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Wood-Framed Design in Seismic Areas: Re-tooling for the 2006 IBC in Light of Current Design Practice

Scope:

This article is focused upon typical light-frame buildings and issues related to the transition to the 2006 IBC from either the 2003 IBC or 1997 UBC. Issues related to the new wind design provisions, essential facilities or structures located in the flood plain are not considered in the article.

The topics are organized below in three general categories:

1) determining forces and allowable stresses
2) system limitations
3) new prescriptive wood code issues

Symbols and Abbreviations:

See Publication for Symbols and Abbreviations (in context)

References:

I) IBC 2006 (or 2007 CBC) chapters 16 and 23,
II) ASCE 7-05 chapters 11 and 12,
III) ANSI/AF&PA "Special Design Provisions for Wind and Seismic" (SDPWS) – 2005.

Authorship:

This publication represents views endorsed by the SEAOC Seismology Committee. It may differ from the views, methods, policies and interpretations of some building authorities. Engineers are cautioned to ascertain such views, methodologies, policies and interpretations in advance of design.

This document was originally authored by Tom VanDorpe, Robert Chou and Andrew Fennell and was subsequently approved by the SEAOC Seismology Committee October 2007.

2007-2008 SEAOC Seismology Committee Membership:

Acknowledgement

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1) Introduction

With every new code adoption, practitioners are challenged to efficiently find the resultant effect upon current practice. Therefore to assist practicing structural engineers and plans examiners we have identified items that promise to change the status quo of light-frame wood construction and have specifically targeted areas where code provisions have outpaced common practice. In some areas, the IBC has reinforced existing code language which may lead to more uniform enforcement..

To limit this article we focused upon typical light-frame buildings and issues related to the transition to the 2006 IBC from either the 2003 IBC or 1997 UBC. Issues that effect a UBC transition, (i.e. not a change for 2003 IBC users) have the code section ***italicized***. Also not included are issues related to the new wind design provisions, essential facilities or structures located in the flood plain.

The code documents you should have available are:

I) IBC 2006 (or 2007 CBC) chapters 16 and 23,
II) ASCE 7-05 chapters 11 and 12,
III) ANSI/AF&PA "*Special Design Provisions for Wind and Seismic*" (SDPWS) – 2005.

The topics are organized below in three general categories:

1) determining forces and allowable stresses,
2) system limitations, and
3) new prescriptive wood code issues.

2) Determining Forces and Allowable Stresses.

The 2006 IBC uses ASCE 7-05 to determine the base shear and system limitations, with only a few minor exceptions. Note that ASCE 7-05 Chapter 14 is excluded (IBC 1613.1), so you must refer back to the appropriate IBC chapter, rather than ASCE 7-05 for material specific seismic design and detailing requirements. Also, it can be shown that LRFD has about a 12% seismic capacity advantage over ASD on wood frame shear walls. However ASD is allowed and expected to remain the standard practice. Here is a list of key issues to be aware of:

IBC 1603.1 General. The IBC requires roof, floor and rain loads, wind and earthquake design data including S_s, S_1, S_{DS}, S_{D1}, C_S, R, and SDC, the base shear and the analysis procedure to be shown on the "construction documents". There has been some debate on this topic from practitioners who resist putting the base shear on the drawings. Also impacting this debate is IBC Commentary section 1603 which states the term "construction documents" includes the calculations.

IBC 1604.3 Serviceability. New deflection limitations are required on roof members supporting non-plaster ceilings, exterior walls and interior walls with 5 PSF minimum horizontal load per 1607.13. These new deflection checks should be incorporated into your standard calculation routines.

IBC 1605.3 Load Combinations using Allowable Stress Design. The load combinations have been slightly modified and expanded to include rain, fluid, earth, and self-straining (temperature) loads. As before, the "basic" ASD load combinations allow the wood C_D factor, which per the NDS may be taken as 1.6 for seismic and wind load combinations. As before, the C_D factor cannot be combined with the "generic" 1.33 increase of the "alternate" load combinations. Therefore wood design will be most economic to use the "basic" load combinations, and items such as proportioning footings will use the "alternate" load combinations.

The vertical earthquake component, E_v, is no longer exempted for ASD design, except at the footings. Therefore, walls or columns or beams supporting any dead load will need the dead load slightly increased. ASCE 7-05 12.4.3 shows the resultant seismic load combinations with a higher dead load coefficient. In the case of a beam supporting a shear wall, the E_v must (still) be considered in combination with overturning.

IBC Table 1607.1 Live Loads. There are some new detailed live load provisions for wood roofs. A good example is that attics in one- and two-family residential with spaces 42 inches high and 24 inches wide or larger require a special non-concurrent live load of 10 psf (20 psf if designated for storage). It is a good idea to note if storage is allowed and add this item to your shop drawing review checklist. Please review the table for more details.

IBC Table 1607.1 Live Loads. Other changes relative to the UBC include: Balconies require 100 psf, except single family and duplex balconies less than 100 square feet can be 60 psf. For multifamily or commercial projects, public rooms and corridors still require 100 psf. Also, for roofs with a 4:12 pitch, the basic roof live load is increased from 16 psf to 20 psf which can be reduced for greater slopes or areas over 200 square feet per IBC 1607.11.2.1.

IBC Section 1613. Earthquake Loads. From the UBC, the base shear formulas have been reworked. Additionally, the seismic zone and soil type have been combined into a new SDC (*Seismic Design Category*) parameter determined by ASCE 7-05 11.4. The SDC ranges from category A through F and is used to determine the R factor, system height limits, level of ductile detailing (ordinary, intermediate, special), inspection requirements and numerous other prescriptive items throughout the code. The SDC is therefore fundamental to the seismic design.

The procedure for finding the SDC and base shear has three basic steps: First, use the longitude and latitude of the building site to determine the S_s and S_1 map parameters from the USGS website. Next, multiply these parameters respectively by the site coefficients F_a and F_v (refer to ASCE 7-05 Table 11.4-1 and 11.4-2) which consider the effect of soft or stiff soils. Combining the formulas in ASCE 7-05 11.4, the resultant base shear for short period structures can be expressed as $V=[2/3(F_a)(S_s)I/R]W$. Note that the building period cannot exceed $T_s=S_{d1}/S_{ds}$ for this expression to be valid. Also, there are several other detailed prescriptive modifications (e.g. IBC 1613.5.6.1, ASCE 7-05 12.8.1.3, 12.8.2.1) that can be used for less conservative results in certain cases.

It is a good idea to experiment with the values. You will find that where the S_s and S_1 values are high, they will trigger a higher SDC. Where they are moderate, the soil type alone can trigger a higher SDC. For example, in moderate seismic areas such as New York, soft soil can trigger SDC D. Finally, note that SDC E is only triggered by the S_1 parameter exceeding 0.75 and SDC F only applies to essential facilities. Therefore most light-frame "high-seismic" designs will be in SDC C, D and occasionally E.

ASCE 7-05 Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems. There are significant changes for wood frame structures using the common bearing wall system with wood shear panels. First, the *system response factor,* R, is now 6.5 for lateral systems with "light framed walls". Comparatively, the base shear is significantly decreased from the 1997 UBC which used 5.5 for "light-framed walls three stories and less" and 4.5 for "all other light-framed walls." Further, the height limit of 65' now applies only to SDC D-F with the other design categories unlimited, although IBC Table 503 and IBC 504.2 will still limit the height based on type of construction.

When using a bearing wall system with stucco or drywall shear walls, the *system response factor,* R, is significantly lower, which will yield higher seismic demands than the UBC. These brittle structural systems are prohibited in SDC E and F and limited to 35 feet in height for SDC D.

The table also includes a new *deflection amplification factor*, C_d, which should not be confused with the wood *load duration factor* C_D. For typical wood sheathed bearing wall systems a deflection amplification factor of 4 is used to determine the story drift in accordance with ASCE 7-05 12.8.6. The C_d factor should be a straightforward modification to whatever story drift routine you are currently using.

ASCE 7-05 Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems. The important table that contains the R and Ω_o factors is now found in ASCE 7 and is no longer reprinted in the IBC.

IBC 2305.3 Design of Wood Shear Walls. Shear walls with openings are given two design methods, Force transfer around openings (FTAO) and the Perforated Shear Wall (PSW) method. Since the perforated shear wall method is limited to 490 plf prior to reductions, its usefulness in high seismic design is severely limited. The FTAO can be any rational method to calculate the benefits of sheathing above and/or below the opening.

IBC Table 2305.3.4. Maximum Shear Wall Dimension Ratios. A change has occurred to allow aspect ratios on wood shear walls to exceed 2.0 and be as slender as 3.5:1. The height and width of the full height panel (or pier adjacent to an opening) are determined the same as with the UBC. Full allowable stresses are permitted at the 2:1 aspect ratio, and a sliding reduction from 0% to 43% occurs as the 3.5:1 maximum aspect ratio is reached. The reduction applies only on the slender segment or pier and not the entire wall. Similarly, the reduction does not apply to wind demands. The intent was to stiffen the short segment with a tighter nailing pattern, therefore it will be important to change the mindset that only one type of nailing pattern can occur on an individual shear wall line.

NDS and SDPWS. Within the NDS there have been several changes worth mentioning that affect wood members. First, the F_v stresses for sawn lumber were approximately doubled. Accordingly, the NDS assumes that significant splits and checks have not formed in shear critical areas of the wood member, which is a change from the previous thinking. Second, the nail penetration requirements for shear walls and diaphragms have been decreased. Third, a double 2x stud can be used in lieu of 3x members at adjoining panels. Fourth, there is a less severe adjustment factor for species other than Douglas Fir or Southern Pine in the shear wall table. Fifth, values are given for high load diaphragms that were previously only shown on research reports. Note also that the allowable shear values for wind have been increased 40% and may be combined with gypsum per 2305.3.9.

3) System Limitations

Once familiarity with the forces and allowable stresses is gained, the IBC has some more items related to the type of structural system used and building geometry:

IBC 1613.6.1 and ASCE 7-05 12.3.1. Diaphragm Flexibility. The topic of how to implement a light-frame rigid diaphragm analysis still promises to cause some variation in practice. Although the IBC exempts smaller structures, it does not yet acknowledge problems in modeling stiffness of code-allowed stucco and drywall shear wall systems, or how should diaphragm boundary conditions be modeled where there are interior shear wall lines. However, the code does provide some improved guidance for the determination of rigid versus flexible diaphragms. ASCE 7-05 now classifies diaphragms as “Semi-Rigid” unless proven as flexible or rigid. This definition is deemed beneficial because it acknowledge that engineering judgment be applied to the spurious results that are occasionally produced in either flexible or rigid analyses.

ASCE 7-05 specifically allows one- and two-family residential light-frame construction of any height to be idealized as flexible, even with a nonstructural concrete floor topping.

Interestingly, the IBC has an optional set of provisions: Per 1613.6.1, the designer may elect to assume a flexible diaphragm for any light-frame occupancies with the following restrictions:

1) Maximum non-structural topping of 1.5 inches, and
2) Drift needs to be checked at each line (not just at the center of mass),
3) Shear walls must be light-frame with wood panels or steel sheets, and
4) Cantilever diaphragms must conform to the rigid diaphragm requirements of IBC 2305.2.5.

Note that item three disallows mixed-systems. Thus, comparing the ASCE 7-05 provisions to the IBC provisions, the question for wood-framed structures other than single family and duplex is, “do you have a mixed system or a floor topping.” Effectively, multi-family and commercial projects can have either a floor topping or a mixed-system (but not both) and qualify for the flexible diaphragm assumption. Therefore, since it is typical to have a floor topping and a mixed system in commercial and multi-family projects, rigid diaphragm analysis will be common in multi-story structures.

ASCE 7-05 12.2.5. System Specific Requirements. Steel frames are common in all styles of wood-frame construction. In particular, it is common to have a one or two-story steel frame in multi-story apartment, condominium and hotel projects where there is a lobby, recreation area or similar "non-stacking" amenity area. ASCE 7-05 addresses frames in Table 12.2-1, which indicates that OMF and IMF are not allowed in SDC D-F, however a number of complex exceptions are referenced. In particular, sections 12.2.5.6 and 12.2.5.7 allow steel OMF and IMF in SDC D or E with a complex set of height, story and weight restrictions. These restrictions appear suited to metal buildings rather than to wood construction and uniform interpretation may be a challenge to wood framed structures. In our opinion, there are many situations where an IMF could be appropriate and should qualify for the exception. Note that the height of the frame is regulated by these specific requirements, rather than the building height regulated by Table 12.2-1.

ASCE 7-05 12.2.3.2. R, C_d, and Ω_o Values for Horizontal Combinations. Mixed systems such as steel frames or cantilever columns with wood framed shear walls generally require the entire direction of analysis to use the lowest R value. An exception permits use of the lowest R value on each independent line of resistance, provided the building does not exceed 2 stories in height and is of light-frame construction, or has flexible diaphragms.

ASCE 7-05 12.3.4. Redundancy. The Redundancy Factor provisions are significantly revised. In SDC D-F the Redundancy Factor, ρ, is taken as 1.3 unless one of two conditions is met, whereby ρ is taken as 1.0. The first condition only applies to regular structures. The second condition should be met in most light-frame projects and the procedure is difficult to automate but it can be expressed as three tests: A) The code requires that all the slender shear walls or piers, defined as those wall segments with aspect ratio over 1.0, be individually "removed". If the story strength "loss" is less than 33% with any one slender wall segment removed, then ρ =1.0. B) Only the lower floors that carry more than 35% of the base shear need to be checked, and C) the procedure does not apply to any buildings with an extreme torsional irregularity. By inspection a building would need to have very few shear walls to have one shear wall removal result in a 33% drop in story strength. Therefore, most light-frame residential projects will have the redundancy factor taken as 1.0.

ASCE 7-05 12.8.1.3 Maximum S_s value for Determination of C_s. The base shear may be calculated with S_s limited to 1.5 if the structure is regular, less than 6 stories and where the period is 0.5 seconds or less. Be aware that any of the irregularities, including angled shear walls and re-entrant corners voids this exception.

ASCE 7-05 12.14. Simplified Alternative Structural Design Criteria for Simple-Bearing Wall or Building Frame Systems. The primary benefits of the simplified method are 1) uniform rather than "inverted triangle" vertical distribution of lateral loads, and 2) no requirement for checking story drift. On the other hand, there are many limitations including a three story height limitation, higher seismic forces, special limitations on irregularities and the structure cannot be located on soft soil. Unfortunately the benefit of the provisions is significantly diminished by the efforts of checking the limitations and overall conservatism. Therefore we do not expect to see these provisions incorporated into production design.

IBC 2305. General. The shear wall and deflection formulas have been revised and corrected. Compared to older formulas the results are similar if using the new coefficient values for shear modulus and nail slip.

SDPWS T4.2.4. Maximum Diaphragm Aspect Ratios. Horizontal or sloped diaphragms of unblocked wood structural panels are limited to 3:1. With blocking the aspect ratio is extended to 4:1.

4) Prescriptive Wood Code Issues

Fortunately most of the new prescriptive code issues can be handled through simple changes to your specification sheets. The following sections will require attention as you transition to the 2006 IBC:

IBC 1707.3 Structural Wood. Periodic special inspection is required in SDC C-F of the wood lateral force-resisting system. Inspection is not required if shear walls have nail spacing more than 4 inches on center. Our opinion is that this provision be based upon the nail spacing required by design and not triggered by an extra nail. Also, where only portions of the building have shear walls with 4 inch nailing, the amount of diaphragm and collectors required to be inspected should be reasonably determined.

IBC 2304 Truss Placement Diagrams. The IBC has some code language allowing truss placement drawings to be prepared without the direct supervision of a registered design professional. The designer is defined as the "individual or *organization* responsible for the design of trusses." Based upon our field experience, we believe it is very beneficial to require an engineer in responsible charge of the roof truss system where the roof is relatively complex (i.e. more complex than a tract home). Also, note that state laws regulating the practice of engineering may restrict the sizing of hangers, setting of cambers and other engineered items generally shown on the layout drawing to registrants. Therefore we recommend engineers and building officials consider when to add a note requiring the truss placement drawings be engineered to work as a system and appropriately sealed by a professional.

IBC 2304.3.3. Shrinkage. This interesting UBC requirement is new to the IBC: In buildings over two stories, wood shrinkage and the effect on roof drainage must be considered. Fortunately, the shrinkage coefficients are found in many references and it is a simple calculation. Note that shrinkage parallel to grain is generally negligible and that the vertical studs do not contribute much.

Since shrinkage is governed by horizontal lumber, note that the variance of shrinkage between two parallel walls is a function of the number separate horizontal wood pieces stacked in each wall. In other words, the probability of having all high-shrinkage pieces stacked together in the same bearing wall, and getting all the good pieces in the opposite bearing wall is low. Therefore, with 4 or 5 horizontal wood members typical to a platform framed floor-ceiling assembly, the issue of differential shrinkage *decreases* with every added story. Accordingly we have yet to find a roof ponding problem due to differential shrinkage of the wood members on a building of 3 or more stories.

IBC 2304.9.2 Sheathing Fasteners. Nails are required to be driven flush to sheathing. Fortunately the American Plywood Association has some useful guidelines listed in APA form TT-012A (Jan 2007): first, no reduction in shear capacity is recommended if the nails are less than 1/16 inch overdriven in dry conditions. Second, if the "overdriven" condition is a result of the wood sheathing swelling after being properly installed, no reduction is recommend. Third, if less than 20% of the perimeter nails are overdriven no more than 1/16 inch, no reduction is recommended. Fourth, you can add 1 new nail for every 2 overdriven nails if more than 20% of the nails are no more than 1/16 inch overdriven or if any nails are over 1/8 inch overdriven. Where extra nails don't fit it may be possible to use staples. The APA also cautions that nails should be properly staggered with the stagger occurring across the grain.

IBC 2304.9.5 Fasteners in preservative-treated and fire-retardant-treated wood are required to be hot-dip galvanized. A proposed exception to allow electro-plated nails fasteners is being evaluated but does not currently apply to nails. Please be aware some nail guns will not shoot more than a 0.131 inch diameter hot-dipped galvanized nail. Check with your contractor the installation method for 16d common hot-dipped nails.

IBC 2304.11.2.2 Wood supported by exterior foundation walls. The standard exterior wall detail showing 6 inches separation between the sill plate and exposed earth has the dimension increased to 8 inches. Sill plates in contact with concrete are required to be pressure treated or naturally durable, therefore the measurement could be taken to the bottom of the untreated stud, however the practice has been to dimension from finish grade to top of concrete slab.

IBC 2304.11.6 Termite Protection. Wood floor framing shall be pressure-treated or termite resistant in areas of "very heavy Infestation". IBC Figure 2603.8 indicates California, east Texas, Louisiana, Mississippi, Alabama, Florida, Georgia and South Carolina are "very heavy". This code section appears reasonable only where joists are over exposed earth.

IBC 2304.12 Long Term Loading. Concrete thicker than 4 inches can be supported by wood framing if long term deflection is considered. The NDS appendix F indicates that creep will result in 1.5 to 2.0 times the initial deflection. Wood shear walls can resist seismic forces generated by horizontal concrete but not from concrete or masonry walls per IBC 2305.1.5.

IBC 2305.3.11 Sill Plate Size and Anchorage in Seismic Design Category D, E, or F. This section requires 3"x3"x0.229" sill plate washers with an "optional" diagonally slotted hole at anchor bolts. Further, the bolt values have been dramatically reduced on edge conditions per ACI appendix D. One option for designers is to use a strap type mudsill anchor, however it should be cautioned that sheathing typically must be on the exterior and double sided shear walls may not be compatible. Corrosion also continues to be a concern with all types of sill anchorage.

IBC 2508.5 Horizontal Gypsum Board Diaphragm Ceilings. A new provision exists that allows gypsum diaphragms, however the shear values are too low to be of much practical use in SDC C or higher.

Finally, a major change was made to the UBC in the area of Regular versus Irregular buildings. In the 1997 UBC, irregularity definitions triggered dynamic analysis and buildings less than 65 feet were exempt from dynamic analysis. In ASCE 7-05, there are implications for having in irregular light-frame structure less than 65 feet tall, which are summarized below:

12.3.3.1 Prohibited Irregularities
On SDC E or F
Extreme torsional irregularity not allowed.
Stiffness-Extreme Soft Story irregularity not allowed.
Weak story or extreme weak story irregularity not allowed.
On SDC D
Extreme Weak Story irregularity not allowed.

12.3.3.2 Extreme weak Stories prohibited if building is > two stories or > 30’ in height. Exception: if weak story is capable of resisting Ωo times seismic force.

12.3.3.4 Requires 25% force increase in SDC D-F on chords, collectors and diaphragm connections to the vertical elements, if structure has out-of-plane, in-plane, re-entrant corner, or diaphragm discontinuity, etc. Exception: if Ωo factor used per 12.10.2.1.

12.3.4.2 Irregularity interferes with options available for reducing Redundancy factor, ρ, from 1.3 to 1.0.

12.5.3 On SDC C-F with non parallel system, the design must consider orthogonal effects in calculations.

12.7.3 Torsion, out-of-plane or non-parallel irregularity shall be analyzed with “3-D” representation.

12.7.4 Rigid elements shall not inhibit performance of moment frames and the affect of infill elements are to be considered in determining irregularity.

12.8.1.3 Cap on S_s of 1.5 seconds if structure is regular and < 5 stories.

12.8.4.3 Light frame structures exempt from amplification of torsional irregularity.

12.12.1 Consider added drift due to torsional irregularity (where occurs).

12.14.1.1.11 No In-Plane or out-of-plane irregularity in system permitted in Simplified analysis.
Exception for light frame , < 3 story with 2.5x overturning forces applied.

There are many other topics that would be included if space permitted. We hope the information presented above aids your transition process.