Structural Engineering Forensic Evaluation of Misdiagnosed Concrete Masonry Wall Cracking

W. C. Bracken

***Abstract*—**Given that concrete masonry walls are expected to experience shrinkage combined with thermal expansion and contraction, and in some cases even carbonation, throughout their service life, cracking is to be expected. However, after concrete masonry walls have been placed into service, originally anticipated and accounted for cracking is often misdiagnosed as a structural defect. Such misdiagnoses often result in or are used to support litigation. This paper begins by discussing the causes and types of anticipated cracking within concrete masonry walls followed by a discussion on the processes and analyses that exists for properly evaluating them and their significance. From here, the paper then presents a case of misdiagnosed concrete masonry cracking and the flawed logic employed to support litigation.

***Keywords*—**Concrete masonry, masonry wall cracking, structural defect, structural damage, construction defect, forensic investigation.

I. Introduction

EACH year, within the US, billions of dollars are spent on litigation and litigation-related activities resulting from flawed forensic investigations based in part on misdiagnoses and/or flawed analyses. This paper focuses on the misdiagnosis of cracking within concrete masonry construction and highlights the use of these misdiagnoses and/or flawed analyses in supporting unwarranted litigation.

II. Anticipated Cracking within Concrete Masonry Walls

According to US building codes, referenced standards and design guides, cracking within concrete masonry walls is to be anticipated.

The referenced standard within US building codes relating to concrete masonry is the ACI 530 *Building Code Requirements and Specifications for Masonry Structures*. This referenced standard states that "consideration shall be given to effects of forces and deformations due to … shrinkage, expansion, temperature changes …and differential movement" [1], all of which are to be anticipated and can be calculated.

The design guide, referenced within the *ACI 530*, is the NCMA *TEK 10-1A: Crack Control in Concrete Masonry Walls*. This design guide also identifies the primary causes of "common" or anticipated cracking to be shrinkage, thermal expansion and contraction, and carbonation [2], all of which can be calculated. These primary causes each create internal stresses that when relieved by the wall result in mortar joint cracking, often referred to as "hairline" cracking.

W. C. Bracken is with Bracken Engineering, Inc., Tampa, FL 33618 USA (phone: 813-243-4251, e-mail: wbracken@brackenengineering.com).

A. Shrinkage

Concrete masonry shrinkage occurs due to the reduction in volume of both the block and mortar as they each cure (dry) and gain strength. Shrinkage due to drying is dependent on several variables including method of curing, initial moisture content of the cementitious mix, the amount of cement paste used and the type and size of aggregate used in both the mortar and the block masonry. Therefore, the rate and amount of shrinkage experienced by concrete masonry assemblages varies.

Established shrinkage coefficients do however exist. According to these published coefficients, typical drying shrinkage will result in a reduction of 0.0002 to 0.00045 in/in (mm/mm) of a wall [2] throughout its service life. NCMA's *TEK 10-1A* provides the following example of anticipated reduction due to shrinkage: *100 lineal feet* (30.48 m) *of masonry wall will experience a reduction in length of 0.24 to 0.54 inch* (6.1 mm to 13.7 mm) *from drying shrinkage* [2].

B. Thermal Expansion and Contraction

Building materials including concrete masonry, concrete mortar and concrete grout within the walls will expand and contract when subjected to changes in temperature. When heated, concrete masonry assemblages will expand resulting in an increase in volume and, correspondingly, the length of concrete masonry walls. Conversely, when cooled, concrete masonry assemblages will contract resulting in a decrease in volume and, correspondingly, the length of concrete masonry walls.

Thermal expansion and contraction, often referred to as thermal variation, also has established coefficients. According to these published coefficients, a change in volume of 0.0000045 in/in//°F (0.0000081 mm/mm/°C) of wall is anticipated. The published values however range from 0.0000025 to 0.0000055 in/in/°F (0.0000045 to 0.0000099 mm/mm/°C) [2]. NCMA's *TEK 10-1A* provides the following example of anticipated reduction due to thermal contraction: *100 lineal feet* (30.48 m) *of masonry wall will experience a reduction in length of 0.38 inch* (9.7 mm) *with a 70-degree* (21.1 °C) *temperature drop* [2].

C. Carbonation

Carbonation is the reaction between cementitious materials and carbon dioxide in the atmosphere. This reaction occurs throughout the service life of the concrete masonry assemblage and results in a reduction in volume or shortening of the wall. As a relatively newly-understood phenomenon its long term effects are still being studied. For this reason, NCMA's *TEK 10-1A* suggests that a value of 0.00025 in/in (mm/mm) [2] of wall be used to estimate the total reduction throughout an assemblage's service life. NCMA's *TEK 10-1A* provides the following example of anticipated reduction due to carbonation: *100 lineal feet* (30.48 m) *of masonry wall will experience a reduction in length of 0.3 inch* (7.6 mm) [2].

D. Causes of Anticipated Cracking

In order for concrete masonry to structurally perform as intended, the walls must be restrained. This restraint is accomplished by structurally connecting the wall to the foundation, structural members and adjoining diaphragms at corners and changes in geometry. All of which, while necessary for the proper structural performance of the wall, result in restraints acting on the wall restricting movements. These restraints also serve to prevent or significantly restrict concrete masonry assemblies from expanding or contracting; thereby, resulting in the buildup of internal stresses. Relief of these internal stresses leads to cracking or separations within and/or between the assembly's constituent parts (i.e., between the concrete masonry units, adjacent to internally filled cells and adjacent the mortar serving to hold the masonry in place). The important point is that while this cracking and separation may constitute distress, it does not constitute structural damage in that it does not typically compromise the structural integrity of the wall. A point emphasized within NCMA's TEK 10-1A's discussion regarding Control Joints when it states, "shrinkage cracks in concrete masonry are an aesthetic rather than a structural concern" [2].

III. Identifying Anticipated Cracking

When concrete masonry assemblages shrink, the cracking that results forms distinct patterns depending on where and how the wall acts to relieve the stress. Typically, shrinkage cracks manifest themselves at changes in geometry, such as adjacent to corners, areas of weakness within the wall (e.g., openings for windows or doors), or adjacent internally-stiffened elements such as filled cells. Anticipated cracking typically manifests in stair step, horizontal and/or vertical configurations. Anticipated cracking also occurs within the interfaces of different components such as the wall-to-foundation interface or the wall-to-bond-beam interface [3].

A. Identifying Types of Crack Manifestation

The process of identifying cracking can be broken up into two phases with the first phases being the identification of the type of crack manifest.

As previously discussed, the most common types of cracking manifest consist of stair step mortar joint cracking, horizontal joint cracking and vertical cracking. For purposes of analysis it is often helpful to map the locations and configurations of the cracking observed. This mapping serves to record the configuration, orientation and extent of propagation of the cracks manifest. See Figs. 1 and 2 for examples of crack mapping showing the types and common locations of cracking.

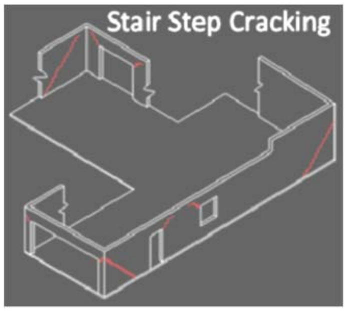

Fig. 1 Stair Step Crack Mapping

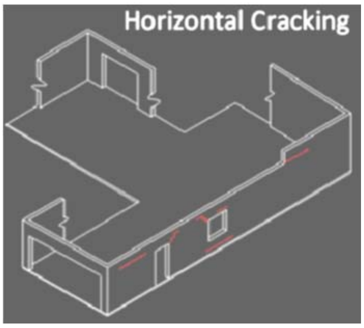

Fig. 2 Horizontal Crack Mapping

B. Quantifying Cracking

Next in the process of identifying cracking is quantifying the type and amount of displacement exhibited within or across the crack.

The types of displacement consist of in-plane, out-of-plane, and rotational displacement. In-plane displacement can best be described as a separation within the field of the wall that does not result in any out-of-plane offsets or out-of-plane discontinuities.

Out-of-plane displacement is best described as a separation within the field of the wall that does result in an out-of-plane offset or out-of-plane discontinuity across the face of the crack and within the field of the wall.

Rotational displacement is best described as rotation of one element with respect to the other. This type of displacement will generally exhibit a significant difference in the width of a separation with that difference being smaller closer to the point of rotation and greater as the distance from the point of rotation increases. Rotational displacement can manifest itself as either in-plane or out-of-plane cracking.

The amount of displacement exhibited within or across a crack or separation is to be quantified through measurement and can be notated as part of the crack mapping. Documenting not only the type of cracking but the location, and type of displacement and amount of displacement, are key to properly interpreting the cracking.

IV. Evaluating Anticipated Cracking

As established, cracking resulting from shrinkage, expansion and contraction, and carbonation within concrete masonry typically does not affect structural integrity or the wall's ability to serve its intended function. This is a point that is also emphasized within NCMA's TEK 10-2C's introduction when it states, "shrinkage cracks in concrete masonry are not a structural concern" [4]. Therefore, when the type of cracking that has manifested is of a type that is anticipated, and the amount of cracking or separation measured is equal to or less than that anticipated, the cracking can be considered aesthetic and of no structural significance.

A. Identifying Aggravating Factors

While the primary causes of anticipated cracking include the material effects previously described, when combined with inadequate design and/or construction, cracking is assured. One example of a design and/or construction deficiency would be a lack of expansion joints or means of controlling anticipated movements which can serve to facilitate or exacerbate anticipated cracking.

According to NCMA's *TEK 10-2C*, "the proper application of crack control measures, including control joints when required, can help ensure satisfactory performance of the concrete masonry" [4]. A point is reiterated within the ACI *Building Code Requirements and Specification for Masonry Structures*, when it states that "movement joints are used to allow dimensional changes in masonry, minimize random wall cracks, and other distress" [5]. Therefore, the process of evaluating cracking should also consider design and/or construction so as to differentiate between anticipated cracking and structurally significant cracking.

B. Structural Condition Assessment

To further identify and differentiate between anticipated cracking that is possibly being aggravated versus structurally-significant cracking, a structural condition assessment is to be conducted.

According to *SEI/ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings*, when performing a structural condition assessment of an existing building for purposes of "Evaluation", the structural condition assessment is to determine the structural adequacy of the building or component for its intended use and or performance [6].

Regarding determining structural adequacy within a structure that merely exhibits anticipated cracking, there is no need to calculate the amount of capacity within the structure because by permitting and constructing in accordance with legally adopted and enforced building code(s), the structure is presumed to have the requisite structural capacity. In addition, there is no need to calculate the amount of capacity lost within the structure because the structure is performing as designed and intended as evidenced by the fact that it fails to exhibit damage resulting from conditions that exceeded those anticipated by the code(s) to occur throughout its life. Therefore, in cases where the structure: was permitted and constructed in accordance with legally adopted and enforced building code(s), has remained legally occupied without any change of use, and fails to exhibit damage resulting from conditions that exceeded those anticipated by the code(s) to occur throughout its life, further analysis is not required.

V. Evaluating the Structural Impact of Cracking

In cases where: the type of cracking that has manifest is not of a type anticipated, the amount of cracking or separation measured is greater than that anticipated, or the cracking is determined to be of a structural significance, whether misdiagnosed or not, further numeric analysis is required to determine the extent of impact on the structure's integrity or the concrete masonry wall's ability to serve its intended function. Similarly, in cases where a structural condition assessment identifies damage to a structure resulting from conditions that exceeded those anticipated by the code(s), further numeric analysis is required so as to determine the amount of capacity lost [6].

A. Impact of Reinforcing

One of the first steps in further evaluating a concrete masonry wall or assemblage is establishing whether the wall is reinforced or unreinforced.

When reinforcing is present within the wall, the tension capacity of the masonry can be neglected because the steel serves to transfer the tensile forces [1], [5]; therefore, a crack, regardless of its level of anticipation, will still allow loading to be transferred without affecting the wall's structural integrity. In short, if both a compressive and tensile load path remains within a reinforced concrete masonry wall, the structural integrity or the wall's ability to serve its intended function has not been compromised.

B. Computational Analysis

If, however, a compressive or tensile load path does not remain or the structural integrity of the wall is in question, then a numeric analysis is to be performed and is to address in-plane and out-of-plane loading as appropriate.

If and when portions of a concrete masonry assemblage or concrete masonry are called into question, they are required to be evaluated by means of performing calculations. Specifically, damage can change the properties of the structure's components and that is when you would need to determine if the damaged member or connection can remain in place and continue to carry loads as required or if that member needs to be supplemented or replaced. The process of analyzing a concrete masonry wall is to begin by considering the wall in its current condition, then performing a computational evaluation looking for any overstressed conditions resulting from axial loading, in-plane shear, and out-of-plane bending.

Axial Loading: The evaluation of axial loading begins by performing a basic load path assessment. Further, because the tensile capacity of concrete masonry is to be neglected [1], [5], the load path assessment consists of examining the concrete masonry wall to determine if there is a loss of bearing contact preventing the loads from transferring down through the wall to its support or foundation. A loss of bearing contact within the field of the concrete masonry wall can result in redistribution of loads possibly increasing loads and forces placed on the concrete masonry units themselves or on the wall's support or foundation.

International Science Index, Civil and Environmental Engineering Vol:11, No:5, 2017 waset.org/Publication/10006918

If there is not a loss of bearing contact and the concrete masonry units remain supported atop one another, there would be no added increase on the concrete masonry units themselves, the wall's foundation and/or the foundation's supporting soils. If, however, there is a loss of bearing contact within the masonry, the bearing capacity of the foundation along with its supporting soils are to be evaluated so as to determine whether the structural integrity of the wall or its ability to serve its intended function has been affected. To perform this evaluation, the stress within the concrete masonry resulting from the applied loads is to be compared against allowable stresses. In order to compute the stress resulting from the applied axial loads, the actual bearing stress f_a of the concrete masonry is equal to the applied load divided by its bearing area A_{br} [1], using (1):

$$f_a = Applied\ Load / A_{br} \tag{1}$$

In order to compute the maximum allowable stress of the concrete masonry in cases where the ratio of the height of the wall h divided by its radius of gyration r is less than the value of 99, *ACI 530-13* states that the allowable axial stress, F_a [1], is computed using (2):

$$F_a = (1/4)\ f'_m\ [1 - (h/(140 * r))^2] \tag{2}$$

Therefore, if f_a is less than F_a then any loss of bearing contact within the wall has not affected the concrete masonry's ability to support its axial loading. In order to compute the stress within the supporting soils resulting from the applied axial loads, the bearing stress q_a placed on the supporting soils is equal to the applied load per unit length divided by the footing's width b using (3):

$$q_a = Applied\ Load / b \tag{3}$$

In order to compute the allowable stress of the supporting soils, if the wall is supported atop a continuous footing and the soil properties are known, the bearing stress of the supporting soils [7] is computed using (4):

$$q_{ult} = (c * N_c) + (\gamma * D_f * N_q) + (0.5 * \gamma * B * N_\gamma) \tag{4}$$

Therefore, if q_a is less than q_{ult} then any loss of bearing contact within the wall has not affected the supporting soil's ability to support the axial loading placed atop the wall.

In-plane Shear: Shear failure is described within the commentary portion of *ACI 530-13* as manifesting itself as diagonal cracking through the concrete mortar and masonry units, horizontal cracking within bed joints, and full-depth stair step mortar joint cracking [1]. The evaluation of in-plane shear loading is performed so as to determine the impact, if any, of cracking within the field of the concrete masonry wall on its ability to resist externally applied loads resulting in in-plane shear. While this evaluation should consider the increase in strength provided by horizontal joint reinforcing, it may either consider or neglect the increases afforded by vertical reinforcing.

If the wall is, at a minimum, vertically reinforced at a spacing not exceeding the height of the wall divided by two and/or any cracking within the wall is merely anticipated cracking, in-plane shear typically does not control. If, however, concerns exist for the in-plane shear capacity, the wall is to be evaluated so as to determine whether its structural integrity or its ability to serve its intended function has been compromised. To perform this evaluation, the stress within the concrete masonry resulting from the applied loads is to be compared against allowable stresses. In order to compute the stress resulting from the applied axial loads, the applied or anticipated load V [1] is used in (5):

$$f_v = (V * Q) / (I_n * b) \tag{5}$$

Neglecting any increases afforded by vertical reinforcing, when computing the maximum allowable stress of concrete masonry laid in a running bond and not fully grouted, *ACI 530-13* states that the allowable shear stress, F_v [1], is not to exceed any of the following within (6)–(8):

$$1.5 * [f'_m]^{0.5} \tag{6}$$

$$120\ psi\ (0.827\ MPa) \tag{7}$$

$$37\ psi + 0.45 * N_v / A_n \tag{8}$$

Therefore, if f_v is less than F_v then any loss of bearing contact or unanticipated cracking within the wall has not affected the concrete masonry's ability to resist in-plane shear.

Out-Of-Plane Bending: Out-of-plane bending failure results when externally generated loads applied uniformly and normal to the face of the wall exceed the concrete masonry wall's ability to resist bending. Given that the tensile capacity of concrete masonry is to be neglected [1], [5], the concrete masonry wall's ability to resist bending is to be based on axial loads in combination with reinforcing within the wall. In other words, any cracking within the field of the wall will have no bearing on the wall's ability to resist out-of-plane bending.

The vertically reinforced sections within the wall are designed to resist both uplift and out-of-plane bending. According to *ACI 530-13*, the effective width of each reinforced section [1], for purposes of design, is computed based on the least of the following (9)-(11):

$$6 * \text{Wall Thickness} \tag{9}$$

$$72 \text{ inches } (1.83 \text{ M}) \tag{10}$$

$$\text{Spacing of Vertical Reinforcement} \tag{11}$$

However, from an analysis perspective, any unreinforced sections, whether within the effective width of the reinforced sections or not, will act to span either horizontally or vertically depending upon the configuration of the wall. In the case of a wall where the spacing of the vertically reinforced sections exceeds those recommended for design, an Arching analysis [8] should be performed. This analysis will determine if the unreinforced sections will be able to span horizontally between the vertical reinforcement, thereby increasing the width of the reinforced sections, for purposes of computing out-of-plane bending, to the center-to-center spacing of the vertical reinforcement.

The performance of this evaluation begins by computing a maximum allowable external load based on the allowable stress of the concrete masonry and the wall segments' restraint conditions and then comparing it against the applied or anticipated load [8] using (12):

$$P = 8 * C * (\gamma t - \Delta o) / h^2 \tag{12}$$

Provided the maximum allowable external load P is greater than the actual applied or anticipated load, the concrete masonry wall has the ability, through Arching, to allow the out-of-plane bending analysis to consider a section of wall based on the center-to-center spacing of the vertical reinforcement. At this point, if there still remains a concern for the concrete masonry's ability to resist out-of-plane bending, the reinforced masonry sections spanning vertically can be analyzed for axial loads acting in combination with out-of-plane loads. The performance of this evaluation begins by computing the actual bending moment based on the applied or anticipated loads.

In the case of wind loads,

$$M_w = w * l^2 / 8 \tag{13}$$

can conservatively be used with w being the applied wind loads. This value is then compared against the concrete masonry wall's allowable moment M_n [1] using (14), (15):

$$M_n = ((A_s * F_y) + P_u) * ((d - (a / 2)) \tag{14}$$

$$a = ((A_s * f_y) + P_u) / (0.8 * f'_m * b) \tag{15}$$

Therefore, if M_w is less than M_n then any cracking within the wall has not affected the concrete masonry's ability to resist out-of-plane bending.

VI. Misdiagnoses in Support of Litigation

As established above, cracking within concrete block walls resulting from shrinkage is an anticipated and quantifiable occurrence that rarely compromises the structural integrity of a concrete masonry wall. Nonetheless, whether through ignorance or intent, misdiagnoses continue to be used to support litigation. The balance of this paper will highlight an exemplar where anticipated cracking within concrete masonry walls was misdiagnosed as unexpected and "indicators of structural damage".

A. Fact Pattern

The following represents a redacted and summarized fact pattern presented for use in this paper.

1) Single story residential structure, roughly 10 years old,
2) Normalized footprint, roughly 2,500 square feet (232 m^2),
3) Building owner reports "damage",
4) Engineer #1 examined the structure and reported:

… multiple vertical cracks emanating from the corners of the window openings sporadically throughout the exterior. These cracks measured up to 1/64 inch. This condition was found to be consistent with expansion and contraction within the masonry wall system concentrated at changes in geometry.

… multiple stair step cracks emanating from the window openings sporadically throughout the exterior. These cracks measured up to 1/64 inch. This condition was found to be consistent with material shrinkage within the masonry wall system concentrated at changes in geometry.

… multiple stair step cracks within the field of the exterior walls. These cracks measured up to 1/64 inch. This condition was found to be consistent with material shrinkage within the masonry wall system reflected through the stucco cladding.

5) Engineer #1 opined:

While the [structure] *did evidence damage, distress and/or minor differential displacement, these conditions were of a non-structural nature.*

6) Engineer #2 examined the structure and reported:

The [structure] exhibited signs of widespread negligible (hairline) to very slight (~ 1/32") masonry cracking. The majority of this cracking is the result of differential settlement of the foundation [shown in Fig. 3].

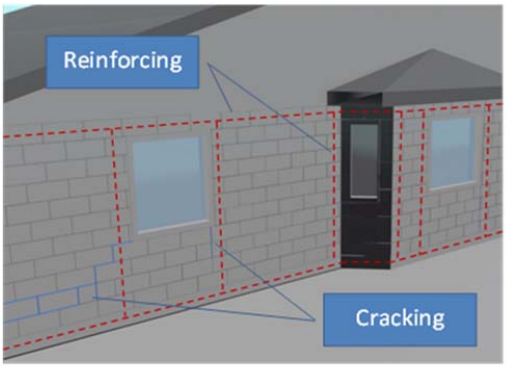

Fig. 3 As-observed by engineer #2

The location of vertically-orientated rebar/concrete filled masonry block cells were identified and found to be on either side of every opening and at a maximum spacing of 8 feet within the field of the walls [shown in Fig. 3].

7) Engineer #2 opined:

There was settlement related damage to the structure which has affected the ability of the building or foundation to carry loads for which it was designed.

B. Basis of Opinions

Engineer #1's basis for their opinion was that the conditions observed throughout the interior and exterior of the subject structure were found to be consistent with anticipated material shrinkage, minor settlement, thermal expansion and contraction and/or wear resulting from either age, anticipated conditions or original construction deficiencies.

Whereas, Engineer #2's basis for their opinion was that:

1) Partially reinforced masonry construction relies on the

tension capacity of the mortar to transfer out-of-plane wind loads through uncracked sections to adjacent reinforced sections and/or lateral support locations.

2) During the wind load case, considering that both uplift axial and out-of-plane bending forces must be transferred and resisted by the masonry structural components (masonry, mortar and the bond between the two) at the same time, the uplift counteracts the gravity loads limiting the flexural capacity gained by self-weight and dead load. Therefore, a wall segment that has been weakened by consecutive settlement related cracks may not adequately resist the building code mandated out-of-plane loads resulting in the failure shown in Fig. 4.

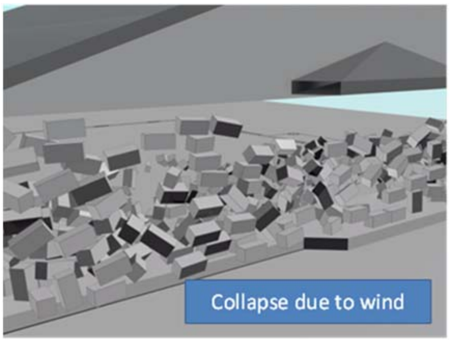

Fig. 4 Failure as theorized by Engineer #2

In other words, Engineer #2 opined that when the wind blows the wall will fall down based on the following:

3) When the wind blows it will generate sufficient uplift to "lift the roof" off its supporting walls and relieve all axial loads,
4) Because the wall has cracks, it is incapable of resisting out-of-plane bending,
5) With no axial load to resist bending, the cracking will allow the wind to blow the wall down, and
6) If the wall cannot resist wind loads, it is structurally compromised.

VII. Conclusion

Clearly, based on the fact pattern as presented, Engineer #2 has failed to consider the impact of anticipated cracking as the cause of the negligible, non-structural "hairline" to "very slight" (< 1/32" or 0.8 mm) masonry cracking observed. In addition, Engineer #2 neglected to consider the full impact of the reinforcing within the wall.

This failure to consider causes other than differential settlement, combined with a failure to consider the impact of the reinforcing, combined with a failure to perform any computational analysis in support of the opinion, resulted in a catastrophically flawed analysis which was used to promote and prolong litigation. Furthermore, continued failure to consider anticipated causes of cracking within concrete masonry walls combined with a failure to perform a proper analysis, will only continue to result in or be used to support litigation.

References

[1] ACI 530-13 & 530.1-13 / TMS 402-13 & TMS 602-13 / ASCE 5-13 & ASCE 6-13: Building Code Requirements and Specification for Masonry Structures. ACI/ASCE/TMS 2013, pp. C-36 – C-41, C-50, C-98, C-108 & C-114 – C-117.

[2] National Concrete Masonry Association, "TEK 10-1A: Crack Control in Concrete Masonry Walls." NCMA, TEK 10-1A, 2005.

[3] W. Bracken & J. Busquets, "White Paper: Hairline Cracking within Concrete Block Walls" unpublished.

[4] National Concrete Masonry Association, "TEK 10-2C: Control Joints for Concrete Masonry Walls – Empirical Method." NCMA, TEK 10-2C, 2010.

[5] ACI 530-08 & 530.1-08 / TMS 402-08 & TMS 602-08 / ASCE 5-08 & ASCE 6-08: Building Code Requirements and Specification for Masonry Structures. ACI/ASCE/TMS 2011, pg. SC-15.

[6] SEI/ASCE 11-99: Guideline for Structural Condition Assessment of Existing Buildings. SEI/ASCE 1999, pp 3 – 11.

[7] L. Chen & J. Evett, Soils and Foundations. Englewood Cliffs, NJ: Prentice-Hall, 1987, pg. 192.

[8] R. Drysdale & A. Hamid, Masonry Structures Behavior and Design – 3rd Edition. Boulder, CO: The Masonry Society, 2008, pp. 272 – 275 & 286 – 287.

William C. Bracken earned his Masters of Science in Civil Engineering from the University of South Florida, Tampa, Florida, USA in 1994 and his Bachelors of Science in Civil Engineering from the University of South Florida, Tampa, Florida, USA in 1989.

He is currently the President & Principal Engineer of Bracken Engineering in Tampa, Florida, USA. His career has centered on the practice of structural engineering while specializing on its application in the fields of Codes, Fire Rescue and Standards of Care. His practice has encompassed design, analysis, research, publishing, instruction and forensics. Mr. Bracken has published and routinely presents on topics of forensic engineering and structural rehabilitation.

Mr. Bracken currently serves as the Chairman of Florida's engineering licensure board and as a Master Instructor for the International Code Council (ICC). In addition, he serves as an Urban Search & Rescue Structural Specialist. He is a recognized Fellow within the Structural Engineering Institute (SEI) and the American Society of Civil Engineers (ASCE). He is also a Board Certified Diplomate of the National Academe of Forensic Engineers (NAFE).