The Rational Design of Anchored Masonry Veneer

By W. Mark McGinley, Ph. D., P.E.

Anchored masonry veneer wall systems are commonly used in residential, commercial and institutional construction. As shown in *Figure 1*, these exterior wall systems include an outer wythe (layer) of masonry veneer attached across an air space to a backing wall by anchors. These backing wall systems may be sheathed wood and steel stud walls, concrete masonry walls or poured concrete walls. The veneer wythe is typically constructed with units of clay or concrete masonry with a nominal thickness of 2⅝ to 4 inches.

By definition, the exterior masonry portion of this system is nonload-bearing and is usually considered only an exterior finish for the building envelope.

For masonry design, model building codes reference the Masonry Standards Joint Committees' *Building Code Requirements for Masonry Structures*, TMS402/ACI530/ASCE5 (MSJC). Chapter 6 of this document (2008 edition) describes two methods for veneer design, the prescriptive and rational methods. The prescriptive method requires that the backing wall be designed to resist the entire out-of-plane wind loading, and provides prescriptive thickness, tie spacing, and tributary area details for the veneer. Almost all masonry veneer wall systems are designed prescriptively. However, that method does not extend to wind zones in excess of 130 MPH, cavity widths in excess of 4½ inches, new tie systems, and is costly to apply in rehabilitation designs.

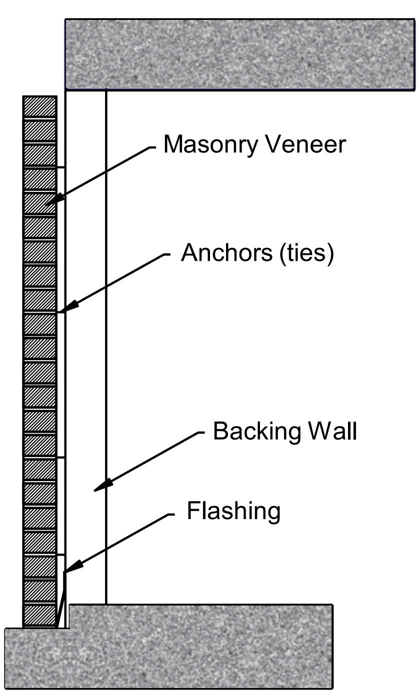

Figure 1: Schematic of Anchored Masonry Veneer.

The rational design method is more time-consuming to design, and the code provisions give very little guidance. Rational veneer design requires that:

1) The forces applied to the veneer are distributed using the principles of mechanics.
2) The backing deflection is limited.
3) Veneer is not subjected to either the flexural stress limits defined in the allowable stress provisions in Chapter 2 of the Code, or the modulus of rupture values in Chapter 3.
4) Designers comply with the general provisions (Chapter 1), with the exception that a specified compression strength (f'_m) is not required.
5) Meet the prescriptive requirements for stack bonded masonry and higher seismic zones.

Implicit in any veneer design is that it can accommodate differential movement and resist moisture penetration.

For design, the wall behavior under in-plane and out-of-plane loading must be addressed. Under in-plane loads, little load will be transferred between the veneer and backing if the wall is detailed properly and the in-plane deformations are controlled. To ensure the backing wall carries all in-plane loads, it is suggested that the veneer contain horizontal and vertical movement joints and that vertical deformation and inter-story drift be limited.

For out-of-plane loads, an understanding of the wall system behavior is necessary. A good overview is available in *Masonry Veneer Wall Systems* (Structural Engineering Report #156, Department of Civil Engineering, University of Alberta, Edmonton, by McGinley et al.), *Brick Veneer/Steel Stud Walls, Technical Note 28B* (2005 by the Brick Industry Association), and the *Design Guide For Anchored Brick Veneer Over Steel Stud Systems* (Western States Clay Products Association).

Figure 2 shows a schematic of the out-of-plane load distribution within the wall system. Under these loads, the veneer spans over a variable anchor support locations where the loading is transferred to the backing wall. The anchor load distribution and veneer support depends on the relative stiffness of the veneer, anchors and backing system. This distribution is further complicated by variation in anchor configuration (especially free play and stiffness). However, analysis and tests data suggest that, for stud backing walls, the veneer generally spans between the anchors near the top and bottom of the wall until the veneer cracks. This induces larger anchor loads near the veneer top and bottom, if the backing is flexible relative to the veneer. As load on the veneer is increased, it usually cracks in a bed joint near mid-height at a loading well below the peak design levels.

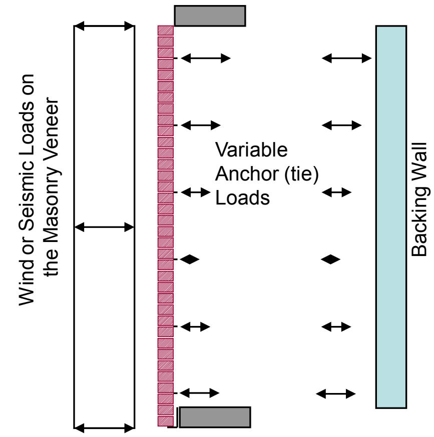

Figure 2: Out-of-Plane Load Distribution of Uncracked Wall System.

After the veneer cracks, it acts as two separate pin connected sections over a variable support and the anchor loading near mid-span is greater than elsewhere. However, non-linear analyses suggest that if anchor/backing stiffness is relatively low, and/or if the anchors are reasonably ductile, the anchor loads will be approximately uniform at ultimate loading. Reasonably ductile or flexible anchor systems allow redistribution of anchor loads. In light of the variability of the anchor stiffness and thus the load distribution, approximating the anchor load as uniform is reasonable.

Out-of-plane cyclic load tests of these systems also suggest that the critical anchor loading will always be tension; under compression, there are sufficient mortar droppings in the cavity to support the anchors and veneer and transfer the loads to the backing wall.

The above behavior suggests the following design methodology:

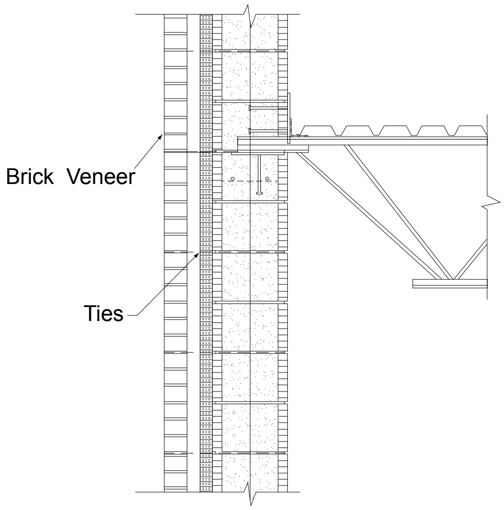

Figure 3: Brick Veneer and Masonry.

1) Assume the veneer cracks in a bed joint near mid-span of the wall. A bed joint can be raked at mid-height to ensure this happens.
2) Design the backing system to resist the resulting out-of-plane loading.
3) Design the anchors to resist out-of-plane loads in proportion to the load distribution and tributary area.
   a) For stud-backed systems with stiff and brittle anchors, it might be assumed that the anchors near mid span resist reactions produced by the two sections of veneer spanning from the top and bottom to mid-height. Although anchor loads are best determined by finite element analysis, they might be approximated by assuming the anchors within one anchor spacing of the mid-height and the top and bottom of the wall resist the veneer reactions.
   b) For flexible/ductile anchor systems, and for stiff backing walls, it appears that a reasonable approximation can be obtained by assuming that all anchors are loaded uniformly based on tributary area.
4) Design the masonry veneer to span between anchor supports without further cracking by limiting the veneer moments to those predicted by the stress limits in either Chapter 2 or Chapter 3 of the MSJC. Although MSJC provisions specifically exempt veneers from having to meet these limits, they are a good means to limit the crack isolation of veneer sections.

Example

A rational veneer design of the wall system shown in *Figure 3* follows. This reinforced, 8-inch, concrete masonry (CMU) with a 4-inch clay brick veneer wall system is in a warehouse located in a 140-mph wind zone. Based on the simplified procedures in ASCE 7 for component and cladding wind loads, peak wind service level wind loads of 38.4 psf can be expected for the corner regions of this 20-foot tall building. Wind loads govern for out-of-plane loads since the building is in a Seismic Design Category B. Only the corner design is given.

Because the wind speed exceeds 130 MPH, a rational design must be conducted. Based on the proposed design methodology, the CMU wall is designed to resist the entire out-of-plane wind load, and the veneer be designed to span between the anchors. A relatively ductile anchor (tie) system will be chosen so there will be adequate redistribution of forces to ensure a uniform loading assumption.

The backing wall design was performed and the 8-inch CMU wall, reinforced with a # 5 rebar (60 ksi steel) at 16 inches on center, can resist the uniform wind 38.4 psf over a 20-foot span.

For the veneer design, the maximum anchor spacing must be set to preclude additional veneer cracking at ultimate load levels. To ensure this behavior, the stresses in the clay masonry veneer will be limited to the modulus of rupture values listed in Table 3.1.8.2.1 of the MSJC.

Using Type S masonry cement mortar, MSJC Table 3.1.8.2.1 defines a flexural tensile stress limit of 60 psi for stresses normal to bed joints (vertical span) and 120 psi for stress parallel to bed joints (horizontal span). If simple supports are conservatively assumed in both the horizontal and vertical directions, and the factored wind load, W_u is equal to 1.6 × 38.4 = 61.4 psf, the maximum vertical and horizontal veneer spans can be determined as follows:

For unreinforced masonry the resistance factor, $\phi = 0.60$

Vertical Span:

$$\phi f_{tv} = 0.60\times 0.60 \text{ psi} = \frac{M_u}{S} = \frac{\dfrac{61.4 \text{ psf} \times (L_v)^2}{8} \times 12\ {}^{\text{in.}}/_{\text{ft.}}}{\dfrac{12 \text{ in.}\,(3.63 \text{ in.})^2}{6}} \Rightarrow L_v = 3.21 \text{ ft} = 38 \text{ inches}$$

Horizontal Span:

$$\phi f_{tv} = 0.60\times 120 \text{ psi} = \frac{M_u}{S} = \frac{\dfrac{61.4 \text{ psf} \times (L_h)^2}{8} \times 12\ {}^{\text{in.}}/_{\text{ft.}}}{\dfrac{12 \text{ in.}\,(3.63 \text{ in.})^2}{6}} \Rightarrow L_h = 4.53 \text{ ft} = 54 \text{ inches}$$

Note that the nominal 4-inch clay units have a specified thickness of 3.63 inches.

These distances define the limits of the anchor spacing that are possible without further veneer cracking being likely. However, it is possible that a bed joint containing a line of ties may crack. If this occurs, a section of veneer would have to span as a cantilever from the tie line, either above or below the cracked joint. However, there is usually enough keying between the unit cores and the mortar to at least approximate a pinned connection at this location. There is, however, a section of veneer that typically must act as a cantilever. That is the section of veneer at the top of the wall above the last set of ties. To ensure that the veneer will not crack off in this area, the flexural tension stress on the cantilever section must be kept below the Code-prescribed maximums. This limits section cantilevering from the top anchors to:

$$\phi f_{tv} = \frac{\dfrac{61.4 \text{ psf} \times (L_{v\,top})^2}{2} \times 12\ {}^{\text{in.}}/_{\text{ft.}}}{\dfrac{12 \text{ in.}\,(3.63 \text{ in.})^2}{6}} = 0.60\times 60 \text{ psi} \Rightarrow L_{v\,top} = 1.60 \text{ ft} = 19 \text{ inches}$$

This same crack isolation is not likely in the horizontal direction. Thus, the highest level of anchors must be placed within 19 inches of the top of the veneer. If the designer was concerned about the load transfer at a cracked bed joint, this vertical spacing limit can also be applied to the rest of the anchors, but this may be overly conservative.

The anchors must now be designed, but there is little guidance on how to do this in the MSJC. The MSJC commentary does reference the Canadian *CSA A 370 (Connectors for Masonry) Standard*. This document provides limit state design provisions for masonry anchors and ties, and requires that the nominal capacities of the ties (determined based on tests or analysis) be reduced by a capacity-reduction factor of 0.9 for material failures and 0.6 for anchorage or buckling failures. The factored capacities then must meet or exceed the anchor loading produced by factored design loads.

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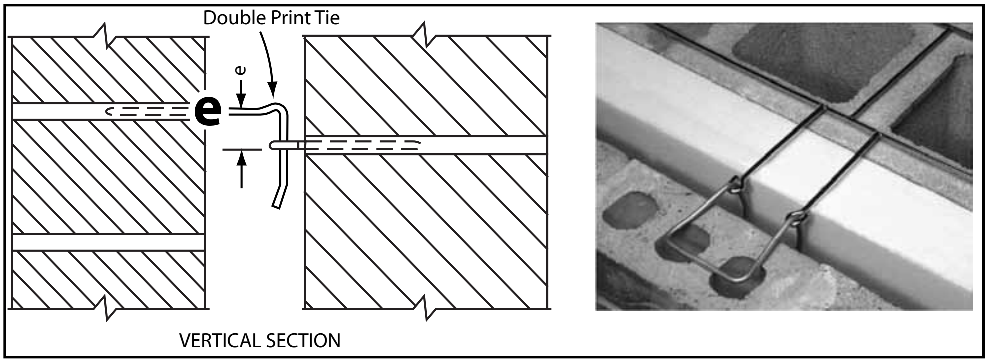

Figure 4: An Adjustable Pintle and Eye Anchor System.

A number of manufactures provide ultimate strength test data for design, usually on the web. A brief web search identified an anchor system shown in *Figure 4*. This adjustable pintle and eye system is used commonly in multi-wythe masonry construction, and allows for differential vertical movements between the veneer and the backing wall. It also accommodates coursing variations between the two wythes of masonry.

The ultimate resistance for this tie system (tension or compression) is 980 pounds for a zero pintle eccentricity, 200 pounds for a 0.75 inch pintle eccentricity and 100 pounds for a pintle eccentricity of the code allowed maximum of 1.25 inches. Note that the tie flexibility varies over 400% over this same range. The maximum pull-out strength of the pintle from a mortar joint is 800 pounds.

The average eccentricity of the ties at the peak loading location is not likely to be at the code allowed maximum (an average of 0.75 inches was assumed). Also, the pintle capacity is limited by a ductile bending failure of the legs, allowing significant load redistribution. Thus, the factored capacity of the anchor is the smallest of:

$\phi Tie = 0.9 \times 200 = 180$ *pounds* $\Leftarrow$ *governs*

or $= 0.6 \times 800 = 480$ *pounds*

This calculation suggests that the critical tie capacity is 180 pounds. Assuming that the tie forces are uniformly distributed, then the

$$\text{Maximum tie tributary area} = \frac{180 \text{ lb}}{61.4 \text{ lb/ft}^2} = 2.93 \text{ ft}^2$$

The above calculations indicate that ties with a capacity of 180 pounds can be spaced so that their individual tributary areas do not exceed 2.94 square feet. Veneer stresses also limited the anchor vertical spacing to 38 inches and 54 inches horizontally, and there must be a row of anchors within 19 inches from the top of the wall.

The anchor system pictured in *Figure 4* is prefabricated so that the horizontal spacing of the eyes is 16 inches. Standard practice would then have the eyes and joint reinforcing placed in every other block course, or 16 inches vertically, even though the vertical spacing of the ties could be increased to 24 inches without violating the vertical spacing limits or the maximum tributary limits.

In this example, a spacing of 16 inches vertically, along with the predefined 16-inch horizontal spacing was used. This allowed a larger average eccentricity to be present in the pintle legs and matched common practice. It should be noted that additional anchors should be provided at larger openings as described in the Code Section 6.2.2.11(c), with anchors within 12 inches of the opening and at a maximum spacing of 24 inches or the minimums described above (which control for this wall).

Finally, the rational veneer design provisions would require the system deflections be limited to ensure veneer stability. The concrete masonry backing wall is so stiff this is not going to be an issue for this design. If a stud backing wall was used in the design, deflections should be limited. Some documents suggest limiting the stud deflections to a maximum of the stud span divided by 360 (L/360), and some suggest that this limit should be L/600. The less stringent L/360 limit is likely to be more than sufficient to limit veneer instability. Proponents of the more stringent limit argue that it will reduce the amount of moisture that penetrates the veneer. However, the veneer is assumed to be cracked, and limiting the crack width at one location under transient wind loads will not appreciably change the amount of water penetrating the wall system once the crack is present.

The previous discussion presents the author's interpretation of the current code rational design provisions for veneer. Although these interpretations are based on a significant experience with this type of wall system, both research and in the field, readers are encouraged to review these recommendations and make there own judgments.▪

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