Design of Directly Bolted Shelf Angles Using Force Method and Virtual Work

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ABSTRACT

Shelf angle design represents the intersection of masonry design, steel design, and depending on the building's primary structural system, concrete, wood, or steel design. Therefore, shelf angle research in this area of masonry is often regarded as a "steel design" or "concrete design" problem, rather than a masonry design problem. There are few papers published on the topic and even less actual testing, with the experimental work to date focusing on shelf angles anchored to wood-frame floors. Complex interactions between the tied masonry veneer and the steel angle, as well as beam behavior of the brick veneer between anchor bolts are difficult to capture with models simple enough for hand calculations. A new design approach is proposed which more accurately accounts for the interaction between the tied masonry veneer and the shelf angle. The proposed design method more accurately reflects field observations of masonry veneer where a L102mm mm x 102mm x 6.4mm (L4in. x 4in. x ¼ in.) supports 7.315 to 9.144 m (24 to 30 feet) of 90 mm (3-5/8 in.) clay brick veneer without evidence of structural distress. This translates into a 5% to 8% cost savings on the shelf angle. The proposed design method uses the Force Method in combination with Virtual work to solve the 1-degree statically indeterminate system that results from the introduction of the tie restraining force at the first course of ties. This new method was then compared to the traditional statically determinate method as well as 2D and 3D finite element models. The proposed method also allows parameters like; veneer height, veneer type, shelf angle size and thickness, air space depth, and bolt hole location to be easily altered without the time-consuming effort of redrawing the system in finite element software.

KEYWORDS

Shelf Angle Design, Masonry Veneer, Deflection, Force Method, Virtual Work.

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INTRODUCTION

Shelf angle design represents the intersection of masonry design, steel design, and depending on the building's primary structural system, concrete, wood or steel design. Therefore, shelf angle research in this area of masonry is often regarded as a "steel design" or "concrete design" problem, rather than a masonry design problem. There are few papers published on the topic [3], [4], [5], [6], [7], and even less actual testing, with the experimental work to date focusing on shelf angles anchored to wood-frame floors [1], [2]. In fact, Dillon concluded that there is no consensus by design professionals or the literature as to the best approach for shelf angle design and suggested the use of readily available finite element models is the best solution [7]. Shelf angle design is not as simple and intuitive as previously thought [7]. Complex interactions between the tied masonry veneer and the angle and beam behavior of the brick veneer between anchor bolts are difficult to capture with models simple enough for hand calculations. Design simplifications to a statically determinate system are often used [6] but produce conservative results that unnecessarily increase the size and cost of the shelf angle as well as the size and frequency of the anchor bolts. A statically determinate system is easily solved using simple statics, but ignores the restraining force produced by the masonry ties and the beam action of the brick veneer identified by McGinley [5]. These impacts on the shelf angle can be captured in 3D structural models with finite element software as recommended by Dillion [7], but as mentioned, are difficult to reproduce with simple hand calculations. The introduction of the first course of masonry ties results in a statically indeterminate problem. However, ignoring the restraining force of the ties produces shelf angle designs that appear to be extremely conservative to performance of these systems as observed in the field, particularly when it comes to estimating the deflection. Fig. 1 demonstrates the impact of both the beam effect of the brick veneer and the restraining force of the ties on a brick veneer of an actual brick veneer in the field. On this building, the shelf angle supporting an approximately 20-foot span of clay brick veneer has fallen to the ground due to anchor bolt withdrawal. The horizontal crack developing into step cracking has formed due to the absence of support of the weight of the veneer, but the veneer has not fallen to the ground. It appears to be the result of the brick veneer supporting its own self weight through beam behavior (or flat arch arching action) between anchor bolts (Fig. 1), while the masonry ties continue to laterally anchor the veneer to the wall.

Figure 1: Failed Shelf Angle – Illustration of Beam Action and Masonry Ties on Veneer due to shelf angle support failure with step cracking visible

Therefore, a new design approach is proposed which more accurately accounts for the interaction between the tied masonry veneer and the shelf angle[8]. The proposed design method more accurately reflects field observations of masonry veneer where a L102mm mm x 102mm x 6.4mm (L4in. x 4in. x ¼ in.) supports 7.315 to 9.144 m (24 to 30 feet) of 90 mm (3-5/8 in.) without evidence of structural distress. It is clear from "as-built" performance of masonry veneers that the deflections and load distribution on a shelf angle are less than those estimated with current simplified design methods. Fig. 2 illustrates a typical brick veneer with wood stud back up detail at the foundation used in Alberta, Canada.

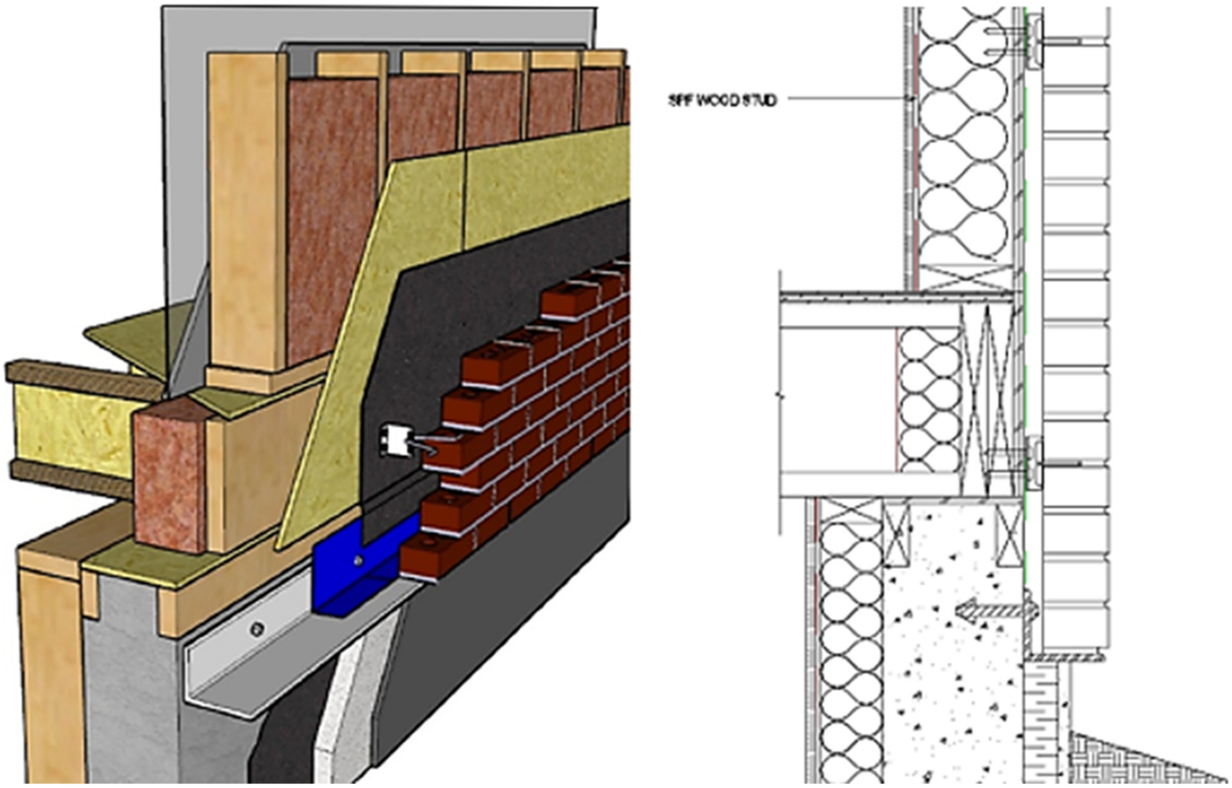

Figure 2: Brick Veneer Wood Stud Backup Wall – Foundation Detail

Design simplifications to a statically determinate system are often used [6]. However, this method produces conservative results compared to finite element models and field observations. The impact of the masonry ties and deflection reductions introduced by the stiffness contributions of the brick can be captured with 3D finite element [7] software using beam, shell or solid elements. However, 3D finite element models are typically too complex for hand calculations. The proposed design method employs the use of the Force Method in combination with Virtual work to solve the 1-degree statically indeterminate system that results from the introduction of tie restraining force at the first course of ties. This new method is compared to the simplified statically determinate method as well as 2D and 3D finite element models with the SAP 2000 software. Use of the proposed design method reduces the size of the shelf angles and anchor bolts and better reflect actual performance of constructed masonry veneers at the same height. The proposed method also allows parameters like; veneer height, veneer type, shelf angle size and thickness, air space depth, and bolt hole location to be easily altered without the time-consuming effort of redrawing the system with 3D finite element software.

FORCE METHOD AND VIRTUAL WORK TO DETERMINE THE REACTIONS

To better represent the actual performance of the system, the restraining force provided by the ties should be accounted for in the calculation creating a statically indeterminate structure as depicted in Fig. 3.

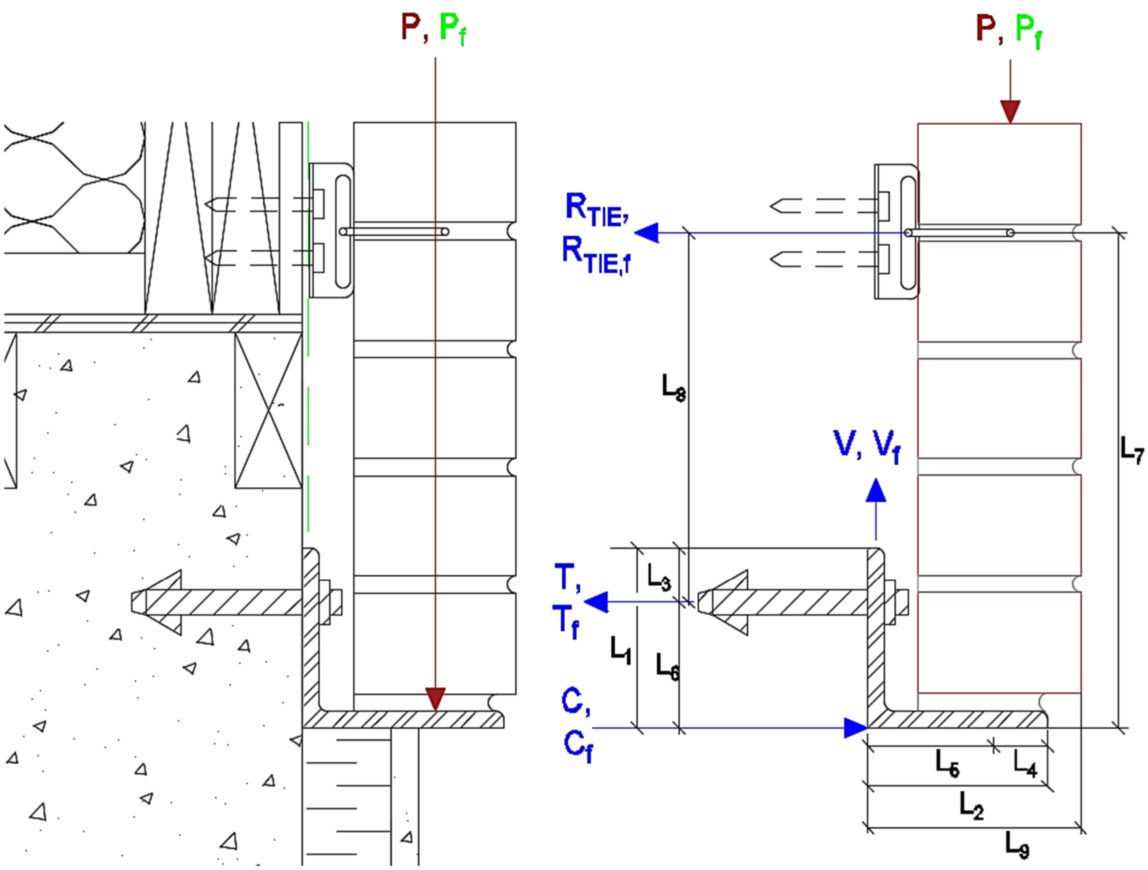

Figure 3: Free body Diagram of Shelf Angle at Foundation – Statically indeterminate.

The following values were used to populate Fig. 3 parameters:

P = Unfactored (service load) of masonry veneer and shelf angle (N) per meter = 17,614 N (3960 lbf)
P_f = Factored Load of masonry veneer and shelf angle (N) per meter = 24,659 N (5544 lbf)
R_{TIE} = Reaction force in the brick ties at the first course of ties (kN) per meter
V_f = Reaction force in the brick ties at the first course of ties (kN) per meter
T_f = Reaction force in the brick ties at the first course of ties (kN) per meter
C_f = Reaction force in the brick ties at the first course of ties (kN) per meter
L1 = Vertical Leg length 102.6 mm
L2 = Horizontal Leg length = 102.6 mm
L3 = Vertical distance to the center of the bolt hole = 38.1 mm
L4 = Centroid of brick veneer (typically 45 mm for metric modular 92 mm brick)
L5 = Eccentricity of veneer load = Air space + (veneer thickness / 2) = (25.4 + 46.1) = 71.5 mm
L6 = L1 - L3 (mm)
L7 = Max 300 mm from base support = 278.35 mm.
L8 = Vertical distance between the ties and the center of the bolt hole = 214.9
L9 = Total length of horizontal leg = air space + veneer thickness = 25.4 + 92.1 = 117.5 mm
t_{angle} = thickness of the horizontal leg of the angle = 6.35 mm
b = 1000 mm (1 meter of wall design length)

veneer_thick = thickness of the masonry veneer = 92.1 mm
V-tie_radius = 2.4 mm
f'_m = the compressive strength of the masonry veneer = 12 MPa
E_m = Modulus of Elasticity of the masonry veneer = 850f'm
Es = Modulus of Elasticity of structural steel = 200,000 MPa
$I_{angle} = b \cdot t_{angle}^3 / 12 = 21,337 \text{ mm}^4$
$I_{veneer} = b \cdot t_{veneer}^3 / 12 == 65,102,497 \text{ mm}^4$
$I_{tie} = 4 \cdot [\pi \cdot (\text{V-tie\_radius})^4 / 4] = 104.23 \text{ mm}^4$

To use the Force Method, the statically indeterminate structure was made determinate by the introduction of a release. In this case the release chosen was at coordinate 1 in Fig 4.

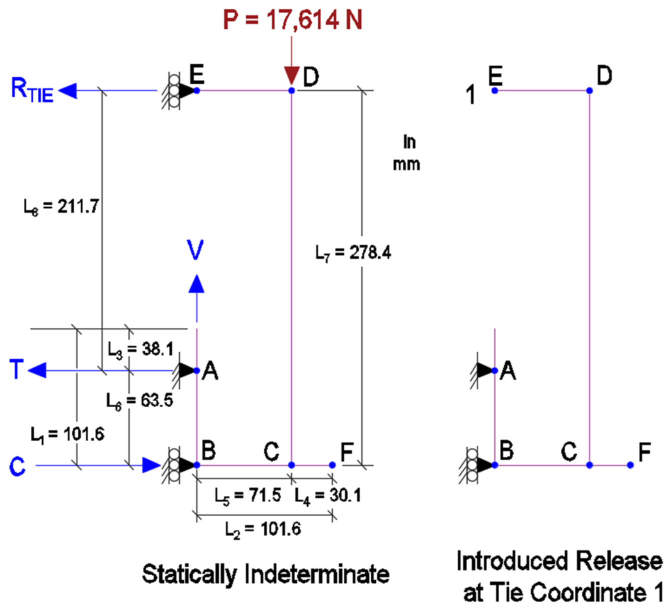

Figure 4: Force Method Steps and Reactions on the Released Structure.

The service load, P and a unit load at coordinate 1 were then applied to the released (determinate) structure. The reactions were solved with 2D statics and the bending moment diagrams generated for these loadings. The results are presented in Fig. 5 without proof.

Calculation of Displacements and Flexibility Using Virtual Work

Virtual work is required to determine values for the displacement, *D* and the flexibility, *f*. The general form for virtual work for moments and neglecting axial deformations is:

(1) $$\Delta = \int \frac{m_u \cdot M_S}{E \cdot I}$$

Where m_u is the moment equation when a unit load is applied at the release and M_s is the moment equation for the released (statically determinate) structure subjected to the loading. The bending moment diagrams (Fig. 5) were determined from the reactions when the service load was applied to the released structure (Fig. 4) and the when the unit load was applied to the released structure (Fig. 4). The displacements D_n and flexibilities f_n parameters of the four (4) frames creating the structure must be aggregated into a total using Eq. 2. Starting at the anchor bolt and moving counterclockwise, member 1 is A to B, member 2 is B to C, member 3 is C to D, and member 4 is D to E. The moment equations to determine the displacements, in Eq. (3) to (7) can be obtained from Fig. 5 and are presented without proof.

(2) $D_n = \int_0^{L_i} \frac{[m_{u,n}] \cdot [M_{s,n}]}{E_n \cdot I_n} dx$ and $f_n = \int_0^{L_i} \frac{[m_{u,n}] \cdot [m_{u,n}]}{E_n \cdot I_n} dx$

(3) $D_1 = \int_0^{L_6} \frac{[T_u \cdot x] \cdot [T_s \cdot x]}{E_s \cdot I_{angle}} dx = \int_0^{63.5} \frac{(-4.3835 \cdot -19{,}818.9) \cdot x^2}{E_s \cdot I_{angle}} dx$ = 1.7375 mm

(4) $D_2 = \int_0^{L_5} \frac{[T_u \cdot L_6] \cdot \left[\frac{T_s \cdot L_6}{L_5} \cdot (L_5 - x)\right]}{E_s \cdot I_{angle}} dx = \int_0^{71.5} \frac{[-278.35] \cdot [17614x - 1{,}258{,}497]}{E_s \cdot I_{angle}} dx$ = 2.9325 mm

(5) $D_3 = \int_0^{L_7} \frac{[L_7 - x] \cdot [0]}{E_m \cdot I_{veneer}} dx = 0$

(6) $D_4 = \int_0^{L_7} \frac{[0] \cdot [0]}{E_s \cdot I_{tie}} dx = 0$

(7) $D = \sum_{n=1}^{4} D_n = 4.6701$ mm

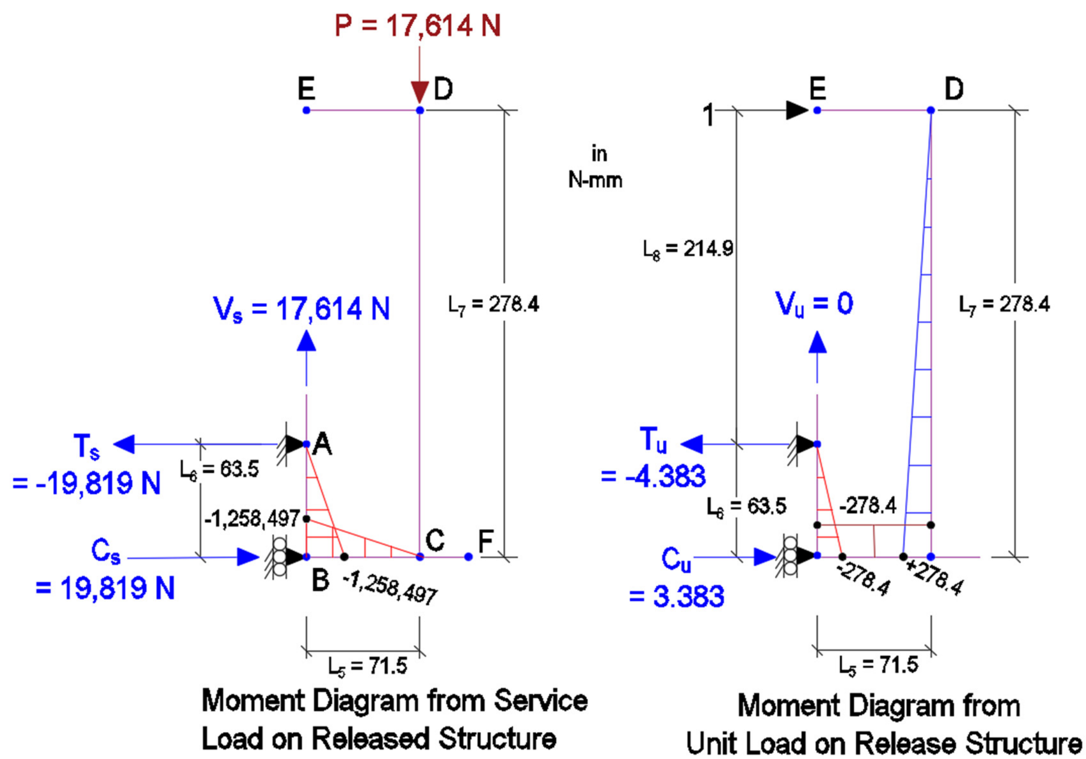

Figure 5: Reactions and Moment Diagrams of the Released Structure.

Next the flexibility *f* is calculated from the moment equations determined from Fig. 5. Once again, the moment equations are presented without proof in the following flexibility equations.

(7) $f_1 = \int_0^{L_6} \frac{[T_u \cdot x] \cdot [T_u \cdot x]}{E_s \cdot I_{angle}} dx = \int_0^{63.5} \frac{(-4.3835 \cdot x)^2}{E_s \cdot I_{angle}} dx$ = 0.00038429 mm/N

(8) $f_2 = \int_0^{L_5} \frac{[T_u \cdot L_6] \cdot [T_u \cdot L_6]}{E_s \cdot I_{angle}} dx = \int_0^{71.5} \frac{(-4.3835 \cdot 63.5)^2}{E_s \cdot I_{angle}} dx$ = 0.0012972 mm/N

(9) $f_3 = \int_0^{L_7} \frac{[L_7 - x] \cdot [L_7 - x]}{E_m \cdot I_{veneer}} dx = \int_0^{278.4} \frac{(278.35 - x)^2}{E_m \cdot I_{veneer}} dx$ = 0.0000108256 mm/N.

(10) $f_4 = \int_0^{L_5} \frac{0 \cdot 0}{E_s \cdot I_{tie}} dx = 0$

(11) $f = \sum_{n=1}^{4} f_n = 0.0016923$ mm/N

Calculation of the Redundant Force (*R_{TIE}*) at the Release

The redundant force can now be calculated according to Eq. (12).

(12) $F = R_{TIE} = -f^{-1} \cdot D = -\left(0.0016923 \ \frac{mm}{N}\right)^{-1} \cdot 4.6172 \ mm. = -2{,}760 \ N = -2.760 \ kN$

However, a generic equation for the redundant force, F is ideal as it permits easy manipulation of parameters and easy implementation in either a spreadsheet or programmed into an app. MathCAD was used to describe $R_{TIE} = F$ symbolically, and yielded:

(13) $$R_{TIE} = \frac{\frac{-P \cdot \left(2 \cdot L_5 \cdot L_6 \cdot L_7 + 3 \cdot L_5{}^2 \cdot L_7\right)}{6 \cdot E_S \cdot I_{angle}}}{\frac{L_6 \cdot L_7{}^2 + 3 \cdot L_5 \cdot L_7{}^2}{3 \cdot E_S \cdot I_{angle}} + \frac{L_7{}^3}{3 \cdot E_m \cdot I_{veneer}}} = \frac{\frac{-17614 \cdot (2 \cdot 71.45 \cdot 63.5 \cdot 278.35 + 3 \cdot 71.45^2 \cdot 278.35)}{6 \cdot 200{,}000 \cdot 21{,}337}}{\frac{63.5 \cdot 278.35^2 + 3 \cdot 71.45 \cdot 278.35^2}{3 \cdot 200{,}000 \cdot 21{,}337} + \frac{(278.35)^3}{3 \cdot 10{,}200 \cdot 65{,}102{,}497}} = \text{-2,760 N}$$

Entering values in Eq. (13) from the “Statically Indeterminate” structure in Figure 4 yields the identical result to the complicated integration equations above.

Calculation of the Reactions of the Statically Indeterminate structure using the Redundant

Once the Redundant it is known, 2D static equilibrium can be used to solve for the remaining reactions. Without proof, the remaining reaction equations are:

(14) $V = P = 17{,}614 \ N = 17.61 \ kN$

(15) $C = \frac{(P \cdot L_5 + R_{Tie} \cdot L_8)}{L_6} = 10{,}483 \ N = 10.48 \ kN$

(16) $T = -(C + R_{Tie}) = -7{,}723 \ N = -7.723 \ kN$

The results for the reactions factored for dead load only (1.4 DL) using the values for the L102x102x6.4mm (L4in. x 4 in. x ¼ in.) shelf angle supporting 30 feet of veneer in this example are easily determined by either substituting P_f = 24,659N for P into Eq. (13) to (16) or simply multiplying R_{TIE}, V, C and T by 1.4. The factored reactions without proof are:

(17) $R_{TIE,f} = 1.4 \cdot R_{TIE} = -3{,}864 \text{ N} = -3.864 \text{ kN}$

(18) $V_f = 1.4 \cdot V = 24{,}659 \ N = 24.66 \ kN$

(19) $C_f = 1.4 \cdot C = 14{,}675 \ N = 14.67 \ kN$

(20) $T_f = 1.4 \cdot T = -10{,}812 \ N = -10.81 \ kN$

Virtual Work to Determine the Deflection at Point C

Virtual Work must once again be used, however, this time it is used to determine the defection at point C. Given the similarity to the virtual work procedure in the previous section the moment equations from the moment diagrams in Fig. 6 for the deflection at point C will be given without proof.

It is important to note that the statically indeterminate structure is now used and that the generic equation for R_{TIE} must be applied when solving the reactions to generate the moment diagram of the unit load applied at C.

(21) $R_{TIE,u\Delta} = \dfrac{\dfrac{-1\cdot\left(2\cdot L_5\cdot L_6\cdot L_7+3\cdot L_5{}^2\cdot L_7\right)}{6\cdot E_S\cdot I_{angle}}}{\dfrac{L_6\cdot L_7{}^2+3\cdot L_5\cdot L_7{}^2}{3\cdot E_S\cdot I_{angle}}+\dfrac{L_7{}^3}{3\cdot E_m\cdot I_{veneer}}}$ = -0.1567 N

(22) $V_{u\Delta} = 1\ N$ Where the values were defined above

(23) $C_{u\Delta} = \dfrac{(1\cdot L_5+R_{Tie}\cdot L_8)}{L_6} = 0.5951\ N$

(24) $T_{u\Delta} = -\left(C_{u\Delta} + R_{TIE,u\Delta}\right) = -0.4384\ N$

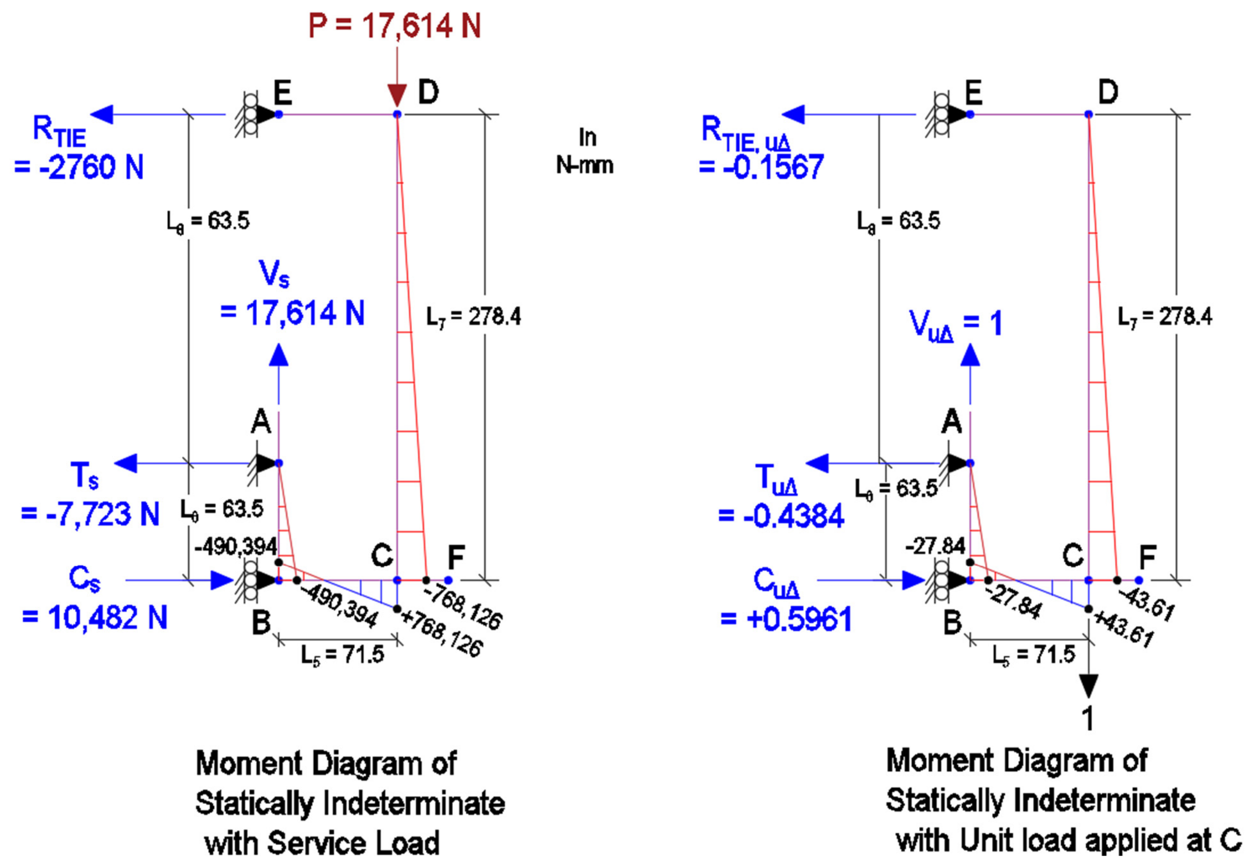

Figure 6: Reactions and Moment Diagrams of the Statically Indeterminate Structure.

(21) $\Delta_1 = \int_0^{L_6} \dfrac{[T_{u\Delta}\cdot x]\cdot[T_S\cdot x]}{E_S\cdot I_{angle}}\,dx = \int_0^{63.5} \dfrac{(-0.4384\cdot -7723)\cdot x^2}{E_S\cdot I_{angle}}\,dx$ = 0.0677 mm

(22) $\Delta_2 = \int_0^{L_5} \dfrac{[T_{u\Delta}\cdot L_6 + V_{u\Delta}\cdot x]\cdot[T_S\cdot L_6 + V_S\cdot x]}{E_S\cdot I_{angle}}\,dx = \int_0^{71.5} \dfrac{[x-27.84]\cdot[17614.x-490{,}394]}{E_S\cdot I_{angle}}\,dx$ = 0.1438 mm

(23) $\Delta_3 = \int_0^{L_7} \dfrac{\left[\frac{T_{u\Delta}\cdot L_6 + V_{u\Delta}\cdot L_5}{L_7}\cdot(L_7-x)\right]\cdot\left[\frac{T_S\cdot L_6 + V_S\cdot L_5}{L_7}\cdot(L_7-x)\right]}{E_m\cdot I_{veneer}}\,dx = \int_0^{278.4} \dfrac{[0.1567x-43.6x]\cdot[2{,}760x-768{,}126]}{E_m\cdot I_{veneer}}\,dx$

= 0.00468 mm

(24) $\Delta_4 = \int_0^{L_7} \dfrac{[0]\cdot[0]}{E_S\cdot I_{tie}}\,dx = 0\ mm$

(25) $\Delta_C = \sum_{i=1}^{4} \Delta_i = 0.2162$ mm

The generic formula for the deflection Δ from MathCAD at point C is:

(26) $$\Delta_C = \frac{3 \cdot L_5{}^2 \cdot L_6 \cdot (\boldsymbol{P} \cdot T_{u\Delta} + T_S) + 2 \cdot L_5{}^3 \cdot \boldsymbol{P} + 6 \cdot L_5 \cdot L_6{}^2 \cdot T_S \cdot T_{u\Delta} + 2 \cdot L_6{}^3 \cdot T_S \cdot T_{u\Delta}}{6 \cdot E_S \cdot I_{angle}} + \left(\frac{L_7 \cdot (L_6 \cdot T_S + L_5 \cdot \boldsymbol{P}) \cdot (L_6 \cdot T_{u\Delta} + L_5)}{3 \cdot E_m \cdot I\_veneer} \right) = 0.2162 \text{ mm}$$

Once the value of the deflection at point C was obtained, the deflection at point F was estimated using the equation for deflection of a cantilever of Length, L subjected to an end moment, M (Eq. 27) where the end moment can be obtained from Fig. 6 and is equivalent to R_{TIE} multiplied by L_7:

(27) $$\Delta_{cantilever} = \left(\frac{ML}{2 \cdot E \cdot I} \right)$$

(28) $$\Delta_F = \left(\frac{-R_{TIE} \cdot L_7}{2 \cdot E_S \cdot I\_angle} \right) \cdot L_4 + \Delta_C \quad = 0.2189 \text{ mm}$$

2D Finite Element Model Comparison

To confirm the accuracy of the results using the Force Method/Virtual Work approach, a 2D finite element model with frame elements was created with the SAP 2000 finite element software. A 3D rendering of the 2D model can be seen in Fig. 8. The frame elements were assigned a 1 m in length and the same dimensional properties as described in the parameters below Fig. 3. The reactions and deflections obtained from this model were tabulated in Table 1 for easy comparison to the Force Method/Virtual Work approach.

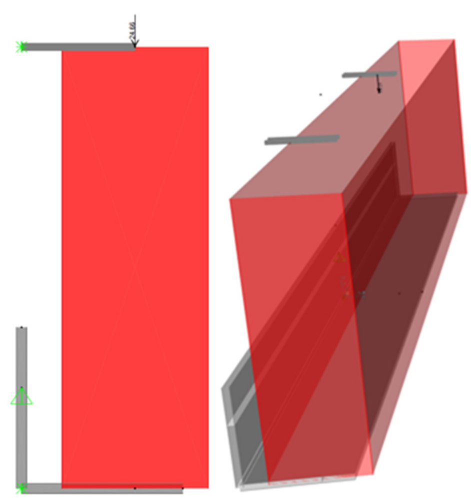

Figure 8: SAP 2000 Model of Force Method/Virtual Work Approach using Frames

3D Finite Element Model Comparison

A 3D finite element model with shelf angle and brick veneer modeled with shell elements and masonry ties modeled with frame elements was undertaken (Fig. 9). The results were used to compare the accuracy of the Force Method/Virtual Work approach and the 2D Finite Element model as both are unable to account for torsional effects between the anchor bolts. A 1016 mm (approximately 1-meter-long shelf angle section was explored with anchor bolts at 406 mm o.c. The reactions and deflections obtained from this model were tabulated in Table 1.

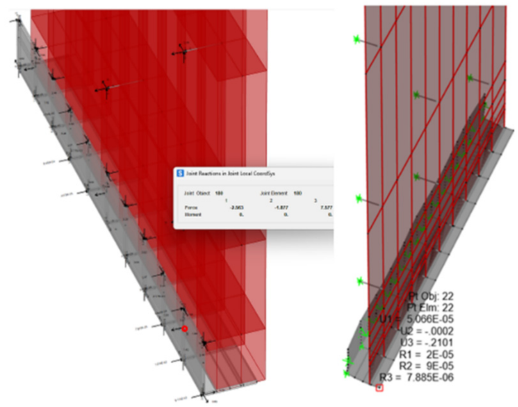

Figure 9: SAP 2000 Model Using Shells for Angle/Veneer and Frames for Ties.

Discussion

A comparison between the statically determinate system typically used to design shelf angles [6], the proposed Force Method/Virtual Work method, and the 2D and 3D finite element (FE) models was undertaken. The results can be found in Table 1 below.

Table 1: Comparison of results for a shelf angle supporting 9.144 m (30 ft.) of Brick Veneer

Model	Tie Reaction $R_{Tie,f}$ (kN/m)	Anchor Bolt Loads V_f (kN/m)	Anchor Bolt Loads T_f (kN/m)	Found. Load C_f (kN/m)	Max. Deflect. (mm)
Statically Determinate	**N/A**	**24.66**	**27.75**	**27.75**	**1.544**
Force Method/Virt. Work	**3.863**	**24.66**	**10.81**	**14.67**	**0.2189**
SAP 2000 – 2D FE Model	**3.865**	**24.66**	**10.81**	**14.67**	**0.2249**
SAP 2000 – 3D FE model	**3.776**	**25.07**	**8.384**	**12.63**	**0.2101**

From Table 1 it can be seen that the statically determinate model provides extremely conservative results with a deflection that is 7.4 times the value of the SAP 2000 – 3D FE model. The Force Method/Virtual Work approach produced a deflection value 9.4% larger than the SAP 2000 – 3D FE model. It can also be seen that the Force Method/Virtual Work approach produced nearly identical results to the SAP 2000 – 2D FE model (within 2.7% on the deflection). One assumption with the Force Method/Virtual Work method that might be considered overly generous, is the moment connection between the brick veneer and the horizontal leg of the shelf angle. A pinned connection or rotational spring connection may be more accurate. However, the brick veneer element is only 278 (10.9 in.) long and not the 9.144 m (30 ft.) full height of the veneer, thereby contributing far less to the overall stiffness of the system. For this reason and justified by the comparisons to the SAP 2000 - Shell elements finite element model, the simplification appears to produce a reasonably accurate estimate of the horizontal leg's deflection and reaction forces. Another simplification is that only one row of ties is considered. However, the 3D Finite Element Model demonstrated that the further the ties are away from the shelf angle, the less lateral load they attract with virtually no lateral load being attracted by the brick ties above 1.22 m (48 in.). In the SAP 2000 – 3D FE model, the reaction of the middle tie located at 278 mm (10.9 in.) above the shelf angle is 1.06 kN (239 lb.) while the middle tie at 1.22 m (48 in.) above the shelf angle is 0.033 kN (7.42 lb.) and decreases even more the further the tie is from the wall base. These results further lend support that the 2D simplification using a single row of ties at the first course of ties produces reasonable, hand-calculatable results.

CONCLUSIONS

A new design method was proposed which more accurately accounts for the interaction between the tied masonry veneer and the shelf angle. The proposed approach uses the Force Method in combination with Virtual work to solve the 1-degree statically indeterminate system that results from the introduction of a tie restraining force at the first course of ties. The equations that result can be placed into a spreadsheet or online application and allow a designer to alter parameters like; veneer height, veneer type, shelf angle size and thickness, air space depth and bolt hole location without the effort of redrawing the system with finite element software. A more accurate estimate of the deflection of the horizontal leg of the shelf angle and the reaction forces acting on the anchor bolts with the new method translates to more efficient designs that better reflect actual field performance. A more accurate estimation of the deflection and the forces acting on the anchor bolt result in a decrease in the shelf angle size from 12.7 mm (½ in.) when using a traditional hand calculation [6] to 6.35 mm ($^{1}/_{4}$ in.). In Alberta, this translates to a cost savings of 35.8% per lineal foot for the shelf angle or approximately 5% to 8% reduction in the total cost of the brick veneer (depending on the number of floors and frequency of shelf angles). Given the absence of experimental data for the structural capacity of shelf angles anchored to concrete, it would be prudent to conduct experiments with shelf angles directly bolted to concrete foundations and anchored at the first course to a backup wall to assess the viability in the lab of the method proposed herein.

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