# TOWARDS A CONSISTENT AND ECONOMICAL DESIGN OF SHELF ANGLES

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## ABSTRACT

Energy code requirements for buildings have increasingly become more stringent in the amount of insulation required in the building envelope system. In order to satisfy the continuous insulation requirement, designers are opting to place the insulation in the cavity, which necessitates a larger width. The recently published TMS 402-16 includes an increase of the maximum prescribed cavity depth requirement for masonry veneers. The typical cavity width in US construction will certainly increase as the TMS 402-16 requirements are legally adopted by local jurisdictions. Now appears to be an ideal time to rejuvenate the topic of shelf angle design. Relatively little has been published regarding the design of shelf angles for supporting masonry veneers. Shelf angle design is not as simple and intuitive as it might appear at first glance. Design simplifications often produce overly conservative results that unnecessarily increase the size and cost of the shelf angle and its installation. Common design assumptions are discussed and compared to previous studies. A comparison of analysis approaches from the literature confirmed that most approaches produce results that are extremely over-conservative. A new design model is proposed which more accurately accounts for the interaction between the masonry panel and the shelf angle. Results from analysis of a case study are presented and compared for multiple shelf angle design models. The analysis results show that the deflections and load distribution on a shelf angle are less than what is typically assumed in design. The proposed model shows that shelf angle deflections are smaller than what is determined from the other models. Use of the proposed model has the potential to reduce the size and installation costs of shelf angles.

**KEYWORDS:** *cavity wall, design, energy conservation, shelf angle, veneer*

## INTRODUCTION

Relatively little has been published regarding the design of shelf angles for supporting masonry veneers. One of the first design narrative was assembled by Grimm and Yura [1], who covered a broad range of topics regarding masonry shelf angle design. Tide and Krogstad [2] presented a case study of

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an existing shelf angle that, based on the previous design recommendations and the given service loads, had exceeded the established failure criteria but was performing satisfactorily.

Energy code requirements for buildings have increasingly become more stringent in the amount of insulation required in the building envelope system. In order to satisfy the continuous insulation requirement, designers are opting to place the insulation in the wall cavity, which necessitates a larger width. The Masonry Society recently approved the publication of TMS 402-16 [3], which includes an increase of the maximum prescribed cavity width requirement for masonry veneers. Beginning with the 2016 edition, the prescribed cavity width, measured from the inside face of the masonry veneer to the backup, may be up to 168 mm (6 $^{5}/_{8}$ inches).

While previous editions of TMS 402 have not prohibited the use of larger cavity widths, the prescriptive requirements were not applicable for larger cavity widths and the veneer support system was required to be rationally designed. There appears to be some hesitancy on the part of designers to using the rational design approach. This hesitancy can be attributed to one or more factors: misunderstanding of the TMS requirements, unfamiliarity with masonry analysis, reduction of design cost, and/or aversion to risk. In keeping with the generalization of Parkinson's Law, the typical cavity width in US construction will almost certainly increase as the TMS 402-16 requirements are adopted by local jurisdictions.

Increasing the masonry cavity width affects several design parameters. Since the center of gravity of the veneer is typically outboard of the structural framing, the masonry veneer support system is typically composed of multiple components which transfer the gravity load of the veneer to the supporting structure. These components vary depending on the selected framing system and the façade attachment strategy, and may include: shelf angles, floor slabs, pour stops, wedge inserts, spandrel beams, roll beams, kickers, stiffeners, hangers, gusset plates, spacers, shims, and connectors [4]. Using the current design rationale, increasing the cavity width will increase the load demand on these supporting members and necessitate larger members be used.

## BACKGROUND

Shelf angle design is not as simple and intuitive as it might appear at first glance. The interaction between masonry and shelf angles is complex due to the greater flexural stiffness of the masonry veneer, rotation of the angle principal axes, and eccentrically-placed loading on the angle. As in other engineering problems, simplifications are frequently made to ease and speed analysis. However, in the case of shelf angle design, these simplifications can result in overly conservative designs that unnecessarily increase the cost of the shelf angle and its installation.

Shelf angle design is fundamentally based on verifying that the selected angle section will not exceed the prescribed demand and deflection limit states. The stresses and deflections of a shelf angle are dependent on the load distribution of the brick veneer. Conversely, due to the rigidity of the veneer, the load distribution is dependent on the vertical deflection of the shelf angle. This second fact does not appear to be widely considered, except by some authors [1][2][6].

Vertical deflection of the shelf angle has three components: torsional rotation of the angle, longitudinal bending of the angle, and local bending of the horizontal leg, which are illustrated in Figure 1. Using superposition, the total deflection can be calculated as the sum of the three parts. As segments of a shelf angle deflect due to torsion and bending, the loading on those segments will generally decrease. Conversely, as the cantilevered toe bends downward relative to the heel, the resultant force resisting the veneer load will increase. This complex relationship between load and deformation cannot be adequately described using common design approaches.

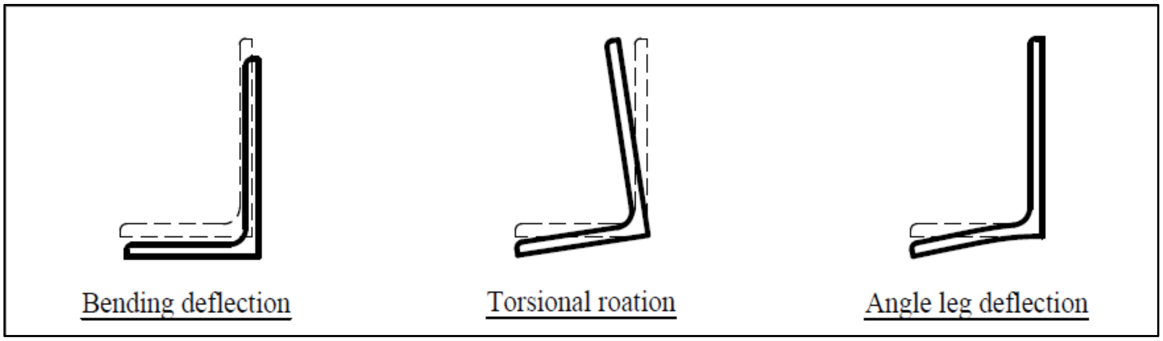


**Figure 1: Vertical deflection components of shelf angles**

***Load Distribution***

One approach that is frequently used in shelf angle design is to assume that the masonry arches over the opening, resulting in a triangular load distribution with the peak at mid-span. Such approach implicitly assumes that the masonry has no rupture strength so the triangular segment beneath the arching section must be supported. In cases where arching is not practical or simplified analysis is desired, the masonry load is frequently assumed to be uniformly distributed over the shelf angle span, which implies that the masonry remains plastic during construction.

The triangular load distribution places more load at the midspan of the shelf angle where the eccentricity of the load creates the greatest torsional rotation and deflection of the angle. The uniform distribution eliminates the peak at the midspan but typically results in an overall greater load and deflection because the load is typically assumed to be the full height of the masonry panel whereas the midspan height of the triangular distribution is only half the span length. Both load distributions are incorrect for most cases, are not representative of the actual masonry load on the shelf angle, and will result in excessively large angle section and/or smaller support spacing, leading to increased construction costs.

As the masonry above a shelf angle is constructed and the mortar sets, the rigidity of the masonry will typically exceed that of the steel angle supporting it. In many cases, the masonry is stiffer by several orders of magnitude [2][6] and can be assumed to act as a rigid body. As the masonry height increases, the masonry weight spanning between the supports is increasingly resisted by the masonry itself.

For the masonry to be considered rigid, the masonry must have enough strength to span between the shelf angle supports. Based on the modulus of rupture values from TMS 402-16, only a few courses of unreinforced masonry are needed to support its own weight for shorter span lengths. The depth required for masonry to be self-supporting increases linearly with increasing span length and is typically less than that prescribed by masonry arching theory.

Tide and Krogstad [2] noted that it is reasonable to assume that the bottom courses of masonry above a shelf angle will behave plastically until the mortar sets. Since the height of brick laid in a workday does not typically exceed a few feet, the bottom courses of brick will have obtained sufficient rigidity and strength to resist the next day's brickwork. As subsequent courses are added over the following days, the midspan of the shelf angle will deflect more than the masonry being supported, resulting in decreased loading at the midspan and increased loading at the supports.

***Bearing Point***

The bearing point of the veneer is commonly assumed to be located at the center of gravity of the masonry. Grimm and Yura [1] noted that as the horizontal leg of the angle deflects downward due to the weight of the masonry, the bearing location of the masonry will move inward. The authors assumed the bearing point to be located two-thirds of the bearing width from the toe of the angle, which is analogous of a triangularly-sloped load distribution. Tide and Krogstad [2] argued that the assumption by Grimm and Yura is only valid if the angle were shored during construction and assumed that the bearing point would move further inward to the inside face of the veneer.

Tide and Krogstad are correct in assuming that the veneer bearing point is located closer to the inward face due to the outward sloping across the angle leg due to leg deflection and angle rotation. However, the recommendation to calculate the eccentricity at the inner face may become overly unconservative for greater masonry heights. Vertical bearing load of the masonry on the shelf angle must by distributed over an equivalent bearing width $a$. The center of this stress block will be located outward and away from the inward face of the masonry by a distance $^{a}/_{2}$. Using the bearing strength provisions from TMS 402-16, the width of the bearing area is

$$a = \frac{w_u}{0.8 f'_m} \tag{1}$$

The load eccentricity of the veneer is

$$e = b_{vnr} + \frac{a}{2} - \frac{t}{2} \tag{2}$$

where $b_{vnr}$ is the distance from the angle heel to the inner face of the veneer and $t$ is the thickness of the angle. The $t/2$ factor is used to adjust for the distance from the heel to the shear center of the angle.

Moving the bearing resultant back toward the inner face of the brick reduces the torsional eccentricity and demand on the shelf angle. For a typical nominal 200 mm (4 inch) brick, the distance from the centroid to the inside face of veneer is approximately 95 mm (1.81 in.), which is almost as large as the increase in permissible cavity width introduced in TMS 402-16. By changing the location of the bearing point, the decrease in eccentricity almost cancels out the increase introduced by the wider cavity. Moving the bearing point requires the designer to verify that the first few rows of veneer anchors above the shelf angle can resist the overturning moment in addition to the negative wind load.

Tide and Krogstad [2] noted that the two-thirds requirement for bearing width should be maintained to facilitate construction of the initial courses of masonry until the anchors are installed and the mortar sets. Sufficient angle leg width is also important to provide adequate support for through-wall flashing.

***Torsion***

The greatest contribution to vertical deflection of the angle at the bearing point is typically due to torsional rotation caused by the eccentricity of the veneer load. Shelf angles and other open steel shapes have relatively poor torsional resistance properties [7]. The design approaches proposed by Grimm and Yura [1] and by Tide and Krogstad [2] both assume that torsional warping is unrestrained, which results in larger shelf angle rotations. Unrestrained warping is encountered in cases where both legs are free to warp at every support and would only occur in a single-span shelf angle that is pin-supported, i.e., a lintel. In typical cases, shelf angles are continuous across multiple supports, so warping at the middle supports is restrained.

Accounting for the warping restraint in the analysis requires the use of the torsion equation

$$T = GJ\theta' - EC_w\theta''' \tag{3}$$

The solution to the third-order differential equation (3) varies depending on the loading and boundary condition and is too complex to be practically used in hand calculations. Symbolic and graphical representations of many solutions are presented in AISC Design Guide 9 [7], but they are limited to relatively small spans and cannot be used for many shelf angle scenarios. In addition, the solutions presented in Design Guide 9 become ill-conditioned at larger span lengths and require algebraic manipulation in order for numerical methods to generate accurate results.

Given the computational tools that are available to modern designers and the repetitive nature of shelf angle design, the difficulty in accounting for warping restraint should no longer be an excuse to exclude it from analysis. Accounting for warping restraint in shelf angle analysis will decrease torsional rotation and, consequently, shear stresses in the angle section, which will permit the use of a smaller angle section.

***Flexural Bending***

The deflection due to torsion is several times larger than that due to flexural bending. Grimm and Yura [1] claim that deflection due to flexural bending is so small in comparison to the deflections caused by torsion that it can be neglected. Results from the study presented herein, however, show that vertical deflection caused from longitudinal bending cannot be neglected in shelf angle analysis.

Designers commonly use the geometric section properties of the angles for shelf angle design assuming bending about the axis normal to the wall surface, which is only valid if the angle is laterally restrained along its length. Even in cases where a shelf angle is attached to a slab edge or to a backup wall, the angle should be held off by shims at the supports to account for construction tolerances [4]. The variability inherent in construction practice makes it impractical to assume that lateral restraint can be provided without including special provisions for it in the design documents.

Tide and Krogstad [2] assumed a friction coefficient of 0.2 between the steel angle and masonry and that the friction could be used to restrain the angle section laterally; however, this assumption may not be valid in all circumstances. A common practice is to set the flashing on top of the shelf angle in a full bedding of non-skinning butyl sealant. Since the sealant remains permanently fluid, it effectually serves as a lubricant between the shelf angle and flashing, negating any possible lateral restraint that could be imposed by the masonry. As detailing of the through-wall flashing typically lies outside of the structural engineer's purview, any possible friction restraint from the masonry should be neglected.

When a shelf angle is not laterally restrained over its length, the use of geometric angle section properties leads to unconservative values for stresses and deflections. The principal axes of a shelf angle are oriented at an angle to the vertical direction. Any vertical load applied to the angle will cause bending about both the strong and weak axes. These forces are proportional to the load components in the principal axis directions.

When an angle is bent about its weak axis, the normal stresses at both toes have the same sign while the heel has the opposite sign. When an angle is bent about its strong axis, the normal stresses at the toes have opposite signs while the heel will be located near the neutral axis. Except in the case of equal leg angles, the distance of the two toes from either axis are not equal, indicating that in the typical case of a continuously spanning shelf angle, the normal stresses should be calculated at all three points for bending about each principal axis for maximum positive moment and again for the maximum negative moment. In addition, leg local buckling should be considered where the toes of the angle are in compression and lateral-torsional buckling should be checked. The moment capacity of a shelf angle computed using biaxial bending is always less than that assuming uniaxial bending about the geometric axis normal to the wall surface.

Biaxial bending of shelf angles due to vertical loading produces deflections in both principal axis directions. When these deflections are transformed back into the geometric coordinate system, the shelf angle deflection will have components in both the vertical and lateral directions. The lateral deflection is smaller than the vertical deflection and can generally be neglected in analysis. The total vertical deflection computed using the moments of inertia about the principal axes will always be larger than that assuming uniaxial bending about the geometric axis normal to the wall surface.

***Deflection from Leg Bending***

The cantilevered toe of the shelf angle will deflect under the load of the veneer. Designers typically assume that the veneer load is uniformly applied along the length of the angle. However, it has been established that the veneer load distribution is concentrated closer to the supports [1][2][6]. Since the cantilever deflection of the leg is proportional to the load, the local deflection of the toe relative to the heel will be the greatest at the supports and small or null at midspan. Such a deflection profile counteracts, at least partially, the downward deflection of the toe due to torsion rotation, which is null at the supports and greatest at midspan.

Grimm and Yura [1] assumed that, due to the relative stiffness of the masonry, the veneer is rigid and that the deflection along its bottom is uniform. They also assumed that the angle deflection at the bearing point is equal along its length. However, this assumption is not valid in all cases because the angle is not physically attached to the veneer. The angle deflection at midspan could exceed that of the veneer wherein a gap would open between the angle and the veneer.

Grimm and Yura [1] assumed a linear behavior for the cantilevered angle leg. Tide and Krogstad [2] used this assumption in an analysis of their case study and calculated a flexural strain that was twice the yield strength of the material. The authors noted that they observed no evidence of yielding of the angle leg during field observations; however, yielding would not have occurred at a deflection of less than 0.8mm ($^{1}/_{32}$ in.), which would not have been perceptible. In order for accurate results to be obtained, analysis should account for the bilinear behavior of the material.

***Deflection Limit***

The current recommendation is that shelf angle deflection under service loads should not exceed L/600 [3][5]. McGinley[6] conducted a finite element analysis of the interaction between shelf angles and masonry and concluded that the masonry is self-supporting and only bears near the supports, so limits on angle deflection should not be required since deflection will occur only during wall construction. Such an assertion would be correct if the entire veneer load were concentrated at the supports, but since the load is also on either side of the support, the load will contribute to midspan deflection. Even if the masonry does not bear on the angle at the midspan, the midspan deflection should be limited to prevent the angle from bearing on the masonry below.

A compressible filler is typically placed in the joint below the shelf angle. The filler prevents debris from lodging into the joint and allows for expansion of the brick veneer below the joint and deflection of the shelf angle above the joint. Compressible joint fillers have a limited range of compressibility and if the shelf angle deflection and brick expansion exceed that range, then part of the shelf angle load will be transmitted through the joint to the masonry below and could affect other elements in the wall. While shelf angle deflection may not be a limiting factor for the brick veneer that is being supported, it should be limited to prevent impact on other wall elements.

## PROPOSED MODEL

Shelf angle behavior can be more accurately described using the model presented in Figure 2. The masonry panel is assumed to act as a rigid body and the shelf angle is assumed to behave as a beam. The vertical deflection of the beam is taken as the sum of the bending deflection and the torsional rotation multiplied by the load eccentricity. The beam length is divided into a series of discrete segments and the cantilevered angle leg of each segment is assumed to behave as a bilinear spring. Each spring is assumed to have zero tensile resistance, linear compression resistance up to the yield strength, and constant compression resistance after the yield point, as shown in Figure 2. The series of independent springs is placed between the beam and the masonry panel.

The solution to the proposed model requires an iterative approach due to the interdependent relationship between the load and deflections and the assumed plastic behavior of the angle leg. Solutions are computed using the following general procedure:

1. The beam is divided into $n$ discrete segments and the spring properties of each segment are determined.
2. The masonry panel is assumed to deflect uniformly a small distance downward, causing the cantilevered leg to deflect.
3. The stress-strain relationship of each spring is used to calculate the load on each beam segment.
4. The load on each segment is used to calculate the bearing width and the load eccentricity.
5. The load on each segment is treated as a concentrated load applied at the middle of the segment. The bending deflection along the length of the beam is computed by summing the deflections caused by each concentrated load.
6. The concentrated load on each segment is multiplied by the respective eccentricity to determine the torque applied on the segment. The torsional rotation along the length of the beam is computed by summing the rotations caused by each concentrated load. The torsional deflection of each segment is calculated by multiplying the rotation by the eccentricity.
7. The bending and torsional deflections are subtracted from the masonry panel deflection to recalculate the cantilever deflection of the angle leg.
8. The steps above are repeated by using the leg deflection computed at the end of step 7 to recalculate the load distribution on the angle.

The solution is obtained when the difference between total vertical loads from subsequent iterations falls within a predefined tolerance. The process is repeated for increasingly larger panel deflections to generate a force-deflection curve for the shelf angle.

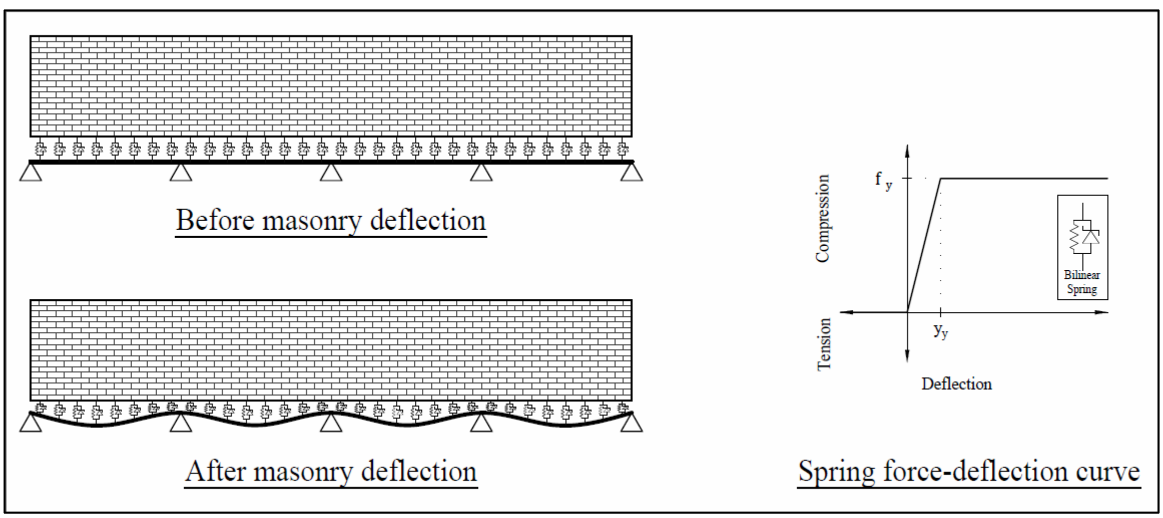


**Figure 2: Proposed model**

The above procedure was developed into an algorithm using Python. During implementation of the algorithm some additional considerations were required for successful implementation of the procedure. The torsion equations were modified algebraically into an equivalent form that is well-conditioned far larger span lengths to minimize the effect of compounded errors and produce accurate results. Additionally, the convergence of the algorithm was aided by the addition of a stabilization subroutine to smooth out fluctuations between subsequent iterations.

## CASE STUDY

An analysis was repeated on the case study specimen presented by Tide and Krogstad [2]. The properties of the shelf angle and masonry panel are provided in Table 1. The results of the proposed model are presented for comparison in Table 2 with the results from the Grimm and Yura [1] model, Tide and Krogstad [2] model, and uniformly loaded model.

**Table 1: Case Study Specimen Properties**

| **Masonry panel properties** | | | |
|---|---|---|---|
| Panel height | $h$ | 7.01 m | (23 ft) |
| Masonry load | | 0.276 MPa | (40 psf) |
| Span length | $L$ | 1981 mm | (78 in.) |
| Total weight | $P_a$ | 26.6 kN | (5.98 kip) |
| Cavity width | $b_{vnr}$ | 60.3 mm | (2 $^3/_8$ in.) |
| **Steel shelf angle properties** | | | |
| Steel section | | L152x192x9.5 LLV | (L6x4x3/8 LLV) |
| Yield stress of steel | $f_y$ | 248 MPa | (36 ksi) |
| Thickness of angle | $t$ | 9.53 mm | ($^3/_8$ in.) |
| Elastic modulus | $E$ | 200 GPa | (29,000 ksi) |
| Shear modulus | $G$ | 77.2 GPa | (11,200 ksi) |
| Torsional constant | $J$ | 73,700 mm$^4$ | (0.177 in.$^4$) |
| Warping constant | $C_w$ | 99,100,000 mm$^6$ | (0.369 in.$^6$) |

**Table 2: Case Study Results for Various Shelf Angle Models**

| Approach | Deflection at Midspan | | Deflection at Support | |
|---|---|---|---|---|
| Uniform | 13.2 mm | (0.518 in.) | 0.0011 mm | (0.000044 in.) |
| Grimm & Yura | 0.717 mm | (0.0282 in.) | 0.717 mm | (0.0282 in.) |
| Tide & Krogstad | 2.62 mm | (0.103 in.) | 2.37 mm | (0.0932 in.) |
| Proposed | 0.517 mm | (0.0204 in.) | 0.485 mm | (0.0191 in.) |

The load distribution of the masonry panel on the shelf angle for the proposed model is shown in Figure 3. The load distribution is very different from the uniform or triangular distributions that are commonly assumed in analysis. The shape is similar to that assumed by Grimm and Yura [1] in that the load is greatest at the supports, but Grimm and Yura did not consider that the load could be zero in the vicinity of the midspan. Tide and Krogstad [2] assumed a uniform load of masonry with a height equal to half the bolt spacing. The height of masonry bearing on the shelf angle was only 14% of the total panel height and yet the calculated deflection was approximately 3.7 times greater than that calculated using the Grimm model and five times that calculated by the proposed model. The Tide and Krogstad model will become increasingly over-conservative as the ratio of panel height to spacing length decreases.

The vertical deflection along the top of the angle is shown in Figure 4. The lines between the heel and toe deflections represent the deflection at ¼ in. (6.8 mm) increments along the width of the cantilevered leg measured perpendicular to the longitudinal axis. The shaded regions represent the area where the brick panel bears on the shelf angle. The deflection of the toe is greater than that of the brick and it must be considered when specifying the joint filler to be placed between the bottom of the shelf angle and the brick panel below the angle.

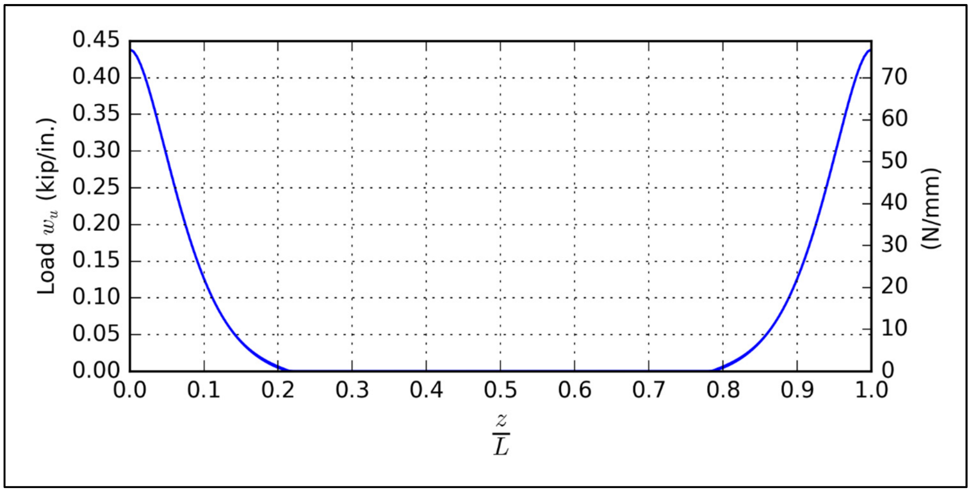


**Figure 3: Variation of distributed load along the length of shelf angle**

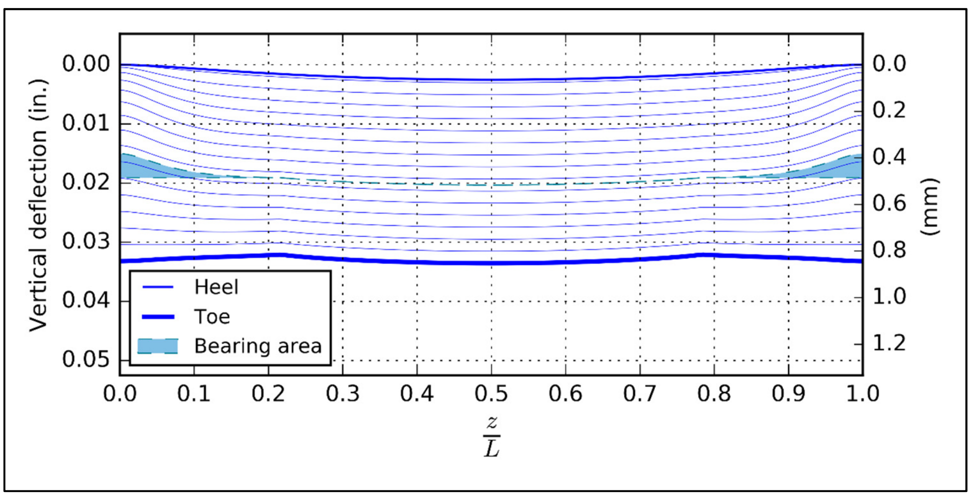


**Figure 4: Deflections along the horizontal leg of the shelf angle**

To aid in visualizing the deflected shape of the angle, cross sections of the angle at the supports ($^z/_L$ = 0 and 1) and at the midspan ($^z/_L$ = 0.5) are shown in Figure 5. The calculated deflections were nearly unperceivable, so the deflections in Figure 5 have been magnified to better present the deflected shapes. At the supports, the deflection is solely due to cantilever deflection of the angle leg because the other two modes of deflection are restrained by the supports. At the midspan, the deflection is a result of longitudinal flexure and torsional rotation. The much-stiffer masonry bridges across the middle of the span and does not bear on the angle, so there is no load and, consequently, no cantilever deflection of the leg at the midspan.

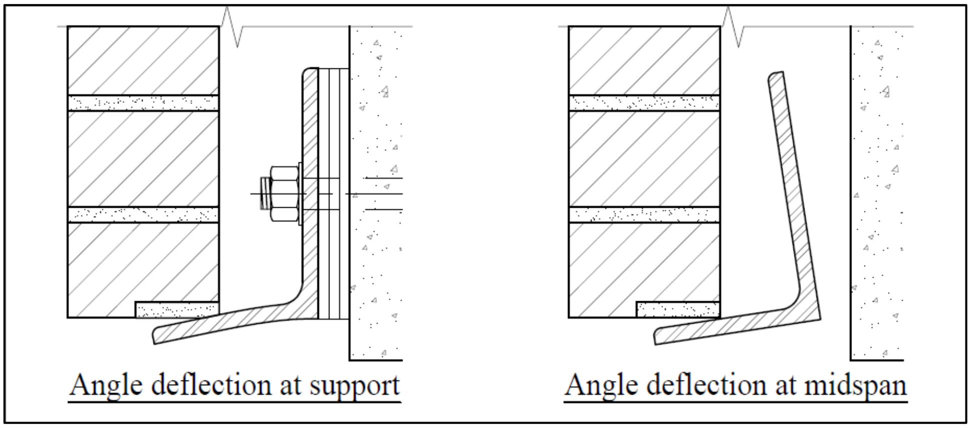


**Figure 5: Magnified deflected cross-sections**

## CONCLUSIONS

There is no consensus among design professionals nor in the masonry literature about the correct procedure for the design of shelf angles. The analysis model presented herein more accurately accounts for the interaction between the masonry panel and the shelf angle. A comparison of analysis approaches confirmed that most approaches produce results that are over-conservative. The proposed model indicates that actual shelf angle deflections are smaller than what is determined from the other models. The model confirms the previous observation made by Tide and Krogstad that shelf angles can support greater loads than what is assumed using traditional design approaches without failing. Use of the proposed model has the potential to reduce the size and installation costs of shelf angles. Future work should be performed to validate the proposed model through experimental testing.

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