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Nineteenth International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, U.S.A, October 14 & 15 2008

Estimating the Effective Yield Strength of Cold-Formed Steel Light-Frame Shear Walls

Reynaud Serrette [1]

Abstract

Characterizing the seismic response of lateral force-resisting elements often requires an expression of the capability of these elements to sustain some portion of their peak strength at displacements well beyond their elastic limit. This paper presents an energy-based method for estimating the effective yield strength (elastic displacement limit) of cold-formed steel shear walls. The method considers the maximum usable wall displacement, the hsyteretic envelope response of a wall and the expected performance of the system in which the wall is used. The resulting effective yield strength limit is shown to be consistent with interpretations of yield strength in performance-based engineering design and provides a rational basis for comparing the elastic stiffness of alternative shear wall configurations.

Introduction

The seismic provisions in ASCE/SEI 7 (2005) limits the use of cold-formed steel (C-FS) light frame shear walls to bearing wall or building frame systems. For each system, seismic performance coefficients and factors (response modification coefficient, R, system overstrength coefficient, Ω_o, and deflection amplification factor, C_d) are specified depending on the sheathing material attached to the C-FS frame, the building height, use of the structure and the anticipated intensity of ground shaking. These coefficients and factors reflect

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the expectation that the dynamic characteristics, lateral resistance and energy dissipation capacity of the lateral-force resisting elements, when incorporated into the defined system, will result in some acceptable range of performance.

FEMA 450-2 (2004) notes that the basic objective of the current building code is the provision of “reasonable and prudent life-safety” at the code-level forces and lateral displacement limits. It is further noted that this objective “considers property damage as it relates to occupant safety for ordinary structures” and the expectation that for a major earthquake (2% chance of exceedance in 50 years) there is “some” margin of safety against collapse with associated structural damage that may not be economically reparable. Beyond the life-safety objective, however, the building code provides no explicit guidance for assessment of performance.

In response to the costly damage associated with wood light-frame construction in the 1994 Northridge earthquake (EERI 1996), a comprehensive 4-year woodframe research project was undertaken to “develop reliable and economical methods of improving woodframe building performance in earthquakes” (CUREE 2004a, 2004b). CUREE (2004a) describes the optimal performance of lateral-force resisting elements in wood light-frame construction as behavior that can “provide sufficient stiffness and high yield strength to survive a minor earthquake with minimal or no damage, and reparable structural damage and limited non-structural damage in a moderate earthquake.” Thus, at the element level, it appears CUREE associates the yield strength limit with “minimal to no damage.” Even though the term “yield strength” is used, CUREE (2004b) remarks that the notion of a defined yield strength in wood shear walls may not be appropriate due to the early onset of inelastic behavior in these elements. However, the notion of yield strength in the context of minimal to no damage of an element in an earthquake may be a useful analysis and design parameter.

SEAOC (1999) presented a set of “Tentative Guidelines for Performance-Based Seismic Engineering.” These guidelines identified five different system structural performance (SP) levels. For each SP level, two criteria, force-based and displacement-based, were proposed to define the target behavior/response at the specific level. Brief descriptions of these SP levels are presented in Table 1.

Although the SP level recommendations address system performance, SEAOC notes that until research shows otherwise, the system characteristics may serve as an acceptable surrogate for the performance requirements of elements. Adopting this approach, the yield strength limit/elastic displacement limit of a cold-formed steel frame shear wall may be interpreted as that point in the measured wall response corresponding to minimal to zero inelastic displacement demand (that is, minimal to no damage).

Both the CUREE recommendations (2004a) and SEAOC's seismic performance level guidelines (1999) appear to support the concept of an effective yield strength limit based on minimal to no damage or minimal to zero inelastic displacement demand of the lateral element.

Table 1. SEAOC (1999) seismic performance (SP) levels

<table>
<tr><th>Structural Performance Level</th><th>Strength-Based</th><th>Displacement-Based</th></tr>
<tr><td rowspan="2">SP-1</td><td colspan="2">Damage is negligible. Structural response corresponds to the effective yield limit state. Inelastic displacement capacity is substantially unused.</td></tr>
<tr><td>Structures designed to remain elastic. Strength design to achieve SP-1 at R = 1.0.</td><td>Approximately 0% of the inelastic displacement capacity is used. IDDR[1] ≈ 0. System displacement ductility, $\mu_{system} = 1.0$.</td></tr>
<tr><td rowspan="2">SP-2</td><td colspan="2">Damage is minor to moderate. Inelastic response at ½ the level expected for the 10% in 50-year earthquake.</td></tr>
<tr><td>Strength design to achieve SP-2 at ½ the code specified R.</td><td>Approximately 30% of the inelastic displacement capacity is used. IDDR = 0.3. System displacement ductility, $\mu_{system} = 2.9$.</td></tr>
<tr><td rowspan="2">SP-3</td><td colspan="2">Damage is moderate to major. Inelastic response at the level expected for the 10% in 50-year earthquake.</td></tr>
<tr><td>Strength design to achieve SP-3 at the code specified R (essentially the life-safety limit state addressed in the building code).</td><td>Approximately 60% of the inelastic displacement capacity is used. IDDR = 0.6. System displacement ductility, $\mu_{system} = 4.8$.</td></tr>
<tr><td rowspan="2">SP-4</td><td colspan="2">Damage is major. Repairs may not be economically feasible. Residual strength, stiffness and margin against collapse are significantly reduced.</td></tr>
<tr><td>Strength design to achieve SP-4 at 1.5 times the code specified R.</td><td>Approximately 80% of the inelastic displacement capacity is used. IDDR = 0.8. System displacement ductility, $\mu_{system} = 6.0$.</td></tr>
<tr><td rowspan="2">SP-5</td><td colspan="2">Partial collapse is imminent or has occurred.</td></tr>
<tr><td>Should not be used as a design target.</td><td>100% of the inelastic displacement capacity is used. IDDR = 1.0. This performance level should not be considered a design target.</td></tr>
<tr><td colspan="3">[1] IDDR: Inelastic displacement demand ratio</td></tr>
</table>

Considering the intent of the building code as expressed in FEMA 450-2 (2004), the recommendations resulting from the CUREE studies (2004a, 2004b) and the recommendations contained in the SEAOC performance-based guidelines (1999), this paper presents a method for estimating the effective yield strength for cold-formed steel light-frame shear walls. The method is based on the concept of minimal to zero inelastic displacement demand at the effective yield strength limit state.

ASTM E2126 Yield Strength Model

For light-frame shear walls, the most current adopted method for estimating the yield strength of the wall is described in ASTM E2126 (2007). E2126 states that the yield limit state (yield point) of a light frame shear wall may be determine as the point in the load-displacement relationship where the [secant] elastic shear stiffness of the assembly decreases 5 % or more. E2126 further suggests that for "nonlinear ductile elastic responses," the yield point may be determined using the equivalent energy elastic-plastic (EEEP) curve to represent the envelope response of a tested shear wall (see Figure 1).

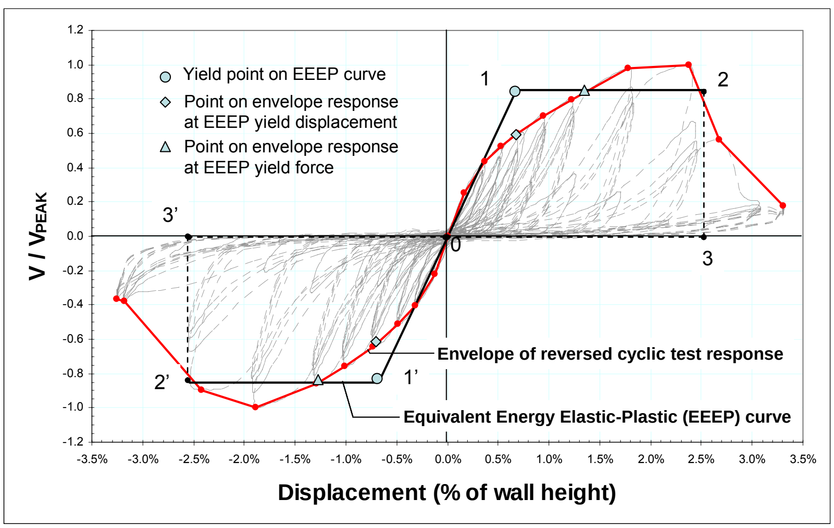


Figure 1. Yield point determination using the EEEP methodology

The 2007 North American Standard for Cold-Formed Steel Framing – Lateral Design (AISI S213) provides a commentary on the use of the EEEP methodology in the development of design values in this Standard.

Although, ASTM E2126 states when the EEEP method may be used, the Standard does not provide a basis for determining what constitutes "nonlinear ductile elastic response," the trigger for using the EEEP method. For seismic design, ASCE/SEI 7 (2005) identifies three levels deformability (ratio of ultimate deformation to limit deformation) for elements: high-deformability elements, limited-deformability elements and low-deformability elements. These three levels are illustrated in Figure 2. If the idea that a ductile response is required to employ the EEEP method of analysis, a criterion related to element deformability may be useful for application of the EEEP method.

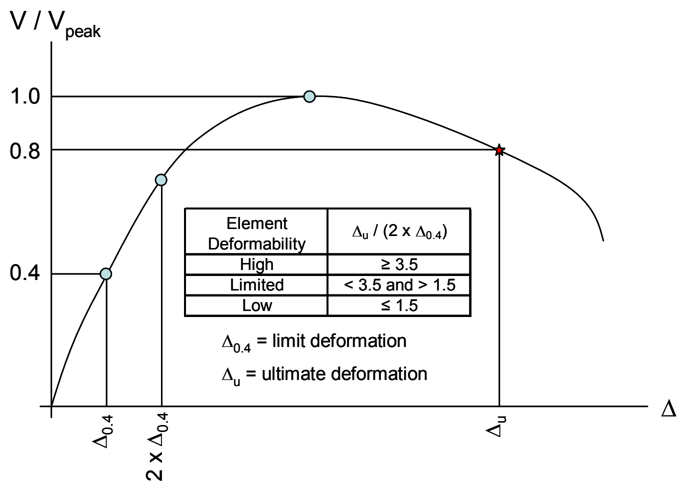
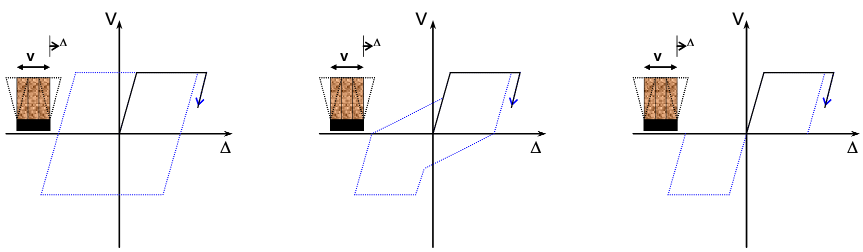


Figure 2. ASCE/SEI 7 Element deformability levels

The minimum yield strength permitted under the EEEP method is defined at 80% of the peak strength of the shear wall. Though this limit may have an historical reference, it does not appear to have a rational basis. At 80% of a cold-formed steel shear wall's peak strength, lateral displacement is likely to exceed SEAOC's (1999) structural performance level 1 (SP-1) limit, damage is likely to be beyond minimal with significant permanent displacement, and the assumption of an elastic response as defined in ASTM E2126 may not be applicable.

Application of the EEEP method alone to determine the yield strength limit does not capture the beneficial energy dissipating attributes of a more robust hysteretic response. Figure 3 illustrates, schematically, three hysteretic response envelopes for lateral-force resisting elements that may be installed in cold-formed steel light frame construction. Under the EEEP method, all three elements would be assigned the same performance characteristics, unless hysteretic energy is somehow taken into account. It is clear that the energy dissipated by the element with the robust hysteretic response should provide a superior performance, compared to the other responses, in terms of the energy dissipated within the system.



(a) Robust (b) Pinched (c) Severely pinched

Figure 3. Schematic representation of hysteric response envelopes

Hysteretic Envelope Energy Balance (HEEB) Yield Strength Model

The hysteretic envelope energy balance (HEEB) methodology presented in this paper attempts to incorporate the equivalent energy elastic-plastic concept expressed in ASTM E2126 with the recommendations in CUREE and the SEAOC guidelines. The HEEB method employs a hysteretic model similar to that used for nonlinear dynamic analysis of buildings with light frame shear walls (Stewart 1987, CUREE 2002) with the exception that only the envelope response is considered.

Figure 4 shows the non-dimensionalized response of a reversed cyclically tested cold-formed steel shear wall. The envelope force-displacement response is overlaid on the hysteresis plot. To apply the HEEB method, the envelope hysteretic response is determined by considering the maximum usable displacement Δ_u, the elastic stiffness K_o and the "pinching stiffness" K_p, as illustrated in Figure 4. To compute the energy enclosed by the envelope curve, it is assumed that at Δ_u, the lateral element unloads with stiffness K_o. Unloading is followed by loading in the opposite direction with an initial degraded, pinched stiffness K_p before the stiffness K_o is again achieved.

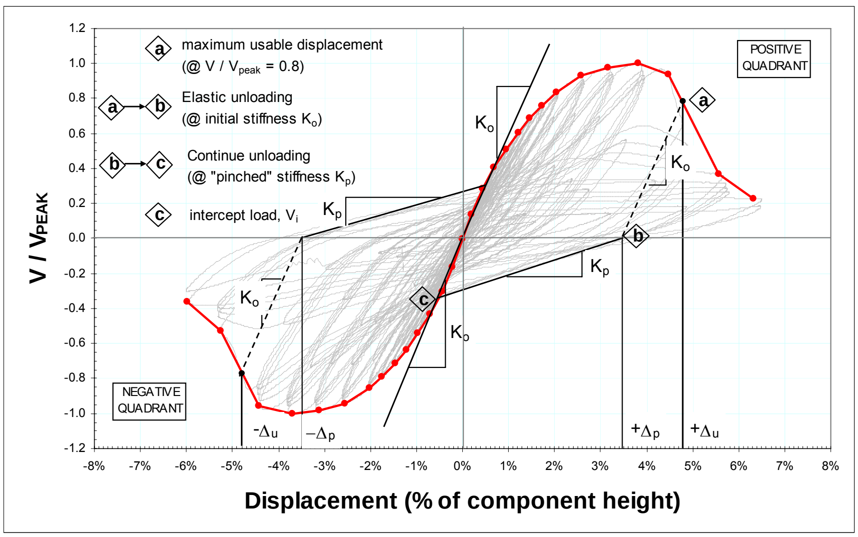


Figure 4. Development of the envelope hysteretic response envelope curve

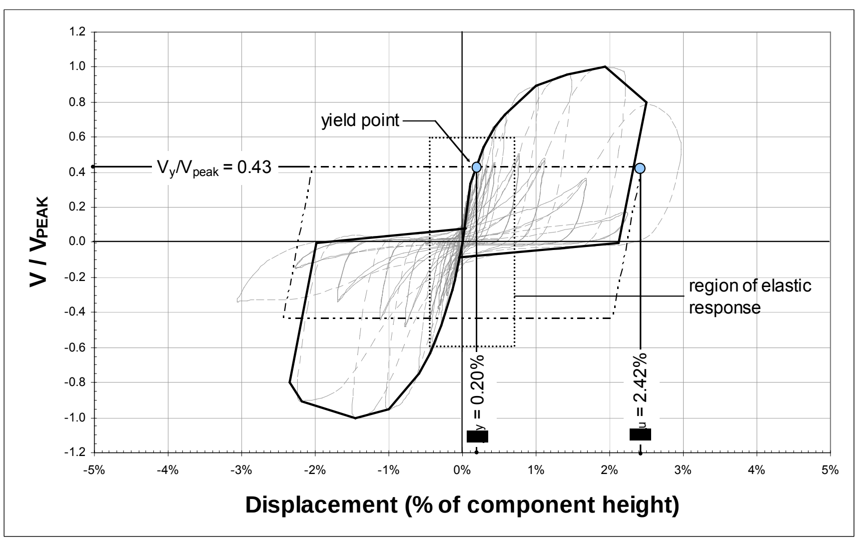
Referring to Figure 4, application of the proposed HEEB method is outlined below:

1. Develop the envelope curve for the lateral element.
2. Determine the peak lateral resistance V_{peak} and the corresponding lateral displacement Δ_{Vpeak} at V_{peak}.
3. Compute $0.4V_{peak}$ and determine the lateral displacement $\Delta_{0.4Vpeak}$ at $0.4V_{peak}$. $0.4V_{peak}$ is the limit deformation defined in ASCE/SEI 7. $0.4V_{peak}$ also corresponds to maximum allowable stress design strength of a cold-formed steel frame shear wall based on a safety factor (Ω) of 2.5, as stated in the AISI Lateral Standard (AISI S213).
4. Compute the secant elastic stiffness, K_o as $0.4V_{peak}/\Delta_{0.4Vpeak}$.
5. Define the maximum usable displacement Δ_u at 80% of V_{peak} after the peak load point. Δ_u is the ultimate deformation defined in ASCE/SEI 7.
6. Compute the permanent lateral displacement Δ_p assuming the lateral element unloads elastically with an unload stiffness K_o.
7. From Δ_p determine V_i, the intercept load for reload in the opposite direction using the pinched stiffness K_p.

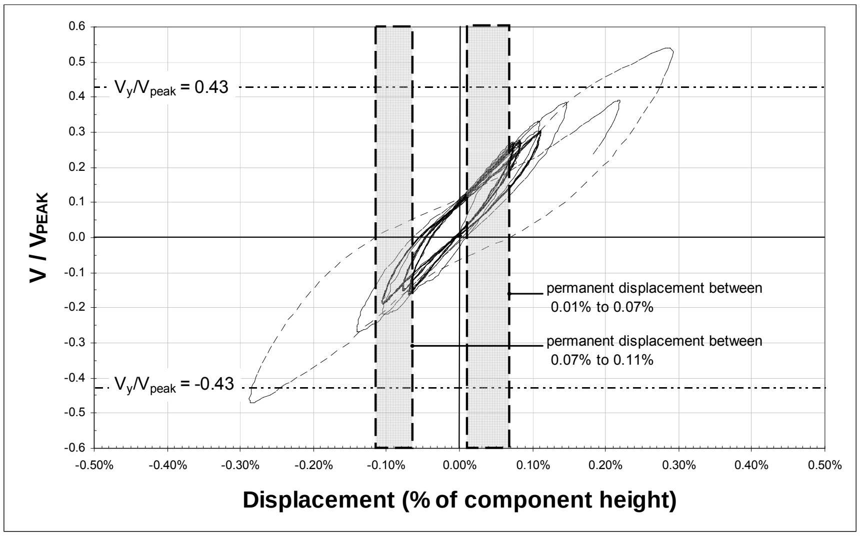
8. Repeat steps (2) through (7) for the loading in the opposite direction.
9. Compute the energy E_T enclosed by the resulting hysteretic envelope response.
10. Determine an equivalent robust elastic-plastic hysteresis response envelope defined by P_{yield} and the average (positive and negative quadrants) Δ_u.
11. Determine the Δ_y using P_{yield} and K_o.

Application of the HEEB Yield Strength Model

Application of the HEEB procedure described above is illustrated in Figures 6 and 7 using data from Branston (2004) and Serrette (1996). The Branston data represents the response of a 1220 mm long by 2440 mm tall shear wall with 11 mm OSB rated sheathing attached to 43-mil framing with No. 8 screws. The screw schedule for the Branston wall was 152 mm at the panel edges and 305 mm in the panel field, and the wall was tested using the CUREE protocol (Krawinkler 2002). The Serrette data represents the response of a similar wall: 1220 mm long and 2440 mm tall with 11 mm OSB rated sheathing attached to 33-mil framing with No. 8 screws. The screw schedule was also similar to the wall in the Branston test and the wall was tested using the sequential phased displacement (SPD) protocol (SEAOSC 1997).

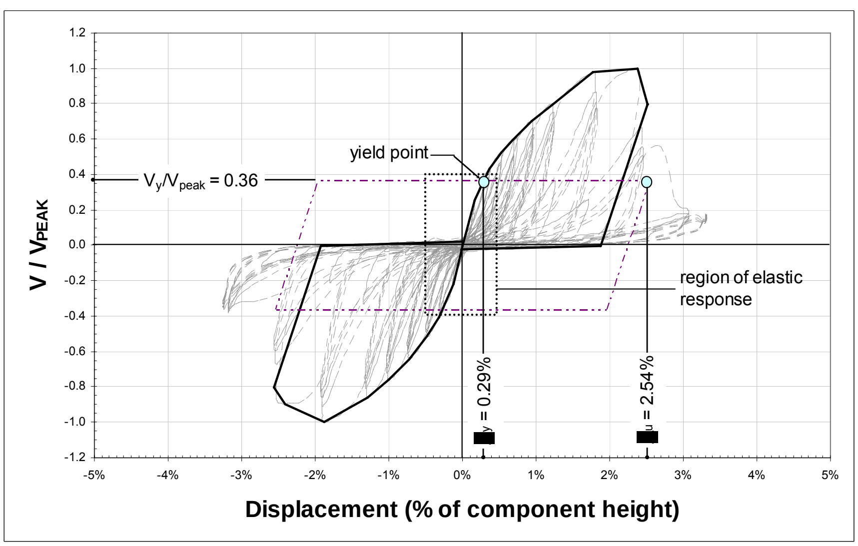


(a) Yield point

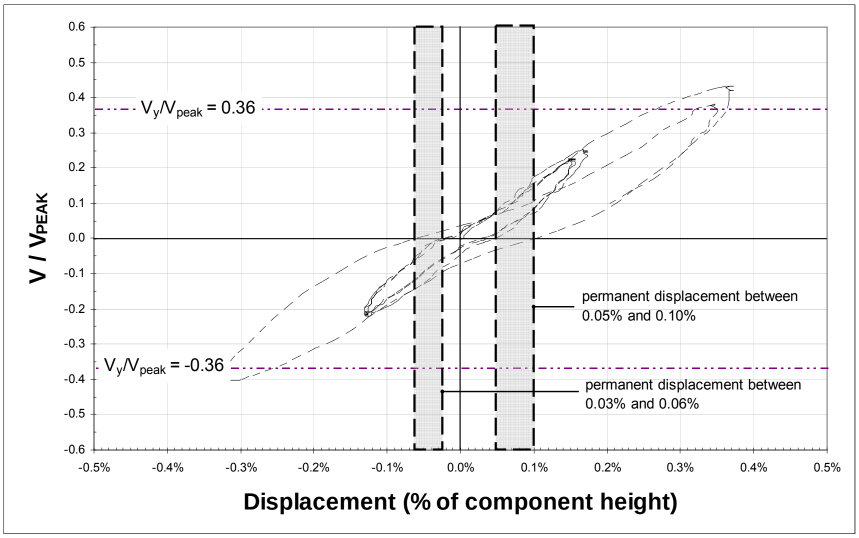


(b) Inelastic displacement demand at yield

Figure 6. HEEB analysis of Branston test data



(a) Yield point



(b) Inelastic displacement demand at yield

Figure 7. HEEB analysis of Serrette test data

As illustrated in both Figures 6(b) and 7(b), at displacements in the region on the computed yield strength/elastic limit, the permanent (unload) displacement from both tests is less than 0.11% of the wall height. At this displacement level, the behavior is essentially elastic and there is minimal demand on the inelastic displacement capacity of the walls. Thus, it appears that the HEEB model provides a result consistent with both the CUREE (2004a, 2004b) recommendations and the SEAOC performance-based guidelines (1999). Additionally, the HEEB yield point provides a relatively accurate assessment of the region in the shear wall response where a shift in the dynamic response (period shift) is likely to occur.

Comparison of ASTM E2126 and HEEB Yield Strength Models

Figures 8 and 9 compare the computed effective yield points for the Branston and Serrette tests, respectively, using the EEEP and the HEEB methods. As shown in these figures, the load and displacement defining the EEEP yield point occur at different positions along the envelope curve, and the yield point itself may not be in close proximity to the response envelope. Unlike the EEEP yield point, the HEEB yield point lies on the response curve (or very close based on averaging of the positive and negative excursions).

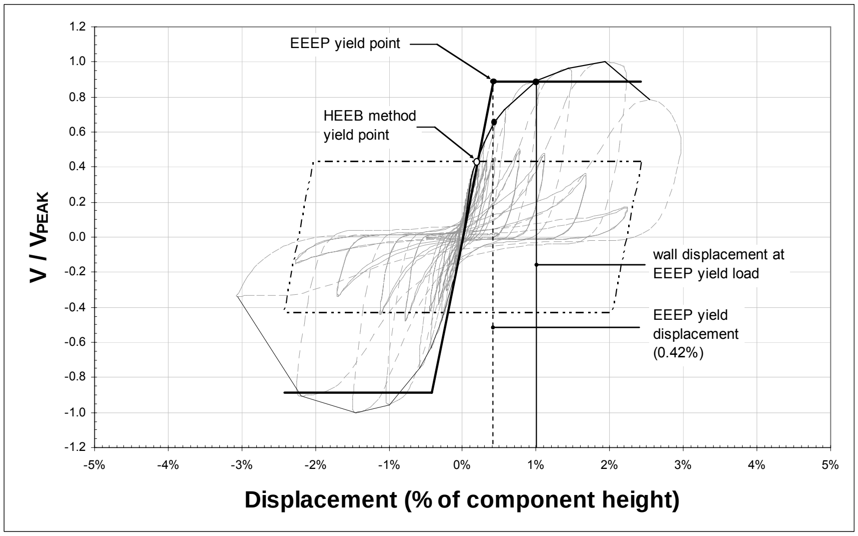


Figure 8. Comparison of EEP yield and HEEB yield—Branston's data

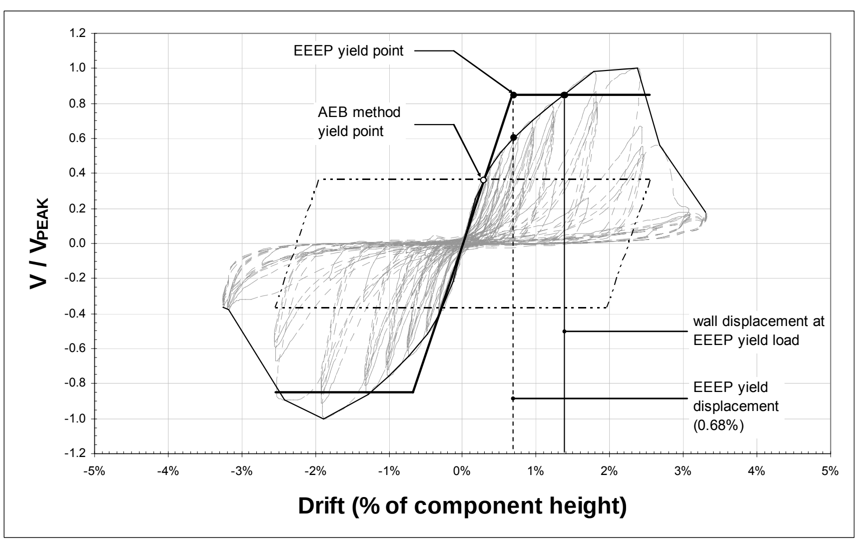


Figure 9. Comparison of EEEP yield and HEEB yield—Serrette's data

Another distinctive difference between the results from EEEP and HEEB methods is the ratio of maximum usable displacement, Δ_u, to the yield/elastic limit displacement, Δ_y. In the examples presented, the EEEP Δ_u to Δ_y ratios for the Branston and Serrette tests were 5.76 and 3.73, respectively. The corresponding HEEB ratios were 12.1 and 8.76.

The SEAOC performance-based engineering guidelines recommended use of the system performance requirements for the elements in the system (pending the development of alternative requirements). Thus, for the maximum considered earthquake (MCE), equivalent to SP-4 in Table 1, the strength level displacement may be related to the maximum usable displacement by the factor 1.5R. Assuming R = 6.5 (wood structural panel or sheet steel cold-formed steel frame shear walls in bearing wall buildings—ASCE/SEI 7), the ratio of the displacement at MCE to the strength level displacement would be 9.75 (= 1.5 x 6.5). If the yield value from the HEEB methodology is considered representative of or close to the strength level design value for the walls, the 12.1 and 8.76 values appear reasonable. Probable relationships between the yield strength and design values using the HEEB yield strength is beyond the scope of this paper.

Conclusion

This paper presented an energy-based method for estimating the elastic limit displacement/effective yield strength of cold-formed steel frame shear walls. The method, referred to as the hysteretic envelope energy balance (HEEB) method, was shown to provide results consistent with the assumption of minimal to no damage or minimal demand on the inelastic displacement capacity of the wall at the effective yield strength. In addition, the derived effective yield strength provided a relatively accurate assessment of the point at which a shift in the dynamic response of the shear wall is likely to occur.

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