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PROCEDURE MODIFICATION

Modification of the MCA Procedure for Strength and Stiffness of Diaphragms used in Post-Frame Construction

Introduction: The Metal Construction Association in 2004 published a manual that extended the diaphragm research conducted for the Steel Deck Institute by Dr. L.D. Luttrell to steel framed light-gauge metal clad buildings. The manual was titled 'A Primer on Diaphragm Design' by L. D. Luttrell and J. A. Mattingly (2004). The MCA method presented in the manual determines diaphragm strength as the smallest strength for field fasteners, panel (sheet) corner fasteners or panel out of plane buckling. The stiffness of the diaphragm is dependent on the warping of the panel at the ends due to shear flow along the diaphragm perimeter and the corrugated sheet profile, the shear strain of the panel material and the flexibility of the fasteners attaching the panel to the frame and to other panels at the overlapping panel edges. The MCA method has been successfully and widely used in the steel building industry.

In the mid 2000s, interest was expressed by the NFBA T&R committee about adapting the MCA method to post-frame construction. Successful adaption of the MCA method to post-frame buildings would reduce the need for testing light-gauge metal sheathed lumber framed diaphragms used in post-frame construction which reduces building costs and gives design engineers more flexibility to develop diaphragm systems that meet the strength and stiffness requirements of the building being designed. Before the MCA method could be applied to post-frame construction it was necessary to determine if the method could predict the strength and stiffness of light-gauge metal sheathed diaphragms on lumber frames established by testing accurately. To this end G. A. Anderson (South Dakota State University) and J. A. Leflar (Colorado State University graduate student and a Senior Equipment Engineer at Avago Technologies) undertook the task to determine if the MCA method could accurately predict the strength and stiffness of diaphragm systems used in post-frame construction and develop modifications to the MCA method as and when required so light-gauge metal clad lumber framed diaphragm strength and stiffness from testing could be accurately predicted. The work is reported in Leflar's thesis titled 'A Mathematical Model of Steel-Clad Wood Frame Diaphragms' submitted in 2008 at Colorado State University. The MCA method with modifications accurately predicted the strength and stiffness of 25 different diaphragm construction made with five different light-gauge metal panels (generally three replication of each construction). The strength had a ratio (Modified MCA method/test) of 0.98 with a coefficient of variation of 16 percent while the stiffness ratio was 0.97 with a coefficient of variation of 23 percent. The ratios for strength and stiffness are approximately 1.00 and the coefficients of variation are within the realm of the coefficients of variation for diaphragm test data (15 percent for strength and 25 percent for stiffness), thus validating that the Modified MCA method with modifications accurately predicts the strength and stiffness of light-gauge metal clad diaphragms on lumber frames as determined by testing (Leflar 2008).

The objective of this paper is to present the significant modification to the MCA method that yielded the good validation results and to briefly discuss the modifications. The terminology and symbols used in the paper are taken from the MCA manual so readers can refer back to the manual and compare the modifications to the material in the 'Primer on diaphragm Design.'

Panel strength properties:

The MCA method limits the ultimate strength and the yield strength of the steel used to make the panels to 65ksi and 60ksi respectively. The modified method allows the designer to use the yield and ultimate tensile strength of the panel steel. Panels used in post-frame construction often are made of thin high-strength steel. The high strength of the steel offsets the smaller thickness resulting in similar strength to thicker sheet with a lower ultimate and yield strength. Limiting the ultimate and yield strength would penalize the light-gauge metal sections used in post-frame construction.

Safety factor:

The MCA method uses a safety factor of 3 if the predicted failure involves wood; field fasteners or panel corner fasteners. Past experience in the industry has shown a safety factor of 2.5 has been satisfactory for this type of construction. The mean predicted strength ratio was 0.98 indicating the Modified MCA method yields the same result on average as the tests do. The test data also has variation similar to that found for the strength ratio. The method is modified to use a safety factor of 2.5 for field fastener and panel corner fastener strength. The safety factor for buckling has been left as recommended by the MCA manual: 2.0.

Flat width to form a pitch:

The flat width to form a pitch (s) has been increased by 0.125 inches to account for the material used to form the minor ribs and any material used in small corrugation on the top of the rib. Increasing "s" causes the ratio s/p (p-corrugation pitch) to increase thereby decreasing the shear stiffness of the diaphragm.

Field Fastener Strength:

The MCA manual provides an equation for each size of field fastener for both wood failure and steel failure (0.145 inches diameter nail, #9, #10, #12, and #14 screws). Rather than providing an equation for each size of fastener for wood failure, the method was modified to yield one equation.

$$Q_{fw} = 32\ Gd^2$$

Where G is the specific gravity of the wood (purlin) and d is the diameter of the fastener. The modification allows the method to be expanded to other dowel fastener sizes than presented in the MCA manual. The equation also provides the same strength as does the MCA manual multiple equation system.

Similarly, the equations for sheet metal failure around the fasteners for #10 screw size and smaller has been modified.

$$Q_f = 2.22F_u dt$$

Where F_u is the ultimate tensile strength of the steel panel material and t is the thickness of the panel. The equation more readily represents the bearing strength of the steel panel and allows a larger variety of dowel fastener sizes to be used. It also provides the same strengths as the MCA manual equations do for the same fastener size. The MCA equation for #12 and #14 screws has not been modified. The same equation is used for both #12 and #14 screws. The equation presented in the MCA manual can be used for dowel fasteners larger than #14 screws.

Elevated sidelap structural fasteners anchored in wood:

The MCA manual provides equations for 0.145-inch nail, #9 and #10 screws that pass through both ribs at the sidelap and penetrate into the purlin. This connection is limited in location to where two sheets over lap over a purlin. The modification for the screw fasteners is:

$$Q_f = 2.22F_u dt$$

The reader may notice the above equation is the same equation as given for field fastener strength bearing on the panel steel in the proceeding section. When the fastener passes through both ribs into the purlin, the fastener is prevented from rotating leaving the fastener to bear on each of the over lapping panels that are moving in opposite direction due to the displacement of the frame (purlins). The equation for Q_f presented is a bearing strength equation for cold formed steel making it appropriate in this case. Since the failure for the elevated sidelap structural fastener anchored in wood is a bearing failure, it is reasonable to extrapolate the bearing field fastener strength for #12 and larger screws. The elevated sidelap structural fastener anchored in wood strength therefore is:

$$Q_f = 1.25F_y t(1-0.005F_y)$$

Where F_y is the tensile yield strength of the panel material. The equation is applicable to screws #12 and larger.

In the case of elevated sidelap structural fasteners anchored in wood, screws and nails do not behave in a similar fashion. Self drilling screws tend to cut a smooth surface in the panel metal which provides a smooth bearing surface for the screw shank on the panel. Nails driven through the panel tend to puncture the panel leaving holes with cracks radiating from the center of the hole outward. The shank of the nail then bears on the jagged surface and may more readily fail the panel by propagating the cracks thus formed. The equation for nails is:

$$Q_f = 1.5F_u dt$$

The reduced strength for nails in the above equation is not seen in the equations for field fasteners. A possible explanation is when the nail is driven through a single sheet in contact with the purlin, the pieces of panel pushed downward as the nail penetrates go into the purlin wood which tends to support the edges of the punctured panel and the punctured panel pieces in the wood may actually help transfer load between the panel and the purlin thus reducing the effect of the punctured panel.

The first and third equations presented in this section yield elevated sidelap structural fastener anchored in wood strengths within 3 percent of the MCA manual equation for an ultimate tensile strength of 82ksi. An ultimate tensile of 82ksi is common for cold formed light-gauge metal panels. With the fastener strength being within 3 percent when an ultimate tensile strength of 82ksi is used supports the proposal of using ultimate and yield tensile strengths of the panel material rather than limiting it to 60ksi (yield) and 65ksi (ultimate).

The MCA manual requires the fastener penetrate 7d into the wood for full design strength and that the fastener must penetrate 4d into the wood in order for it to have any strength. The design strength is to be reduced proportionally for wood penetration depths between 7d and 4d. The author's past experience has not included witnessing an elevated sidelap structural fastener anchored in wood failing because of lack of fastener penetration into the wood. Dowel type fasteners in wood tend to transfer shear with little or no bending moment between the two connected parts. Dowel fasteners in elevated sidelap structural connections anchored into wood tend to transfer no shear between the metal and wood connected parts, but do transfer a bending moment:

$$M = Pt$$

Where P is the shear load transferred from one sheet to the other sheet in the overlap.

The moment calculated above is applied to the dowel fastener at the surface of the wood. The loading is more analogous to a post embedded in soil with a ground line moment than it is to a shear test of a dowel connection in wood. Assuming the stress in the wood is due to the applied bending moment can be treated as two equal areas (one half the penetration depth) with a constant stress and equating it the dowel bearing strength (Q_{fw}) while the panel force P is equated to the bearing strength of the panel and solving for n_d (required number of diameters the fastener must penetrate the wood so the full bearing strength of the panel can be achieved) gives:

$$n_d^2 = 0.2775(F_u/G)(t/d^2)^2$$

The above equation is for #10 and smaller fasteners. For #12 and larger fasteners the equation becomes:

$$n_d^2 = (5F_y t^2(1-0.005F_y))/(32Gd^5)$$

The penetration depths ($n_d d$) found from the equations above should be treated as minimum values that allow the full bearing strength of the connected panels to be developed. For thin panels (29 gauge) the required penetration depth is much less than 7d. It is recommended that a minimum penetration depth be set at 2d which would apply for wood with a high specific gravity and/or thick panels.

Contribution of fasteners:

The contribution of fasteners term B, has been expanded so there may be three fastener patterns on the sheet; one edge (end of the sheet) and two interior ones. The field fasteners at this time must still be the same though. The addition of the second interior fastener pattern will allow two fastener patterns to be used on the interior purlins allowing design engineers more options when establishing fastener patterns for the required diaphragm strength and stiffness. It also will allow diaphragms to be analyzed that are continuous over the ridge because the ridge purlins can be treated as the second fastener pattern and include additional field fasteners that attach the ridge cap. The contribution of fasteners is:

$$B = n_s\alpha_s + 2n_p\alpha_p^2 + 2n_{p2}\alpha_{p2}^2 + 4\alpha_e^2$$

Where n_s is the number of stitch connections, α_s ratio of Q_s/Q_f, Q_s is the stitch connector shear strength, n_p is the number of interior purlins with fastener pattern 1, α_p^2 is interior fastener weighting factor for fastener pattern 1 squared, n_{p2} is the number of interior purlins with the second fastener pattern, α_{p2}^2 is the interior fastener pattern weighting factor squared, and α_e^2 is the weighting factor for the end panel fastener pattern squared.

Fastener flexibility coefficient:

The fastener flexibility coefficient (C) must be adjusted to account for the affect two different interior fastener patterns have on diaphragm stiffness. The modified equation is:

$$C = ((Ets_f/w))((24L/(2\alpha_e + n_p\alpha_p + n_{p2}\alpha_{p2} + 2n_s(s_f/s_s))$$

Where E is the modulus of elasticity of the panel material, s_f is the structural fastener flexibility, w is the width of a panel, L is the diaphragm length, α_e is the end fastener pattern weighting factor, α_p is the field fastener weighting factor for the first interior purlin structural fastener pattern, α_{p2} is the weighting factor for the second interior purlin structural fastener pattern, and s_s is the stitch connector flexibility.

Structural connectors per unit length on end purlins:

The fasteners on the end of the diaphragm transfer shear parallel to the purlins due to shear flow around the boundary of the diaphragm and shear parallel to the major panel ribs. The two shears are summed as vectors. The shear parallel to the end purlins is assumed to be shared equally between all structural fasteners in the panel end while the shear parallel to the major panel ribs is assumed to be greatest at the fastener closest to the side of the sheet thus making the fastener closest to the panel corner the most heavily loaded fastener. The elevated sidelap structural fastener anchored in wood at the panel corner is assumed to carry shear parallel to the end purlin in a similar fashion as it does carrying shear between the overlapping panels parallel to the panel major ribs. The structural connectors per unit length on the end purlin is modified to include the sum of all structural fasteners in the flats on the end purlin and elevated sidelap structural fasteners anchored in the end purlin wood divided by the panel width.

Shear connectors/blocking:

The diaphragm analysis is based on the assumption the structural fasteners are equally weighted on each side of the panel center line parallel to the major ribs. Shear connectors/blocking make the panel stiffer and stronger on one side of the panel compared to the other. The number of panel to shear connector fasteners is divided by the number of panels to include partial panels in the diaphragm width. The result is termed the number of phantom purlins in the diaphragm. Each phantom purlin is assume to have a structural field fastener at the same distance from the panel center line as the shear connector fasteners are from the panel center line. The fastener weighting factor is calculated (squared term too) for the one fastener and averaged with the weighting factors for the interior purlins relative to the number of interior purlins and phantom purlins.

$$\alpha_{pa} = (n_p\alpha_p + n_{pp}\alpha_{pp})/(n_p + n_{pp})$$

Where n_{pp} is the number of phantom purlins and α_{pp} is the weighting factor for the phantom purlin fastener. The same procedure is used to find the squared weighting factor (α_{pp}^2) from the squared interior and phantom purlin weighting factors.

The terms α_{pa} and α_{pa}^2 are then used to determine the flexibility coefficient and contribution of fasteners factor in place of α_p and α_p^2. It should be noted the Modified MCA method can be used to predict test diaphragm strength and stiffness for assemblies that have two seams or overlapping sheet sides which provides one interior panel to act as assumed in the MCA method equation development.

Structural fastener flexibility:

Leflar (2008) evaluated data from Kelley and Anderson (1995 and 1996) and determined the structural field fastener flexibility over a range of fastener sizes, sheet thickness and wood densities was relatively constant. The modification to the MCA method is to set s_f to a constant value of 0.2in/kip. This value is an order of magnitude more flexible than predicted by the MCA method.

Elevated sidelap structural fastener anchored in wood flexibility:

Elevated sidelap structural fasteners anchored in wood are not free to rotate since the purlin prevents rotation. The flexibility of the fastener was set equal to that of a stitch fastener in relative thick panel material that would also inhibit fastener rotation.

$$s_{purlin} = 3/(1000(t)^{1/2})$$

Stitch screw flexibility:

Stitch screws in thin metal are expected to rotate thereby loosing stiffness or increasing flexibility of the connection. Evaluation of trial analysis data indicated the flexibility of the stitch fastener was 1/3 greater than the flexibility of the fasteners in the proceeding section.

$$s_s = 4/(1000(t)^{1/2})$$

The flexibilities s_{purlin} and s_s are averaged together relative to the number of each type of fastener in the sidelap as was done with the weighting factors (α) when phantom purlins were used.

Corrugations between fasteners on panel ends:

The ends of the panel tend to warp because of the shear flow along the perimeter of the diaphragm. Fasteners around the ribs tend to reduce panel end warping. The counter V was found by trial and error to be 1 for fasteners on on both sides of a rib and to be 1.6 with a fastener on one side of the major ribs. The fasteners should be placed close enough to the rib so the warping rib does not lift up panel material in the flat area between the ribs. Also, elevated sidelap fasteners anchored in wood at the panel ends were not evaluated. The elevated fastener would likely restrain the rib from warping as much or more than a fastener on each side of the rib. Therefore, V should be 1 for a rib with an elevated sidelap structural fastener anchored in wood in the end purlin.

Purlin-rafter connection and shear connector contribution to test diaphragm stiffness:

Test diaphragm assemblies often include purlin-rafter and shear connector slip. These stiffness components are incorporated into the diaphragm stiffness. The purlin-rafter connection stiffness is multiplied by the number of purlin-rafter connections and added to the number of shear connectors multiplied by the shear connector stiffness which results in the rafter connection stiffness, K_R. The purlin-rafter connection and shear connector stiffness values are developed from data found in Leflar (2008) and Anderson (1990). The resulting equations are:

$$G'net = G'\frac{\frac{a}{b}}{\frac{1}{\left(\frac{b}{a}\right)} + \frac{2}{K_R}}$$

and

$$Cp = G'net\left(\frac{b}{a}\right)$$

Where a is the diaphragm width, b is the diaphragm length, K_R is the combined purlin-rafter and shear connector stiffness, Cp is the in plane diaphragm stiffness, and G' is the diaphragm shear stiffness from the MCA method.

The purlin-rafter connection stiffness is taken to be 1k/in and the shear connector stiffness is ten times greater, 10k/in. The purlin-rafter connection and shear connector slips often reduce the diaphragm stiffness by and order of magnitude relative to the MCA manual stiffness.

Summary:

The MCA method presented by Luttrell and Mattingly (2004) as modified by Leflar (2008) can accurately predict the strength and stiffness of light-gauge metal diaphragms on lumber frames used in post-frame construction as demonstrated with comparisons to strength and stiffness data obtained from tests. The MCA method was modified as presented in this paper so the parameters in the model that affect the strength and stiffness of diaphragms used in post-frame construction are more accurately represented in respect to post-frame construction. The net result is that with the Modified MCA method diaphragm strength and stiffness of metal clad lumber frame diaphragms can be estimated without testing of the different assemblies. The modification used in the comparisons to test data were briefly discussed and explained. ***FBN***

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