLT Shear Wall Structures.
strands and for simulating the floor diaphragms that connect the neighboring PT CLT shear walls, respectively. The ShellMIT4 element combined with the J2Plasticity material law was used for simulating the CLT wall panels. The ZeroLength element combined with the ENT material law was used for simulating the contact stiffness of the interface between the wall panel and the foundation. Based on structural modal analysis, for the 4-, 8-, 12-, and 16-storey benchmark PT-CLTstrs, their fundamental period $T_1$ was 1.40 s, 1.97 s, 2.40 s, and 2.99 s, respectively.

For the model of each PT-CLTstr, a set of 20 ground motions ($V_{30}$ was between 280-480 m/s) was selected from the Pacific Earthquake Engineering Research Center’s Next Generation Attenuation (NGA) database. These 20 ground motions used as the excitations of the structural dynamic analysis should match the target seismic response spectrum corresponding to the defined performance level of the DDD procedure (i.e., CP hazard level) with the fundamental period $T_1$. Then, nonlinear dynamic analysis was conducted on the model of each PT-CLTstr, using the corresponding assemble of the 20 ground motions. The maximum inter-storey drift (MaxISDR) of each PT-CLTstr were obtained, forming a cumulative curve of MaxISDR based on the empirical cumulative distribution functions. Since the probability of non-exceedance (PNE) was defined as 95% based on the Chinese code GB 50068 (2018), the inter-storey drift limitation of the PT-CLTstrs for the CP hazard level can be determined in the range of 2.0% - 2.4%, as shown in Figure 4. This drift range of 2.0% - 2.4% is in agreement with the recommended drift limitation of 2.2% for the PT-CLTstrs under the CP hazard level (Sun et al. 2019 and 2020c), indicating that the models of the PT-CLTstrs were calibrated.

### 3. Fragility-based Reliability Analysis

#### 3.1. General Introduction

In the fragility-based reliability analysis, fragility analysis was used for estimating the structural non-performance probability conditional on a given seismic hazard level.
(e.g., IO, LS, CP). This conditional non-performance probability was then multiplied by the annual probability of exceeding a given seismic intensity measure (IM), producing a structural failure probability. Therefore, for one PT-CLTstr, its structural failure probability can be calculated using equation 1, in which, \( f_R(z) \) is the probability density function of the seismic fragility; \( H_{IM}(z) \) is the hazard function for earthquakes with a given IM. The discrete form of equation 1 can be expressed as equation 2, in which, the \( F_{RI}(IM_i) \) is the conditional non-performance probability (i.e., structural fragility) for the \( i \)-level IM; \( H_{IM}(IM) \) is the annual probability of exceeding the given \( i \)-level IM.

\[
P_f = \int f_R(z) \cdot H_{IM}(z) \cdot dz
\]

\[
P_f = \sum \left( F_{RI}(IM_i) - F_{RI}(IM_{i-1}) \right) \cdot H_{IM}(IM_i)
\]

3.2. Fragility Analysis

The fragility analysis can be conducted using probability seismic demanding analysis (PSDA) or incremental dynamic analysis (IDA). Since it was proven that both PSDA and IDA can generate similar fragility curves but the PSDA required less computational efforts (Zhang and Huo 2009), in this paper, the fragility analysis for estimating the conditional non-performance probability was conducted using the PSA method; besides, the PGAs (peak ground accelerations) of the ground motions were recommended as the IM (Padgett et al. 2008). A total of 10 earthquake ground motions \( (V_{so} \text{ was between 280-480 m/s}) \) were selected as the input excitations for PSDA (Table 2), including 6 shallow crustal ground motions from NGA database, 3 subduction ground motions from the Japanese database of the K-NET and KiK-net, and 1 Wenchuan (a city of Sichuan) ground motion. These selected ground motions with PGAs evenly distributed in the range of 0.1 g - 1.5 g can reflect the seismic environments of Sichuan, producing unbiased seismic performance evaluation on the PT-CLTstrs.

In this paper, the seismic reliability of each PT-CLTstr was calculated for three earthquake hazard levels, including the IO hazard level with an average return period of 50 years, the LS hazard level with an average return period of 475 years, and the CP hazard level with an average return period of 2475 years. The MaxISDR of the PT-CLTstrs was adopted as the engineering demanding parameter (EDP) for the PSDA; besides, the damage index of the MaxISDR (\( \theta_{dis} \)) was adopted as 0.7%, 1.4%, and 2.2% for the IO, LS, and CP hazard levels, respectively (Sun et al. 2019). The fragility curves of the PT-CLTstrs providing their conditional non-performance probabilities are shown in Figure 5.

3.3. Seismic Hazard and Failure Probability

The annual probability of exceeding a given PGA \([i.e., H_{IM}(PGA)]\) can be calculated using equation (3) (Cornell et al. 2002). For the IO with a 50-year return period, LS with a 475-year return period, and CP with a 2475-year return period, their mean PGA is recommended as 0.20 g, 0.55 g, and 0.82 g, respectively (Shu et al. 2019). The seismic hazard curve for annual exceedance probabilities was obtained (Figure 6) by fitting the mean PGA of each hazard level with its corresponding return period. The

### Table 2. Ground Motions Selected for Reliability Analysis

<table>
<thead>
<tr>
<th>No.</th>
<th>Event</th>
<th>Date</th>
<th>Station</th>
<th>Component</th>
<th>PGA (g)</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Chi-Chi, Taiwan</td>
<td>1999.9.20</td>
<td>TCU036</td>
<td>TCU036-N</td>
<td>0.124</td>
<td>Shallow</td>
</tr>
<tr>
<td>2</td>
<td>Irpinia, Italy-01</td>
<td>1980.11.23</td>
<td>Sturno (STN)</td>
<td>STU000</td>
<td>0.227</td>
<td>Shallow</td>
</tr>
<tr>
<td>3</td>
<td>Northridge-01</td>
<td>1994.1.17</td>
<td>Pardee-SCE</td>
<td>PAR-T</td>
<td>0.302</td>
<td>Shallow</td>
</tr>
<tr>
<td>4</td>
<td>Superstition Hills-02</td>
<td>1987.11.24</td>
<td>Parachute Test Site</td>
<td>PTS315</td>
<td>0.384</td>
<td>Shallow</td>
</tr>
<tr>
<td>5</td>
<td>Cape Mendocino</td>
<td>1992.4.15</td>
<td>Centerville Beach</td>
<td>CBF360</td>
<td>0.478</td>
<td>Shallow</td>
</tr>
<tr>
<td>6</td>
<td>Cape Mendocino</td>
<td>1992.4.15</td>
<td>Petrosia</td>
<td>PET090</td>
<td>0.662</td>
<td>Shallow</td>
</tr>
<tr>
<td>7</td>
<td>TOHKAMACHI</td>
<td>2004.10.23</td>
<td>NIG021</td>
<td>NS</td>
<td>0.832</td>
<td>Subduction</td>
</tr>
<tr>
<td>8</td>
<td>Kobe, Japan</td>
<td>1995.1.26</td>
<td>KJMA</td>
<td>KJM000</td>
<td>0.832</td>
<td>Subduction</td>
</tr>
<tr>
<td>9</td>
<td>Wenchuan</td>
<td>2008.5.12</td>
<td>Wolong</td>
<td>EW</td>
<td>0.977</td>
<td>Subduction</td>
</tr>
<tr>
<td>10</td>
<td>SHIOGAMA</td>
<td>2011.4.7</td>
<td>MYG012</td>
<td>EW</td>
<td>1.457</td>
<td>Subduction</td>
</tr>
</tbody>
</table>
regression analysis yielded a decay factor $k_d$ of 2.131 and a scale factor of 0.000657. Then, the specific $H_{IM}(PGA)$ of the PT-CLTstrs can be calculated using the equation 3.

$$H_{IM}(PGA) = k_d \cdot (PGA)^{-k_s}$$

Based on the equation 2, product of both the $H_{IM}(PGA)$ and the structural fragility was integrated over a PGA range of 0.1 $g$ - 1.5 $g$, producing a failure probability $P_f$ and a corresponding reliability index $\beta$ for each PT-CLTstr or for each seismic hazard level. Results of the fragility-based reliability analysis are listed in Table 3. For the IO hazard level, the $\beta$ of the four PT-CLTstrs is in the range of 1.484 - 1.942, with the largest $\beta$ for the 4-storey PT-CLT. For the LS hazard level, the $\beta$ of the four PT-CLTstrs is in the range of 2.200 - 2.614, with the largest $\beta$ for the 8-storey PT-CLT. For the CP hazard level, the $\beta$


<table>
<thead>
<tr>
<th>Storey</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>$2.606 \times 10^{-2}$</td>
<td>1.942</td>
<td>$4.793 \times 10^{-3}$</td>
</tr>
<tr>
<td>8</td>
<td>$3.732 \times 10^{-2}$</td>
<td>1.783</td>
<td>$4.476 \times 10^{-3}$</td>
</tr>
<tr>
<td>12</td>
<td>$6.888 \times 10^{-2}$</td>
<td>1.484</td>
<td>$8.976 \times 10^{-3}$</td>
</tr>
<tr>
<td>16</td>
<td>$6.431 \times 10^{-2}$</td>
<td>1.520</td>
<td>$13.889 \times 10^{-3}$</td>
</tr>
</tbody>
</table>

**Figure 5.** Fragility Curves of the PT-CLTstrs.

**Figure 6.** Seismic Hazard Curve for Annual Exceedance Probability.

**Table 3.** Fragility-based Reliability Analysis Results
of the four PT-CLTstrs is in the range of 2.908 - 3.162, with the largest $\beta$ for the 8-storey PT-CLT.

4. Reliability Based on Response Surface Method

4.1. General Introduction

Compared to the fragility-based reliability analysis method, the response surface method (RSM) can consider more random variables, which is always coupled with common reliability analysis methods [e.g., first-order reliability method (FORM)]. In the RSM for the PT-CLTstrs, the performance function that serves as an effective tool for reliability estimation or structural optimization can be expressed as equation 4, in which, $\delta$ is the inter-storey drift capacity of the PT-CLTstrs ($\delta = H \theta _{DI}$); $\Delta$ is the MaxISDR demand, which a function of the seismic intensity measure $IM$, the characteristics of the ground motions $r$, the structural total floor number $n$, the response surface fitting error $\varepsilon$, and the design factors of interest $F_d$. The characteristics of the ground motions $r$ can be considered by selecting a set of representative ground motions reflecting the site record-to-record variability. The PGAs of the ground motions were used as the $IM$, which follow a lognormal distribution with a mean PGA of 0.25 g and with a COV (coefficient of variation) of 0.6 (Li et al. 2012); it coupled with an assumed annual Poisson arrival rate of 0.1/year. In this paper, the uncertainties including the $IM$ (i.e., PGA), the total floor number $n$, and the fitting error $\varepsilon$ were considered in the performance function of the PT-CLTstrs (equation 5).

$$G = \delta - \Delta(IM, r, n, \varepsilon, F_d)$$  \hspace{1cm} (4)

$$G = \delta - \Delta(PGA, n, \varepsilon)$$  \hspace{1cm} (5)

15 PGA levels (i.e., from 0.1 g to 1.5 g with intervals of 0.1 g) and 4 total floor number levels (i.e., 4-, 8-, 12-, and 16-storey) were used for generating the database of the MaxISDR. In this paper, the 10 ground motions listed in Table 2 were scaled with respect to each PGA level, generating 15 groups of ground motions with incremental PGA levels. For each PT-CLTstr, over a group of 10 ground motions scaled to one PGA level, the structural MaxISDR was calculated based on the time-history dynamic analysis. For each PT-CLTstr and for each PGA level, both the mean value ($\mu_{sm}$) and the standard deviation ($\sigma_{sm}$) of the MaxISDR were calculated based on the structural model simulations; then, for all the combinations of the PGA levels and the floor number levels, a discrete set of the $\mu_{sm}$ or the $\sigma_{sm}$ was generated (Table 4).

### Table 4. Statistical data of the structural simulation results ($\mu_{sm}$ and $\sigma_{sm}$)

<table>
<thead>
<tr>
<th>PGA (g)</th>
<th>4</th>
<th>8</th>
<th>12</th>
<th>16</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\mu_{sm}$ (mm)</td>
<td>$\sigma_{sm}$ (mm)</td>
<td>$\mu_{sm}$ (mm)</td>
<td>$\sigma_{sm}$ (mm)</td>
</tr>
<tr>
<td>0.1</td>
<td>4.248</td>
<td>0.961</td>
<td>7.860</td>
<td>3.292</td>
</tr>
<tr>
<td>0.2</td>
<td>14.647</td>
<td>11.959</td>
<td>15.404</td>
<td>6.233</td>
</tr>
<tr>
<td>0.4</td>
<td>35.092</td>
<td>24.338</td>
<td>30.149</td>
<td>11.797</td>
</tr>
<tr>
<td>0.5</td>
<td>28.281</td>
<td>15.244</td>
<td>36.548</td>
<td>14.444</td>
</tr>
<tr>
<td>0.6</td>
<td>57.101</td>
<td>45.413</td>
<td>43.605</td>
<td>16.273</td>
</tr>
<tr>
<td>0.7</td>
<td>44.711</td>
<td>24.906</td>
<td>50.806</td>
<td>19.171</td>
</tr>
<tr>
<td>0.8</td>
<td>48.845</td>
<td>21.817</td>
<td>57.790</td>
<td>20.744</td>
</tr>
<tr>
<td>0.9</td>
<td>50.675</td>
<td>19.396</td>
<td>64.892</td>
<td>23.914</td>
</tr>
<tr>
<td>1.0</td>
<td>70.001</td>
<td>31.453</td>
<td>71.206</td>
<td>26.615</td>
</tr>
<tr>
<td>1.1</td>
<td>59.505</td>
<td>26.451</td>
<td>77.985</td>
<td>29.281</td>
</tr>
<tr>
<td>1.2</td>
<td>67.848</td>
<td>26.463</td>
<td>85.799</td>
<td>32.202</td>
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<tr>
<td>1.3</td>
<td>59.569</td>
<td>18.897</td>
<td>91.926</td>
<td>34.381</td>
</tr>
<tr>
<td>1.4</td>
<td>65.762</td>
<td>23.871</td>
<td>97.742</td>
<td>36.665</td>
</tr>
<tr>
<td>1.5</td>
<td>76.541</td>
<td>20.215</td>
<td>105.824</td>
<td>34.702</td>
</tr>
</tbody>
</table>
error between the polynomial fitting results and the model simulation results; superscripts $i$, $j$, and $k$ are the orders of the polynomials. In this paper, third-order polynomials with nine coefficients were adopted (i.e., $i$ and $j = 1, 2, 3$) in the polynomial functions (equations 8-9).

\[ \mu_{rs} = \sum a_{ij} \cdot PGA^j \cdot n^i \]  
\[ \sigma_{rs} = \sum a_{ij} \cdot PGA^j \cdot n^i \]  
\[ \mu_{rs} = a_{11} \cdot PGA \cdot n + a_{12} \cdot PGA^2 \cdot n^2 + a_{21} \cdot PGA^3 \cdot n^3 + a_{22} \]  
\[ \times PGA^2 \cdot n^2 + a_{13} \cdot PGA \cdot n^3 + a_{31} \cdot PGA^3 \cdot n + a_{23} \]  
\[ \times PGA \cdot n + a_{32} \cdot PGA^3 \cdot n^2 + a_{33} \cdot PGA^3 \cdot n^3 \]  
\[ \sigma_{rs} = b_{11} \cdot PGA \cdot n + b_{12} \cdot PGA^2 \cdot n^2 + b_{21} \cdot PGA^3 \cdot n + b_{22} \]  
\[ \times PGA^2 \cdot n^2 + b_{13} \cdot PGA \cdot n^3 + b_{31} \cdot PGA^3 \cdot n + b_{23} \]  
\[ \times PGA \cdot n + b_{32} \cdot PGA^3 \cdot n^2 + b_{33} \cdot PGA^3 \cdot n^3 \]  

When taking the fitting error $\varepsilon$ into account, the fitting errors of the generic $i$-th combination of the random variables were calculated using equations 10-11. The mean and standard deviation of the overall fitting errors (i.e., $\varepsilon_\mu$ and $\varepsilon_\sigma$) can be obtained when all the combinations are considered, both of which are assumed to follow a normal distribution. Assuming that the seismic response of MaxISDR follows a lognormal distribution, the performance function (i.e., equation 5) can be rewritten as equation 12, in which, $\mu$ is the mean value of MaxISDR demand calculated using equation 13; $V$ is the COV (i.e., $\sigma/\mu$), $\sigma$ is the standard deviation of MaxISDR demand calculated using equation 14. $R_n$ is the standard normal distribution $R_n(0, 1)$.

\[ \varepsilon_\mu = \frac{\mu_i - \mu_{sm}}{\mu_{rs}} \]  
\[ \varepsilon_\sigma = \frac{\sigma_i - \sigma_{sm}}{\sigma_{rs}} \]  
\[ G = \delta - \frac{\mu}{\sqrt{1 + V^2}} \exp \left( R_n \ln \left( \frac{1 + V^2}{1} \right) \right) \]  
\[ \mu = \mu_{rs} \cdot (1 - \varepsilon_\mu) \]  
\[ \sigma = \sigma_{rs} \cdot (1 - \varepsilon_\sigma) \]  

The polynomial response surfaces for both the mean and standard deviations are shown in Figure 7. The polynomial response surface fitting results versus simulation results are shown in Figure 8.

**Figure 7.** Polynomial Response Surfaces: (a) Mean Values; (b) Standard Deviations.

**Figure 8.** Polynomial Response Surface Fitting Results Versus Simulation Results.
values and the standard deviations of the MaxISDR are shown in Figure 7; the nine fitted polynomial coefficients are also listed for each polynomial response surface. The polynomial fitting errors for the sets of both the $\mu_{sm}$ and the $\sigma_{sm}$ are shown in Figure 8. Most of the data points are located near the perfect agreement line, indicating that good fitting results with small fitting errors can be achieved by using the third-order polynomial response surfaces.

Then, the structural failure probability $P_f$ and the reliability index $\beta$ can be estimated based on the FORM using the software of $Rt$ (Mahsuli and Haukaas 2013). The RSM-based reliability analysis results with respect to both the 15 PGA levels and the 4 total floor number levels are listed in Table 5. For the IO hazard level, the $\beta$ of the four PT-CLTstrs is in the range of 1.098 - 1.392. For the LS hazard level, the $\beta$ of the four PT-CLTstrs is in the range of 1.979 - 2.346. For the CP hazard level, the $\beta$ of the four PT-CLTstrs is in the range of 2.577 - 2.802. The largest $\beta$ is for the 4-storey PT-CLTstr under all seismic hazard levels.

5. Comparison and Discussion

The seismic reliability of the PT-CLTstrs under different seismic hazard levels was evaluated, based on the fragility analysis combined with the seismic hazard analysis as well as based on the RSM combined with the FORM, respectively. The reliability indices of the PT-CLTstrs are shown in Figure 9. The RSM-based reliabilities of the PT-CLTstrs that consider more intervening random variables (i.e., the response surface fitting error, the total storey number, and the earthquake ground motions) are slightly less than the fragility-based reliabilities. Overall, the reliability method based on the fragility analysis and that based on the response surface can provide similar reliability indices; for these studied 4-, 8-, 12-, and 16-storey PT-CLTstrs, following their fragility-based reliabilities, the 8-storey PT-CLTstr with the largest $\beta$ under the LS and CP hazard levels is subjected to the least seismic vulnerability; while, following their RSM-based reliabilities, the 4-storey PT-CLTstr with the largest $\beta$ under all hazard levels is subjected to the least seismic vulnerability.

When a random variable (e.g., the ground motions) is the dominant reliability-influencing one, then the fragility-based reliability analysis method seems to be more straightforward for obtaining the structural seismic reliabilities. Because compared to the fully coupled reliability analysis method (i.e., RSM), the fragility-based reliability analysis method separates the structural response analysis from the seismic hazard analysis. Overall, both methods can be used as efficient approaches for assessing the seismic reliabilities of the PT-CLTstrs.

6. Conclusions

In this paper, seismic reliability assessment was conducted on a set of mid- and high-rise PT-CLTstrs designed with the direct displacement-based seismic design method, based on the fragility analysis combined with the seismic hazard analysis as well as based on the RSM combined with the FORM, respectively. The main conclusions can be drawn as follows:

1. Based on the fragility-based reliability analysis method, the reliability indices of the PT-CLTstrs are in the range of 1.484 - 1.942, 2.200 - 2.614, and 2.908 - 3.162, for the IO, LS and CP hazard levels, respectively.

2. Based on the RSM-based reliability analysis method, the reliability indices of the PT-CLTstrs are in the range of 1.098 - 1.392, 1.979 - 2.346, and 2.577 - 2.802, for the IO, LS and CP hazard levels, respectively.

3. The RSM-based reliabilities considering more random variables are slightly less than the fragility-based reliabilities. Overall, both the RSM and the fragility-based reliability method can be used as efficient approaches for assessing the seismic reliabilities of the PT-CLTstrs.

4. For these studied 4-, 8-, 12-, and 16-storey PT-CLTstrs, following their fragility-based reliabilities, overall, the 8-
storey PT-CLTstr is subjected to the least seismic vulnerability; while, following their RSM-based reliabilities, the 4-storey PT-CLTstr is subjected to the least seismic vulnerability.

Acknowledgements

The authors also gratefully acknowledge the support from National Natural Science Foundation of China (Grant NO. 51778460) and China Scholarship Council (Grant NO. 201706260124). Ground motion data for Japanese earthquakes and worldwide crustal earthquakes were obtained from the K-NET/KiK-net database at http://www.kyoshin.bosai.go.jp/, and the PEER-NGA database at http://peer.berkeley.edu/nga/index.html, respectively.

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