Comparison of Chilean and US Seismic Design Provisions for Timber Structures

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Abstract

The seismic design provisions for timber structures in the United States have advanced significantly in recent years. Significant attention has been given to the proper design and detailing required for acceptable ductility and toughness for surviving the lateral loads experienced during an earthquake. The process of developing a design philosophy for light-frame construction has also begun under the auspices of the Building Seismic Safety Council (2008), which will describe in detail the preferred structural mechanism and the necessary detailing requirements to insure this performance.

The Chilean timber design code is currently in the process of being revised in an effort to move the timber design code to one of the leading design codes in the world. Also, a draft of a building code update has been written by consultants at the British Columbia Institute of Technology (BCIT). The Chilean forest industries and timber engineering experts will now make the decisions on how to fill the format with the technical provisions that are appropriate for Chilean societal objectives. A parallel effort to update the timber design standard is also required. As part of this process, an initial evaluation of timber design codes from other countries is being undertaken, and comparisons with the current Chilean design code made. Upon completion of this review, decisions on which technical provisions will be adopted is to be made.

A second part of the design process is the determination of the seismic design forces. This paper is part of this process in that it makes comparisons between the current design provisions (and additional reference materials) available for use in the United States and those currently used in Chile. The effects of some of the more pertinent differences are also discussed in an effort to provide additional information for the committee to use in making their decisions.

In the United States, the forces are determined following ASCE 7-05 (2005) *Minimum Design Loads for Buildings and Other Structures*. In Chile, the forces are determined using NCh 433.Of96 (1996) *Deseño sismico de edificios*. While the Chilean standard is based on the *Uniform Building Code* (ICBO 1995), which is also based on ASCE 7-95 (1995), there are significant changes that effect the magnitude of the seismic design forces and the assumed relative performance of concrete, masonry, steel, and timber structures.

Comparisons

The first comparison between the two codes is the basis of the earthquake response. In the United States, the magnitude of the earthquake is that the maximum considered earthquake (MCE) has a 2 percent probability of exceedence in 50 years, which translates to a return period of about 2500 years. This earthquake magnitude is then reduced by 33 percent to obtain the design earthquake, which has a magnitude of about 10 percent exceedence in 50 years. This results in designing for a lower magnitude earthquake over a geographical area that is defined by the MCE. The Chilean code bases the design earthquake on a return period of 472 years. This is close to a 10% probability of exceedence in 50 years. Therefore, the conclusion can be made that both countries are designing for approximately the same magnitude earthquake; the difference is that the U.S. requires that the event be defined by the geographical region defined by the MCE. Chile also required that the design event be used for a wider region than the actual hazard maps would require because they use a zone map, which results in a step function defining the magnitude of the design event.

In both countries' standards, three options for design are available. These are 1) Equivalent Linear Lateral Force Analysis (or Static Analysis), 2) Modal Response Spectrum Analysis (Spectral modal analysis), and 3) Non-linear Time History Analysis. For low-rise structures (< 5 stories), only analysis methods 1) and 2) are typically used. Non-linear time history analysis is a specialized method of analysis that requires significant expertise to obtain reliable results. Therefore most, if not all, timber buildings are currently designed using Method 1) with a few being designed using Method 2). This paper will focus on Method 1), which is currently the predominant method for designing timber buildings. In addition, the assumptions for design comparison are presented in Table 1.

Table 1: Base assumptions for seismic analysis.

Variable	**U.S. Value**	**Chilean Value**
Building Type (Light-Frame, 3-story, Bearing wall system Area < 3,000 Occupancy <100)	Type II	Category C
Map Acceleration (g)	0.4	0.4
Soil Class	D	II
Response Modification Factor (R)	6.5	5.5
Overstrength Factor (Ω_0)	3.0	N/A
Displacement Amplification Factor	4.0	$R = 5.5$ *
Fundamental period of building	0.2 sec	0.2 sec

* Not written in Chilean Code, but used by a few designers. Most designers use 1.0.

Both design codes require that the base shear for the building be determined using an equation of the form

$$Q_0 = CIP = C_s W$$

Where: Q_0 = the base shear in Newtons or pounds
CI = C_s = the seismic coefficient
P = W = the total weight of the structure above the foundation

The CI or C_s term is where differences between the two design requirements start to show up. In the Chilean code, *I* is the importance factor that is included in the C_s for the United States. Both countries would assign a value of 1.0 for this term for the type of building being designed in this example.

The seismic coefficient for Chile is determined using the equation

$$C = \frac{2.75 A_0}{gR}\left(\frac{T'}{T^*}\right)^n \leq \frac{0.40 SA_0}{g}$$

Where: A_0 = Maximum horizontal acceleration
g = Acceleration of gravity
R = Response modification factor
n, S, T' = parameters relative to the foundation soil type.
T^* = Period of the highest translational equivalent mass in the direction of analysis

For the United States code, the value of C_s is determined using the equation

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

Where: S_{DS} = the mapped acceleration short period structures and is compared to limits associated with the mapped acceleration value for structures with a fundamental period of 1.0 seconds.
R = the response modification factor

I = the importance factor = 1.0 for the building used for this example.

If the values for each variable are inserted into the associated equation, the base shears determined are

$$\text{Chile: } Q_0 = \frac{2.75(0.4g)}{g(5.5)}\left(\frac{0.35}{0.20}\right)^{1.33}(1.0)P = 0.42P \le [0.40(1.0)(0.4g)/g]P = 0.16P$$

With a minimum value of:

$$Q_0 = \frac{A_0}{6g} IP = \frac{0.4g}{6g}(1.0)P = 0.067P$$

$$\text{U.S.: } V = \left(\frac{0.40}{6.5/1.0}\right)W = 0.062W$$

In other words, the building designed in Chile will be required to resist 16% of the building weight acting in a lateral direction, while the same building designed in the United States would be required to resist 6.2% of is weight acting in a lateral direction. This is the same as saying that the Chilean society wants its buildings to be 2.6 times as strong as the citizens of the United States expect their buildings to be.

Both design codes require that an accidental torsional load be superimposed upon the lateral design of the structural system. For static analysis, the Chilean code requires that an accidental torsional analysis be determined using an eccentricity for the mass of each floor of $\pm 0.10\, b\, Z_k/H$. Where *b* is the dimension perpendicular to the acceleration direction, Z_k is the height to the particular story, and *H* is the total height of the building. The U.S, code requires an accidental torsional for be determined using an eccentricity of 0.05 *b* for each story. What the additional accidental torsion effectively does is require that the walls of the building be even stronger. The Chilean requirement effectively increases this additional strength requirement to being twice that required in the U.S. for short buildings, and ratios the effect for taller buildings so that the mass associated with the upper stories has more effect on the torsional response. The U.S. requirement assumes that the torsional effects of all stories have an equal effect on the overall response, but the effect is only one-half that expected in Chile.

Together these two requirements effectively make Chilean buildings be designed to resist forces on the order of 2.75 – 3.0 times higher than the same building designed in the United States. One might be able to significantly reduce the penalty of using a 10 percent eccentricity for the torsional analysis if the design were to use modal superposition to determine the loading rather than equivalent linear static analysis. One can gain the benefits associated with the more elaborate analysis procedures.

The relative level of force with competing materials has a large impact on which material is chosen for a given project. For this purpose, reinforced concrete, and reinforced concrete block masonry are chosen as the competing materials since they are common materials used to construct building systems using walls as the lateral force resisting element. The respective values for the seismic response factor, *R*, for wall type building systems for these materials for each code are presented in Table 2. The seismic response factor is the factor in the base shear equation that determines the relative acceleration level that the structure is required to resist. The values are currently not based on rational analysis in either country, but rather are based upon post-earthquake evaluation of the relative performance of the different structural systems and engineering judgment. The higher the value of *R*, the lower the effective acceleration the building must resist.

If the values in Table 2 are investigated further, in the U.S. light-frame wood shear walls have the highest value of *R* and reinforced masonry has the lowest value, with reinforced concrete in between. In Chile, reinforced concrete has the highest value for *R* and reinforce masonry has the lowest, with wood in between. This gives wood construction a market advantage in the US while, reinforced concrete has the market advantage in Chile. If one compares the values, in the U.S. concrete and masonry are required to resist 30% and 225% higher accelerations than wood construction, respectively. In Chile the same comparisons show that concrete and masonry are required to resist 21% lower and 25% higher accelerations than wood construction, respectively. These differences illustrate the effect of the committee membership, and the member's relative familiarity with the various materials, when the values of the seismic response factor were being set. It also illustrates the need for a rational method for determining the seismic response factor and associated design variables such as the method developed by the Applied Technology Council ATC-63 (2008) project.

Table 2: Seismic response factors for concrete, masonry, and wood shear wall structural systems.

Material	**ASCE 7-05**	**NCh 433.Of96**
Light-Frame Shear Walls with Wood Structural Panels	6.5	5.5
Special Detailed Reinforced Concrete	5.0	N/A
Reinforced Concrete	4	7
Special Detailed Reinforced Masonry	5	N/A
Intermediate Reinforced Masonry	3.5	N/A
Confined Reinforced Masonry*	N/A	6
Ordinary Reinforced Masonry	2	4

*Confined reinforced masonry has reinforced concrete beams and columns confining the masonry walls and restricting the displacements of the masonry when racked. The concrete must resist more than 50% of the story shear.

The second check a designer must make when designing a building to resist earthquakes is a drift check. The drift is defined as the difference in the deflections of two adjacent stories divided by the story height. In Chile, all buildings are required to not deform more than 0.002h. Where, h is the story height. In the U.S., the requirement is 0.025 h for all structures 4 stories and less, except for masonry which is required to meet a drift requirement of 0.01h. In other words, the Chilean code requires that the timber buildings be over 10 times as stiff as in the U.S.

If one investigates this drift requirement further, 0.002h represents an inelastic drift of no more than 4.8 mm for a 2.4 m wall. McMullin and Merrick (2001) determined that damage to gypsum does not initiate until a drift of approximately 6 mm. Therefore, the drift requirement imposed upon the design by the Chilean code essentially requires the building to remain elastic. If the displacement amplification factor of R=5.5 that is used by some designers were to be applied, the allowable elastic drift would be (0.002h/5.5) = 0.00036h. For a 2.4 m high wall, the allowable elastic drift would be 0.9 mm! Even if the deflection amplification factor were taken as 1.0, the allowable drift would be only 4.8 mm, which still is in the initial elastic response region of the response curves. Obviously, the drift of the building will control in Chile because the code drift requirements are all but impossible to meet if the higher accelerations are used. Tests from several researchers (Dolan 1998, Salenikovich 2000) show that a 2.4 x 2.4 m wall would be restricted to less than 1 kN/m loading when the capacity of the wall would be on the order of 9-10 kN/m. If the drift requirements of the U.S. were imposed, the associated resistance design value would be close to the capacity of the wall.

Conclusions

A comparison of the effects of the seismic load determination and drift requirements was presented. The comparison showed that the drift requirements of the Chilean design code required that the building respond in an elastic manner and did not allow the wood shear walls to utilize their full strength. In fact, the drift requirements would result in eliminating almost all damage associated with the design event.

The Chilean strength requirements result in wood buildings being designed for accelerations (forces) 2.75-3.0 times higher than the equivalent building designed under the U.S. requirements.

When the relative strength requirements for competing materials were investigated, the U.S. design code give a 30% and 225% advantage to wood construction over reinforced concrete and reinforced masonry for the level of acceleration that must be resisted, respectively. The Chilean design code gives a 21% advantage to reinforced concrete over wood construction and a 25% advantage to wood over reinforced masonry for the level of acceleration that must be resisted. These differences illustrate the need to rationally set the values for the seismic design parameters, especially the seismic response factor, R.

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