RELIABILITY ASSESSMENT OF TIMBER SHEAR WALLS UNDER EARTHQUAKE LOADS

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SUMMARY

A modified version of the BRANZ procedure for lateral capacity rating of bracing walls was used to determine the sustainable lateral mass of a 910-mm wide '2x4' timber shear wall. The key modifications involve: (1) the use of a multi-criteria system identification method to determine a structural model that fits test data from both cyclic testing and pseudo-dynamic testing; and (2) probabilistic treatment of ground motions (i.e., using suites of site-specific earthquake records with 2%, 10% and 50% exceedance probability in 50 years as input loads in Monte Carlo simulation). Then the reliability index for the wall system that was rated according to the modified BRANZ procedure was estimated when subjected to a range of earthquake intensities in Tokyo. For this particular wall, we obtained reliability indices (at the safety limit state) ranging from 0.94 to 5.20, depending on the displacement capacity determined from the static cyclic test, and the suite of earthquakes from which the sustainable mass was calculated from. Thus, it is desirable to quantify and include the inherent uncertainty in displacement capacity and ground motions in the analysis. The method presented herein is general and can be applied to allow the direct use of laboratory data, from cyclic or pseudo-dynamic testing, for dynamic and seismic reliability analyses of lateral resisting systems with no distinct yield point.

INTRODUCTION

Current seismic design procedures for light-frame timber buildings have evolved through experience, field observations and a limited number of simple experiments. Design target performance and reliability are unknown or un-quantified. The 1994 Northridge earthquake and the 1995 Hyogo-ken Nanbu (Kobe) earthquake have not only caused extensive damage to timber buildings but they have also badly shaken the generally high level of confidence that most people have in the seismic performance of timber buildings. These earthquakes have inflicted severe emotional, social and economic difficulties on affected people and communities.

Until recently, most programs in earthquake disaster mitigation do not address issues related to residential buildings. Whether the buildings are engineered or not, their reliability under earthquake loads is not known. We have learned from recent earthquakes that their overall impact in heavily populated areas cannot be effectively contained without serious efforts to improve the seismic performance of light-frame timber buildings. Many positive developments are expected in this area in the next several years with the recent federal and state funding of the CUREe-Caltech Woodframe Project in the US. The proceedings of this project's first technical workshop provides a snapshot of current developments and identifies areas of needed research [Seible et al., 1999].

One of the important topics that needs to be addressed is determining the reliability of timber buildings under earthquake loads. A simple method of estimating failure probability based on results of nonlinear random vibration analyses of hysteretic timber systems has been developed [Foliente et al., 1996a; 1996b]. Ceccotti and Foschi [1998] used a response surface approach together with DRAIN-2DX, with a special hysteretic element, to determine the force reduction factor for buildings with timber shear walls that best satisfies the target

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performance specified in the National Building Code of Canada. Suzuki and Araki [1998] have presented a combined analytical-numerical method of reliability analysis of hysteretic systems with uncertain properties under deterministic and random excitations. They demonstrated the method for timber buildings with uncertain properties under specified seismic excitations (i.e., deterministic). Foliente et al. [1999] applied the reliability analysis method used in code calibration in Japan [Saito et al., 1998] to estimate the reliability of Japanese post-and-beam wall construction, modelled by an inelastic degrading and pinching single-degree-of-freedom (SDOF) system, subjected to ground motions in Tokyo and Osaka. These site-specific accelerograms were generated using a non-stationary stochastic process model and historical earthquake data.

In this paper, we extend the application of the method by Saito et al. [1998] and Foliente et al. [1999] in determining the dependable seismic mass restrained by a ‘2x4’ timber shear wall using a modification of the procedure developed by the Building Research Association of New Zealand (BRANZ) [King and Deam, 1999]. The reliability index for a wall system rated according to the modified BRANZ procedure, when subjected to a range of earthquake intensities in Tokyo, was then calculated. The method is general and can be applied to allow the direct use of laboratory data, from cyclic or PSD testing, for dynamic and seismic reliability analyses of lateral-resisting systems with no distinct yield point.

STRUCTURAL MODEL

Experimental Basis

Most light-frame timber buildings are of platform construction, also known as ‘2x4’ construction. Originally developed in the US, it has been adopted in many parts of the world. This form of construction has also been slowly gaining market share in Japan since its introduction there in the 1970’s.

The Building Research Institute (BRI) has tested many different types and configurations of walls under static monotonic, static cyclic, PSD and shaketable loading. Figure 1a shows a schematic diagram of one such wall. In this paper, we have chosen a 910 x 2450 mm shear wall, which is typically found in Japanese houses and is commonly regarded as the minimum size shear wall in Japan. This wall was sheathed with a 9.5-mm thick JAS No.2 (conifer) plywood on one side. The framing members were 38 mm x 89 mm Spuce-Pine-Fir (S-P-F), JAS Standard grade. JIS CN50 nails, 50 mm in length and 2.87 mm in diameter, fastened the sheathing panel to the frame at a spacing of 100 mm along the four edges and 150 mm along the centre stud. At the end of the wall, the double studs were connected to the steel base with a hold down. Lateral load was applied to the top of the wall following the static cylic protocol shown in Fig. 1b. Similar walls were subjected to PSD and shaketable testing with the North-South component of the 1995 Kobe earthquake recorded at the Kobe Marine Meteorological Observatory, scaled to 60%, as input load. Details of the test program are given in Kawai [1998].

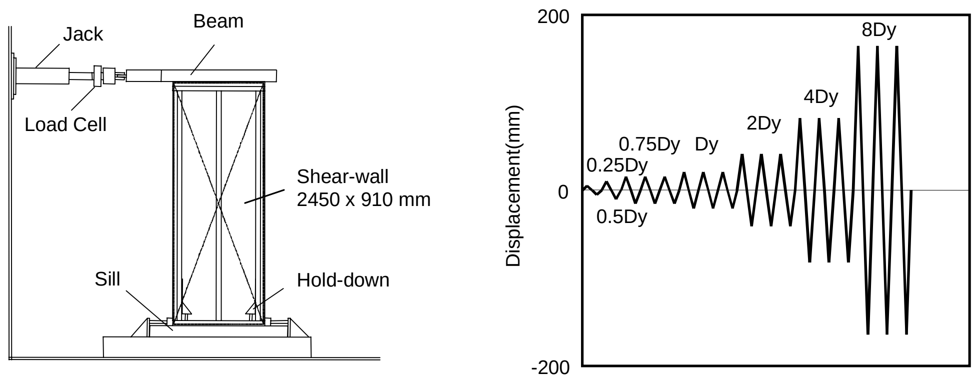


Figure 1: Static-cyclic test setup and loading sequence of BRI shear wall (from Kawai [1998])

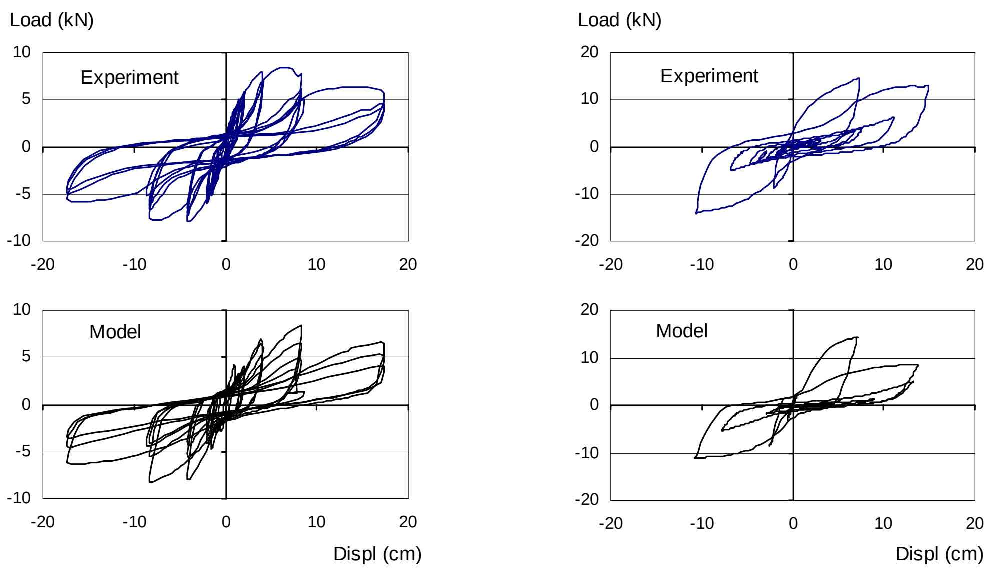
System Identification

The wall system was modeled by a SDOF shear model with the modified Bouc-Wen-Baber-Noori (BWBN) differential hysteresis model. This model is capable of producing various hysteresis shapes, and exhibits strength and stiffness degradation and pinching (including slip) [Foliente, 1995]. These hysteretic features, when observed in test results, cannot be ignored in analysis for convenience's sake because doing so may lead to non-conservative estimates of peak responses and reliability estimates [Paevere and Foliente, 1999].

The parameters of the modified BWBN hysteresis model can be estimated using [Foliente et al., 1998; Zhang et al., 1999]: (1) a Genetic Algorithm method; (2) the Generalised Reduced Gradient (GRG) method; (3) the constrained simplex method; or (4) the constrained Extended Kalman Filter (EKF). In this paper, a multi-criteria GRG method was used to estimate model parameters based on the BRI wall response data from both cyclic and PSD tests. The GRG algorithm is an extension of the Wolfe algorithm to accommodate both a nonlinear objective function and nonlinear constraints. In essence, the method employs linear, or linearized constraints, defines new variables that are normal to some of the constraints, and transforms the gradient to this new basis. In this study, the objective function was to minimise the cumulative error between: (1) the predicted and measured restoring force from the cyclic test, and (2) the predicted and measured displacements and restoring forces from the PSD test.

Figure 2 shows a comparison of the hysteretic response of the wall from static cyclic and PSD loading with the identified SDOF model. Figure 3 compares the displacement time histories from the PSD test and from the model during the first 12 seconds of the scaled North-South component of the 1995 Kobe earthquake. The identified model has natural frequency ω_o=2.6 Hz, damping ξ_o=5%, and the following hysteresis parameters: A=1.0, α=0.01, β=0.45, γ=–0.2, n=1.0, q=0.07, ζ_s=0.9, λ=25, p=6.5, ψ_o=0.008, δ_ψ=0.0001, δ_ν=0.05 and δ_η=0.1.

Parallel identification of the model for different types of loading pattern is needed because the estimation of hysteresis model parameters is load protocol (or load-path) dependent. When a set of model parameters is obtained based on only one type of protocol (or loading pattern), the estimated parameters will not necessarily capture other possible behaviour of the same specimen when subjected to a very different loading pattern (e.g., static cyclic pattern vs. arbitrary seismic displacement pattern). A hysteresis model which is fitted only to a static-cyclic experiment may under-estimate the rate of system degradation under a real earthquake, whereas a model fitted only to a shaketable or PSD test may over-estimate system degradation and hence over-estimate the response. The confidence of any analysis prediction is greatly increased by having a model which fits to multiple experiments under multiple excitations on the same system. Overall, the match between the cyclic and the PSD test data and the identified model using the multi-criteria system identification method we employed, as shown in Figures 2 and 3, is very good and hence the confidence in our analysis predictions is high.



(a) **(b)**

Figure 2: Experimental and fitted hysteresis loops for (a) static-cyclic test and (b) pseudo-dynamic test

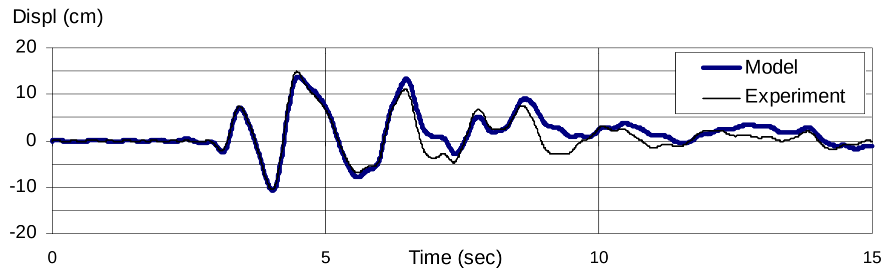


Figure 3: Experimental (from PSD test) and fitted displacement response of timber shear wall under a 0.6*g* Kobe earthquake record

EARTHQUAKE GROUND MOTIONS

The input earthquake ground motions were generated based on the design acceleration response spectrum, $S_A(T)$ (where T is the natural period of a building), given in the Architectural Institute of Japan (AIJ) Guideline *Recommendations for Loads on Buildings* [1993]. We assumed that the maximum earthquake acceleration and the ground amplification factor are random variables, and the spectral coordinates of $S_A(T)$ follow the log-normal probability distributions. The estimated values of mean and coefficient of variation (COV) of the 50-year maximum ground acceleration for Tokyo were 0.23*g* and 60%, respectively. The COV of the ground amplification factor was assumed to be 30% [Ahmed et al. 1996]. The mean values of the maximum response spectrum, $S_A(T)$, in 50 years are plotted in Fig. 4.

The non-stationary stochastic process model used in generating the ground motions was based on a stationary stochastic process having a specified power spectrum – which was determined to be compatible with the sample value of design response spectrum $S_A(T)$ – modified with a Jennings-type envelope function [Saito et al. 1998]. Two different envelopes were adopted: one was an envelope with a short duration time corresponding to Richter Magnitude, $M = 5$ and the other was an envelope with a long duration time corresponding to $M = 8$. Considering the frequency of earthquake occurrences in Tokyo, reliability estimates using the two envelopes were averaged; this is a simple way of taking into account ground motion duration effects. Figure 5 shows samples of generated ground accelerations for Tokyo.

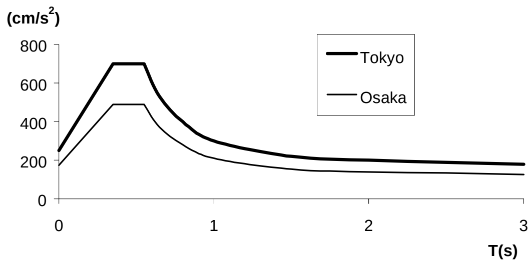


Figure 4: The mean values of maximum response spectrum, SA(T), in 50 years (from Saito et al. [1998])

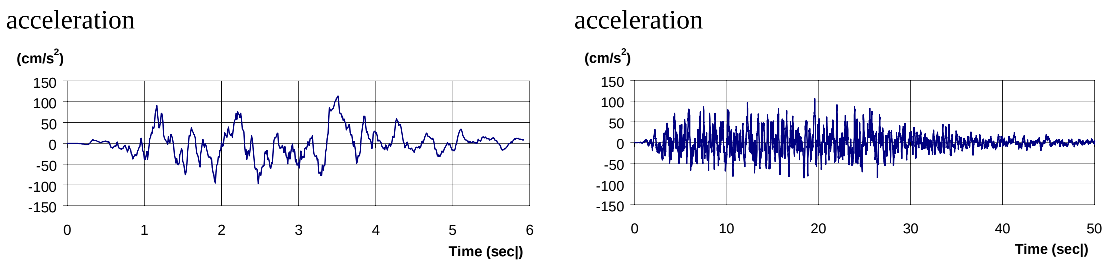


Figure 5: Sample of generated short duration (left) and long duration (right) acceleration records for Tokyo

ESTIMATION OF SUSTAINABLE LATERAL MASS

The BRANZ evaluation procedure to assess the lateral resistance provided by wall systems that are commonly used in low-rise residential buildings in New Zealand [King and Deam, 1999] has many advantages over current methods of using isolated wall test results to obtain information or data required in conventional force-based seismic design. Among other applications, the BRANZ procedure can be used to obtain the sustainable lateral mass of other lateral resisting systems with no distinct yield point, for which ductility assumptions may be meaningless. To estimate the sustainable lateral mass for the 910-mm wide BRI shear wall, we used a modified version of the BRANZ procedure, which is summarised as follows:

- The hysteresis data from both the static-cyclic and pseudo-dynamic tests were fitted simultaneously with the BWBN hysteresis model using a multi-criteria, parallel system identification technique.

- Displacement spectra for various levels of earthquakes were generated. Figure 6 shows the displacement spectra for the BRI wall considering earthquake records with 2%, 10% and 50% probability of exceedance in 50 years in Tokyo; these exceedance levels correspond to average peak ground acceleration of 0.75*g*, 0.56*g* and 0.26*g*, respectively. Each point on the spectra represents the median of the peak displacements obtained from an ensemble of ten site-specific accelerograms, generated using the procedure described earlier.

- The maximum dependable displacement capacity of the shear wall was estimated from the static-cyclic test results. Then, the corresponding period of vibration, *T*, for this displacement was obtained from the appropriate displacement spectra as shown in Figure 6.

- The mass, *m*, effectively restrained at this value of *T* is then calculated based on the frequency term in the equation of motion for a linear oscillator, i.e. $m = kT^2/4\pi^2$, where *k* is the elastic stiffness of the wall. Table 1 presents the sustainable mass for the wall at three displacement limit levels.

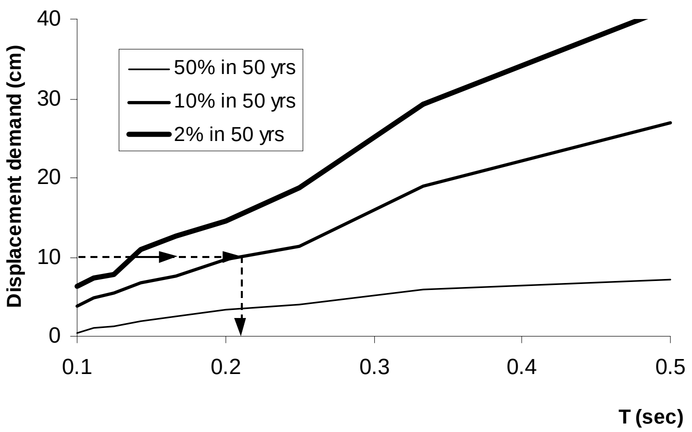


Figure 6: Displacement spectra for shear wall under three levels of excitation

Table 1: Sustainable seismic mass for the shear wall at different displacement levels

Displ. Capacity (cm)	Wall Subjected to Earthquakes with 10% Prob. of Exceedance		Wall Subjected to Earthquakes with 2% Prob. of Exceedance	
	T (sec)	*M* (Tonne)	*T* (sec)	*M* (Tonne)
10.00	0.21	0.66	0.14	0.27
7.50	0.17	0.40	0.11	0.18
6.25	0.14	0.28	0.10	0.15

RELIABILITY ANALYSIS

Reliability Index Calculation Procedure

In this study, system or material variability has been assumed to be not as large a contributor to the overall response variability as the ground motion variability is [Toki, 1998], and is ignored. In the next stage of the study, we will investigate implicit and explicit methods of incorporating system or material variability.

The procedure used to estimate the reliability index β for the 910-mm wall, which has been rated using the modified BRANZ procedure, is described below:

- The nonlinear structural dynamic response of the walls in Table 1 was calculated by Monte Carlo simulation using the generated ground motion records for Tokyo scaled by X =0.97, 2.10, 2.42, 2.78 and 3.0, which correspond to 50-year exceedance probabilities of 50%, 10%, 5%, 2% and 1%, respectively.

- The relations between the input scale, X, and the maximum displacement, Y, were determined by linear regression analysis. After transformation of the original variables, the coefficients a and b of the linear function $V = aU + b$ [where $U = \ln(X)$ and $V = \ln(Y)$] were determined. The linear fit is excellent, with coefficient of determination $R^2 \geq 0.96$. Since the input scale X is standardized using the mean value of maximum response spectral coordinates $S_A(T)$, the variable X follows a lognormal probabilistic distribution function (PDF). Because of the linear relationship between ln (X) and ln (Y), the maximum response Y also follows a lognormal PDF.

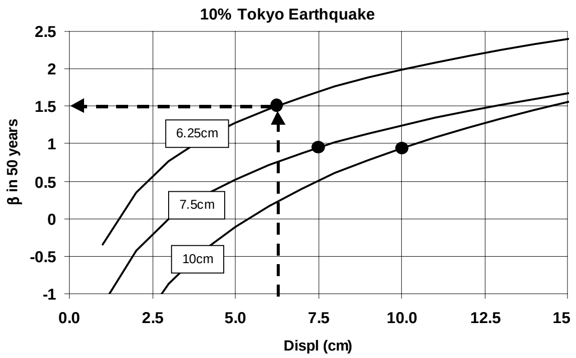
- The reliability index β was defined from the safety probability P_s by means of the standardized normal distribution function Φ as follows:

  $$\beta = \Phi^{-1}(P_s) \tag{1}$$

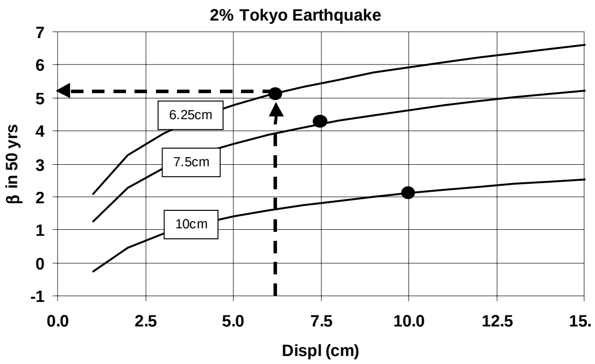
  where P_s is the probability of the maximum response Y not exceeding the design criterion y_c. Details of the calculation of β are given in Saito et al. [1998] and Foliente et al. [1999].

Results and Discussion

Figure 7 shows the reliability curves for the 910-mm wide shear wall, for different values of dependable displacement capacity. Figure 7a has been derived from the displacement spectrum under the 2% probability level earthquakes (i.e., 0.75g) for Tokyo (Fig. 6). Figure 7b has been derived from the displacement spectrum under the 10% probability level earthquakes (i.e. 0.65g).



(a)



(b)

Figure 7: Reliability curves for the shear wall with sustainable mass obtained from Tokyo earthquake records with: (a) 10% probability of exceedance, and (b) 2% probability of exceedance in 50 years

Figure 7 can be used to determine the implied reliability of the shear walls rated using the modified BRANZ procedure for various values of displacement capacity and earthquake level. The reliability index is obtained by determining the intersection of the reliability curve for a given displacement capacity and a vertical line at the same displacement value in the abscissa. Then, from this intersection point in the curve, we can obtain the 50-year reliability index for the system by projecting a horizontal line from this point to the ordinate axis. For example in Fig. 7a, the system with a dependable displacement capacity of 6.25 cm points to a 50-year reliability index of about 1.5. If we do this for all the curves in Fig. 7, we obtain the 50-year reliability index for three levels of displacement capacity under two levels of design earthquake. These calculations are summarised in Table 2. The reliability index values differ markedly for the different displacement capacities and excitation levels (β = 0.94 to 5.2). The range of design earthquakes (0.65 to 0.75*g*) and displacement capacities (6.25 to 10cm) considered are all reasonable for the wall type and site under consideration. This implies that in order to determine a more accurate system reliability, both the design earthquake level and the dependable displacement capacity must be chosen carefully. For this reason, it would be desirable to include also the inherent uncertainty in displacement capacity and building period in the analysis; and this will be the focus of a future study. Furthermore, since the wall is just one part of the whole building, in design, the compatibility of the stiffness and displacement capacity of the wall with other parts of the house should be ensured.

We should note that the above method of obtaining reliability index has been used to obtain the implied reliability of timber, reinforced concrete and steel buildings against a specified, and fixed, drift or displacement limit state, e.g., say a code-specified limit state of 1/30 radian [Foliente et al., 1999; Saito et al., 1998].

Table 2: Reliability indices associated with shear wall at three displacement levels

Displ. Capacity (cm)	50-year Reliability Index, β	
	Wall with Mass Obtained Based on Earthquakes with 10% Prob. Exceed.	Wall with Mass Obtained Based on Earthquakes with 2% Prob. Exceed
10.00	0.94	2.12
7.50	0.95	4.20
6.25	1.55	5.20

CONCLUSIONS

A modified BRANZ procedure for rating the lateral capacity of wall systems and a procedure for reliability assessment of shear walls under earthquake loads have been presented. The overall method is general and can be applied to allow the direct use of laboratory data, from cyclic or pseudo-dynamic testing, for dynamic and seismic reliability analyses of lateral-resisting systems with no distinct yield point. Our key modifications to the BRANZ procedure involved: (1) the use of a multi-criteria, parallel system identification method to determine a structural model that fits test data from both cyclic testing and pseudo-dynamic testing; and (2) probabilistic treatment of site-specific ground motions. Then we estimated the reliability index β for the systems rated using the modified BRANZ procedure under a range of earthquake intensities in Tokyo.

For the 910-mm wide plywood-sheathed shear wall that we analysed in this study, we found that the reliability index β ranged from 0.94 to 5.20, depending on the assumed displacement capacity determined from the static cyclic test, and the intensity of earthquakes used to generate the displacement spectrum. The choice of dependable displacement capacity and design earthquake level can significantly affect the predicted reliability of the wall. This implies that in order to accurately determine system reliability, both the design earthquake level and the dependable displacement capacity must be chosen carefully. Since the wall's reliability is quite sensitive to these parameters, it is desirable to quantify and include the inherent uncertainty in displacement capacity and ground motions in the analysis. Quantifying and incorporating other potential sources of uncertainty such as natural period for a given displacement level, and other system properties may be necessary. This is the primary goal in the next phase of our research.

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