Seismic Analysis of Light-framed Wood Multi-Story Residential Buildings

Samuel Shiotani, S.E., Associate
Douglas Hohbach, S.E., Principal
Joaquim Roberts, S.E., Principal
Hohbach-Lewin, Inc.
Palo Alto, CA.

Abstract

In current structural engineering practice in California, there are significant variations in how the seismic analyses of light-framed wood multi-story residential buildings are performed. The most significant variations are in the analysis techniques, as well as in the areas of modeling diaphragm and shear wall rigidity. For background code information on this topic, refer to Mochizuki and Fennell, (2002).

This paper will provide a brief summary of the application of flexible and rigid diaphragm design assumptions, current design practices and code criteria, shear wall deformations, shear wall modeling techniques and a step by step three dimensional modeling procedure that incorporates non-linear shear wall behavior into a linear analysis. This procedure provides a reasonable distribution of shear forces to shear walls and facilitates the calculation of building drifts as required by code. The goal is to demonstrate that analysis of rigid diaphragm behavior, multi-story shear wall deformations, and building drift via three dimensional modeling is feasible and practical for this building type.

Although this paper focuses on light-framed wood multi-story buildings, many of the methodologies and conclusions are likely also appropriate for light-framed light gauge steel multi-story buildings.

Building Type Description

Light-framed wood multi-story residential buildings typically consist of a floor to floor repetitive pattern of individual residential units. The unit separation walls (party walls) are back-to-back walls with an approximate 1 inch air gap that separates the units for acoustical purposes. At the core of the building, a set of corridor walls typically runs the entire length of the structure. Corridor and party walls are used as shear walls due to their length and regularity, as well as their lack of penetrations. The building's exterior walls are typically highly fenestrated and often stagger out of plane which, if

Figure 1. Typical building exterior

Figure 2. Typical residential building plan with central corridor (Bold walls represent shear walls)

these walls are used as shear walls, makes chord development and collector force distribution extremely challenging. When used as shear walls, this is especially true if a force distribution based on a flexible diaphragm model is utilized. Figures 1 and 2 depict typical building exteriors and building plans respectively.

A typical floor assembly, see Figure 3, consists of a ¾ inch to 1½ inch topping slab over ¾ inch minimum plywood over I-joist framing with two layers of 5/8 inch gypsum board ceiling supported by resilient channel.

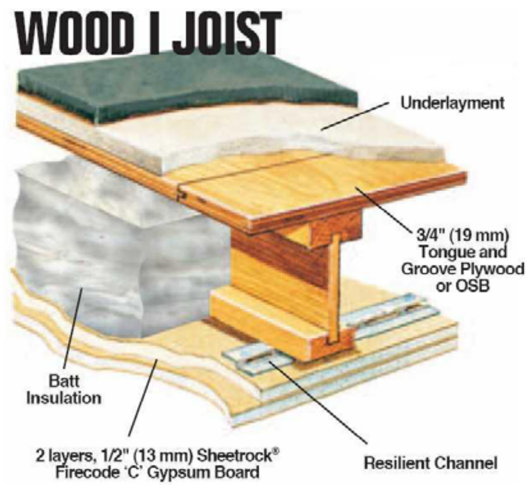

Figure 3. Typical floor assembly
Source: www.maxxon.com

Introduction

The 2007 California Building Code (CBC) explicitly allows the assumption of flexible, or if appropriate, rigid diaphragm behavior, in lieu of more complex semi-rigid diaphragm analysis, but requires the calculation of the relative rigidities of diaphragms and shear walls. Determining wood diaphragm and shear wall rigidities and deformations is difficult due to the non-linear load deformation behavior of panel assemblies, notwithstanding the contribution of finish materials. For three and four story wood buildings, deformation calculations used for determining stiffness, vis-à-vis the vertical lateral force resisting elements, are further complicated by the need to analyze the flexural deformation of multi-story shear walls acting as a single unit. Currently, this issue is rarely addressed in light framed wood multi-story building design.

Flexible Diaphragm Behavior and CBC Flexible Diaphragm Determination Criteria

ASCE 7-05 Section 12.3.1 requires that structural analysis shall consider the relative stiffness of the diaphragms and of the vertical elements of the seismic force-resisting system, unless the diaphragm can be idealized as either flexible or rigid. Diaphragm deflection, for a blocked wood structural panel, is calculated per 2007 CBC Section 2305.2.2.

$$\Delta = \frac{5vh^3}{8EAb} + \frac{vL}{4Gt} + 0.188he_n + \frac{\Sigma(\Delta_c X)}{2b}$$

This equation calculates the maximum mid-span deflection of an assumed simple span diaphragm. Using this equation and comparing the diaphragm deformations to the shear wall deformations, it can be determined if the diaphragm can be idealized as flexible. Diaphragms are permitted to be idealized as flexible, per ASCE 7-05 Section 12.3.1.3, if the computed maximum in-plane deflection of the diaphragm is more than two times the average story drift of adjoining vertical elements. Note that in most instances the diaphragm will not be simply supported; the calculated deflection may be scaled to account for continuous beam or propped cantilever support conditions.

Also, the 2007 CBC Section 1613.6 permits the idealization of the diaphragms of wood buildings as flexible when all of the following conditions are met:

- Toppings of concrete or similar materials are not placed over wood structural panel diaphragms except for non-structural toppings no greater than 1 ½ inches thick.
- Each line of vertical elements of the lateral-force-resisting system complies with the allowable story drift.
- Vertical elements of the lateral-force-resisting system are light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets.
- Portions of wood structural panel diaphragms that cantilever beyond the vertical elements of the lateral-force-resisting system are designed in accordance with Section 2305.2.5 if the CBC. These are the rigid diaphragm provisions.

In addition, ASCE 7-05 Section 12.3.1.1 permits diaphragms of untopped wood structural panels in one-and two-family residential buildings of light frame construction to be idealized as flexible.

In general, diaphragms in multi-story residential buildings cannot be classified as flexible using the calculated flexible diaphragm conditions of ASCE 7-05 12.3.1.3 because their diaphragm aspect ratios do not result in deflections two times larger than the adjacent shear wall deflections. However, given the above, it is feasible by code to design this building type based on a flexible diaphragm assumption. Nevertheless, it is not prudent to do so if it results in underestimating the contribution of long stiff walls. Per SEAOC Seismology (2007)

> "As the 1994 Northridge Earthquake illustrated, long stiff walls become critical after initial seismic cycles soften up the short wall elements….Accordingly engineers should consider that earthquake performance of wood frame structures tends to ultimately depend upon the longer shear walls. "

Also note that the requirement that each line of vertical elements comply with the allowable story drift requires the determination of shear wall deformations. For this to be done accurately, the multi-story nature of the walls must be incorporated into the calculations as described later in this paper.

Rigid Diaphragm Behavior and Open-front Structures

In multi-story wood buildings, due to their small diaphragm aspect ratios, diaphragm deflections are usually significantly smaller than the shear wall deflections, particularly at the upper stories, indicating that rigid diaphragm analysis has validity. Although semi-rigid seems like a reasonable categorization, the authors are unaware of any rigorous ways to approach such an analysis for a wood panel diaphragm. The issue of semi-rigid diaphragms in multi-story wood buildings, focusing specifically on system limitations, was addressed by SEAOC Seismology (2007), implying that the only practical avenue to a semi-rigid analysis was through a rigid diaphragm analysis.

> "Since it is typical to have a floor topping and mixed system in commercial and multi-family projects, rigid diaphragm analysis will be common in multi-story structures."

The 2007 CBC Section 2305.2.5 permits the design of open-front wood diaphragm structures, by assuming rigid diaphragm behavior, Figure 4 illustrates the definitions of width and length relative to the open-front. The code specifies a maximum length to width ratio of 0.67 for structures over one story in height, as well as a maximum length, *l*, of 25 feet; however, per the exception to this section, where calculations can show that diaphragm deflections can be tolerated, the length is permitted to be increased to a maximum length to width ratio of 1.5. Because the length is permitted to be increased with no maximum defined, it could be interpreted that there is no maximum limit to the diaphragm length provided the length to width ratio and the story drift requirements are met. In any case, the code's focus on an arbitrary limit length limit rather than an aspect ratio does not seem appropriate.

Multi-story wood buildings of this configuration present less risk to excessive deformation if designed as an open-front structure due to their small diaphragm aspect ratios ranging from 0.2 to 0.3. Furthermore, usually they have many interior shear walls that will contribute to additional rotational stiffness. For these reasons it is likely that diaphragms in these buildings will display rigid behavior, particularly if only slender shear walls are provided at the exterior. However, it is always desirable to design with lateral force resisting elements in these locations, preferably interlinked by strong spandrels.

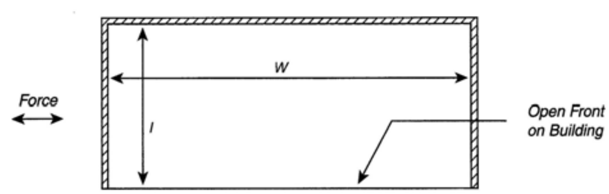

Figure 4. Open-front Diaphragm
Source: 2007 CBC Section 2305.2.5 Figure 2305.2.5(1)

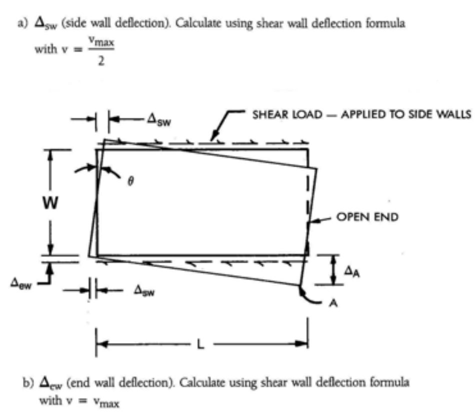

Figure 5. Open-front Diaphragm Deflection
Source: 1991 APA Design/Construction Guide

Professional judgment and careful analysis should be used if designing wood open front structures. Cobeen, Russell and Dolan, (2002) provide discussion of the vulnerabilities in open-front building design.

Due to the inaccuracies of calculating slender multi-story shear wall deformation and the difficulty of analyzing wood semi-rigid diaphragms, the building deformation at the exterior of the building can be conservatively calculated excluding the exterior walls and treating the structure as an open-front structure. However, the deflection calculation of an open-front diaphragm is not defined in the 2007 CBC. The American Plywood Association at one time provided a sample calculation in their *APA Design/Construction Guide, Diaphragms.* The design example assumes negligible flexural and chord splice slip deformation, while calculating shear deformation, nail slip, rotation due to side wall shear deflection and end wall shear deflection.

$$\Delta_A = \frac{vL}{2Gt} + 0.375Le_n + \frac{2\Delta_{sw}L}{b} + \Delta_{ew}$$

This is the method used to determine if diaphragm deflections can be tolerated per the exception of Section 2305.2.5. If Δ_A is less than the allowable story drift as defined in Table 12.12-1 of ASCE 7, the diaphragm deformation is considered acceptable.

2006 Structural/Seismic Design Manual

Often buildings of this type are currently being designed using a conventional envelope procedure as illustrated in the SEAOC Structural Design Manual (S.D.M.), where both flexible and rigid diaphragm assumptions are utilized. In the longitudinal direction, the exterior wall design is governed by a flexible diaphragm force distribution and the corridor wall design is governed by a rigid diaphragm force distribution. In the transverse direction, the party walls are designed based on the governing force distribution from either flexible or rigid diaphragm force distributions. The rigid diaphragm analysis assumes the relative rigidity of the shear walls are based on the story height of each wall at each level rather than the multi-story rigidity. The forces at each level are distributed independently from the distributions at other levels. While this approach is straightforward and usually conservative, it does not provide a direct avenue to calculating overall building drifts or accurately distribute shears to the various walls, particularly in cases where the vertical or lateral force resisting elements are not uniform top to bottom. Also, this approach overestimates the rigidity of more slender walls at upper stories and in many cases the exterior walls may have unreasonably large holdown hardware and compression posts due to the relatively large design forces.

Designing with an envelope method neglects the importance of understanding building deformation. Per Table 12.2-1 of ASCE 7-05, the Response Modification Coefficient, R, for light-framed walls sheathed with wood structural panels has increased from 4.5 to 6.5 for structures greater than 3 stories. This increase results in a 44% decrease in the seismic design base shear. The reductions in base shear can potentially drive the design criterion from strength controlled to drift controlled if minimum nailing of shear walls is utilized. Building drift may also be required to determine appropriate seismic separations and may be important in addressing deformation demands for architectural components.

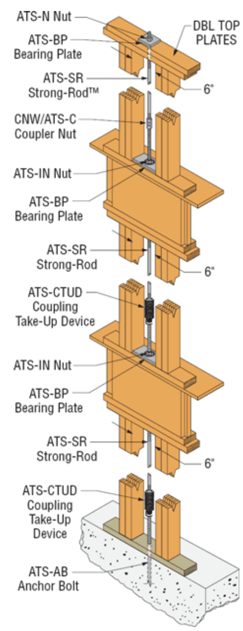

Figure 6. Typical continuous holdown assembly
Source: www.strongtie.com

Multi-Story Shear Wall Deformation

Modeling wood shear wall stiffness/deformation is the most difficult process in accurately modeling expected behavior of this building type. The non-linear nature of the expected shear wall behavior and the shear wall deformation equation does not lend itself well to analysis with commonly used computer programs. Shear wall deformations are generally approximated based on the shear wall deflection equation per 2007 CBC Section 2305.3.2.

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b} d_a$$

Hopefully improvement to this equation will be forthcoming when as part of the NEES Wood project a full scale multi-story wood shear wall building will be tested. However for design purposes useful results from the testing may not be available for a few years.

It is reasonable to assume multi-story shear walls with a continuous holdown system will act as a single element. Figure 6 shows a continuous tie-down system anchored at each floor.

Although it is common practice to assume that wood shear walls are pinned at each floor, this assumption would appear to have little validity for this building type, where walls that contribute significantly to overall building stiffness are either long and much stiffer than the diaphragm's out-of-plane stiffness or are shorter but interconnected by spandrel elements.

Note that per section "C805.4 Shear Wall Deformation", SEAOC (1999) *Recommended Lateral Force Requirements and Commentary*:

> "Testing reported in ATC [ATC,1995] and in Dolan and Heine [1996] indicates that for aspect ratios higher than 2:1, wall drift increases significantly, and further that the increase in deflection could not be adequately predicted using this equation."

Thus, it is implied that in order to appropriately use the shear wall deformation equation in a four story, forty foot tall building, a typical multi-story shear wall should be a minimum of twenty feet in length to meet the height to width ratio for these walls.

In the shear wall deflection equation, the most difficult term to quantify is the tie-down system deformation due to the myriad of factors that influence it: rod elongation, device slip and deflection, sill and top plate crushing, shrinkage, etc., that affect the deformation. Design considerations for shear wall drift are discussed by Nelson, Patel and Arevalo, (2002). The article references that some jurisdictions have placed a device deformation limit on the tie-down components of 1/8 inch, for design purposes a total tie down system deformation of 1/4 inch per floor is typically used. To minimize shrinkage, all lumber is specified to have a maximum moisture content, at time of installation, of 19%. Tie-down devices that anchor at each floor with non-slip shrinkage compensators are also highly recommended.

Shear Wall Modeling Techniques

Two modeling techniques utilizing linear properties for the wood shear walls were investigated to determine an effective and practical method of calculating and analyzing shear wall deformations. These modeling techniques are based on the following design information:

- In order to reduce iterations, a capacity based approach is used to determine shear wall rigidity. The maximum strength level shear force for an assumed nailing pattern and wall length is used to calculate shear wall deformation. Although this is an arbitrary point on a shear wall's load-deformation curve, it arguably the most significant one.
- Short shear walls (approximately six to eight feet in length) are generally ineffective unless nailed much more heavily than required for strength. These then can be modeled at a level below their capacity.
- Hohbach, Roberts and Cheng, (1996) presented a case study of the analysis of a four story wood framed building designed to meet enhanced seismic performance goals. A complex model incorporating a tri-linear force-deflection element for the shear walls was used to assess building behavior. It included the stiffness contribution of spandrels, gypboard and stucco finishes as well as separately modeling the hold-down devices. Utilizing standard strength and stiffness values for the finishes, they found that the strength and stiffness contribution of the finishes was only significant at lower force levels and was not significant at the strength capacity of the wall.
- Based on the above, since shear walls are being evaluated at their capacity, and for the purpose of simplicity, finishes are neglected.

The first technique follows the design approach followed by the S.D.M. of calculating shear wall rigidities based on the wall story height at each level. However, this approach is taken a step further by "stacking" these walls to determine force distribution and deformation in a three dimensional model, rather than assuming walls to be pinned at each story level. The second technique is based on the concept that shear wall rigidity and deformation should be calculated assuming the full multi-story height of the shear wall.

The "stacked" model assumes shear wall rigidities are calculated based on the story height of each wall and then are analyzed as if they are stacked. Figure 7 is a graphical representation of the modeling technique used. The rigidity at each level is calculated for all the walls and the lateral forces are then distributed to the shear walls at each level, based on the rigidity of the shear walls at that level. The "multi-story" model assumes the shear wall rigidities are calculated based on the full multi-story height of the shear wall. To calculate the deformation of a multi-story shear wall using the single story empirical deformation equation above, the shear wall must be dissected into single story components. Figure 9 shows how a wall is broken into its components. Figure 8 is a graphical representation of the modeling technique used. The fundamental design principles for this model are: the sum of the deformation of the components is greater than the deformation of the wall as a whole and the critical control deformation is the deformation at the top of the wall. The uniform rigidity for the full height walls is calculated and the lateral forces are then distributed to the shear wall based on their rigidity.

For simplicity, both methods utilize two dimensional "stick" modeling, where a column section, representing a shear wall, is assigned a moment of inertia, I, to produce the rigidity of the vertical resisting element calculated from the shear wall deflection formula. For the "stacked" model, this is achieved by calculating the moment of inertia to match the deformation of a single story shear wall under the same loading. For the multi-story model, this is achieved by calculating the moment of inertia to match the calculated top of wall deformation based on the wall's component deformation under the same loading. The column's other properties including area, section modulus and out of plane moment of inertia (about the weak axis) were defined to minimize their contribution to stiffness. In both cases, the non-linear behavior of the shear walls has been calculated and accounted for in the column stiffness and thus, the model's primary function is force distribution and overall deformation calculation.

Due to the linearity of the individual models, both techniques produced similar expected shear distributions at each level at each element. However, due to the fundamental non-correlation of the stiffness determination with the three dimensional models, the "stacked" model produced deformations approximately five to six times

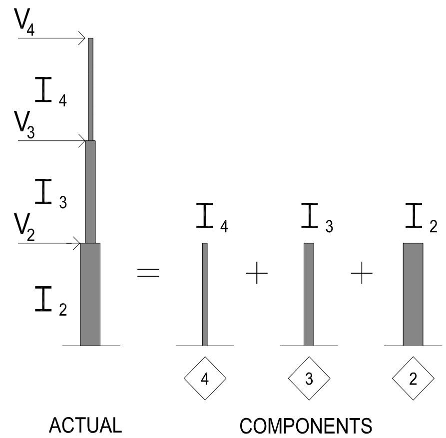

Figure 7. "Stacked" column shear wall

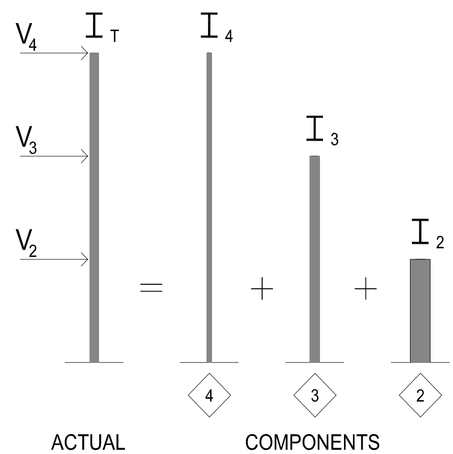

Figure 8. Continuous multi-story column shear wall

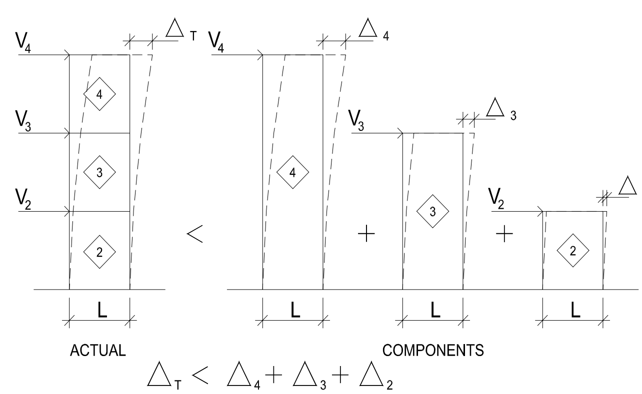

Figure 9. Assumed maximum multi-story shear wall deformation at the roof level

greater than the "multi-story" model since the moment of inertia required to produce the calculated deformation is generally six to eight times lower than compared to the moment of inertia required for the multi-story model. If the fundamental design principle of the multi-story model that the sum of the deformation of the components is greater than the deformation of the wall as a whole is true, then the deformations of the "stacked" model must be too high and incorrect. As such, using the "multi-story" model will give more accurate deformation results.

Models using shells and panels in lieu of columns were originally investigated and used for design purposes in an attempt to more accurately capture strut and spandrel behavior of shear walls. However, as noted above, the non-linear nature of wood shear wall deformation and the difficulty of modeling plywood properties combined with chord elements make using shells or panels challenging. Wood shear wall models of this nature are tools for force distribution and deformation only. Once the geometric properties are altered or defined to produce expected deformation behavior based on empirical formulas, whether it is a shell, panel or column, capturing secondary effects is unrealistic given typical analysis constraints on a design project. Thus, the use of column models is judged to be more appropriate for this level of investigation and design.

Three Dimensional Building Analyses

The use of a three dimensional linear computer model in conjunction with a spreadsheet program is a powerful combination for light framed wood multi-story building design. Spreadsheets are useful to address the multitude of variables and design iterations necessary to capture the building performance. The building's deformation and torsional behavior can effectively be observed, designed, altered and reanalyzed through this type of modeling. Revisions and modifications can be quickly and accurately done once the initial model is set up. The building skeleton itself is a useful graphic for quality control purposes, plan check or peer reviews. Comparable design control and accuracy cannot be achieved by hand calculations alone.

The following is a typical procedure for analyzing a light framed multi-story wood building:

- Shear walls in each principle direction are identified and their centroid relative to a datum is defined.
- Shear walls are grouped corresponding to their height, length and direction of resistance.
- Based on engineering judgment, plywood thickness/type, nailing pattern, holdown and compression post size for each wall group is assumed.
- The deflection of the full height shear wall, based on the assumed maximum multi-story shear wall deformation, is calculated.
- The corresponding moment of inertia for each group is calculated.
- A "stick" is created for each shear wall modeled as a solid rectangular column with its corresponding moment of inertia.
- A model is created incorporating each shear wall's location, height and stiffness.
- A rigid "membrane" diaphragm is created at each floor level and the calculated story shear forces and torsional moments due to accidental eccentricity are applied at the building's center of mass at each corresponding level.
- After analyzing the model, the member forces are post-processed in a spreadsheet and the adequacy of the assumed wall parameters (plywood type/thickness, nailing, compression post, etc.) is verified. If any of the assumed parameters are insufficient or over designed, the model maybe revised and the procedure repeated until a complete and efficient design is achieved.

Summary

As described above, there are multiple design assumptions and modeling techniques that can be used to analyze multi-story light framed wood buildings. As more and more of these building are being designed and built, especially in the high seismic metropolitan areas of the State, it is important that engineers can reasonably predict the building's behavior in a seismic event as well as construct a reliable and economical design. Three dimensional modeling as presented can more accurately determine the expected behavior of the building than the common hand calculation methods. Expected values of required seismic separations at adjacent buildings can be calculated. Furthermore, it gives engineers the ability to redesign efficiently when modifications to the design are made by the owner and/or architect. In more complicated applications, using similar design methods, modeling moment resisting frames in conjunction with wood shear walls can also be accomplished.

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