Warping of Brick Cladding Corners

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Walls constructed of rigid materials, such as brick or concrete, provide designers with interesting challenges when it comes to isolating the wall from building movements.

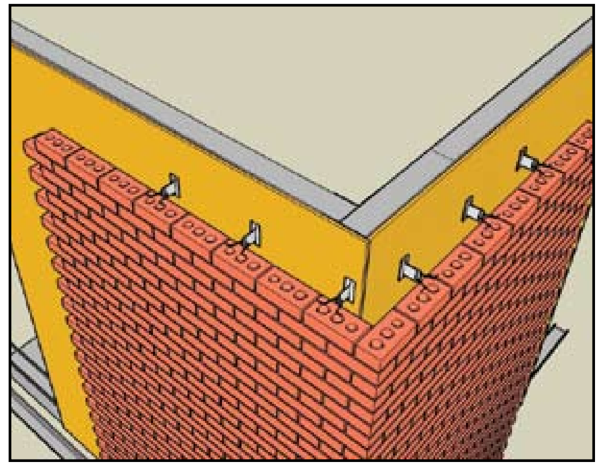

Figure 1: Brick Veneer on Steel Studs Corner.

During seismic events, one floor of a building moves in the horizontal plan relative to another. This movement can be in any horizontal direction. The differential horizontal movement between floors can result in damage to the wall, a falling hazard and economic loss if the wall is not properly designed.

There are many ways to attach walls to buildings. The attachment scheme has a direct link to the performance of the wall. For flat walls, it is relatively easy to design attachments that produce high performance. Corners are more difficult.

In a recent project, a brick veneer on steel stud system was selected. It was decided to use the warping corner strategy of attachment. The design displacement for "immediate occupancy" was ½ inch, and the displacement for "life safety" (⅔ MCE) was 1½ inches.

Figure 1 shows the normal location of brick ties without applying the warping concept. A vertical joint of 1-inch or more at the corner (not shown) would be required to meet the "immediate occupancy" standard.

Figure 2 shows the ties removed from the corner; the first tie is located 4 feet away. Instead of having a vertical joint, the brick wall is designed to warp. This can be visualized by considering the wall on one side of the corner as a rectangle. Three corners of the rectangle do not move. The two corners are fixed by the wall return and the third by the floor. The upper corner, where the tie is located, moves in the direction perpendicular to the surface of the rectangle. In other words, the rectangle warps.

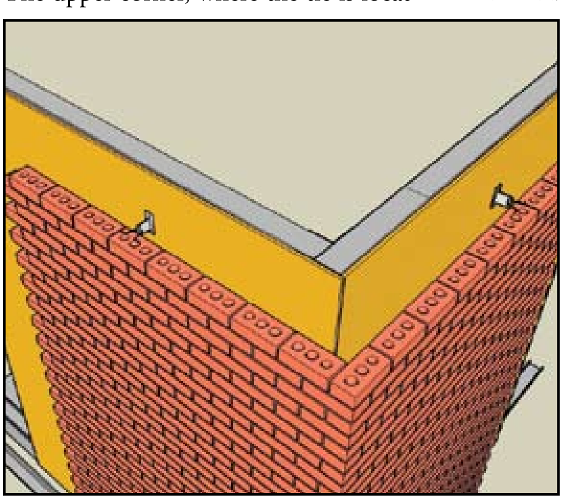

Figure 2: Brick Veneer on Steel Studs – Warping Corner.

It was decided to test a corner to see if the brick could displace the ½-inch distance without unacceptable cracking. Unacceptable cracking was defined as cracking of the brick. Cracking of the mortar joint between the brick and mortar was considered acceptable.

Figure 3 shows the test specimen. Loading the specimen at the center of the panel, and not at the upper corner, saved on test set-up costs. The relationship between the two loading points would be determined by analysis. The frame shown is for the support of the dial gages and not to resist the test loading. The plate at the center of the closest wall is the location of the load cell on the other side of the wall.

The ½-inch defection requirement occurs at the top corner, away from the panel corner. A finite element model predicted the behavior prior to the test. The model predicted that in the elastic range, the deflection at the center is one-fifth the deflection at the upper corner, if the load were moved to the corner and the deflection were measured at the corner. Thus, the required deflection at the location of the test loading is 0.10 inch. The model predicted a deflection of 0.14 inches at the location of the load cell at the center of the wall. This translates to a 0.70-inch differential movement between floors before cracking.

Figure 4 shows the chart of the center deflection as a function of the load that was used during the test. The actual deflections at the load cell are the circles. As can be seen, the predicted was slightly more flexible than the actual but still provided acceptable performance.

The test provided information on the validity of the material properties given in the building code (TMS 402). The estimated values for the shear modulus of brick is $E_v = 0.4\ E_m$, where $E_m = 700f'_m$. Tests of the brick resulted in a brick average compressive strength of 9,197 psi, and a modulus of rupture of 500.7 psi. Using the TMS 402 Specification, Table 1, results in a predicted compressive strength of the masonry of 3,287 psi. This corresponds to E_m = 2,300,000 psi, and E_v = 920,400 psi.

Adjusting the model elastic properties to match the test behavior is not a valid process. Masonry is more orthographic than isotropic as assumed in the model, plus many other factors are not considered. Nevertheless, doing the simple adjustments to match the actual values results in E_m = 1,041,000 psi and E_v = 416,000 psi, or less than half the values predicted by code.

The code offers no predictions for torsional strength of masonry. The first major torsion crack occurred at a load of 3,454 pounds. The model predicts a maximum torsion of 450 lb-in/in at this load. For the 3½-inch brick, this translates to a torsional stress of 110 psi. A rule of thumb is to double this value, as normally the masonry joint is cracked and the brick alone is resisting the load. Alternatively in running bond, it is assumed the brick is one-half the section. Thus, the brick torsion failure was at about 220 psi. This is considerably less than the modulus of rupture.

The old rule of thumb stating that the modulus of rupture is 10% of the compressive strength is not a good predictor for this brick. The modulus of rupture is more like 5% of the compressive strength.

The test verified that using the warping strategy for a brick corner is an acceptable alternative to providing a large vertical joint at the corner. However, this strategy cannot be generally applied without an engineering assessment of the project-specific situation. The performance of a warping corner depends on brick mortar and geometry being used.▪

Figure 3: Test Panel.

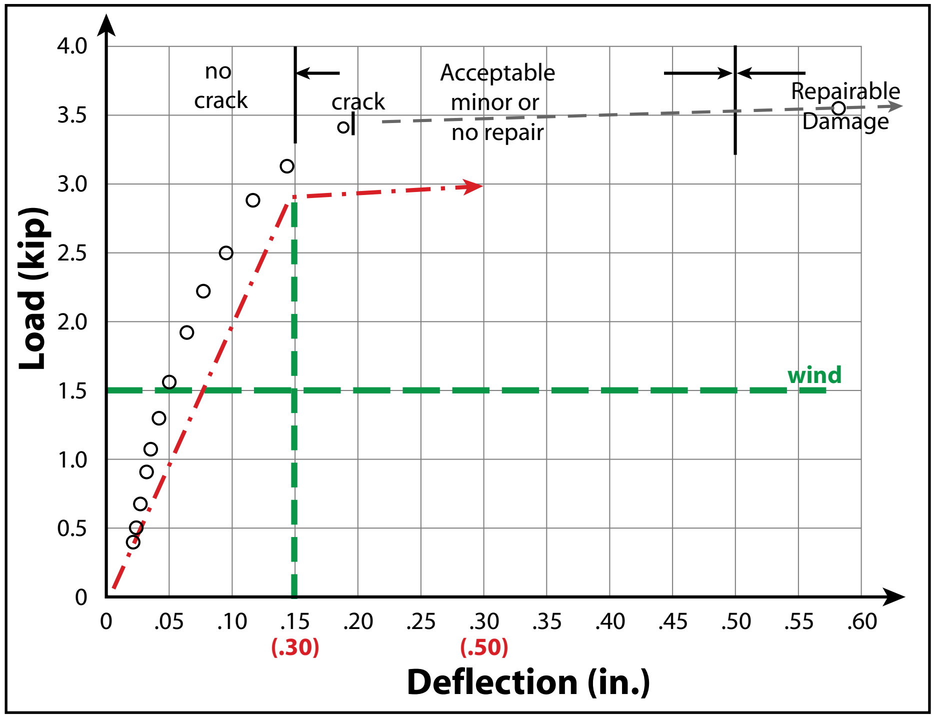

Figure 4: Comparison of Test Deflection to Predicted Deflection.

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