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The effect of eccentric overturning restraint in complete shear wall assemblies

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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, three dimensional 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 that this reduction is proportional to post stiffness and nail spacing.

1 INTRODUCTION

There has been much discussion recently in the USA concerning the proper design of end posts in light-framed wood shear walls. The end posts serve as shear wall boundary members and resist the tension and compression caused by overturning forces. This paper deals only with the forces in the tension end post and how to estimate them. Traditional manufactured overturning restraint systems (holdowns) attach to the side of the post with mechanical fasteners. Because they attach to the side they are eccentric to the post, and this eccentricity causes bending moments in the post in addition to the tension caused by the overturning forces. At issue is whether or not the sheathing that is nailed to the post, and causing the post tension, offers any resistance to the post bending forces induced by eccentric holdowns. If the sheathing/nailing does not aid the post in any way, then the end posts should be designed considering the full eccentric moment in the determination of internal stresses.

Extensive full scale shear wall testing has been performed that suggests that the effect of bending due to holdown eccentricity is minor or does not control for the configurations tested (Ship, Erickson and Rhodebeck, 2001). 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 (Nelson, 2001). It has also been suggested that the state of stress in the post can be readily ascertained using simple statics (Hamburger and Nelson, 1999). This is true only if the force vectors from each nail attached to the post are properly determined and applied to a static free-body diagram. 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 (Nelson, 2001).

The lack of an epidemic of laboratory or real world shear wall post failures in posts that appear to be overstressed by factors between 5 and 10 suggests to many engineers that the analysis techniques used to determine the demand must not be adequate for the problem. To date, estimates of post bending and tension demand have come from very simple analytical models of what is really a highly statically indeterminate condition. To better understand the demand on the end posts, nonlinear finite element analysis (FEA) 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 ultimate capacity, referred to as the strength limit state (SLS). This was then compared to the demand that is predicted by the simplified model that assumes that the sheathing/nailing provides no support to the post in resisting moments caused by holdown eccentricity (the “bare post” approach).

2 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 stress in addition to the direct tension stress. 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. 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 to the right, and the tension end post is on the left. Figure 1 shows the bare post free-body diagram for double 38 mm x 89 mm stud end post removed from an 2.44 m tall wall which carries a design shear of 12.7 kN/m at the top of the wall. 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 30.99 kN. 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 traditional bottom-of-wall placement.

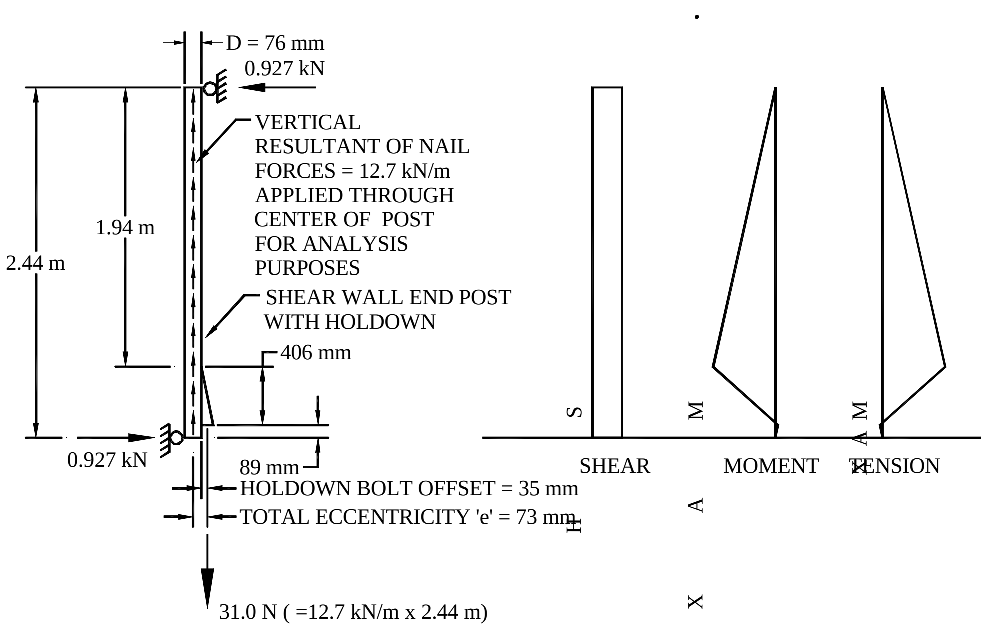


Figure 1: ASD Shear Level Free-body and Force Diagrams for Tension End Post Using Bare Post Approach

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 (AF&PA, 1999). 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, 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 and are included explicitly in the modelling used to support the findings in this paper. In any event, these issues do not 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.

3 FINITE ELEMENT MODELING

A commercially available finite element analysis package, MSC visualNastran for Windows, was used to investigate the internal forces in complete shear walls using nonlinear static analysis procedures. Three dimensional FEA models were developed for complete shearwalls and compared to full-scale cyclic test results. Like the premise of the bare post analysis of Figure 1, the full scale tests were of a wall 1.22 m x 2.44 m, framed with DFL wood (double 38 mm x 89 mm #2 end posts, 64 mm x 89 mm pressure treated sill), with 12 mm Structural 1 OSB and 10d common nails (3.76 mm x 76.2 mm) at 51mm on center at the edges and 305 mm on center in the field, and Simpson Strong-Tie PHD8 holdowns for overturning resistance, which gives the wall an ASD level code design rating of 12.7 kN/m (Uniform Building Code, 1997).

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 (Fonesca, Rose and Campbell, 2002), 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 could simulate lift off of the sill plate. 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 73 mm (35 mm for the holdown itself plus 38 mm for one half of the post depth). Figure 2 shows the typical results of the modeling compared to the test results.

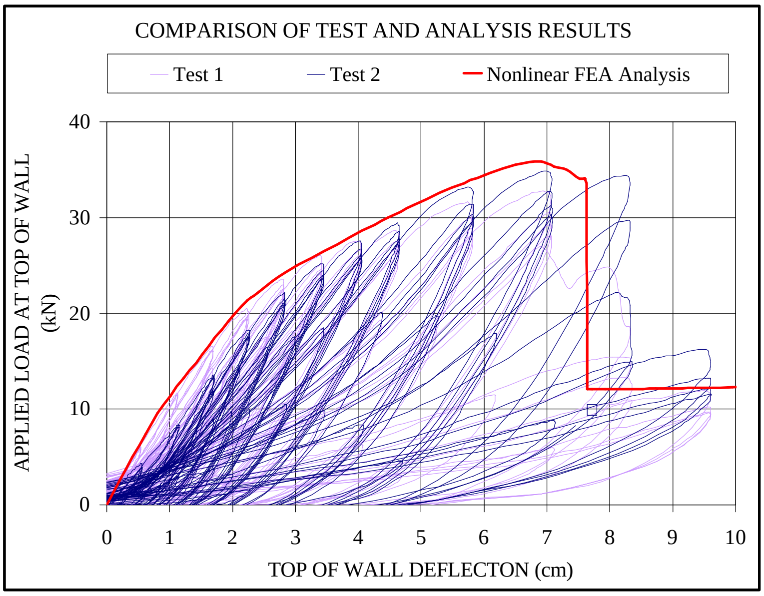
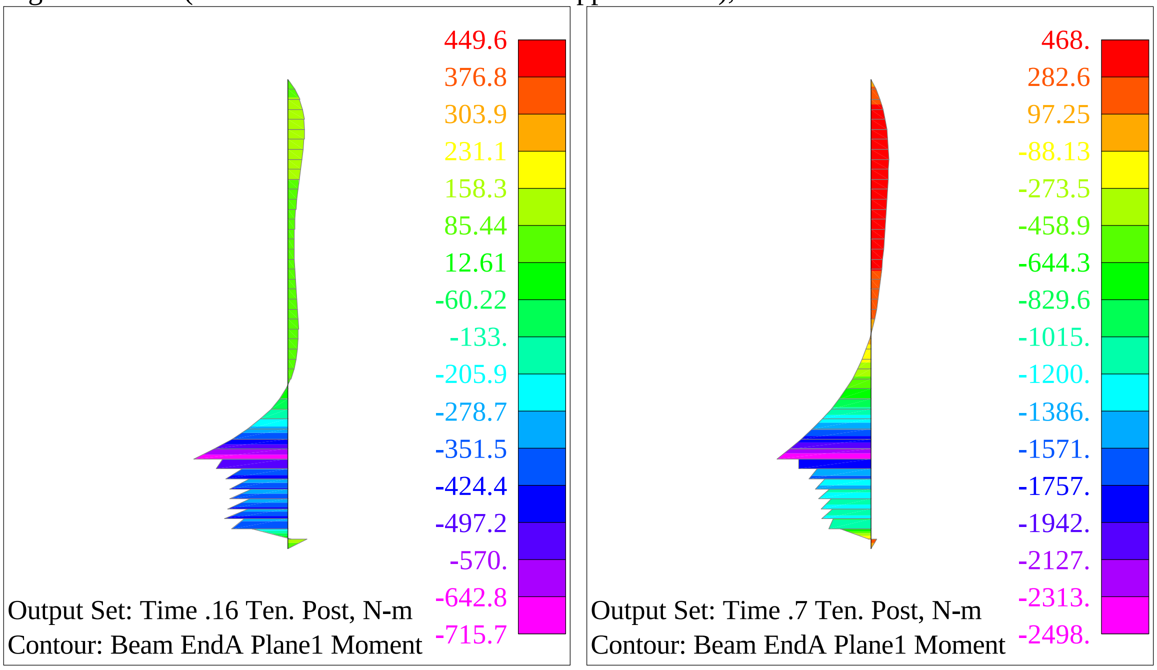


Figure 2: Testing vs. Analysis Results – Wall Stiffness and Capacity

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 3 shows the bending moment diagram for the shear wall tension post at both the design shear rating of the wall (12.7 kN/m * 1.22 m = 15.5 kN applied shear), and when the wall reached its SLS.



a: ASD Shear Level   b: SLS Shear Level

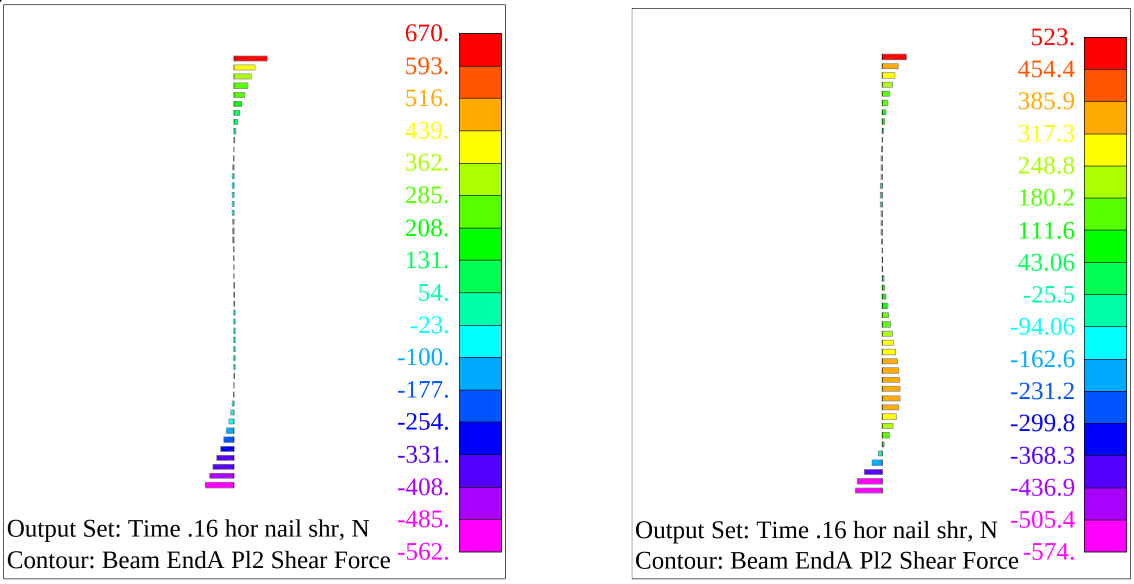
Figure 3: Tension Post Moment Diagrams (N.m)

Comparing Figures 1 and 3, 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 at both the ASD level and at the SLS level. 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, 1800 N.m vs. 715.7 N.m. The bare post analysis approach significantly over predicted the internal bending moment at the design level. At the SLS, the holdown tension was computed to be 75,300 N. This would create a bare post moment of 4383 N.m, which is 75% higher than that shown in Figure 3b for the complete shear wall assembly. Thus, even at the SLS, there is still significant interaction between the sheathing/nailing and the post that reduces the internal bending moments. The reason for the reduction in moment can be seen in the horizontal component of the nail shear forces, which differ from a concentric holdown case to an eccentric holdown case due to the flexibility of the end post.

It should be mentioned that no such overestimate applies to the tension developed in the post. When the FEA model reaches 31,000 N of tension in the holdown, the corresponding maximum tension is in the first post element above the holdown and has a value of 24,900 N, or 260 N 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.

4 HORIZONTAL NAIL SHEAR

Unlike a concentric holdown, when an eccentric holdown is used to restrain post uplift the applied moment creates curvature in the post. This curvature causes the post to interact with the sheathing/nailing in a different way than it would were it restrained with a concentric holdown. This is clearly seen by comparing the magnitude of the horizontal shear components in the nails for the concentric and eccentric cases. Figures 4a, b show the magnitude and direction of the horizontal component of nail shear for the two cases. 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 suggested by Nelson (Nelson, 2001), 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. Horizontal nail shear is developed even in the concentric holdown case because of the relative rotation of the sheathing with respect to the post. Thus even concentric holdowns create moments in the post, and in this case the maximum post moment at the ASD design level was about one third of the moment for the eccentric case shown in Figure 3a.



a: Concentric Holdown (Dbl 38mm x 89 mm Post) b: Eccentric Holdown (Dbl 38mm x 89 mm Post)

Figure 4: Horizontal Component of Nail Shear in Tension Post Over Height of Post

5 ANALYSIS RESULTS

5.1 Single Story

Four different post size/grade combinations, along with three different edge nail spacings, were analysed resulting in twelve different wall configurations. From an analysis standpoint, the modulus of elasticity (E) changes with different grades of lumber. Each of the walls 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 these 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 3a, b. 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 affected.

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 5. For both the ASD and SLS levels, the bare post moment over prediction is larger for smaller, lower E posts and higher nail densities. This makes sense because smaller (i.e., lower moment of inertia, I, for bending in the plane of the wall), lower E 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 double 38 mm x 89 mm DFL No. 2 posts (which did not fail in the full scale testing). To aid designers in estimating internal bending stress, Figure 6 was developed. It shows a well-fit regression curve that can be used to estimate the over prediction of internal bending moment by the bare post method (which is easily performed). Thus, actual internal moment is estimated by dividing the bare post moment by the value of y from the regression equation.

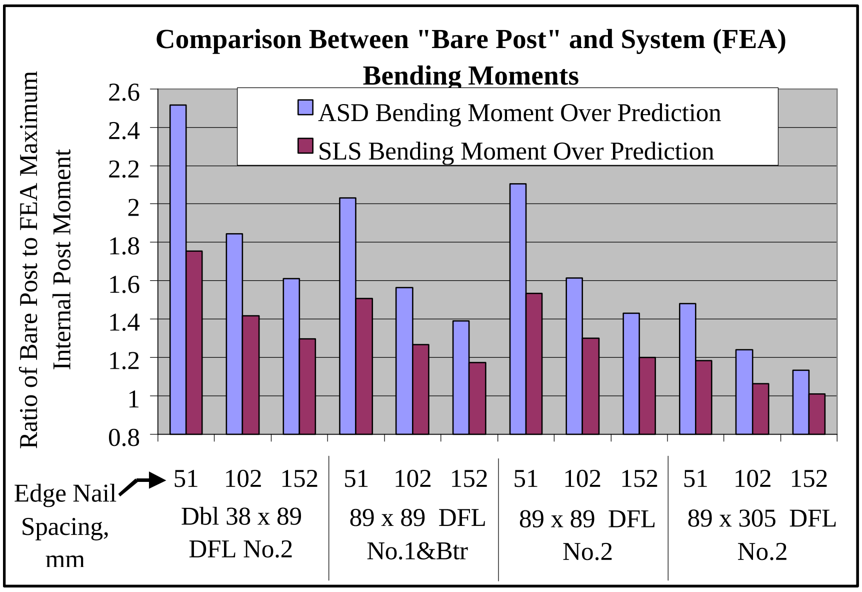


Figure 5: Ratio of bare post Bending Moments to System Bending Moments

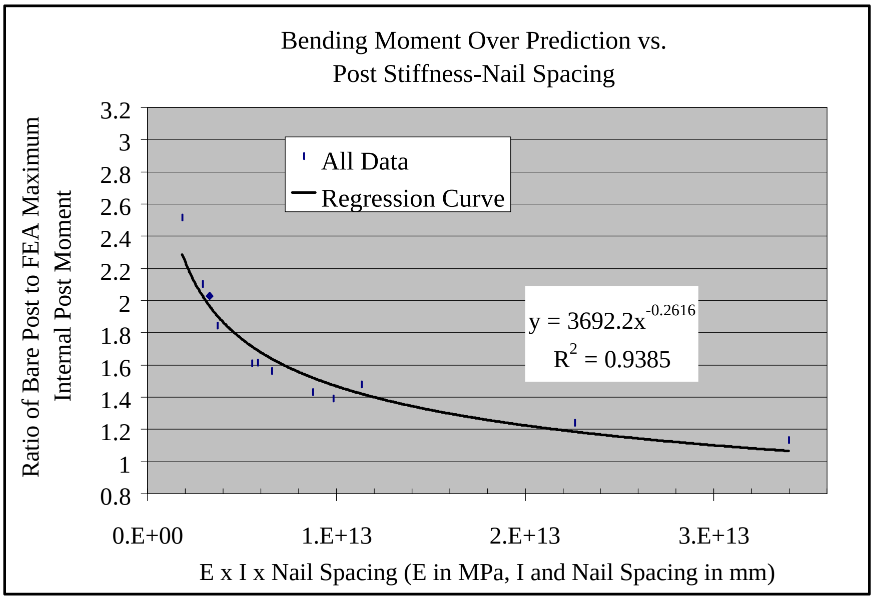


Figure 6: Relationship Between Post Stiffness, Edge Nail Spacing and Maximum Internal Post Moment

5.2 **Two Story**

For two story walls, the analysis was repeated with an additional eccentric holdown 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 (Uniform Building Code, 1997). 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, 89 mm x 140 mm DFL No.1&Btr with 10d edge nailing at 51 mm on center was added for the two story analysis. Similar to Figure 1, the bare post free-body and force diagrams for the two story condition was developed but is not shown here. Figures 7, 8 present the same information as shown in Figures 5, 6, but for the maximum moment above the lower holdown in the two story analyses.

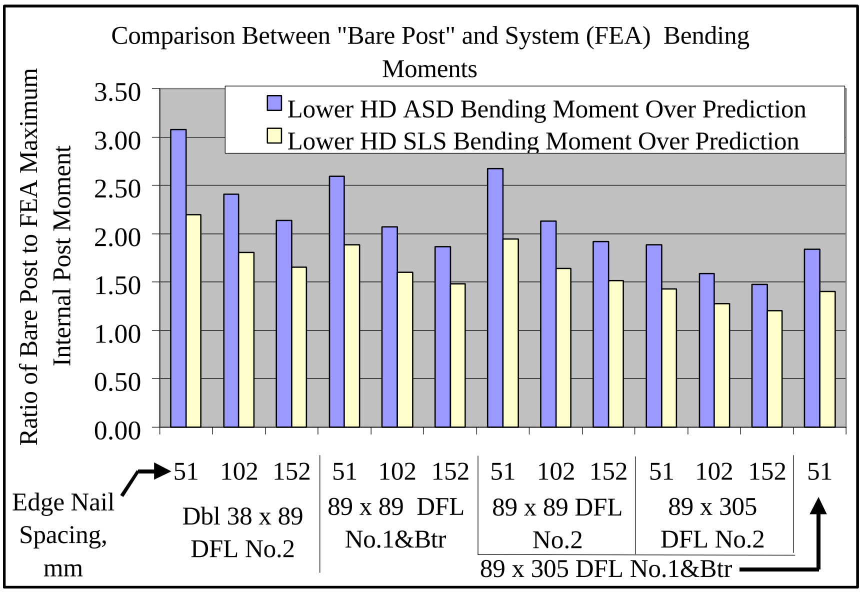


Figure 7: Ratio of Bare Post Bending Moments to System Bending Moments

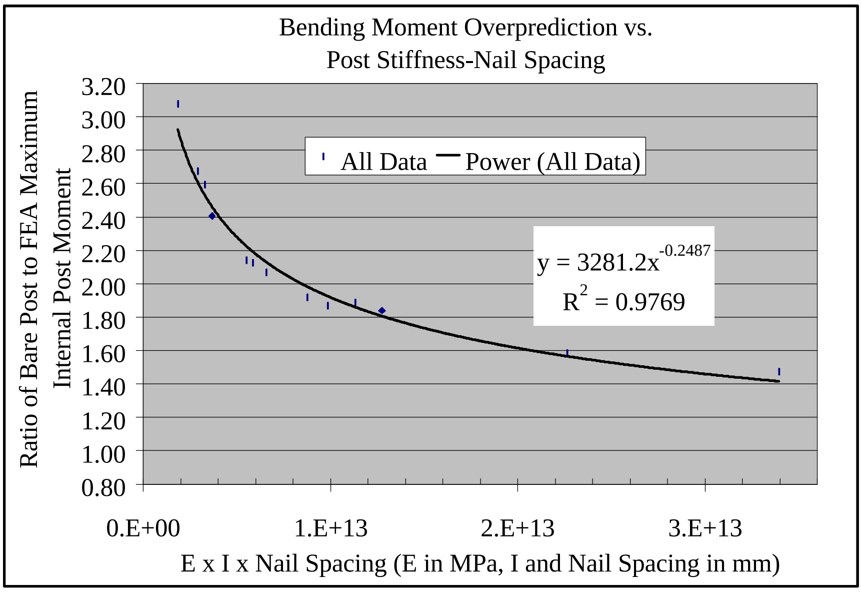


Figure 8: Relationship Between Post Stiffness, Edge Nail Spacing and Maximum Internal Post Moment

Similar results for the top holdown region of the post are not shown here because it doesn’t control the design of the post, and the bare post free-body diagram overestimates the internal post moment significantly more than it does at the bottom holdown region of the post. At the ASD level, the over prediction ranged from a factor of 3.38 to 5.62, and at the SLS it ranged from 2.11 to 4.39.

6 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 a 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 found to follow a distinct trend for various combinations of end posts and nail spacings. Equations were presented that allow the practicing engineer to easily and accurately estimate the internal bending moment by adjusting the results of a simplified free-body analysis of the tension post.

Additional testing and analytical 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. As more information becomes available it will be posted at www.strongtie.com.

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