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IMPROVED SEISMIC ANALYSIS OF LIGHT-FRAMED MULTI-STORY RESIDENTIAL BUILDINGS

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ABSTRACT

In current structural engineering practice in North America, there often are significant inaccuracies in how the seismic analyses of light-framed multi-story residential buildings are performed, particularly with respect to the treatment of diaphragm and shear wall behavior. Improvement in analysis techniques, including in the modeling of diaphragm and shear wall rigidity and computing building deflections, may be warranted. This paper provides a brief summary of the application of flexible and rigid diaphragm design assumptions, current design practices and code criteria in the United States and Canada, calculation of shear wall deformations, shear wall modeling techniques and introduces three-dimensional modeling procedures that incorporate non-linear shear wall behavior into a linear analysis.

Introduction

Light-framed multi-story residential buildings are more challenging to analyze than buildings constructed of more homogeneous materials. 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 light framed buildings, deformation calculations used for determining diaphragm 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 multi-story unit. Currently, this issue is not commonly addressed in light framed multi-story building design, but should be to improve understanding of the seismic behavior of this building type, and so that multi-story light frame building analysis correlates better with the analysis of buildings of other materials.

This paper is informed by structural engineering practice on the West coast of North America and references the 2006 International Building Code (IBC), which incorporates ASCE 7-05 and the 2006 British Columbia Building Code (BCBC) which is based on the National Building Code of Canada.

Building Type Description

A typical floor assembly, for light-framed multi-story residential buildings, consists of a gypsum concrete topping slab over plywood over joist framing, with a gypsum board ceiling attached to the underside of the joists. A typical wall assembly consists of stucco over plywood studs, with a gypsum board interior finish. Light-framed

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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 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 (Figure 2). 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 (Figure 1) and often stagger out of plane (Figure 2), which, if these walls are used as shear walls, makes chord development and collector force distribution extremely challenging.

Figure 1. Typical building exterior

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

Figures 1 and 2 depict typical building exteriors and building plans respectively.

A typical feature of this building type is uniformly distributed shear walls of roughly equivalent stiffness, however where this is not the case, more attention to analysis must be paid.

British Columbia Wood Frame Provisions

The BCBC in 2009 incorporated new mid-rise wood frame building provisions allowing construction of wood frame buildings up to six stories in height. Apparently recognizing the uncertainties inherent in current commonly utilized analysis procedures for this building type, two types of structural irregularities tabulated in BCBC Table 4.1.8.6 are prohibited, the in-plane discontinuity in Vertical Lateral-Force-Resisting Element and Out-of-Plane offsets.

Diaphragm Rigidity

If vertical lateral force resisting elements are not of roughly equivalent stiffness and uniformly distributed, then understanding the degree of diaphragm rigidity is important to understanding expected building seismic response and seismic design.

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 per definitive criteria.

Diaphragm deflection, for a blocked wood structural panel, is usually calculated per 2006 IBC 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.

The BCBC does not make any statements with respect to diaphragm rigidity, but in Section 4.18.15.(1) requires that diaphragms and their connections shall be designed so as not to yield. Even though this provision would likely be more consistent with a rigid diaphragm assumption, a flexible diaphragm is commonly utilized for the design of this building type. However, in the upcoming version of the code it is proposed that diaphragms that exhibit ductile behavior will be allowed to yield, and where acting in combination with wood shear walls, the seismic design force can be used for the design of the diaphragm rather than the shear walls.

IBC Flexible Diaphragm Criteria

The 2006 IBC explicitly allows the assumption of flexible, or if appropriate, rigid diaphragm behavior, in lieu of a more complex (and undefined) semi-rigid diaphragm analysis. Also, the 2006 IBC 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 IBC Section 2305.2.5.

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 prescriptive requirements, 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 to resist lateral load.

"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." (SEAOC Seismology, 2007)

Also note that the above 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. Finally, in some cases these buildings have mixed systems, for instance steel moment frames at the first floor level, disqualifying them in the United States from analysis using a flexible diaphragm assumption.

Rigid Diaphragm Criteria

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), "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." implying that the only practical avenue to a semi-rigid analysis was through a rigid diaphragm analysis.

Assuming rigid diaphragm behavior, the 2007 IBC Section 2305.2.5 permits the design of open-front wood diaphragm structures. Figure 3 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 length limit rather than an aspect ratio does not seem appropriate.

The current BCBC no longer has any prescriptive requirements with respect to open front structures; such requirements were last incorporated into the 1989 version.

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 likely favorable to design with lateral force resisting elements at the exterior to reduce drift at these walls. It also should be noted that the stiffness of the perpendicular walls significantly influences the amount of drift at the "open front" exterior walls.

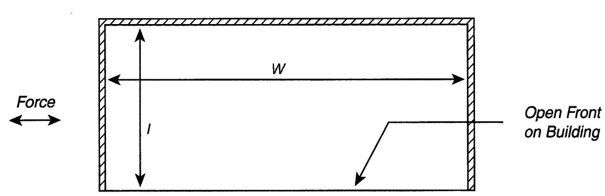

Figure 3. Open-front Diaphragm

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.

Importance of Calculating Drifts

Story drift limits are specified in ASCE 7-05 Section 12.12.1 and BCBC Section 4.1.8.13, however these limits are often not rigorously checked for multi-story light frame buildings in either U.S. or Canadian practice. For typical buildings, including multi-story light frame, the specific limits are .02h and .025h respectively, with each code also specifying stricter limits for special occupancy buildings.

Per Table 12.2-1 of ASCE 7-05, the Response Modification Coefficient, R, for light-framed walls sheathed with wood structural panels is now 6.5 rather than 4.5, as previously noted in Table 16-N of the code previously in force in California (UBC 1997) for structures greater than 3 stories. This increase results in a 44% decrease in the seismic design base shear. This reduction in base shear often results in a shift of the governing design criterion from strength controlled to drift controlled, particularly where shorter shear walls are utilized. The equivalent value in the BCBC of R_dR_o for nailed shear walls: wood based panel of 3.0 x 1.7 = 5.1 also sometimes results in drift governing the design.

Calculation of building drift is required to determine appropriate seismic separations at conditions where there are adjacent buildings or required seismic separations. Drift calculations also may be important in addressing deformation demands on architectural components as described in Chapter 13 of ASCE 7-05 and BCBC Section 4.1.8.17.

Calculation of building drift for the purpose of determining the period of the building by rational methods can be done to obtain a lower seismic design base shear, as is demonstrated in the Canadian design example (APEGBC, 2009) described in the following paragraph..

Past Design Examples

Buildings of this type may be designed using a conventional envelope procedure as illustrated in the SEAOC Structural Design Manual (SEAOC, 2006), 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 hold down hardware and compression posts due to the relatively large design forces.

The design example in the BC publication on 5 and 6 storey buildings (APEGBC, 2009) also utilizes a flexible diaphragm assumption to distribute the base shear to the shear walls, and notes that an analysis utilizing a rigid diaphragm assumption should also be performed, but proceeds quite differently from that point. Wall deflections are calculated, not floor by floor but over the height of the building, incorporating terms intended to capture the overall flexural deformation of the wall. These deflections are utilized to determine a building period by rational methods to determine a new base shear. The design is then iterated until a building that conforms to the drift design criterion is designed.

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 have been generally approximated based on the shear wall deflection equation per 2006 IBC Section 2305.3.2.

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

Recently a simplified 3-part equation has been introduced (SDWPS, 2005) that evaluates the shear stiffness of the plywood and its nailing at its design strength and thus makes the shear term in the equation linear.

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. 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.

Although it is common practice to assume that wood shear walls are pinned at each floor, this assumption has 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.

Reported testing by (Dolan 1996) for wood shear wall aspect ratios higher than 2:1, wall drift increases significantly beyond that predicted by this equation, and further that the increase in deflection could not be adequately predicted using this equation. Thus, it is implied that in order to use the shear wall deformation equation to accurately calculate drifts, in a typical four story, forty foot tall building, where the beams are not stiff enough to restrain the walls, 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.

The contribution of finishes affects shear wall load-deformation behavior, however there is evidence that their influence may be neglected at loadings near the shear wall's capacity. A case study, (Hohbach, Roberts and Cheng, 1996) of the analysis of a four story wood framed building designed to meet enhanced seismic performance goals, utilized a complex model incorporating a tri-linear force-deflection element for the shear walls to assess building behavior. It included the stiffness contribution of spandrels, gypsum board 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.

Three Dimensional Building Analysis

A practical three dimensional modeling method (Shiotani et al, 2008) has been described for this building type utilizing "off-the-shelf" computer programs. For simplicity, the method utilizes 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. 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. 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) are defined to minimize their contribution to stiffness. The non-linear behavior of the shear walls is calculated and accounted for in the column stiffness and thus, the model's primary function is force distribution and overall deformation calculation. The column sections are input into a computer model with rigid "membrane" diaphragms at each 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.

The building's deformation and torsional properties can effectively be evaluated, 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.

In the first commercial customization of a software analysis program for light frame construction known to the authors, in September 2009 RISA Technologies introduced wood analysis modules for its RISA Floor and RISA 3D programs. Utilizing the first two terms of the shear wall deflection equation in the 2005 Special Design Provisions (SDPWS, 2005), plate elements are used for the shear walls. The shear wall chords are also modeled. Hold down slip is only incorporated into the model at the lowest level of the wall. Hold down slip at each floor level is potentially a significant factor to the overall wall deformation. By incorporating hold down slip only at the lowest level of the wall there is an inherent design assumption that there is a continuous hold down system used and that slip at each floor level is being minimized by non-slip shrinkage compensators. The program apparently has the capability of iterating to determine required shear wall nailing and computes building drifts.

Summary

As described above, there are multiple design assumptions and modeling techniques that can be used to analyze multi-story light framed buildings. As this building type continues to be built in the high seismic regions of the continent, it is important that engineers can approximately predict the building's behavior in a seismic event as well as create a reliable and economical design. Three dimensional modeling utilizing rigid diaphragm assumptions likely provides more insight into the expected behavior of the building than the common hand calculation methods. Expected values of required seismic separations at adjacent buildings can be calculated. In more complicated applications, using similar design methods, modeling mixed systems and discontinuities in vertical lateral-force resisting elements can also be accomplished.

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