J.B. SPEED SCHOOL OF ENGINEERING

# Alternative Designs of Masonry Veneer & Shelf Angle Supports Session 1

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Masonry Day

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Louisville KY

MIH Masonry Institute of Hawaii

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## Course Description

Chapter 12 of the TMS 402 2016 includes provisions for the design of anchored and adhered masonry veneer. Although the vast majority of anchored masonry veneer systems are designed using the prescriptive provisions within this standard, veneers in high wind zones, high seismic zones, or constructed with large distances between the backing walls and the veneer cannot be designed using these prescriptive provisions. In addition, the prescriptive provisions do not often provide the most cost effective solutions for the rehabilitation and retrofit of masonry veneers. The TMS 402 does have provisions for alternative rational design provisions for masonry veneers. These provisions will be explained and a method for the rational veneer wall system design will be presented. This presentation will also describe a method for the design of the vertical Shelf angle support of the walls systems.

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## Learning Objectives

- **Describe the alternative design procedures for masonry veneers in TMS 402 (Chapter 12).**
- **Present an example of a rational veneer design.**
- **Describe how steel shelf angles provide vertical support to masonry veneers.**
- **Present sample shelf angle designs.**

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## Introduction

- TMS 402/ACI 530/ ASCE 5 – 2016 Ch12 has provisions for anchored and adhered masonry veneer.

  **“12.1.2** *Design of anchored veneer*
  Anchored veneer shall meet the requirements of Section 12.1.6 and shall be designed rationally by Section 12.2.1 or detailed by the prescriptive requirements of Section 12.2.2.”

- Almost all anchored masonry veneer systems are prescriptively designed using the prescriptive provisions.

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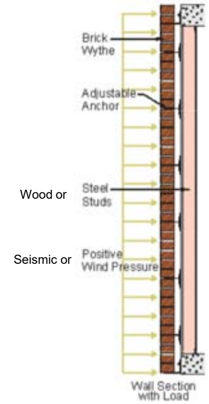
## Introduction

- In high wind & seismic zones, or unusual configurations, Veneer cannot be designed prescriptively.
- Also, the prescriptive provisions are often not the most cost effective solutions for the rehabilitation of veneers.
- The TMS 402 has alternative rational design provisions for masonry veneers.

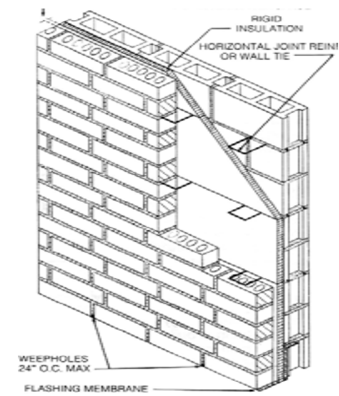


## Masonry Veneer Wall Systems

Wood or Steel Stud Backed
From BIA TECH Note




CMU Backed
From Maconline






## Code Provisions

**12.2 — Anchored veneer**
**12.2.1** *Alternative design of anchored masonry veneer*
The alternative design of anchored veneer, which is permitted under Section 1.3, shall satisfy the following conditions:
(a) Loads shall be distributed through the veneer to the anchors and the backing using principles of mechanics.
(b) Out-of-plane deflection of the backing shall be limited to maintain veneer stability.
(c) The veneer is not subject to the flexural tensile stress provisions of Section 8.2 or the nominal flexural tensile strength provisions of Section 9.1.9.2.
(d) The provisions of Section 12.1, Section 12.2.2.9, and Section 12.2.2.10 shall apply. (Stack Bond and Seismic)



## Code Provisions-Commentary

**12.2 — Anchored veneer**
**12.2.1** *Alternative design of anchored masonry veneer*
There are no rational design provisions for anchored veneer in any code or standard. The intent of Section 12.2.1 is to permit the designer to use alternative means of supporting and anchoring masonry veneer. See Commentary Section 12.1.1 for conditions and assumptions to consider. The designer may choose to not consider stresses in the veneer or may limit them to a selected value, such as the allowable stresses of Section 8.2, the anticipated cracking stress, or some other limiting condition. The rational analysis used to distribute the loads must be consistent with the assumptions made. See Commentary Section 12.2.2.5 for information on anchors. The designer should provide support of the veneer; control deflection of the backing; consider anchor loads, stiffness, strength and corrosion; water penetration; and air and vapor transmission.

# Code Provisions-Commentary

**12.1.2** *Design of anchored veneer*

When utilizing anchored masonry veneer, the designer should consider the following conditions and assumptions:
a) The veneer may crack in flexure under allowable stress level loads.
b) Deflection of the backing should be limited to control crack width in the veneer and to provide veneer stability.
c) Connections of the anchor to the veneer and to the backing should be sufficient to transfer applied loads.
d) Differential movement should be considered in the design, detailing, and construction.
e) Water will penetrate the veneer, and the wall system should be designed, detailed, and constructed to prevent water penetration into the building

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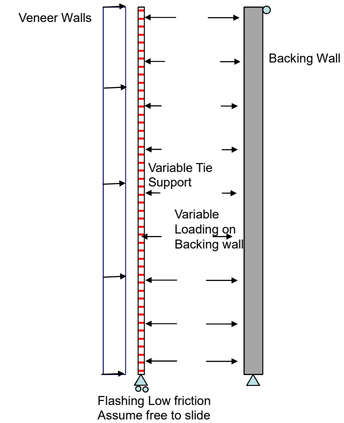
# Masonry Veneer Wall Systems Under uniform OOP loads




Typical design is to either:

1; Design backing wall to resist all out of plane load and assume veneer is thick heavy paint.

2. Rationally design veneer, anchors and supports. **How?**

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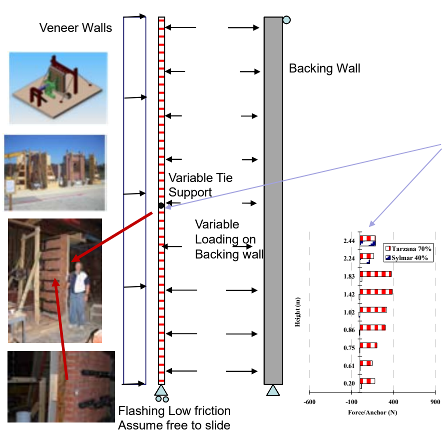
# Masonry Veneer Wall Systems

## Measured Behavior Under Uniform OOP loads




Based on many wall tests – Static, Quasi-static and Dynamic:

1.If stud backing walls are used and the veneer is uncracked, The ties at the top and bottom are loaded more.

2. Once the veneer cracks, the ties near mid-height are loaded more.

3. Veneer cracking occurs almost always before design loads and always before backing wall deflects L/1000 (no need to limit backing wall deflections to L/600!) .

4. CMU backing walls with eye and pintel tie systems much closer to uniform tie loads.

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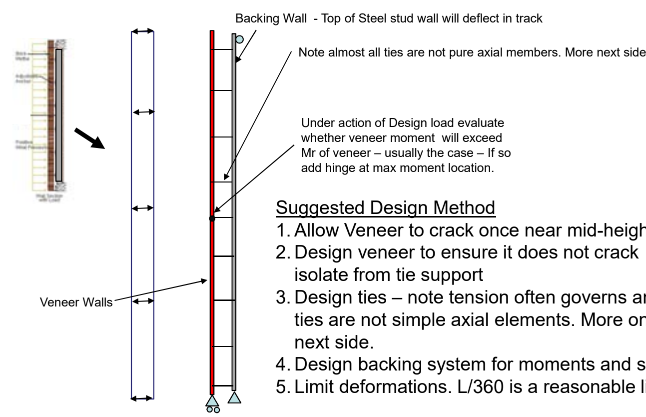
# Design of Veneer Wall Systems

Use a simple FE frame Model




Suggested Design Method
1. Allow Veneer to crack once near mid-height.
2. Design veneer to ensure it does not crack isolate from tie support
3. Design ties – note tension often governs and ties are not simple axial elements. More on next side.
4. Design backing system for moments and shear.
5. Limit deformations. L/360 is a reasonable limit

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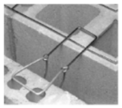
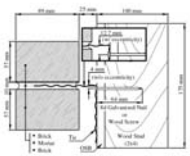
# Design of Veneer Wall Systems
## Ties

CMU ties – Adjustable eye and pintel

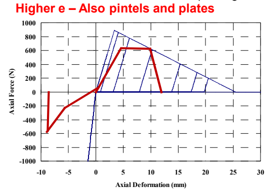


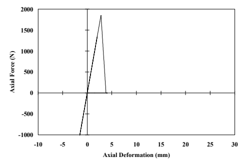
Typical Steel Stud Adjustable



Wood Stud Ties – Typically Corrugated 22 to 16 gauged nailed or screwed - Lefave

Okail- Flexible tie behavior no e. (Corrugated tie
**Higher e – Also pintels and plates**




Okail More rigid tie behavior no e. **Must account for e as well – lower stiffness**

Tie Failure modes:

1. Yielding of tie metal – Usually combined bending and tension/comp
2. Buckling of tie in compression- less likely than you think.
3. Pullout of veneer mortar joint.
4. Pullout of backing wall.

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# Harbor Courts Complex Baltimore

Office/Condominium

8-Story Hotel

Parking D

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# Investigation of Problems

Localized Cracking and Distress of Brick

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# Mortar Blockage in Horizontal Expansion Joints Below Shelf Angle

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# Horizontal Expansion Joints

## Problems

Popped out – no ties

Cracked away

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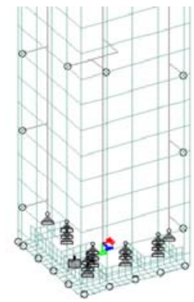
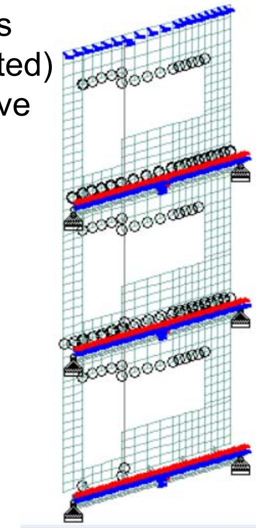
To solve these problems (mostly shelf angle related) you get to use FE to save money and lawyers are paying.





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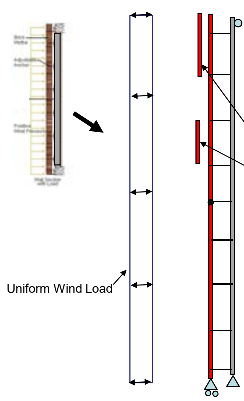
# What About Simplified of Veneer Wall Systems ?




Suggested Simplified Design Method

1. Design backing system for moments and shear.
2. Limit deformations. L/360 is a reasonable limit
3. Assume veneer cracks at mid-height.
4. Design veneer to ensure it does not crack isolate from tie support – can use simple span and propped cantilever at top – check vertical and horizontal spans.
5. Design ties – What are loads? They range from almost uniform in stiff backing systems with ductile tie systems to some multiple of Ave. tie load - Note KPFF BV SS design guide suggests using 2+ x average (based on trib. area) when designing stiff ties with steel studs.

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# CMU Backed Veneer Wall Example

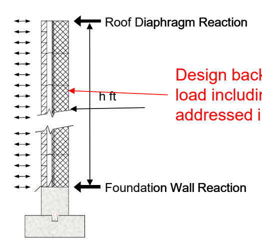
Cavity wall 8" CMU 4 " Clay brick veneer – Example - The winds would produce a *maximum wind loading of*

*V = 150 MPH wind speed maps Cat II Assume loads for veneer and anchors – note veneer and anchors do not see internal wind pressures.* $A_{trib}$ *= 10 ft²*

Walls (based on 10.0 ft². Assume loads are determined as shown.

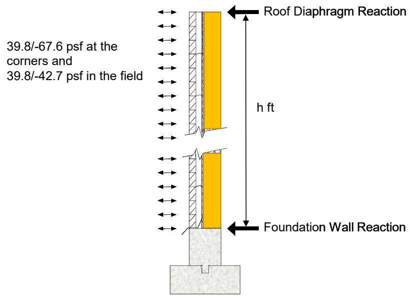
•$P_{(4)}$ =39.8 or -42.7 psf

•$P_{(5)}$ = = 39.8 psf and -67.6 psf



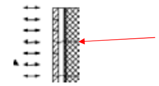

Design backing wall to resist entire wind load including internal gust coeff. – Not addressed in this example.

# CMU Backed Veneer Wall Example




Note the components and cladding wind loads were determined based on an effective wind area of 10 $ft^2$ (required for anchors) and are for the design of the veneer and anchors only. For the backing walls the loads are lower. The wind code would suggests the loads should be determined using a wind area no lower than the $span^2/3$. The effective wind area is used to determine the components and cladding pressure coefficients and thus the wall pressures. These pressures are then applied over the actual tributary area.

# CMU Backed Veneer Wall Example



Design veneer to span between ties – Conservatively simple spans

Note that wind load provisions would require that the connectors (ties) be designed for the actual tributary area. Since loads are the same for areas below 10 $ft^2$, this can conservatively be used for effective wind area for the veneer as well.

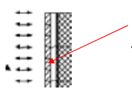
Vertical Span: Type S masonry cement mortars See Table 9.1.9.2 (TMS 402)
In the general field.

$$\phi\, f_{tv} = 0.60\times 80.0\text{ psi} = \frac{M_u}{S} = \frac{\frac{1.0\times 42.7\text{ psf}\times (L_v)^2}{8}\times 12\,\text{in.}/\text{ft}}{\frac{12\text{ in.}(3.63\text{ in.})^2}{6}} \Rightarrow L_v = 4.44\text{ ft} = 53.3\text{ in.}$$

At the corners

$$\phi\, f_{tv} = 0.60\times 80.0\text{ psi} = \frac{M_u}{S} = \frac{\frac{1.0\times 67.6\text{ psf}\times (L_v)^2}{8}\times 12\,\text{in.}/\text{ft}}{\frac{12\text{ in.}(3.63\text{ in.})^2}{6}} \Rightarrow L_v = 3.53\text{ ft} = 42.3\text{ in.}$$

# CMU Backed Veneer Wall Example



Design veneer to span between ties – Conservatively simple spans

Horizontal Span:
In the General Field: Type S masonry cement mortars See Table 9.1.9.2 (TMS 402)

In the general field.

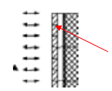
$$\phi\, f_{th} = 0.60\times 160\text{ psi} = \frac{M_u}{S} = \frac{\frac{1.0\times 42.7\text{ psf}\times (L_h)^2}{8}\times 12\,\text{in.}/\text{ft}}{\frac{12\text{ in.}(3.63\text{ in.})^2}{6}} \Rightarrow L_h = 6.28\text{ ft} = 75.4\text{ in.}$$

At Corners:

$$\phi\, f_{th} = 0.60\times 160\text{ psi} = \frac{M_u}{S} = \frac{\frac{1.0\times 67.6\text{ psf}\times (L_h)^2}{8}\times 12\,\text{in.}/\text{ft}}{\frac{12\text{ in.}(3.63\text{ in.})^2}{6}} \Rightarrow L_h = 4.99\text{ ft} = 59.9\text{ in.}$$

These distances are the maximum spans for the veneer under the design wind loads, and represent the maximum spacing between tie systems. (Ignore self-weight.)

# CMU Backed Veneer Wall Example



Upper section of veneer designed as a propped cantilever

**Vertical Span:**

In the General Field:

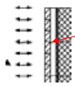
$$\phi\, f_{tv} = \frac{\frac{42.7\text{ psf}\times (L_{v\,revised})^2}{2}\times 12\,\text{in.}/\text{ft}}{\frac{12\text{ in.}(3.63\text{ in.})^2}{6}} = 0.60\times 80\text{ psi} \Rightarrow L_{v\,revised} = 2.22\text{ ft} = 26.7\text{ in.}$$

At Corners:

$$\phi\, f_{tv} = \frac{67.6\,\frac{190.4\text{psf}\times (L_{v\,revised})^2}{2}\times 12\,\text{in.}/\text{ft}}{\frac{12\text{ in.}(3.63\text{ in.})^2}{6}} = 0.60\times 80\text{ psi} \Rightarrow L_{v\,revised} = 1.77\text{ ft} = 21.2\text{ in.}$$

Crack isolation is not likely in the horizontal direction.

## CMU Backed Veneer Wall Example



Design the Ties

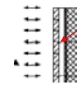
If ties are reasonably ductile and the backing wall is stiff, you get load re-distribution and thus can distribute loads ~ w.r.t. trib. Area.

Little code guidance for tie design. TMS 402 allows anchor test data to establish nominal strengths in Section 9.1.6.2.

Try Canadian CSA A 370 (Connectors for Masonry) 2004.

Those provisions require that the nominal capacities of the ties (determined based on tests or analysis), when reduced by a capacity-reduction factor of 0.9 for material failures and 0.6 for anchorage or buckling failures, equal or exceed the factored design loads. The tie minimum capacity in each direction (tension or compression) is considered to govern. (This is consistent with TMS 402 SD anchor design)

## CMU Backed Veneer Wall Example



Design the Ties

However most ties do not list ultimate load capacities based on loads, they simply list allowed values – You can used these allowable values! – Convert load to ASD

Using a typical tie allowable load = 200 lb.

If ties are reasonably ductile you get close to uniform load distribution and thus can distribute loads w.r.t. trib. area.

Maximum tie tributary area in the general field of the wall:

$$= \frac{200\ \text{lb}}{47.2\ \text{x}\ 0.6\ \text{lb}/\text{ft}^2} = 7.06\ \text{ft}^2$$

Maximum tie tributary area at the corners of the wall:

$$= \frac{200\ \text{lb}}{67.2\ \text{x}\ 0.6\ \text{lb}/\text{ft}^2} = 4.96\ \text{ft}^2$$

## Summary

- You can space ties not to exceed 4.4 ft and 3.5 ft (corners) vertically and 6.2 ft and 4.99 ft (corners) ft. horizontally.
- Last ties at top should be within 2.2 ft. and 1.7 ft. of the top of the wall.
- Trib. area of ties cannot exceed 7.06 $ft^2$ and 4.96 $ft^2$.
- As is typical, the capacity of the ties typically govern the tie spacing.
- For this wall - ties @ 3 ft. x 2 ft. (vertical) would work in all areas except the corners. Use 2 ft x 2 ft in these areas and ½ spacing at top of wall.

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## Questions on Out-of-Plane Design?

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# Vertical Support

In general, masonry veneer (stud backed) must be supported at very floor over 30 ft in height (except at gables). Also typically done for CMU backed veneers (differential movements).

This usually requires that shelf angles be placed at each floor above 30 ft. and designed to support the weight of veneer above.

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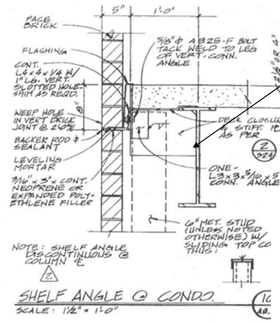
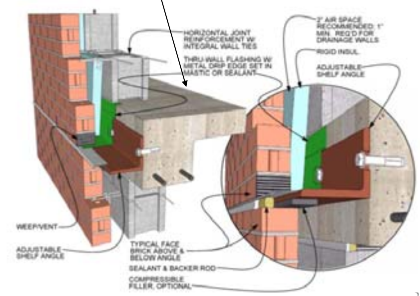



Shelf Angles tied back to Spandrel beam
Or slab widening

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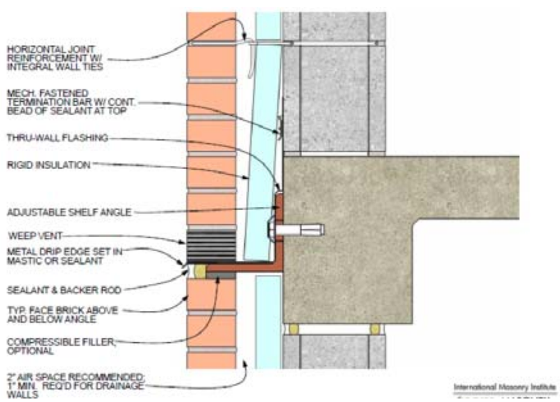


More "realistic detail" – Note the angle thickness and expansion joint height.

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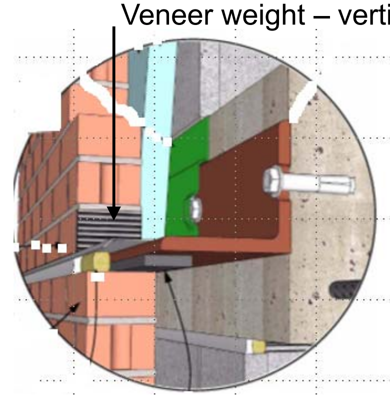
Design

Veneer weight – vertical load



Shelf angle must be designed to resist vertical veneer weight load

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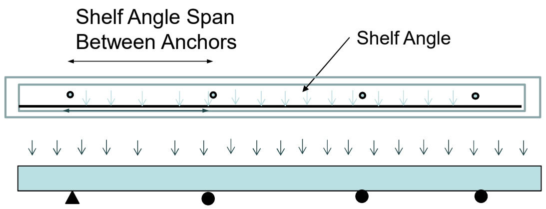


Common shelf angle design is to assume a uniform dead load from the veneer and design the angle to span as a beam between anchors OR

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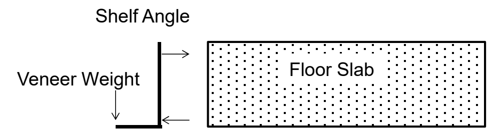
## Shelf Angle Frame Design Model




B) Anchored Frame Shelf Angle Model

Assuming the angle legs act as a bolted frame with a vertical dead load applied to the end of the horizontal leg

Question how much of the angle length is effective ?

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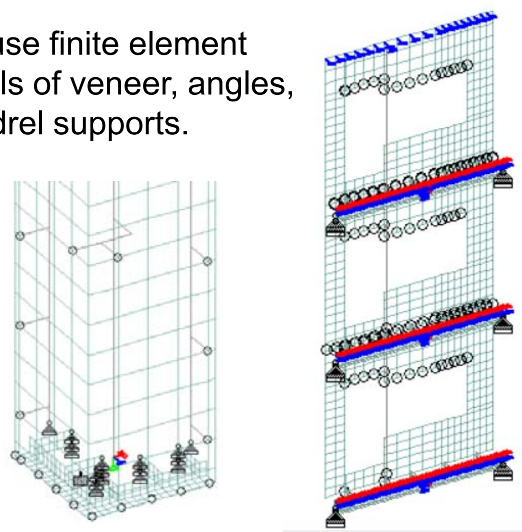
Can use finite element models of veneer, angles, spandrel supports.



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We found connection details are important!!!! More later

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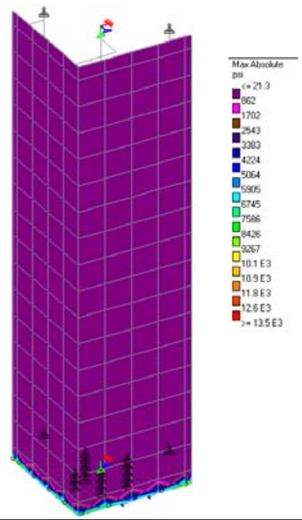
These analysis showed that the stiff brick (vertically) spans between the less flexible sections of the angle with little stress in the brick

(Dead load)



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Based on finite element analyses and field testing, a number of observations about the shelf angle veneer interaction can be made:

1. The veneer is very stiff relative to its supports.

2. Shelf angles are poor in torsion and do not really act as beams between anchors.

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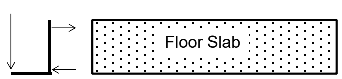
Based on finite element analyses and field testing, a number of observations about the shelf angle veneer interaction can be made:

Near Anchor




3. Near the anchors, the angle acts as a frame element and provides support to the veneer.

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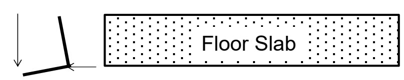
Based on finite element analyses and field testing, a number of observations about the shelf angle veneer interaction can be made:

Away from Anchor




4. Away from the anchor, the angle rotates away from veneer and provides little support to the veneer.

5. The brick can span a significant distance.

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## Veneer and Angle Behavior

Design:

- The veneer will span between steel angle anchors and will not likely govern.
- Anchor spacing will affect the loads on the angle anchors and the size of the steel angle.
- The length of the steel angle that resists this veneer loading is difficult to determine.
- One method to determine the effective length of the steel angle is to assume it is 4 x (the nominal veneer thickness).
- Another method to assume the effective length of steel shelf angle is about 25% of the anchor spacing Based on FE by Dillion et-al
- Once the veneer loads and the effective length of the angle are determined, the thickness of the shelf angle can be determined.

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## Veneer and Angle Behavior

**Table 1: Maximum Spans for Brick Supporting its Own Weight**

| Height of Veneer, h – (ft) | Maximum Brick Span for $F_t$ = 20 psi ($F_t$) | Maximum Brick Span for $F_t$ = 5 psi (ft) |
|---|---|---|
| 1.0 | 5.39 | 2.69 |
| 3.0 | 9.33 | 4.66 |
| 5.0 | 12.04 | 6.02 |
| 10.0 | 17.03 | 8.52 |
| 20.0 | 24.08 | 12.04 |
| 30.0 | 29.50 | 14.75 |

brick sections are able to support its own weight for even for relatively low heights.

It should be noted that the analysis presented in Table 1 was based on simple beam theory. This theory becomes increasingly inaccurate as span to depth ratios drop below 2 but is conservative.

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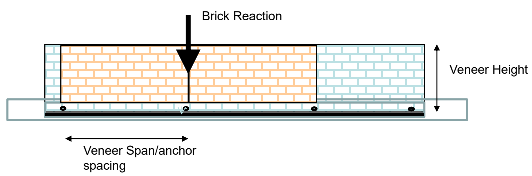
## Angle Design: Example 1




A shelf angle supporting a 10 ft. height of 4in., clay brick veneer. Anchor spacing = 6 ft.

Veneer reaction loading:

Veneer Reaction =40 psf (10'x 6')= 2400 lb

Adding 10 lb/ft for the angle results in R = 2460 lb.

Table 1 shows less than 1 ft. of veneer can span at least 2.6 ft

Effective length of angle (25% of the spacing of anchors) 18 in.

If an equal legged angle was used to support the veneer, the approximate angle loading shown in Figure 6 can be assumed.

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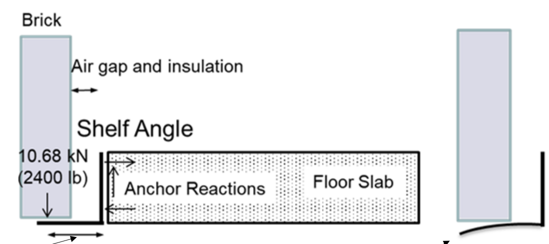
## Design Examples - Angle




Eccentricity of veneer weight = $e_b$
= 1 + 1 + ¼+ 3.625/2 = 4.06 in

Figure 6 Veneer weight in center of veneer as angle deflects away

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# Design Examples - Angle

Max. moment on the lower leg :

$M_{max}$ = 4.06 in. x 2400 lb = 9,744 lb. in

ASTM A36/A36M steel the $S_x$, required to resist the factored dead load moment would be: Use $M_y$ as the limit

$$S_x = \frac{M_{\text{max factored}}}{0.9\ F_y} = \frac{1.2\ (9{,}744)}{0.9\ (36{,}000)} = 0.361\ \text{in}^3$$

For the horizontal leg, $S_x = bt^2/6$ , Assuming the effective length is 25% of the Anchors spacing = 0.25 x 6 x 12 = 18 in.

The minimum thickness of angle should be t = 0.347 in.

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# Design Examples - Angle

You could also make an argument to use $\phi M_p = \phi\ Z_x\ F_y$
And $Z_x = bd^2/4$ -

If you used 1.4 D & $Z_{req}$=.421

Angle Z = 0.562 $in^3$ , t would be about the same either way

A 3/8 in. thick, A36 steel angle would work to support this load. Note that the eccentricity used in the above calculation has a significant impact.

**Use a 3/8" x 6" x 6" angle (A36)**

Note the vertical leg has tension and flexure but T stress less than 2% of $F_y$.
Note also there is no defection check of angle.

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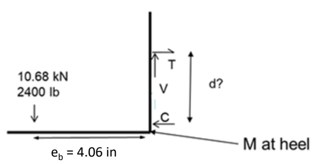
# Design Examples - Anchor

Shelf Angle



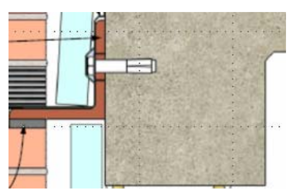



**Figure 7: Angle Anchor and Loading Configuration**

Assuming the anchor configuration and a concrete strength of 4000 psi,

Published factored shear strength of 7,570 lb and a factored tensile strength of 7,400 lb for a galvanized 5/8 in. x 4-3/8 in. expansion anchor. This would be compared to factored loads of:

Factored Shear = 1.4 x 2460 = 3,444 lb
(assume d = 3 – 0.5 = 2.5)
Factored Tension = 1.4 x (2400) x 4.06/2.5 = 5,457 lb

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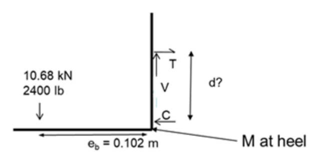
# Design Examples - Anchor

Shelf Angle



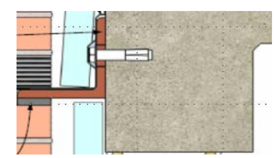



**Figure 7: Angle Anchor and Loading Configuration**

As these are combined loadings these must be dealt with using an interaction equation (Eq. 12) as shown below (as per manufacturer's recommendations).

$$\left(\frac{N_u}{\varphi N_n}\right)^{5/3} + \left(\frac{V_u}{\varphi V_n}\right)^{5/3} = \left(\frac{5.46}{7.40}\right)^{5/3} + \left(\frac{3.44}{7.57}\right)^{5/3} = 0.78 < 1.0 \text{ thus OK}$$

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## Design Examples - Bearing

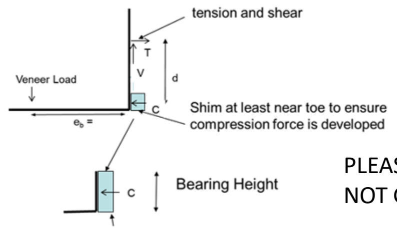


Figure 9: Assumed Bearing Conditions behind angle at Anchor Location

PLEASE NOTE THAT AIR IS NOT GOOD IN BEARING!!!

If the bearing of the steel is limited by the concrete bearing capacity, Using the ACI 318 concrete bearing provisions, a 1 in high shim 3 in. wide would provide a bearing capacity of 6,630 lb≥ 5,460 lb compression force developed at this location. Note the shim height must be limited to 1.0 in.

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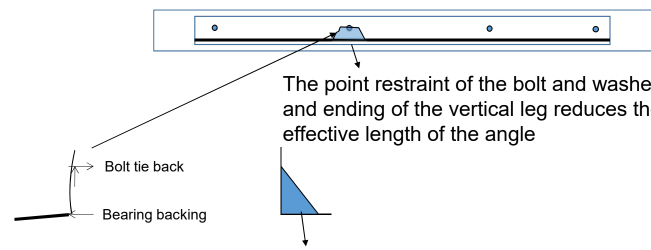
## Angle Design: Example 1

Note on the Vertical Leg

The vertical leg of the angle is being held back by the bolt at a point location



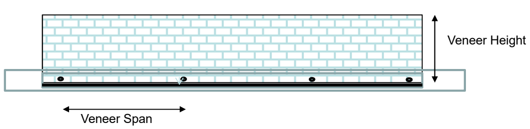

The point restraint of the bolt and washers and ending of the vertical leg reduces the effective length of the angle

However the Moment on vertical leg falls off away from the corner at about the same rate as the reduction in effective length

## Angle Design: Example 2 – 5 inch Cavity

10 ft height of 4” clay brick veneer

But now assume that there is 4” of backing wall (cavity) insulation.




Try an anchor spacing of 4 ft. – The veneer can span this distance for heights over 3 courses.

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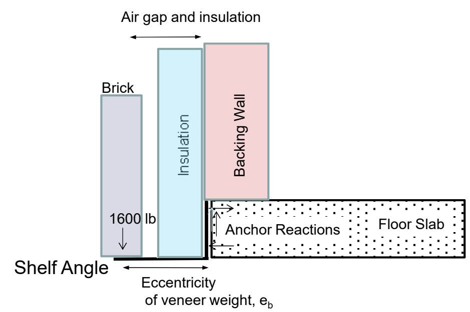
## Angle Design: Example 2 – 5 inch cavity

The distance between the interface of the veneer and support wall is 1 + 4 = 5 in.




Note ties may have to be rationally designed as tie length exceeds prescriptive code limits for most ties .

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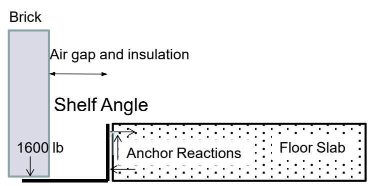
## Angle Design: Example 2 – 5 inch cavity

Using the same procedures as before – 4 ft anchor spacing




**Use a 7/16 x 8 x 6 angle (A36)**

— note 7/16 x 6 x 8 may not be available

**Use a 5/8" x 4-3/8" Galvanized Kwik Bolt 3 Anchor @ 4' OC**

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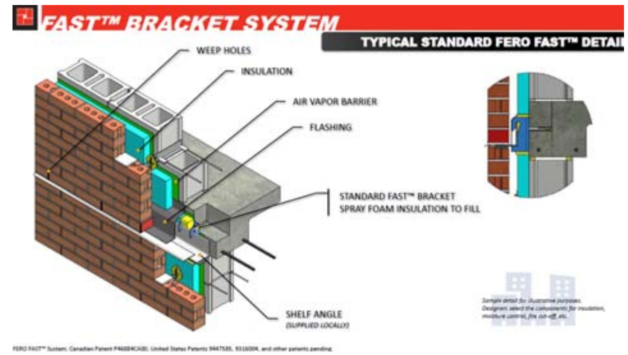
## Angle Design: alternatives




Anchor Reactions will depend on the configuration on the back of angle. Is plates to increase moment are, maybe reduce thermal bridging

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## Angle Design: Alternatives

**Now design the anchors**

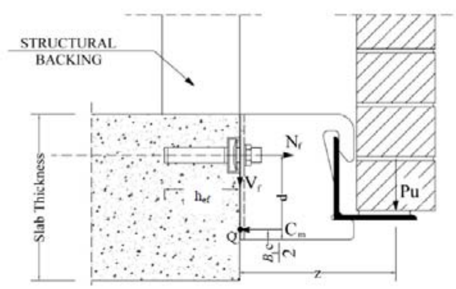


Figure C.3: Straining actions acting on the anchor at FASTTM connection

Increased level arm reduces anchor reactions

## Summary

- Presented a brief overview of Masonry Veneer Wall systems and their vertical supports
- Discussed how these systems behave
- Presented methods how they can be rationally designed
- Presented few examples for design

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# Questions???

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