BSSC TS7 DRAFT JULY 2002

THE EFFECT OF ECCENTRIC OVERTURNING RESTRAINT IN COMPLETE SHEAR WALL ASSEMBLIES

Steven E. Pryor, P.E., S.E., Lateral Force Resisting Systems Manager
Simpson Strong-Tie, 4120 Dublin Blvd, #400Dublin, CA 94568

ABSTRACT

In this paper various aspects of the effect of eccentricity in overturning restraint devices used in light-framed wood shear walls is discussed. To accurately assess the demand on the tension end post, nonlinear finite element modeling is used to investigate the effect that sheathing/nailing can have on the boundary posts in shear walls. Results are presented at both the allowable stress design level and at the strength limit state, and these results are compared against a bare post analysis. Additionally, both one and two story conditions are analyzed. The results show that the sheathing/nailing can significantly reduce the internal post stresses, and this reduction is proportional to post stiffness and nail spacing. Finally, various aspects of the In-Grade testing program used to determine wood resistance values in the United States are discussed because of the impact they have on the actual distribution of strength in a given grade of lumber.

INTRODUCTION

There has been much discussion recently regarding the effects of eccentricity in the shear wall overturning restraint systems traditionally offered by various manufacturers [4;6;8]. Extensive full scale testing has been performed that suggests that these effects are minor or don't control for the configurations tested [8:27]. Others have countered that this conclusion is incorrect because a: the worst strength-reducing characteristic of a particular grade of wood may not have been present at the point of critical stress; and b: the tests were conducted in a single story configuration and would, therefore, not be applicable to multi-story configurations [6:37-38]. Additionally, a claim has been made that sheathing/nailing cannot offer any resistance to the boundary posts to reduce the effects of an eccentric overturning connection [6:39].

To many, the lack of an epidemic of post failures in walls that appear to have posts overstressed by factors between 5 and 10 suggest that there must be something else going on. To date, estimates of post demand have come from overly simplified models of a highly statically indeterminate situation. To better understand the demand on the end posts, nonlinear finite element modeling of one and two story shear walls was performed to evaluate the internal stresses in the tension post when restrained against uplift by an eccentric device. In the discussions that follow, it is important to remember that dead load and building interaction effects were not considered in the analysis, thus the demand estimates are conservative. Biaxial bending and tension demand was evaluated when the walls reached their Allowable Stress Design (ASD) rated shear resistance and when they reached their strength limit state (SLS). This was then compared to the demand that is predicted by assuming there is no interaction between the sheathing/nailing and the post (the "bare post" approach). For all of these demand levels and analysis assumptions the 1997 NDS combined stress ratio was evaluated.

While this provides a more accurate picture of the real demand, a better understanding of the real resistance is also needed to understand why post failures in full-scale shear wall tests are rare. A review of the in-grade testing program is presented to show why the stress distribution in a shear wall end post will enable higher strengths to be realized than predicted by the in-grade test methods. Also, the effect of the presence of wood 'above grade level' in a given grade of wood will be discussed.

ECCENTRICITY

With eccentric tension connections, internal post stresses cannot be determined without assuming some value for the eccentricity in the connection. The downward restraint force on the post is delivered through the anchor bolt attached to the holdown, so the bolt's physical position is well known. The upward forces on the post come from the upward component of force in the nails that attach the sheathing to the post. The distance between these two forces creates a couple that produces bending stresses in addition to the direct tension stresses. If the holdown anchor bolt force were colinear with the nail force, there would be no couple, and thus no bending moments due to the holdown (there are still moments due to the relative rotation between the sheathing and the post as will be discussed later). So, the position of the nails must be assumed, and in this case the nails are assumed to be located in the middle of the face of the post.

For all of the discussions in the paper, the orientation of shear walls is taken such that applied load is the right, and the tension end post is on the left. Figure 1 shows the bare post free body diagram for a double 2x4 end post removed from an 8 ft. tall wall which carries a design shear of 870 plf at the top of the wall (this is referred to as the bare post approach because any forces not parallel to the post that the sheathing/nailing may impart to the post are not considered in the free body diagram). Without dead load, it can be shown that the uplift will be (approximately) the wall unit shear times the height of the wall, or in this case 7.0 kips. Also shown are the shear, moment and axial force diagrams, with values of each given at the critical section of the post at the top fastener of the holdown. Notice that the value of the moment and tension in the post are highly influenced by the vertical position of the holdown on the post. The internal post moment is the product of the end shear and the farthest distance to the top or bottom fastener in the holdown (whichever is greater), and the end shear is constant regardless of where the holdown is placed. Thus, holdowns placed at mid-height would reduce the moment and tension by a factor of nearly 2 over a bottom-of-wall placement.

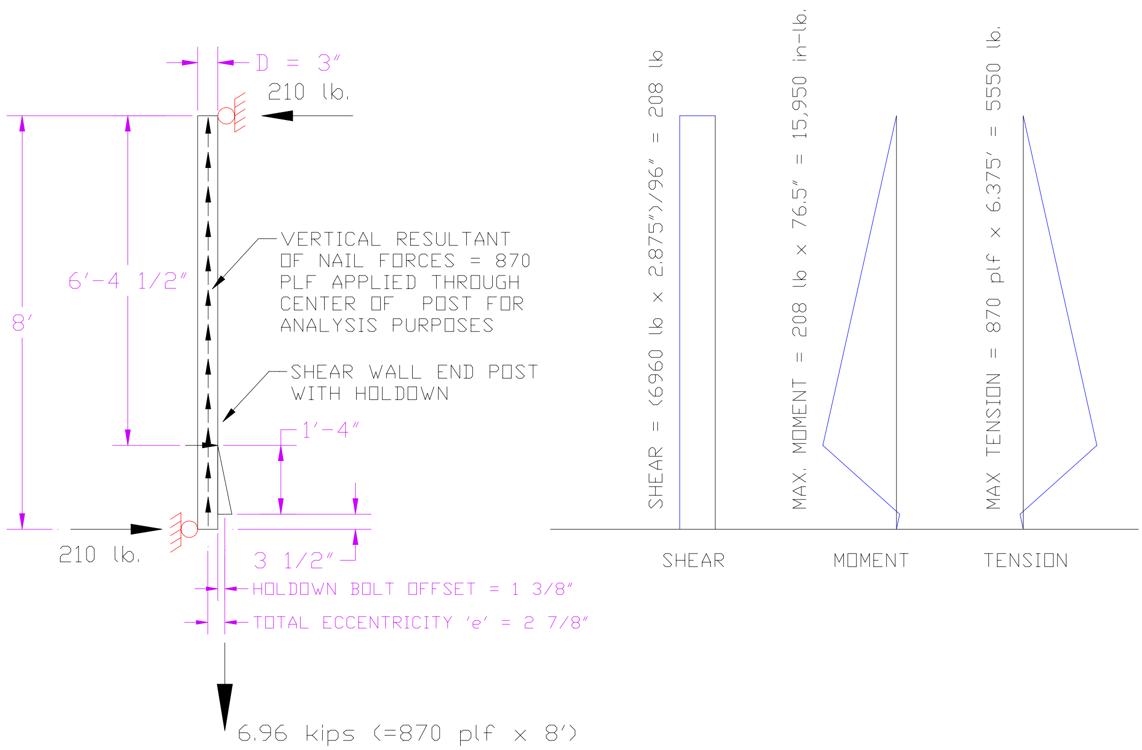


Figure 1: ASD Shear Level Free Body and Force Diagrams for bare post Approach

It has been suggested that perhaps the eccentricity in the connection can be determined by looking at the way the holdown fasteners deliver their load into the post, such as the discussions in the AF&PA Technical Report No. 12 [6:38]. By itself, this report cannot provide the answer. AF&PA Technical Report No. 12 does not consider the tension that will develop in the fasteners from an eccentric holdown connection [1:5], nor does it discuss the compression forces between the body of the holdown and the post, both of which are a result of the eccentricity of the holdown itself. As shown above, these issues don't have any bearing on the global free body diagram used to predict the value of internal tension and moment at the critical section above the holdown anyway.

FINITE ELEMENT MODELING

A commercially available finite element analysis package, MSC visualNastran for Windows, was used to investigated the internal forced in complete shear walls. To start with, the bare post of Figure 1 was modeled using elastic finite element analysis (FEA) in order to compare the moment diagram obtained from FEA with that shown in Figure 1, and the result is shown in Figure 2 (units are inches, pounds).

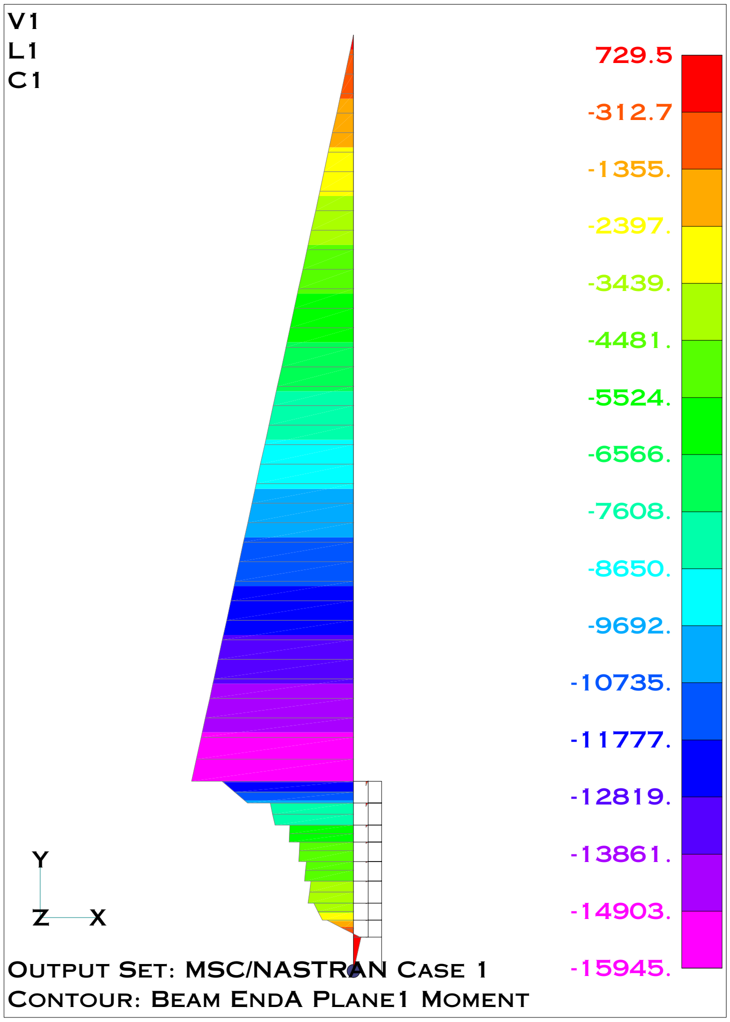


Figure 2: ASD Shear Level Moment Diagram From bare post FEA

As one would expect, the maximum moment from FEA at the top of the holdown, 15945 in-lb, is the same as that obtained in the simple analysis of Figure 1.

Next, nonlinear FEA models were developed for complete shearwalls and compared to full-scale cyclic test results. Figures 3 and 4 show the typical results of this modeling compared to the test results. Like the premise of the bare post analysis of Figure 1, the full scale tests were of a wall 4'x8', framed with DFL wood (double 2x4 #2 end posts, 3x4 pressure treated sill), with 15/32" S1 OSB and 10d common nails at 2" on center at the edges and 12" on center in the field, and PHD8 holdowns for overturning resistance, which gives the wall a code design rating of 870 plf [9:288].

In the model, elastic beam and plate elements were used for the framing members, holdown body and fasteners, and sheathing. Nonlinear springs were used for the nails and compression post support. Nonlinear nail data was derived from research performed as part of the CUREE-Caltech Woodframe Project [3], and the nonlinear compression post support spring was calibrated from the full-scale test data. In addition to three points of restraint along the length of the sill plate to simulate the sill bolts, the sill was also supported by gap elements that would allow upward movement, but not downward movement. Connections between vertical framing members and the top and bottom plates were modeled to transfer shear and compression, but not tension or moment. The anchor bolt for the holdown element was offset from the neutral axis of the post 2.875" (1.375" for the holdown itself plus 1.5" for one half of the post depth).

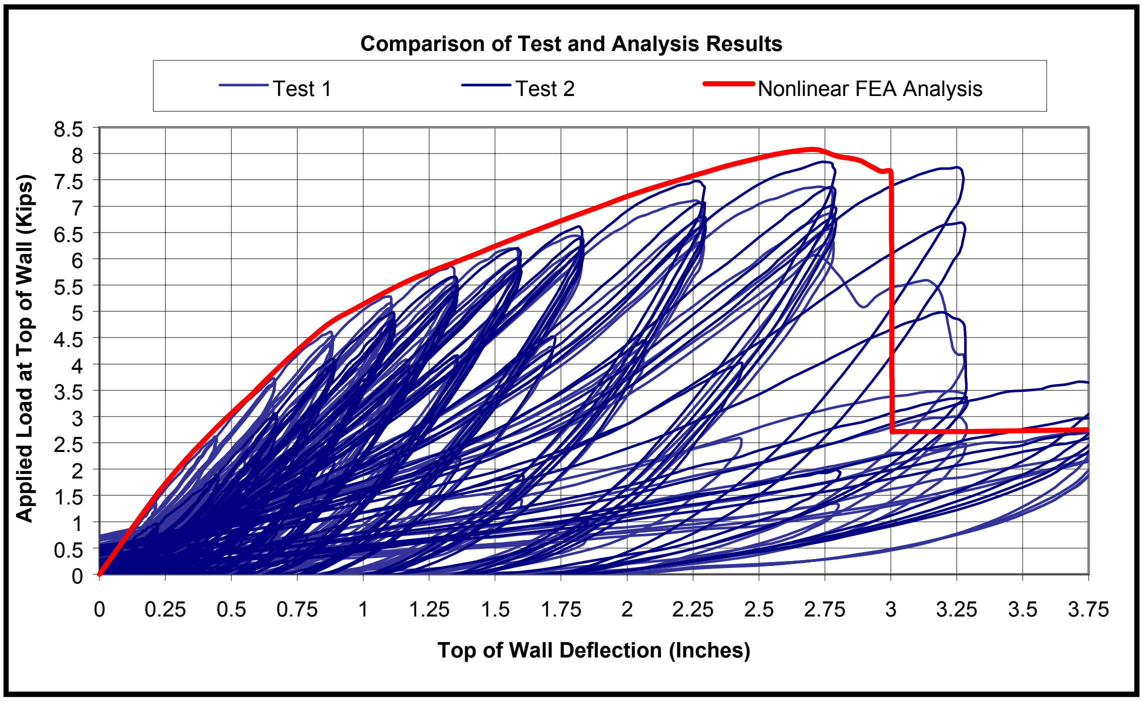


Figure 3: Testing vs. Analysis Results – Wall Stiffness and Capacity

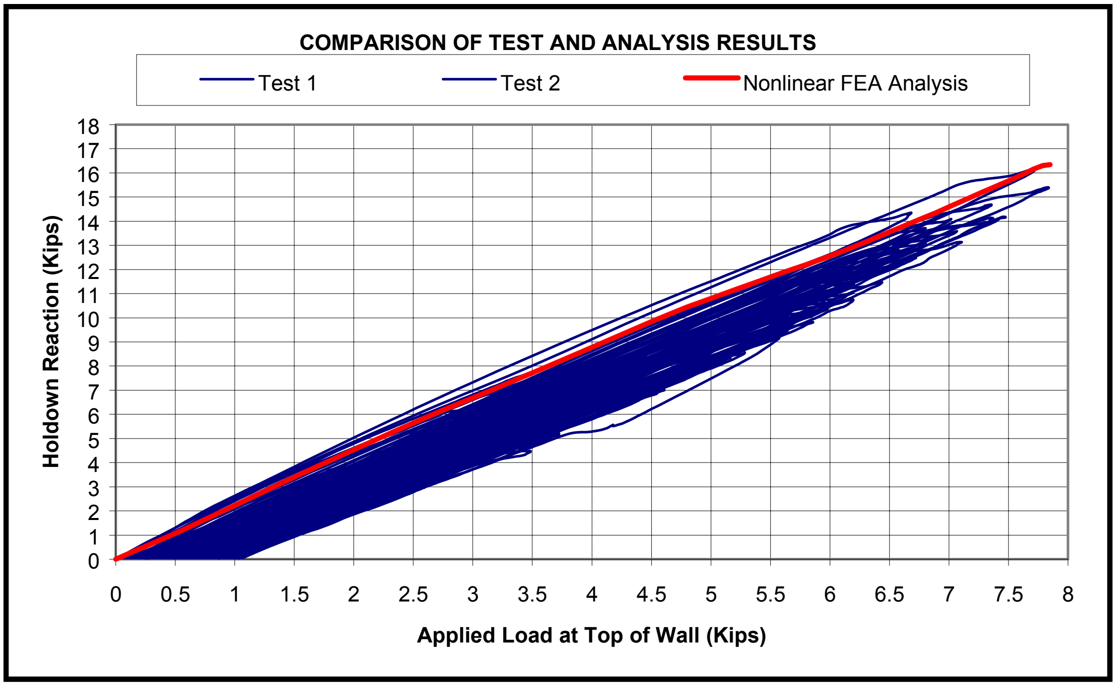


Figure 4: Testing vs. Analysis Results – Holdown Reactions

Having achieved good agreement between the analytical model and the test results, the model was then used to look at the forces developed in the posts to which the eccentric holdown is attached. Figure 5 shows the bending moment diagrams for the framing members at the design shear rating of the wall (870 plf * 4' = 3480#). Note that all FEA bending moment diagrams have units of inches and pounds.

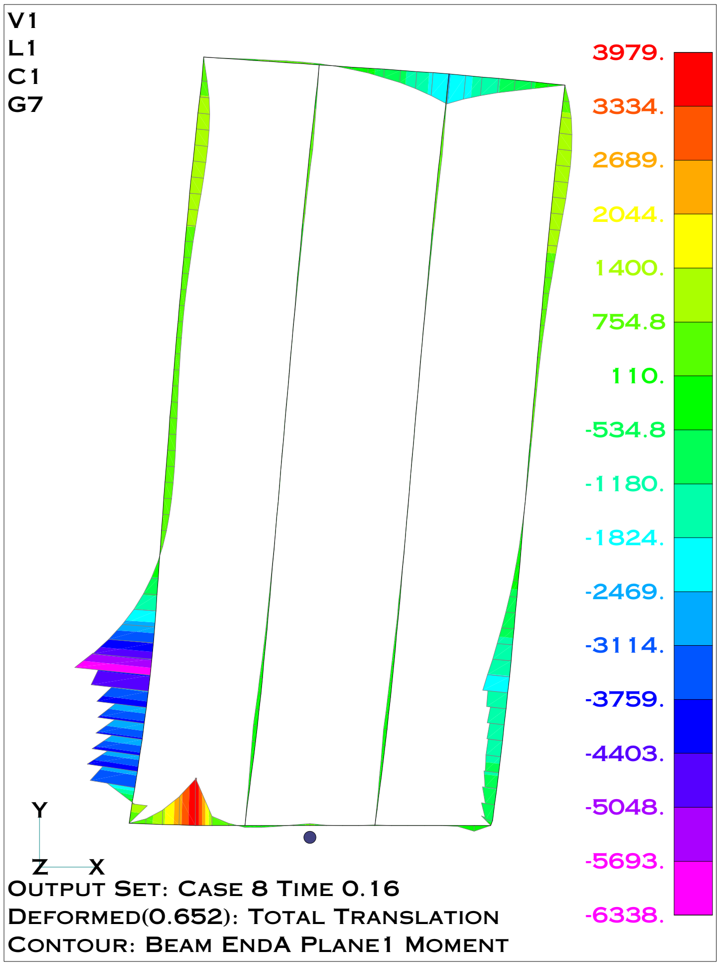


Figure 5: ASD Shear Level Framing Member Moment Diagrams

Comparing Figures 2 and 5, one can see that the shape of the moment diagram of a post in a complete wall assembly is very different from that of a bare post. Also, at the ASD level shear capacity of the wall the value of the bare post maximum moment at the top of the holdown is 252% of the moment in the complete shear wall assembly, 15950 in-lb vs. 6338 in-lb (see Figures 1 and 2).

It should be mentioned that no such overestimate applies to the tension developed in the post. When the FEA model reaches 7 kips of tension in the holdown, the corresponding maximum tension is in the first post element above the holdown and has a value of 5596 lb, or 46 lb higher than the bare post free body diagram prediction in Figure 1. Also, the shape of the tension diagram is the same as in Figure 1. Thus the focus is only on the bending moment reductions due to sheathing/nailing interaction.

For seismic demand, one needs to be concerned with the post behavior at the ultimate capacity of the wall in addition to its behavior at the ASD or LRFD design level. Figure

6 shows the moment diagrams for the framing members when the same wall has reached its peak capacity, or Strength Limit State (SLS).

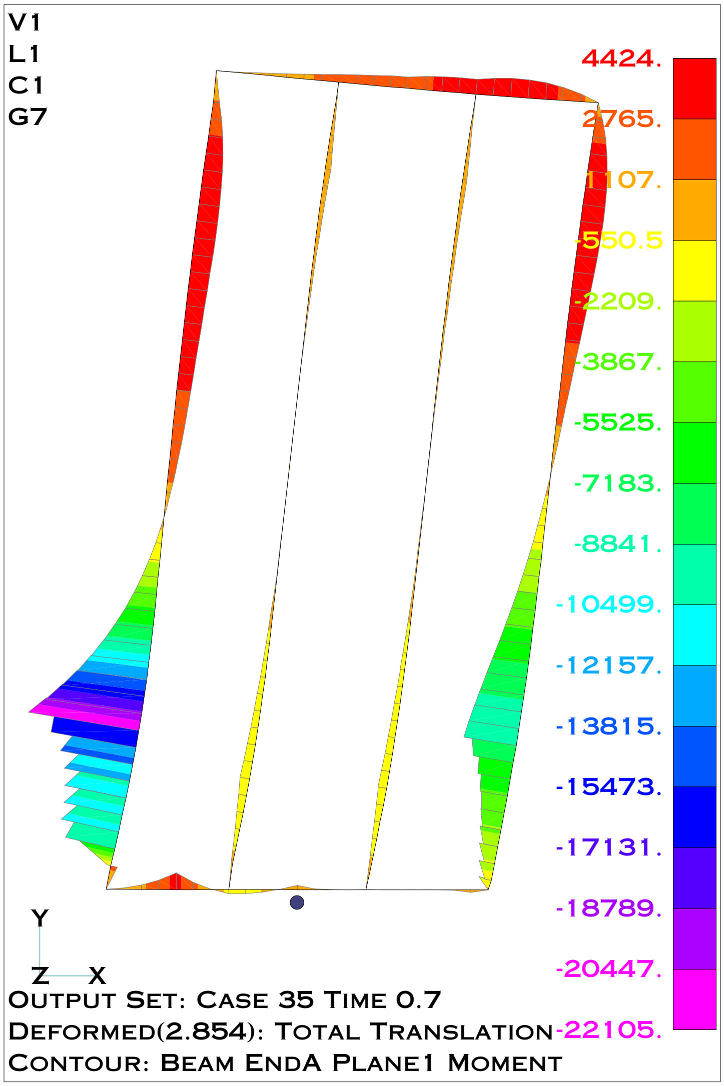


Figure 6: SLS Shear Level Framing Member Moment Diagrams

One can see that the moment diagram for the tension post has the same general shape at the SLS level as it was for the ASD level, and it still does not resemble a bare post. At this point in the analysis the holdown tension was computed to be 16927 lb, which would create a bare post moment of 38790 in-lb, which is 75% higher than that shown in Figure 6 for the complete shear wall assembly. The reason for the reduction in moment can be seen in the relative displacements between the sheathing and the post, which differ from a concentric holdown case to an eccentric holdown case, due to the flexibility of the end post.

Using concentric holdowns will not remove all the moment from the end posts. Figure 7 shows the ASD shear level moment diagram of the same single story wall discussed in Figures 1 through 6, but with concentric holdowns (the bottom of the post was modeled with a pin support). Moments are present in the end posts, even though the holdowns are modeled concentrically. The relative movement between the sheathing and the post causes horizontal forces to develop in the nails, and this in turn creates the moments.

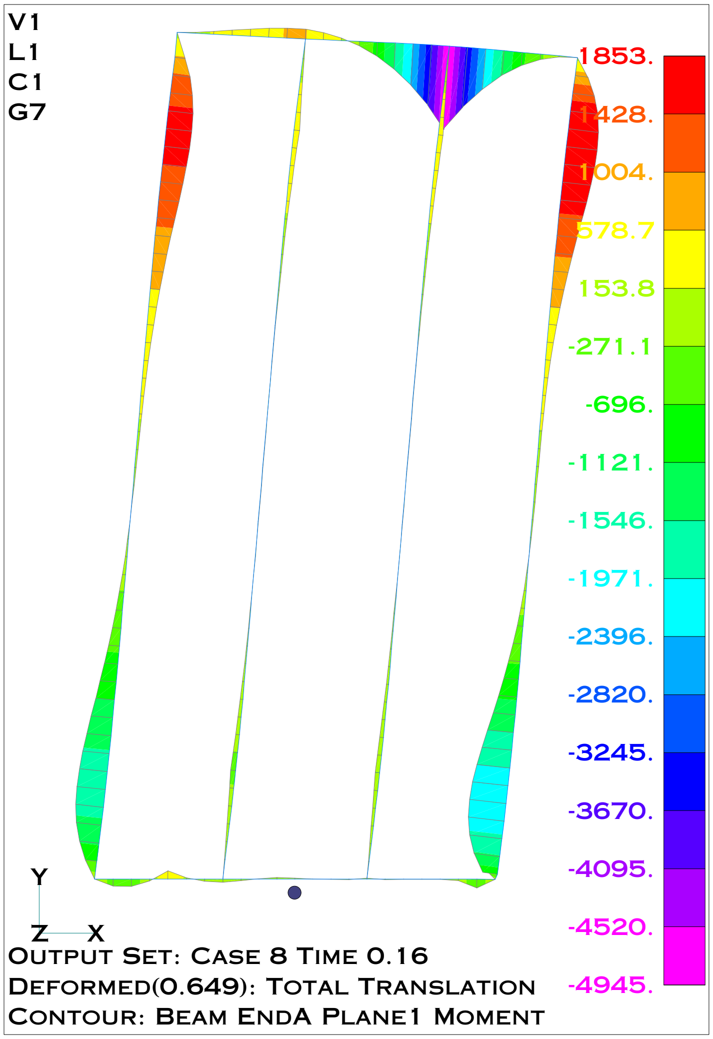
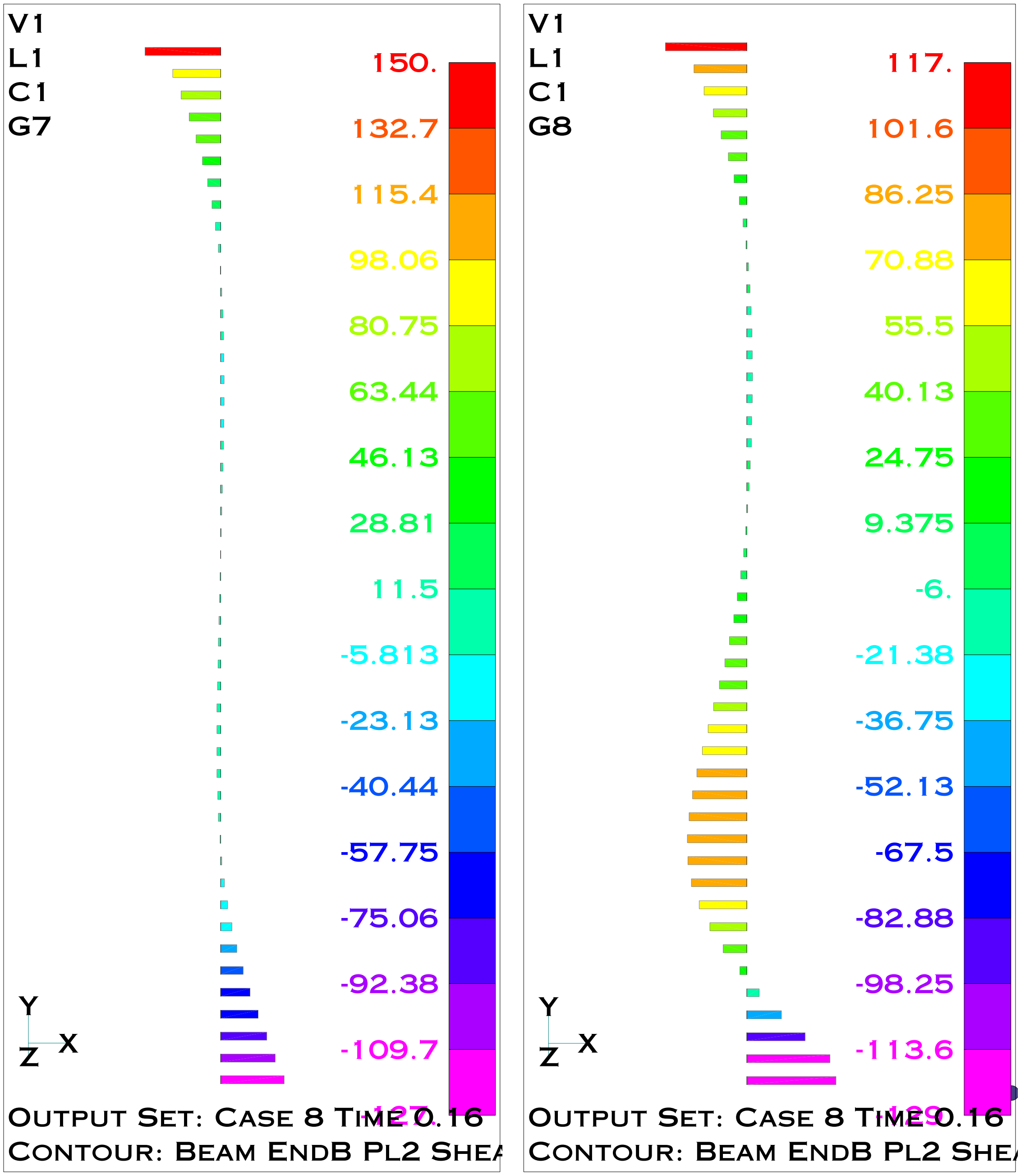


Figure 7: ASD Shear Level Moment Diagram of End Posts in Shear Wall With Concentric Holdowns

These horizontal nail forces along the length of the tension post are shown in Figure 8, along with the horizontal nail forces that exist for the eccentric holdown case. In the upper region of the post, they are very similar, but the concentric holdown case develops higher horizontal nail shears. In the lower region of the post there is a distinct difference between the concentric and eccentric case. Instead of pulling only to the right as shown in [6:39], the post flexibility interacts with the sheathing/nailing in the eccentric holdown case to create horizontal nail shears acting to the left, thus counteracting the effects of eccentricity in the overturning restraint device.



a: Concentric Holdown (Dbl 2x Post) **b: Eccentric Holdown (Dbl 2x Post)**

Figure 8: Horizontal Component of Nail Shear in Tension Post

The importance of considering the post flexibility in our understanding of wall behavior was tested in the following way. If the stiffness of the posts in the same shear wall model with eccentric holdowns were to be increased to the point where it was essentially infinitely rigid with respect to the wall, then one would expect to see the moment diagram change to something like the bare post moment diagram. However, the computed moments near the top of the holdown should be higher than predicted in a bare post analysis because of the moment created by sheathing rotation as discussed earlier. This was tested by increasing the modulus of elasticity (MOE) of the post members from 1.6×10^{6} psi to 1.6×10^{11} psi, increasing the stiffness by a factor of 100,000. The results of this analysis match the prediction very well and are shown in Figures 9. For comparison, Figure 10 shows the horizontal force component of the nail shears corresponding to the rigid post analysis.

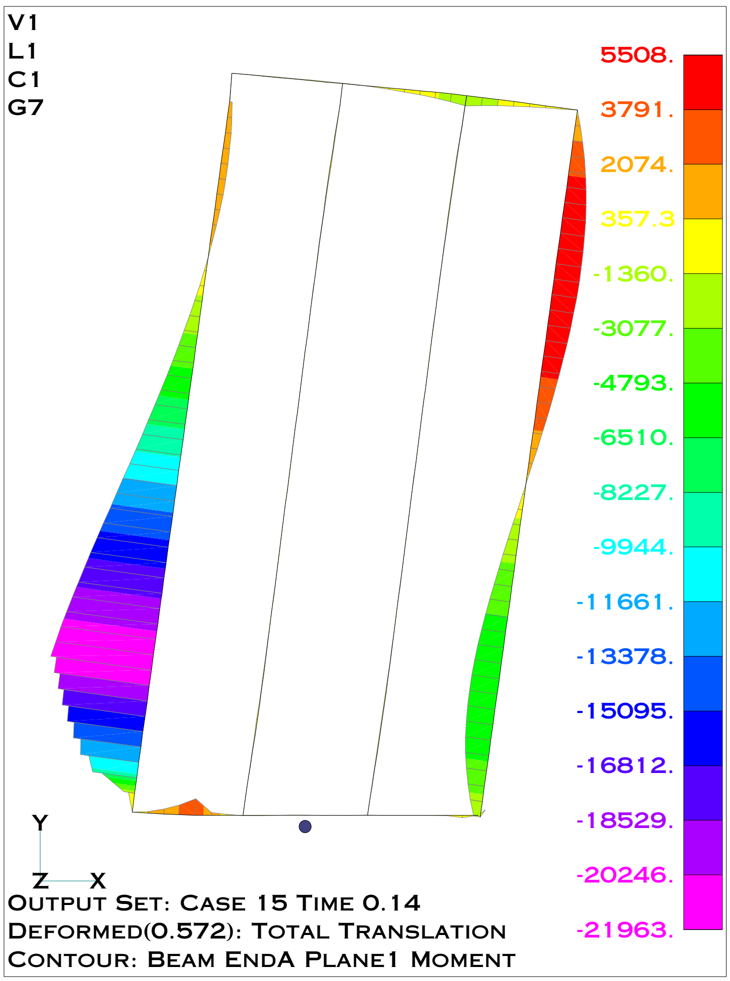


Figure 9: ASD Shear Level Moment Diagrams for Eccentric Holdown Wall With Post Stiffness Increased by a Factor of 100,000

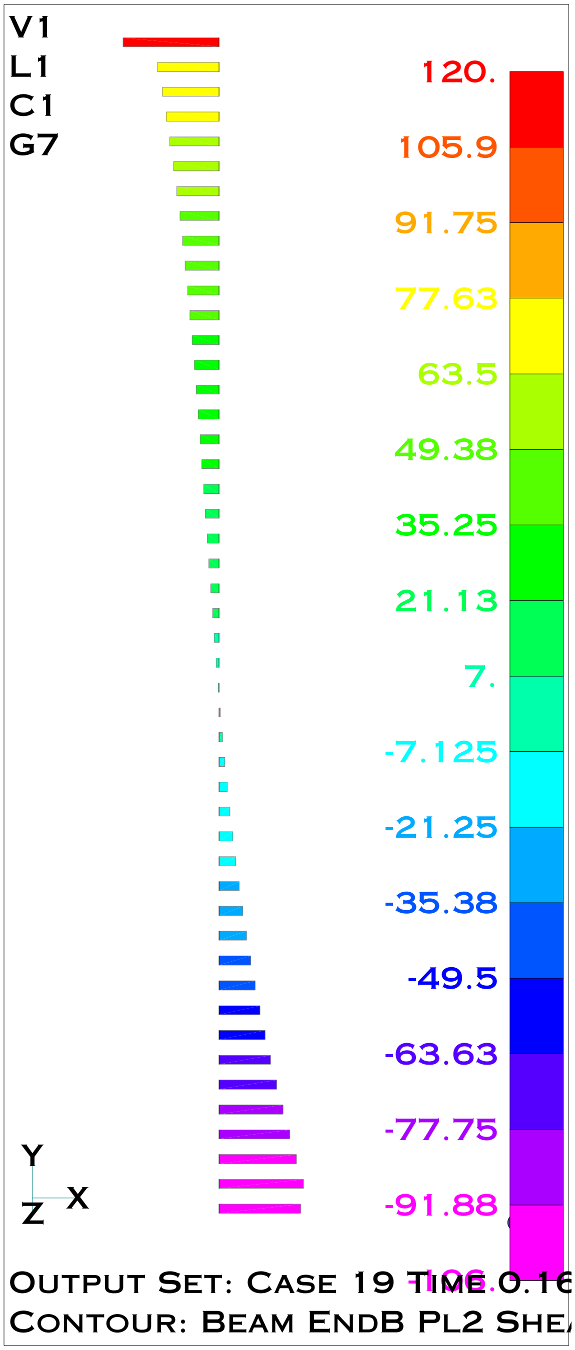


Figure 10: Horizontal Component of Nail Shear in Rigid Tension Post

RESULTS

Single Story

Four different post size/grade combinations, along with three different edge nail spacings, were selected resulting in twelve different wall configurations. Each of these was analytically displaced laterally at the top of the wall until failure had occurred in the nails connecting the sheathing to the framing. The resulting tension and bending about both axes of the post was recorded at two points in the analysis: when the wall had reached its ASD design capacity and when it had reached its peak strength, or SLS.

In all cases theses maximum forces occurred in the post element immediately above the holdown, as was expected. Additionally, in all cases the profile of the bending moment diagram mirrored those shown in Figures 5 and 6. As was mentioned earlier, the internal post tension behaved exactly as predicted by a bare post analysis. Even at the SLS, the FEA calculated maximum internal post tension was within 3% of that predicted by a bare post analysis (using the holdown tension from the FEA analysis). So, in terms of stress reduction because of a sheathing/nailing interaction, it is only the bending interaction that is relevant.

The ratio of maximum moment predicted by the bare post analysis to that of the FEA analysis for the twelve different analysis combinations is shown in Figure 11. For both the ASD and SLS levels, the bare post over prediction is larger for smaller, lower MOE posts and higher nail densities. This makes sense because smaller, lower MOE posts can flex more and higher nail densities have more to offer in terms of resistance to this flex. At the SLS, the over prediction was smaller but still significant, especially for Dbl 2x DFL No. 2 posts (which did not fail in the full scale testing).

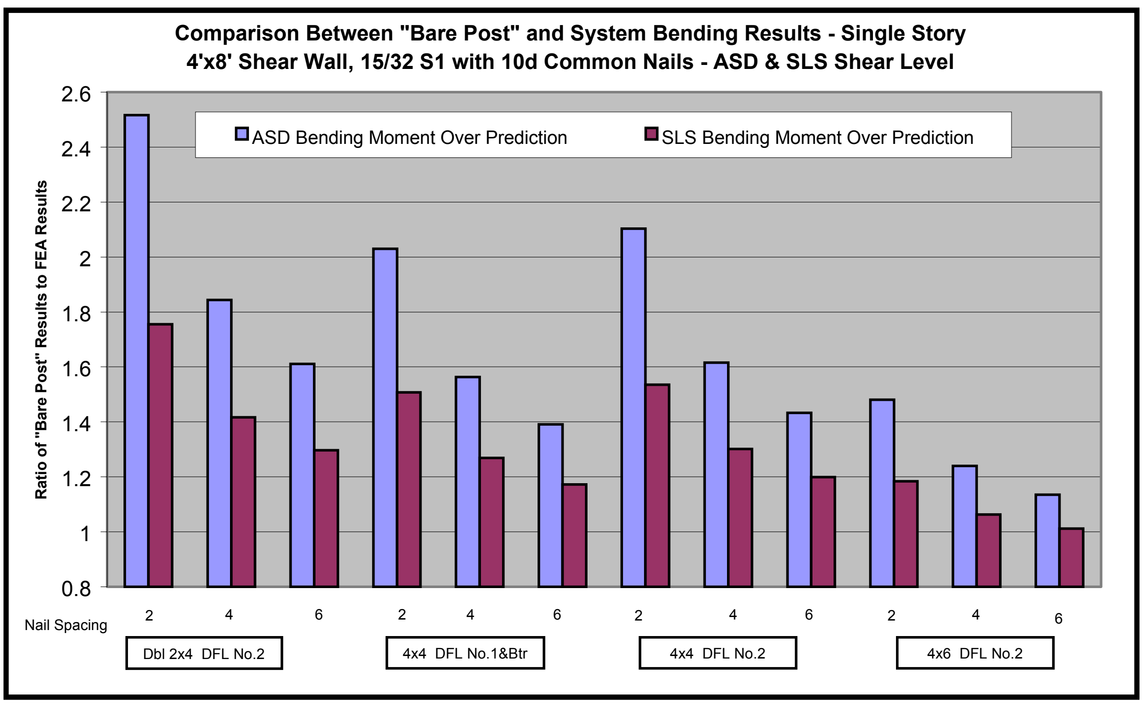


Figure 11: Ratio of bare post Bending Moments to System Bending Moments

Figures 12 through 15 show the combined stress interaction results from evaluation according to the 1997 NDS at both the ASD and SLS level and for load duration factors (LDF) of 1.33 and 1.6. For the ASD level results, axial compression and perpendicular to grain force was assumed equal to the nominal uplift (unit wall shear times height), and the axial buckling factor, K, was conservatively taken as unity. At the SLS, the buckling and perpendicular to grain forces were taken from the FEA analysis as the maximum compression in the bottom of the compression post.

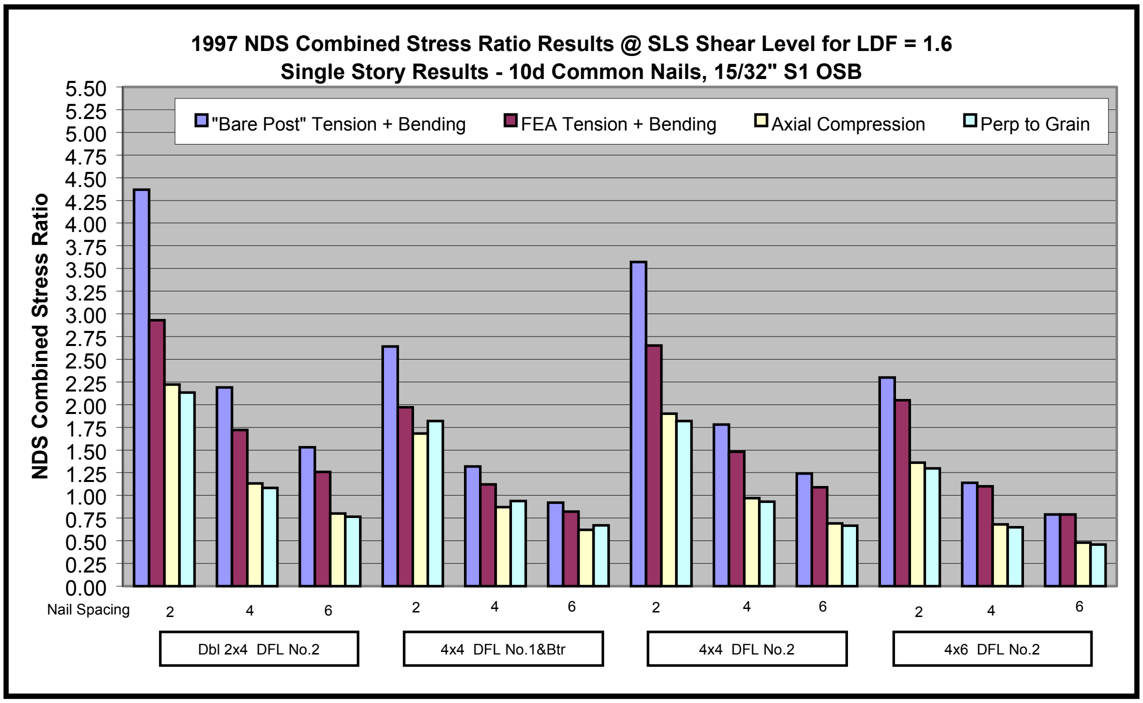


Figure 12: 1997 NDS Evaluation at SLS for LDF=1.6

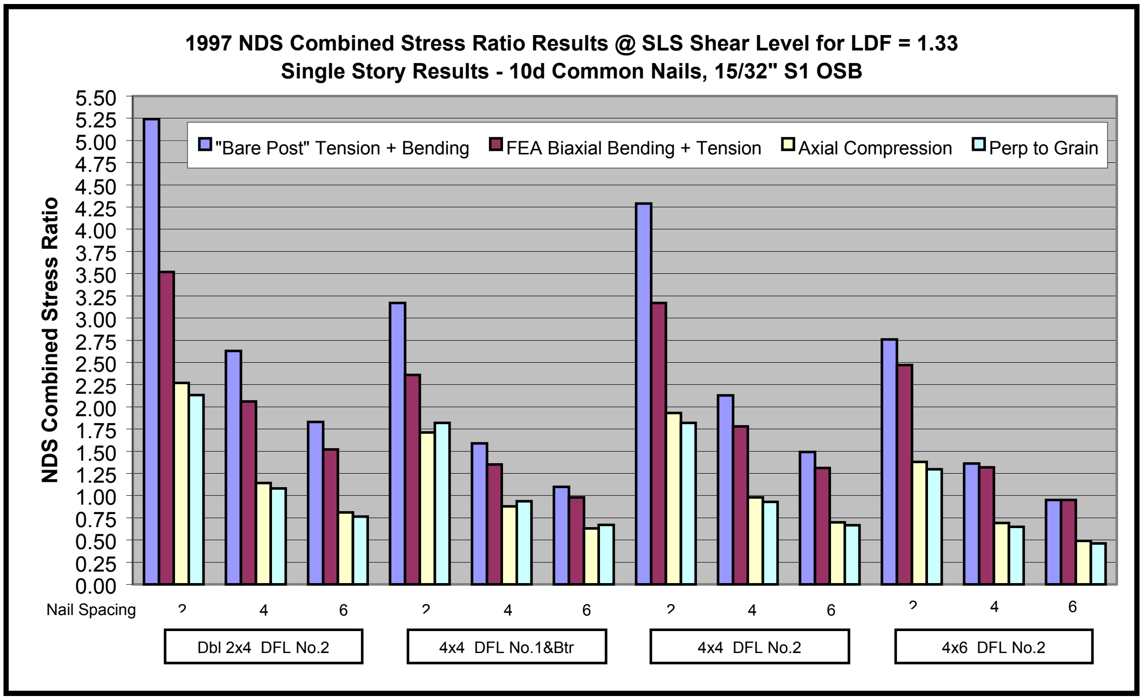


Figure 13: 1997 NDS Evaluation at SLS for LDF=1.33

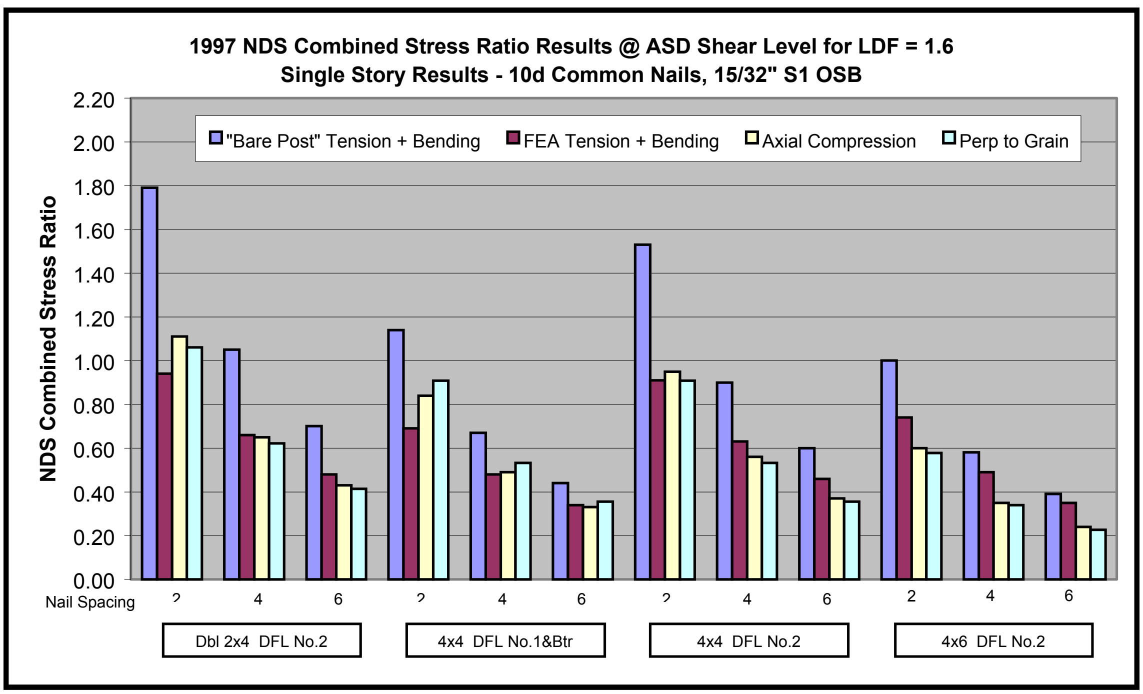


Figure 14: 1997 NDS Evaluation at ASD for LDF=1.6

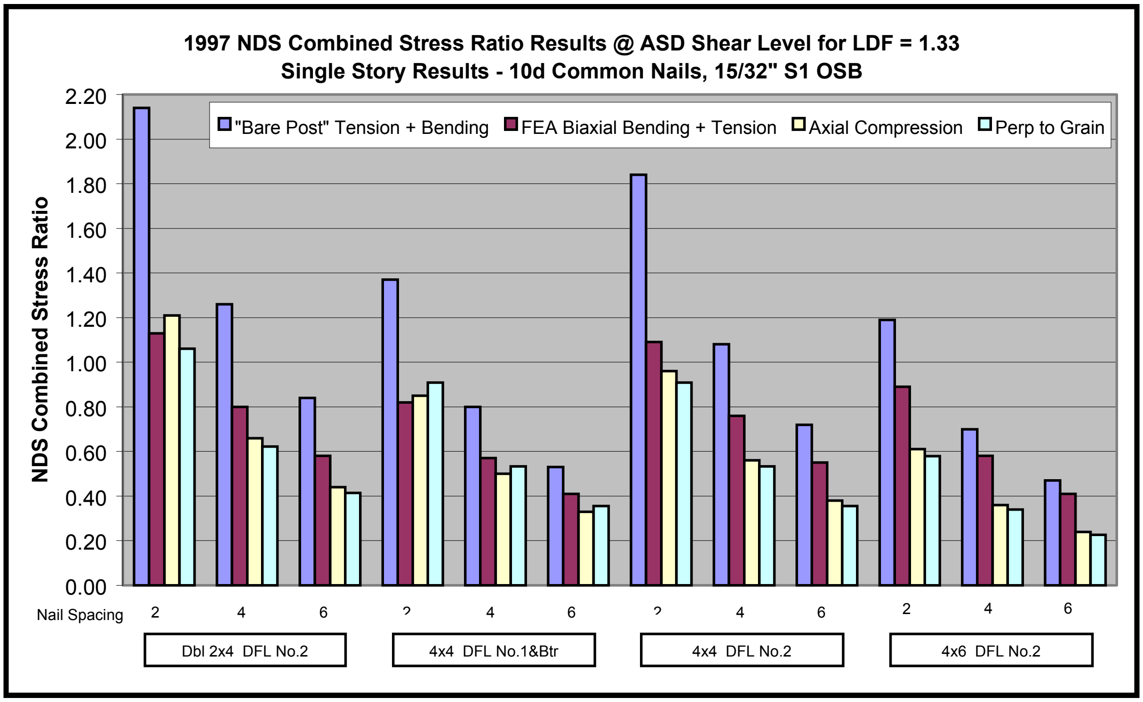


Figure 15: 1997 NDS Evaluation at ASD for LDF=1.33

In addition to the NDS evaluation, another area of interest is the magnitude of the maximum combined stress (tension and bending about both axes) in the critical area of the tension post. In the analytical models, the maximum out-of-plane moment on the post varied from 1% to 4% of the primary in-plane moment. For the bare post approach, no out-of-plane moment was included. Figure 16 shows these maximum stresses for the twelve different analytical models at both the SLS and ASD levels.

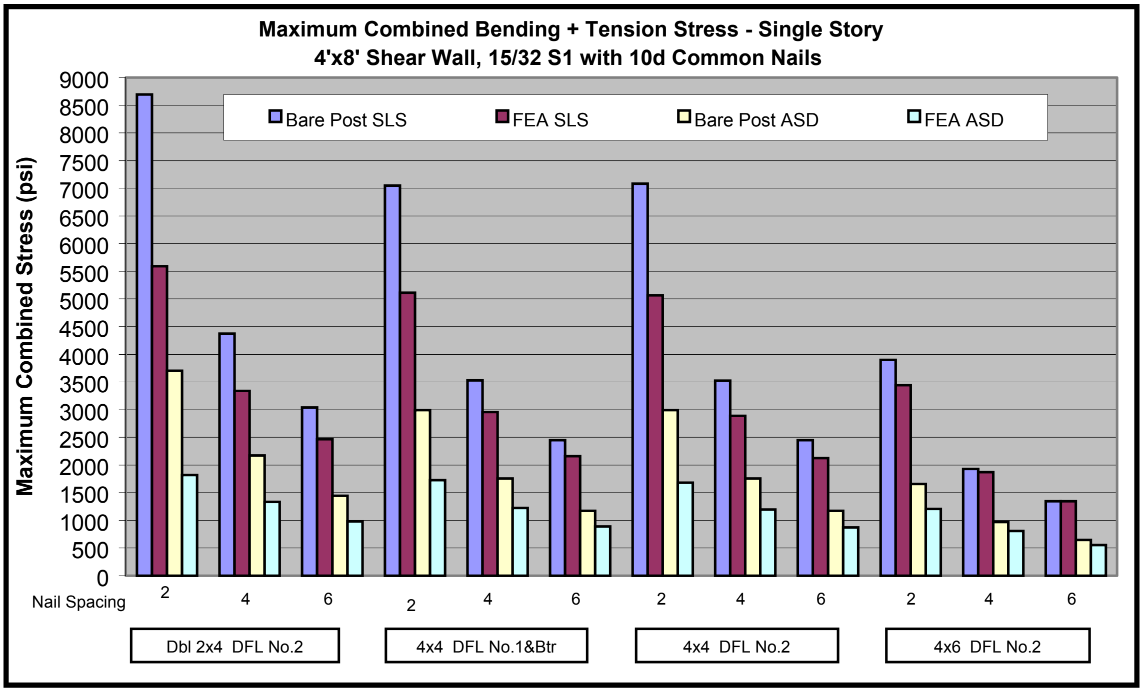


Figure 16: Single Story Maximum Combined Bending + Tension

Two Story

For two story walls, an additional eccentric holdown was placed at the top of the tension post to create an additional upward force simulating the transfer of uplift from a shear wall above through the boundary post of the shear wall below and from there into the foundation. Uplift on this holdown was assumed to be 2/3 of the uplift created by first story shear alone. This was chosen because of the assumed triangular distribution of base shear used for regular structures in an equivalent lateral force elastic seismic analysis. A matching downward force was placed on the top of the compression post. In addition to the four post size/grade combinations investigated for the single story models, 4x6 DFL No.1&Btr with 10d edge nailing at 2" on center was added for the two story analysis. The bare post free body and force diagrams for the two story condition (4x6 end posts) are shown in Figure 17 for reference.

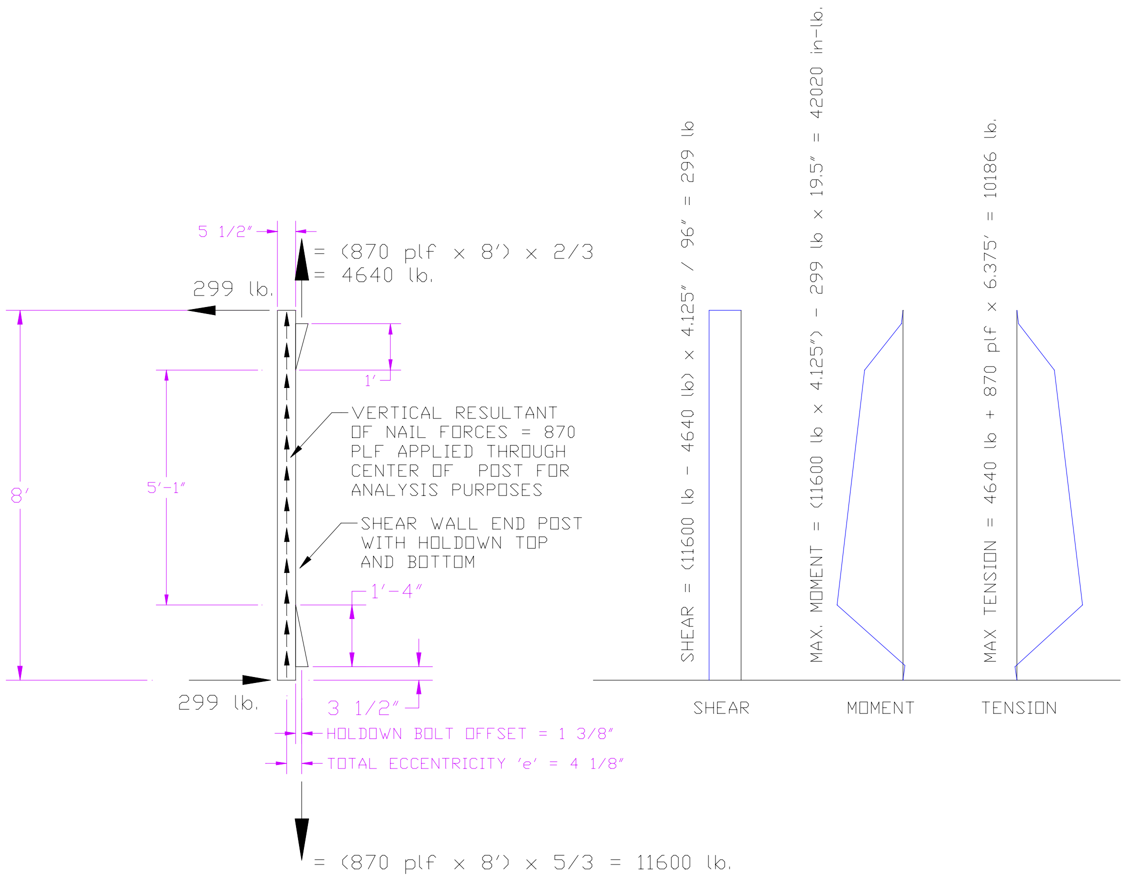


Figure 17: ASD Shear Level Free Body and Force Diagrams for a Two Story Bare Post Condition

Figure 18 shows the state of bending for a matching two story nonlinear FEA analysis (using 4x6 DFL No. 2 end posts) when the wall had reached 870 plf shear.

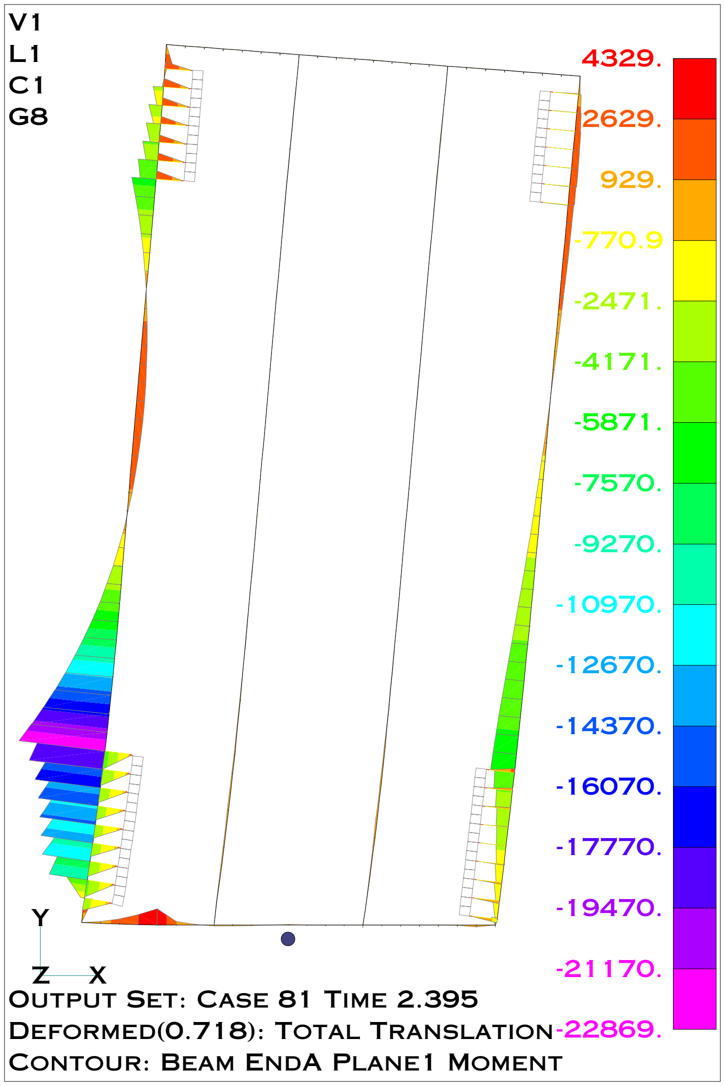


Figure 18: ASD Shear Level Moment Diagram for Two Story Wall

As with the single story analysis, the internal bending moments in the post with additional eccentric uplift at the top are very different than that predicted by a bare post analysis. In this case, the maximum bare post bending moment at the top of the lower holdown is 184% of moment when analyzed as a complete shear wall assembly. Although not shown, analysis was performed with a very rigid post, and again, this caused the computed moment diagram to take on the same shape as that shown in Figure 17, further emphasizing the importance of relative rigidities in understanding complete shear wall behavior.

Figures 19 and 20 show the over prediction of internal post moment when using the bare post method to determine the maximum moments at the lower and upper holdowns.

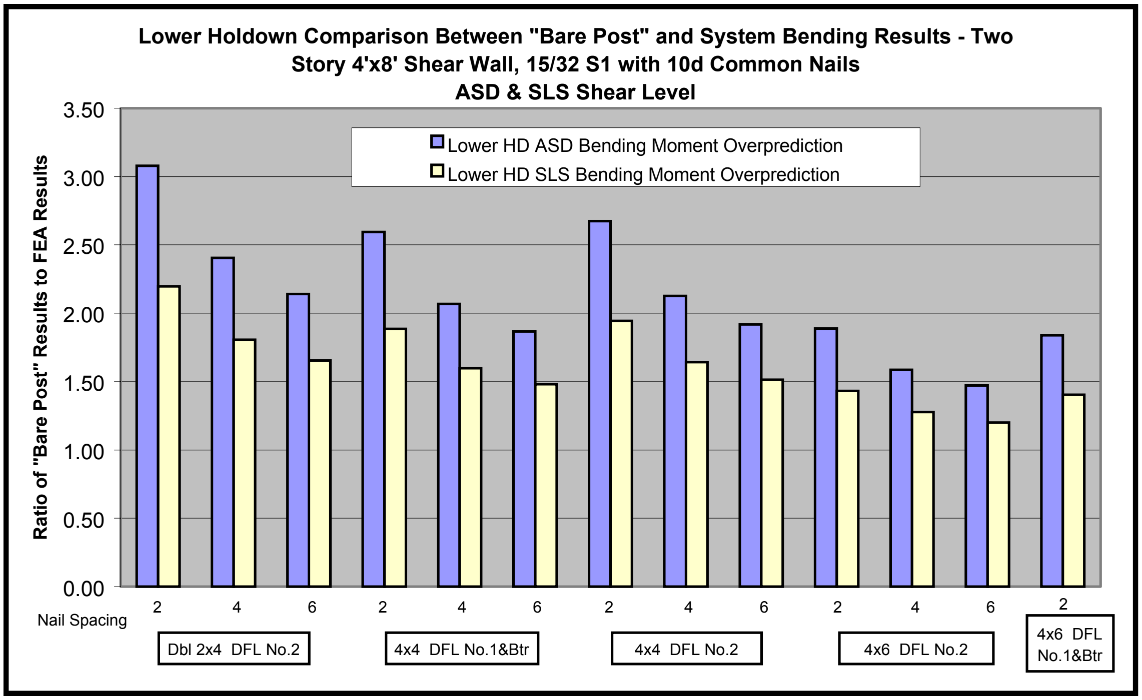


Figure 19: Lower Holdown Ratio of bare post Bending Moments to Systems Bending Moments

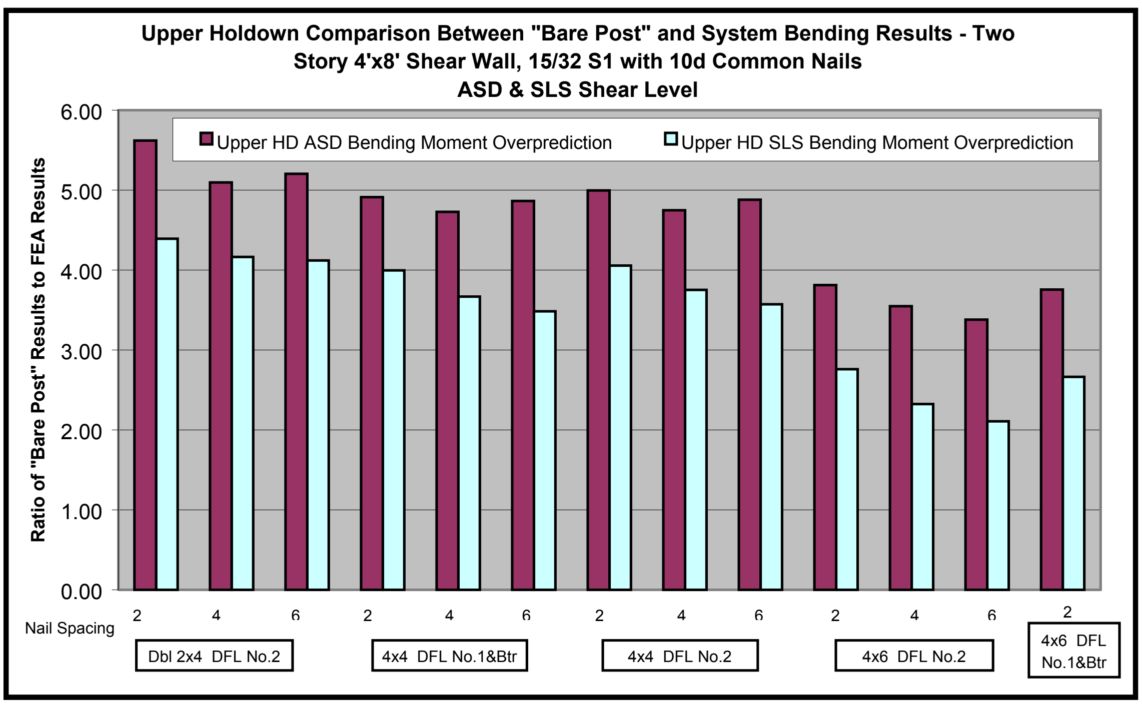


Figure 20: Upper Holdown Ratio of bare post Bending Moments to Systems Bending Moments

For the area of maximum stress just above the lower holdown, Figures 21 through 24 show the combined stress interaction results from evaluation according to the 1997 NDS at both the ASD and SLS level for load duration factors of 1.33 and 1.6. The magnitude of the maximum combined bending and tension stress for the area just above the lower holdown is shown in Figure 25. FEA results include moments about both axes of the post, whereas the bare post analysis considers only the in-plane post moment.

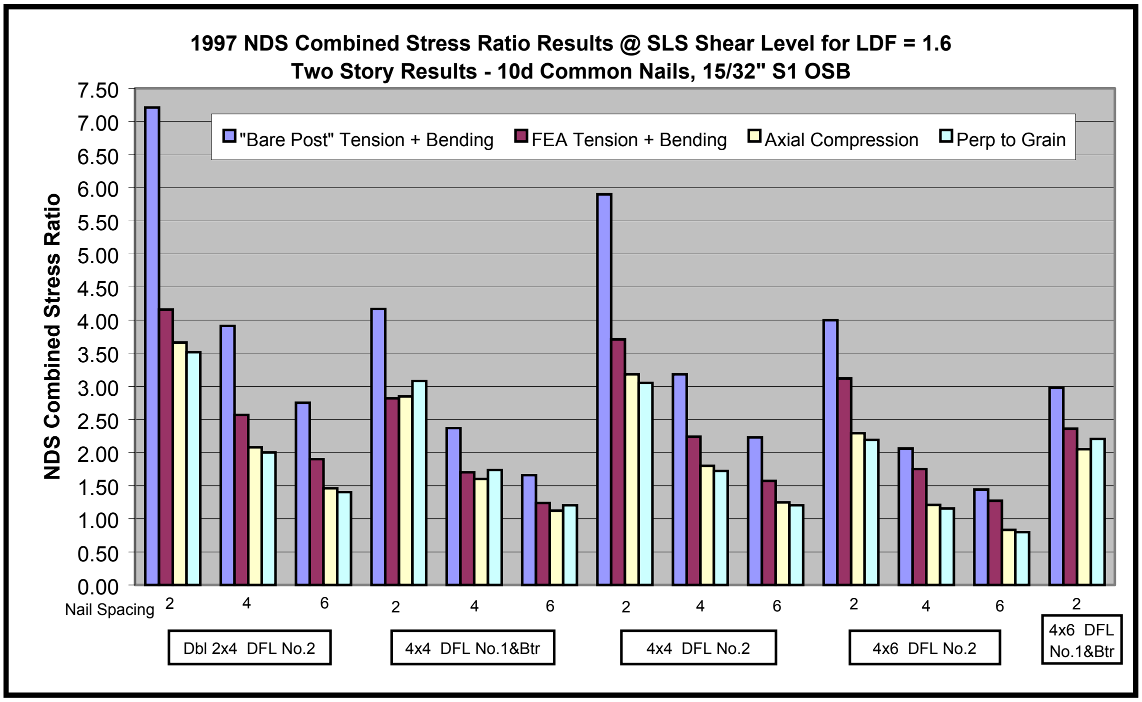


Figure 21: Two Story 1997 NDS Evaluation at SLS for LDF=1.6

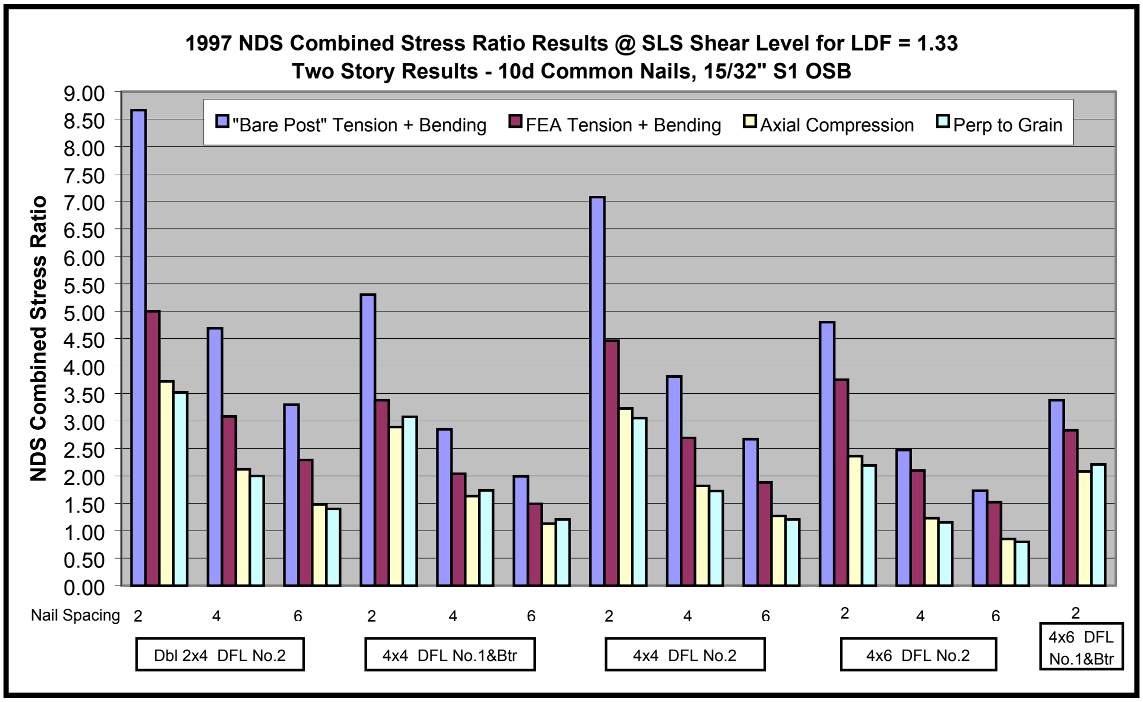


Figure 22: Two Story 1997 NDS Evaluation at SLS for LDF=1.33

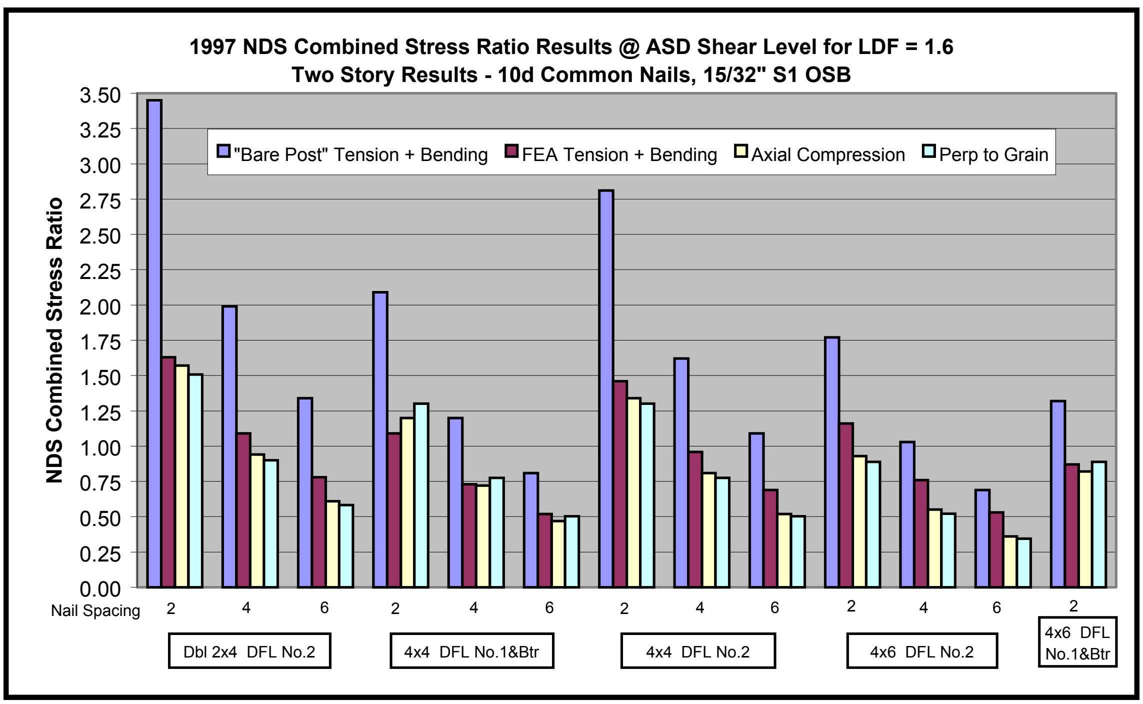


Figure 23: Two Story 1997 NDS Evaluation at ASD for LDF=1.6

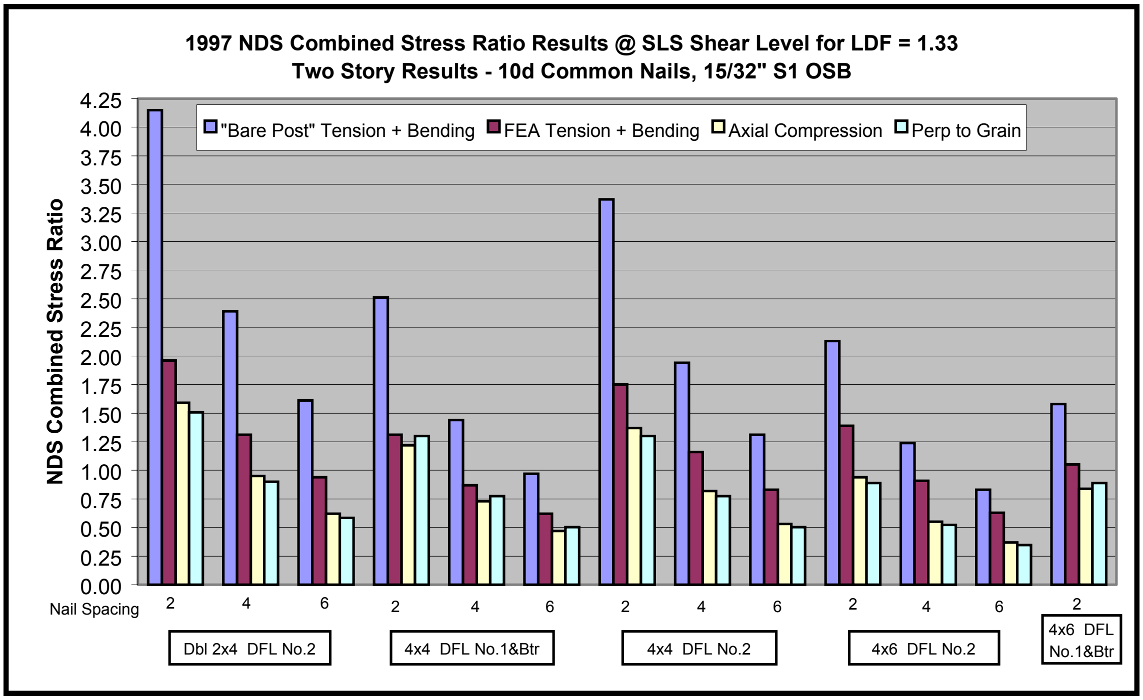


Figure 24: Two Story 1997 NDS Evaluation at ASD for LDF=1.33

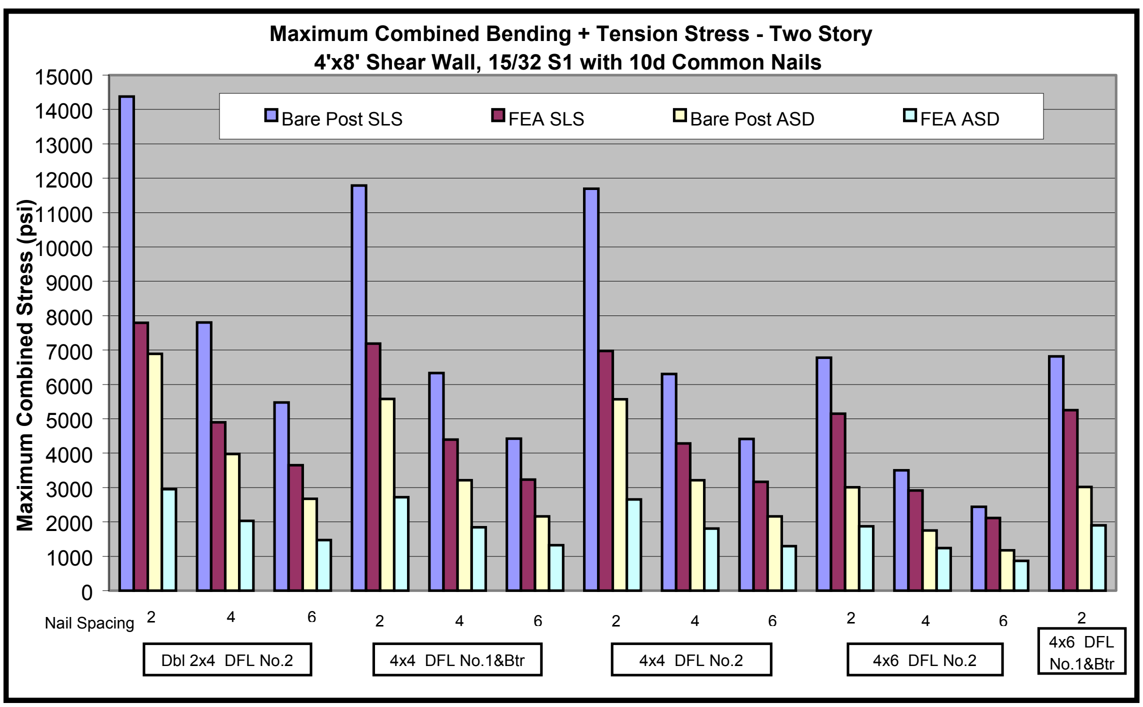


Figure 25: Two Story Maximum Combined Bending + Tension

Strength Reduction

In past discussions the author has participated in regarding the ability of the sheathing/nailing to reduce internal post stresses, it has been suggested that if there is an interaction it will come with the penalty of reducing the peak capacity of the wall. This would be because the nails already have a job to do, i.e. lift up on the post, so if they are required to push sideways on the post they will have a reduced capacity in terms of lifting up on the post. The analysis shows that this is not the case.

The FEA computed force vs. deflection diagrams for all the single story walls are shown in Figure 26 below. The curves with lowest capacity are the 6" on center analysis, and the highest are the 2" on center analysis. As can be seen, there is no appreciable difference between the curves within a given nailing schedule. As previously shown, there are horizontal force vectors in the nails even without eccentric holdowns. So the question isn't if the nails are pulling sideways on the post, but which way they are pulling.

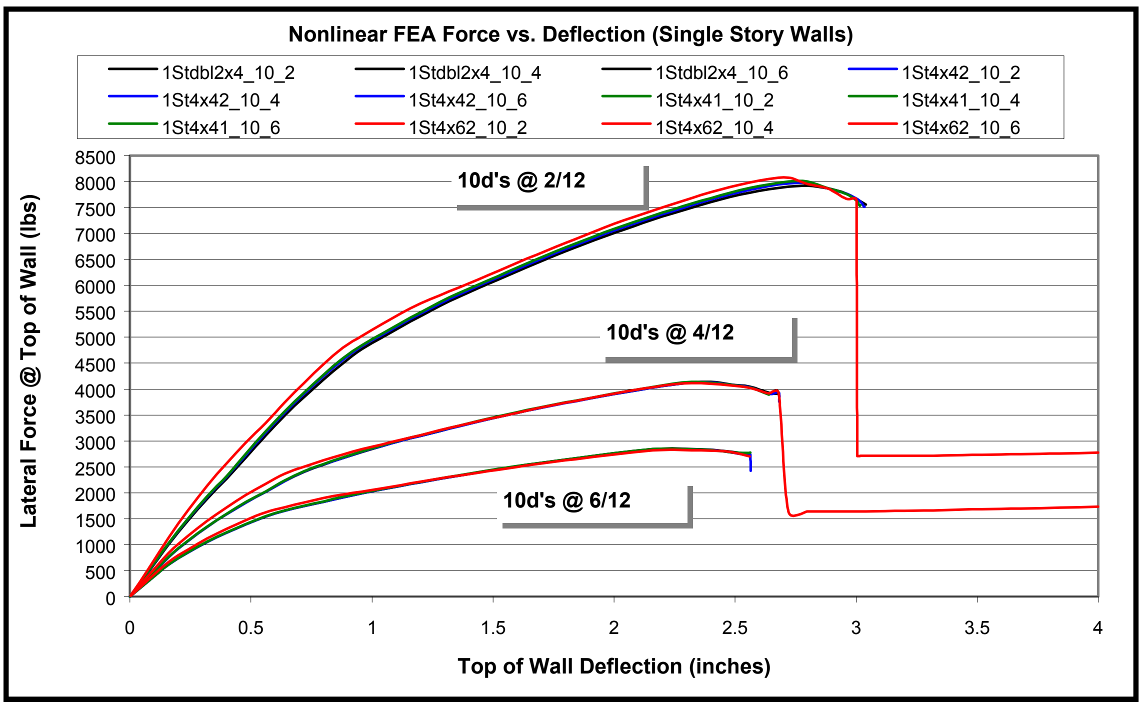


Figure 26: Force vs. Deflection Results for Single Story Walls

IN-GRADE TESTING

The driving force behind the In-Grade testing program was to establish a testing program that simulated, as closely as possible, the way lumber was actually used in construction. To that end, if the location of strength-reducing characteristics in actual construction is a random event, then the location of strength-reducing characteristics in In-Grade testing should also be determined randomly. In this way, the results of the testing would properly reflect the probable strength of lumber in the built environment. The material in this section comes largely from [5], and the reader is referred there for a more complete discussion of this subject.

The test setup created to establish bending design properties was a board simply supported, with equal loads placed at third points. This creates a region of constant moment in the middle third of the span, and was chosen because it was easy to perform while approximating the common situation of uniform loading. Along with the decision to use third point loading, the decision was also made in the United States to ensure that boards chosen for testing would have a worst strength-reducing characteristic for the grade in question somewhere in the span. The Canadians were well underway in their In-Grade testing program and had chosen to test boards with a truly random placement of strength-reducing characteristics, but since the US is such a large market for Canadian lumber, they repeated the tests according to US practice.

When establishing the In-Grade characteristic strength for a given grade of lumber, the strength is established as that value for which 95% of the tested boards are stronger. The basic strength used for ASD design in the NDS is this number divided by 2.1 (2.1 is the product of a load duration adjustment factor of 1.6 and a safety factor of 1.3 on the 5th percentile stress). This statistical 5th percentile load is greatly affected by the probability of having a strength-reducing characteristic impact the outcome of the test. If the test setup for bending created constant moment throughout the length of the board being tested, then the probability of having a strength-reducing characteristic impact the test results would be much greater than a test with a different moment diagram, such as the third point loading. And in turn, a third point loading test has a much greater chance of being affected by a strength-reducing characteristic than say a single, mid-span loading that produces a V shaped moment diagram.

This suggests that the shape of the moment diagram, or pattern of loading, has a large effect on the outcome of the test and on the reliability of engineered design. In fact, this effect has been verified through extensive testing [5:32,237-283] but is not currently addressed in our building codes. Research has also shown that the length of a bending member also impacts the probability of having a strength-reducing characteristic where it can do the most harm [5:274]. The combined effects of load diagram and length can be summed up this way: as the volume of wood subjected to high stresses goes down, so does the probability of being affected by a strength-reducing characteristic in that volume of wood, and, in turn, the strength of lumber goes up. It was concluded that the 5th percentile bending strength increased 43% just for going from a third point bending moment pattern to one seen in a shear wall end post, and there was another increase of about 20% for bending that occurred over short spans [5:281,282].

The length of the piece being tested also affects the 5th percentile tension stress. This is because increased length means an increased probability of a large strength-reducing characteristic being present. This length effect has been quantified in [5:283] and suggests that for small cross sections and lengths less than two meters an increase of 17% to 35% in the 5th percentile tension stress is warranted. This increase, too, is based on the assumption of uniform tension throughout the length of the piece, like a truss chord between panel points. However, in a shear wall the tension in an end post will not be constant - it will vary from zero at the top to a maximum at the top fastener in the holdown (for a single story shear wall). This means that in addition to length, the loading pattern will also affect the 5th percentile tension stress (as it does for bending) because of the reduced probability that the strength-reducing characteristic for tension will be in the region of critical tension at the top of the holdown. While information on load pattern adjustment values for tension could not be found in the literature, it seems reasonable to compare this situation to that of bending where there is constant moment throughout the span versus where there is only a point load at mid span yielding the V shaped moment diagram. For these conditions, the increase in 5th percentile values was 40% [5:281].

There is yet another phenomena that increases the 5th percentile load of commercial lumber. The visual grading rules provide a lower bound on what qualifies as a certain grade. In the process of visually grading lumber, many pieces get included that are actually far better than the grade they are sorted into. In the No. 2 and Better grade, for instance, it has been found that as many as 50% of the boards actually qualify for S.S. [5:67]. Additionally, a piece of lumber may be sorted into a lower grade because of visual characteristics that are not strength-reducing characteristics. The results of the Canadian random strength-reducing characteristic placement testing show no difference in strength at the 5th percentile level between DFL #1 and #2 [5:73]. The combined effects of load pattern, length, and real strength within a grade substantially increase the statistical 5th percentile resistance of commercial lumber.

As was stated in the introduction, there has been criticism of full scale shear wall tests in that the end posts were not chosen and fabricated in such a way as to place the strength-reducing characteristic in the region of highest stress immediately above the holdown. Instead of criticizing, though, we should applaud because this is representative of what happens in the built environment – strength-reducing characteristics are randomly placed, and testing that does not emulate this will not give us any information on the reliability of our designs with lumber.

DISCUSSION AND RECOMMENDATIONS

The data shows that less stiff posts see more support from the sheathing/nailing, and that higher nail densities offer more support to the post regardless of stiffness. If post stiffness is quantified as the product of moment of inertia (I) for the primary bending axis (in the plane of the wall) multiplied by the modulus of elasticity (E) of the post, this stiffness value can then be multiplied by the edge nail spacing (N) to provide an index which segregates less stiff posts with more nailing from stiffer posts with less nailing. Thus we would expect that as I x E x N gets smaller, the bare post method should over estimate the internal post moments more and more. The data does in fact show a strong correlation to this product, and Figures 27 through 30 show these relationships for the one and two story analysis at both the ASD and SLS shear levels. A regression curve that has been fit to the data is also shown, along with the R^2 value for the curve.

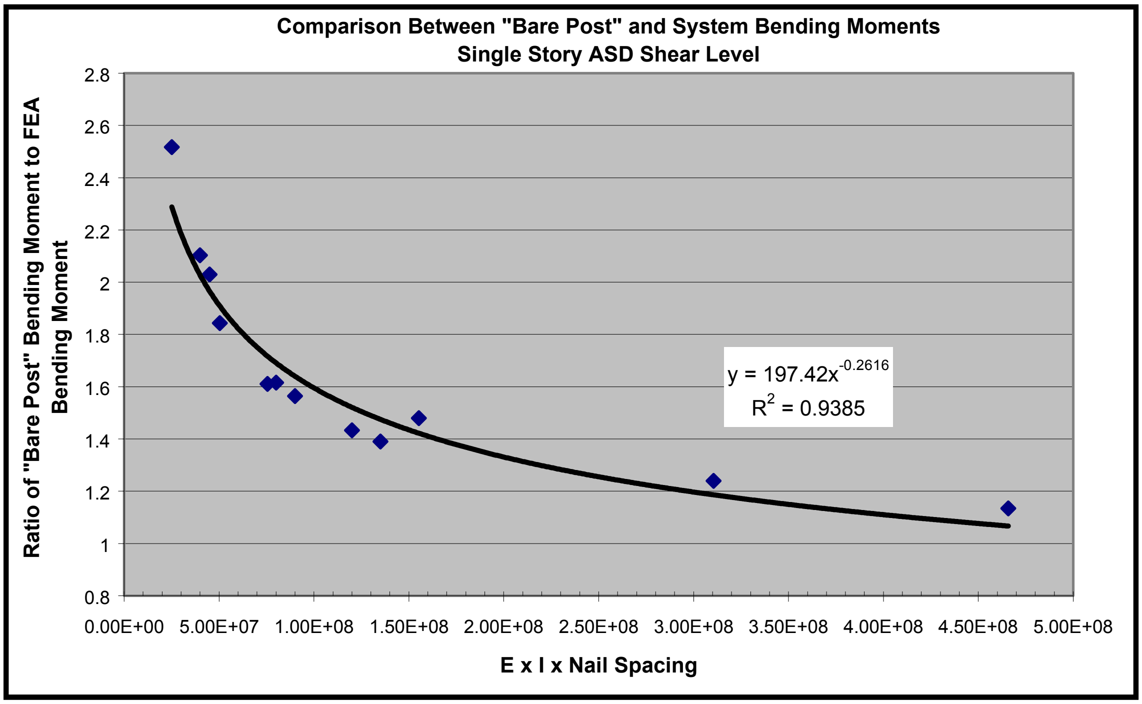


Figure 27: Single Story ASD Shear Level Bending Moment Ratios

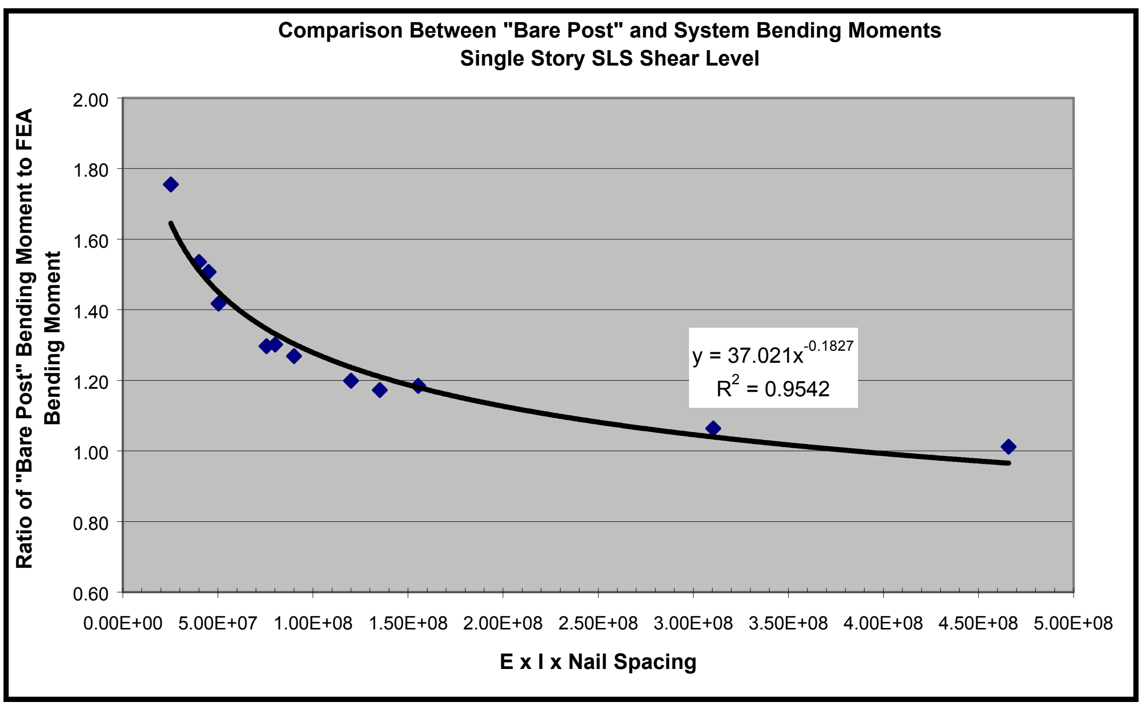


Figure 28: Single Story SLS Shear Level Bending Moment Ratios

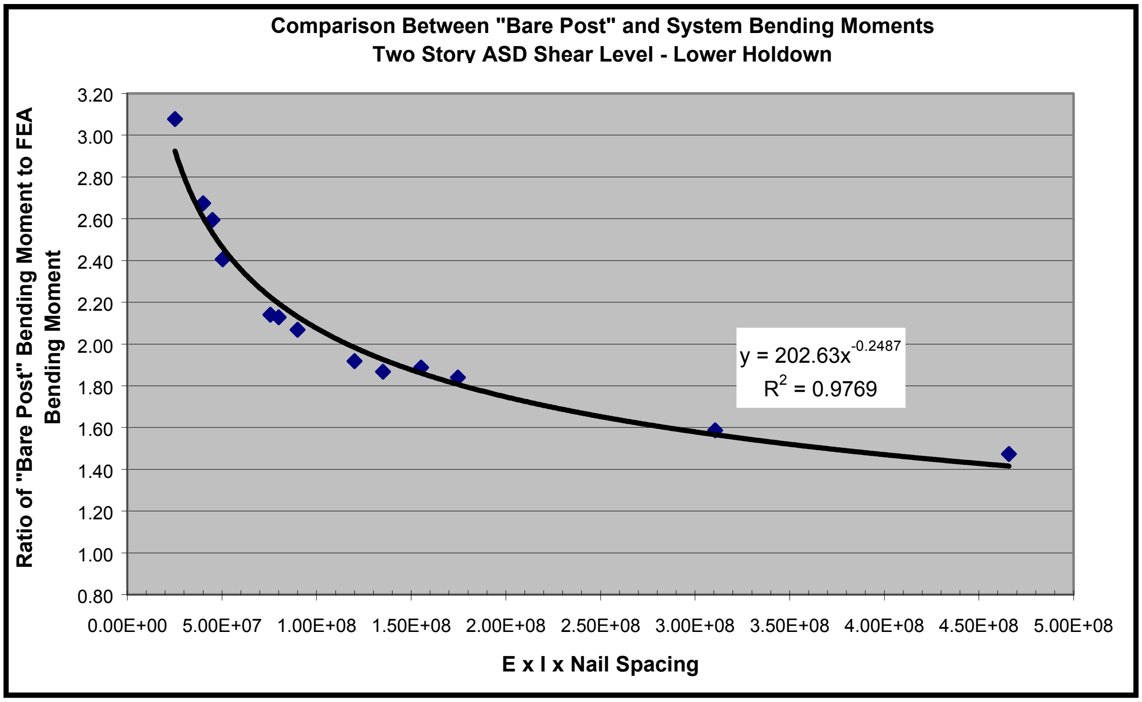


Figure 29: Two Story ASD Shear Level Bending Moment Ratios

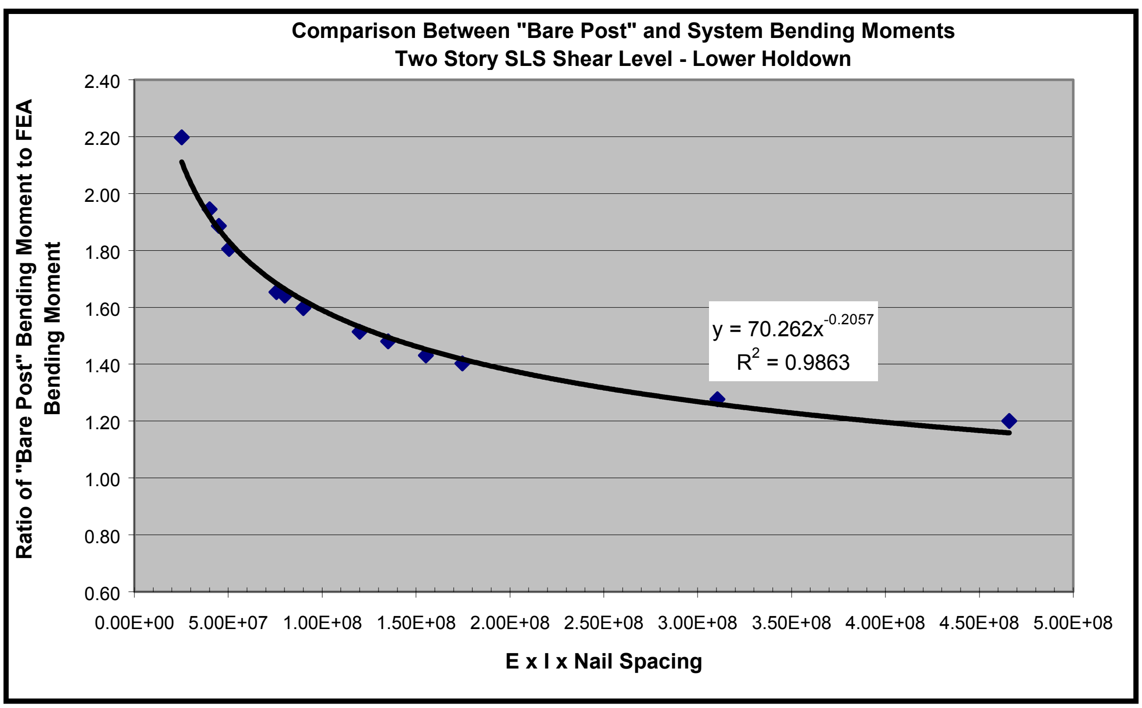


Figure 30: Two Story SLS Shear Level Bending Moment Ratios

From the preceding four figures it is clear that the internal bending stresses at the critical area above the lower holdown can be readily defined by dividing the moment from a bare post analysis by the ordinate of the appropriate figure. As stated earlier, internal post tension is appropriately defined by a bare post analysis. This, then, addresses the "demand" side of the equation.

The figures also show that the sheathing/nailing interaction is somewhat greater at design levels than at strength limit state levels. This creates the problem of choosing which shear level reduction equation to use. The 1997 UBC states that for light-framed shear wall systems, the system overstrength factor, Ω_0, is 2.8 [9:2-32]. This, though, is for demand calculated at a LRFD level. It would have to be increased by an additional factor of 1.4 to make it relevant to the ASD level, which brings $\Omega_{0\text{-ASD}}$ up to a value of 3.92. If one then uses this to estimate the strength limit state of a shear wall to be 3.92 times the ASD capacity the resulting demand predictions will be too high.

In this study the "overstrength" in the analysis, relative to the ASD shear level, varied from 2.0 to 2.32. This is consistent with several hundred full scale shear wall tests the author has witnessed and analyzed that used 10d common nails. The UBC overstrength factor is more consistent with a "system" overstrength factor and not a shear wall component overstrength factor. This is because secondary sheathing elements, such as gypsum board and stucco, will add additional strength to a wall line. Additionally, shear wall end posts will have additional stiffening from the secondary sheathing elements that was not addressed in the study. **This leads to the first recommendation: use the ASD level over prediction equations of Figures 27 and 29, along with the bare post moment procedure of Figures 1 and 17, to estimate the internal bending moments for use in satisfying the NDS combined stress interaction equations.**

Just as important as the modeling work to better define the internal post demand, the discussion regarding In-Grade testing weighs heavily on the issue of why shear wall end posts don't break in epidemic proportions as predicted by a bare post analysis and current NDS tabulated wood strengths. To help visualize the rapidly changing nature of stresses in a shear wall end post, Figure 31 shows a close up picture of the combined bending plus tension stresses in a double 2x4 end post that is part of a single story shear wall with 10d's at 2" on center edge nailing. Also shown in the figure is a length scale so that one can see the distance over which the changes are occurring. The results have units of inches and pounds.

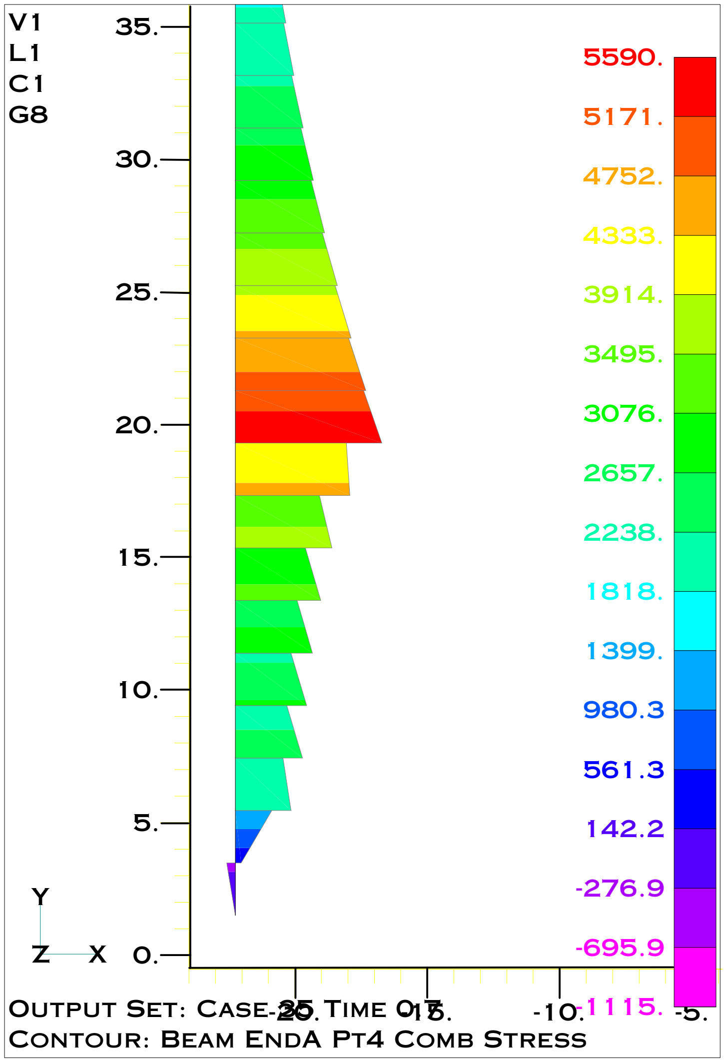


Figure 31: Spatial Distribution of Maximum Combined Bending Plus Tension Stress at the Strength Limit State

In this case, the maximum combined stress at the top of the holdown was 5590 psi. Recall from the discussion on In-Grade testing that the base tabulated value for allowable bending in the NDS (including the size factor and flat use factor as appropriate) is the 5th percentile strength divided by 2.1. For a DFL No.2 2x4, the base bending value would be 1485 psi (900 psi x 1.1 flat use factor x 1.5 size factor). When multiplied by 2.1, the result is a 5th percentile stress of 3119 psi. Based on this, there is only about 10" of wood that is subjected to a stress higher than 3119 psi. Note that in these 10 inches, only a part of the cross section is subjected to tension stresses, and only a portion of that exceeds 3119 psi. This is why accounting for the random nature of the distribution of strength-reducing characteristics, and accounting for the pattern of load, is so important to understanding the real distribution of strength within a grade. If the factors for bending shape and length that were proposed in [5], 1.43 and 1.2 respectively, are applied to 3119 psi, the resulting adjusted 5th percentile bending strength would be 5352 psi, or 4.4% less than the predicted maximum demand at the strength limit state. Also, 5352 psi is a value in which 95 out of 100 boards tested would still be *stronger* than that, if loaded with the same force distribution as seen in a shear wall end post. Given the overstrength of boards within a grade as discussed earlier, it is no wonder why shear wall end post breakage is so rare, even in lab tests. If one adds in the presence of dead load (not present in the analysis and not present in most lab tests) and building interaction effects (secondary sheathing further stiffening the end post and reducing the demand on the shear walls themselves) the probability of failing a shear wall end post restrained by an eccentric device goes down even further.

This leads to the second recommendation: use a load duration factor of 1.6 and post properties based on the net section when checking for NDS compliance at the ASD shear level for combined tension plus bending. Even with an increase of 1.6 on the NDS bending stress, the 5th percentile bending stress will still be 1.3 x 1.43 x 1.2 = 2.23 times higher than that. This provides a balance between the demand at the SLS and the 5th percentile bending stress. Again, dead load and systems effects will serve to decrease the demand, thus increasing the effective margin beyond 2.23.

In rounding out the discussion of results, there are a few more items that should be addressed. While most of the attention of this paper has been on the tension post bending interaction with the sheathing/nail in complete shear walls, the designer must also address the posts for compression demands. In addition to having enough axial buckling capacity, the bearing area of the post must be large enough to satisfy the allowable perpendicular to grain stress given the calculated demand. In the case of the two story analysis using 2" on center edge nailing, this would automatically eliminate anything smaller than a 4x6.

CONCLUSION

The question of proper design procedure for shear wall end posts has been around for some time. The issue has not been whether or not to satisfy the Code requirements for wood design, but rather what is an appropriate force level to use in that design. Prior to this study, no substantiated information on the bending interaction between shear wall end posts and the sheathing/nailing attached to them existed. Consequently, engineers either dismissed the bending effect of eccentric holdowns due to lack of field problems, or they took the other extreme and assumed the worst based on simplistic analysis. As building costs continue to rise, though, the industry must do its part to keep from becoming overly conservative.

In this study, bending and tension demand on shear wall boundary posts restrained against uplift by an eccentric device was investigated using three dimensional nonlinear finite element analysis of complete light-framed shear wall assemblies. Two story situations that involved an additional eccentric holdown at the top of the tension post were also investigated. The bending interaction between the tension post and the sheathing/nailing in complete shear wall systems was investigated and quantified showing a significant reduction in the internal post moments when compared to a simplified analysis that would ignore this interaction. For the two story case, this interaction was even larger for the top holdown area than the bottom holdown area. The magnitude of internal tension and moment at the critical post section was quantified and found to follow a distinct trend for various combinations of end posts and nail spacings.

A review of the In-Grade testing program was presented with information showing that the load pattern within a shear wall tension post will allow for a substantial increase in the 5th percentile strength of lumber, both in bending and in tension. When taken together, these increases in resistance along with the more accurate estimates of demand explain why end post failure in complete shear wall tests is a rare event. Recommendations were made for establishing the compliance of shear wall tension end posts with the NDS.

Recommendations for practicing engineers were made for the conditions outlined in this paper. Additional research with other sheathing/nailing sizes is planned to investigate the use of 8d nails and thinner sheathing. While it is expected that the end post moment reduction will decrease over that found in this study, so too will the uplift demand decrease from the use of smaller nails and thinner sheathing.

REFERENCES

1. American Forest & Paper Association, General Dowel Equations for Calculating Lateral Connection Values – Technical Report 12. American Forest & Paper Association, Inc., 1999.

2. ANSI/AF&PA NDS-1997. American Forest and Paper Association. Washington, D.C., USA.

3. Fonseca, Fernando S., Rose, Sterling K., Campbell, Scott H. (2002) CUREE-Caltech Woodframe Project Task 1.4.8.1 – Nail, Wood Screw and Staple Fastener Connections. http://www.structures.ucsd.edu/andre/Task1-4-8-1%20Final%20Report%20(Fonseca).PDF

4. Hamburger, Ronald O., Nelson, Ronald F. "Holdown Eccentricity and the Capacity of the Vertical Wood Member." Building Standards (November-December 1999), 27-28.

5. Madsen, Borg. Structural Behaviour of Timber. North Vancouver: British Columbia Lumber Engineering Ltd., 1992.

6. Nelson, Ronald F. "Hold-Down Connectors and Wood Member End-Post Capacity." Structure (September 2001), 36-41.

7. NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000. Building Seismic Safety Council, Washington, D.C., USA.

8. Ship, John G., Erickson, Todd W., Rhodebeck, Mike. "Findings from Cyclic Testing of Plywood Shear Walls." Structure (March 2001), 27-31.

9. Uniform Building Code 1997 Volume 2. International Conference of Building Officials, Whittier, CA, USA.