BEN L. SCHMID
CONSULTING STRUCTURAL ENGINEER
203 PEARL AVENUE
BALBOA ISLAND CALIFORNIA 92662
BUS: (949) 673-6339 FAX: (949) 673-6047

September 14, 2009

Subject: ICC –ES AC 322-1009-R1 Hearing – October 6, 2009

Paragraph 4 2 2 6 — Initial Pretension of Overturning Restraint: ----) Pretension shall not exceed ~~500 pounds (2,225 N)~~ 1200 pounds (5,400N).

Reason: AC 269 has a pretension force of 1200 pounds (2,225N), Par. 4 2 6 Since the sheathing is similar in use, the pretension force should be the same.

Paragraph 4 4 1 1 — Drift Limit (Seismic): The ASD load which satisfies the draft limit of ASCE 7, Section 12 8 6 shall be computed as follows:

Comment: Section 12 8 6 Story Drift Determination refers to Fig. 12 8-2 and Equation 12 8-15. The frame is a rigid frame, fixed at the base. The header at story 1 of 2 is not described It is difficult to imagine a frame, defined in ASCE 7, Para 11 2 as being CBF, EBF, IMF, OMF or SMF , which frame is being used (with its special qualifications) Fig. 12 8-2 shows a fixed base, which needs to be shows how 100% fixity is obtained

Paragraph 5 0 — Conditions Of Use: Establishment of Seismic Design Coefficients

----, the provisions of Appendix A shall be used to provide equivalency.

Comment: Appendix A is totally different in nomenclature than Paragraphs 5.2.1.2, 5.2 2, 5 2 3 and 5.2 4 or AC 130

As examples:

Δ_U AC 130 $\geq \cancel{11}\ \underline{6}$
$\Delta_{0\ 8VP}$AC322

$\Delta_U \geq 0.028$ 0.028H AC 130
$\Delta_{0\ 8VP} \geq 2.8\%\ h_x$ AC322

$2.5 \leq \frac{P_{PEAK}}{P_{PSD}} \leq 5.0$ AC 130

$2.5 \leq \frac{V_P}{V_{ASD}} \leq 5.0$ AC 322

Comment: A continuing debate is going on between manufacturers and seismology committees of Structural Engineers Associations in comparing the R factor listed in ASCE/SEI Table 12 2-1, Item A 14 (light-framed walls sheathed with wood structural panels for rated shear resistance), with manufactured narrow panels. There is a current recognition of deflection control in matching panels of different sheathing, steel frames, etc as used in wood structures With acknowledgement and awaiting final reports of current testing an R factor or 4 for IBC is recommended for inclusion in AC120.

Paragraph 4 (b) ----- Equal to greater than ~~11~~ <u>6</u>.

Comment: 20 years of panel testing Δ_{SUS} versus $\Delta_{YLS/1.4}$.

Paragraph 4 (c) A displacement corresponding to ~~6~~ 2. Percent of the wall height

Comment: A deflection of 6 inches for an 8 ft. high panel and frame may be achievable for narrow panels, but in reasonable deflection control, 2 inches is a realistic drift Note that ASCE 7, Table 12.12-1 allowable story drift has a notation that "with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts " CUREE testing reports on panels, two story houses and a 3 story tuck-under apartment building, during the past 8 years, have described an reiterated that wood framing structures become un-inhabitable when drift exceed 1½ percent per floor ICC and Manufacturers should recognize the damage threat placed upon an unsuspecting owner or tenant Major discussions are requested by the writer on these un-acknowledged testing results.

Appendix B

Comment: The two illustrations do not truly provide design protocol. Typically, Shear Walls and Shear Panels have a T (tension) force for the hold-down and a compression force on the far post For very narrow manufactured panels, it should be up to the supplier to provide a stress block from testing. The Triangular Stress Block spears suitable for a base concrete grade beam. The Rectangular Stress Block appears suitable for a wood supporting beam.

Respectfully submitted,

Ben L. Schmid
Fellow, SEAOC

September 17, 2009

Kurt Stochlia, P.E.
Technical Director
ICC Evaluation Service
5360 Workman Mill Road
Whittier, CA 90601

<u>RE: Proposed Revisions to Acceptance Criteria for Prefabricated, Cold-formed, Steel Lateral-Force-Resisting Vertical Assemblies, Subject AC 322-1009-R1</u>

Dear Mr. Stochlia:

Thank you for the opportunity to comment on the proposed revisions to AC 322. Our staff has reviewed the criteria and we have some concerns relative to the proposed methodology:

<u>Comment #1 - Section 4.1:</u> We agree with the concerns expressed in your comment and feel the most important boundary member connection is via the anchor bolts. To be consistent with the benchmark wood structural panel wall system, the prefabricated product should not rely upon anchor bolt stretch as a primary movement mechanism. One solution might be to add something similar to the following as Section 1.2.1:

> *1.2.1 This criteria applies to prefabricated wood shear-resisting wall assemblies that yield within the tested assembly during a cyclic in-plane shearwall test. Yielding of the anchor bolts or panel-to-load beam connections shall not be used as mechanisms for assembly yield.*

<u>Comment #2:</u> The inclusion of provisions for equivalency to prescriptive braced walls permitted by the IBC and IRC is a complex question and likely will take further study. We are willing to work with a task group to propose changes for an upcoming ICC-ES hearing.

<u>Comment #3 - Section 4.1.1:</u> Your comment questions whether failure modes should be consistent between replications of a given wall configuration for the results to be averaged. It should be noted that wood structural panel/wood stud benchmark shearwalls tested with the same configuration often experience multiple failure modes, including: sheathing edge tearout, nail withdrawal, fastener head pull-through, or some combination of the above. Given that requiring the same failure mode for all walls of a given configuration would be problematic for the code-listed benchmark system, we do not agree with this proposal. However, we do agree that a framing failure in any racking test specimen that limits the vertical load capability should signal a need for further investigation. Such a failure mode observed in any panel test would place the panel outside the scope defined by Section 1.2 for recognition.

P.O. Box 8449, Boise, Idaho 83707❖Phone (208) 364-3600❖Fax (208) 364-1436

Comment #4 - Section 4.2.2.3: We agree with the concerns expressed in this comment. A significant amount of detail needs to be added to AC 322 to address recognition in multi-story applications. That detail should standardize basic elements like: the definitions and limitations of the systems to be recognized, boundary conditions to be used for testing, and the allowable load derivations. We are willing to work with a task group to propose changes for an upcoming ICC-ES hearing.

Comment #5 - Proposed Section 4.2.2.4: We feel the proposal is written in a way that could easily be abused and doesn't provide enough detail. It is not clear if the test is conducted on a beam that is rigidly restrained with the deflection of the beam added analytically, or if the beam is spanning some distance with the beam deflection included in the testing measurements.

- If the beam is rigidly restrained, the fastener properties of the beam are probably more relevant than its strength and stiffness, as the failure mechanism should be initiated through this connection.

- If the beam is spanning a distance, testing it with a minimum strength beam could be misleading. When establishing a design load for the system commonly controlled by deflection, the rotation of the beam could be used to mask the lack of ductility in a prefabricated product. Also, the beam will undoubtedly see other loads in application that are not part of the wall test that will impact the measured strength and stiffness of the assembly relative to the tested condition. These loads could combine with the panel overstrength to cause a brittle failure of the beam if it is not properly designed for all imposed loads.

- The clause that allows use of raised floor data in lieu of beam-mounted test data may not be justified in all cases. Raised floor details are typically attached through a floor platform to a rigid foundation. Systems attached to a second story beam typically are not.

To address this situation, we suggest the following alternative wording for Section 4.2.2.4 and an added Section 7.X:

> *4.2.2.4 Wood Beam: Panels intended to be installed on a wood beam shall be tested by placing the panel on a beam with fastener properties and an attachment configuration that defines the lower bound permitted in application. The beam element shall be restrained from rotation and lateral displacement in the test.*
>
> *7.X When a prefabricated wood shear panel is placed upon a beam in application, the assumed lateral stiffness of the combined assembly used in design shall include wall deformation induced by beam deflection under all vertical applied loads in the applicable load combination. Where consideration of earthquake loading is required by IRC Section R301.2.2 or IBC Section 1613.1, the design of the beam shall consider the panel overstrength as required by Section 12.3.3.3 or Section 12.14.1.1 (11) of ASCE 7-05.*

Comment #6 - Proposed Section 4.2.2.5: We feel the proposed changes are vague. They also seem to conflict with the first three sentences of the section. What stiffness is being considered? Axial stiffness of the wall? Vertical stiffness of the floor framing members? Lateral stiffness of the wall and framing members? We suspect that a detailed clarification of

P.O. Box 8449, Boise, Idaho 83707❖Phone (208) 364-3600❖Fax (208) 364-1436

the recognition requirements for multi-story applications is needed. We are willing to work with a task group to propose changes for an upcoming ICC-ES hearing. In the interim, please consider deleting the proposed modifications in favor of a more comprehensive solution.

Proposed changes to Section 4.2.2.11: We feel the proposed requirement for shrinkage compensator devices is not necessary and should be deleted based on the following rationale:

- The requirements on the anchor bolt tightness currently included in Section 4.2.2.6 of AC322 and Section 6.2.3 of ASTM E2126 have already been imposed specifically to address this issue.

- Similar requirements do not exist for site-built shearwall construction. Shrinkage is not included in the allowable load derivations for site-built shearwalls and should not be imposed upon prefabricated products.

- Current literature suggests that the stiffness and drift of light-frame walls tested without finishes and non-structural components underestimates the actual stiffness that can be expected in application. This is also true with prefabricated panels. Adding this further penalty to a design load derived from a bare wall test seems unreasonable.

We ask that the proposed wording be removed. At a minimum, please consider adding the following sentence applying to assemblies constructed with engineered wood product platforms that will not experience the perceived shrinkage effects:

> *Use of engineered wood products manufactured with a moisture content less than 10% to frame the floor platform is considered an acceptable means to mitigate wood shrinkage effects.*

Comment #7 – Sections 4.3.1.4 & 4.4.1.4: This comment suggests that the allowable design loads for the AC322 walls should be established by calculation. Our interpretation of Section 4.4.1 and 4.3.1 is that the panel design capacity should be governed by the lower of the tested capacity or the calculation limit. Given this, we do not believe that it is necessary to run a hybrid calculation that combines the empirical and calculated data. However, we do hope that calculations being reviewed by ICC-ES include application of the assumed overstrength factor to the axial tension/compression chord loads. That magnification is required by Section C5.1.2 of the AISI Lateral Standard to address compression chord crumpling for cold-formed steel site-built shearwall systems that use R=6.5. The same failure mode is a concern for cold-formed steel pre-fabricated products and should be part of the calculation procedure.

Comment #8 – Section 4.5: At this time, we do not agree that combined vertical and lateral load testing should be mandatory. We have some experience in this area and have found that combined vertical/lateral load testing is complicated and poorly defined. We aren't convinced that the few labs that claim to have this capability can accurately apply a consistent vertical load with a reasonable tolerance throughout a lateral load cycle. Our preference would be to limit recognition to light-frame bearing wall products that do not rely upon buckling of the vertical load elements to achieve ductility and drift capacity in a lateral load-only test. This would be consistent with the benchmark light-frame wood panel/stud shearwall system. However, in the absence of an established test standard, we think the existing AC322 Section 4.5 combined with a calculation check that includes overturning overstrength represents a reasonable compromise for panels that rely upon axial buckling to achieve lateral performance. It should also be noted that a panel doesn't necessarily need to be recognized to have vertical load capabilities.

Proposed Section 7.6: LRFD to ASD Conversions: We do not understand the LRFD-to-ASD conversion factor of 1.5 proposed in Section 7.6. This seems to be inconsistent with both the IBC and ASCE-07 load provisions. We believe the appropriate conversions should be 1.4 for seismic load combinations and 1.6 for wind combinations. Please consider replacing the second to last sentence of the proposed paragraph with:

> *Strength level capacities for wind and seismic design may be converted to allowable stress design level capacities by dividing by factors of 1.4 and 1.6, respectively.*

Proposed Section 7.6: EEEP Level Loads for Anchorage Design: We do not agree with the last sentence proposed for Section 7.6 that suggests designing the foundation for the panel's ASTM E2126 EEEP yield load as a way to permit the panel to serve as the ductile "attachment" required by ACI 318 Appendix D. This is breaking new ground and there are a number of problems with this proposal:

- The intent of ACI 318 Section D3.3.5 is to require the "attachment" to yield, not the wall. This provision does not meet that intent. We agree with the wording shown in staff's comment #10 that the attachment should be shown as yielding in lieu of yielding occurring elsewhere in the panel.

- Using the ASTM E2126 EEEP panel yield load to design the foundation creates a larger disparity with what is permitted for site-built construction where there are no similar provisions. With regard to prefabricated products, it creates a disconnect between foundation requirements in engineered construction versus what is typically used for prescriptive construction. These disparities do not need to be amplified by making engineered construction overly conservative. To do so will only encourage prescriptive uses of these products which may not be an improvement in overall building performance in the field.

- The EEEP yield loads are often 75-90% of the panel's ULTIMATE capacity. Tying the concrete strength DESIGN load to this level ignores the relative overstrength capabilities of the concrete foundation. The specified steel and concrete strengths are lower fractiles. The foundation design process also induces further conservatism (i.e. phi factors, assumption of cracked concrete, etc). Requiring the foundation to be designed for the EEEP yield load of the panel is unnecessary conservative, especially when one considers that the EEEP yield load does not even reside on the backbone curve. To date, the E2126 yield point has been used primarily as a uniform means to determine a yield displacement for consistent calculation of ductility for a non-linear load-displacement curve. To our knowledge, this is the first time that it has been tied to a derivation of a design strength requirement and we do not think it is appropriate.

- The proposed provision does not work well if the yield load measured in the wall test is based on stretching of the anchor bolt. Given that anchors have yield strengths in excess of their design value and an additional phi factor is applied in design, a stronger anchor will now be required in design than was used in the test to correspond to the EEEP yield load. The use of this stronger anchor will change panel

P.O. Box 8449, Boise, Idaho 83707❖Phone (208) 364-3600❖Fax (208) 364-1436

performance and will fail to ensure that the anchor/panel combination will experience a ductile yield prior to concrete failure.

Based on the above, we ask that the last sentence proposed for Section 7.6 be deleted and a new section added per our comments below.

Proposed Section 7.X: Design Capacity vs. Assumed Demand: The proposed Section 7.6 highlights a significant issue that occurs in the marketplace. Prefabricated panels are often chosen as much for their geometry as for their published design load. It is not uncommon for building designers to proportion the anchorage, and attachment to the surrounding construction, based upon the load they want the panel to carry instead of the full published design load. Significantly scaling down the foundation anchorage and top connection not only has the potential to undermine the intended panel attributes (i.e. ductility, drift capacity, etc), it also does not protect against the situation where the panel attracts loads beyond those accounted for in the building design. We believe the following section should also be added as part of these revisions:

> ***Section 7.X*** *Where consideration of earthquake loading is required by IRC Section R301.2.2 or IBC Section 1613.1, the panel anchorage and connections to the surrounding construction shall be designed for the full published seismic design capacity of the panel. Reductions in these attachments to reflect the demand on the panel in an individual application are not permitted.*

Thank you for consideration of these comments. If you have any questions regarding these comments please don't hesitate to contact me at 208-429-3715 or at Daniel.Cheney2@Weyerhaeuser.com

Sincerely,

Daniel W. Cheney (sent via e-mail)

Daniel W. Cheney, P.E.
Director of Codes and Product Acceptance

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

16 September 2009

Mr. Kurt Stochlia, P.E.
Technical Director
ICC Evaluation Service, Inc.
Los Angeles Business/Regional Office
5360 Workman Mill Road
Whittier, CA 90601

Re: Proposed Revisions to the Acceptance Criteria for Prefabricated, Cold-formed, Steel Lateral Force-Resisting Vertical Assemblies, Subject AC322-1009-R1

Dear Kurt:

This letter is offered in response to your 1 September 2009 memorandum to interested parties on the proposed acceptance criteria revision. I offer my comments below as one who was a member of the joint ad hoc committee that assisted ICC-ES with development of the present acceptance criteria.

In general, I support the proposed updates of the AC 322 criteria. I do however have some comments which I offer for your consideration:

1. **Proposed Section 4.2.2.3.1 – Wood Beams**. I disagree that assemblies intended to be installed on flexible elements, such as beams, should be tested in such configuration and that the supporting member in the test should be at the lower bound of the range permitted in the application. The effect of a flexible base in a test will be to reduce the amount of distortion that must be accommodated by deformation of the shear wall element itself. This can create the illusion of a shear wall product that is capable of greater deformation than it really is and could be quite unconservative. I believe that Simpson's intent here is to provide a "rated capacity" for walls that considers the deflection of the panel including its boundaries and attempts to ascertain that the story deflection under design load will not exceed the permissible drift under the code. I believe this approach is problematic, at best. A product supplier cannot hope to understand all of the possible boundary conditions that an engineer may incorporate into a design. If an engineer designs a structure such that there is a very flexible base beneath the panel, the engineer should not be permitted to rely on catalog load values for the panel that attempt to automatically include consideration of deflection. Instead, the engineer should be required to calculate the story drift, considering the deflection of the panel and the base. This would of course, require that the panel manufacturers provide a rated deflection of the panel at design loading, so that the engineer could perform this calculation.

   I recommend that the acceptance require testing of such assemblies on rigid bases, or alternatively, that any deformation of the test specimen resulting from flexibility at the base be mathematically removed from calculation of the shear panel's force-deformation behavior. Rather than attempting to account for base support flexibility in the test program, designs of buildings in which shear panels will be supported on beams or other flexible elements should account for this added flexibility in their calculation of wall stiffness and story drift, as discussed above.

2. **Proposed revision to Section 4.2.2.4**. I believe the proposed requirement to include in qualification testing, representation of non-strandard flexible mounting conditions is impractical. The manufacturer cannot possibly anticipate as part of a qualification program, the variety of uses engineers may come across on individual designs. Rather, the burden should be on engineers who elect to use the products in novel approaches to demonstrate through calculation that the differing flexibility and strength at the base proposed in their design is adequately accounted for. Please see my additional comments above, on :"wood beams" for more discussion of this issue.

3. **Proposed revision to Section 4.2.2.10**. I oppose a requirement to include calculations of drift associated with shrinkage. To be meaningful, any such calculation must consider the nonlinear effects of closure of gaps that are created by the shrinkage. This is an impossibly complex procedure for the design of typical structures. Further, there has been no demonstration that drift effects associated with drying shrinkage have lead to poor performance in the past.

4. **Proposed Section 4.4.1.5 Calculation Limits**. While I concur that the design of actual structures must include complete load paths with adequate strength to resist the required seismic forces, I do not believe it is practical to require such calculations as part of the qualification procedure for a proprietary shear product as the complete load path will not be known until a specific structure within which the product is to be placed has been conceived. If the intent of the proponent is to require that the design loads for panels include consideration of all possible limit states that are directly associated with the panel, such as its connection to building drag and collector lines, or connection to foundations, this is appropriate, but this requirement should be worded in a more direct and clear manner.

5. **Proposed Modifications to Section 7.4, renumbered as 7.6**. It is not clear why an LRFD to ASD strength conversion factor of 1.5 has been selected. The code more typically uses a value of 1.4 for this ratio for seismic applications. For wind applications, a value of 1.6 should be used to convert from ASD to LRFD, to be consistent with the load combinations contained in Section 1605 of IBC-2009 and Section 2.3 of ASCE 7-05.

In your memorandum to interested parties you raise several specific questions. I do wish to offer response to several of these questions as follows:

Question1: I believe you are asking if it should be permissible to allow anchorage strength to be the governing factor in setting the rated capacity for a shear panel product. I strongly oppose the concept of establishing the rated capacity for a shear panel product based on the strength or stiffness of an anchor in a test. This is because the product supplier will not have any control over the design of the anchors actually used by engineers on specific projects. It is conceivable that engineers will use higher capacity anchors than those used in the testing, or anchors that do not have the ductility of those used in the testing. In either case, the product would not perform as indicated by the test. Please note that my comments apply to yielding of anchor bolts, as opposed to yielding of the shear panel's attachment to the anchorage. Such attachments are wholly within the control of the panel supplier and can be designed and manufactured so as to assure consistent strength and ductility properties.

Question 3: I do not believe it is necessary to achieve consistent failure modes in the tests for a particular panel product. It is likely that when products of different aspect ratios are tested, different failure modes will occur. This should be acceptable, as long as the tests for each series panel passes the acceptance criteria. There is precedent for this in other qualification criteria. For example, the AISC 341-05 Appendix S criteria for prequalification of steel moment connections does not require that each test specimen fail in the same way, but rather that each specimen demonstrate acceptable behavior.

Question 4 – I do not believe it is practical to test these panel products in all possible configurations that an engineer may desire to use them in, for individual building designs. I believe that single story panel tests, with fixed bases should be adequate for all applications as long as the engineer is required to account for the boundary conditions affect on drift and overturning in their actual design.

Question 5 – There is no reason that an engineer should not be able to use these products on a steel beam. However, as noted in my response to the previous question, I don't think that it is necessary to test panels mounted on steel beams. Rather, the engineer who designs such a building should perform an analysis to determine that the steel beam has adequate strength, and that the flexibility of the steel beam does not result in unacceptable story drift.

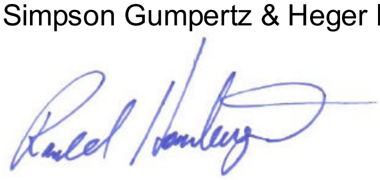
Question 8 – While it is clearly important to assure that panel products will retain their gravity load carrying capacity when subjected to lateral loading, it is very difficult to test such panels under combined vertical and gravity loads. One method that could be used to assess the ability of panels to support both vertical load and lateral load would be to reject tests in which failures occur that would result in loss of vertical load carrying capacity. Such failure modes could include buckling, crushing, or fracture of vertical load-carrying elements of the panel.

Question 10 –Please see my response No. 6, above. I believe that a value of 1.6 should be used for wind load resistance and a value of 1.4 for seismic load resistance.

Question 11 – I do not believe it is possible to provide stress block rules that will be appropriate for all possible panels. Some panel systems will resist overturning loads primarily through the actions of end posts while others will distribute overturning across the base of the panel. I believe that ICC-ES must permit the application of rational engineering mechanics for these stress blocks, but must perform adequate review to assure that this is reasonably done.

I regret that it will not be possible for me to offer oral testimony at the October hearings, however, I appreciate the opportunity to provide this comment. Please feel free to contact me, should you have any questions on the above.

Sincerely yours,
Simpson Gumpertz & Heger Inc.



Ronald O Hamburger, SE
Senior Principal
CA License S-2951 (Structural)
I:\SF\Projects\2007\077031-WCC1\Correspondence Out\l09roh99_AC322.doc