Wood Construction: Commonly Missed Items and Misconceptions

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About the speaker

- Structural intern at 3 different firms 2013-2015
- B.S. Civil Engineering –April 2015
- Licensed as a Civil Engineer in UT and CA
- Currently Project Engineer at Vector Structural Engineers
- Work with timber structures including; residential, commercial, heavy timber, and mixed materials.
The Vegas experience

Outline

- Topics:
  - Gravity Topics
  - Floor vibrations
  - Lateral topics
    - Chords and collectors
    - Common mistakes with perforated and FTAO walls
  - Construction Topics
    - Panelized construction details
Gravity Topics

Vibration issues

https://www.youtube.com/watch?v=X50qwgBuXpY
Case Study

- Large residence with suspended stairs
- Well within code deflection limits

Case study cont.
Thinking about floor vibrations, even in residential applications, is the kind of work that can set an engineer apart from the crowd. Clients may feel like their design is not adequate if there is easily perceived vibrations. There are a few resources for assessing the permissible floor vibrations. The paper by Woeste and Dolan addresses the issue from the standpoint of simply the natural frequency of the floor beams. They recommend avoiding frequencies in the “sensitive range” for humans (7-10 Hz).

It is possible that combined framing elements who individually have acceptable vibrations may combine to form a system with unacceptable vibrations. The simplistic view of looking at only the natural frequencies of the system may fall short in these circumstances, and the approach of AISC DG 11 may be more appropriate in these cases.
**Simpler than you think**

\[ f_n = 0.18 \frac{\sqrt{g}}{s} \]

- Determine peak acceleration:
  
  \[ \frac{\Delta p}{g} = \frac{P_o e^{-0.35f_n}}{\beta V} \]

- Compare to suggested limits

![Graph showing peak acceleration vs. frequency](image)

**Avoiding vibration pitfalls**

- Have a conversation with the client up-front. Understanding their expectations is critical. You can have an idea almost immediately if there will likely be problems.

- Reducing amount of deflection is helpful, but even L/800 is often not enough.

- Keep floor joists continuous wherever possible.
  - Intuitively, a beam or joist that is fixed at one end will be stiffer
  - Anecdotal evidence suggests the same
  - 4.1.2 AISC DG11 allows for 50% increase in effective panel weight when beams are continuous at support.

![Diagram showing more susceptible vs. least susceptible](image)

**Checking vibration isn’t actually a big deal, because you already know almost all the parameters**

**Things you already have calculated:** Effective weight, Dead load deflection

Damping values and forcing function amplitude are straight out of tables, usually the same for all timber projects. Be careful to remember the chart axis is in percent.

Be careful with dead loads, as we consistently overestimate dead loads in design, but in this case, it is non-conservative.
Lateral Topics

Chord and collector tips

- Chord and collector forces are critical to a reliable seismic force resisting system, but we may not always give them detailed attention in timber design.
- Good rules for chords and collectors:
  1. Utilize total line load and chord force relationship
  2. Pick a standard strap
  3. Resolve the discontinuities
  4. Make sure it really gets there
Line load and chord force relationship

- For a simple flexible diaphragm, there's a fixed ratio where the chord force exceeds the total load on a shearwall line.

\[
V = \frac{wt}{2} \quad TC = \frac{wl^2}{8h}
\]

\[
V = TC
\]

\[
\frac{wl}{2} = \frac{wl^2}{8h}
\]

\[
1 \quad \frac{l}{2} = \frac{8h}{l}
\]

\[
4h = l
\]

- The aspect ratio at which the chord force exceeds the force in the shearwall, is also the maximum limit allowed per code.
- Is your force in the shearwalls less than your chord splice capacity? Congratulations, you have verified your chord works.
- Once the total shear resisted on a line exceeds splice capacity, then you need to start more detailed calculations.

Pick a standard strap and splice

- Every office should have a standard inexpensive strap that is designated for all discontinuities.
- Know by heart the capacity of your typical top plate splice.
- Make the two roughly the same, so that your brain only needs one trigger.

The fastest way to deal with chords and collectors is to ensure that there is a known threshold where your design becomes non-standard. In our office, the ST6224 strap is typical for all drag connections. Its capacity of 2535 lbs is almost identical to the 2576 lbs capacity of our typical top plate splice. Whenever we are dealing with total line forces lower than that, we know that our chords and collectors are adequate. Any special consideration doesn’t need to happen until that value is exceeded.
There are many discontinuities that can cause poor lateral performance for a timber structure. I will touch on a few common examples here.

The wall jog is very common, and hopefully is as simple as strapping to a beam or girder truss, but sometimes the configuration of the framing won’t allow that. In those cases we should provide continuity straps that can transfer the diaphragm forces, or provide additional shearwalls.

I’m certain that this crowd doesn’t need the importance of load path reiterated. We can’t allow correct structural detailing to be ignored or omitted. Probably the biggest offender in our area is blocking. Framers don’t understand what it does, and architects want to vent through it. We must have the diaphragm connection to the wall or we’ve wasted our time.

I don’t think it’s a common problem among experienced engineers, but we often interview newer engineers with some timber experience who have trouble detailing the correct load transfer. As engineers who know, we should consider it part of our duty to make sure the concepts of shear transfer are taught and understood by those we use to perform calculations.
Make sure it really gets there

- Don’t look at the shearwall plans in isolation. Without the framing plans the load path doesn’t happen.

It seems like a common mistake we engineers make on wood construction is being in a hurry, and forgetting the vital detailing that brings the loads to the shearwalls. To make sure that detailing happens, the shearwall/floor plans cannot be viewed in isolation from the framing plans.

Common perforated wall misconceptions

- Perforated walls have fewer requirements than other methods and enable flexibility in using fewer holdowns
- Special requirements for uniform uplift restraint and perscriptively increased demands
What is Co doing?

\[
v_{\text{max}} = \frac{V}{c_0 \sum L_i}
\]

\[
T = C = \frac{V h}{c_0 \sum L_i}
\]

\[
V_{\text{wall}} = V c_0 \sum L_i
\]

\[
L_{\text{max}} = \frac{V}{c_0 \sum L_i}
\]

The perforated wall reduction factor Co is a prescriptive penalty based on testing. Testing shows that the modification to the shear demand ensures the entire wall is designed for the largest concentrated shears that occurs at the openings. Holdown tension is likewise increased to the worst case that was seen to occur in testing.

The additional uplift requirements are required to provide some kind of restraint at wall segments that will have tension away from the holdowns. They must have a uniformly distributed uplift restraint equal to Vmax. This uplift capacity may be provided by plate washers on foundation anchor bolts or by uniformly spaced straps.

Simple mechanics would tell us that perforated shearwalls have a problem, if there are only holdowns installed at the outside ends and not the interior. I have had plans examiners argue that they are required at door openings at perforated walls. They're not entirely wrong, but more research into the code provisions and the testing involved show us that those forces are being resolved in an alternate requirement.
Common misconceptions about FTAO walls

- SDPWS 4.3.5.2 “Design of force transfer shall be based on a rational analysis”
- What methods are rational?
- The APA clarified in 2011 with the release of Form No. M410: Evaluation of Force Transfer Around Openings – Experimental and Analytical Findings

Evaluation of Force Transfer Around Openings – Experimental and Analytical Findings
The less rational (or less ideal)

Sometimes analysis of FTAO at doors uses the pier height not only to calculate the aspect ratio and its penalties, but also the overturning moment. Reduced overturning height may or may not be accurate depending on the strength of the assembly above the opening. The converse of this would be to design the wall with additional holdowns at the wall interior and treating the wall like two independent piers. Certainly this design would have strength equal to or greater than the calculated design, but the addition of more holdowns would not make you popular with owners or contractors.

The ideal (according to the AWC)

- Testing by the AWC found that the most accurate rational analysis for FTAO walls is the Diekmann method.
- The wall is purposes of overturning, internal forces are resolved.
- treated as a rigid body for While computationally more intensive, the results are far more accurate.

Predicted strap forces for the Diekmann technique varied from 18% conservative to 87% conservative in testing. Cantilever beam method was 64% conservative at the lowest and 963% at the highest. Drag strut technique underestimated bottom strap forces in all but one configuration. For examples and a complete walkthrough see Design of Wood Structures—ASD/LRFD by Donald E. Breyer, P.E.
The force transfer wall diagram in the code lists the "wall pier height" in a somewhat ambiguous way, that has led to at least one situation where an engineer interpreted the code to allow for a very interesting design. The wall height used in overturning calculations was the height of the opening, and the resisting couple stretched across the length of the wall as a whole. Though I don't work for the AWC, I'm fairly certain this is not their intent.
Panelized construction

- All wall framing is panelized construction at some level. Nobody frames the wall standing up straight. The key difference is when and how the sheathing and top plates are applied.
- Both top plates can be applied in the factory, which is problematic.
- Sheathing is applied in factory also. This makes the shearwall panels discontinuous.

The key differences in panelized construction from traditional really have to do with the sequence and location of the fabrication. All walls are framed in a panelized manner, on the jobsite the wall is framed as a single panel that is then erected, but in the plant the panel sizes are limited by transportation concerns. A good panel factory has practices that minimize the work on the engineer of record, a lower quality one may not. There is rarely any problem with panel factories providing the gravity systems as designed, but the lateral concerns may be overlooked. The key items are top plates and shearwall splices. We will need to coordinate with the contractor regarding when and how those take place.

How do we appropriately coordinate for panelized construction?

- Panelized walls should be shipped with a single top plate, and the top plate splice applied in field.
- Alternatively, provide a strap alternative for the top plate splice.
- Discontinuous shearwall panels shall be joined per SDPWS 4.3.6.1.1. This can be provided as standard language in the shearwall schedule.