Reliability-based Capacity Rating of Wood Shear Walls under Seismic Loading

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ABSTRACT

The purpose of this study was generate the reliability-based seismic capacity rating of wood shear walls. The rating is based on the seismic design provision of the 1994 Uniform Building Code. The shear wall resistance was established based on the provision of Eurocode 5, (2004). Uncertainties in the design variables were fully accommodated. It was established that, variability in the ground motion has very serious effect on the safety of wood frame structures under seismic loading. A review of seismic design code to fully accommodate uncertainties through probabilistic design is advocated in this study. Hence, the use of the developed design chart will ensure uniform reliability in seismic capacity rating of wood shear wall.
Keywords: Wood shearwall, capacity rating, Action Reduction Factor

INTRODUCTION

Wood is among the oldest material used by man from time immemorial particularly for the construction of houses as well as monumental buildings, bridges and other special purpose construction. Apart from the advantageous mechanical and construction properties that are the cause of large use of wood in building construction, the material is natural, and renewable. From the aspect of sustainable management, wood is likely to be the predominant building material of the future (Lucas, Olorunmisola and Adewole, 2006). There is significant variability in the strength properties of timber members. On the other hand the loading applied to structures in general are highly uncertain, especially, the environmental loading like earthquake and wind loads. Most modern design codes use limit state design approach.

They recognize the presence of uncertainties through the use of partial safety factors, within deterministic design framework (Afolayan, 1992). Uncertainties in engineering analysis and design can best be undertaken using probabilistic method (Ang and Tang, 1975; Melchers, 1999; Ditlevsen and Madsen, 2005). In this paper attempt was made to investigate the performance of wood frame shear walls located in seismic prone areas using probabilistic technique. The reason for the selection of shear walls, is for the fact that, this component serve as the major lateral load resisting system of timber structures during earthquake (Ming, 1999; Folz and Filiatrault, 2002; Rosowsky, 2002; Rosowsky and Ellingwood, 2002; Lind and Mathew, 2003). The provision for the design of structures in seismic region is based

on elastic spectral acceleration to determine the required lateral strength of the structures at elastic range (UBC, 1994; Eurocode 8, 2004). The design lateral strength of the structure is then obtained by dividing the elastic strength by the force reduction factor *"R"*. The force reduction factor represent the inherent over-strength and global ductility of the lateral force resisting system (Stehn and Johanson, 2002). The required lateral strength of structures of buildings under seismic action is the product of the induced acceleration and the building mass. The capacity of buildings under seismic action is therefore analogous to is its maximum sustainable mass.

In this study, the sustainable tributary loading for a Single-Degree-of-Freedom wood shear wall subjected to seismic design trend of seismic zone four based on the UBC (1994) seismic zone map were generated using probabilistic technique and the non-linear pushover analysis database of a SDOF shear wall generated by Folz and Filiatrault (2002). The reason for the adoption of seismic zone four is that, the trend in earthquake resistant design is towards larger design loads (Breyer, 1988). Seismic zone four is of high seismicity, with average Peak Ground Acceleration (PGA) of $39.24 m/s^2$ and maximum return period of 474 years. The PGA represents an extreme loading condition (Lagorio, 1990). The lateral (racking) load capacity of the shear wall was established based on the Eurocode 5 (2004) requirements.

Non-Linear Cyclic Analysis of Wood Shearwall: Folz and Filiatrault (2001) developed a simple numerical model capable of predicting the load displacement response and energy dissipation characteristics of wood shear walls under general quasi static cyclic loading. In the model the shear wall is comprised of three structural components: rigid framing members; linear elastic sheathing panels; and non linear sheathing-to-framing-connectors. The hysteretic model for the sheathing-to-framing connector takes account of pinching behavior and strength and stiffness degradation under cyclic loading. A robust displacement control solution strategy was utilized to predict the wall response under general cyclic loading protocol. The shear wall model has been incorporated into the computer programme CASHEW (Cyclic Analysis of Wood Shear Wall) as part of Federal Emergency Management Agency (FEMA) funded CUREE (Consortium of Universities Research in Earthquake Engineering) - Caltech Wood frame project (Folz and Filiatrault, 2002; Folz and Filiatrault, 2001).

As an application, Folz and Filiatrault (2002) performed pushover analysis on a planar SDOF shear wall. The dimensions of the wall are 2.4m x 2.4m. All framing materials are 38mm x 89mm Spruce. The top plate and the end studs consist of double members, while the bottom plate and interior stud are single members. Studs are spaced at 400mm at center. Conventional hold down at corners are used to prevent overturning of the wall and to ensure racking mode of deformation. The sheathing panel is 9.5mm thick Oriented Strand Board (OSB). The sheathing-to-framing Connectors are pneumatically driven 50mm long spiral nail, corresponding to 11 gauge (3mm diameter) (Dowrick, 1977). The cyclic loading test shown that, the planar shear wall treated as an isolated sub-assembly of wood frame structural system requires 75mm nail spacing between panel to intermediate studs connection,

and 300mm nail spacing between panel to end studs connection, when located in seismic zone four (Uniform Building Code, 1994)

The UBC 1994 Seismic Design Provision: According to the UBC (1994), static procedure, the total earthquake force (base shear) on a structure is given by:

$$V = \left(\frac{ZIC}{R} \right) W \tag{1.0}$$

where:

Z = Seismic zone factor (Table 16.1 of the UBC (1994)). It express zone seismicity in terms of effective peak ground acceleration with 10% probability of being exceeded in 50 years (10/50).

I = Importance factor, which reflect the need to protect essential facilities that operate after earthquake (Table 16-k of the UBC, 1994).

C = Dynamic response spectrum value, which account for how the building and soil can amplify the basic ground acceleration, given by equation (28-2) of the UBC (1994), as:

$$C = \left(\frac{1.25S}{T^{\frac{3}{4}}} \right) \leq 2.75 \tag{2.0}$$

in which, T is the building fundamental period of vibration which influence the buildings response to motion, obtained using equation (30-8) of the UBC (1994) as:

$$T = C_Z \left(h_n B \right)^{\frac{3}{4}} \tag{3.0}$$

CZ in equation 3.0 is generally taken as 0.02 for wood buildings, and hn is the height of the building in metres divided by a constant B equal to 0.3 . (UBC, 1994). However, since most one storey timber buildings have fundamental period in the range of 0.1 to 0.6 seconds (Dowrick, 1977), the value of "C" in equation (2.0) would be greater than the default value of 2.75. Hence the default value is appropriate for the structure considered for analysis in this study. It is therefore adopted for the analysis of the mathematical model.

S in equation 2.0 is the site coefficient (Table 16-J of the UBC (1994)). The code specified that, without sufficient geotechnical investigation to determine soil profile at the building location, S is taken as 1.5. When equation 2.0 is used to establish a plot of C against period of vibration the magnification spectrum is obtained.

R = Structural system coefficient, which is a judgment factor that accounts for building Ductility and damping. For plywood shear panel building. *R*, = 8.0 (Table 16-N of The UBC, 1994).

W = the tributary seismic weight.

Direct Displacement Design Methodology: The following is the procedure for the determination the sustainable tributary loading of the shear wall:

1. Determine the hysteretic response parameters corresponding to 75mm/300mm nail pattern from pushover analysis database. In this research work the data obtained in Folz and Filiatrault (2002).

2.   Plot the load-slip curve from the model parameters obtained in equ. 1 above.
3.   Determine the equivalent lateral stiffness of the wall at target displacement corresponding to FEMA drift limit of 2% for life safety (Folz and Filiatrault, 2002).
4.   Determine the equivalent elastic period of the wall.
5.   Determine the sustainable tributary loading at top of wall, which is given by:

$$W = \left(\frac{g K_{eq} T_{eq}{}^2}{4\pi^2} \right) \qquad (4.0)$$

where: W is the seismic weight at the maximum ductility.
g is the acceleration due to gravity
T_{eq} is the equivalent elastic period of the wall from the design displacement response spectrum (UBC, 1994),
K_{eq} is the equivalent lateral stiffness of the wall.

6.   Determine the required ductility R_R of the sheathing to framing connection, as an output of the reliability analysis.
7.   Determine the sustainable seismic weight W_s

$$W_s = \left(\frac{W}{R_R} \right) 8.0 \qquad (5.0)$$

First Order Reliability Method (FORM): The aim of structural design is to ensure that the applied action is greater than the structural resistance, that is:

$$R - S \geq 0 \qquad (6.0)$$

Where R is the structural resistance, S is the load effect and Z is the safety margin. This is based on deterministic approach. However the probability of failure is best established by modeling the uncertainties associated with each of the variables that defined the applied load and structural resistance. This reliability-based approach is adopted in this research. The probability of failure of the structure is given by the convolution integral (Melchers, 1999; Ditlevsen and Madsen, 2005).

$$P(Z < 0) = \int_{-\infty}^{0} f_z(\xi) d\xi \qquad (7.0)$$

$$= \Phi(-\beta) \qquad (8.0)$$

where $\beta = \dfrac{\mu(Z)}{\sigma(Z)} \qquad (9.0)$

and $f_Z(\xi)$ = probability density function of Z
    $\Phi(-\beta)$ = standard normal distribution for the variable β
    β = β is the safety index. It is simply a measure (in standard deviation) of the distance that the mean μ_G is away from the safety-failure interface.

It is supposed that the reliability function can be linearised, so a tangent plane in a point on its surface can be expressed by a first order Taylor series expansion (the basis of FORM). The acronym FORM is short for First Order Reliability Method.

$$Z_{Lin} = Z(X*_1, X*_2, ...X*_n) + \sum_{i=1}^{n}\left(\frac{\partial Z}{\partial X_i}\right) Xi = X_i^* \cdot (X_{i\_} X_i^*) = 0 \qquad (10.0)$$

where

Z_{Lin} = linearised reliability function Z is linearised in $(X^*_1, X^*_2, ...X^*_n)$

n = number of stochastic variables in the reliability function

$\left(\frac{\partial Z}{\partial Z}\right) Xi = X_i^*$ = partial derivative of Z, with respect to $X_i - X_i^*$

The mean value and the standard deviation of Z_{Lin} are

$$\mu_{Lin} = Z(X*_1, X*_2, ...X*_n) + \sum_{i=1}^{n}\left(\frac{\partial Z}{\partial X_i}\right) Xi = X_i^* \cdot (\mu_{i\_} X_i^*) = 0 \qquad (11.0)$$

And

$$\sigma Z_{Lin} = \sqrt{\sum_{i=1}^{n}\left(\frac{\partial Z}{\partial X_i}\right) 2Xi = X_i^* \ \sigma_{xi}^{2}} \qquad (12.0)$$

The probability of failure can be approximated by:

$$P(Z<0) = \int_{-\infty}^{0} f_{ZLin}(\xi)d\xi \qquad (13.0)$$

$$= \Phi(-\beta) \qquad (14.0)$$

Where

$$\beta = \frac{\mu(Z_{Lin})}{\sigma(Z_{Lin})} \qquad (15.0)$$

The probability of failure is given by (Melchers, 1999):

$$\Phi(-\beta) = \left(\frac{1}{(\beta\sqrt{2\pi})}\right)^{-\beta^{\frac{2}{2}}} \qquad for \ \beta > 2.0 \qquad (16.0)$$

FORM5 is a computer package developed by Gollwitzer, Abdo and Rackwitz (1988) in FORTRAN 77 to perform First Order Reliability Investigation. A compatible computer program written in this study using FORTRAN 77 was linked with FORM5 and the reliability-based assessment required in this study was fully automated.

Development of the Reliability Function: The design lateral load carrying capacity $R_{i,v}$ (the racking resistance) under a force F_{iv} acting at the top of a cantilevered panel (Fig. 1.0) secured against uplift by vertical action or by anchorage based on the Eurocode 5 (1995) (clause 9.2.4.2) is determined from the following simplified method of analysis for walls made up of a single panel, with sheet fixed to one side of a timber frame (cl 9.2.4.2(1)), provided that:

i. The spacing of fasteners is constant along the perimeter of every sheet, and

ii. The width of each sheet is at least $^h/_4$.

Is given by:

$$R_{iv.d} = \frac{R_{fd} b_i}{S} \qquad (17.0)$$

where:

> $R_{f,d}$ is the lateral design capacity of an individual fastener,
>
> S is the fastener spacing
>
> b_i is the wall panel width

The characteristic resistance R_k of a well designed ductile sheathing-to-frame connection is given in Annex D.1.3 Eurocode 5 (2004)

$$R_k = 1.15 k_{cal} \sqrt{(2M_{y,k} f_{h,1,k} d)}$$

where; R_k = the load carrying capacity per shear plane per fastener

$f_{h,1,k}$ = embedding strength of the sheathing

d = diameter of fastener

$M_{y,k}$ = yield moment of fastener

k_{cal} = a factor to account for axial forces which develop in fastener.

Eurocode 5 (2004), recommends the following values for the characteristic embedding strength for plywood or oriented strand board (OSB):

$$f_{h,k} = 0.11 \rho_k d^{-0.3} N / mm^2 \qquad (18.0)$$

where ρ_k is the characteristic density in kg/m^3 and d the nail diameter in mm.

In this study the sheathing panel is Oriented Strand Board (OSB) having a mean density of 500kg/m^3 (Rio, 1999). The framing members are SPF corresponding to timber strength class C24: EN 338 (Blass et al, 2008).

The characteristic yield moment for common smooth steel wire nails made from a wire having a minimum tensile strength of 600N/mm^2 (Blass et al, 1995; Eurocode 5, 2004) is given by:

$M_{y,k}$ = 180d$^{2.6}$Nmm (19.0)

The reliability function for racking mode (deformed configuration of shear wall subjected to lateral displacement in which the bottom and top plates of the wall remain horizontal given to the wall a parallelogram geometry) is then given by

$$G(X) \left[R_{iv,d} - \left(\frac{F_{iv,d}}{R_R} \right) \right] \qquad (20.0)$$

Where, $R_{iv,d}$ is the design capacity of the panel against racking mode, and

> $F_{iv,d}$ is the applied lateral force at top of wall
>
> R_R is the required ductility of the sheathing to framing connection.

The statistics of the stochastic variables are presented below (Table 1).

Table 1: Probabilistic Models of the Basic Design Variables

S/No	Basic Variable (x)	Distribution Model	Mean	Standard Deviation	Coefficient of Variation
1	Density of Sheathing (kg/m^3)	Lognormal	500 kg/m^3	7.5	0.015
2	Nail Diameter (mm)	Lognormal	3.0 mm	0.15	0.05
3	Zonal Coefficient	Gumbel	4.0	2.40	0.60
4	Seismic Weight (kN)	Lognormal	50.0 kN	1.00	0.05
5	Partial safety factor for panel	Normal	1.2	0.06	0.05
6	Partial Safety Factor for Nail	Normal	1.3	0.07	0.05
7	K_{mod}	Normal	1.1	0.06	0.05
8	Fastener Spacing (mm)	Normal	75.0 mm	3.75	0.05
9	Panel Width (mm)	Normal	2400.0 mm	120.0	0.05

RESULTS AND DISCUSSION

Based on the provision of Uniform Building Code (1994), for a well designed wood shear wall with high dissipative panel-to-framing connection, the value of Action Reduction Factor (R) equal to 8.0 is recommended. It is observed in Fig. 1 that when the coefficient of variation of ground acceleration are 60, 50, 40, 50, 30, and 20 percent safety indices of up to 2.30, 2.60, 3.10, 3.70 and 4.50 can respectively be achieved. Safety index greater than 4.5 can be achieved when the Coefficient of Variation of ground acceleration is 10%. From Fig. 2, the values of Sustainable tributary seismic loading for the shear wall with the defined configuration in this study were extracted and presented on Table 2. The results show significant effect on variability of ground acceleration on the capacity of wood shear walls.

For example, at a target safety index of 3.0, the sustainable seismic weight corresponding to 10% coefficient of variation for ground acceleration is 95kN. This value dropped to 80kN, 60kN, 50kN, 40kN and 35kN for coefficient of variation of ground acceleration equal to 20%, 30%, 40%, 50% and 60% respectively. It is clear from this investigation that, uncertainty in ground motion is very large, and uncertainty can only be fully accommodated in design through probabilistic method. Figure 3 is the design chart for wood shear wall in seismic zones, based on target safety indices. The rating of any shear wall with similar structural configuration investigated in this study can be made, by considering the rating in the design chart as that of a single wall unit. For wall consisting two units, the rating for the wall is doubled and so on.
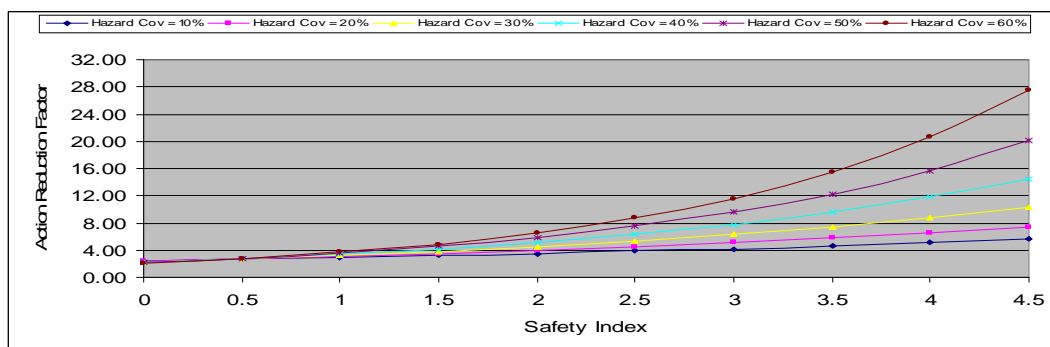

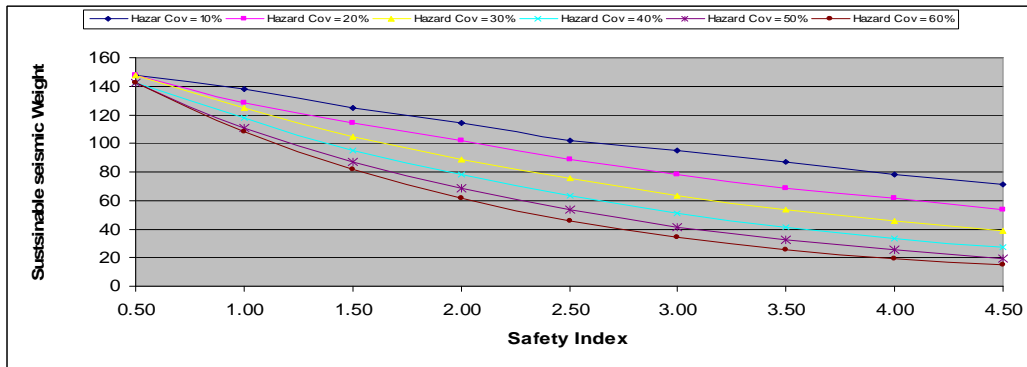**Fig. 1:** Energy Dissipation Demand at Various Safety Level

Fig. 2: Sustainable Seismic Weight at Various Safety Level

Table 2: Sustainable Tributary Seismic Loading (R ≤ 8.0)

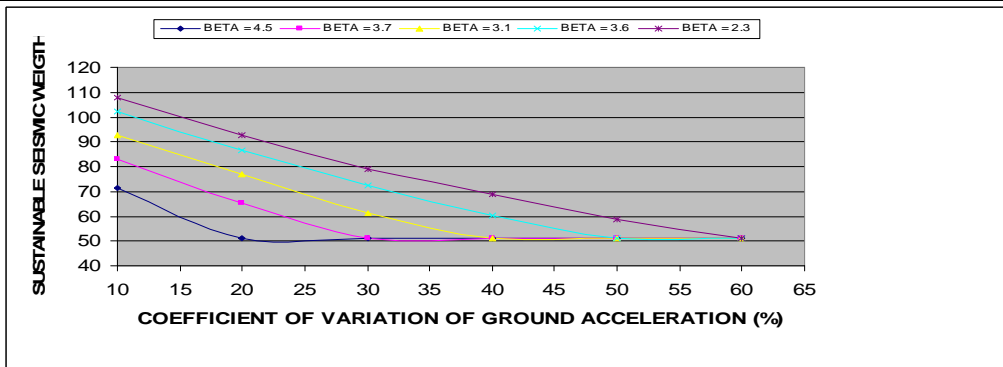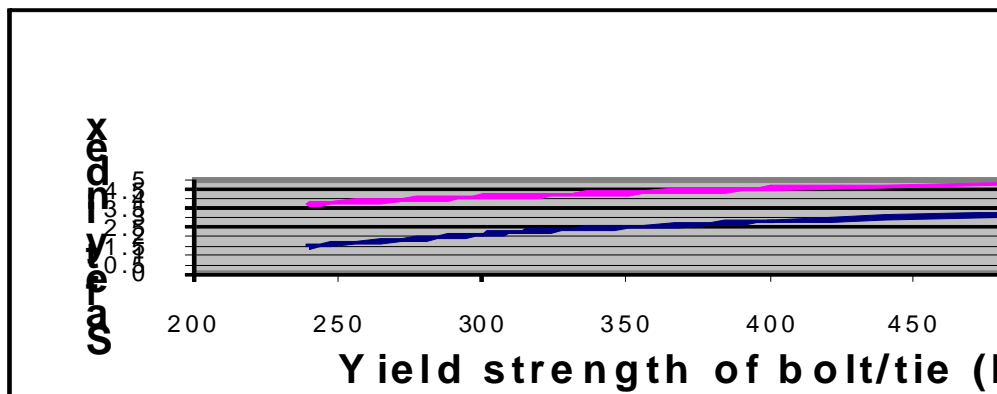| Cov (%)	Target Safety Index				
	4.5	**3.7**	**3.1**	**2.6**	**2.3**
10	71.26	83.14	92.81	102.32	107.86
20	51.16	65.42	76.74	86.75	92.81
30	51.16	51.16	61.39	72.56	78.81
40	51.16	51.16	51.16	60.46	68.80
50	51.16	51.16	51.16	51.16	58.69
60	51.16	51.16	51.16	51.16	51.16

Fig. 3: Sustainable Seismic Loading at Various Target Reliability Indices (Action Reduction Factor = 8.0)

CONCLUSION

The reliability-based seismic capacity rating of wood shear wall was presented in this paper. The rating is based on the seismic design provision of the 1994 Uniform Building Code. The shear wall resistance was established based on the provision of Eurocode 5, (2004). Uncertainties in the design variables were fully accommodated. It was established that, variability in the ground motion has very serious effect on the safety of wood frame structures under seismic loading. Review of seismic design code to fully accommodate uncertainties through probabilistic design is advocated in this study. The use of the developed design chart will ensure uniform reliability in seismic capacity rating of wood shear wall.

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